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HYDRAULIC MODEL STUDY OF BRANTLEY DAM SPILLWAY

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16. ABSTRACT

Hydraulic model studies on the spillway at Brantley Dam in New Mexico confirmed the design of the spillway, slotted bucket energy dissipator, and the spillway tailwater channel. Side piers at the spillway entrance were modified to improve spillway flow; and rating curves were obtained for spillway discharge versus reservoir elevation for both free flow and various gate openings. Sediment deposits, immediately upstream from the spillway, were also investigated. Spillway approach velocities washed sediment over the spillway, with minor lowering of the spillway discharge. The slotted bucket energy dissipator worked very well. Immediately downstream from the energy dissipator, the tailwater channel was excavated out of bedrock; and then transitioned upwards to the bottom of the outlet channel. The maximum discharge readily eroded the flood plain downstream from the energy dissipator. A riprapped berm on each side of the tailwater channel protected the dam against erosion.

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by

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INTRODUCTION

Brantley Dam is located in the Southwest Region near Carlsbad, New Mexico, see figure 1(a). The composite dam will be a concrete and embankment structure about 108 feet (33 m) high at the highest point and about 21,000 feet (6400 m) long. Located on the right side of the river is the concrete control structure, which has an overflow section (spillway), and on each side of the spillway is a concrete nonoverflow section that extends into the embankment dam. The portion of the embankment dam that ends at the concrete control structure is designated the terminal cone, see figure 1(b). The spillway has six radial gates and a slotted bucket energy dissipator. The purpose of the model study was to confirm the design of the spillway; slotted bucket energy dissipator; and spillway tailwater channel.

CONCLUSIONS

1. Without sediment deposits, the 300-foot (91.4-m) wide spillway passed the 342,000-ft^3/s (9684-m^3/s) maximum spillway discharge at a reservoir water surface elevation of 3302.5 feet (1006.6 m), just slightly below the 3303.5-foot (1006.9-m) maximum water surface elevation.

2. Model tests showed sediment deposits eroded immediately upstream from the spillway with a 342,000-ft^3/s spillway discharge at a 3303.5-foot reservoir water surface elevation (sedimentation test No. 3, fig. 12).

3. The recommended side pier design (fig. 7) improved flow through the outside bays of the spillway and increased the spillway capacity. Some flow separation still occurred, but the design was believed an appropriate compromise between improved flow capacity and structural costs.

4. The slotted bucket energy dissipator worked well for all discharges, and was safe from "sweepout."

5. The recommended design of the spillway tailwater channel (fig. 18) was developed after making a series of tests. Large flood flows will wash overburden material off the bedrock, but the riprapped berms will protect the terminal cones of the zoned embankment dam.

6. Some areas with 2-foot (0.6-m) riprap might require additional protection or stabilization (figs. 21 and 22). Model tests indicated it was extremely important that a high degree of stabilization be provided for the area on the upstream side of the spillway, as shown on figure 22.

7. When spillway discharges are controlled by the radial gates, it is best that all gates operate with the same gate opening. Individual gate operation can create flow currents that bring sand and gravel into the stilling basin (fig. 23). However, when low spillway releases are necessary, gates 3 and 4 (fig. 22) should be opened first. If higher discharges are required, additional gates should be opened proceeding outward to gates 2 and 5, and then gates 1 and 6.

THE MODEL

The model scale was 1:66. A headbox, upstream from the model spillway, contained the reservoir and upstream face of the dam (fig. 2). A tailbox, downstream from the spillway, contained the slotted bucket energy dissipator and discharge channels. The spillway, piers, and slotted bucket energy dissipator were made with a hard, dense, plastic foam (fig. 2(b)). Flow entering the model was measured by Venturi meters, which had been volumetrically calibrated, and a rock baffle dampened turbulence from the inlet pipe (fig. 3). The reservoir topography extended upstream from the spillway to a curved bulkhead. The shape of the bulkhead (plan view) was made to approximate a potential line of a flownet for water approaching the spillway. Model topography was formed using pit-run sand, see table 1.

Gravel, ⅜- to ¾-inch (10- to 20-mm) in diameter, was placed on flow surfaces of the earth dam to simulate riprap. Two types of topography were used in the model tailbox (fig. 4): a concrete mortar that simulated the underlying bedrock downstream from the spillway and pit-run sand for the erodible overburden covering the bedrock. It should be noted that the erosion was not modeled in the sense that an eroded depth in the model would be 66 times deeper in the prototype; instead, model erosion was used to help make qualitative judgments about effectiveness for various appurtenances tested in the model.

Table 1. – Size analysis for one sample of pit-run sand.

Sieve size		Percent
No.	SI metric	passing
4	4.75 mm	100
8	2.36 mm	93
16	1.18 mm	65
30	600 µm	39
50	300 µm	18
100	150 µm	5

SPILLWAY TESTS

Reservoir Water Surface Elevation

Conditions in the model headbox were such that a direct measurement of the reservoir water surface elevation could not be made. Some head loss occurred as water flowed over the topography bulkhead (fig. 3), which meant that any measurements taken upstream from the bulkhead would not be correct. Model water surface elevation measurements were made with a static head probe located at the spillway centerline at a prototype distance of 150 feet (45.7 m) upstream from the spillway. Tubing connected the static probe to a measuring well on the outside of the box. To obtain the reservoir water surface elevation, the velocity head was added to the static head probe measurement. Velocities were computed by dividing discharge by flow area; the flow area being along the dashed line shown on figure 3.

Side Piers

The model spillway was constructed according to the preliminary design, and the piers at each side of the spillway had the same shape and dimensions as the intermediate piers. Observations were made for a range of free flow discharges passing over the spillway. At high discharges, excessive flow separation occurred on each side of the spillway (fig. 5).

Three modifications of the side pier were tested (fig. 6). These modifications were made in such a manner that they could be readily attached and removed from the right side of the model while water was flowing through the spillway. Thus, each modification could be compared to the preliminary design side pier. Each modification improved the flow. Flow separation decreased, and the crosshatched area (fig. 5) was covered with water. A quantitative measure of flow improvement was made in the following manner: The model was operated with a 352,000-ft^3/s (9968-m^3/s) discharge, and then the reservoir water surface elevation was measured. Using this same discharge, each modification was then placed in the model and the water surface elevation measured. All three modifications lowered the model water surface below that of the preliminary design:

Modification No.	Distance Lowered, inches (mm)
1	0.07 (1.8)
2	0.07 (1.8)
3	0.08 (2.0)

Side pier modification No. 3 was selected and permanently constructed on both sides of the model spillway.

Later in the study, a design change was made to the shape of the concrete nonoverflow section of the dam on each side of the spillway. In the preliminary design, the upstream face of this section was vertical. This face was changed to a 0.25:1 horizontal to vertical slope to provide better compaction of embankment against the dam. This face change required a slight change in the spillway side piers, so new side piers were installed in the model, figure 7. Discharge through the spillway was similar to that of side pier modification No. 3, figures 6 and 9. The side pier shown on figure 7 is the recommended design for Brantley Dam spillway.

Free Flow

Free-flow tests were made with the permanent side pier modification No. 3 in place and for the preliminary design spillway. The spillway width was composed of five 10-foot (3.05-m) thick intermediate piers and six 54.33-foot (16.56-m) wide bays. The maximum design flood of 352,000 ft^3/s readily passed through the spillway. Model test data (discharge and water surface elevation) were plotted on a large scale working graph, and a curve drawn through the slight distribution of data points. Then, at selected reservoir elevations, the spillway discharge coefficient was computed using the following equation:

$$Q = CLH_e^{3/2} \tag{1}$$

where:

Q = spillway discharge, in cubic feet per second;
C = spillway discharge coefficient, square root of acceleration, $(ft/s^2)^{1/2}$;
L = total spillway width between piers, in feet; and
H_e = total head on crest, including velocity head of approaching flow, in feet.

The discharge coefficient C in equation (1) is not a dimensionless coefficient and, therefore, the equation in this form is not readily converted to a metric equivalent. The values for the coefficient of discharge versus reservoir elevation are shown on figure 8.

The discharge of 352,000 ft^3/s at the maximum design reservoir water surface elevation of 3303.5 feet (1006.9 m) was higher than required. The designers used coefficients of discharge obtained from the model studies (sedimentation test No. 3) to route the IDF (inflow design flood) and size

the spillway. A 300-foot (91.4-m) spillway width was adequate with the maximum spillway discharge of 342,000 ft^3/s (9684 m^3/s). Therefore, for recommended design, the clear opening of the spillway was reduced to 300 feet, or 26 feet (7.9 m) less than in the preliminary design. The total spillway width was 350 feet (106.7 m), with five 10-foot (3.05-m) wide intermediate piers and six 50-foot (15.24-m) wide bays.

Although the spillway width was shortened for the recommended design, this change was not made in the model. Instead, the model discharges were increased by a factor of (326/300), thus maintaining the same unit discharge for the 326-foot (99.4-m) spillway as for a 300-foot (91.4-m) wide spillway. Likewise, this unit discharge technique was used to reduce model data to that of the 300-foot-wide spillway, figure 9. Hereafter, all discharges mentioned in this report relate to the 300-foot spillway.

Reservoir Sedimentation

Sediment deposits are expected to occur in the reservoir over an extended time. One estimate has the sediment level at the spillway crest elevation after 100 years. Three reservoir sedimentation tests were made in the model to determine the effect upon spillway discharge.

For sediment test No. 1, the inlet channel to the outlet works was filled to the level of the surrounding topography, about elevation 3230 feet (984.5 m). The sand used for this test was the same size as that used for forming the model topography. Free-flow spillway tests were made, and there was no measurable effect upon spillway discharge capacity. However, some slight erosion was detected near the dam. Some fines had eroded from the upstream surface and formed a small dune in front of the spillway crest. Also, fines were removed adjacent to the face of the dam, figure 10.

For sediment test No. 2, sand was placed up to elevation 3244 feet (988.8 m). The sand placement extended upstream and, near the topography bulkhead, the sand surface was sloped 45° downward to the bulkhead. Near each side of the spillway, some sand eroded below the vertical face of the corbel (fig. 11). Some of the larger sand grains deposited against the vertical corbel face, forming a ramp that allowed eroding sand to be carried over the spillway. Model discharges were progressively increased, and each discharge was allowed to act upon the reservoir bed for 10 to 30 minutes. Discharge capacity through the spillway was slightly decreased (fig. 12).

For sediment test No. 3, sand was placed up to spillway crest elevation 3259.5 feet (993.5 m). As model discharges were progressively increased to 342,000 ft^3/s (9684 m^3/s), the erosion

upstream from the spillway increased. After completion of the test, measurements were made of the erosion (fig. 13). The spillway discharge capacity was slightly less than that of sediment test No. 2 (fig. 12).

Topography immediately upstream from the spillway contributed to a self-cleaning action of sediment in front of the spillway. The concrete spillway is flanked on each side by the terminal cone of the embankment dam. On each side of the spillway, the front face of the embankment dam curves inward toward the spillway (fig. 3). Thus, the topography converges the water flow toward the spillway and flow velocities increase as water approaches the spillway. In the model, these velocities were sufficient to flush sand and riprap-size model gravel over the spillway.

One interpretation of the sedimentation tests was that the extent of erosion would be greater in the prototype than in the model. In the model, pit-run sand, which is larger than the prototype bed material, was used to represent the sediment deposit. The sand (even the coarse particles) moved downstream to the spillway and passed over the spillway. However, some of the largest coarse particles were not carried over the model spillway, but deposited near the upstream face of the spillway crest. Actual size of the prototype sediment in front of the spillway is anticipated to be even smaller than that used in the model tests. The larger size sediment would probably deposit much further upstream in the reservoir. Also, by Froude number scaling, the prototype velocities will be approximately eight times greater than the model velocities. Thus, with higher velocities acting on smaller sediment in the prototype, the erosion is expected to be more extensive in the prototype than in the model.

During review of the spillway design, results of sedimentation tests were questioned. Possibly, the fine sediment would consolidate over time and not readily erode like noncohesive sand. Thus, sedimentation test No. 4 was made with concrete mortar placed in the model reservoir at 3259.5-feet (993.5-m) elevation. At the maximum reservoir elevation of 3303.5 feet (1006.9 m), the discharge was 317,000 ft^3/s (8976 m^3/s) (fig. 12), which is about 7 percent less than the spillway design discharge of 342,000 ft^3/s (9684 m^3/s). The average velocities at 10 and 150 feet (3.05 and 45.7 m) upstream from the spillway were 20 and 13 ft/s (6.1 and 4.0 m/s), respectively. Model tests cannot definitely prove that the prototype spillway will pass the required 342,000-ft^3/s discharge under severe sedimentation conditions. A judgment must be made considering the type of sediment and flow velocities acting on the sediment.

Gate Tests

Model tests were made with different gate openings to obtain discharge ratings, figure 9. Note that the G.O. (gate opening) is defined as "the vertical distance from the crest to the bottom of the gate."

Observations were made for flow through the spillway with the gates in the fully raised position. For the 342,000-ft^3/s flood and sediment test No. 3, water occasionally impinged against the bottom of gates No. 1 and 6 (gates were numbered from right to left facing downstream, fig. 22). Thus, the designers raised the bottom gate position 2.5 feet (0.76 m) above that of the preliminary design. For the recommended design, the bottom of the gate, at the fully raised position, is at elevation 3296 feet (1004.6 m).

ENERGY DISSIPATOR TESTS

The energy dissipator is a slotted bucket. Tests were made over a range of discharges with the corresponding tailwater for each discharge (fig. 14). At low discharges, there was little disturbance of the tailwater surface, figure 15(a). With increased discharges, the tailwater turbulence progressively increased (figs. 15(b), 15(c), and 16(a)); and at high discharges there was a turbulent boil on the water surface.

"Sweepout" tests were made with only the bedrock topography downstream from the slotted bucket. At 345,000 ft^3/s (9769 m^3/s), sweepout occurred at tailwater elevation 3233 feet (985.4 m). The anticipated tailwater elevation for this discharge was 3243 feet (988.5 m).

The top elevation of the slotted bucket side walls was 3238.5 feet (987.1 m). At high discharges, water will flow over the side walls into the slotted bucket (tailwater curve, fig. 14). Note the water level in back of left wingwall of slotted bucket, figures 15(c) and 16(a).

SPILLWAY TAILWATER CHANNEL

Preliminary Design

Figure 17 is a schematic of the preliminary design spillway tailwater channel and outlet channel. The tailwater channel is located between the slotted bucket energy dissipator and the outlet channel. The elevation of the tailwater channel changes in the transition from the end of the bucket to the dual trapezoidal channels of the outlet channel, figure 17 section A-A. The pilot channel will handle outlet works flows up to 500 ft^3/s (14.2 m^3/s), and the larger trapezoidal channel will handle spillway flows up to about 30,000 ft^3/s (850 m^3/s). For discharges greater than 30,000 to 40,000 ft^3/s (850 to 1133 m^3/s), the water progressively spreads out over the flood plain. At 100,000-ft^3/s (2832 m^3/s) discharge, the water extends 1.5 miles (2.4 km) widthwise across the

flood plain. Thus, for discharges greater than 40,000 ft^3/s, the function of the spillway outlet channel for conveying water to the river becomes insignificant.

General

Considerable testing was done for the spillway tailwater channel. The presence of bedrock, about 30 feet (9 m) below the ground surface, was beneficial as a nonerodible boundary. Concrete mortar was placed in the model to represent the bedrock topography and bedrock excavation immediately downstream from the end of the slotted bucket. Pit-run sand was used for the erodible material overlaying the bedrock. Topography was placed in the model tailbox for a prototype distance of 800 feet (244 m) downstream from the spillway. The recommended design for the spillway tailwater channel is shown on figure 18. The only changes from the preliminary design were the slotted-bucket wingwalls and the 160-foot (49-m) long riprapped berms at each side of the tailwater channel.

Slotted Bucket Wingwalls and Sidewalls

The shapes of the wingwalls were developed from the model tests. For the preliminary design (fig. 17), the right wingwall was straight and at about a 90° angle from the stilling basin side wall. The left wingwall was at a 45° angle and then had a dog-leg farther out from the stilling basin side wall. Observations of model flow conditions indicated better performance with the left wingwall; i.e., smaller and less intense eddy action at the left side of the spillway tailwater channel. A 130-foot (39.6-m) long wingwall at a 45° angle was also tried on the right side; however, the left side still had better flow conditions. Next, the wingwall (recommended design as shown on fig. 18) was placed at the right side of the stilling basin. This configuration produced less intense eddy action than the two previous wingwalls.

During high discharges, the tailwater elevation is above the top of the slotted bucket wingwalls and sidewalls. An eddy current overtopped the wingwalls, water flowed back to the sidewalls, and then flowed over the sidewalls reentering the slotted bucket. Potentially, the top of the fill behind the wingwalls and sidewalls could be eroded. Thus, the height of the slotted bucket wingwalls and sidewalls was increased by 1 foot (0.3 m) over that of the preliminary design. The recommended elevation is 3238.5 feet (987.1 m) for the wingwalls and side walls. This modification reduced flow over the top of the walls, which reduced the erosion potential of the top fill behind the walls.

Initial Erosion Test

The initial erosion test was made with the preliminary design configuration using 3-foot (0.9-m) riprap on each side of the tailwater channel, from the wingwall, 400 feet (122 m) downstream to the outlet channel. A 324,000-ft^3/s (9175-m^3/s) flood flow rapidly destroyed the tailwater channel and outlet channel and endangered the terminal cone of the embankment dam near the spillway. The erodible bed was flushed off the concrete mortar downstream from the spillway tailwater channel, figure 4.

This erosion test provided information for the recommended design. The tailwater elevation was above the riprapped side slope and sand eroded from behind the riprap, which caused riprap failure (note each side of tailwater channel on fig. 4). Either the riprapped side slope should extend above the water surface or additional riprap should be placed on the flat surface above the side slope to prevent erosion of the material from back of the riprap. Much of the riprapped side slope did not rest on bedrock, but was on the erodible material. Flow velocities readily eroded this material, undermining the riprap. Therefore, the riprapped side slope should rest on bedrock to prevent this type of failure. Flow velocities removed the sand from the bedrock for a considerable distance downstream from the spillway. Protecting the flood plain from erosion would be prohibitively expensive; however, the flood plain erosion had a compensating effect in that the larger flow area reduced velocities. The economical design for the spillway tailwater channel and spillway outlet channel appeared to be one that would allow erosion of the downstream flood plain and also provide riprapped berms to protect the terminal cone of the zoned earthfill dam.

Recommended Design

The recommended design for the spillway tailwater channel is shown on figure 18. The general design concept was to direct high velocities out to the flood plain. Bedrock would limit erosion in the vertical direction and, in the downstream horizontal direction, erosion would proceed outward until the velocity dissipated. The riprapped berms were angled outward from the spillway tailwater channel, away from the high velocity. Both berms had riprap on the top. The riprap extended down the slopes to bedrock and completely around the end of the berm. On the back side of the left berm, the riprap extended down to the original surface. The 3235-foot (986-m) top elevation at the right berm was similar to the ground surface, and the model riprap was placed outward to the tailbox side wall. Pit-run sand was placed on the concrete mortar to form the spillway tailwater and outlet channels and overburden topography.

The recommended spillway tailwater channel and outlet channel design was tested with a progressive series of discharges similar to a flood routing. At 30,000 ft^3/s (850 m^3/s), no appreciable

erosion occurred; and at 50,000 ft^3/s (1416 m^3/s), slight erosion occurred along the spillway tailwater channel invert at the upstream edge of the 3185-foot (971-m) elevation. At 186,000 ft^3/s (5267 m^3/s), considerable erosion occurred; and sand was flushed off the bedrock surface almost to the end of the tailbox topography. The area of exposed bedrock slightly increased widthwise for the 238,000- and 342,000-ft^3/s (6739- and 9684-m^3/s) discharges, and reached the end of the installed topography. Bedrock would probably have been exposed further than 800 feet (244 m) from the spillway if it had been installed in the model. Erosion tests are shown on figures 19 and 20(a).

A greater depth of sand was eroded from the downstream vicinity of the right berm than from the left berm. This erosion difference was probably affected by proximity of the model tailbox side wall (fig. 18). Thus, for judgment of berm design, more weight should be given to occurrences at the left berm. Adjacent to the left berm, the exposed mortar extended 80 feet (24.4 m) downstream from the wingwall, figure 20(b). Proceeding further downstream, less sand eroded from the berm.

At the end of the berm, very little sand eroded from the 3221-foot (981.8-m) elevation ground surface. Although sand erosion was slight at the end of the berm, it is recommended that the berm be protected down to bedrock. Prototype erosion could possibly be more than was shown by the model because of higher velocities and finer material. The objective was to provide enough riprap coverage and to ensure that riprap failure would not occur by erosion in back of the riprap.

RIPRAP STABILITY

Riprap stability was also investigated in the model test; however, similitude from model to prototype is not exact for riprap movement. Also, factors of prototype riprap shape and placement, such as interlocking of adjacent stones, may not be duplicated in the model. However, the model will indicate areas of riprap erosion. Initially, model tests were made with gravel representing 2- to 3-foot (0.6- to 0.9-m) riprap; however, there was some question that field riprap may be limited to 2 feet. For tests of the recommended design, model gravel representing 1- to 2-foot (0.3- to 0.6-m) riprap was used. With the 342,000-ft^3/s (9684-m^3/s) discharge, sand and riprap eroded from the right berm next to the wingwall. Turbulence was generated at the eroded area as flow moved upstream against the wingwall, and then was deflected downward against the model riprap. Apparently, the sand beneath the model gravel was disturbed. The eroded area was filled with model gravel and tested again, and no significant erosion was observed. A close inspection of the left berm detected areas of apparently thin gravel cover. Underwater observations made with

the model operating at 342,000-ft^3/s flow revealed occasional rock movement. Additional riprap protection is recommended for the sloping portion of each berm for a 40-foot (12.2-m) distance downstream of the wingwall, figure 21. Either 3-foot riprap or stabilization of the 2-foot riprap with concrete or shotcrete should be sufficient. The objective of using concrete or shotcrete would be to adhere individual stones together, making a larger mass to resist movement by the flowing water.

Measurements were made of flow velocities acting on the riprapped berms. An electromagnetic velocity meter sensor was located 10 feet (3 m) (prototype distance) from the riprap boundary. Considerable velocity fluctuation occurred at a given location at 3 to 9 ft/s (0.9 to 2.7 m/s). Thus, only the peak velocity pulses were considered when evaluating potential riprap movement. The flow was in an upstream direction and the velocities varied from 6 to 10 ft/s (1.8 to 3.0 m/s). The berms were not exposed to high velocity spillway flow, only to a large eddy on each side of the spillway tailwater channel.

Gravel on the concrete mortar did not appear as stable as the gravel on the berm as it readily flushed off the bedrock surface, see figure 20(b). Possibly, less force is needed to roll a stone along a relatively smooth surface than to dislodge a somewhat embedded stone. An area more susceptible to this movement is where the berm riprap rests on the bedrock. Additional riprap protection is therefore recommended for this location, figure 21. A remedial measure is needed to prevent stones from rolling, possibly by embedding the first layer of stones in a concrete pad or a key trench that restrains stone movement.

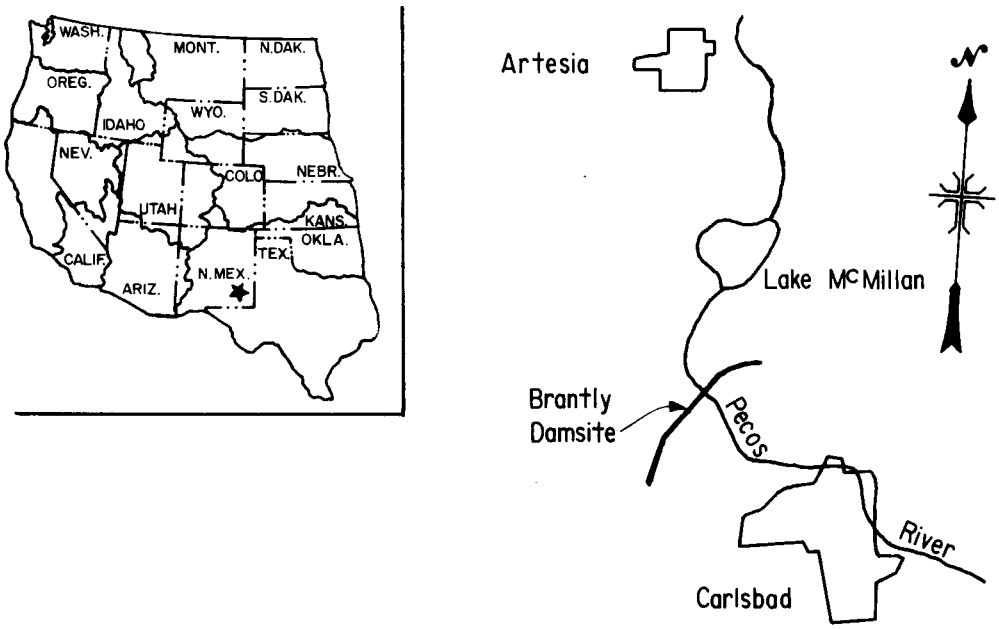
Riprap eroded immediately upstream from the spillway in the vicinity of each side pier. Additional testing was done to determine whether the riprap rolled down the embankment or was carried over the spillway. Model gravel painted orange was placed in the eroded areas, and the model was operated at a 342,000-ft^3/s (9684-m^3/s) discharge. The orange gravel was found downstream of the spillway tailwater channel, proving that the eroded riprap was carried over the spillway. The upstream area requiring riprap stabilization is shown on figure 22. This area is believed to be more critical to dam safety than the downstream areas. A high degree of stabilization is recommended.

LOW DISCHARGE RELEASES FOR SPILLWAY

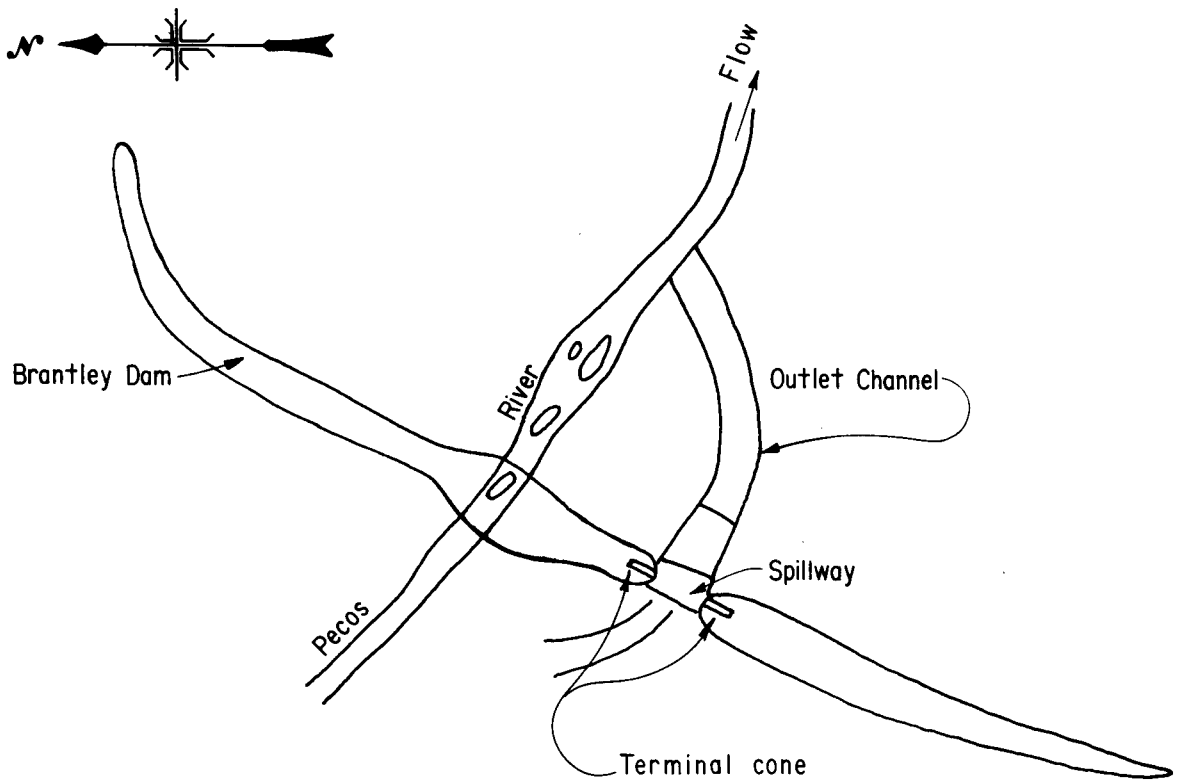
For the best flow conditions downstream from the slotted bucket, all six spillway gates should operate with the same gate opening. However at small discharges, it may be necessary to operate individual gates with relatively small gate openings. With the unbalanced operation, eddies can bring rocks and gravel into the slotted bucket.

Model tests were made with a 3-foot (0.9-m) gate opening to ensure sand movement. Two different operation modes were tried: (1) opening the two inside gates (Nos. 3 and 4), and (2) opening the two outside gates (Nos. 1 and 6). The gates are numbered from right to left, looking downstream (fig. 22). Less sand entered the basin with flow through the inside gates, figure 23(b). Thus, recommended operation is to start with the inside gates, and a maximum gate opening of 1 foot (0.3 m). If practical, an even smaller gate opening would be better. If higher discharges are required, open additional gates proceeding outward, starting with gates 2 and 5 and then gates 1 and 6.

With water standing in the model spillway tailwater channel, sand was placed in the roller bucket. Then, the model was operated with all gates open 3 feet. The sand was readily flushed from the bucket. For this mode of operation, the self-cleaning characteristics of the bucket are excellent.

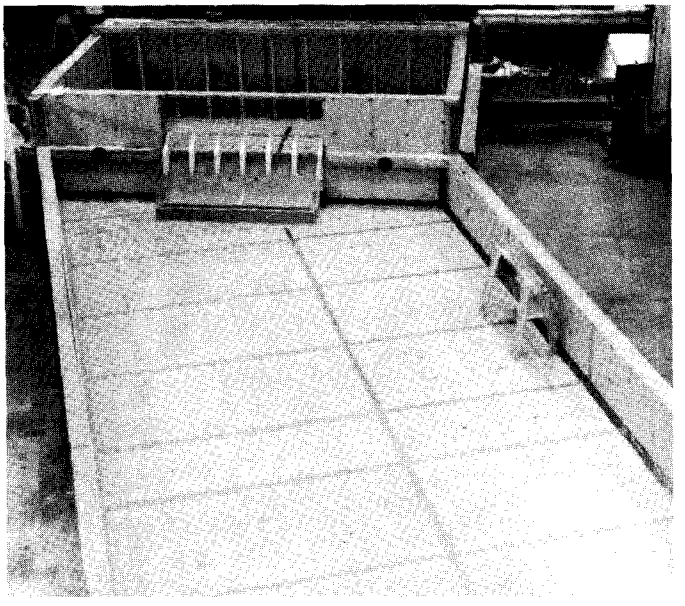


(a) Location maps.

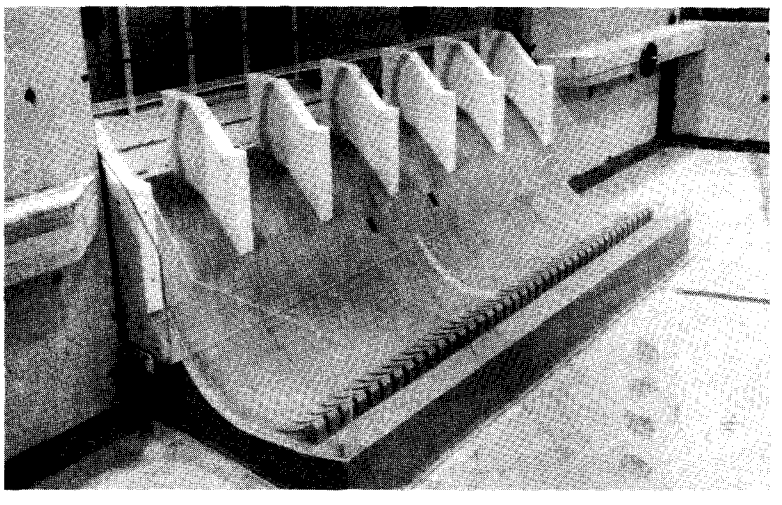


(b) General plan.

Figure 1. – Location of Brantley Dam.



(a) Model headbox and tailbox. P801-D-81129.



(b) Model spillway with slotted bucket energy dissipator. P801–D–81130.

Figure 2. – The model.

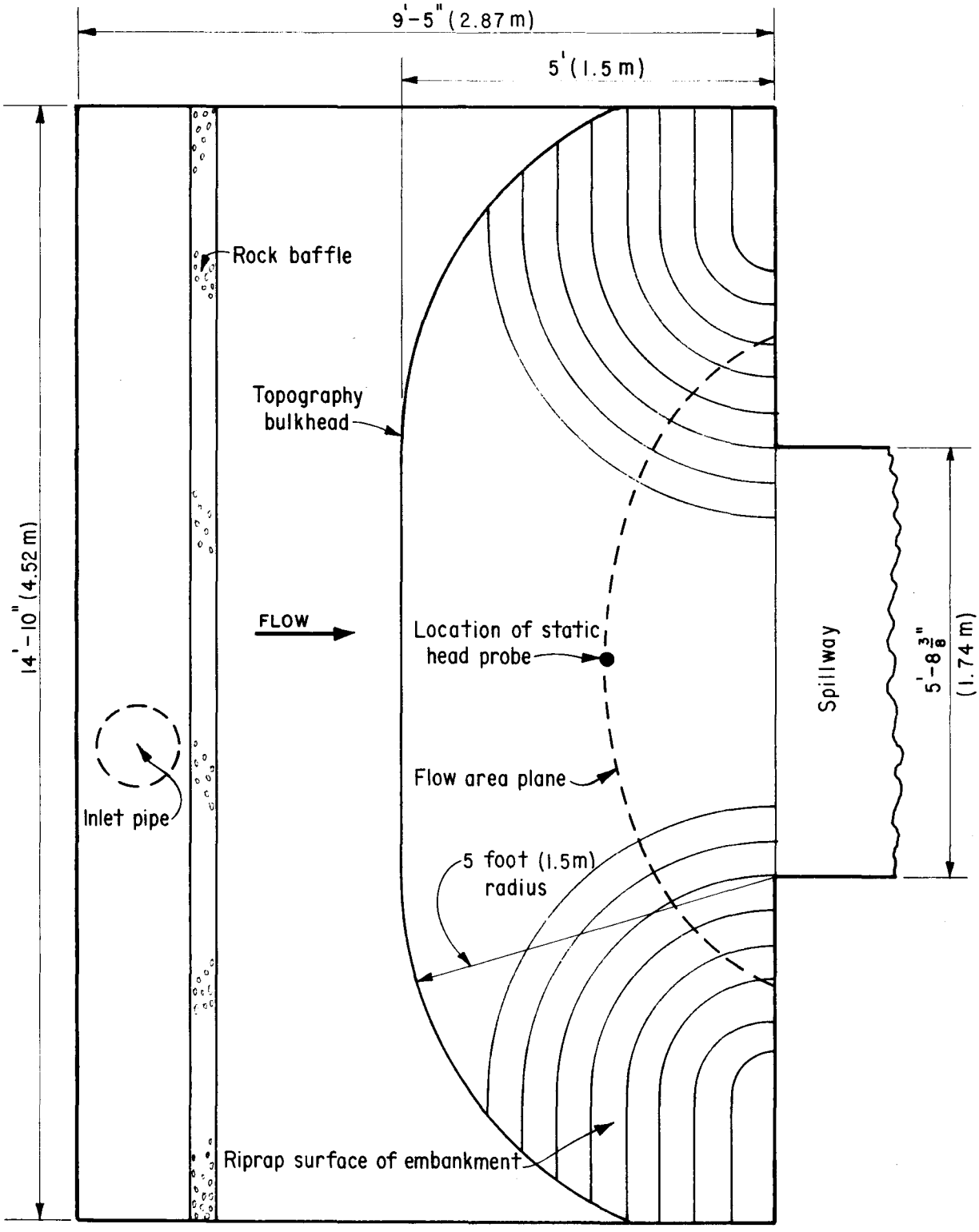


Figure 3. – Model headbox.

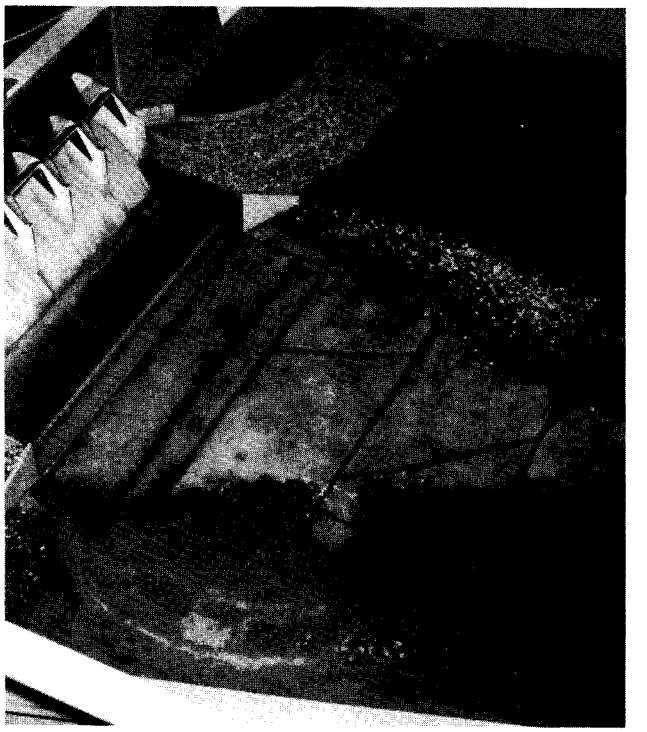


Figure 4. – Topography in model tailbox after erosion test No. 1. P801-D-81131

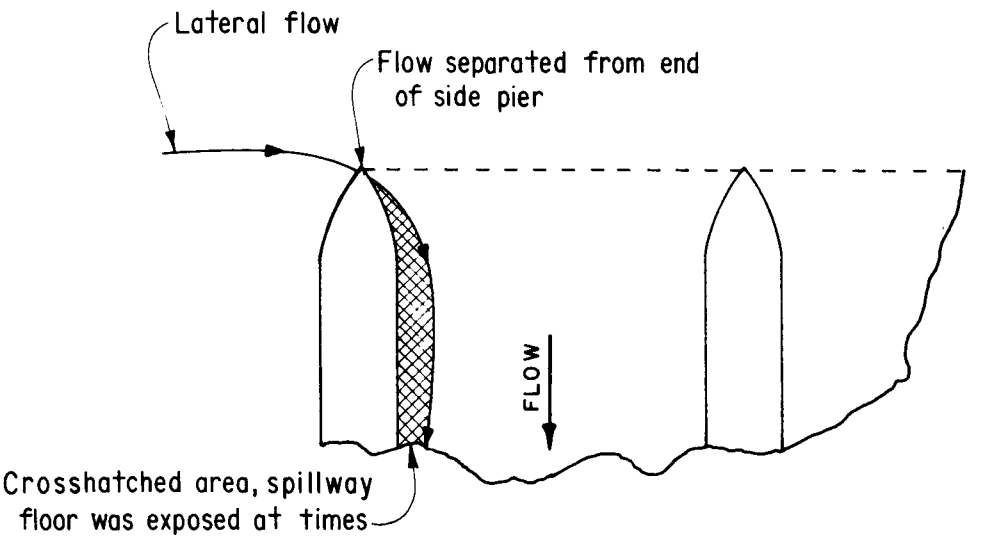


Figure 5. – Schematic of side pier flow separation.

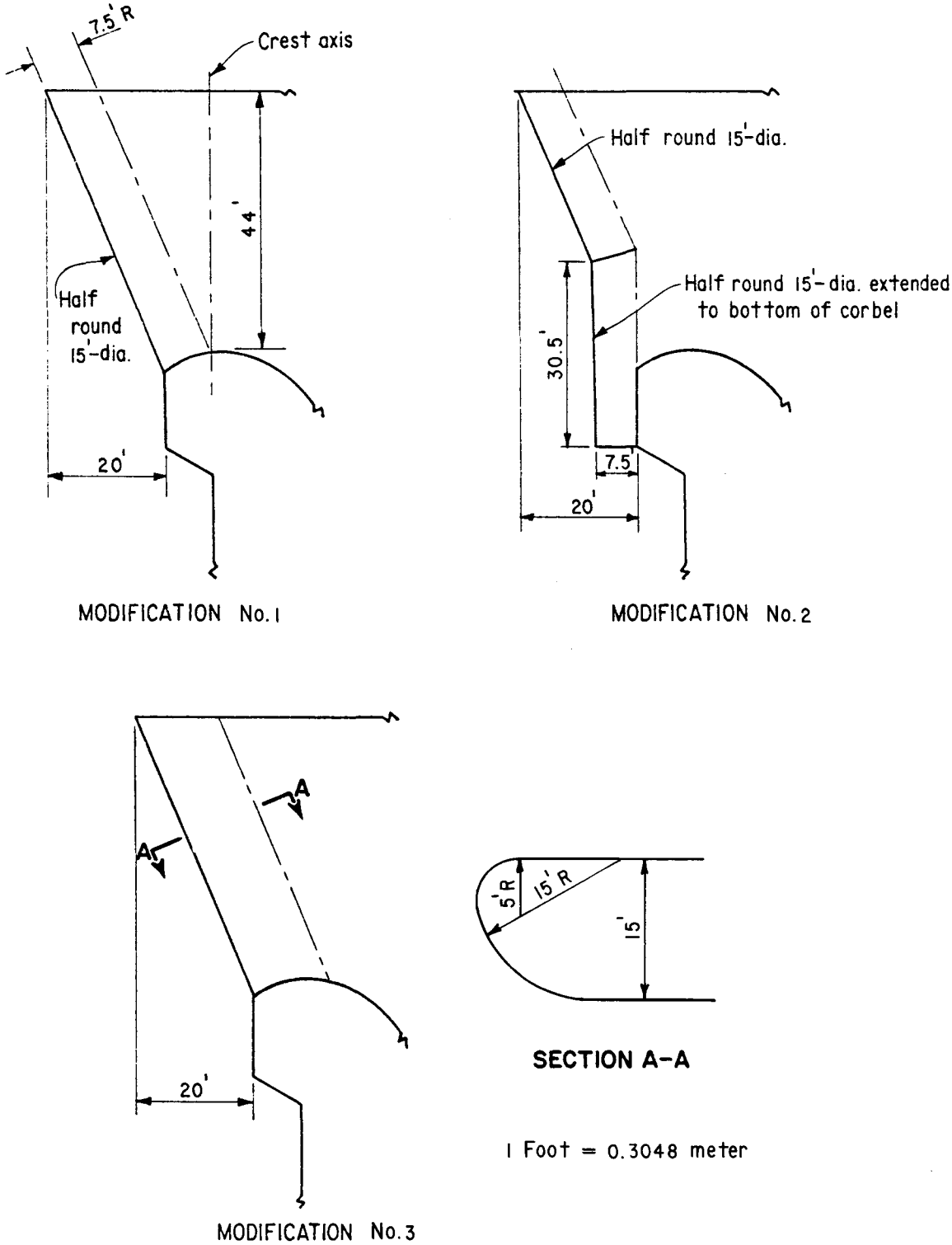


Figure 6. – Side pier modifications.

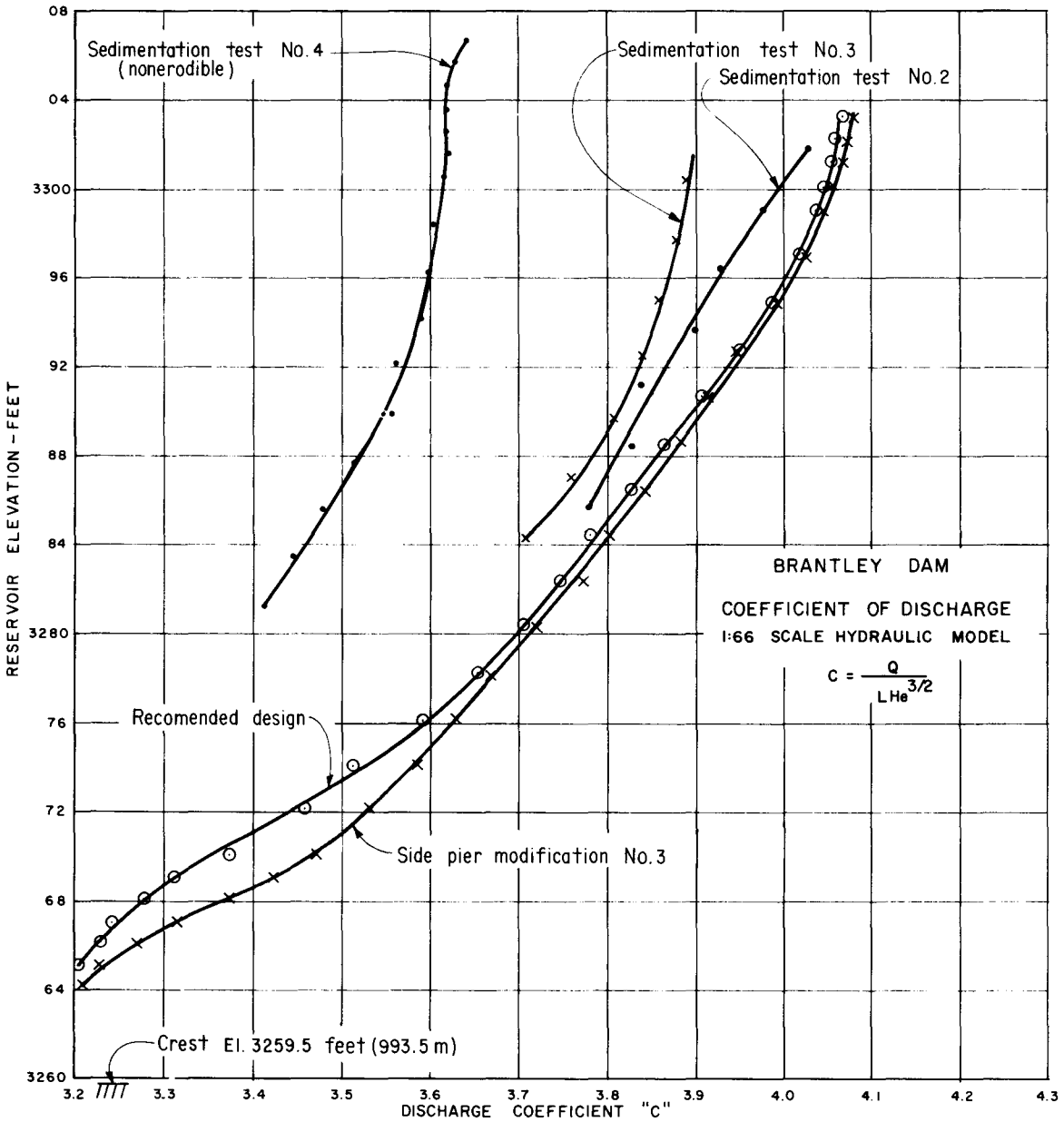


Figure 8. – Coefficient of discharge.

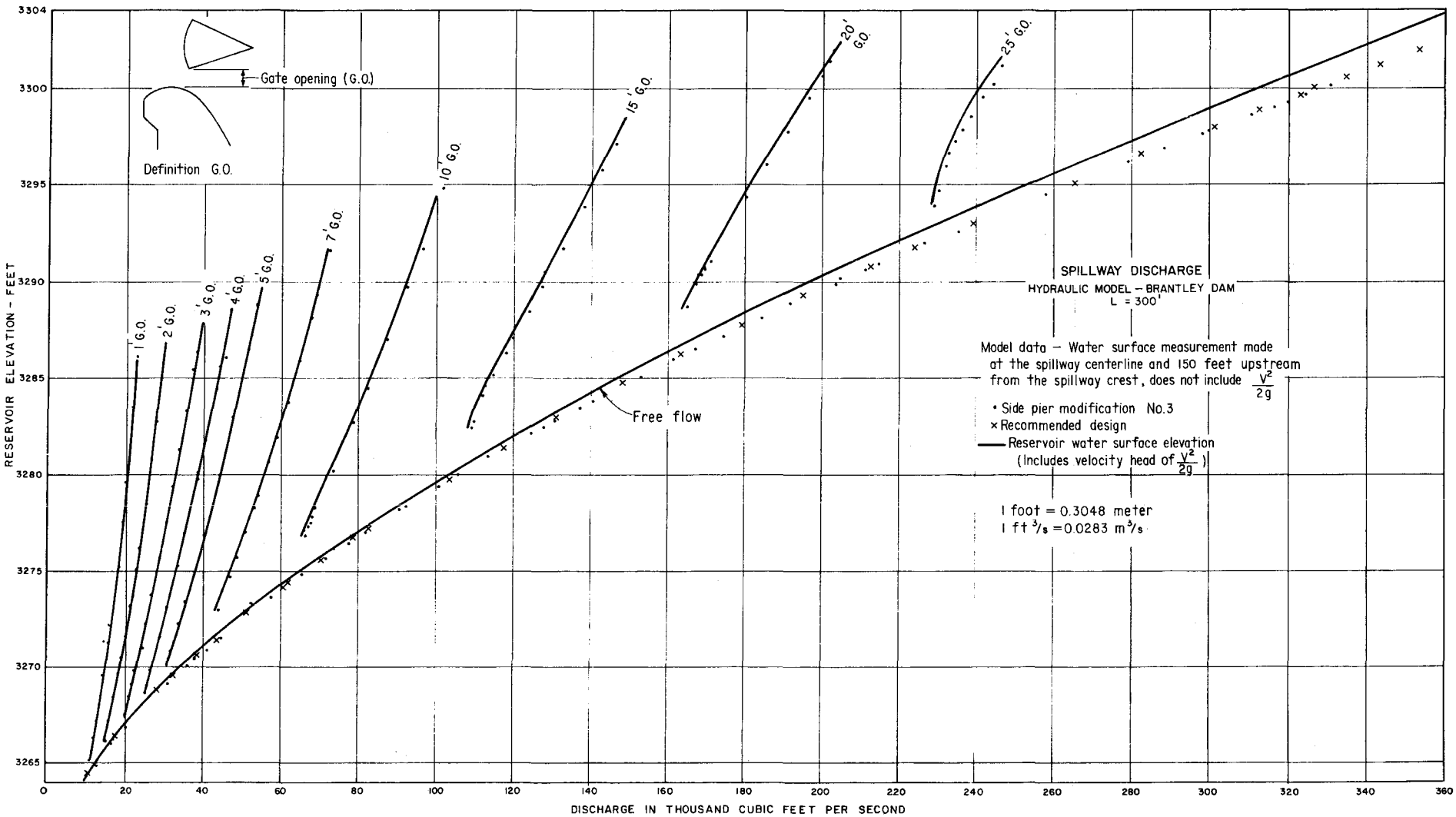


Figure 9. – Spillway discharge.

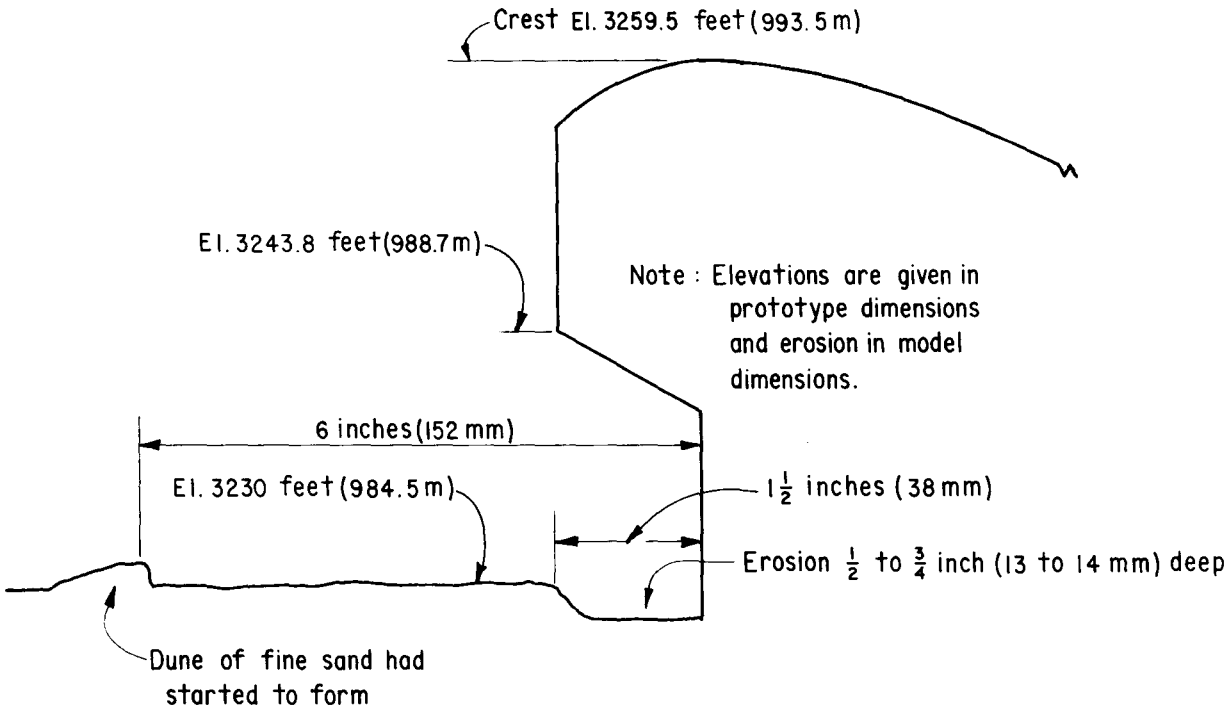


Figure 10. – Sediment test No. 1.

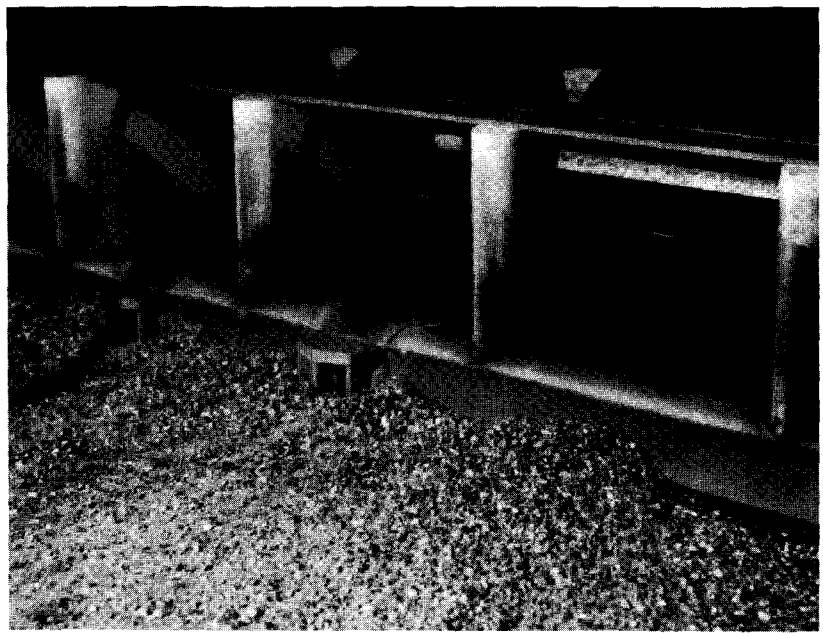


Figure 11. – Sediment test No. 2. P801-D-81132.

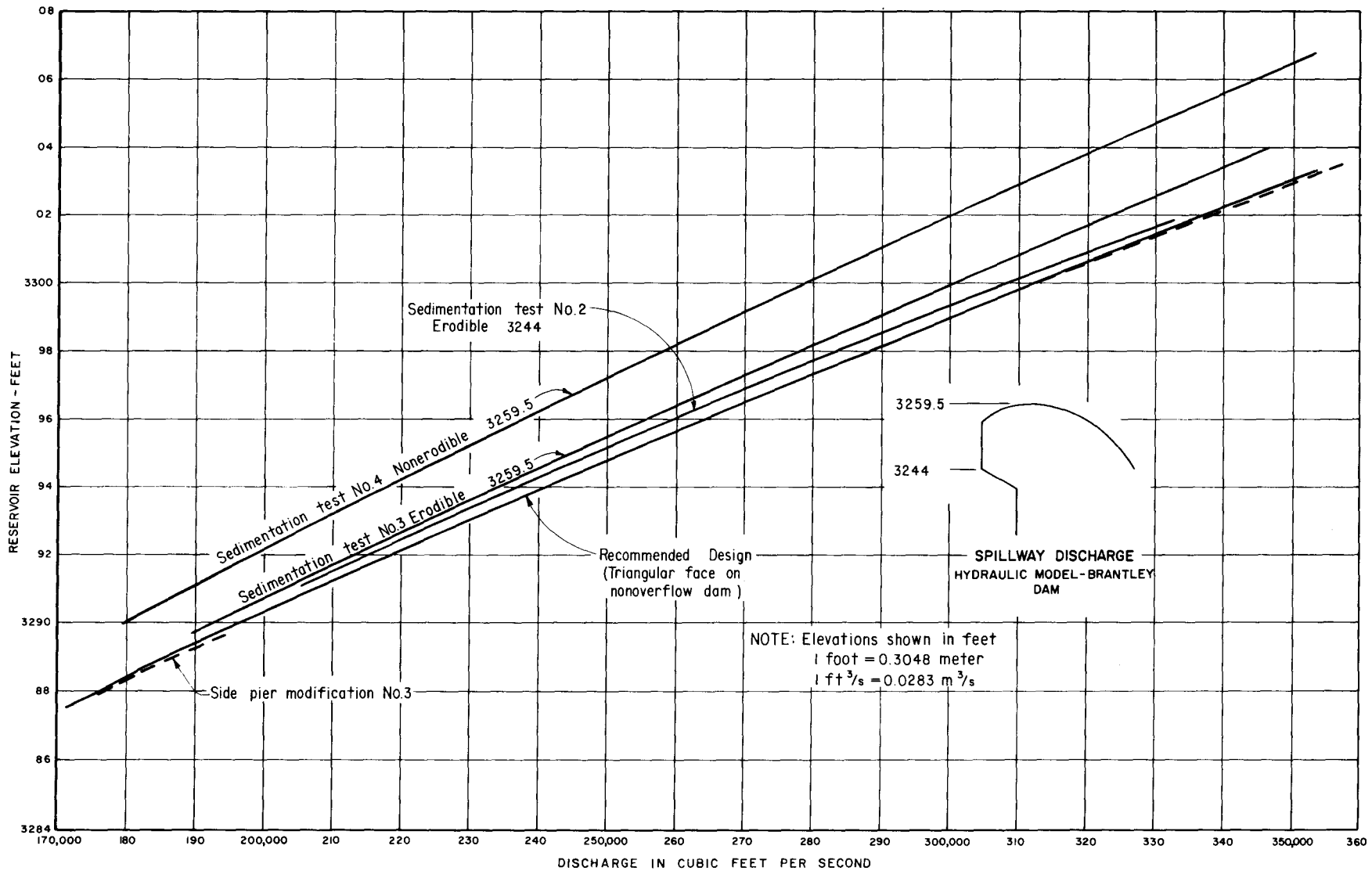


Figure 12. – Effect of reservoir sedimentation on spillway discharge.

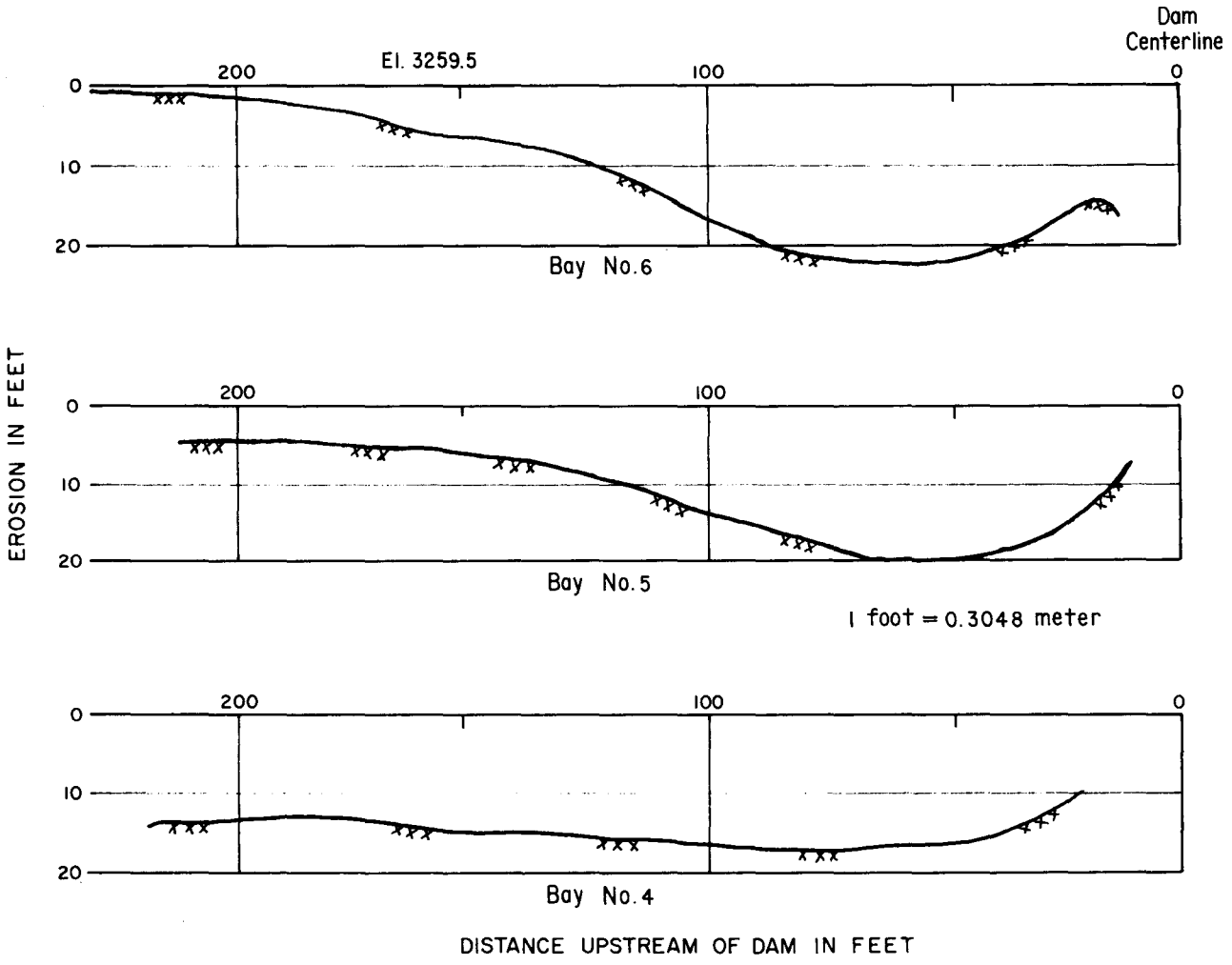


Figure 13. – Erosion for sediment test No. 3.

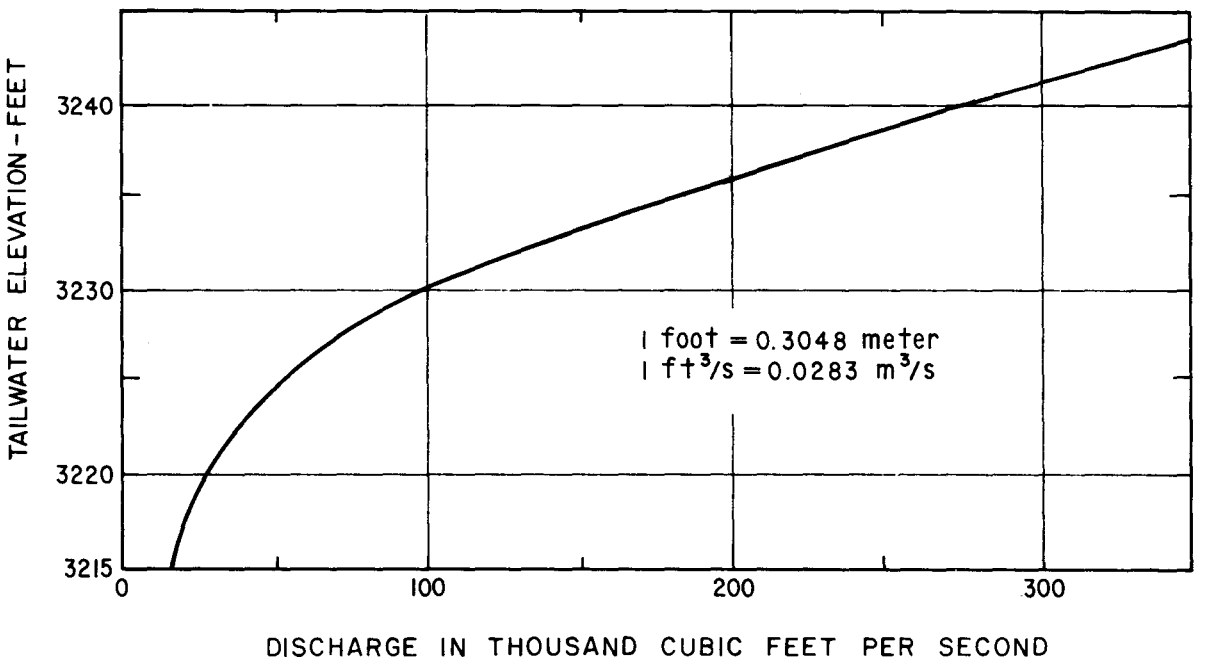
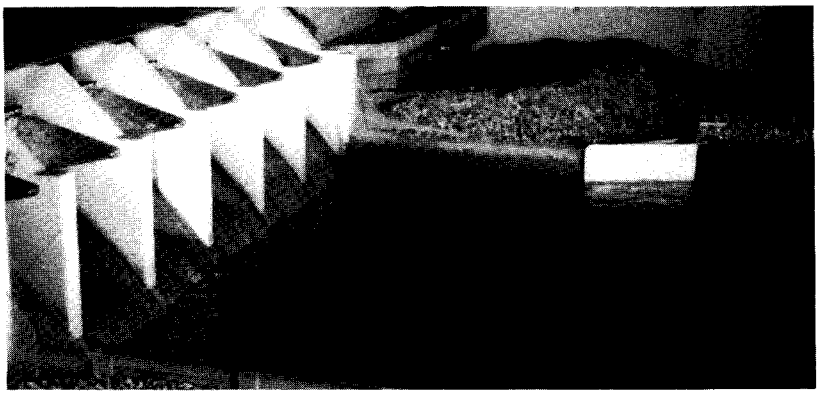
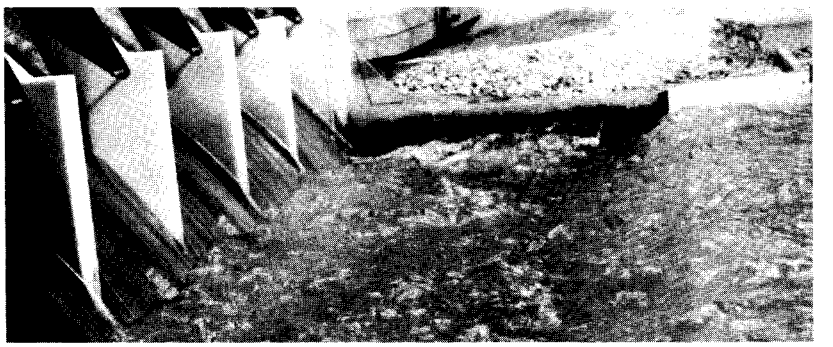


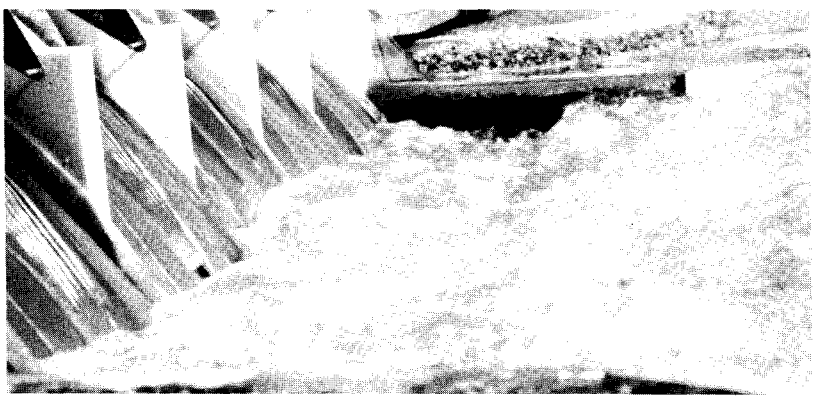
Figure 14. – Discharge – tailwater elevation.



(a) Tailwater El. 3225 feet (983.0 m), Q = 30,000 ft^3/s (850 m^3/s). P801-D-81133.

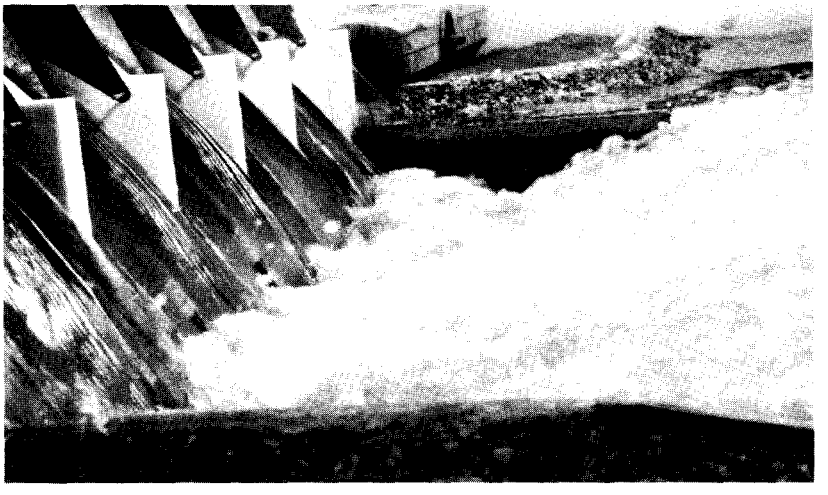


(b) Tailwater El. 3230 feet (984.5 m), Q = 100,000 ft^3/s (2832 m^3/s). P801-D-81134.

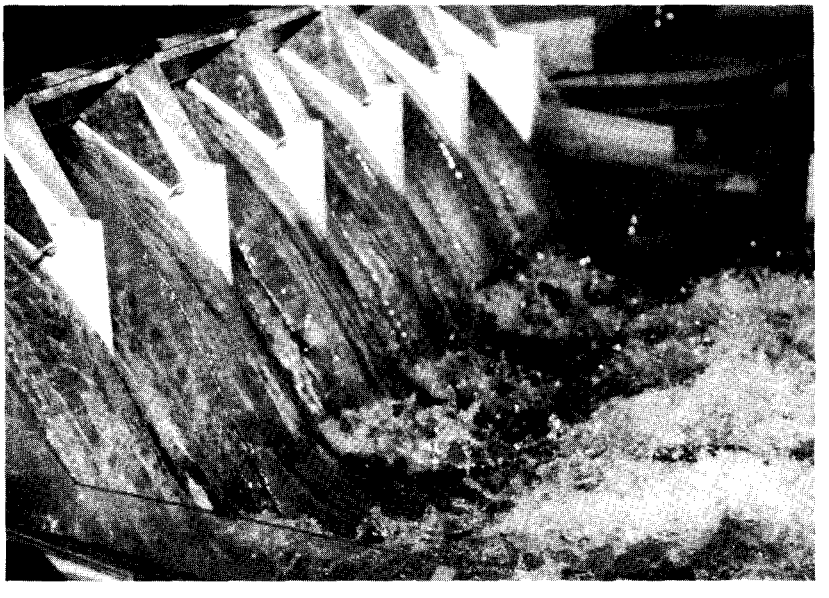


(c) Tailwater El. 3239 feet (987.2 m), Q = 250,000 ft^3/s (7079 m^3/s). P801-D-81135.

Figure 15. – Slotted bucket flows.



(a) Tailwater El. 3243 feet (988.5 m), Q = 342,000 ft^3/s (9684 m^3/s). P801-D-81136.



(b) "Sweepout", tailwater El. 3233 feet (985.4 m), Q = 345,000 ft^3/s (9769 m^3/s). P801-D-81137.

Figure 16. – Slotted bucket flows at maximum discharges.

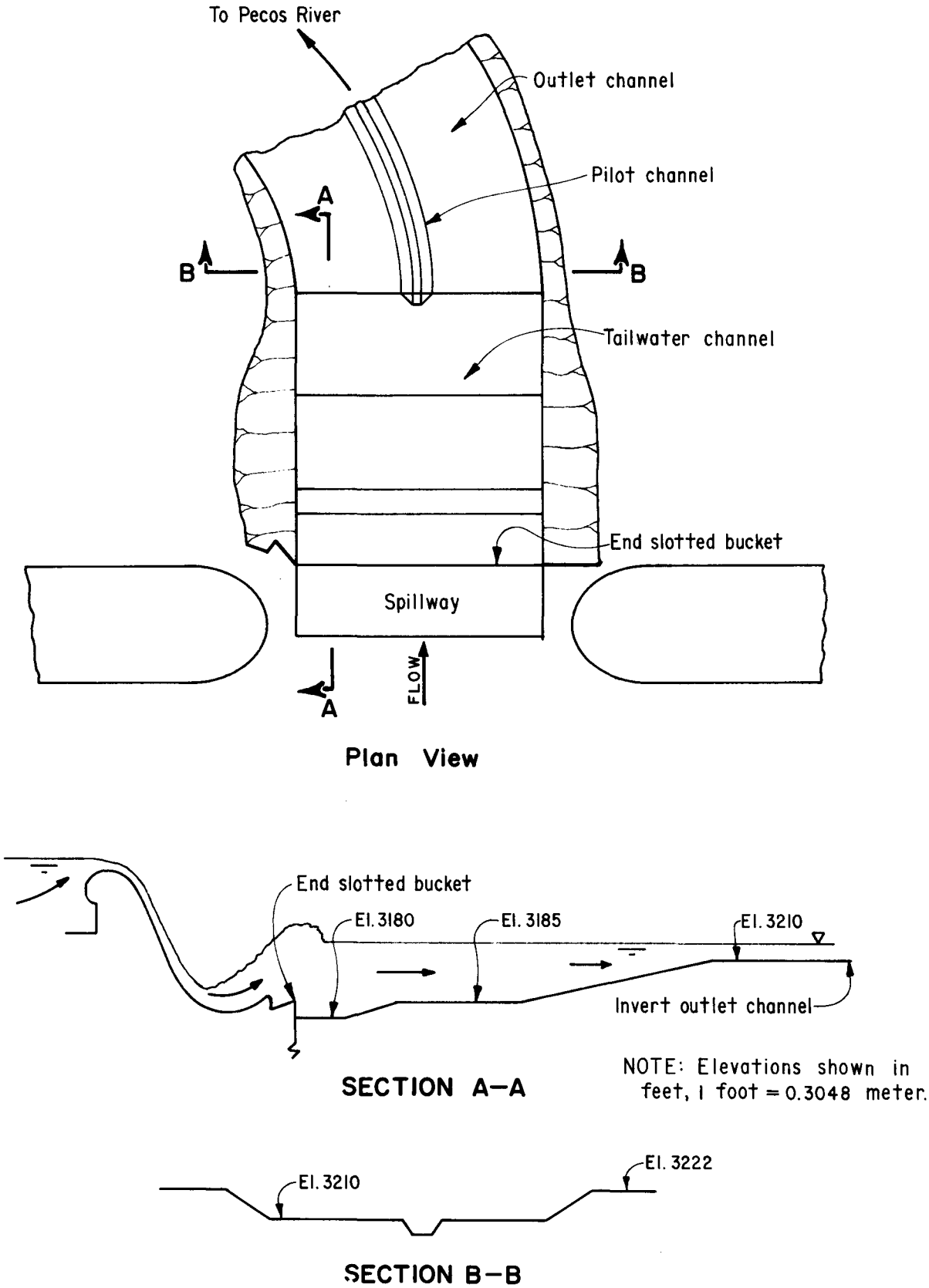


Figure 17. – Schematic of spillway tailwater channel and outlet channel – preliminary design.

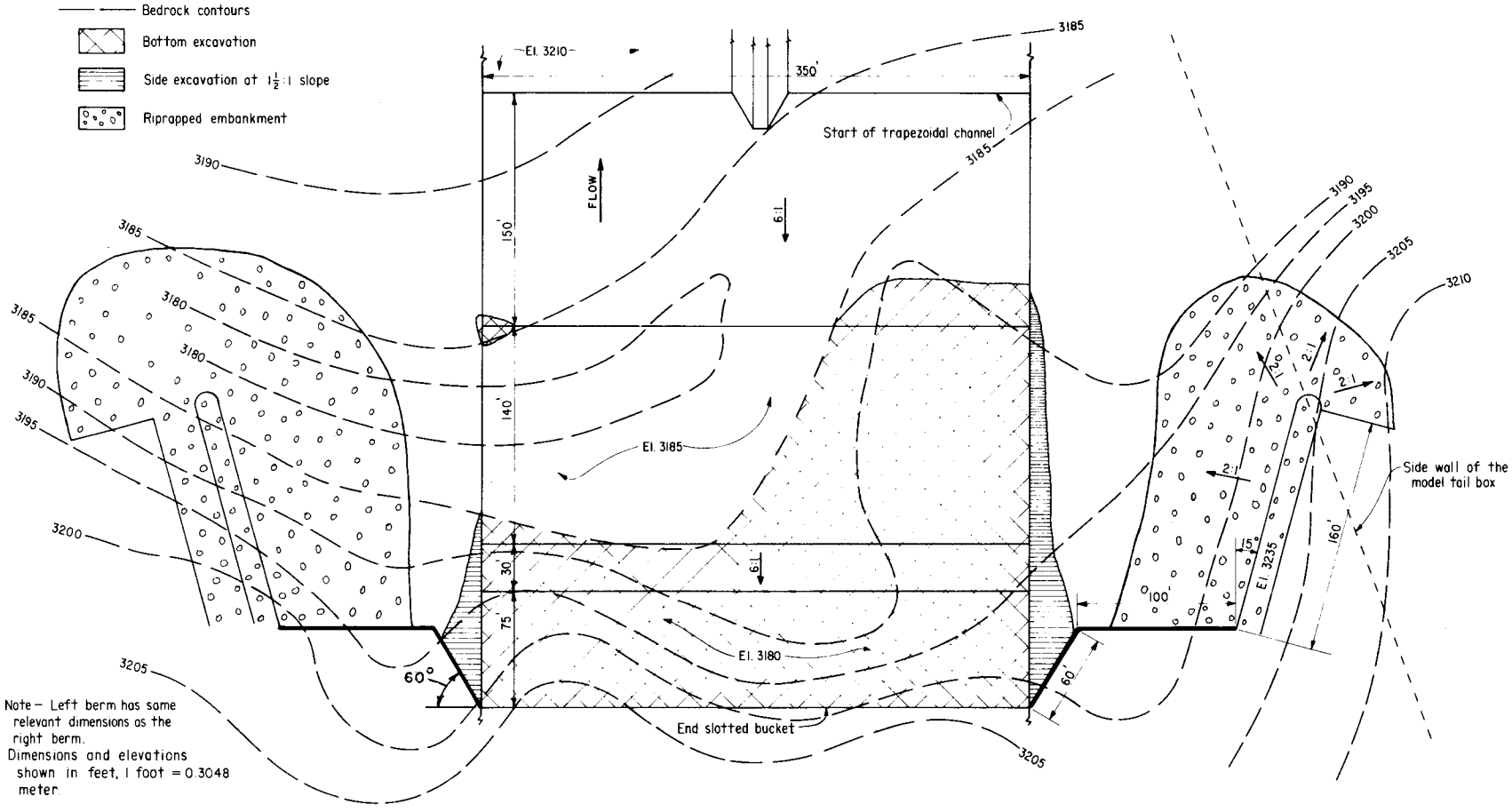
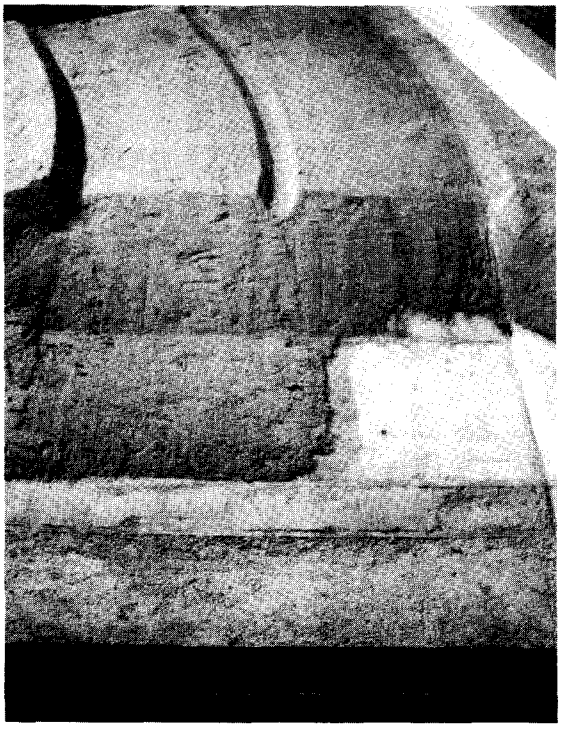


Figure 18. – Recommended design of spillway tailwater channel.



(a) Before. P801-D-81138.

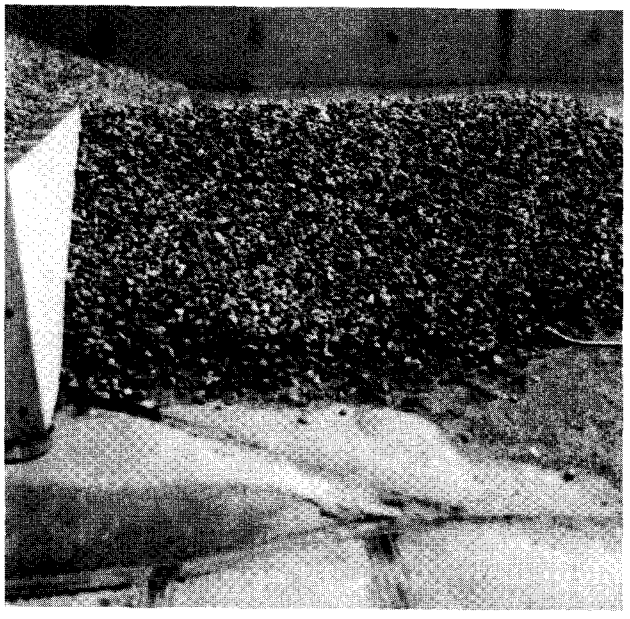
(b) After 50,000 ft^3/s (1416 m^3/s). P801-D-81139.

(c) After 186,000 ft^3/s (5267 m^3/s). P801-D-81140.

(d) After 238,000 ft^3/s (6739 m^3/s). P801-D-81141.

Figure 19. – Erosion tests.

(a) After 342,000 ft^3/s (9684 m^3/s). P801-D-81142.



(b) Left berm after 342,000 ft^3/s (9684 m^3/s). P801-D-81143.

Figure 20. – Erosion tests at maximum discharge.

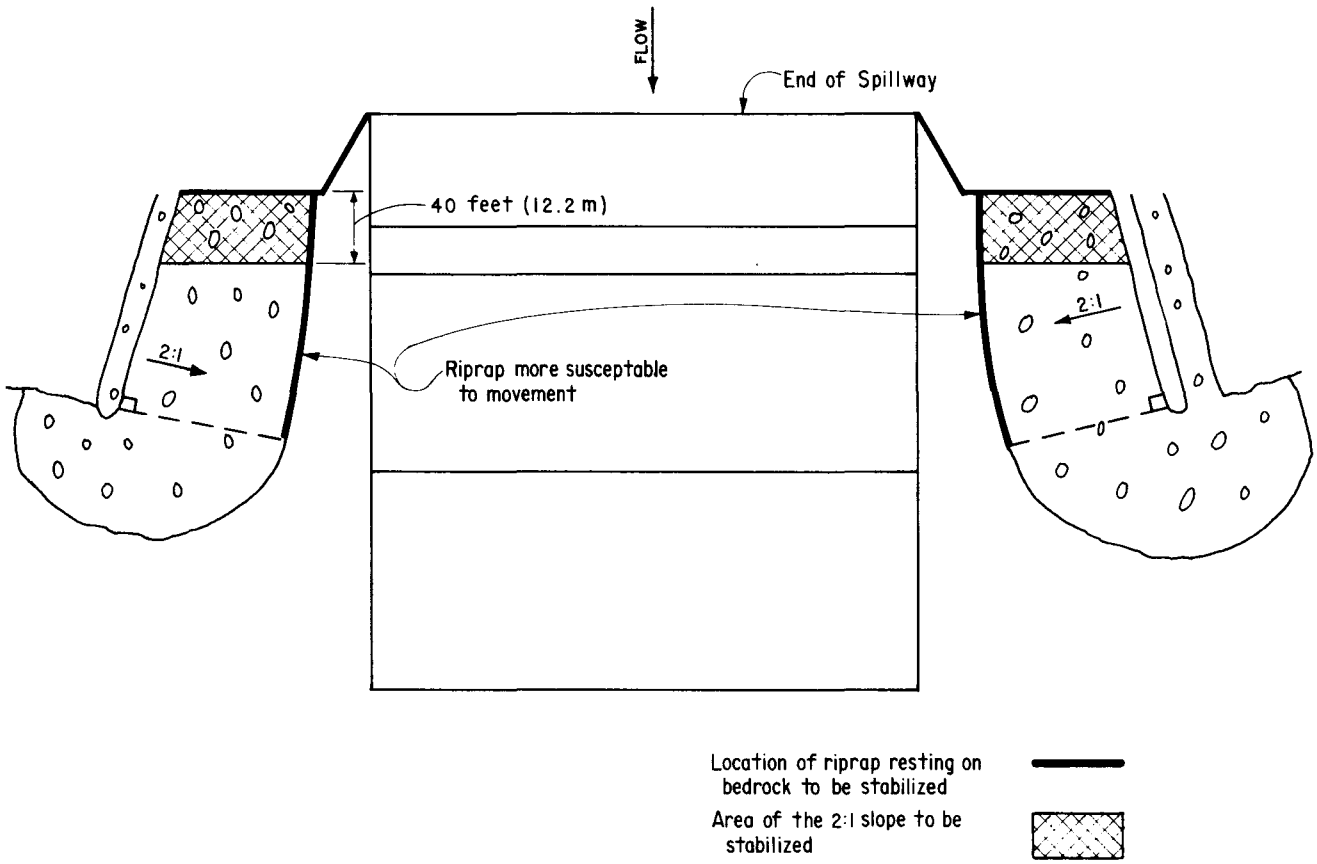
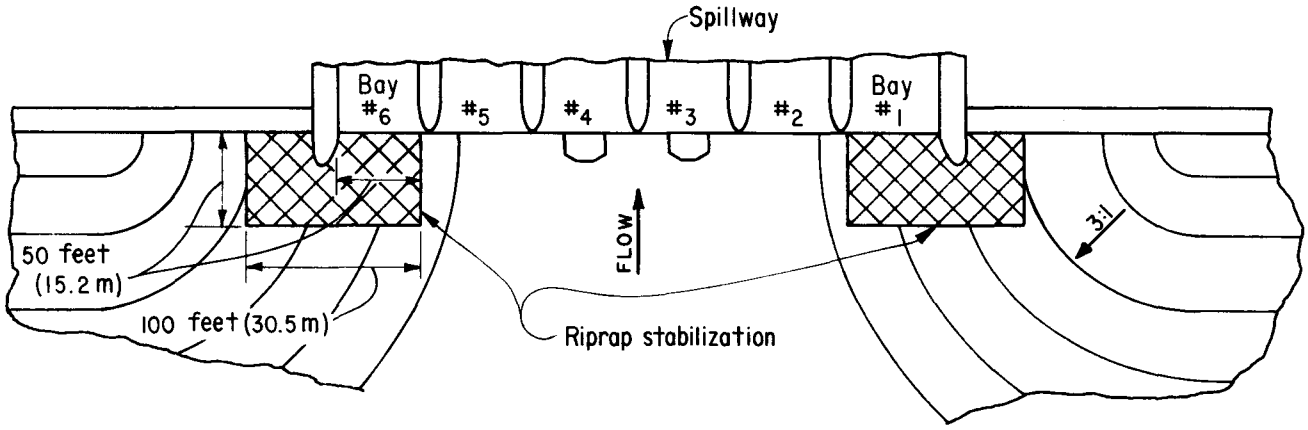
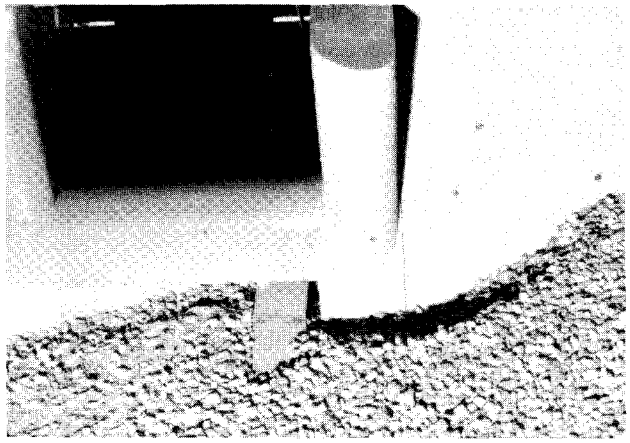


Figure 21. – Riprap stabilization for berms of the spillway tailwater channel.



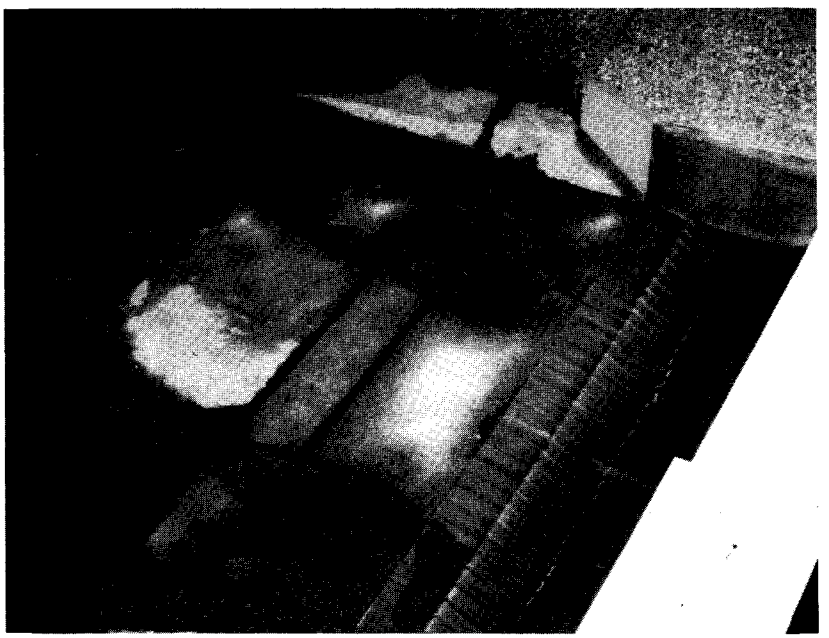
(a) Area needing stabilization.



(b) Erosion of model riprap. P801-D-81144.

Figure 22. – Riprap stabilization upstream of spillway.

(a) After operation of gates No. 1 and 6. P801-D-81145.



(b) After operation of gates No. 3 and 4. P801-D-81146.

Figure 23. – Erosion with two-gate operation.

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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-822A, P O Box 25007, Denver Federal Center, Denver CO 80225-0007.