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HYDRAULIC MODEL STUDY OF THE THEODORE ROOSEVELT DAM SPILLWAY MODIFICATIONS

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16. ABSTRACT

Theodore Roosevelt Dam in Arizona will be raised 65 feet by the Bureau of Reclamation to provide additional flood storage. This raise will require extensive modifications to the existing waterways. The hydraulic model study examined the approach conditions, intake vortex suppression methods for the two top-seal radial-gated spillways, optimization of the spillway chute geometry, plunge pool location and size to accommodate the impinging jets from the dual spillways, and flow conditions in the river channel downstream and near the powerplant road.

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by

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PURPOSE

The hydraulic model study was conducted to investigate the proposed modifications to the waterways as a result of the dam raise. The model study emphasized optimizing the spillway geometry and plunge pool energy dissipation.

INTRODUCTION

Theodore Roosevelt Dam is listed in the National Register of Historic Places. The dam, completed in 1911, was the first major structure to be completed by the Bureau of Reclamation (formerly known as Reclamation Service) under the Reclamation Act of 1902. The dam is located on the Salt River, 76 miles northeast of Phoenix, Arizona. The Salt River Project operates the dam which is part of a multipurpose project providing irrigation water, hydropower, and recreation.

The existing rubble-masonry thick-arch dam is 280 feet high, 723 feet long at the crest (El. 2142), and impounds 1,381,580 acre-feet of water. The existing abutment spillways with crest elevation at 2120.3 feet are unlined channels controlled by a total of nineteen 20- by 15.9-foot radial gates (fig. 1). The spillway discharge capacity is 150,000 ft^3/s at reservoir elevation 2146.

The dam will be raised 65 feet to provide storage capacity, thus reducing the potential for flooding the city of Phoenix, Arizona. The dam will be raised using conventional mass concrete to form a single-curvature arch dam to elevation 2218 (fig. 2). For the new dam, the waterways were redesigned to allow continued passage of 150,000 ft^3/s.

THE HYDRAULIC MODEL

The Froude law was used for scaling the model, as gravitational forces predominate. A 1:40 scale hydraulic model was chosen based on available laboratory space and the need to attain an adequate gate size for the spillways.

Froude law similitude produces the following relationships for the 1:40 scale model:

$$\text{Length ratio} = L_r = 40:1$$

$$\text{Velocity ratio} = L_r^{1/2} = 40^{1/2} = 6.32:1$$

$$\text{Discharge ratio} = L_r^{5/2} = 40^{5/2} = 10{,}119.29:1$$

The model was calibrated using the permanent laboratory Venturi systems to measure the inflows and a point gauge in the head box to record reservoir elevations.

The model was completed in March 1988 to the specifications of the double-curvature arch dam option from feasibility drawings of the dam, spillways, and plunge pool (fig. 3). The model included approximately 800 feet of the reservoir area with the spillway approach channels upstream of the raised arch dam, the left and right abutment spillways, and about 1,150 feet of the downstream river channel, including the plunge pool.

The double-curvature arch dam had thrust blocks with 1:3 sloping upstream faces. At the upstream dam face, the straight conduit section of the spillways was turned inward 6.67° from normal. The piers extended into the reservoir to varying degrees depending upon their elevation with respect to the thrust block slope. The spillway design specified two adjacent, 18-foot-wide by 28-foot-high, straight conduits passing through the dam. Spillway flows were regulated by top-seal radial gates mounted on each conduit at the intersection with the downstream face of the dam. The right spillway conduit and chute were set at a 0.17 slope. The chute flared from 50 to 80 feet beginning downstream of the gates and terminated with a 40-foot-long, 30° flip bucket. The left spillway was designed at a slope of 0.12. Proper orientation to the plunge pool required the chute to follow a horizontal curve. The curved chute was superelevated to maintain a uniform flow depth. Similar to the right side, the chute also flared from 50 to 80 feet and terminated in a 30° flip bucket at the centerline. Both spillways discharged into a common plunge pool. The initial plunge pool was almost rectangular, with a floor elevation of 1868 feet.

Buildings for the powerplant, outlet works, and historic warehouse including the access roadway were modeled to determine if any of these structures would be endangered by flow from the new spillways into the plunge pool. The outlet works discharge was also modeled. The 16-foot-diameter outlet works pipe, split to four 7.5-foot-diameter jet-flow gates, was modeled by using a pipe and spreader box with holes representing the jet-flow gates. The plunge pool and downstream topography were initially modeled to the alluvium elevation using a fixed bed. Some of the fixed bed was later removed and replaced with erodible material to investigate possible erosion or alluvium movement.

An overall view of the 1:40 scale model with the original geometry of the double-curvature arch dam is shown on figure 4.

FINAL DAM RAISE SELECTION

The model was initially constructed with the dam, spillways, and plunge pool of the double-curvature arch dam design. Model investigations began in March 1988, and the spillway and plunge pool designs were optimized with the geometry and alignment that accompanied this dam raise option.

While the hydraulic model studies were being conducted with the waterways of the double-curvature arch dam option, the designers were continuing simultaneous investigations of other dam raise options. During this time, it was found that the tensile stress in the base of the existing dam would exceed the acceptable limit under loading at maximum water surface elevation 2218 with the DCAD (double curvature arch dam) raise option. In December 1988, the design for raising the dam was changed from the original DCAD concept to a SCAD (single curvature arch dam) constructed of conventional mass concrete.

Adopting the SCAD raise option meant several changes to the design of the waterways. The SCAD design moved the spillways closer to the center of the now-vertical dam face and reduced the length of the left spillway, thus moving the impingement point of the spillway jets upstream and closer to the powerplant road. After review of the design changes to the waterways, it was felt that no major changes to the proposed design would be necessary; however, the final design should be modeled to confirm these opinions. Modifications to the model began in late December 1988 and were completed in mid-February 1989.

RESULTS

The results presented are for the waterways associated with the final SCAD design. All aspects of the waterways were retested for the SCAD except the pressure measurements in the plunge pool, as these will remain identical to those recorded for the DCAD.

Spillway Intake Area

The geometry of the approach channel and the alignment of the spillways cause air core vortices upstream of the spillway intakes. To prevent these vortices from residing above the intakes, vortex suppressors are needed above both the left and right abutment spillway intakes. The recommended design for each vortex suppressor is horizontal beams 4 feet high by 2 feet wide with 3-foot openings between each beam. The suppressor structure spans the width of each intake and is attached to the face of the dam from elevation 2150 to 2175 (fig. 5). The suppressors break up the circulation of vortices as they reach the face of the dam, thus inhibiting air-entraining vortices from reservoir elevation 2151 to 2175. Above elevation 2175 intermittent vortices will form and break up.

The spillway approach channels will be dewatered during construction. The concrete walls, constructed to support cellular cofferdams, will remain after construction and be submerged by the reservoir. Although positioned near the spillway intakes, the walls cause no adverse effects on the spillway entrance flow.

Spillway Discharge Curves

To meet the spillway discharge criteria, the top-seal radial gates were widened from 18 to 21 feet for the final design. The wider spillway gates pass the design discharge, 135,000 ft^3/s, at reservoir elevation 2175. The maximum allowable discharge, due to capacity of downstream dams, is 150,000 ft^3/s. With the gates fully open, 150,000 ft^3/s is passed at reservoir elevation 2187.57. Discharge curves were developed for each spillway for both simultaneous and separate operating conditions (figs. 6, 7, and 8). Gate openings were measured normal to the spillway slopes. Different slopes for the right and left spillway inverts produce different gate seat elevations; therefore, each spillway discharge under identical reservoir elevation and gate setting is not equal.

Spillway Chutes

The 30° flip bucket angle is appropriate and produces impingement of the jets in the central area of the plunge pool. The end width for the right spillway chute was reduced by 10 feet to 70 feet. This reduced the flow separation along the side walls and the spread of the jet. This, in turn, improved the impingement of the two spillway jets, thus improving flow conditions at the upstream edge of the plunge pool. Rooster tail fins form downstream of the piers in both spillways. The fin in the left spillway is reduced by the superelevation. However, due to the short length of the right spillway chute, the fin is quite large at intermediate flow rates but is drowned out at high flow rates. Water surface profiles through the chute and side wall pressures through the bucket were measured to provide wall height and stability information for the right spillway (figs. 9 and 10).

Superelevation of the left spillway was significantly reduced from the original design. This improved uniformity of the flow depth across the chute width, thus producing more balanced flow conditions in the plunge pool. Water surface profiles and side wall pressures were also recorded for the left spillway for wall height and stability information (figs. 11 and 12).

Gate Operations

Uniform gate operations are recommended. However, the right spillway may be operated alone up to a discharge of 25,000 ft^3/s at reservoir elevation 2175 without adversely affecting erosion patterns. Further investigations showed the right spillway could be operated up to about 41,000 ft^3/s at elevation 2175 without endangering the adjacent powerplant road retaining wall. Operation of the right spillway above 25,000 ft^3/s will, however, draw loose material into the plunge pool and produce asymmetrical scouring of the unlined plunge pool and adjacent area. Discharges from the left spillway should be added as soon as possible, preferably before releasing 41,000 ft^3/s from the right spillway, to assure proper plunge pool energy dissipation.

Under the design discharge (gates fully open at reservoir elevation 2175) and greater flows, operating with both spillways at the same gate opening results in a strong boil along the left edge of the plunge pool. The right gate of the right spillway may be closed approximately 1.6 feet, normal to the invert, to reduce the boil and thereby reduce the potential for erosion along the powerplant road.

Reservoir Evacuation

During reservoir evacuation, with the gates fully open, the jet trajectory lengths decrease with falling reservoir head, eliminating coincident impingement of the jets. As a result, some enlargement of the plunge pool toward, and erosion of the cliffs underneath, each spillway could be expected. However, these effects should be minimal assuming adequate tailwater and the limited flow duration.

Plunge Pool Location and Geometry

The final plunge pool location and shape tested for the SCAD, shown on figure 13, will be pre-excavated to elevation 1840 and unlined. This plunge pool adequately dissipates the energy of the design discharge, 135,000 ft^3/s at reservoir elevation 2175 (fig. 14).

The potential for plunge pool erosion was investigated by measuring pressures on the plunge pool floor and velocities in the adjacent area under the design discharge. A map of the static pressure for the design discharge (fig. 15) shows a concentrated area of high pressure at the jet impact point with rapidly declining pressures away from the immediate jet impact area. Near the perimeter of the pool, the static pressure heads were nearly equal to the tailwater depth. Dynamic pressure fluctuations in the major impact area were less than 33 feet below and 41 feet above the static level 95 percent of the time. Tailwater velocities measured upstream of the impinging jets are low, and those measured in the river channel are acceptable (fig. 16). The major jet impact and boils occur in and along the middle to downstream end of the plunge pool. It is assumed that the plunge pool will erode beyond the pre-excavated limits during major spillway flows. Flow conditions and pressure and velocity data indicate no plunge pool erosion will occur toward the dam.

Erodible and Movable Bed Studies

Movable bed studies were conducted to investigate the adequacy of the planned alluvium removal adjacent to the plunge pool. Alluvial material was placed over the rock contours, transitioning on 3:1 slopes from the plunge pool to the material at the dam toe and downstream of the plunge pool. The full discharge range was then tested. No material was drawn into the plunge pool or moved from the toe of the dam during outlet works operation. Only a very small amount of material was moved downstream from the toe of the 3:1 slope in front of the outlet works. This was true with the outlets operating alone or in conjunction with the right spillway to obtain a total discharge of 25,000 ft^3/s from reservoir elevation 2151 to 2175. During operation of the right spillway alone at discharges greater than 25,000 ft^3/s, material will move into the plunge pool from the area adjacent to the right downstream side. During design discharge, material within the pool will wash out and redeposit in the river channel downstream. No material moved at the toe of the dam under any operating condition. Erosion of alluvium will occur next to the warehouse and along the sides of the downstream river channel during spillway flows of about 70,000 ft^3/s or greater. These tests indicated that the planned alluvium removal is adequate.

The sides of the upstream half of the fixed plunge pool were replaced with an erodible concrete mixture to determine if there were any high-velocity jets directed toward the toe of the dam. No erosion was observed on the sides of the pool when tested under the design discharge with no material originally in the plunge pool. This indicated that after impingement on the floor of the plunge pool the flow is largely directed downstream. When material was then manually placed in the pool and retested, large material located in the upstream end of the pool was not flushed out of the pool. This material moved around the pool floor, ball milling at the toe of the side wall slopes.

Powerplant Road and Retaining Wall

Initial opening of the left spillway gates at reservoir elevation 2175 will cause flow up to 575 ft^3/s to impinge on the powerplant road. The amount of water impinging on the road did not vary significantly with gate opening rate; therefore, the standard gate opening rate of 1 ft/min/gate is recommended. To reduce the rate of flow impinging on the road, the gates should be opened one at a time, preferably the right gate first. The right gate should be opened until the flip bucket sweeps out before opening the left gate. Some flow may still be hitting the road from the edges of the jet even after the bucket sweeps. Each gate should continue to be opened until the momentum in the jet carries it entirely beyond the road.

Operation of the left spillway will produce spray onto the road even after the jet clears the road. The close proximity of the plunge pool to the powerplant road will probably cause the road to be wet during operation of either spillway.

A highly erodible sand and gravel mixture was then placed between the plunge pool and the powerplant road. A thin layer of cement was sprinkled on the material to form the surface topography and upstream and left side walls of the pool. Progressive tests of outlet works and right spillway discharges up to 25,000 ft^3/s at elevation 2175 were then conducted. The left side walls of the pool were eroded, but not significantly enough to endanger the powerplant road retaining wall. Some erosion occurred along the toe of the road wall. This erosion resulted from a component of surface flow and wave action being diverted down the wall. Material eroded from the side walls did not wash out of the pool but deposited downstream from and behind the right spillway jet.

OPERATING CRITERIA

The normal reservoir water surface elevation is 2151 feet. Between reservoir elevations 2151 and 2175, a maximum of 25,000 ft^3/s may be released from the dam, assuming no flows enter the river downstream of the dam. This discharge would be attained using some combination of powerplant, river outlet works, and spillway operations. The remaining reservoir area above elevation 2175 is to be used for flood storage; therefore, once the reservoir water surface elevation exceeds 2175 feet, the gates for both spillways must be opened entirely to pass the design discharge of 135,000 ft^3/s, assuming no flows intervene. If the reservoir continues to rise, the discharge will increase until reaching a maximum of 150,000 ft^3/s, at which time the gates must be partially closed to maintain this discharge as the reservoir rises to the maximum elevation of 2218 feet. The probable maximum flood (PMF) is 150,000 ft^3/s at reservoir elevation 2218. The spillways must operate under these conditions until the water surface is again below elevation 2175.

The river outlet works and powerplant will be used to pass the majority of the releases. The outlet works capacity is 10,924 ft^3/s at elevation 2151 and 11,517 ft^3/s at elevation 2175. It is expected that the outlet works will be used in combination with the right spillway to release a total of 25,000 ft^3/s between reservoir elevations 2151 and 2175. When the reservoir rises above elevation 2175, the spillways will release the design flow rate and the outlet works will be closed to avoid submerged operation of the jet-flow gates.

TEST PLAN

The model study was conducted to optimize the geometry of the spillways and the energy dissipation of the plunge pool. The following items were investigated in the model, regardless of the dam raise option:

- Spillway approach flow conditions to determine adequacy of proposed excavations, and the effect of the approach channel geometry and spillway alignment on the spillway entrance conditions. Possible structural modifications to the approach channel geometry and spillway entrance to, if necessary, improve flow conditions.

- Flow conditions through the spillway conduit and chute sections including flare angle, water surface profiles, center pier width, flip bucket angle and side wall pressures, and left spillway superelevation.

- Spillway discharge rating for the entire range of reservoir elevations and possible gate openings as specified by the operating criteria.

- Plunge pool location and geometry to provide adequate energy dissipation for the entire flow range from reservoir evacuation to maximum reservoir elevation. With the basic geometry determined, measure pressures on the plunge pool floor and velocities in the surrounding river channel.

- Potential for erosion of the plunge pool, the area adjacent to the powerplant road, and the downstream river channel. This was tested by replacing the fixed plunge pool and river channel topography with loose erodible material to qualitatively indicate locations of erosion.

- Spillway jet impingement on the powerplant road during initial opening of the left spillway gates.

- Effect of outlet works discharge on the plunge pool operation and downstream river channel erosion.

DOUBLE-CURVATURE ARCH DAM - SPILLWAY INVESTIGATIONS

The initial spillway and plunge pool designs were developed based upon several factors:

- The location of the spillways with respect to the load-bearing dam thrust blocks.

- The steep cliffs on each abutment that limited the spillway lengths and thus the final jet trajectory lengths.

- The concept of minimizing the plunge pool excavation by using a common pool for both spillways.

- Protecting the toe of the dam from any high-velocity flows.

The initial spillway and plunge pool designs were constructed in the model as shown on figure 3. The initial discharge, 54,380 ft^3/s at elevation 2175 (fig. 17), was used as the design flow for spillway and plunge pool designs. The plunge pool was to be sized and shaped to produce adequate energy dissipation for this discharge. Initial operation of the model revealed some obvious concerns:

- There were strong vortices upstream of the spillway intakes.

- The flare on the right spillway was too large.

- Superelevation of the left spillway was too great.

- The plunge pool was too shallow and its location and shape required changes.

Additional design considerations including modifications to the outlet works, the reservoir evacuation plan, and the flood return period analysis subsequently required an increase in the design discharge from 54,380 ft^3/s to 135,000 ft^3/s at reservoir elevation 2175. The maximum total discharge, 150,000 ft^3/s, remained identical, but the plunge pool was now required to adequately dissipate a much greater discharge.

The following sections will discuss the extensive model investigations completed for the structures of the original DCAD raise option as they pertain to the development of the final designs for the

SCAD. Some data – discharge curves, for example – developed for the DCAD spillways are not reported as these data do not apply to the SCAD final design.

Approach Conditions

The four spillway top-seal radial gates, operating under increased head, provide the same discharge as the 19 gates of the existing spillways. The existing approach channel boundaries, which force a large turn in the flow, remain almost identical for the new spillway design. The channel invert will be excavated along a slope over the width of the intake to match the invert elevation of the spillway conduits. Each spillway passes through a thrust block with a 1:3 upstream face. The vertical stoplog piers extend up to maximum reservoir elevation 2218. The following approach channel flow phenomena contributed to formation of strong vortices for the initial spillway design (fig. 18):

- Flow along the face of the dam is obstructed by the piers which extend upstream of the dam face.

- Flow in the approach channel is not aligned with the spillway entrance and must turn to enter the spillway.

- The dead flow area outside each spillway at the intersection of the dam and the approach channel curve produces a flow shear zone.

Several types of vortices were observed in the model. Surface dimples or rotations were considered acceptable, as were dye core vortices when the tail did not extend far below the surface. Air core vortices were unacceptable due to possible gate vibration and loading problems during gate control.

Interpreting model scaling of free surface vortices has often been controversial. It has been determined through comparisons of model and prototype vortex characteristics of several intake structures that if good modeling practices are followed, good conformance exists. Froude scales with correct approach channel modeling and attention to vortex type and intensity have produced good model/prototype comparisons (Hecker, 1981).

Extensive investigations were performed to determine the best methods for eliminating air core vortices over the intake structures. Most tests were run under the design discharge condition, 135,000 ft^3/s and reservoir elevation 2175, as the vortices were the most persistent with the gates fully open and at reservoir elevation 2175 and above. The tests included determining guidewall arrangements, modifying the pier nose, making changes to the upstream face of the thrust blocks, filling in dead flow spaces, making the piers flush with the 1:3 thrust block face, and constructing vortex suppression structures. Each of these alternatives was evaluated based upon its effectiveness in eliminating the vortices, the cost, and the ease of construction.

Cellular cofferdams will be used across the approach channels during construction of the spillways. The concrete cofferdam support walls that attach to the face of the dam near the spillway entrances were modeled. The influence of the walls on the approach flow was initially studied. The cofferdam support walls were found to have little effect on approach flow conditions and thus will not require removal after construction is completed.

The guidewall arrangements tested were elliptically shaped, spanning from the cofferdam support walls to the inside pier nose. Initially, the guidewalls were vertical to elevation 2218 for the entire distance, then they were tapered from the cofferdam support to the pier nose (fig. 19). The vertical guidewalls were quite effective in eliminating the vortices. However, the cost of constructing a 75- to possibly 108-foot-high structure, depending upon acceptable reservoir design level, was considered prohibitive. Tapering the walls to reduce the cost greatly reduced their effectiveness to eliminate vortices as the reservoir rose.

After abandoning the use of guidewall arrangements, many changes were made to the piers and thrust block geometry. Vortices were persistent in the inside bays of each spillway between the upstream nose of the piers and the thrust block face. The 1:3 slope was changed to vertical between the spillway bays and on either side of the entrance along the entire length of the thrust block. This reduced the area for recirculation and provided somewhat improved flow around the inside pier (fig. 20). After removing previous guidewall modifications, the inside pier nose was extended to the end of the thrust block by adding a rounded section with a 5-foot radius next to the existing pier. This pier extension did not improve the flow conditions significantly (fig. 21). Generally, modifications to the inside piers reduced the intensity of the vortex above the inside bay of each spillway but caused a more persistent vortex to form above the outside bay.

As none of these previous modifications were acceptable, vortex suppressor racks were mounted between the piers above the intakes. The idea for the suppressors came from those typically designed for low-level outlet works or pumped storage facilities (Colon, 1970; Government of India, 1987). Available racks and screens, of no particular dimension, were used to test the effectiveness of the suppressors. The suppressors were then tested with all the different thrust block and pier configurations discussed previously. The suppressors function by inhibiting the vortex circulation, thus breaking up the air core or "tail" as the vortex passes through the suppressor members.

The suppressors, with a vertical dam face between the bays, prevented air core vortices from entering the spillway, although strong recirculation patterns were still evident near the intakes (fig. 22). Therefore, it was hoped to improve the general flow conditions upstream of the spillway intakes by modifying the entrance to be flush with the face of the 1:3 thrust block. Vortices have successfully been eliminated at other structures by allowing the flow to sweep by the intake. The right spillway intake was so modified; however, the dead flow area in the existing approach channel to the outside of the spillway still produced a strong flow shear boundary and some vorticity. Filling in the area outside or to the right of the right spillway effectively eliminated the vortices; however, this would be too expensive in the prototype (fig. 23).

The vortex suppressors seemed to be the only method that would eliminate the vortices upstream of the spillway intakes. Two different suppressors were designed: one for the right spillway entrance, which had been modified to be flush with the thrust block face, and one for the left spillway entrance with the original pier and thrust block design.

For the right spillway, an open, flow-through to solid structural area ratio of 20 percent was constructed. Previous investigators (Colon, 1970; Government of India, 1987) used suppressor baffle wall designs with 7 to 20 percent flow-through ratios. The suppressor was constructed of solid wood with circular holes drilled for the flow-through area. The suppressor was mounted flush with and on top of the right spillway piers. The suppressor extended initially to elevation 2218, then was later reduced in height to elevation 2175 (fig. 24). A top horizontal member was also added to further reduce the possibility of a vortex forming behind the structure. Two different flow-through ratios were tested with no apparent difference in effectiveness. Therefore, it was determined that the flow-through area to solid area was not critical in this range if there was flow passage. Air core vortices would occasionally still form in the approach channel upstream of the suppressor, but would break up as the tail hit the suppressor.

The left spillway suppressor was designed with 2- by 2-foot horizontal members spaced at 3-foot intervals across the width of each bay. The members were located flush with the pier noses from above the top curve of the spillway intake to elevation 2175, then back toward the dam face (fig. 25). The surface recirculation was very strong; however, no air core vortices formed above the intakes for any operation from elevation 2151 to 2218 (fig. 26).

After evaluating both intake and suppressor designs, the left spillway suppressor configuration was chosen for the final design. In general, flow conditions were less turbulent with the intake and suppressor design for the right spillway entrance; however, the designers determined that the beam design of the left spillway suppressor offered better structural stability and simpler construction.

Spillway Geometry

Spillway geometry was based upon impinging the jets together into a common plunge pool. The geometry was limited by the steep dam abutment areas and the location of the spillways within the thrust blocks. The spillway geometry was therefore investigated to ensure:

- Adequate spillway capacity.
- Proper flow conditions in the chutes.
- Jet trajectories and impingement location in the plunge pool.
- Proper spillway alignment to produce a downstream flow component in the river channel.
- Safety of the dam toe and powerplant road.

The right and left spillways were similar through the straight conduit sections to downstream of the top-seal radial gates. The conduits were 18 feet wide by 28 feet high with a 14-foot-wide center pier for a total width of 50 feet at the gate seat location. The right spillway, with invert slope 0.17 and a straight chute throughout, flared just downstream of the gates to a width of 80 feet at the end of the 30° flip bucket. The left spillway, with invert slope 0.12 and a superelevated chute, also flared to 80 feet at the end of a 30° flip bucket. The steeper invert slope of the right spillway causes the right spillway gates to be set at a lower elevation than the left.

Initial operation of the spillways verified the predicted intersection point of the two spillway jets. However, several aspects of the spillway geometry needed modification to improve chute flow conditions and plunge pool energy dissipation. The flare angle, 9.68°, of the right spillway was too large. This caused some flow separation along the chute side walls and the right spillway jet to carry through that of the left jet at the upstream intersection of the jets. Nonuniform flow depth at the end of the left spillway chute indicated excessive superelevation. The nonuniform chute flow depth also affected the ability of the left spillway jet to offset that of the right spillway jet.

The blunt downstream ends of the piers produce a fin in the chutes, particularly in the right spillway chute, as the flow from either side of the pier meets in the flip bucket. The fins, which were most severe at intermediate gate openings, should not be a concern since they drown out significantly at higher flow rates. The end width of the right spillway was reduced by 10 feet, producing a 70-foot width. This reduced the flare angle to 6.43°, eliminating the flow separation along the side walls and improving the intersection of the jets at the upstream end of the plunge pool. The superelevation of the left spillway had been designed by determining velocities from a flow profile computer program and using them in the calculation method proposed in Hydraulic Laboratory Report No. 74 (Owen, 1940). Results from Laboratory Report No. 74 are good providing initial velocity data are correct. Water surface elevations were measured in the model from which velocities and a new superelevation were computed.

At this point, recalculated flood routings required that more discharge be passed by the spillways, and the design lowered the elevation of the outlet works jet-flow gates, which did not allow their use with the spillways discharging at reservoir elevation 2175 due to submergence by the tailwater.

Therefore, the right and left spillway gates were widened to 19 feet and the left spillway reconstructed after these initial observations. Discharge coefficients were then determined for both spillways with the gates fully open and varying reservoir elevation. As a result of this and changes in operating criteria as explained earlier, the spillway gate widths were again increased - to 21 feet - to pass 135,000 ft^3/s at elevation 2175. Maintaining a 50-foot chute width at the gate seat resulted in an 8-foot pier width. Flow depths in the left spillway were not uniform. To improve the flow, a tapered insert was installed on the inside of the curved chute, thus reducing the flow depths in this area. Remeasurement of both the right and left spillway chutes showed uniform flow depth across the width of the chutes. These measurements were used to determine chute wall heights (table 1).

Uniform flow depth across the left spillway chute improved flow conditions in the plunge pool by better offsetting the jet from the right spillway. The spillways, with 21-foot-wide gates, the right spillway end width at 70 feet, and the redesigned left spillway chute superelevation are shown discharging into the original plunge pool in figure 27.

Discharge curves were developed for the 21-foot-wide gates of the DCAD spillways. During investigation of the spillway geometry and development of these discharge curves, observations were made of the plunge pool dissipation characteristics. Ideally, the jets from the spillways were to impinge, offset one another, and force a largely downstream component within the flow leaving the plunge pool. The initial pool design and location did not produce the desired flow in the plunge pool and downstream river channel. Therefore, the plunge pool was significantly modified, making it deeper and more centered about the impingement point of the spillway jets. Tests with the design discharge after the plunge pool was modified showed a definite downstream component to the flow. As a result, the spillway alignment was determined acceptable, and it was felt that adequate energy dissipation could be achieved with further plunge pool modifications.

The right spillway jet still tended to overpower the jet from the left spillway, producing excessive wave action along the powerplant road. Due to the flow component toward the road, two other changes were made to the right spillway which were not adopted but are worth mentioning. Both changes attempted to break up the right spillway jet, thus reducing the energy before it entered the plunge pool.

Table 1. - Water surface profile data measured vertically (in feet) for the DCAD option.

Station	135,000 (ft^3/s) El. 2175 Rt. wall	135,000 (ft^3/s) El. 2175 Lt. wall	150,000 (ft^3/s) El. 2218* Rt. wall	150,000 (ft^3/s) El. 2218* Lt. wall	187,000 (ft^3/s) El. 2218 Rt. wall	187,000 (ft^3/s) El. 2218 Lt. wall
			Right spillway			
1+85.82	28.13	28.33	29.58	28.33	28.13	28.33
1+95.82	25.00	25.00	25.00	26.67	25.00	26.67
2+05.82	22.92	21.25	23.96	25.00	22.08	25.00
2+15.82	20.00	17.92	22.08	21.25	23.75	21.25
Bucket						
2+39.45	17.92	16.25	20.83	20.63	23.75	20.63
2+49.45	17.50	16.25	20.83	21.67	23.75	22.08
2+59.45	17.54	16.88	20.83	20.83	23.33	22.08
2+69.45	17.08	17.71	20.42	20.83	22.50	22.92
2+79.45	16.66	16.66	20.83	22.08	22.50	22.98
			Left spillway			
1+84.55	27.08	27.92	27.08	27.92		
2+06.55	24.58	24.58	24.58	25.42		
2+19.36	20.83	23.33	20.83	24.58		
2+46.55	18.96	21.46	18.96	24.17		
2+71.55	16.88	18.75	16.88	21.88		
Bucket						
3+04.14	17.29	17.71	17.29	21.67		
3+14.14	17.71	17.29	17.71	21.25		
3+24.14	19.17	17.08	19.17	21.25		
3+34.14	21.25	16.25	21.67	20.21		
3+44.14	20.00	15.63	22.50	18.33		

* Q = 187,000 ft^3/s at elevation 2218 produced very similar flow depths in the left spillway.

The first change was to simulate a slotted flip bucket on the right spillway by extending the 30° flip bucket with alternating wedges of 45° and 30° about 10 feet wide and 5 feet long. It was hoped to fragment the jet, thus promoting aeration and increasing the surface area to volume ratio of the jet before it entered the plunge pool. Observations of the jet after it left the slotted bucket indicated that the dimensions of the slots would have to be much greater to accomplish adequate separation of the jet. Therefore, the slotted bucket was not used. The second change was to split the right flip bucket in half along the spillway centerline. The left side of the bucket remained at 30° and the bucket angle on the right side was decreased to 20°. It was hoped that the left spillway jet could more easily offset the component of the right spillway jet if the right jet were split producing two impingement angles. Due to the large flow depth in the spillway chute at design discharge, the jets were not significantly affected by this change and the plunge pool operation was not visibly altered.

At this point, it was felt that further improvement in plunge pool energy dissipation would have to be made with changes to the pool and not by further modifications to the spillway geometry. The recommended geometry for the spillways of the DCAD option is shown on figures 28 and 29.

DOUBLE-CURVATURE ARCH DAM - PLUNGE POOL AND RIVER CHANNEL INVESTIGATIONS

The design discharge condition for the plunge pool is 135,000 ft^3/s at reservoir elevation 2175. The spillways were aligned to impinge the jets together and thus minimize the size of the plunge pool excavation. The plunge pool studies were performed to ensure that high-velocity flow from the spillway jets was confined to the plunge pool area, thus not causing excessive erosion at the toe of the dam or within the river channel that might produce undermining of the powerplant road retaining wall.

Plunge pool investigations began after the spillway geometry was finalized. The investigation included locating, shaping, and determining the depth of a nonerodible plunge pool in the model. Both static and dynamic pressures were measured in the plunge pool. Velocities were measured around the pool and in the river channel downstream. Portions of the pool were tested with an erodible concrete mixture, and the river channel alluvium was modeled for determining the erosion and deposition patterns surrounding the pool. These studies were used to determine the dimensions of a pre-excavated plunge pool and to address the need for lining the pool.

Theoretical Plunge Pool Depth

Many attempts have been made to predict the depth of erosion for unlined plunge pools for free-falling jets. Several references were used in predicting the maximum depth of erosion for the Roosevelt plunge pool. The scour depth is a function of the jet velocity or unit discharge, head, turbulence intensity, aeration, tailwater depth, and particle size. The scour depth was calculated based on work by Johnson (1974), Ervine (1985), Ervine and Falvey (1987), and Mason (1985, 1989). Equations for predicting scour are usually of the form:

$$D = K\,(q^x H^y)/d^z$$

The equation (Mason, 1985) used for predicting the scour depth for Roosevelt Dam was:

$$D = K\,(q^x H^y h^w)/g^v d^z$$

where:

D = depth of scour below the tailwater elevation, meters
q = discharge per unit width of the jet at impact with the tailwater, m^2/s
H = head drop from the reservoir level to the tailwater, meters
h = tailwater depth, meters
g = acceleration due to gravity, m/s^2
d = particle size of the bed material, meters
$K = 6.42 - 3.10H^{0.10}$
$x = 0.60 - H/300$

$y = 0.05 + H/200$
$w = 0.15$
$v = 0.30$
$z = 0.10$ ($d = 0.25$ m for prototypes and d_{50} or mean size for models)

The plunge pool floor elevation was initially predicted to be elevation 1868, based upon a design discharge of 54,380 ft^3/s at elevation 2175. The design required a pre-excavated plunge pool (either lined or unlined), the depth would be confirmed by model results. Increasing the design discharge to 135,000 ft^3/s required the plunge pool floor elevation to be lowered to elevation 1839 given an assumed jet divergence angle of 3° after leaving the spillways.

The erosive properties of the plunge pool material are also a consideration. The scour depth of alluvium may be predicted theoretically perhaps better than the reaction of fractured rock to the jet impact and pressure fluctuations. Mason (1989) has since conducted further research regarding the effects of jet aeration on plunge pool scour. The equation is quite similar to the previous one except that the head term is replaced by an aeration coefficient. This equation compared model/prototype data, as did the one presented above, to determine its applicability. Results were good for both equations, except for the model and theoretical predictions of the scour at Kariba Dam. It has been postulated that the rock type and the effect of multiple spillway jets produced the unpredicted scour patterns.

Location and Geometry

The location and shape of the initial plunge pool are shown on figure 3. The pool shape and location had been selected from a compromise between a smaller shape centered on the impingement point determined adequate by the designers and a shape covering a much larger area deemed necessary by a consultant review board. Operation of the model under the original design discharge of 54,380 ft^3/s showed that no major modifications would be necessary (fig.17); however, operation under the revised design discharge of 135,000 ft^3/s indicated all components of the plunge pool needed reevaluation (fig. 27). The pool was located too far to the left of the jet impingement area, was too shallow, was improperly shaped, and did not direct the flow toward the river channel.

Numerous plunge pool modifications were then tested with the jets produced by the finalized spillway geometry (fig. 30). The first modification was to make the plunge pool rectangular, 172.5 feet wide by 180 feet long; 21.76 feet deeper to elevation 1846.33; and angled to release the flow into the river channel (fig. 31). Testing with this plunge pool depth and shape indicated that the plunge pool could successfully be further modified to protect the powerplant road and produce a downstream orientation to the released flow. However, the pool from this first modification appeared too small as the flows around the pool, except at the toe of the dam, were quite turbulent (fig. 32). The plunge pool depth seemed appropriate and was maintained while the pool was enlarged and reshaped.

The second pool modification was basically triangular with the upstream apex flat, not pointed. The right side of the triangle was formed by extending the right wall of the previous shape; the left wall was moved back toward the road and oriented almost parallel to the end of the left spillway. The downstream edge of the pool followed the 1880 topography contour between the pool area and the downstream river channel (fig. 30). This shape was chosen to contain more of the boil from the right spillway, thus hopefully protecting the road. The upstream end of the pool was formed by the flattened apex of the triangle and was thus significantly narrower. In the previous tests, a boil had formed between the underside of the jet and the plunge pool boundary. With the sides of the pool close to the jet, flow conditions underneath the jets were improved. Tailwater along the toe of the dam was very calm. The remaining area of concern continued to be flow near the powerplant road and outlet works exit channel. This flow was caused by the left spillway jet not entirely balancing that of the right spillway. Since expanding the pool toward the road did not seem to improve conditions measurably (fig. 33), the next approach was to force the jump in the pool sooner.

The jump was forced by inserting benches from the left corner toward the center of the pool perpendicular to the centerline of the right spillway jet. The first bench tested was about 5 feet high and extended about 85 feet from the left corner toward the center of the pool. Tests were also conducted with the bench width increased, thus moving the step closer to the downstream edge of the left spillway jet. Neither of these bench arrangements had adequate height to significantly improve flow conditions near the road. The bench was then placed across the pool just downstream of the impingement point. This immediately forced the jump from the right spillway jet; however, the left spillway jet was then split by the bench, producing undesirable back flows under the jet.

While attempting to force the jump further upstream in the pool, material between the downstream end of the pool and the river channel was removed. It was hoped this would reduce the height of the boil near the left side of the pool and release more flow to the center of the river channel. Considering the amount of material requiring excavation, the slight difference in plunge pool energy dissipation was not significant. Several other minor changes were made to the shape that did not make noticeable differences in the flow conditions.

Rather than bench the left downstream wall, it was decided to move the wall in along the right spillway centerline to the elevation of the rock contour. This quickly forced the jump in the pool and improved the alignment of the flow leaving the plunge pool with the river channel.

The extensive plunge pool geometry testing revealed that the side of the pool under the left spillway should be close to the jet to eliminate formation of a boil underneath the jet, the upstream edge of the pool should be as close to the intersection of the jets as possible to reduce excavation, and a narrow downstream end of the pool is needed to impose a strong downstream component to the flow. The recommended shape, with dimensions, is shown on figure 30 where it may be compared to the previously tested configurations.

The recommended plunge pool shape for the DCAD design, figure 34, was selected because it:

- Provided adequate energy dissipation for the design discharge (fig. 35).
- Provided a definite downstream component to the flow exiting the pool.
- Prevented a high-velocity jet from impacting on the powerplant road.
- Produced calm flows at the toe of the dam and underneath both spillway jets.

The issues of the plunge pool depth and lining remained. The plunge pool depth had not been altered since the first modification. Because excavation for the plunge pool will be very expensive, further investigation of the floor elevation was undertaken. The above recommended shape was used to determine the floor elevation and to resolve the plunge pool lining issue.

Plunge Pool Impact Pressures

Two pool floor elevations, 1841 and 1861 feet, were used for studying flow patterns and measuring pressures. Data were taken at total discharges of 22,000, 90,000, and 135,000 ft^3/s at reservoir elevation 2175 through both spillways. These data were used to determine the required pool depth and provide information for making the decision on whether or not the pool should be lined.

Instrumentation. - Static pressures and low-frequency fluctuating pressures were measured on the plunge pool floor and walls with 48 piezometer taps. The taps were attached to a manual scanivalve and a 1-lb/in^2 Pace transducer. The transducer signal was then filtered and sent to both a strip chart recorder and an HP3457A digital voltmeter. The mean, maximum, minimum, and standard deviation of the pressure fluctuations at each tap location were recorded. The statistics were based on a sample size of 200 readings. The static pressure results then determined the placement of four 100-lb/in^2 Kistler flush-mount dynamic pressure cells. Three of these were located within the jet impingement area, and one was placed on the downstream left pool wall where the jet from the right spillway caused a large boil. The signal from each dynamic transducer was passed through a digital low-pass filter at 20 hertz and stored on a magnetic tape recorder for later analysis. A map of the piezometer tap and dynamic cell locations is shown on figure 36.

Analysis of plunge pool pressure data. - The plunge pool floor, either elevation 1841 or 1861, was used as a reference elevation allowing direct comparison of the floor and side wall data regardless of the tap elevation. The side wall taps were located at 5 and 25 feet above the 1841-foot floor elevation. Only the 25-foot-high taps remained after raising the pool floor to elevation 1861. The static pressure data were converted to feet of head above the floor elevation and included the tailwater elevation plus the impact pressure. Plots of equal pressure contours based on the mean static data clearly show the jet impact area for each floor elevation and discharge (figs. 37 and 38). To include the side wall pressures in these plots, the pressures were calculated based on the tap location above the floor, then averaged vertically and plotted directly below the wall taps on the floor. The mean static pressure rise from the taps located within the jet impact area for each discharge and pool elevation tested is shown in table 2.

Table 2.- Static pressure data in the jet impact area for each plunge pool floor elevation.

Discharge (ft^3/s)	Floor El. (ft)	Tailwater El. (ft)	Tailwater referenced to floor (ft)	Measured head (ft)	Impact head (ft)
135,000	1841	1937.4	96.4	120	23.6
	1861	1937.9	76.9	110	33.1
90,000	1841	1926.3	85.3	110	24.7
	1861	1925.2	64.2	110	45.8
22,000	1841	1912	71	71	0.0
	1861	1914	53	53	0.0

For the design discharge, floor elevation 1841, the static head of 120 feet included 96 feet of tailwater plus about 24 feet of impact pressure. For the design discharge, floor elevation 1861, the static head of 110 feet included 77 feet of tailwater plus 33 feet of impact pressure. Of the discharges tested, 90,000 ft^3/s has the highest impact heads on the pool floor. This can be attributed to the impact angle of the jets at this discharge compared to the impingement of the jets at 135,000 ft^3/s. Comparison of the static pressure data for the two floor elevations at 90,000 ft^3/s shows about 19 percent of the impact head reaches the pool floor at elevation 1861. This can be reduced to 10 percent by lowering the floor to elevation 1841.

The dynamic pressure data describe the pressure fluctuations around the mean static pressure. A spectrum analyzer was used to analyze the data records in the time domain. Probability density histograms were composed from time records of the dynamic pressure data. Minimum and maximum pressures and confidence limits were then determined. The maximum and minimum pressures and the 95-percent confidence limits are listed in table 3 for the design discharge and 90,000 ft^3/s. (The 22,000-ft^3/s discharge did not produce significant pressure fluctuations.)

Table 3. - Dynamic pressure data in the jet impact area for each plunge pool floor elevation.

Discharge (ft^3/s)	Floor El. (ft)	Mean pressure (ft)	Max. pressure (ft)	Min. pressure (ft)	95% limit
135,000	1841	120	127.3	56.4	32.8 to -40.7
	1861	110	140.4	72.7	39.7 to -47.9
90,000	1841	110	128.6	65.9	29.7 to -36.3
	1861	110	135.2	69.2	29.7 to -39.6

The confidence limits mean that 95 percent of the peak pressures that occur will be between the values given for each flow condition. For the design discharge and floor elevation 1841, this means that 95 percent of the peak pressure amplitudes will be between 33 feet above the mean pressure head and 41 feet below. Instantaneous maximum and minimum values may exceed these limits; however, these pressures would occur infrequently. The time records of the pressure fluctuations show no periodic tendencies, only random fluctuations. Pressure fluctuations caused by the large-scale turbulence of the jet fragmentation in the plunge pool should scale to the prototype in the same manner as the static data. Based on current engineering technology, it is difficult to predict the erosion potential of rock-lined plunge pools in response to dynamic pressure fluctuations. The dynamic pressure data, as did the static data, showed slightly higher pressure fluctuations with the shallower pool. However, the difference measured was not considered significant enough to clearly choose a floor elevation or determine the need for lining the pool.

Further information regarding flow velocities and erosion potential was investigated to supplement the pressure data before making a decision on plunge pool depth and lining.

River Channel Velocities

Velocities were measured for the design discharge for both plunge pool floor elevations, 1861 and 1841. Data were gathered with an OTT propeller meter around the circumference of the plunge pool and in the river channel at stations 200 (adjacent to the warehouse) and 320 feet downstream from the end of the plunge pool. Data were taken in the downstream channel every 40 feet across the width of the channel. Velocities in the downstream river channel were measured near the bottom, center, and top of the water surface, depth permitting, then averaged. The flow conditions around the pool were quite turbulent, thus making accurate velocity measurements difficult. The velocity data measured around the plunge pool were taken near the bottom and top of the water surface, then averaged. Surface flows measured adjacent to the pool had quite high velocities. Velocities near the bottom which could cause undermining of the powerplant road or erosion at the dam toe were much lower.

Lowering the plunge pool floor elevation from 1861 to 1841 had a negligible effect on the velocities measured near the pool boundaries. Figure 39 shows the location, magnitude, and direction of the velocities measured for the recommended DCAD plunge pool shape at floor elevations 1861 and 1841. Velocities at the toe of the dam were low, 1 to 5 ft/s. Velocities measured underneath the right spillway jet indicated flow back toward the cliff and the dam at 6.7 to 8.1 ft/s. Measurements under the left spillway jet indicated velocities toward the powerplant road of 2.3 to 3.8 ft/s. Flow velocities of 6 to 10.4 ft/s were measured exiting the pool toward the outlet works and powerplant road. The sections across the downstream channel indicated higher velocities along the right bank (19 ft/s) and the left bank (14 ft/s) than in the center of the channel (4 to 9 ft/s).

The velocity results also did not produce a clear choice between plunge pool floor elevations.

Resolution of Plunge Pool Floor Elevation

Since the pressure and velocity results were not significantly different for the two pool elevations, it was decided that the movable bed and erodible plunge pool results would also probably be very similar. Therefore, it was decided that pool elevation should be chosen before continuing with the movable and erodible bed modeling. The bed modeling would then be used to verify the issue of plunge pool lining.

Video tape and still photography were used to document the impingement zone and surface flow conditions of the two pool elevations under the design discharge. Observations of the flow conditions for plunge pool floor elevations 1841 and 1861 were:

- Flow conditions at the toe of the dam and river channel downstream of the plunge pool were almost identical with either pool elevation.
- The height of the boil in the plunge pool was less with the floor at elevation 1841.
- Better flow conditions and less wave action occurred along the powerplant road retaining wall and near the outlet works and warehouse with the pool floor at elevation 1841.

Weighing the results from the pressure and velocity measurements, flow observations, and the generally poor quality of the rock, it was decided to pre-excavate the plunge pool to elevation 1840.

The final recommendation on the issue of lining the plunge pool, floor at elevation 1840, was addressed by conducting movable riverbed studies and replacing the fixed pool walls with erodible material. The pool floor remained fixed at elevation 1840, which would produce conservative results for lateral erosion.

Movable Bed Modeling

The fixed model topography, which had previously modeled the elevation of the alluvium, was removed and lowered to model the rock elevations. The excavated plunge pool was still modeled with a fixed nonerodible material, floor elevation 1840. A material gradation for the alluvium was obtained from two independent visual surveys of the prototype river channel downstream of an existing weir. The alluvium gradation was as follows:

20 percent less than 1 foot
60 percent between 1 and 3 feet
20 percent between 3 and 12 feet

Alluvial material was then geometrically sized for the model, mixed, and placed over the rock contours. The material was placed on 3:1 slopes from 25 feet upstream and downstream of the plunge pool cuts to the required alluvium elevations at the dam toe and the downstream river channel. Due to excavations for the powerplant road and the plunge pool, all the alluvium will be removed between the plunge pool and the powerplant road (fig. 40). Material was left in the scour hole, made by the original right spillway, located to the right of the new pool.

Progressive erosion and deposition patterns of the alluvium were studied by increasing the discharge in stages. Visual comparisons, photographs, video, or survey mappings were used to document each flow condition and movement of material before proceeding to the next higher discharge. The test plan was based on the expected prototype operation sequence. First, fully open the outlet works, then add right spillway discharge until a total of 25,000 ft^3/s at reservoir elevation 2175 is attained. For passing discharges above 25,000 ft^3/s, the outlet works are closed and spillway discharges increased according to the operating criteria. Each test modeled about 2 hours of operation, except the 135,000- and 150,000-ft^3/s discharges, as noted in table 4, which lists the progression of the tests.

Testing began with the outlet works operating alone to a maximum of 11,517 ft^3/s at reservoir elevation 2175 (fig. 41). Right spillway flows were then added to the outlet works flow for a total discharge of 25,000 ft^3/s at elevation 2175. The right spillway discharge was then increased to 25,000 ft^3/s at elevation 2175 and the outlet works closed. For this condition, a clockwise flow pattern developed to the right of the plunge pool. Surface flows moved from downstream of the pool upstream along the right side of the pool and underneath the right spillway jet. The recirculating flows were not strong enough to move material into the pool or at the toe of the dam. Material did not deposit in the plunge pool for any of the outlet works or right spillway tests when total discharges were less than 25,000 ft^3/s. Material from the 3:1 slope in front of the warehouse moved slightly downstream from operation of the outlet works (fig. 42).

Both spillways were then operated to a total discharge of 70,000 ft^3/s at elevation 2175. This intermediate discharge was selected for testing because it was the previous maximum release from the existing dam. Material moved into the pool from the old scour hole to the right and just downstream of the plunge pool. The movement of material was observed by using an underwater borescope which was effective when air bubbles were not prevalent in the flow area. The picture was then transmitted to a television monitor for viewing and recorded on a video tape. The boil releasing from the pool produced a high-velocity surface jet which in turn produced a flow reversal underneath, drawing material back toward the pool. After observing the flow conditions, the model was turned off and drained. Material was deposited inside the pool along the right side and in the river channel downstream (fig. 43).

Table 4. - Alluvium studies test plan for DCAD plunge pool.

Operation mode	Reservoir El. (ft)	Total discharge (ft^3/s)	Tailwater El. (ft)	Documentation method
Outlet works only	2151	10,924	1914.3	observation
Outlet works only	2175	11,517	1914.5	photo
Outlet works + right spillway	2175	25,000	1917.5	observation
Right spillway only	2175	25,000	1917.5	map
Both spillways	2175	70,000	1926.2	map
Both spillways	2175	135,000	1938.0	map, 14 h
Both spillways	2218	150,000	1938.0	map, 4 h

The design discharge, 135,000 ft^3/s at elevation 2175, and PMF, 150,000 ft^3/s at elevation 2218, were tested for 14 and 4 hours, respectively. These durations were determined from the flood routings. During both of these tests, material from the right side of the pool continued to enter the pool but was then swept out and deposited downstream. Material below the downstream edge of the pool was no longer drawn into the pool but was eroded and deposited further downstream. Material was eroded from the right side of the channel downstream of the hole, along both sides of the downstream river channel, and from in front of the warehouse to the elevation of the rock contours (figs. 44 and 45). Again, no material moved at the toe of the dam.

An attempt was then made to simulate movement of material eroded from an unlined pool under the design condition. With the erosion pattern remaining from the previous test, the entire floor of the plunge pool was covered with about 20 feet of the same alluvial material. The material was washed from the upstream portion of the pool, depositing in the downstream end of the pool and river channel (fig. 46).

The results of the movable alluvium bed study indicated:

- Operation of the outlet works does not deposit material in the plunge pool.
- No alluvium moved from or was deposited at the dam toe.
- Planned alluvium excavation, 25 feet upstream and downstream of the plunge pool and adjacent to the pool, will be adequate.

- Material eroded from the plunge pool during the design flow rate will move downstream and be deposited in the river channel.

Plunge Pool Wall Erosion Studies

The upstream side walls of the plunge pool were replaced with an erodible concrete mixture to investigate the possibility of high-velocity jets causing erosion upstream toward the dam toe or the powerplant road. Several trial mortar mix samples were prepared and tested for erosion. A mix of 30 parts sand, 1 part cement, and 2 parts water by volume was used. The mortar was placed and allowed to set overnight (fig. 47). Prior to testing, the walls could be eroded by scraping the surface with one's hand. The design discharge was tested first without and then with material in the plunge pool at initial startup. No erosion of the side walls occurred with the pool initially free of material. When tested with material in the pool, most of the material again washed out or armored the downstream slope. However, a couple of large rocks remained in the upstream portion of the pool, ball milling both side walls. No ball milling was observed on the most upstream face of the pool (fig. 48).

Resolution of Plunge Pool Lining Issue

The decision was made to have an unlined, pre-excavated plunge pool at elevation 1840. The decision was based upon the location of the jet impact area, the acceptable static pressure magnitudes, low velocities at the toe of the dam, and the erodible bed studies showing no high-velocity jets directed toward the dam toe that would endanger the dam stability. Theoretical calculations of scour depth indicated elevation 1840 would be a conservative elevation for the erosion potential of the spillway jets. In spite of this, it is assumed that measured dynamic pressure fluctuations will cause further erosion due to rock fracturing. A calculated assumed "worst-case" prototype rock erosion pattern would still not endanger the toe of the dam. The deep, pre-excavated pool will provide adequate stilling action for the design discharge, and, as the pool erodes further, especially in the jet impact area, greater energy dissipation will be achieved. A lined plunge pool would have to be designed to withstand the pressure fluctuations. Also, it would abrade if material should enter the basin from any source. Partial lining was not chosen because if erosion occurs anywhere in the pool, the lining could be undermined and the eroded material would be a source for ball-milling action on the remaining lined portion.

Inspection of the plunge pool, however, is recommended after spillway discharge to map any erosion patterns.

SINGLE-CURVATURE ARCH DAM - FINAL SPILLWAY AND PLUNGE POOL DESIGNS

Selection of the single-curvature arch dam method for raising the dam resulted in several modifications to the spillway designs. Initially, it was felt that the modifications to the spillways, and consequently the plunge pool performance, would be minor. The proposed design changes were initially checked by making minor modifications to the hydraulic model of the DCAD spillways and plunge pool.

The right thrust block face and spillway entrance were modified to produce a vertical upstream face. The left side wall of the plunge pool was also repositioned for the new impingement point to assess the applicability of the previous design recommendations. These tests indicated that the proposed alterations to the spillways would basically be acceptable. However, it was felt that the entire final design should be modeled primarily to confirm extrapolation of the DCAD results for the spillway vortex suppressors and the plunge pool energy dissipation to the final SCAD design.

Geometry Differences

The major differences in the right spillway geometry were:

- The centerline of the spillway was moved toward the center of the dam by 35 feet. The alignment and slope remained the same.
- The sloping upstream dam face, where the thrust blocks were located, was changed to vertical, and the entrance was moved upstream 3.34 feet into the reservoir.
- The spillway chute was lengthened 6.73 feet.
- The pier noses were made normal to the dam face, not parallel to the spillway centerline.

The major differences in the left spillway geometry were:

- The centerline of the spillway was moved toward the center of the dam by 17.5 feet.
- The sloping upstream dam face was made vertical, the entrance was moved upstream 6.66 feet, and the pier noses were aligned normal to the dam face.
- The spillway slope was increased from 0.12 to 0.14.
- The spillway chute was shortened 49.37 feet.
- The radius forming the chute was shortened 1.88 feet.

These differences between the DCAD and SCAD spillway designs required extensive changes to the existing model. Both spillways had to be removed, modified, and reinstalled with new holes cut in the dam for the new spillway locations and the old ones patched. The right spillway required only minor changes; however, the left spillway superelevation was entirely reconstructed. In addition, the approach channel bottom cut slopes were steepened from 0.1 to 0.2, and a rock trap was added at the upstream edge of each spillway entrance. (This change had already been designed for the previous spillways but had not been modeled.) The vertical dam face and the new stoplog pier noses were then added. Because the spillway centerlines were both moved toward the center of the dam, the plunge pool was repositioned. The plunge pool was moved to the left 18.5 feet and upstream 35 feet to keep the impingement point relative to the pool geometry, the same as in the DCAD design.

The final spillway and plunge pool configurations are shown on the general plan (fig. 13). All aspects of the waterways were retested except the pressure measurements in the plunge pool, which were considered to be unaffected since the centroid of the plunge pool was moved to reflect the new spillway alignments. The extensive studies already completed for the waterways of the DCAD design led to very quick results for the final spillway and plunge pool geometries.

Spillway Flow Conditions

The flow conditions remained quite similar for the redesigned SCAD spillways. The flow conditions will be briefly described noting differences between the single- and double-curvature arch dam designs and additional studies.

Spillway discharge curves were recompiled due to the change in centerline locations and the steeper slope of the left spillway. The gate seat elevations for the right and left spillways are 2086.60 and 2088.73 feet, respectively. The discharge curves were developed for 4-foot gate opening intervals, measured normal to the spillway slopes. Discharge curves were developed for each spillway for both simultaneous and separate operating conditions (figs. 6, 7, and 8). The required discharge capacity was still maintained.

The correct left spillway superelevation was essential for producing uniform flow depth across the chute and acceptable plunge pool energy dissipation. New water surface profiles were recorded throughout the spillway chutes that proved the adequacy of the left spillway superelevation and determined wall heights for both spillways (figs. 9 and 11). Side wall pressures through the flip bucket were also measured to confirm theoretical stability computations on the diverging walls of both spillways and the effect of the left spillway superelevation. Pressure data were recorded for 135,000 ft^3/s at elevations 2175 and 2218, and 187,000 ft^3/s at elevation 2218, which is the maximum possible discharge from the dam (figs. 10 and 12). The flare of both spillway chute side walls was acceptable.

Vortex Suppressors

The spillway entrance design with the vertical dam face allows the stoplogs to travel down the face of the dam during installation and does not require the piers, as with the DCAD, to extend to the maximum water surface. The tops of the piers are at elevation 2150 and, therefore, are less of a factor in creating vorticity above the intakes. However, vortices continued to form in the approach channel and above the spillway intakes, largely due to the alignment of the channels with respect to the spillways. The same approach to solving the vorticity problem was used. Vortex suppressors for each spillway were located on top of the piers and attached to the face of the dam from elevation 2150 to 2175. The vortex suppressors were horizontal beams, 4 feet high by 2 feet wide with a 3-foot opening between each beam, that span the width of the intake (fig. 5).

No air-entraining vortices were observed in the model for spillway operation from reservoir elevation 2151 to 2175. Weak vortices, observed as surface dimples, formed in the approach channels and moved toward the spillway intakes. The suppressors effectively broke up the circulation of the vortices as they reached the face of the dam. Intermittent vortices may form above reservoir elevation 2175; however, it was felt that this was acceptable due to the operating criteria. The critical feature of the suppressor design is that water flows through the beams. The beam thickness is primarily dependent on structural integrity and not vortex suppressor effectiveness.

Gate Operations

Four operational modes, other than equal gate openings for both spillways, were investigated:

- Limitations of operating the right spillway only.
- Limiting flow onto the powerplant road during startup of the left spillway.
- Unusual gate operations to improve plunge pool performance.
- Single gate operations for each spillway in case of gate malfunction.

Extensive gate operations were investigated with the final spillway alignment and geometry. Flood control operating criteria require that both spillways be fully opened to release 135,000 ft^3/s should the reservoir elevation reach 2175 feet, assuming no flows intervene. The powerplant and outlet works will pass the normal flows. When spillway flows are needed, it would be desirable to operate the right spillway first because initial opening of the left spillway will cause flow to impinge on the powerplant road. The worst-case scenario would be to assume no intervening flows downstream of Roosevelt Dam and require the full 25,000 ft^3/s to be passed from the right spillway at elevation 2175. This flow is passed by a 12-foot gate opening of the right spillway (fig. 49). Further investigations showed that the right spillway may be operated alone to a gate opening of 20 feet, about 41,000 ft^3/s at elevation 2175 (plus any outlet works discharge) before introducing flows from the left spillway (fig. 50). Discharging more than 41,000 ft^3/s from operation of the right spillway only causes flow to sweep from the plunge pool and encroach on the powerplant road. If larger discharges are required, balancing flows from the left spillway must be introduced for proper plunge pool energy dissipation.

Operation of the right spillway only above 25,000 ft^3/s will, however, draw loose material from downstream and to the right of the plunge pool into the pool. Asymmetrical erosion patterns in the adjacent area and downstream river channel may also result. Since the plunge pool will be unlined and further erosion is expected, flow rates above 25,000 ft^3/s at elevation 2175 should be minimized due to the possibility of scouring an asymmetrical pool.

Initial opening of the left spillway gates is another operational concern. The shorter spillway of the final design increased the probability that flow will impinge upon the powerplant road below. The parapets on the powerplant road were designed to protect the road from the maximum tailwater; thus any flow on the road would be funneled down to the powerplant. The model was used to investigate if flow would impinge on the road and, if so, how to minimize the amount.

Model tests showed initial gate opening caused flow to hit the road, as a hydraulic jump formed upstream of the bucket in the superelevated chute. This low-velocity flow exiting the chute then cascaded down the cliff at the end of the spillway and onto the powerplant road. The initial flip of the jet from the bucket still impinges on the road (fig. 51). However, as the gates continue to open and the discharge increases, the momentum is great enough to carry the jet beyond the road.

A crude test apparatus was used for evaluating the amount of flow impinging on the road. A tray was placed on the road underneath the left spillway to capture the flow. The tray emptied through a pipe into containers of known volume outside the model. The right spillway was set up to pass 25,000 ft^3/s at elevation 2175. The left spillway was then opened and the right spillway simultaneously closed to maintain a constant reservoir elevation. The gates were operated at a constant rate, and a stopwatch was used to measure the time required for the jet to clear the road.

Several gate opening rates and gate sequences were tested for which flow impingement was measured. Tests showed that the average amount of flow hitting the road, roughly 575 ft^3/s, varied little with gate opening rate or single gate sequence, but increased to roughly 900 ft^3/s for dual gate operation. In other words, 900 ft^3/s could hit the road for a short time by opening the gates quickly, together; or the average discharge could be lowered by opening the gates individually. A decision was made to limit the quantity of water on the road at any given time; therefore, the best procedure is to open the gates one at a time, preferably the right gate first. Each gate should be opened until the bucket sweeps out before opening the next. Once both gates are open, they should be opened simultaneously until the jet entirely clears the road, since it may not after initial sweepout of the bucket. Gate travel rates of 1/2, 1, and 2 ft/min/gate were tested using the above sequence. The standard gate opening rate of 1 ft/min/gate was chosen for simplicity of operation.

As a result of these tests, a gate was designed that may be drawn across the road upstream of the powerplant, prior to initial operation of the left spillway. This solid gate would divert flows due to initial opening of the left spillway and prevent flooding of the powerplant while allowing continued traffic flow most of the time. It is expected that jet impingement in the plunge pool and operation of the left spillway will keep the powerplant road wet during spillway discharges.

A strong boil along the left edge of the plunge pool develops for the design and greater flow rates. This component, caused by the right spillway, could not be completely eliminated by spillway or plunge pool modifications. The boil can be reduced by slightly closing the outside gate of the right spillway when both spillways are operating simultaneously. The outside gate of the right spillway should be closed about 1.6 feet, normal to the invert, at the design discharge to reduce the boil height. This will, of course, cause the reservoir elevation to rise, to approximately elevation 2180.41. This procedure should be used if it appears that significant erosion is occurring toward the powerplant road with balanced gate operation.

Standard operating procedures for the spillway gates require the maximum difference between gate openings to be 1 foot except for the previously mentioned condition to, if necessary, reduce the plunge pool boil. This operating procedure should be followed, not only to ensure proper flow conditions in the chutes, but to produce adequate energy dissipation in the plunge pool. The operating criteria require all spillway gates to be fully opened when the reservoir reaches elevation 2175. Uniform operation should be used particularly during opening of the right spillway gates. This is due to the extremely short spillway length and the possibility of the fin from the center pier overtopping the chute wall at intermediate gate openings. The left spillway superelevation produces more acceptable chute flow conditions during uneven opening of the gates; however, uniform flow depths are definitely required from this jet to offset the right spillway jet and produce adequate plunge pool energy dissipation.

Reservoir Evacuation

Flow conditions during reservoir evacuation were also studied in the model. Evacuation was simulated by setting maximum reservoir elevation 2218 with the gates fully open. The discharge was then gradually decreased until the reservoir elevation decreased to the height of the topography in the spillway approach channels. The jet trajectories and flow conditions were noted. With decreasing reservoir head, the spillway trajectory lengths decreased, moving the impingement out of the excavated plunge pool. As the reservoir head decreased, the left spillway jet again impinged on the road as during initial gate opening, and the right spillway jet impinged close to the toe of the cliff. Eventually, hydraulic jumps formed upstream of the buckets in both chutes with flow exiting the spillways and cascading down the cliff faces. Erosion of the cliff faces could be expected unless they are cleared of loose material during construction of the spillways. Some enlargement of the plunge pool toward the direction of each spillway could also be expected; however, this should be minimal depending upon the tailwater and the length of time during which impingement outside the excavated pool occurs. The reservoir evacuation was documented on video tape.

Final Plunge Pool Design

The plunge pool geometry developed for the previous dam option was used for final design with the floor at elevation 1840 (fig. 13). The location of the pool was moved to the left and upstream to maintain the same jet impingement point relative to the pool with the final spillway alignment. Plunge pool energy dissipation remained similar with the exception that flows releasing from the downstream left side of the pool are closer to the road. Measurements of flow velocities between the pool and the road (fig. 16) did not show a significant increase compared to the previous design (fig. 39). The plunge pool adequately dissipated the energy of the design discharge. Flow at the toe of the dam remained calm. The final side wall slopes were modeled and did not have an appreciable effect on plunge pool flow conditions. The alluvial material was replaced in the river channel downstream and showed no significant difference in erosion or deposition patterns. The final spillway and plunge pool designs operating under the design discharge of 135,000 ft^3/s at elevation 2175 are shown on figure 14.

Additional tests were then performed to investigate the effect of outlet works and right spillway flows on the powerplant road retaining wall. There was concern that the close proximity of the plunge pool would cause undermining of the road if the plunge pool eroded further. The fixed upstream and left edges of the plunge pool and the topography between the plunge pool and the powerplant road were replaced with erodible material. The material consisted of a pea gravel and sand mixture. The pool walls and surrounding topography were built up in layers. Each layer of material was compacted and then covered with a thin layer (1/16 inch) of cement. The cement was used to produce thin horizontal lenses within the erodible material. The cement lenses prevented the material from slumping at the angle of repose of the material. A thin layer of cement (1/16 inch) was also placed over the face of the finished pool walls and topography (fig. 52). The material was fixed only enough to hold the shape of the plunge pool walls; there was basically no cohesiveness.

Progressive erosion tests were then documented for a total discharge of 25,000 ft^3/s. During initial filling to the tailwater elevation, the saturated material settled somewhat and the upstream pool wall slumped. As the upstream wall has shown no potential for erosion in previous tests, the slumped material was left and the tests were continued. For the first test, 11,000 ft^3/s was released through the outlet works and 14,000 ft^3/s from the right spillway at elevation 2151. This flow condition is the most likely operation. Some erosion occurred along the side wall near the upstream edge of the right spillway jet with no erosion from the outlet works operation (fig. 53). Material eroded from the side wall was deposited in the pool just downstream and to the right of the edge of the jet. No erosion occurred near the powerplant road retaining wall.

For the next test, 25,000 ft^3/s was released through the right spillway at elevation 2151. The erosion along the left pool wall progressed along the upstream edge of the jet in the flow direction. More of the wall eroded, and about an 80-foot section along the powerplant road retaining wall was eroded. The erosion along the powerplant road retaining wall was not due to undermining from progressive erosion of the plunge pool, but to surface flow from the plunge pool boil hitting the road wall and diving downward to the base. A small roller then deposited material in a ridge between the road and the pool (fig. 54). The downstream left pool wall also experienced some erosion but did not break through the layered area outside the wall. Eroded material again remained in the plunge pool.

The right spillway was then operated to 25,000 ft^3/s at elevation 2175. This basically followed the same erosion pattern as the previous test with more material removed from the previously eroded areas. The extent of the erosion near the base of the powerplant road did not grow parallel to the road, but did deepen slightly (fig. 55). Material continued to be deposited in the plunge pool toward the downstream end and underneath the jet.

Erosion at the toe of the road was caused by a surface jet diving down the road retaining wall, then digging a hole until the energy was dissipated. A 5-foot concrete bench planned for the toe of the retaining wall at elevation 1902 was not represented in the model. This bench will help deflect the flow component directed down the retaining wall out from the rock topography, which will deter the erosion. Placing a protective slab, or slight modifications to increase the rock bench excavations, will also prevent exposing the toe of the road to undermining. The noncohesive material used for modeling will not effectively show erosion depths but will point out the areas of greatest potential erosion. Erosion along the road was not from direct spillway jet impact, but from the surface flows. The plunge pool also did not self-clean under these flow conditions, and should the pool be partially lined, erosion of the lining would be quite possible.

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Figure 1. - Existing Theodore Roosevelt Dam - 1989. C 344- 330-031113 NA.

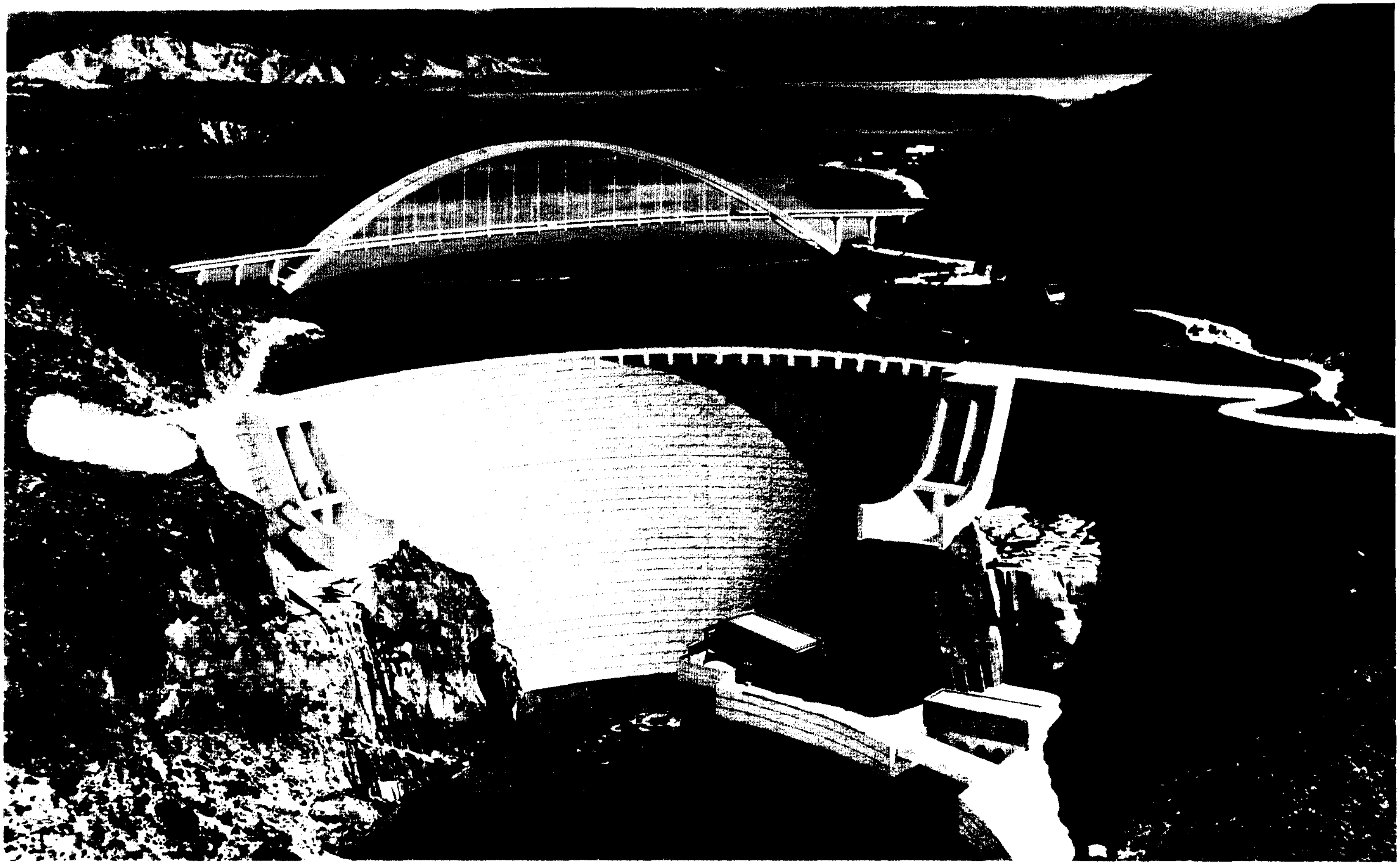


Figure 2. - Artist's conception of the proposed modifications for Theodore Roosevelt Dam - 1989.

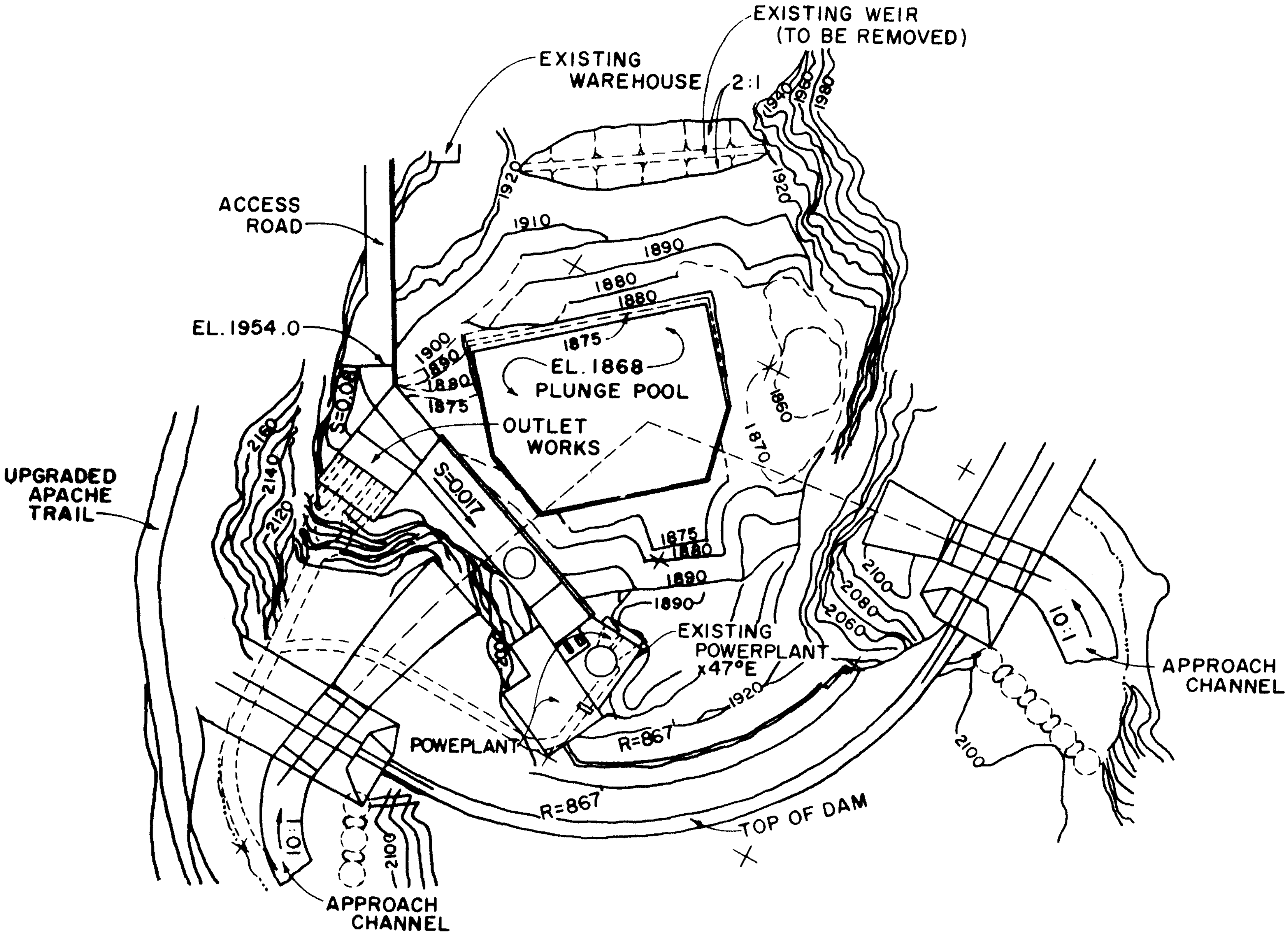


Figure 3. - Original design of the waterways for the double-curvature arch dam.

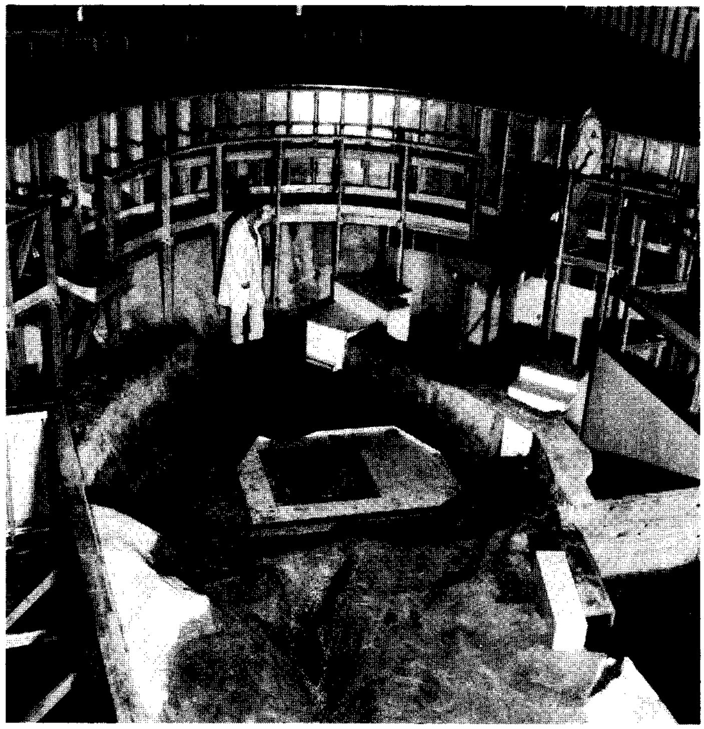


Figure 4. - Overall view of the 1:40 scale model with the original waterway designs of the double-curvature arch dam.

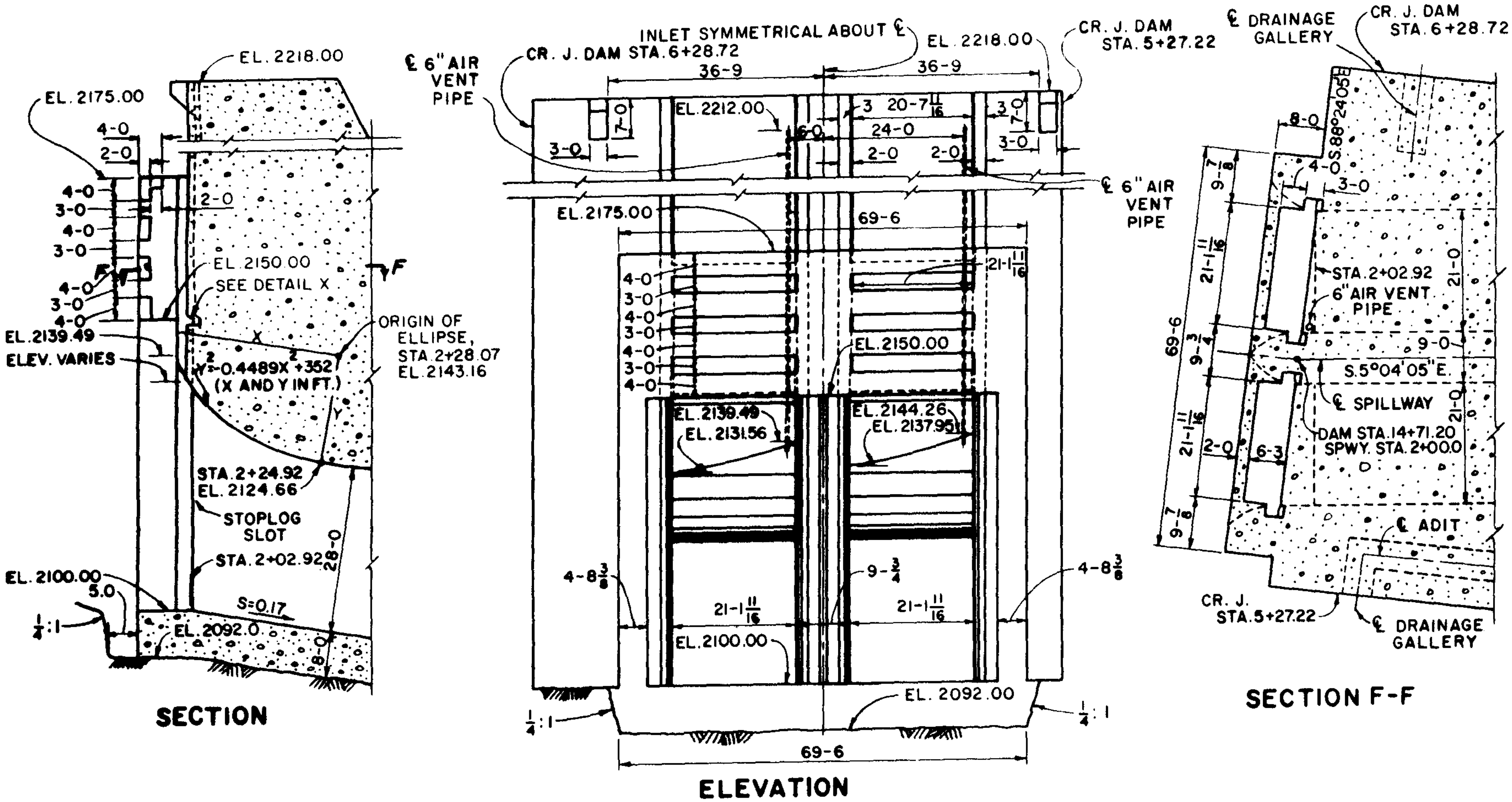


Figure 5. - Final vortex suppressor design.

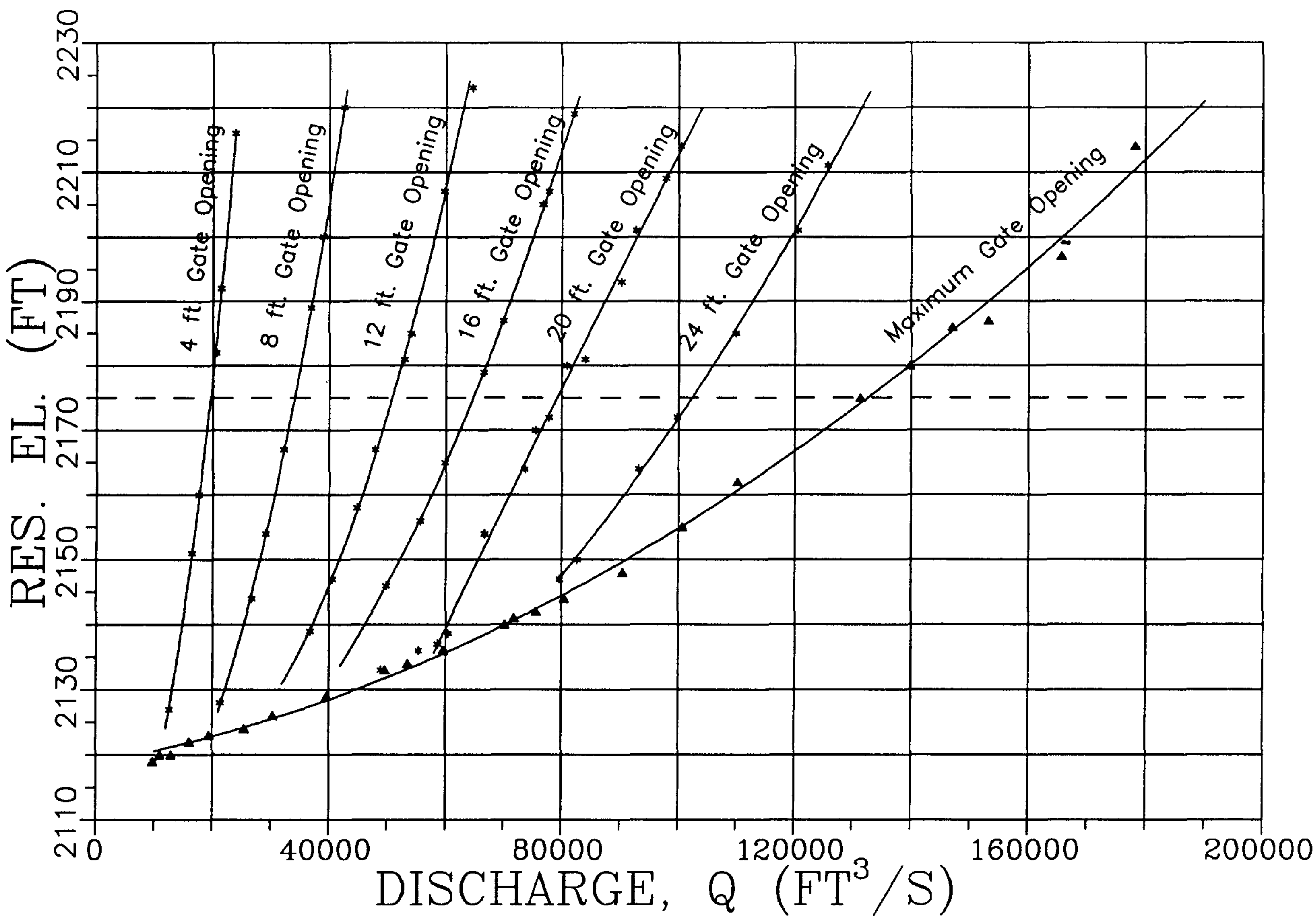


Figure 6. - Discharge rating curve for simultaneous operation of both spillways.

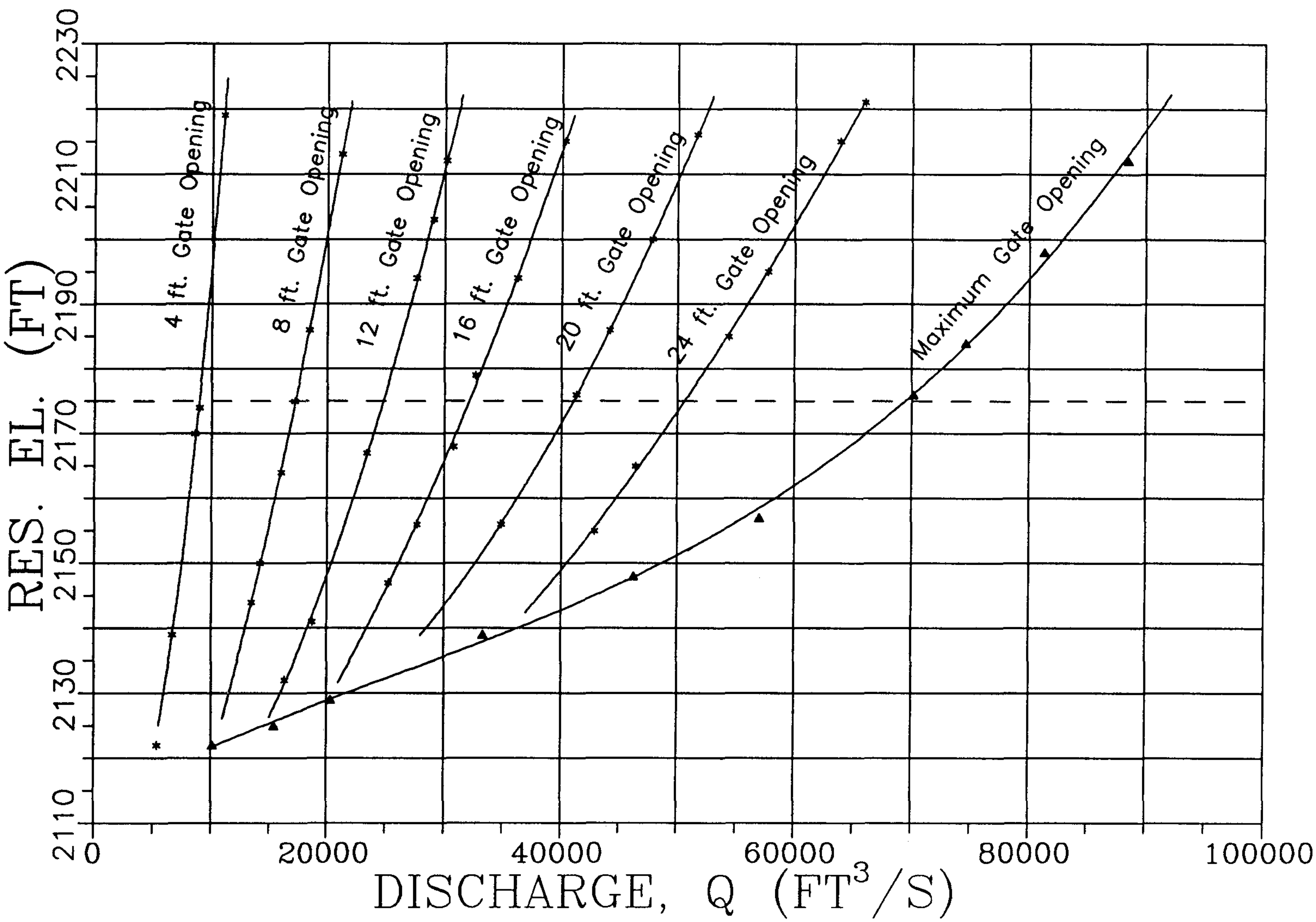


Figure 7. - Discharge rating curve for right abutment spillway operation.

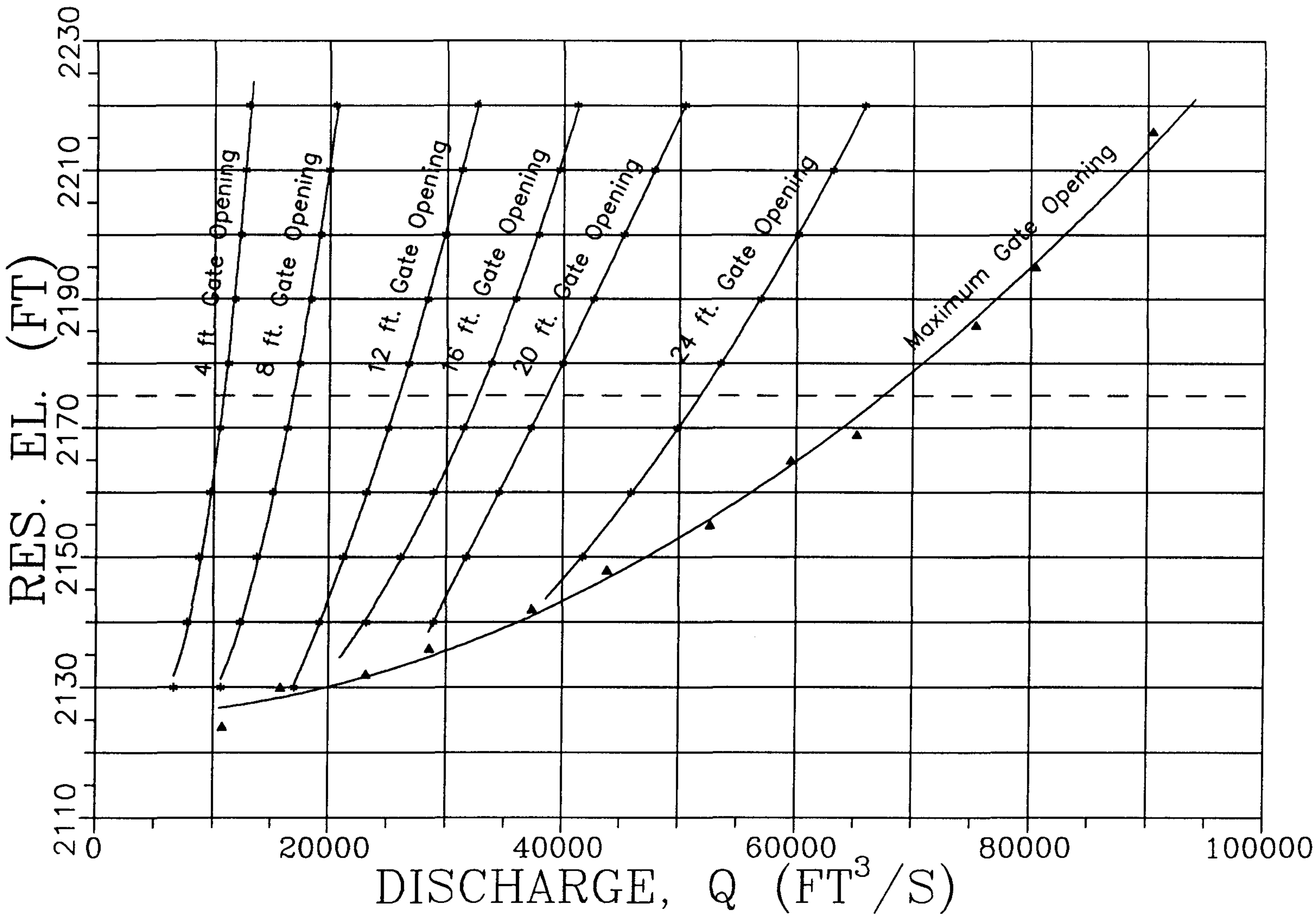


Figure 8. - Discharge rating curve for left abutment spillway operation.

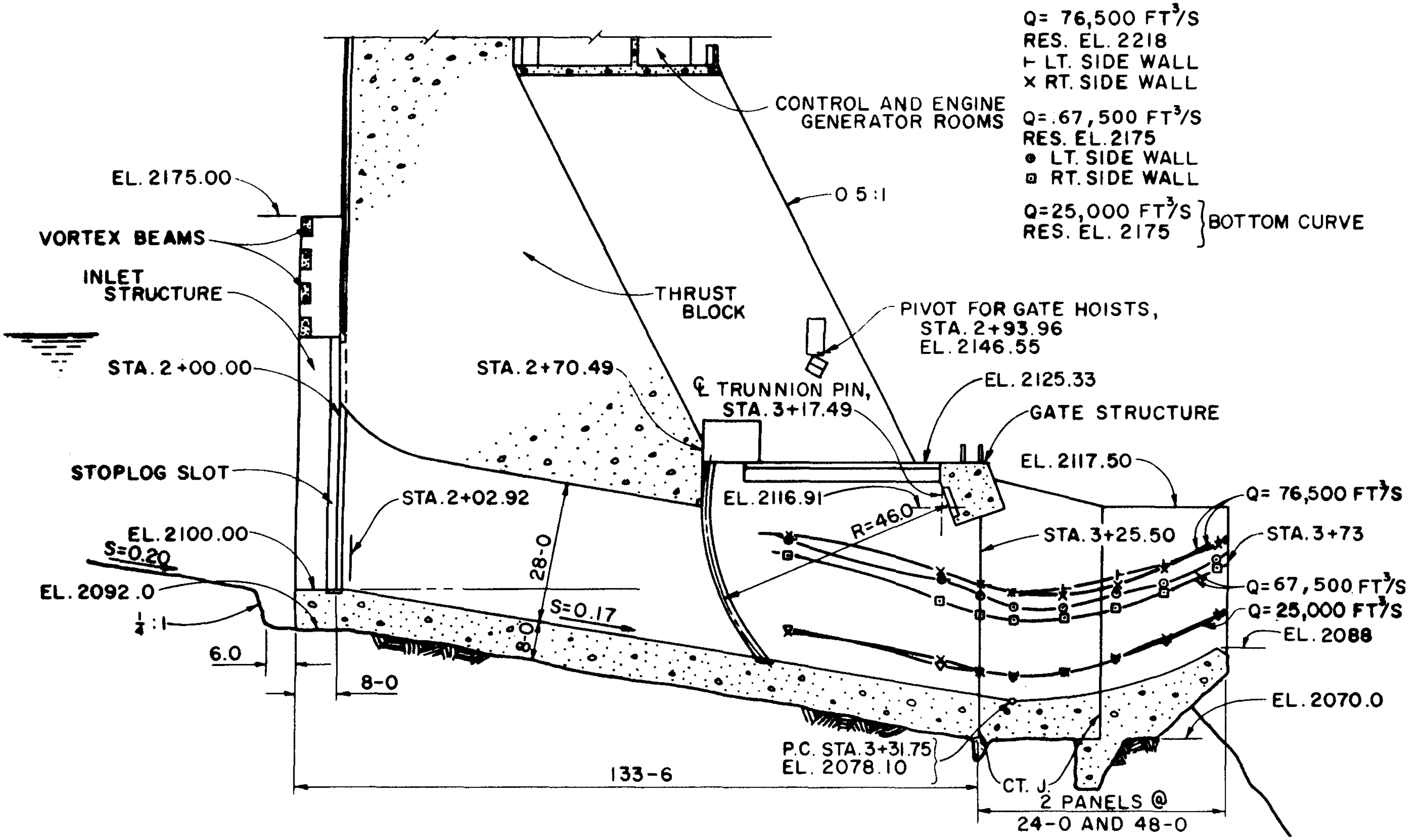
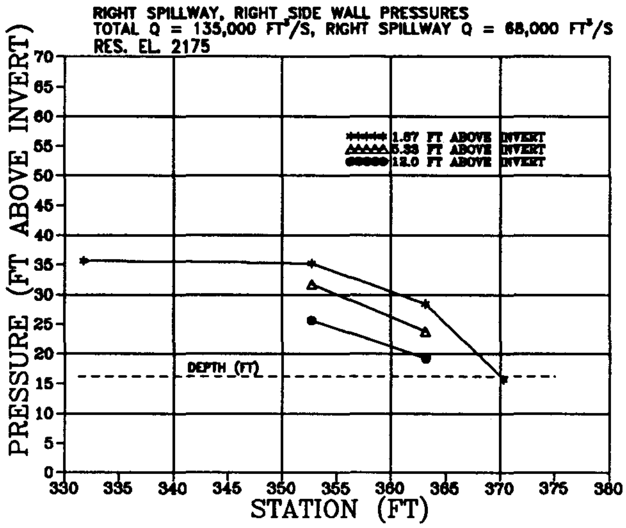
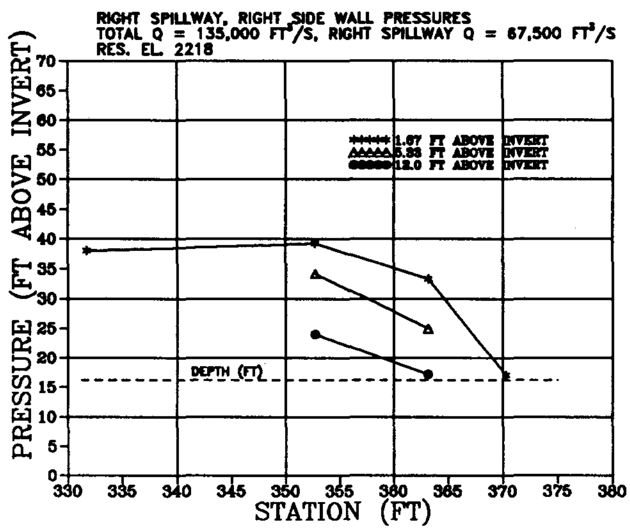


Figure 9. - Chute water surface profiles for the right spillway.





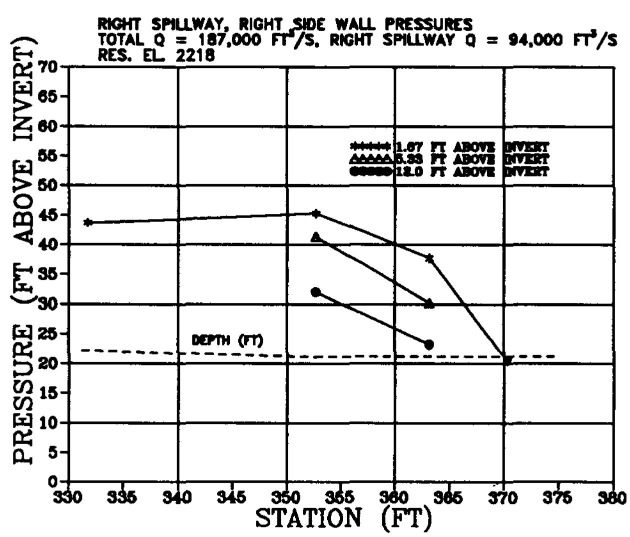


Figure 10. - Side wall pressures through the flip bucket for the right spillway.

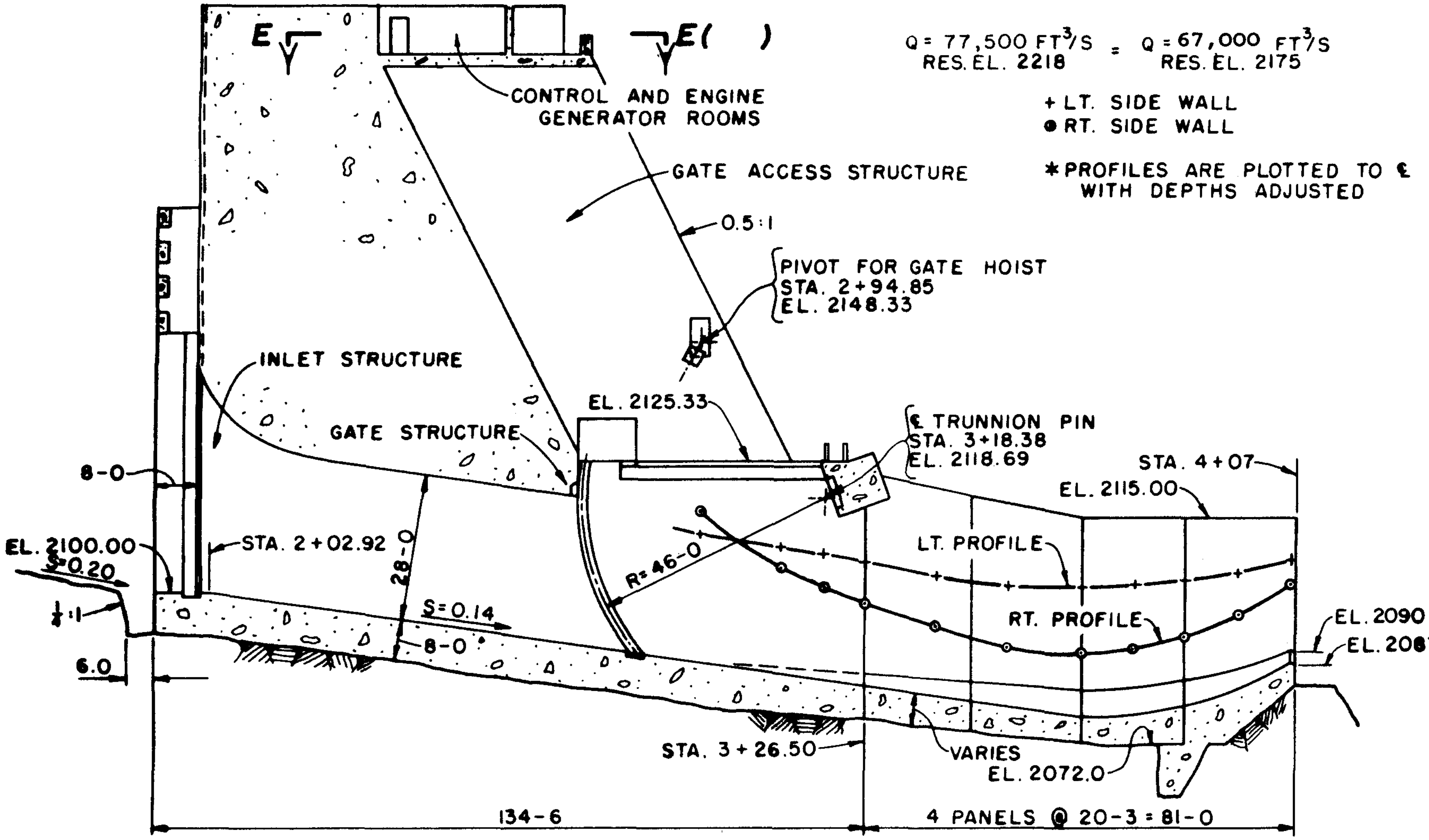
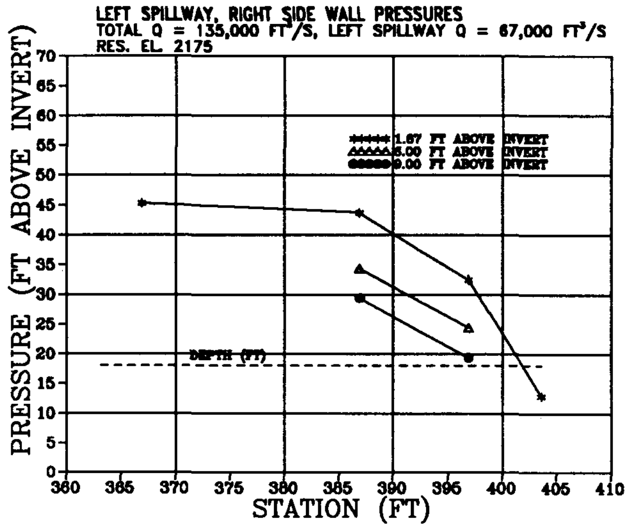
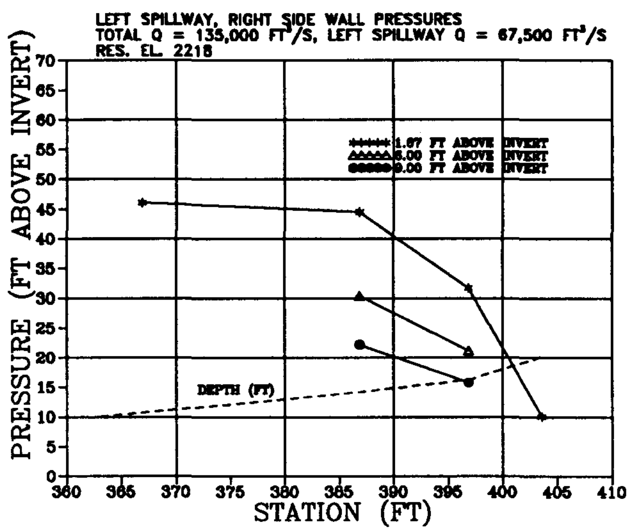


Figure 11. - Chute water surface profiles for the left spillway.





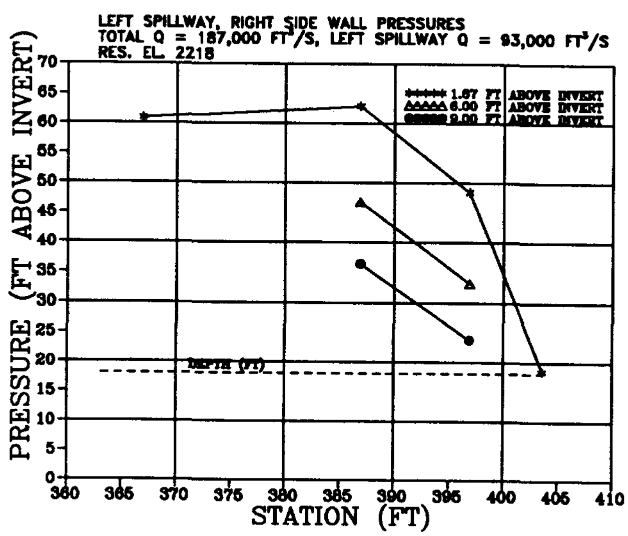
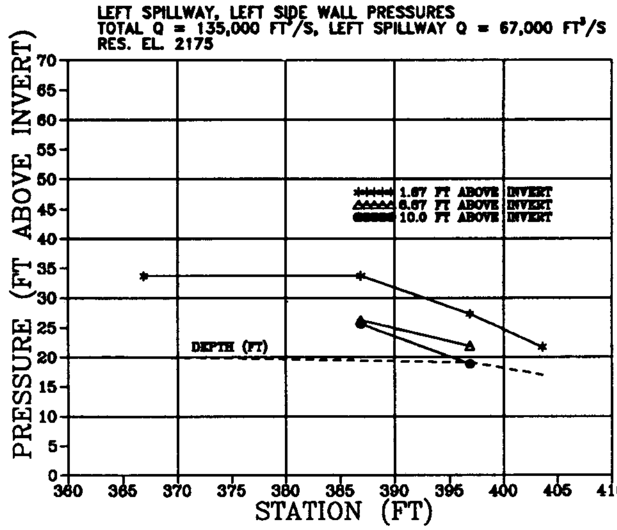
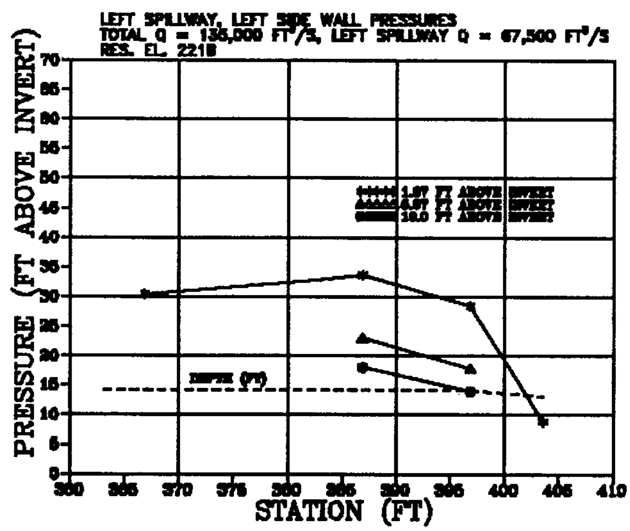


Figure 12. - Side wall pressures through the flip bucket for the left spillway.





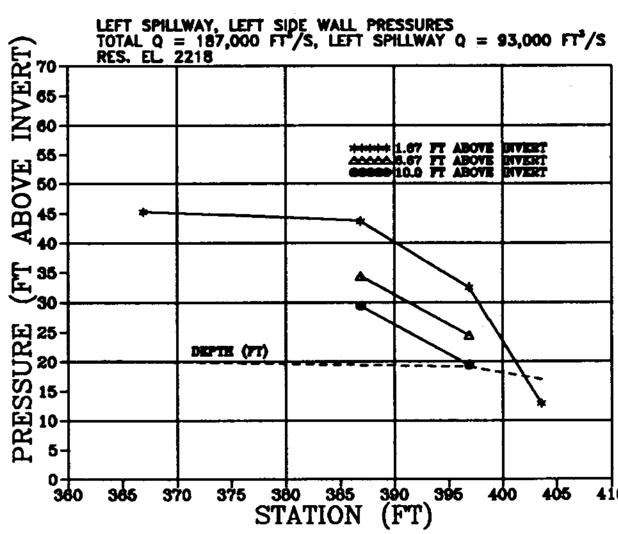


Figure 12. - Side wall pressures through the flip bucket for the left spillway. - Continued

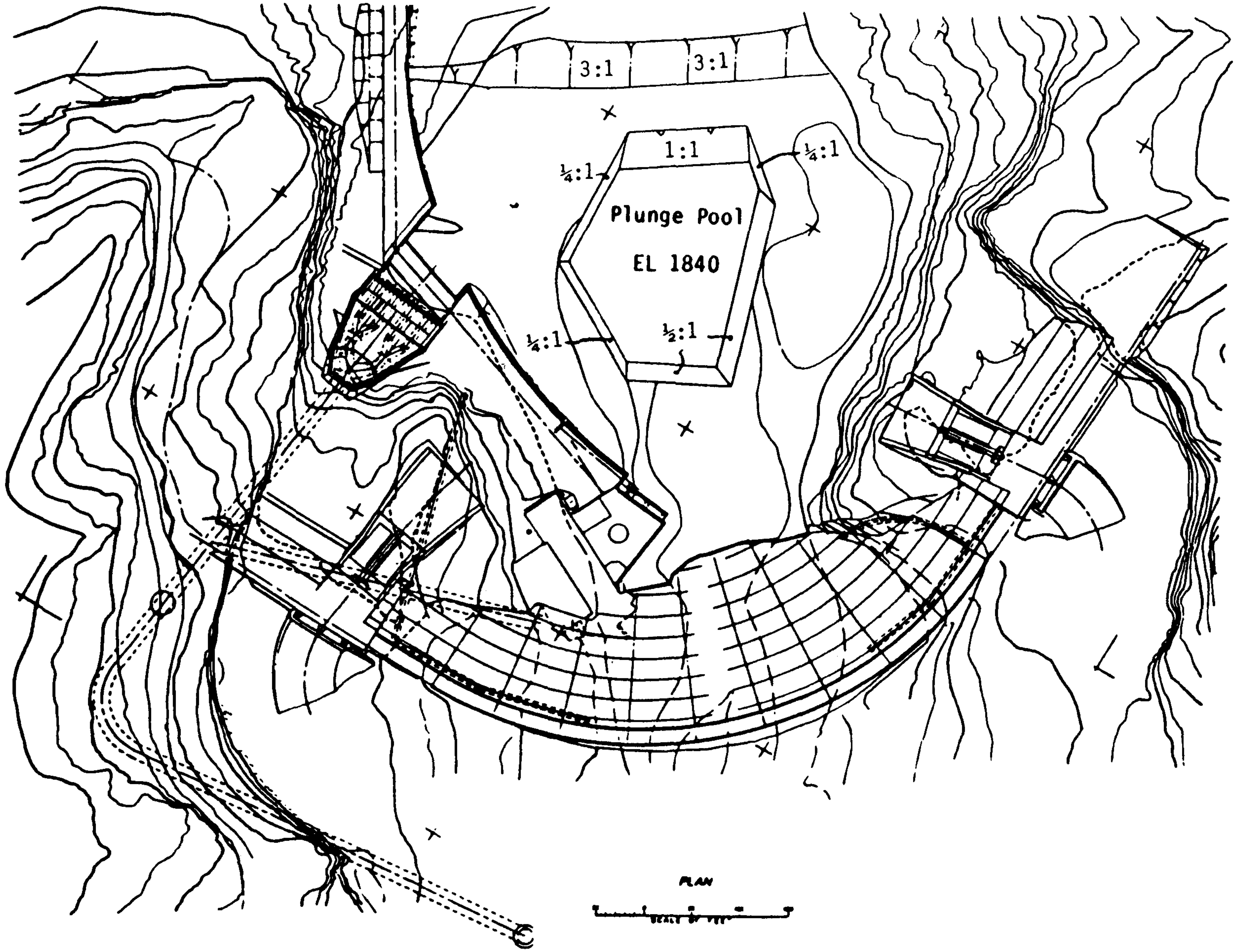


Figure 13. - General plan of the plunge pool and spillway designs for the single-curvature arch dam raise.

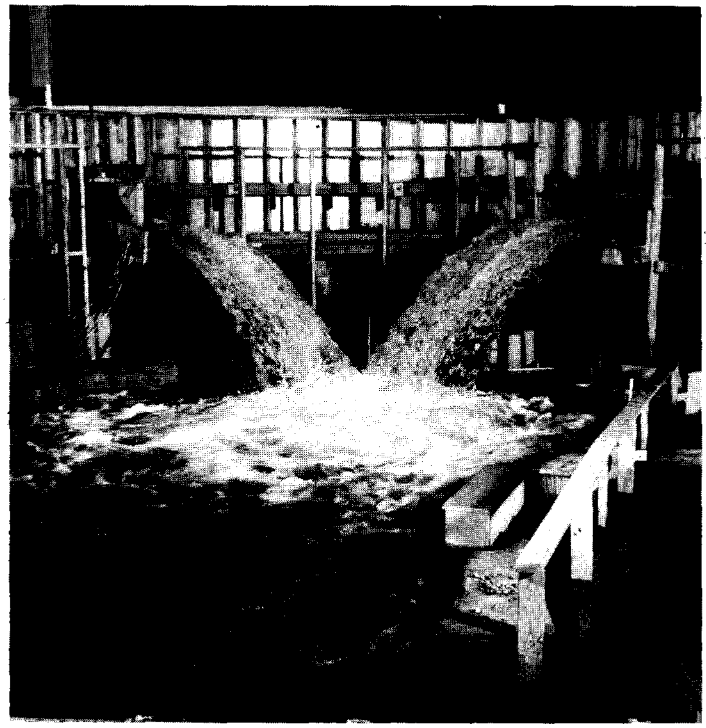


Figure 14. - Final plunge pool design dissipating the energy of the design discharge, 135,000 ft^3/s at reservoir elevation 2175.

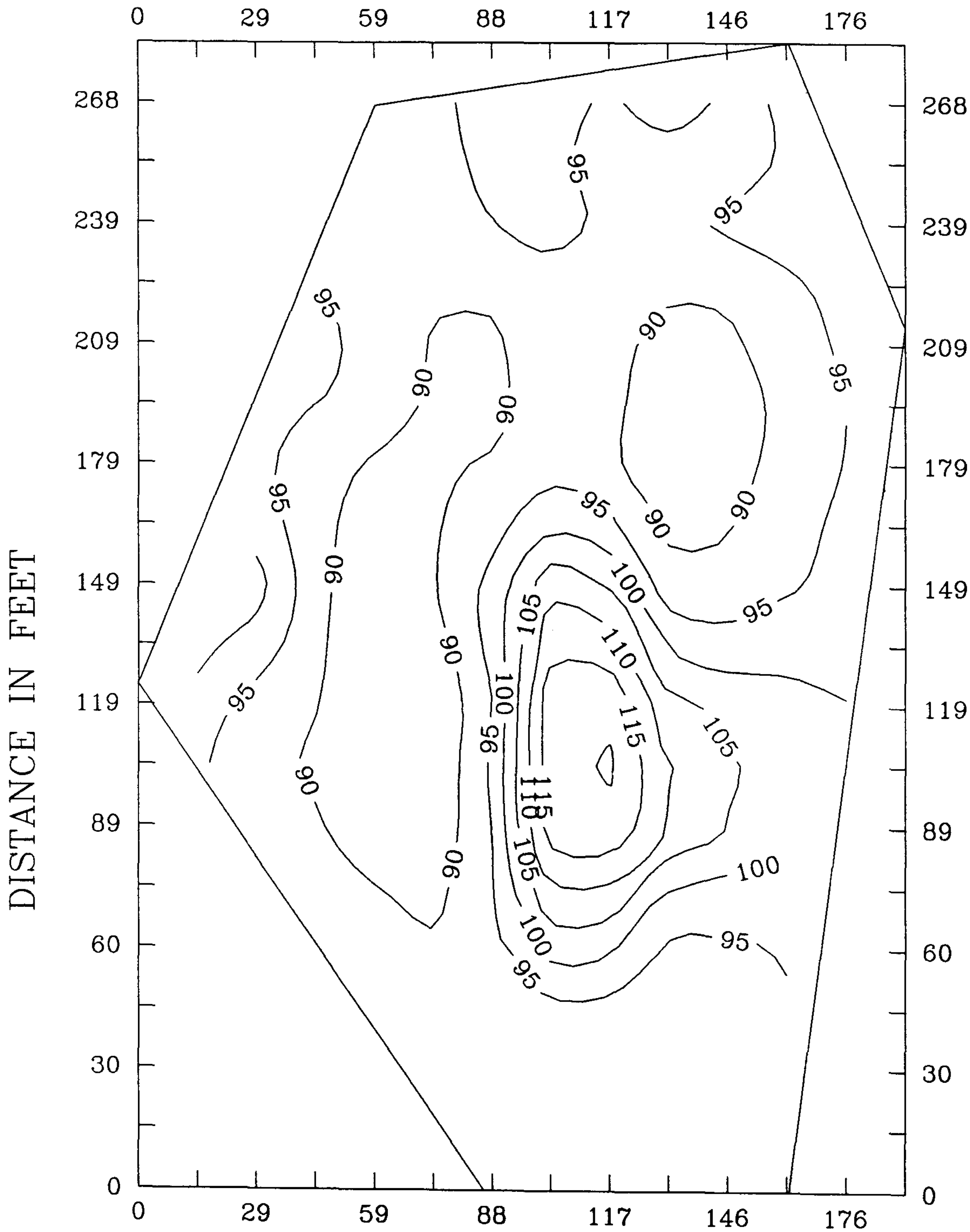


Figure 15. - Map of static pressures above the plunge pool floor at elevation 1840 for the design discharge, tailwater elevation = 1937 feet.

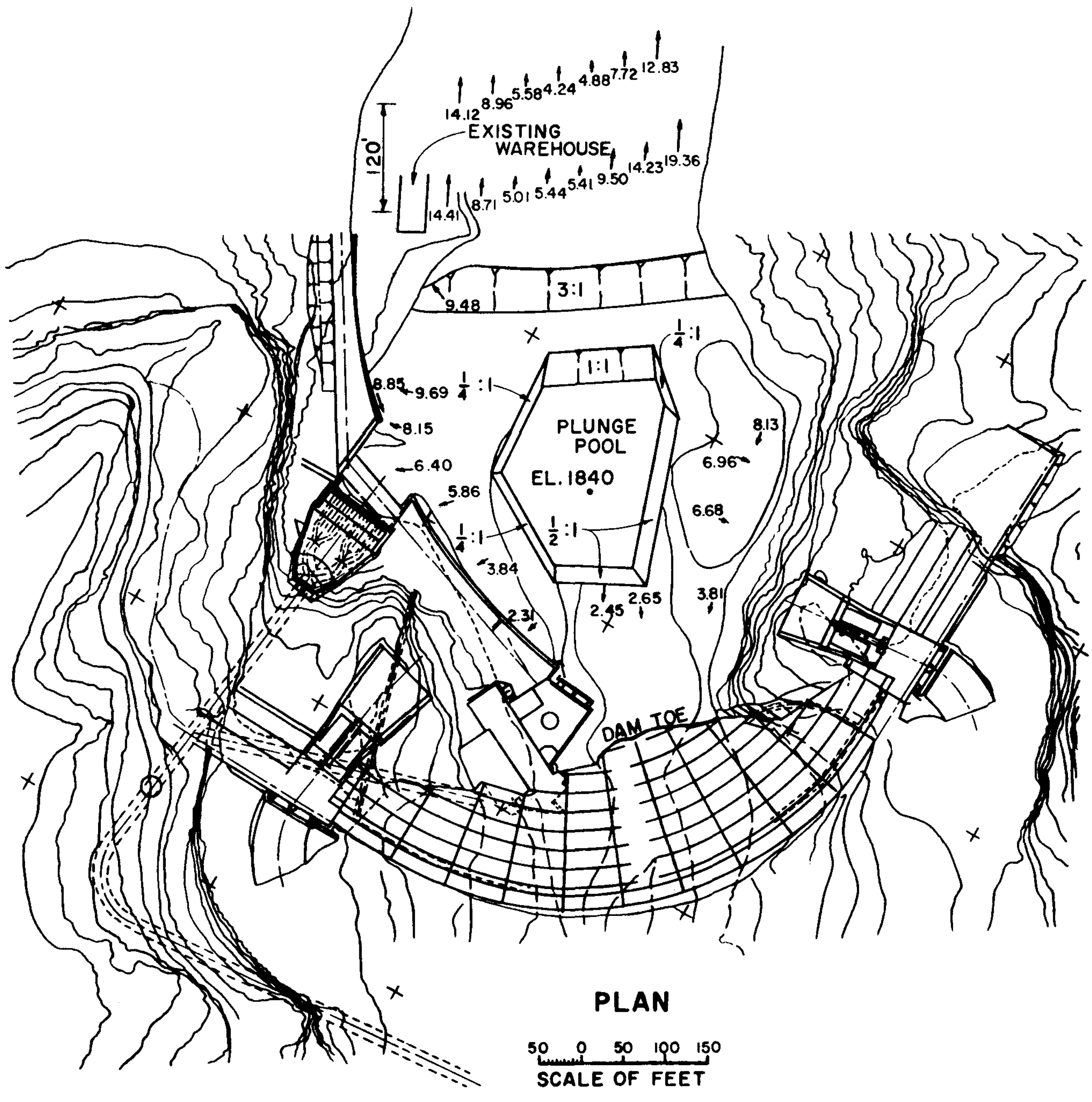


Figure 16. - Velocities, ft/s, around the SCAD plunge pool design and in the river channel for the design discharge.

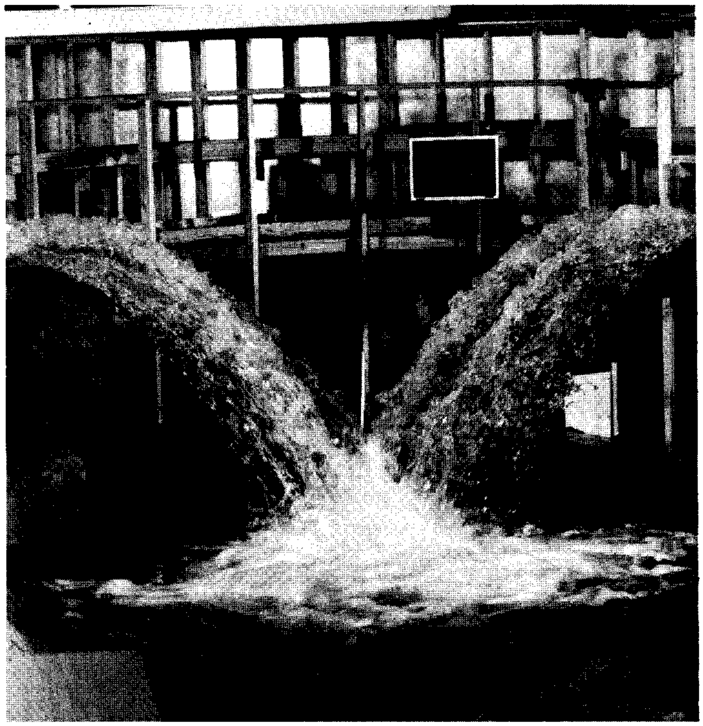


Figure 17. - Initial DCAD spillways and plunge pool with Q = 54,380 ft^3/s at elevation 2175.

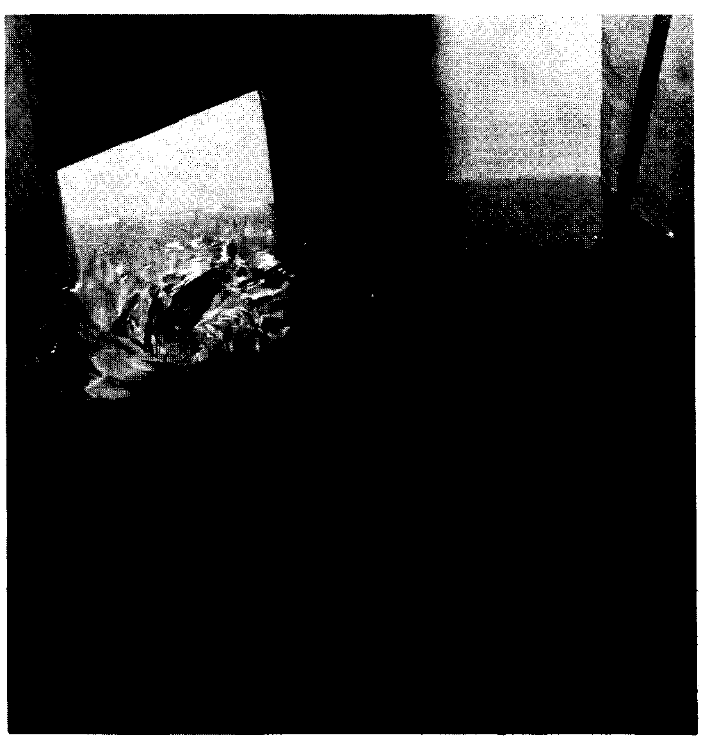


Figure 18. - Strong vortex over the left bay of the right spillway, initial spillway design, Q = 135,000 ft^3/s, elevation 2175.

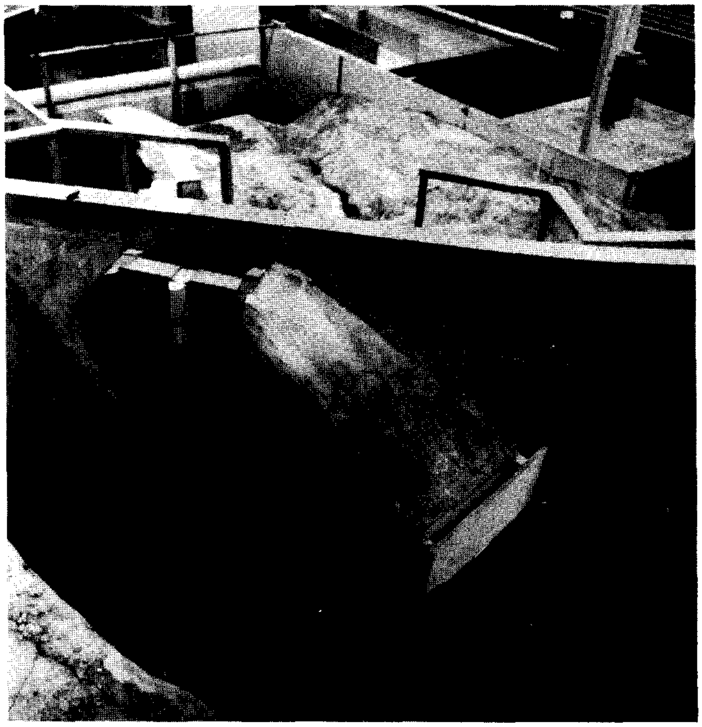


Figure 19. - Tapered guidewall tested in the left spillway approach channel to reduce vortex formation.

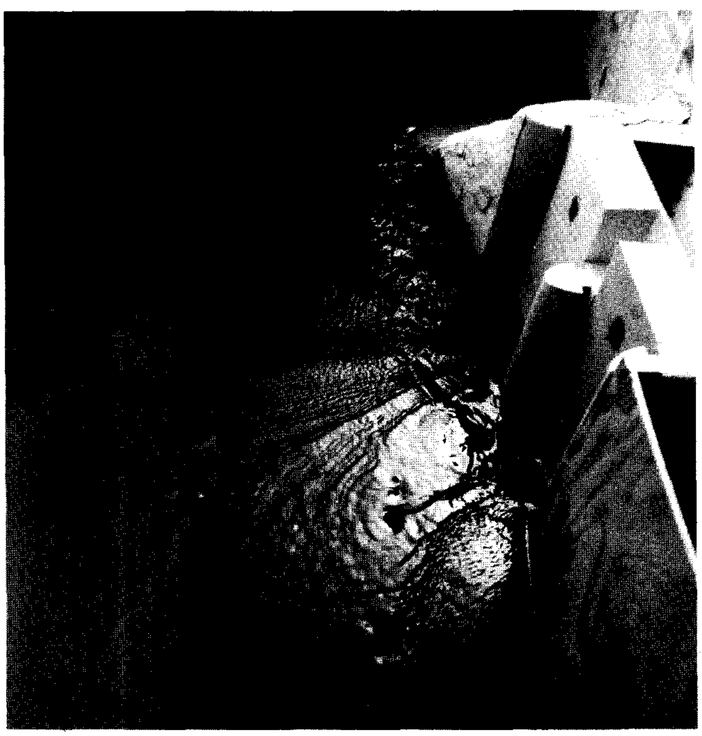


Figure 20. - Vertical thrust block face and tapered guidewall in the right spillway channel tested at reservoir elevation 2175 to reduce vortex formation.

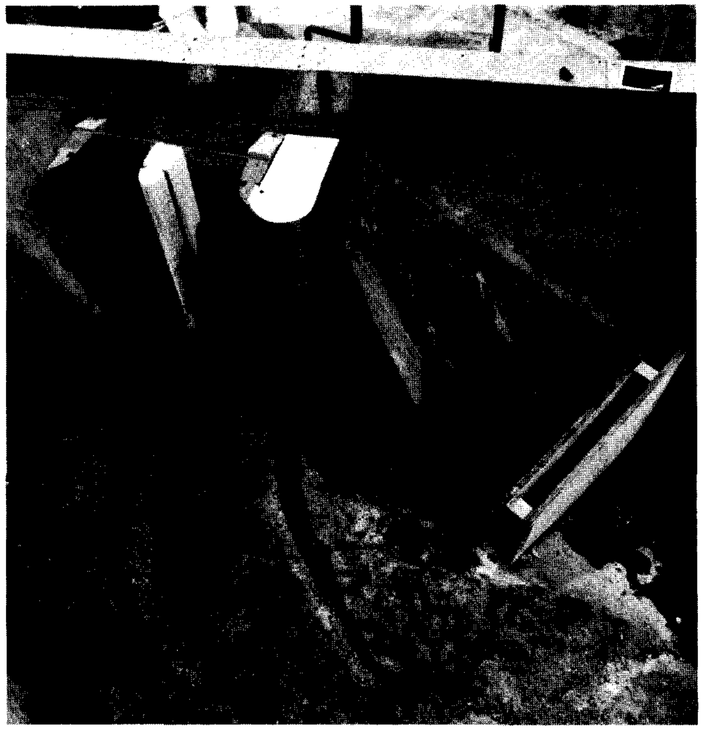


Figure 21. - Nose of the left spillway inside pier extended to the end of the thrust block tested to reduce vortices.

Figure 22. - Initial testing with vortex suppressor racks above the right spillway entrance at reservoir elevation 2175.

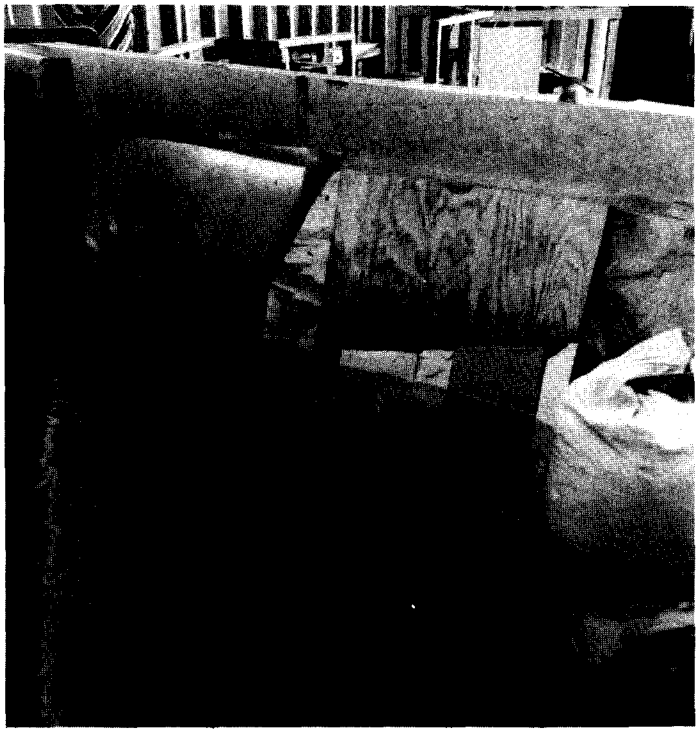


Figure 23. - Modified right spillway entrance flush with the 1:3 thrust block face and "dead space" in approach channel filled (reservoir elevation 2175).

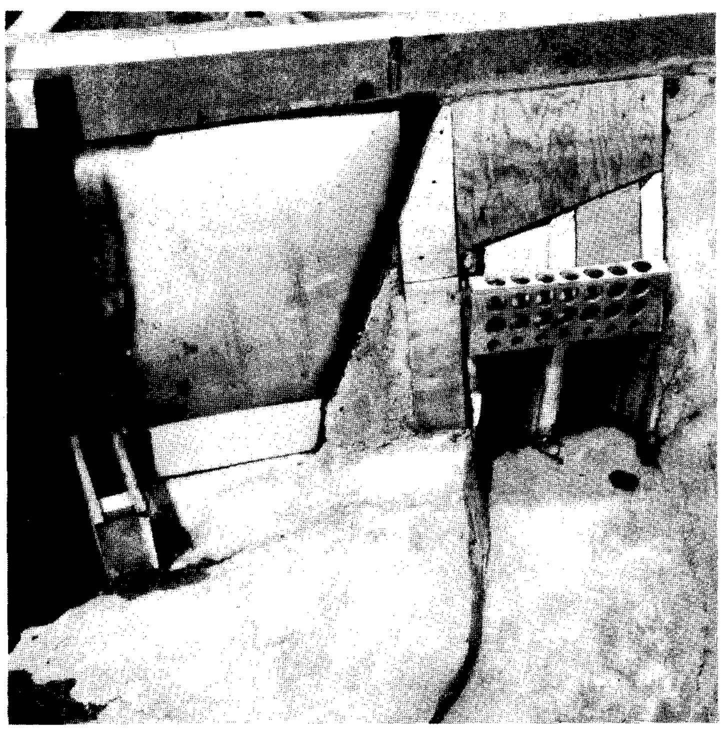


Figure 24. - Vortex suppressor structure mounted from elevation 2151 to 2175 above the right spillway entrance.

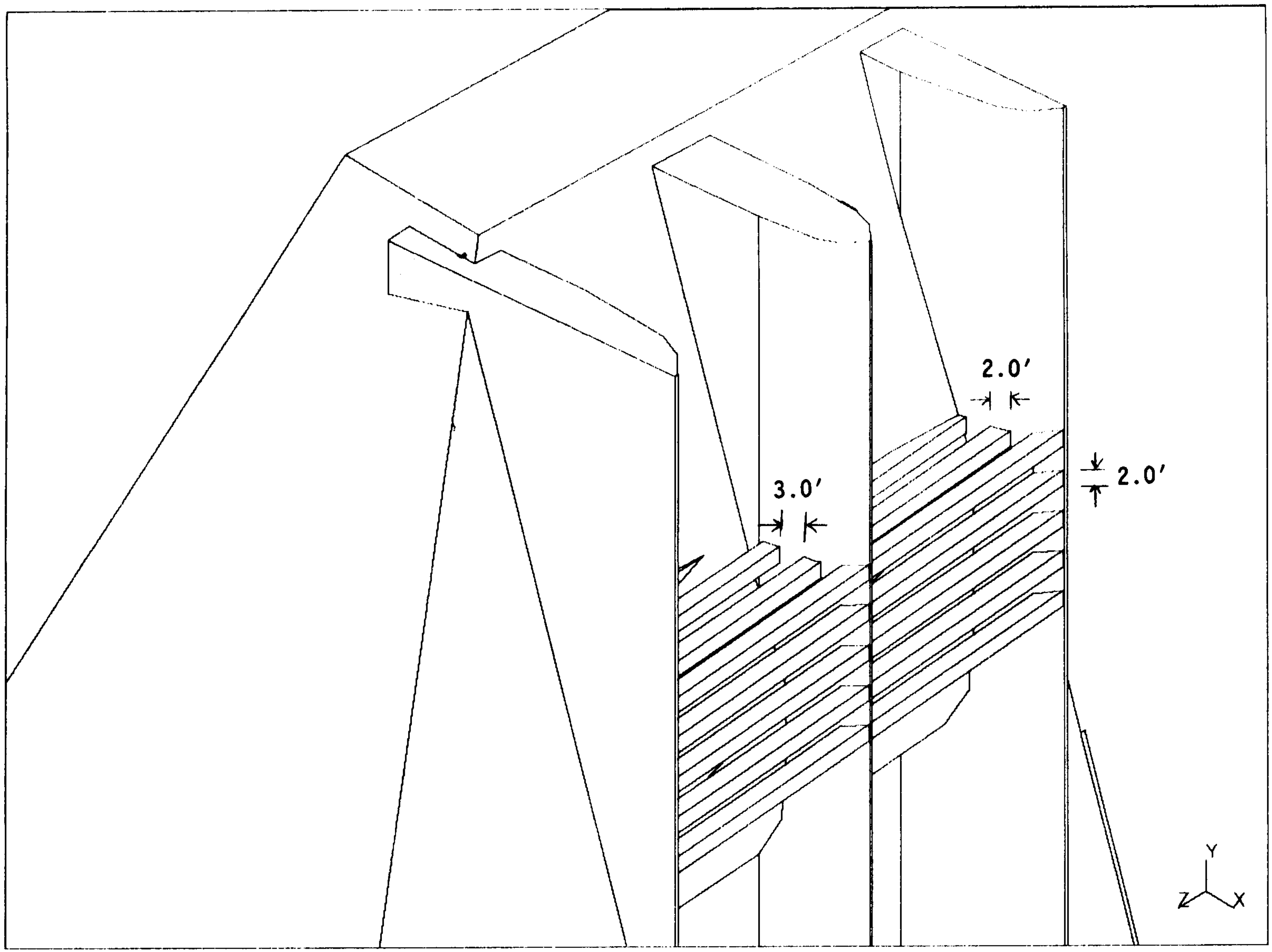


Figure 25. - Recommended vortex suppressor for the DCAD option.

Figure 26. - Weak surface vortices, but no air core vortices over recommended vortex suppressor, Q = 135,000 ft^3/s, elevation 2175, for the DCAD option.

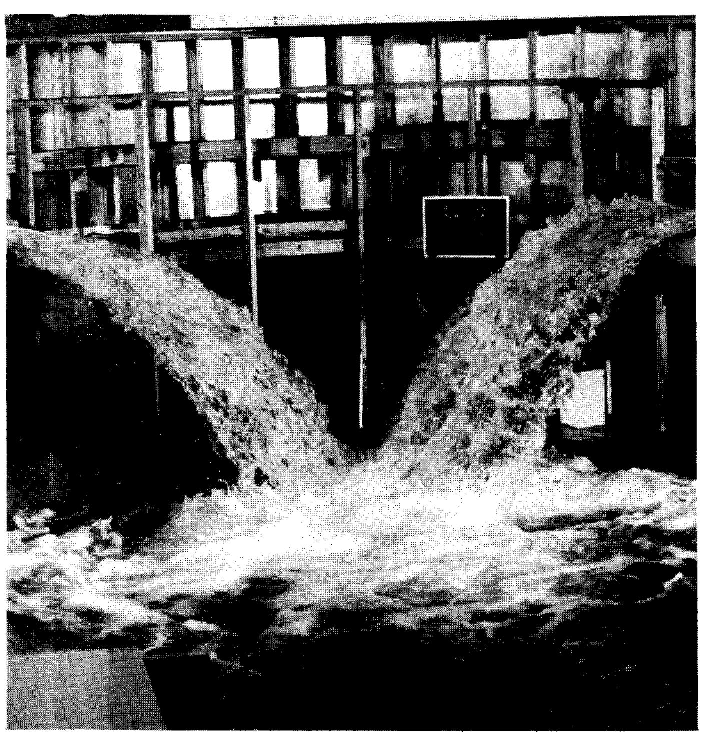


Figure 27. - Recommended spillways for the DCAD option discharging into the initial plunge pool design with Q = 135,000 ft^3/s at elevation 2175.

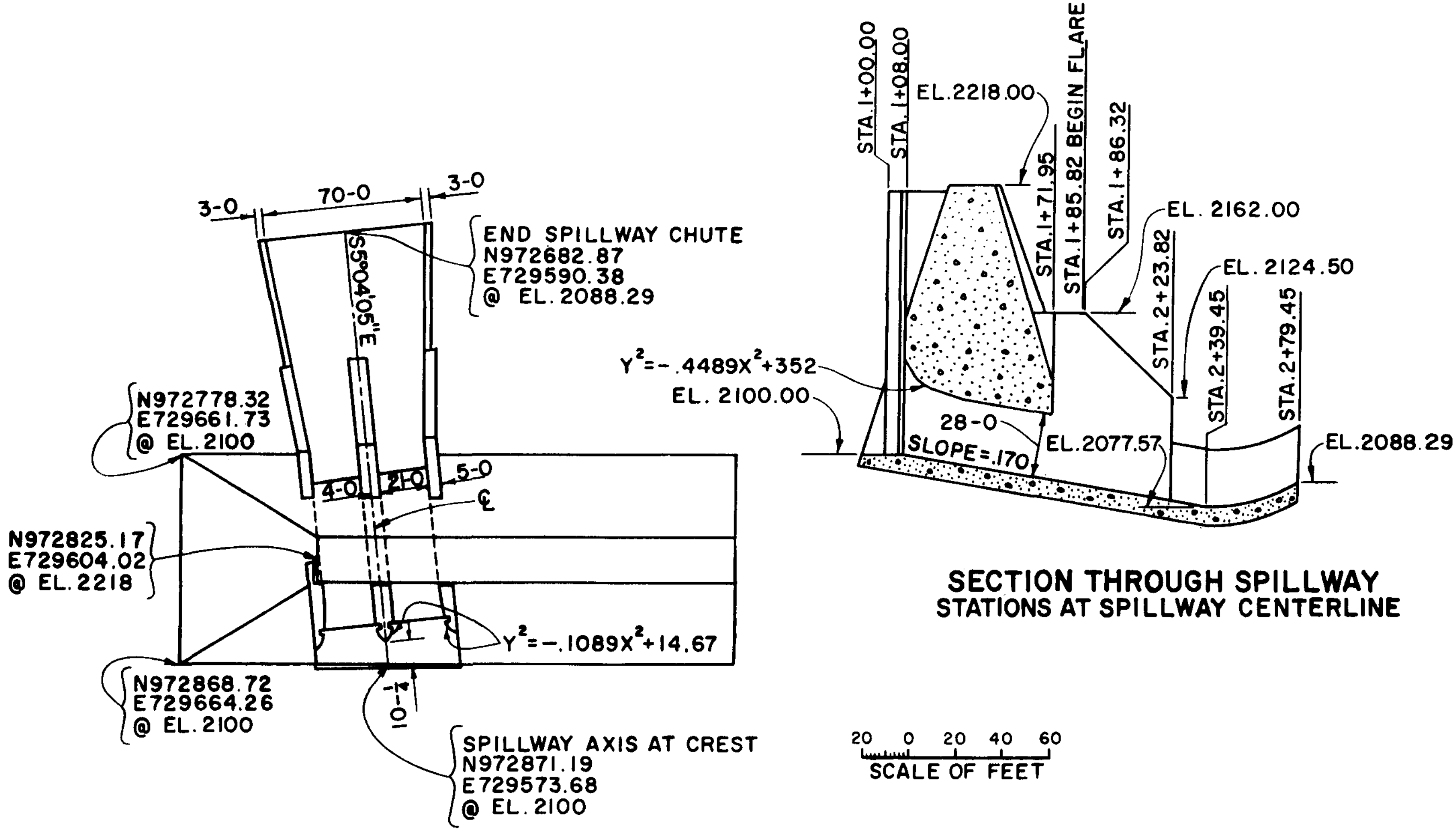


Figure 28. - Recommended right spillway geometry for the DCAD option.

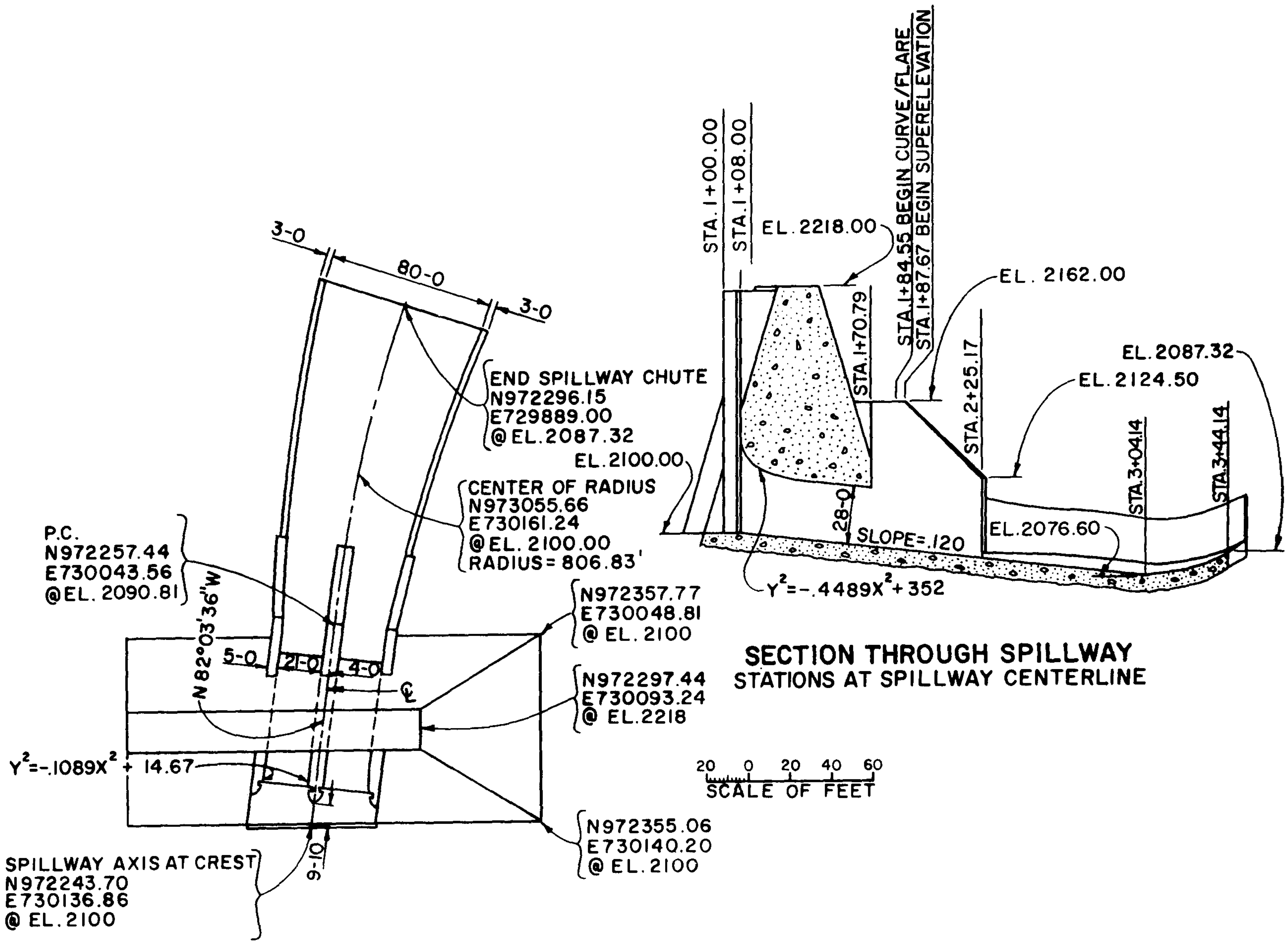


Figure 29. - Recommended left spillway geometry for the DCAD option.

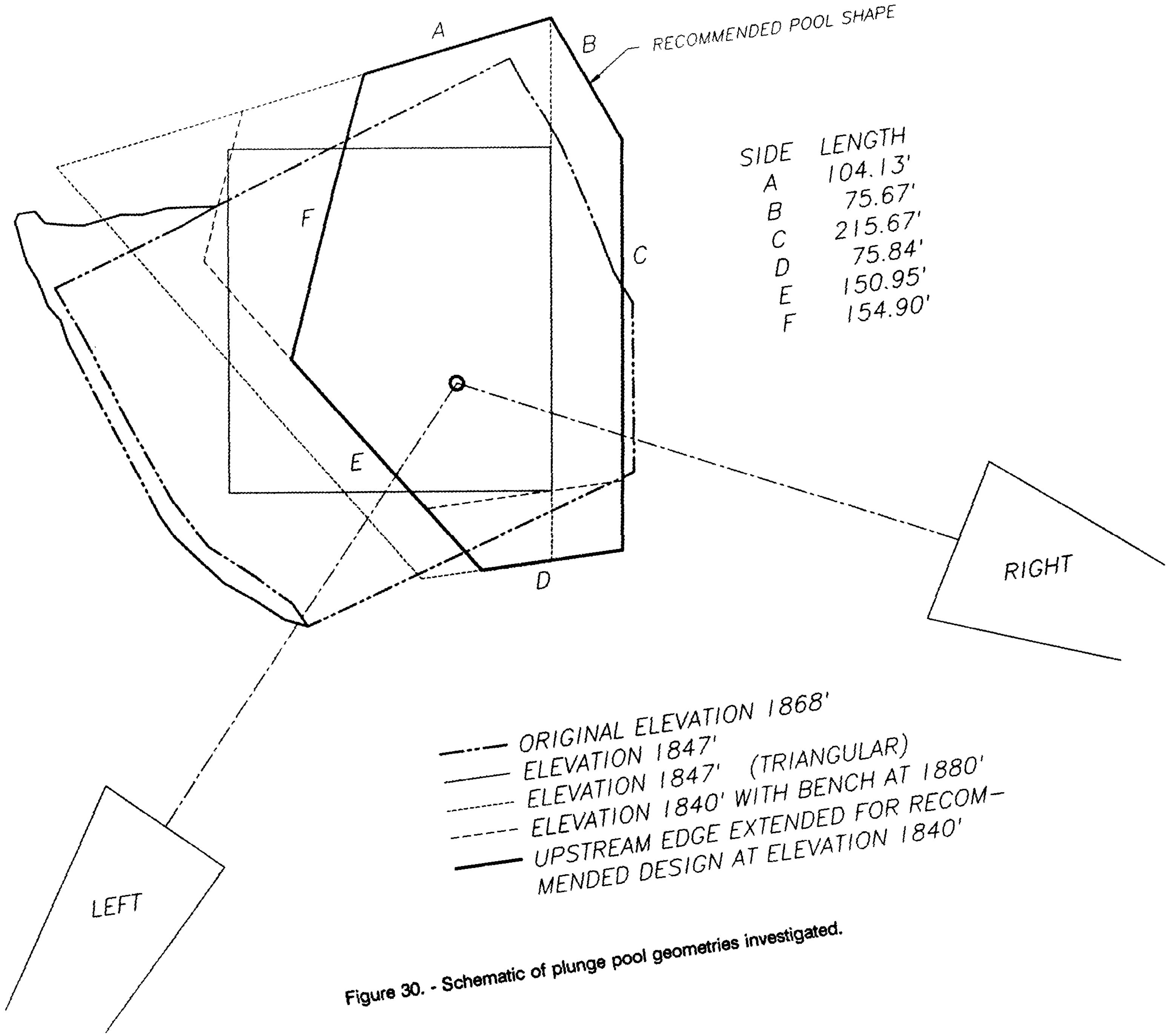


Figure 30. - Schematic of plunge pool geometries investigated.

Figure 31. - First plunge pool modification, floor elevation 1846.33.

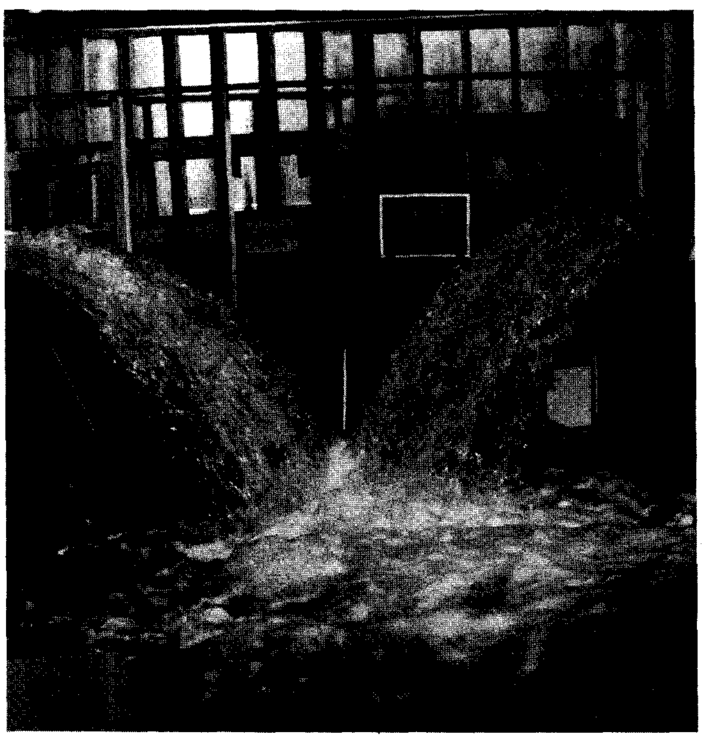


Figure 32. - First plunge pool modification with floor elevation 1846.33, Q = 135,000 ft^3/s, elevation 2175.

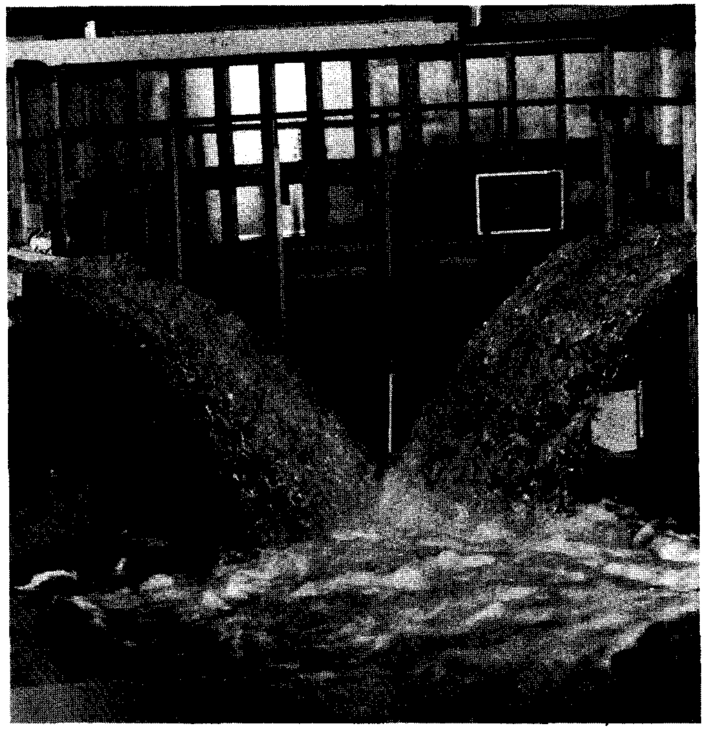


Figure 33. - Triangular plunge pool modification, Q = 135,000 ft^3/s, elevation 2175.

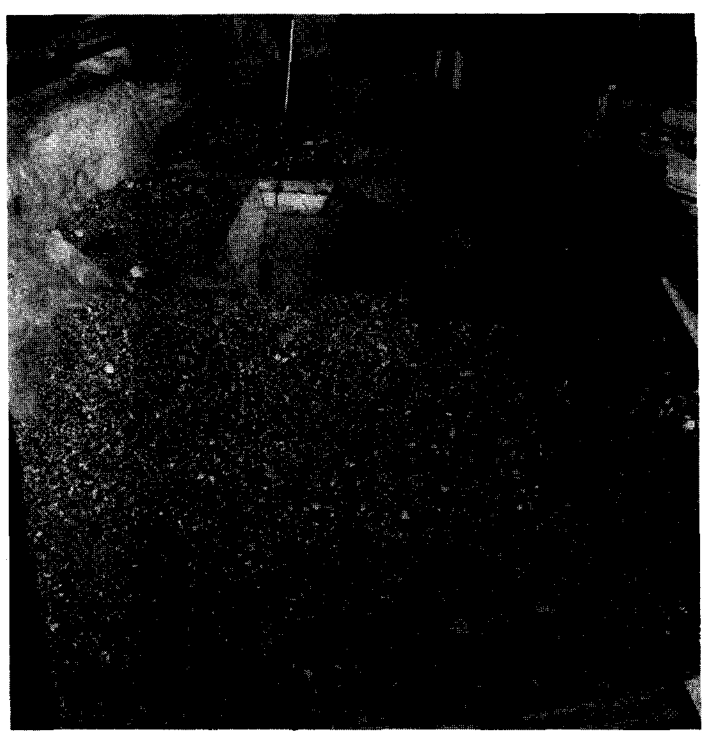


Figure 34. - Recommended plunge pool geometry for the DCAD option.

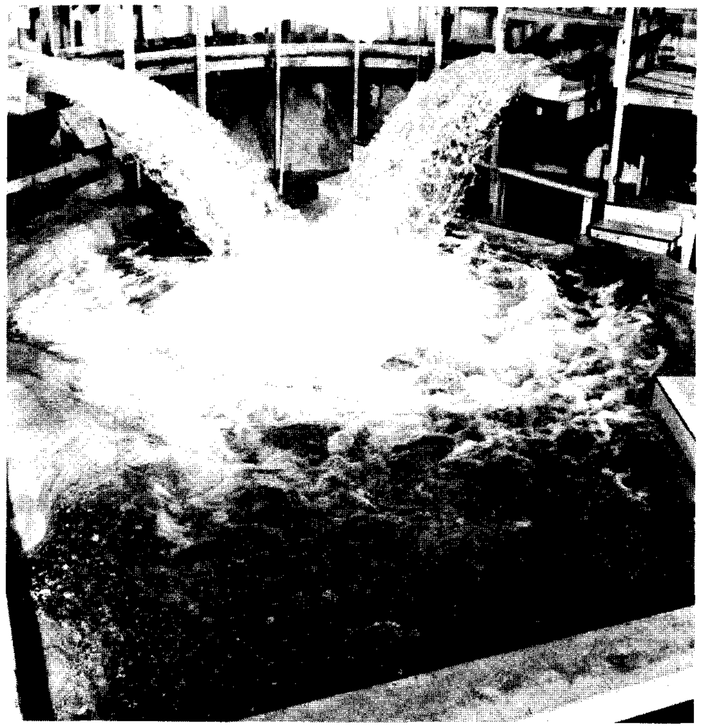


Figure 35. - Dissipation of the design discharge in the recommended pool for the DCAD option.

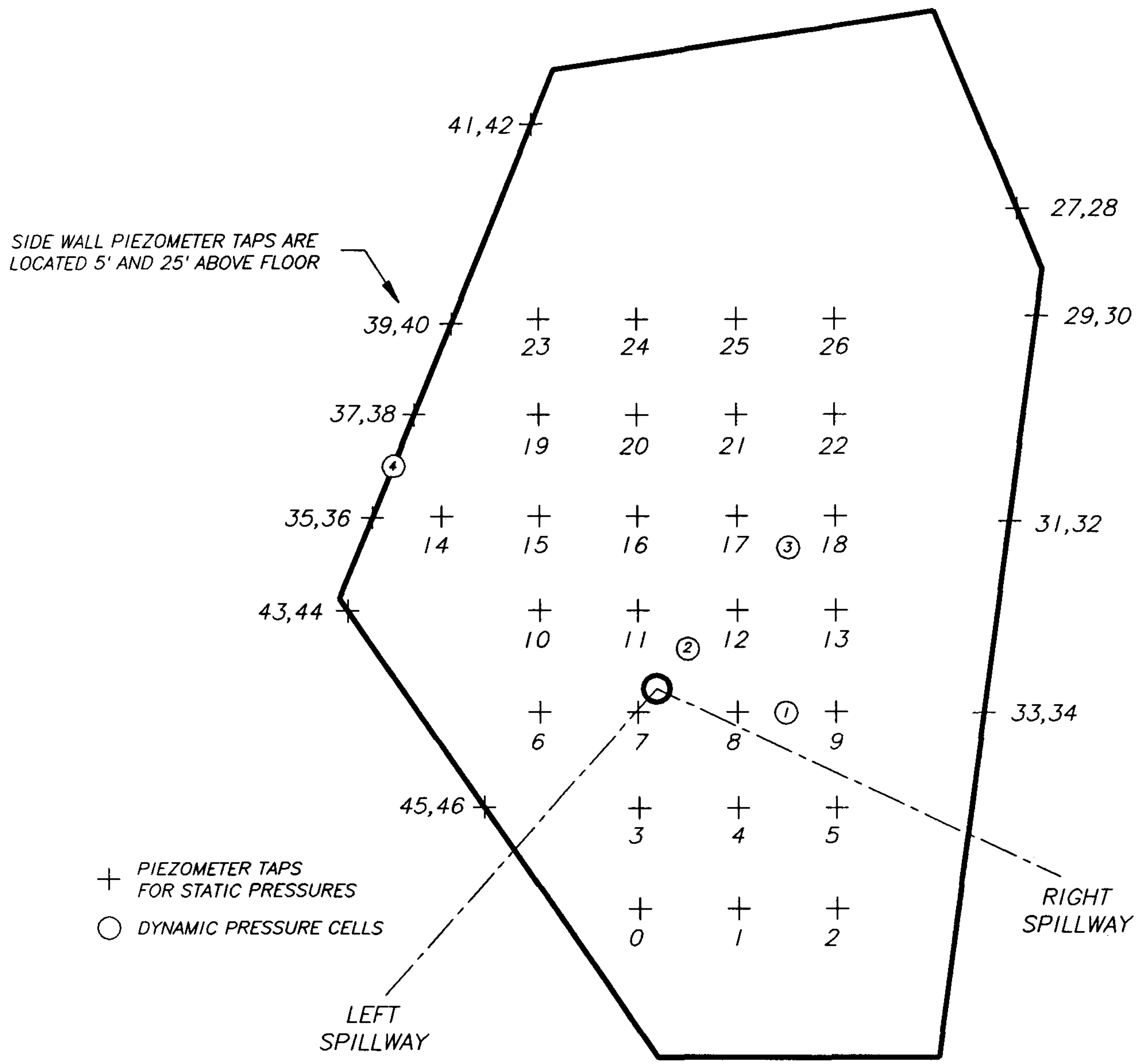
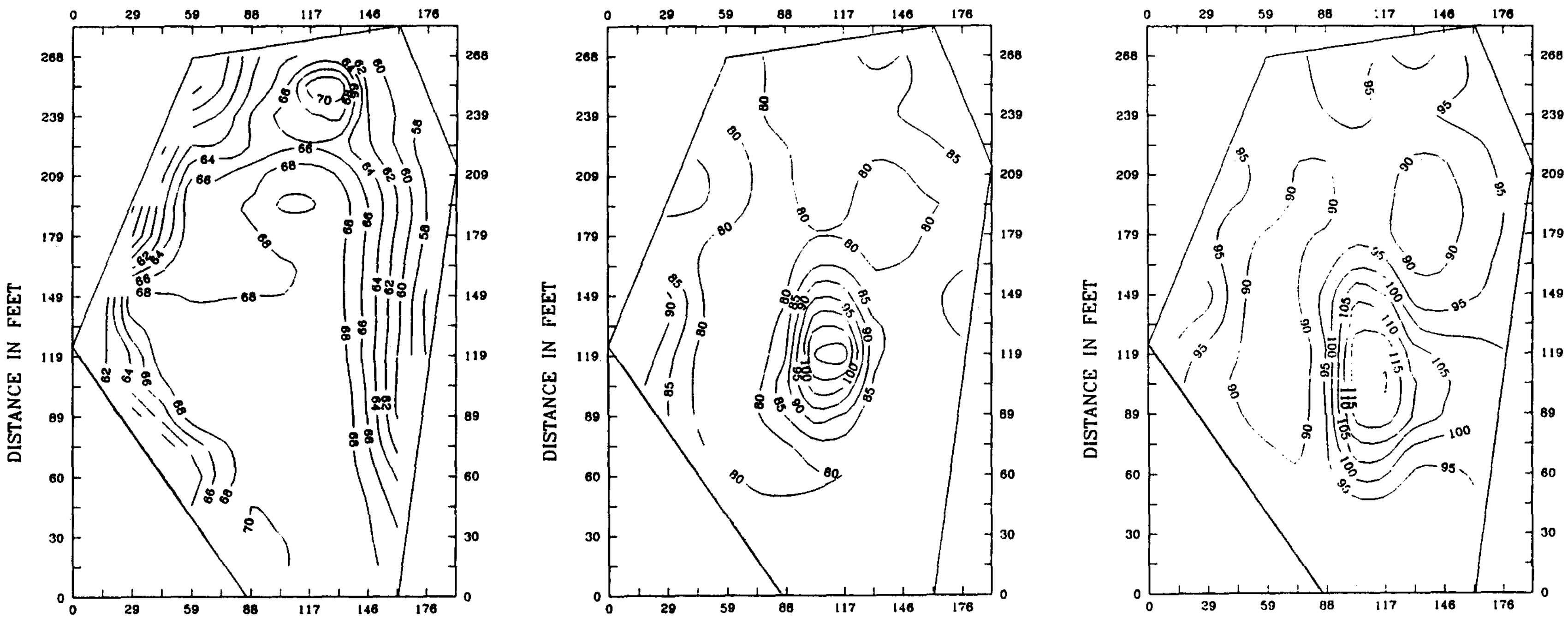


Figure 36. - Map of piezometer tap and dynamic pressure locations in the plunge pool.

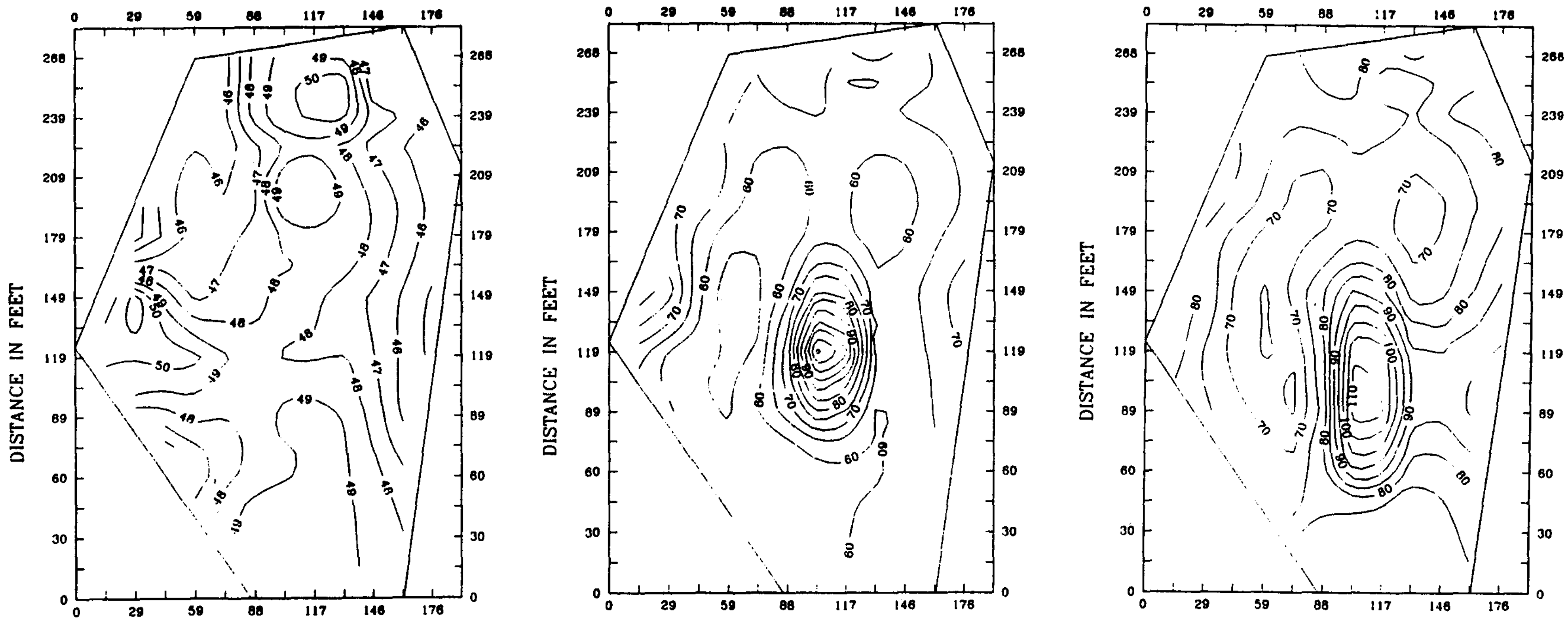


Q = 22,000 ft^3/s
TW = 1912

Q = 90,000 ft^3/s
TW = 1926

Q = 135,000 ft^3/s
TW = 1937

Figure 37. - Equal static pressure contours for pool floor at elevation 1841.



Q = 22,000 ft^3/s
TW = 1914

Q = 90,000 ft^3/s
TW = 1925

Q = 135,000 ft^3/s
TW = 1938

Figure 38. - Equal static pressure contours for pool floor at elevation 1861.

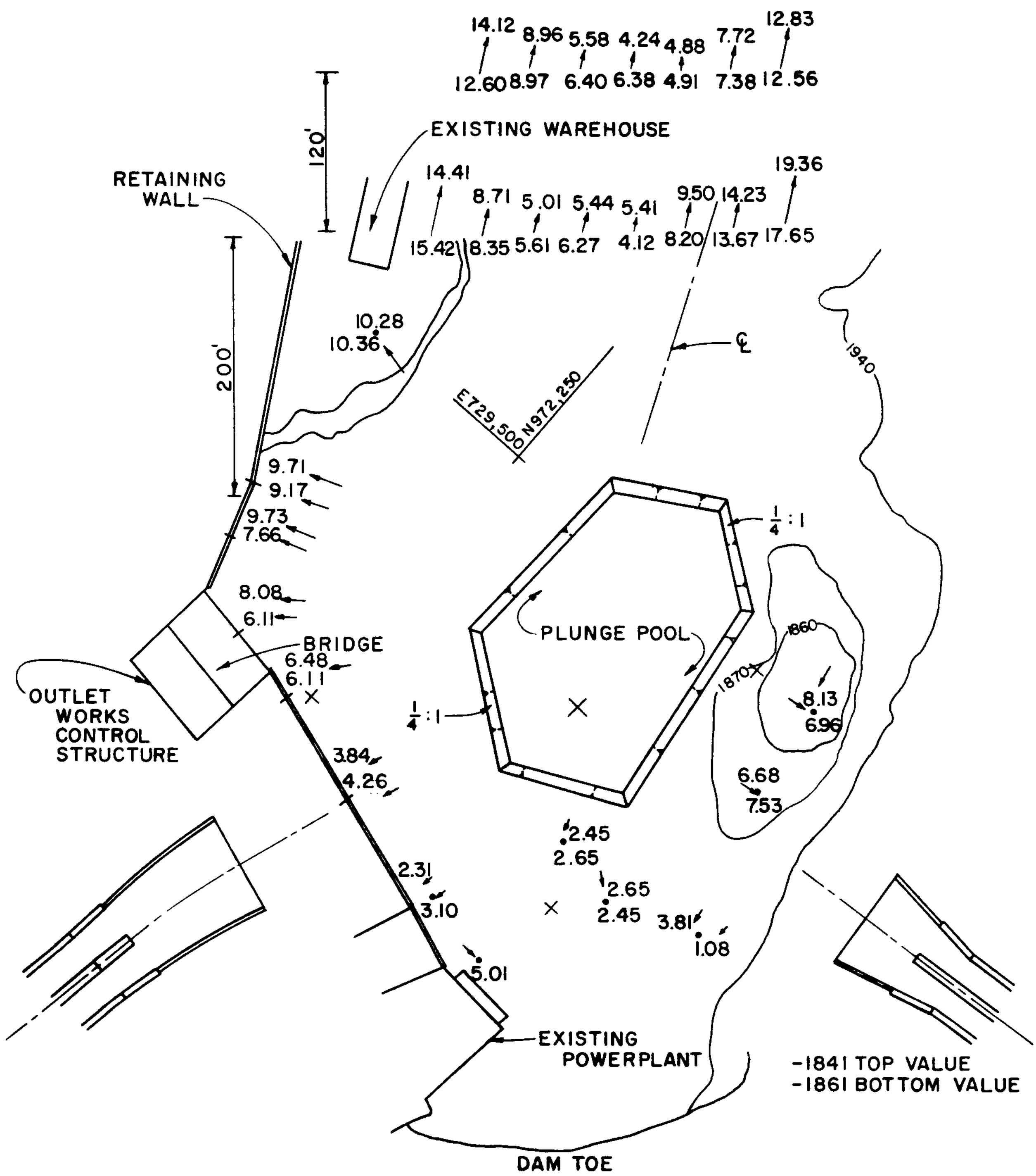


Figure 39. - Velocities adjacent to and downstream of the DCAD plunge pool for floor elevations 1841 and 1861.

Figure 40. - Original alluvium contours with recommended DCAD plunge pool.

Figure 41. - Outlet works operation at Q = 11,517 ft^3/s and reservoir elevation 2175.

Figure 42. - Alluvium patterns after cumulative outlet works and right spillway discharges up to 25,000 ft^3/s at elevation 2175.

Figure 43. - Alluvium patterns after a total of 70,000 ft^3/s at elevation 2175 through both spillways.

Figure 44. - Alluvium patterns after operation of the design discharge.

Figure 45. - Alluvium patterns after operation of the PMF.

Figure 46. - Plunge pool after operating under design discharge with material in the pool at the beginning of the test.

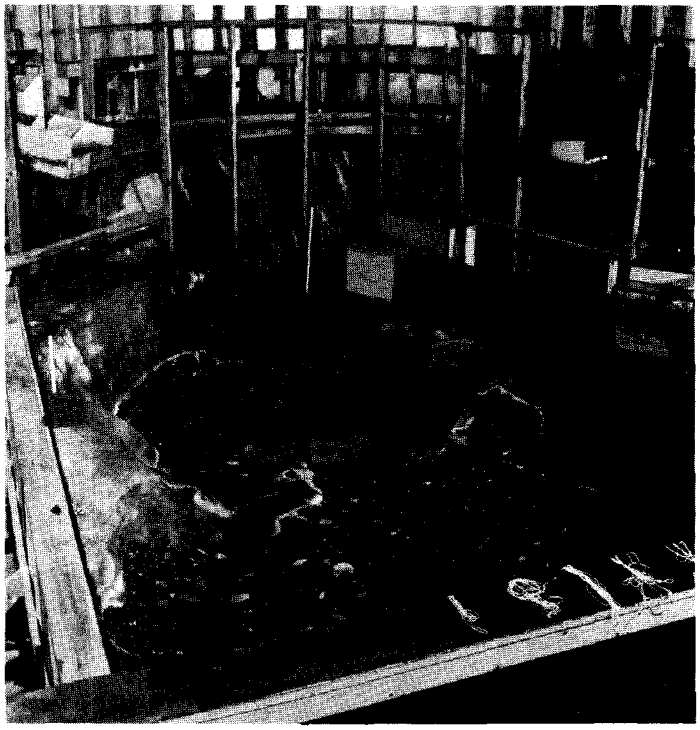


Figure 47. - Erodible concrete forming the upstream side walls of the recommended pool for the DCAD option.

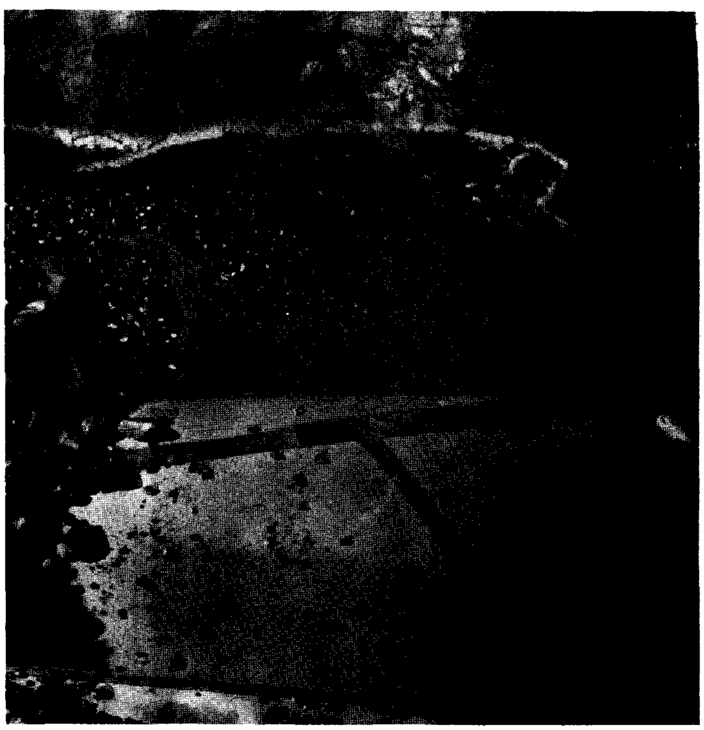


Figure 48. - Erosion of erodible concrete pool walls after tests under design discharge with material in the pool. (Note: Premature failure occurred at the joint with the fixed side wall along the right side.)

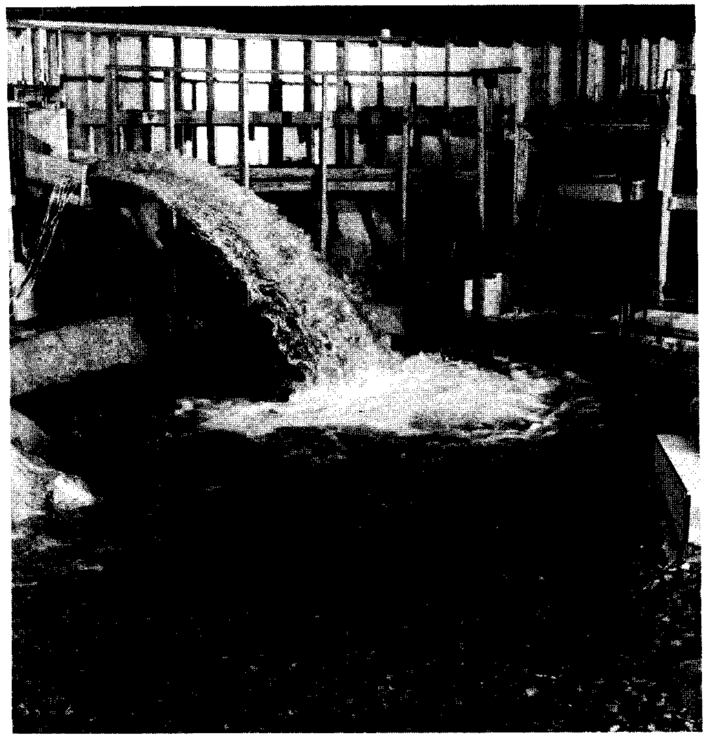


Figure 49. - Right spillway operation at 25,000 ft^3/s and elevation 2175 for the final plunge pool design of the SCAD.

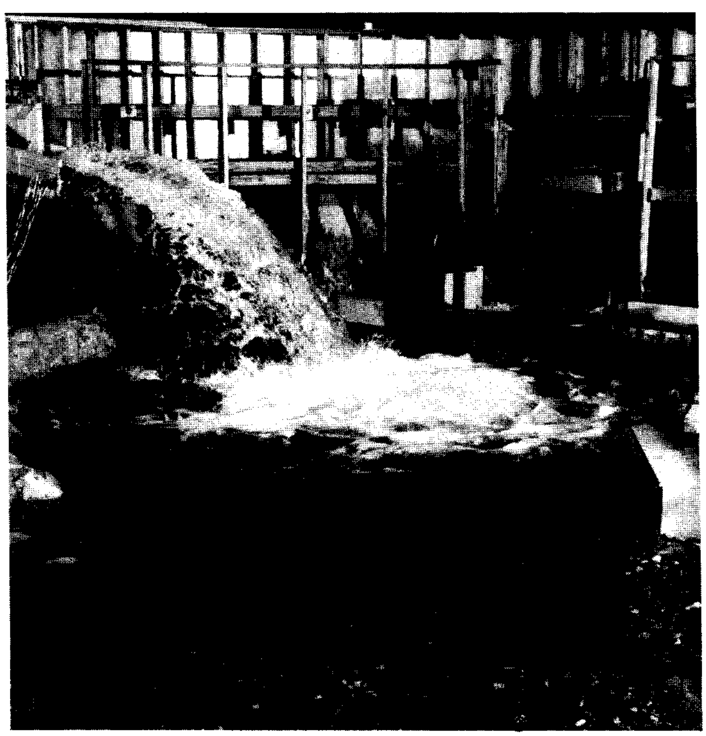


Figure 50. - Right spillway operation at 41,000 ft^3/s and elevation 2175 for the final pool design of the SCAD.

Figure 51. - Flow impinging on the powerplant road from initial opening of the left spillway for the SCAD design.

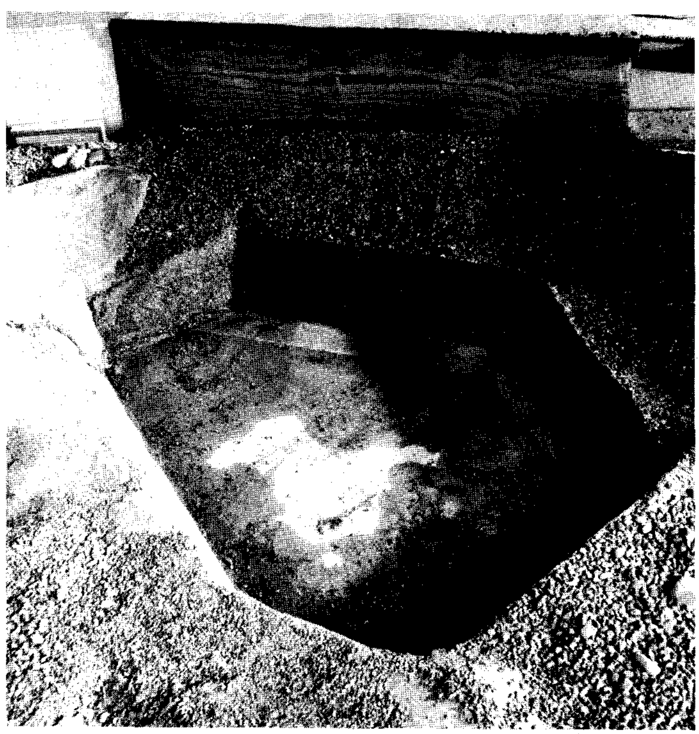


Figure 52. - Erodible bed material forming plunge pool walls and area adjacent to powerplant road for the final SCAD design. (Outlet works structure is shown in upper right corner of photograph.)

Figure 53. - Erosion pattern after operating at elevation 2151 with about 11,000 ft^3/s through the outlet works and 14,000 ft^3/s through the right spillway of the SCAD. (Outlet works structure is shown in upper right corner of photograph.)

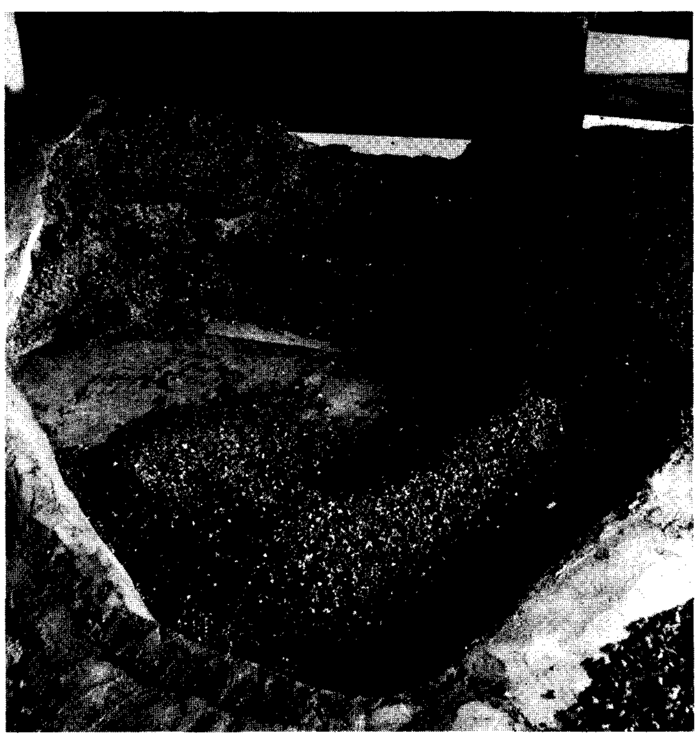


Figure 54. - Erosion pattern after operating at elevation 2151 with 25,000 ft^3/s through the right spillway of the SCAD. (Outlet works structure is shown in upper right corner of photograph.)

Figure 55. - Erosion pattern after operating at elevation 2175 with 25,000 ft^3/s through the right spillway of the SCAD.

★ U.S GPO:1990 773-182

Mission of the Bureau of Reclamation

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.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

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.

A free pamphlet is available from the Bureau entitled "Publications for Sale." It describes some of the technical publications currently available, their cost, and how to order them. The pamphlet can be obtained upon request from the Bureau of Reclamation, Attn D-7923A, PO Box 25007, Denver Federal Center, Denver CO 80225-0007.