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HYDRAULIC MODEL STUDY OF HORSESHOE DAM FUSE PLUG AUXILIARY SPILLWAY

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16. ABSTRACT

The Bureau of Reclamation conducted a 1 to 60 scale hydraulic model study of the proposed fuse plug auxiliary spillway for Horseshoe Dam, Arizona. The study was conducted to determine the discharge capacity of the spillway and investigate erosion of the spillway channel and appurtenant structures during spillway operation. The spillway channel will use a tapered, partial excavation. Operation of the spillway will cause erosion that enlarges the spillway channel. Dynamic erosion testing in the model showed that erosion will be sufficient to develop the required design capacity of the spillway channel, based on assumptions and available geologic data. Erosion testing also showed that a training dike constructed of materials excavated from the spillway channel would direct auxiliary spillway flows downstream, away from the area of Horseshoe Dam and the existing gated service spillway. The dike will also prevent auxiliary spillway flows from endangering the relocated dam tender's facilities.

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PURPOSE

This hydraulic model study was conducted to investigate proposed designs for the new fuse plug-controlled auxiliary spillway at Horseshoe Dam, Arizona. Specific objectives of the study included:

- Establishing the discharge capacity of the spillway and optimizing the intake channel geometry to make efficient use of the auxiliary spillway crest structure
- Investigating details of the splitter wall and fuse plug embankment designs that will affect the reliability of spillway operation and the integrity of the embankments during spillway operation
- Confirming the viability of a narrowed spillway channel excavation that will be enlarged by erosion during spillway operation
- Understanding the dynamics of erosion of the training dike and spillway channel and determining modifications required to prevent auxiliary spillway flows from eroding a channel that would lead back toward Horseshoe Dam

INTRODUCTION

Horseshoe Dam is located on the Verde River 58 miles north of Phoenix, Arizona. It is the most upstream of the two dams on the Verde River, lying about 20 river miles upstream of Bartlett Dam. The dam is a zoned earth and rockfill embankment with a crest length of 1140 ft, crest elevation of 2044 ft, and a structural height of 194 ft. The Salt River Valley Water Users Association constructed the dam between 1944 and 1946; the city of Phoenix added radial gates to the service spillway in 1949 to increase the storage available for domestic water supply. The SRP (Salt River Project) operates the dam. Elevation 2026.0 ft is the top of the active conservation pool. The current maximum reservoir water surface elevation is 2035.5 ft, at which the existing service spillway capacity is 240,000 ft^3/s. Figure 1 shows the dam, service spillway, and new fuse plug auxiliary spillway to be constructed at the site.

Dam Safety Modifications

Hydrologic studies have shown that the PMF (probable maximum flood) at Horseshoe Dam is a winter rain-on-snow event with a peak inflow of 562,000 ft^3/s and a 10-day volume of 2.4 million acre-feet. This flood would cause overtopping of the dam for almost 40 hours, with a maximum depth of over 12 ft, and would lead to catastrophic failure of the dam. To prevent the failure of the dam, a fuse plug-controlled auxiliary spillway will be constructed to the west of the dam to pass excess flood flows. The surcharge storage in the reservoir also will be increased by raising the crest of the dam 8 ft and constructing a 3-ft high parapet wall on the crest. A small closure dike also will be constructed just west of the auxiliary spillway control structure. The closure dike will fill the remainder of the saddle in which the auxiliary spillway will be located.

Following these modifications, the maximum reservoir water surface elevation will be 2052.0 ft. This will increase the maximum discharge capacity of the service spillway to 318,000 ft^3/s and provide increased surcharge storage in the reservoir. The auxiliary spillway will pass flows in excess of the service spillway capacity. A three-dimensional hydraulic model study of the

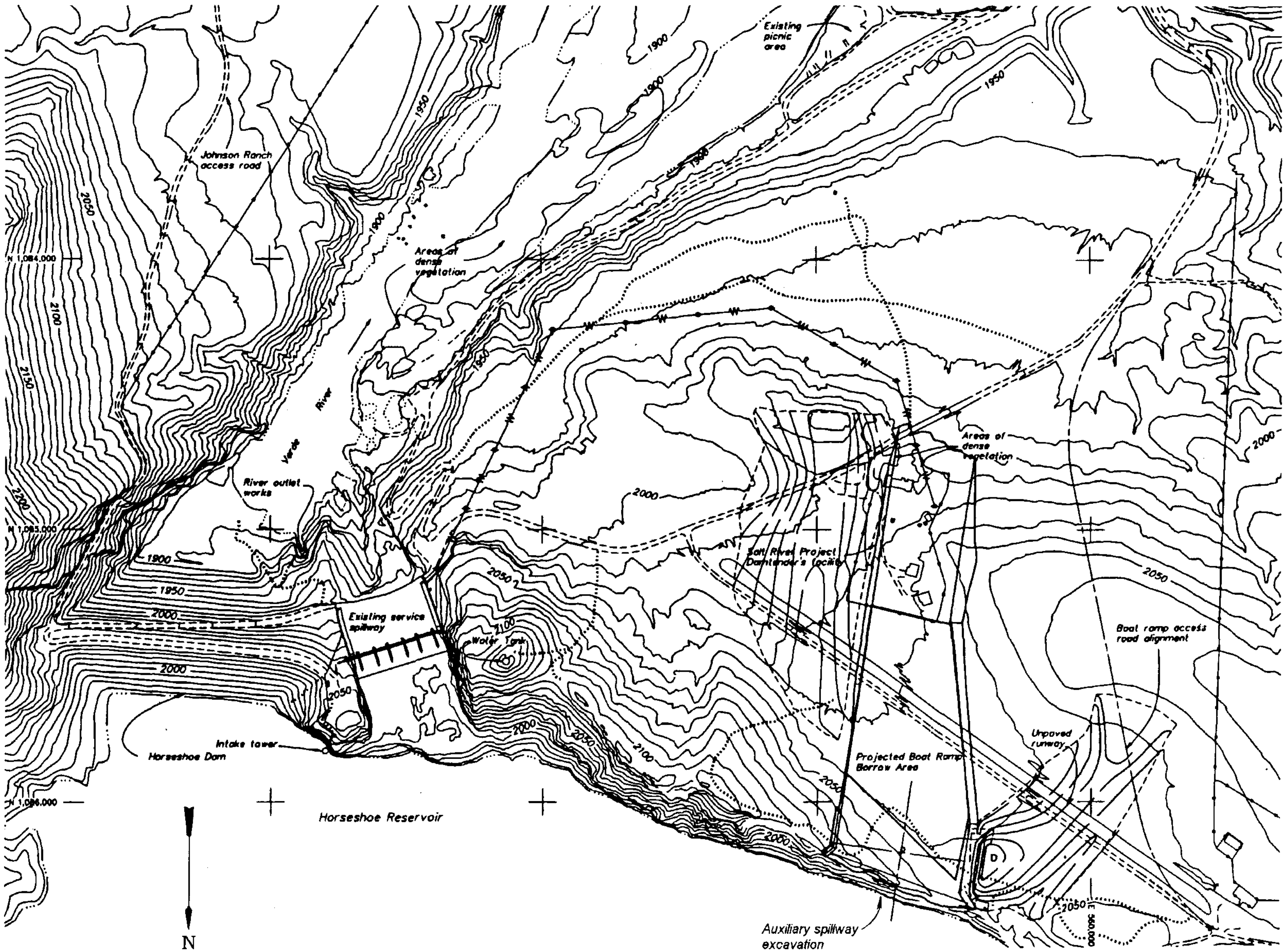

Figure 1. – Horseshoe Dam, the existing service spillway, and the excavation for the proposed auxiliary spillway.

service spillway was conducted to determine its discharge capacity and resolve issues related to the increased discharge and higher head (Wittler, 1993).

Reclamation (Bureau of Reclamation) reevaluated the seismic loading for Horseshoe Dam in 1989. This investigation identified a potentially active fault—the Horseshoe Reservoir Fault—within 1,000 ft of the dam. The maximum credible earthquake for the site is now a magnitude 7.0 event (Richter scale). To ensure the safety of the dam, a stability berm will be constructed at the toe of the dam using materials excavated from the auxiliary spillway channel. The service spillway gates also will be strengthened to resist earthquake loading.

Auxiliary Spillway

The auxiliary spillway will be a trapezoidal channel excavated through rock and alluvial materials on a S8°30′–W heading. The spillway will be located west of Horseshoe Dam, daylighting into the reservoir through a steep cliff on the southern rim of the reservoir. Flow will be from north to south. The crest length was reduced during the model study from 575 to 470 ft as the result of improvements to the discharge capacities of both the service and auxiliary spillways. The right side of the spillway (looking downstream) will be tapered downstream of the crest structure to a final downstream width of 290 ft. Channel side slopes will be 1.5 to 1 (horizontal:vertical), except near the crest where the left side will be excavated at 0.25h to 1v with benches at 40-ft vertical intervals, and the right side will be excavated at 1 to 1. The channel slope will be 0.013 throughout the length of the spillway. The crest structure will be a 1-ft high sill block set at elevation 2021.0 ft, 5 feet below the conservation pool level. A conventional concrete apron will be constructed downstream of the sill block. The total streamwise length of the crest structure—as proposed at the beginning of the hydraulic model study—is 17 ft.

To preserve the conservation storage capacity of the reservoir, three fuse plug embankments—separated by concrete splitter walls—will control the spillway. A fuse plug embankment is designed to function as an embankment dam until the reservoir reaches a predetermined elevation. Once this elevation is reached, the embankment is breached in a controlled manner, thereby opening the spillway channel (Pugh, 1985). Fuse plug embankments have been used extensively in recent years for the control of spillways that operate infrequently, where the cost of a gate-controlled spillway cannot be justified.

The fuse plug embankments for this spillway will be staged to operate at 3-ft increments of reservoir elevation, with the first embankment designed to breach at reservoir elevation 2042 ft. Figure 2 shows the basic configuration of the crest structure and fuse plug embankments.

A training dike will be constructed on the left (east) side of the auxiliary spillway channel using material excavated from the spillway. This dike will direct the auxiliary spillway flow to the south so that the spillway discharges into the Verde River well downstream of the dam and service spillway. The dike also will keep auxiliary spillway flows away from the newly relocated dam tender's facilities. The dike will be sacrificial and will not be protected from erosion.

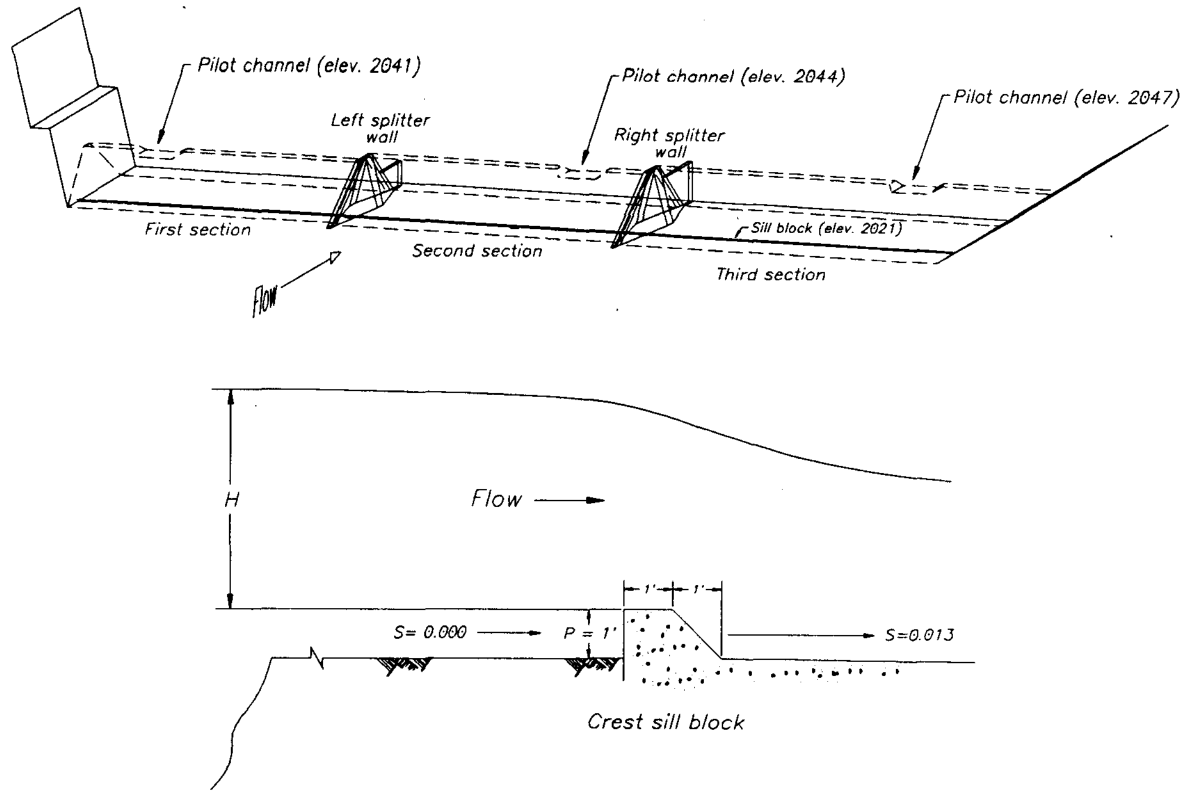

Figure 2. – Basic configuration of the auxiliary spillway crest structure and fuse plug embankments. The embankments are shown with dashed lines. A section of the 1-ft high sill block is shown in the lower view.

RESULTS AND RECOMMENDATIONS

Discharge Capacity

Physical hydraulic model studies showed increases in discharge capacity over assumed design values for both the service spillway (Wittler, 1993), and the auxiliary spillway. This made it possible to reduce the auxiliary spillway crest length from 575 to 470 ft.

The maximum capacity of the recommended auxiliary spillway design, based on the results of the model tests, is 245,800 ft^3/s at reservoir elevation 2052.0 ft.

Model test data were used to develop discharge equations for the auxiliary spillway of the form:

$$Q = CLH^{3/2}$$

where:
- Q = discharge, ft^3/s
- L = nominal crest length, ft
- H = head, ft (measured above the crest sill block elevation)
- C = discharge coefficient

The parameters of the discharge equation for the three phases of spillway operation are shown in table 1.

Table 1. – Auxiliary spillway discharge equation parameters.

Spillway operation	Nominal crest length, L (ft)	Discharge coefficient, C
One section open	145	2.92
Two sections open	300	2.88
Three sections open	470	3.03

The model tests indicated that lateral erosion of the training dike and degradation of the bed will open the downstream reach of the spillway channel sufficiently to pass the maximum discharge during the PMF. Erosion opens the channel sufficiently to prevent any suppression of flow at the spillway crest.

A hydraulic jump can be established downstream of the crest structure if a scour hole forms on the downstream side of the crest sill block. (This study did not attempt to determine whether a scour hole will form.) However, the model tests showed that the jump will not reduce the discharge capacity of the crest. Erosion of the downstream channel prevents the buildup of tailwater downstream of the crest structure, thus preventing the hydraulic jump from moving upstream and suppressing flow at the crest structure.

Tests performed with a nonerodible spillway channel showed that if no erosion occurs in the spillway channel, the maximum spillway capacity would be reduced by about 12 percent. Subsequent testing with an erodible spillway channel confirmed that the assumption of minimal erosion is unreasonable.

Training Dike

The training dike configuration tested in the second dynamic erosion test will maintain the auxiliary spillway flow in a southerly direction through the full range of expected spillway discharges. The dike will prevent the flow from turning back toward the dam tender's facilities, service spillway, and toe of dam area. The erosion test showed that during the PMF a stable channel will be established prior to reaching the peak spillway discharge, and the dike will not be overtopped.

The upstream reach of the dike can be lowered to elevation 2035 ft (fig. 25). This may allow additional flexibility in locating the SRP access road at the site.

Fuse Plug Pilot Channel Elevations

No significant drawdown from the reservoir pool elevation could be detected at either the second or third fuse plug pilot channels, when one or two spillway sections were open. To obtain the 1-ft overtopping head assumed necessary to breach the fuse plugs at reservoir elevations of 2042, 2045, and 2048 ft, the pilot channel inverts should be located at elevations 2041, 2044, and 2047 ft, respectively.

Approach Channel

Significant flow contractions in the approach channel were detected in initial testing of the proposed 575-ft crest length spillway. Entrances beveled 40° away from the spillway centerline were recommended and tested when the spillway crest length was reduced to 470 ft. The beveled entrances improved the maximum discharge capacity by about 3.5 percent.

As flow approaches the auxiliary spillway crest, the Froude number varies from about 0.5 to 1.0. The flow passes through critical depth at the sill block.

Splitter Walls

The concrete splitter walls separating the three fuse plug embankments present minimal obstruction to the flow due to their streamlined shape.

The downstream training walls (on the tail of each splitter wall) prevented the flow from attacking the downstream face of adjacent embankments. The height of the downstream training walls can be reduced from the original design (El. 2041 ft). The first wall (separating the first and second embankment sections) can be reduced to elevation 2033 ft, and the second wall can be reduced to elevation 2035 ft. These elevations allow 4 ft of freeboard above the mean water surface profiles observed in the model.

The 5-ft vertical height upstream training walls effectively reduce the velocities on the upstream faces of the fuse plug embankments during the operation of an adjacent spillway section. The maximum velocity along the second embankment (during operation of the first section) was about 6.6 ft/s and the maximum velocity along the third embankment (during operation of the first two sections) was 7.5 ft/s.

MODEL CONSTRUCTION AND TEST PROCEDURES

A 1 to 60 Froude scale model of the auxiliary spillway area was constructed in the hydraulics laboratory during the fall of 1991. Testing was conducted from January through May 1992. The model, shown in figure 3, included the reservoir, approach channel, crest structure, and downstream topography. The model covered a 1620- by 2250-ft area of the prototype, as outlined on figure 4. The downstream portion of the model was a composite construction designed to model both erodible and nonerodible topography. Data obtained from the field investigation of the site were used to estimate the level to which the downstream channel would be erodible. Nonerodible topography was constructed with concrete and wire mesh fabric placed over plywood contours. Erodible materials were placed over the top of the rock topography up to the ground surface elevation of the proposed spillway channel.

Each fuse plug embankment was constructed in three solid subsections (fig. 3), sized to have approximately equal flow areas. These sections were easily inserted and removed during testing. The solid sections were used because the model's 1 to 60 scale would not have permitted accurate simulation of the fuse plug breaching process. The use of solid sections also eliminated the need for tedious construction and reconstruction of the model embankments.

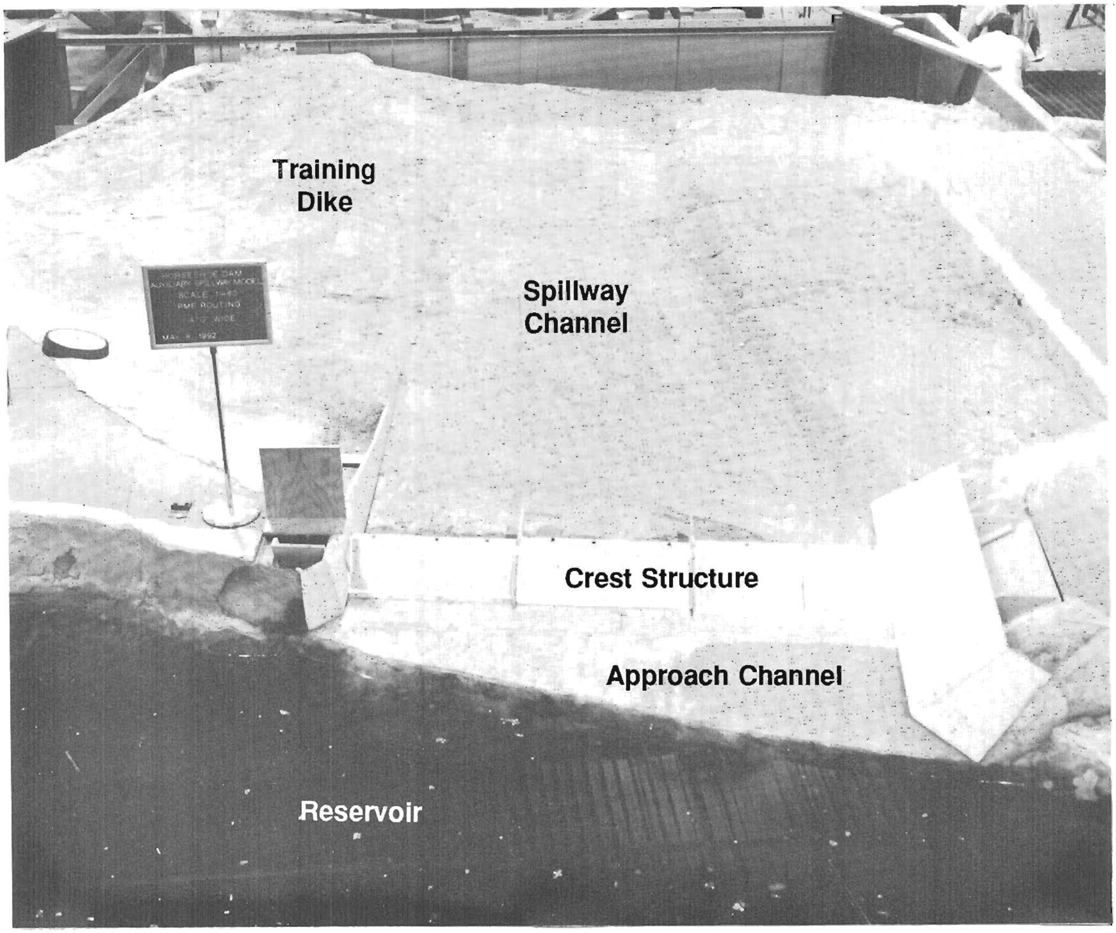

Figure 3. – Scale model (1 to 60) of Horseshoe Dam Auxiliary Spillway, in the Bureau of Reclamation Hydraulics Laboratory, Denver, Colorado.

Site Geology and Topography

The auxiliary spillway will be located about 2500 ft west of the dam. The spillway channel will convey the flow in a southerly direction through a small saddle south of the present dam tender's facilities. Auxiliary spillway flows will eventually enter the Verde River about 3000 ft downstream of the dam.

Three major subsurface geologic features at the site are important in the design and operation of the auxiliary spillway: (1) the rock area at the upstream end of the spillway (south rim of the reservoir), (2) a buried, ancient channel of the Verde River crossing through the midsection of the spillway channel, and (3) a block of uplifted lakebed deposits in the right downstream portion of the spillway. The approximate locations of these features are shown on figure 4.

The upstream reach of the spillway channel will be excavated through rock—primarily andesite. On the left side of the cut, the rock is high quality and extends from the reservoir to about 400 ft downstream of the crest structure. The quality and downstream extent of rock decrease as one moves to the right side of the spillway. The right edge of the cut will daylight out of rock about 200 ft downstream of the crest structure.

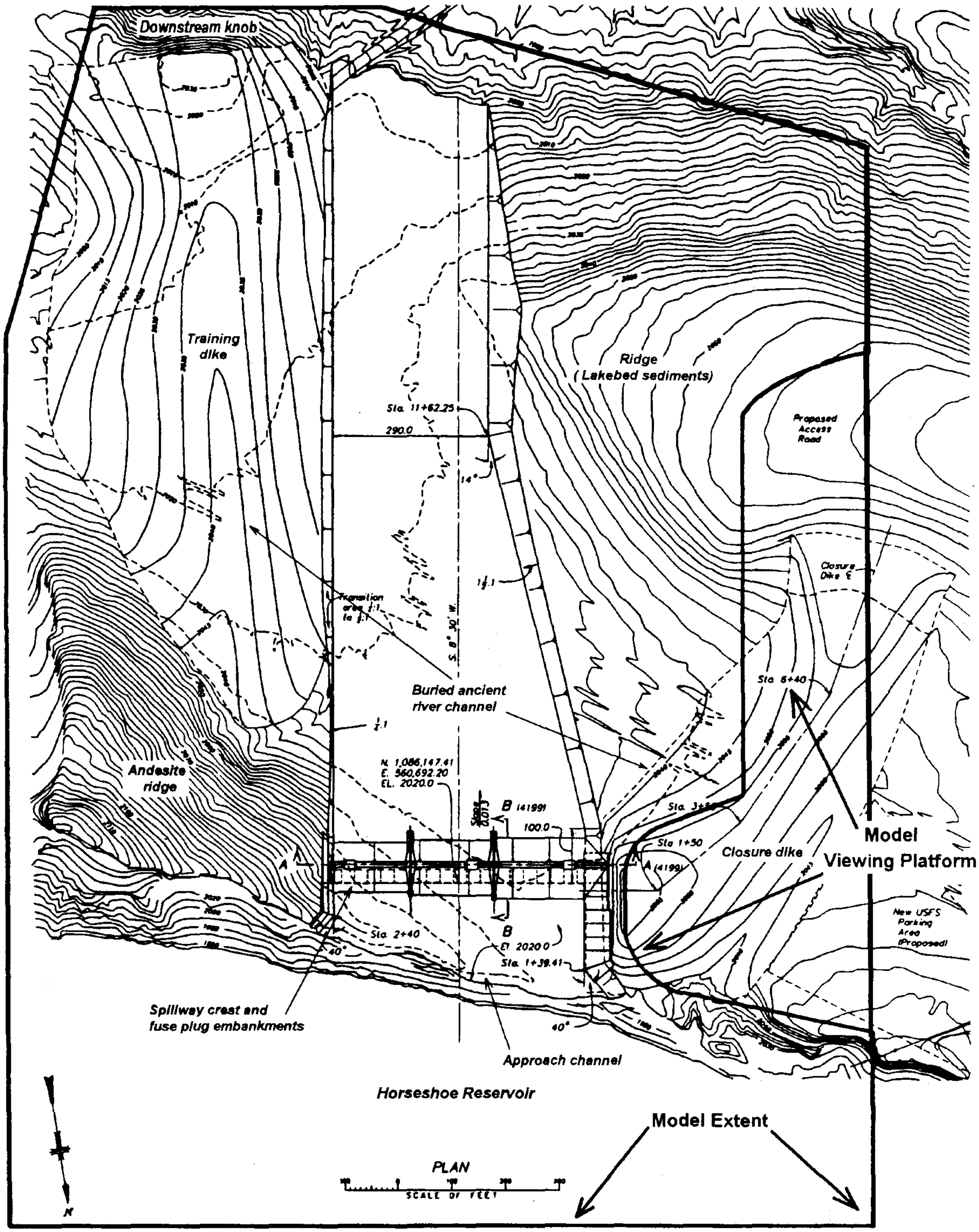

Figure 4. – Plan view of auxiliary spillway site and extent of hydraulic model (470-ft modified spillway).

The midsection of the spillway channel is excavated through a buried, ancient channel of the Verde River that is aligned roughly from west-northwest to east-southeast, along the line of the airstrip currently at the site. The ancient channel is filled with alluvial deposits that would be highly erodible during spillway operation. These deposits were modeled with a noncohesive sand mixture.

The ridge at the downstream right side of the spillway channel consists primarily of uplifted lakebed sediments. These sediments vary in quality, but should be somewhat erodible during spillway operation. The weathered, upper 10 ft of these sediments were considered erodible and were modeled with the same noncohesive sand used for the bed of the spillway channel. Below the 10-foot depth lakebed sediments were considered nonerodible and were modeled using concrete mortar. The assumptions regarding the erodibility of the lakebed deposits are conservative. The concrete construction limited the widening of the right side of the channel. This probably caused increased erosion of the training dike on the left side of the channel.

One additional surface feature at the site affects the spillway operation. At the downstream left end of the spillway channel, a small knob divides the exit of the spillway channel from a natural swale leading back to the service spillway and toe-of-dam area. Geologic data suggest that the knob is made up of lakebed sediments and good quality conglomerates. A portion of the knob also may consist of waste material from the original construction of the dam and service spillway. This knob was nonerodible in the model.

The erodible materials in the spillway channel area fall into two basic categories (Materials Investigation Report, 1991). An upper horizon, about 8 ft thick, in which 94 percent of the material is smaller than 3-in diameter, with about 37 percent mostly plastic fines. The lower horizon, from 8 to 18 ft depth, contains about 6 percent nonplastic fines, and 80 percent of the material is less than 3-in diameter. The depth of the spillway cut will place the spillway invert in the lower horizon materials.

Model Materials

The erodible materials used in the model were scaled to match the prototype materials in the lower horizon. Scaling was done using Froude criteria, with an adjustment for settling velocity (Pugh, 1985). The resulting gradation was well graded, and could not be matched exactly. The materials used in the model were less well graded, but had about the same d_{50} as the desired gradation, as shown on figure 5. The use of poorer graded materials in the model will cause slightly faster erosion than in the prototype due to reduced apparent cohesion by mechanical interlocking.

The erodible materials were placed in the model in a damp condition and compacted using a vibratory compactor. Materials were placed to the proper ground surface elevation by adding successive lifts until the ground surface matched a series of templates suspended over the spillway channel from the edges of the model box.

Test Procedures

The model was tested under three different spillway channel conditions. The three configurations are described in the sections that follow.

Nonerodible channel configuration. — Before performing the erosion tests (described below), a thin concrete cap was constructed over the erodible ground-surface topography to create a nonerodible spillway channel (575-ft crest length). Testing was done in this condition to determine the discharge capacity of the spillway under the assumption that minimal erosion would occur in the downstream portion of the channel. These tests provided a better understanding of the extent to which erosion is required to obtain the desired spillway capacity.

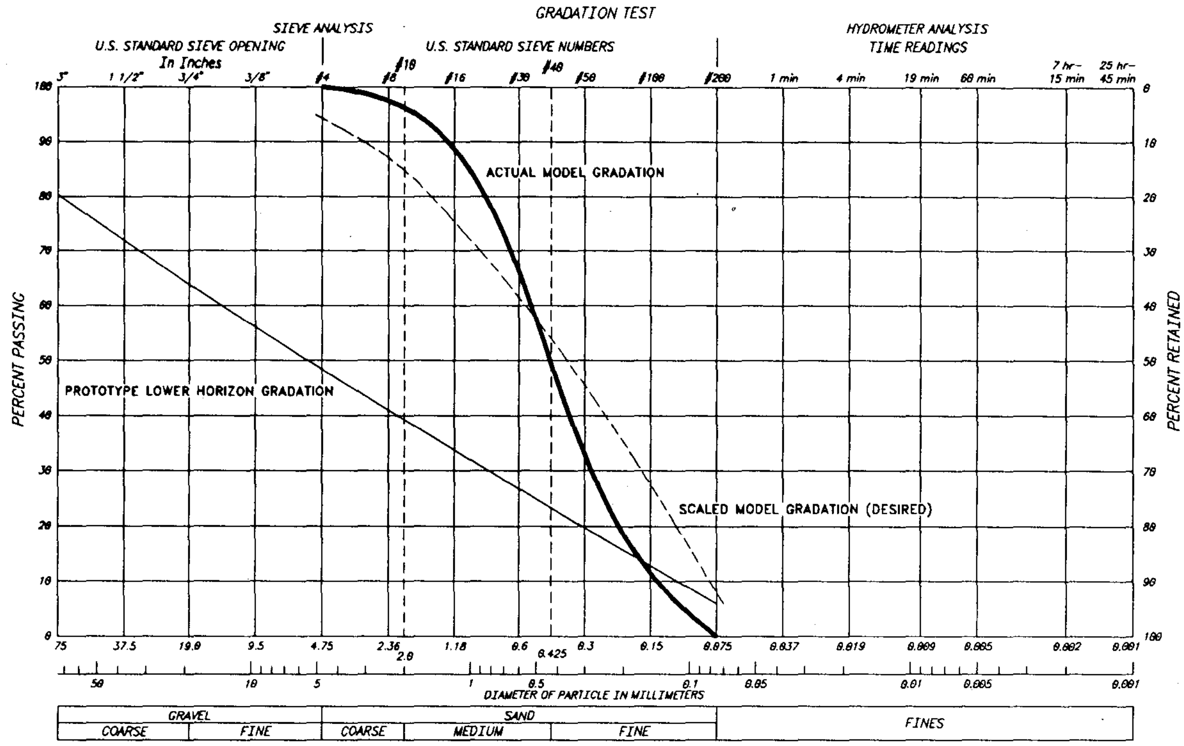

Figure 5. – Prototype and model erodible materials.

Dynamic erosion tests.—Two tests were run with erodible topography in the spillway channel. The objective of these tests was to identify the relevant hydraulic and dynamic aspects of the erosion process and to assess the performance of the training dike on the left side of the spillway channel.

The first test was run for the originally proposed 575-ft crest length spillway, the second for the modified 470-ft crest length spillway. Model inflows were scaled according to Froude criteria. Froude scaling yields a time scale condensed by the square root of the length scale of the model. Thus, 31-hours operation of the prototype spillway was reduced to about 4 hours of model operation.

The total inflow to the model for these tests was determined by routing the PMF through the reservoir and setting the model inflow equal to the sum of the auxiliary spillway flow and the additional inflow required to obtain the proper rate of reservoir rise in the model. A computer using a combination of electronic spreadsheet and process control software controlled the operation of the model. The spreadsheet computed the flow into the model as a function of the elapsed time, and sent the result to the process control software. The process control software then transmitted the required discharge to a closed-loop controller connected to the laboratory venturi meters. The controller operated valves downstream of the venturis to maintain the required discharge. Venturi meters ranging from 4- to 12-in diameter were used for the tests.

As the test proceeded, the solid fuse plug embankment sections were physically removed from the model at the appropriate times, depending on their pilot channel elevations and computed lateral erosion rates. As each embankment subsection was removed, an appropriate quantity of sand was added at the crest location to simulate the fuse plug embankment materials that must be carried downstream through the spillway channel.

Each of these tests was videotaped from two locations in the laboratory, and still photos were taken before, during, and after the tests. Markers were placed on the training dike to indicate the progress of lateral erosion into the dike.

Fuse plug breaching characteristics.—The breaching characteristics of the fuse plug embankments, which were simulated in these tests by timed removal of the solid embankment subsections, were determined using the results of extensive research conducted by Reclamation in the early 1980s (Pugh, 1985). The operation of a typical fuse plug embankment consists of two linked processes. The first is termed *initiation of breach*; flow through the pilot channel (refer to fig. 2) quickly cuts down into the fuse plug embankment, washing away the downstream shell material. This removes the support for the inclined core section, which behaves structurally as a cantilevered beam. As the downstream shell is eroded away, the core repeatedly breaks off, deepening the pilot channel section and further accelerating the breaching process. The second stage of operation is the *lateral erosion* phase. When the pilot channel section has been fully eroded down to the base of the embankment, the embankment then begins to erode laterally. This lateral erosion process continues until the embankment has been completely washed out.

Reclamation's past research has shown that both the initiation of breach and lateral erosion processes are quite predictable in a properly designed and constructed fuse plug. In Reclamation's studies, the breach of a model fuse plug in the laboratory was brought about by rapidly bringing the reservoir level up to a prototype elevation 1 ft above the invert of the pilot channel. This level was then maintained throughout the remainder of the test. Reviewing the videotaped records of these tests showed that about 10 to 15 minutes would be required for the initiation of breach process on the fuse plugs proposed for the Horseshoe Dam Auxiliary Spillway.

The results of the past research, however, only provide a qualitative estimate of the time required for the initiation of breach process. Differences between the proposed design for Horseshoe Dam and the designs tested in Reclamation's research include:

- wider pilot channels to improve passage of debris,
- a deeper layer of gravel surfacing and filter materials in the pilot channel invert to provide additional weather protection for the impervious core, and
- a steeper downstream slope of the embankment.

The wider pilot channels and steeper downstream slope will tend to produce a faster breach; the effect of the additional core protection in the pilot channel invert is uncertain. Also, since the reservoir level in the prototype will rise gradually—as opposed to the rapid rise to the 1-ft overtopping level used in Reclamation's research—there will be a significant time period in which there is flow through the pilot channel, but at a depth less than 1 ft. Rates of reservoir rise during the relevant floods at the site will be in the range of 1 to 3 ft/h. Thus, 20 to 60 minutes may be required for the reservoir to rise from the pilot channel invert to a level 1 ft above the invert. All of these factors make it impossible to exactly predict the timing of the initiation of breach process.

In addition, flood routings performed during the design of the spillway showed that the flood outflows from Horseshoe Reservoir were insensitive to even large differences in the assumed initiation of breach characteristics. Thus, to simplify the conduct of these model tests, the initiation of breach process was assumed to be complete when the reservoir reached a prototype level 1 ft above the pilot channel invert. At this point, the fuse plug subsections were removed at a rate corresponding to the estimated lateral erosion rate of the embankment.

The lateral erosion rate of the fuse plug embankments was estimated using the results of Reclamation's previous research. The steep downstream slope—and thus reduced volume of the downstream shells—in this design produces relatively high lateral erosion rates, ranging from 425 to 500 ft/h. The flood routing analyses also indicated that reservoir outflows were insensitive to large variations in the lateral erosion rate.

Posterosion channel configuration.—Operation of the model with erodible materials in place (described above) confirmed that erosion of the spillway channel would be extensive during any operation of the spillway. Furthermore, the tests showed that erosion was sufficient to prevent any shifting of hydraulic control along the length of the spillway channel; hydraulic control was maintained at the crest structure throughout the erosion tests. Thus, to simplify testing, the development of discharge curves and the testing of details of the approach channel, crest structure, and splitter wall designs were conducted with the channel in a posterosion condition. This limited the contamination of the laboratory water supply and eliminated labor required to repeatedly reconstruct the erodible topography.

CREST STRUCTURE INVESTIGATION

Discharge Capacity

Initial discharge capacity testing was done for a 575-ft crest length spillway with successive fuse plug section lengths of 150, 175, and 250 ft. The left side of the crest structure was excavated on a 0.25h to 1v slope, and the right side was excavated on a 1.5 to 1 slope. These tests showed a significant contraction of the flow at the left entrance to the spillway channel. Preliminary testing indicated that rounding or beveling of this corner could increase the maximum discharge capacity of the spillway by about 2.5 percent. A slight contraction of the flow also was apparent at the right entrance. Both corners were subsequently modified by beveling them 40° away from the spillway centerline.

The contraction off the right side of the spillway opening (the splitter walls) was significant with the first and second fuse plug sections open. The 5-ft high upstream training walls magnified this contraction. With the embankments on both sides of the splitter walls removed, the splitter walls presented little obstruction to the flow.

Another significant influence on the discharge capacity of the spillway was the presence of a region of reduced velocity along the centerline of the spillway with all three sections open. Flow boiling up from deep in the reservoir just in front of the entrance to the spillway channel caused the reduction. With all three sections open, the spillway draws significant flows from both sides of the spillway opening along the steep cliff face. This induces similar flows parallel to the cliff face at depth in the reservoir. When these flows meet at the spillway centerline they boil up to the surface. This caused an area of reduced velocity on the approach that was about 150 ft wide. No feasible solution for this condition could be determined.

Nonerodible channel tests.—When the model was operated with the concrete cap over the channel, the discharge capacities for operation of one or two sections were not significantly different from those determined later with the channel in the eroded condition. For operation of all three sections, the constriction of the channel at the downstream end (to one-half the crest length) caused a buildup of tailwater that reduced the capacity of the crest structure by about 12 percent.

Scour hole tests.—One factor that could reduce the discharge capacity of the spillway would be the formation of a scour hole immediately downstream of the crest. This is the most likely location for formation of a scour hole due to the acceleration of the flow and its downward velocity component as it passes over the crest and through critical depth. If the scour hole provides enough additional resistance, a hydraulic jump could be formed that would move upstream and submerge the crest section. This could cause a decrease in the discharge capacity of the spillway.

To determine if such a scenario is feasible in this spillway, a series of tests was performed to determine how the discharge capacity would be affected by various depths of scour holes downstream of the crest structure. A portion of the downstream apron was removed from the model to open a rectangular basin that began 17 ft downstream of the crest. This is the downstream edge of the crest structure in the original design. The basin extended to 95 ft downstream of the crest (fig. 6). The basin was then filled with 1/4-inch pea gravel and the flow was allowed to scour out a naturally shaped hole. This type of test was run three times with the ultimate scour depth limited to 4, 10, and 16 ft. In each case the flow removed the pea gravel down to the allowed ultimate scour depth, leaving pea gravel only in the corners of the rectangular basin. The tests were run with the downstream channel in the eroded condition, since some time would be required to form a scour hole in the prototype.

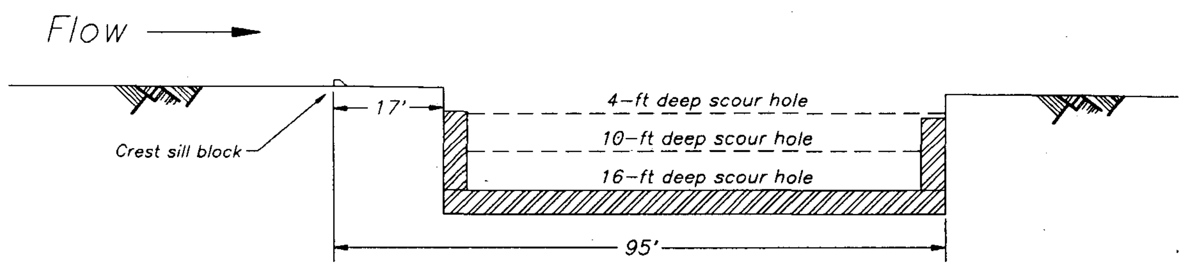

Figure 6. – Schematic of model arrangement for scour hole testing. Dimensions shown are for the prototype.

In each of the three tests a hydraulic jump formed in the scour hole. However, the jump did not move upstream to submerge the crest and thus it had no impact on the discharge capacity of the spillway. Because the downstream channel was in the eroded condition, as the flow exited the jump it quickly passed through critical depth again and flowed into the scoured out ancient river channel. This prevented the tailwater from ever becoming high enough to push the jump further upstream onto the crest.

Modified 470-ft spillway.—Following initial tests of both the service spillway model and the 575-ft crest length auxiliary spillway model, the auxiliary spillway was narrowed to a total crest length of 470 ft, with successive section lengths of 145, 155, and 170 ft. The downstream width of the spillway channel was maintained at 290 ft. At the same time, the crest sill block was moved about 27 ft downstream to provide increased area for construction access. (The new intersection of the spillway centerline and the downstream edge of the crest sill block was at Arizona State plane coordinates N. 1,086,147.41, E. 560,692.20.) The left sidewall of the crest structure was also modified; the concrete sidewall was placed on the excavated side slope, rather than in an overexcavated pocket. This caused the wall to project 18 inches out from the rock face. All of these changes were made in the hydraulic model.

Following the completion of testing, four other changes were made, but not included in the model.

- The crest sill block was moved 40 ft upstream from the position tested in the 470-ft crest length model to improve the foundation conditions under the right side of the crest. The model as tested is conservative with respect to this change, as this reduces the approach length and approach losses. (Final Arizona State plane coordinates of the spillway centerline at the downstream edge of the crest sill block are N. 1,086,187.0, E. 560,698.1)
- The right sidewall was steepened from 1.5h:1v to 1h:1v to simplify field excavation procedures.
- The right sidewall was also modified so that, like the left, the concrete pad will project into the flow rather than being flush with the right side slope.
- The side slopes of the two splitter walls were changed from 0.25h:1v to 0.3h:1v for structural stability reasons.

The latter three modifications reduce the open crest length of the spillway and will reduce the discharge capacity. Corrections for these three modifications have been made by determining the additional percentage of flow area occupied by the modified structures at the critical section (upstream edge of the crest sill block). The reduction in discharge capacity is 0.12 percent for the operation of one section, 0.20 percent for the operation of the first two sections, and 1.65 percent for the operation of all three sections.

The discharge curves for the 470-ft crest length spillway are shown on figure 7. The discharge equations are as follows:

For operation of the first section:

$$Q = 2.92\ (145)\ H^{3/2}$$

With the first two sections open:

$$Q = 2.88\ (300)\ H^{3/2}$$

With all three sections open:

$$Q = 3.03\ (470)\ H^{3/2}$$

where:

Q = discharge, ft^3/s

H = head (measured above the crest sill block elevation), ft

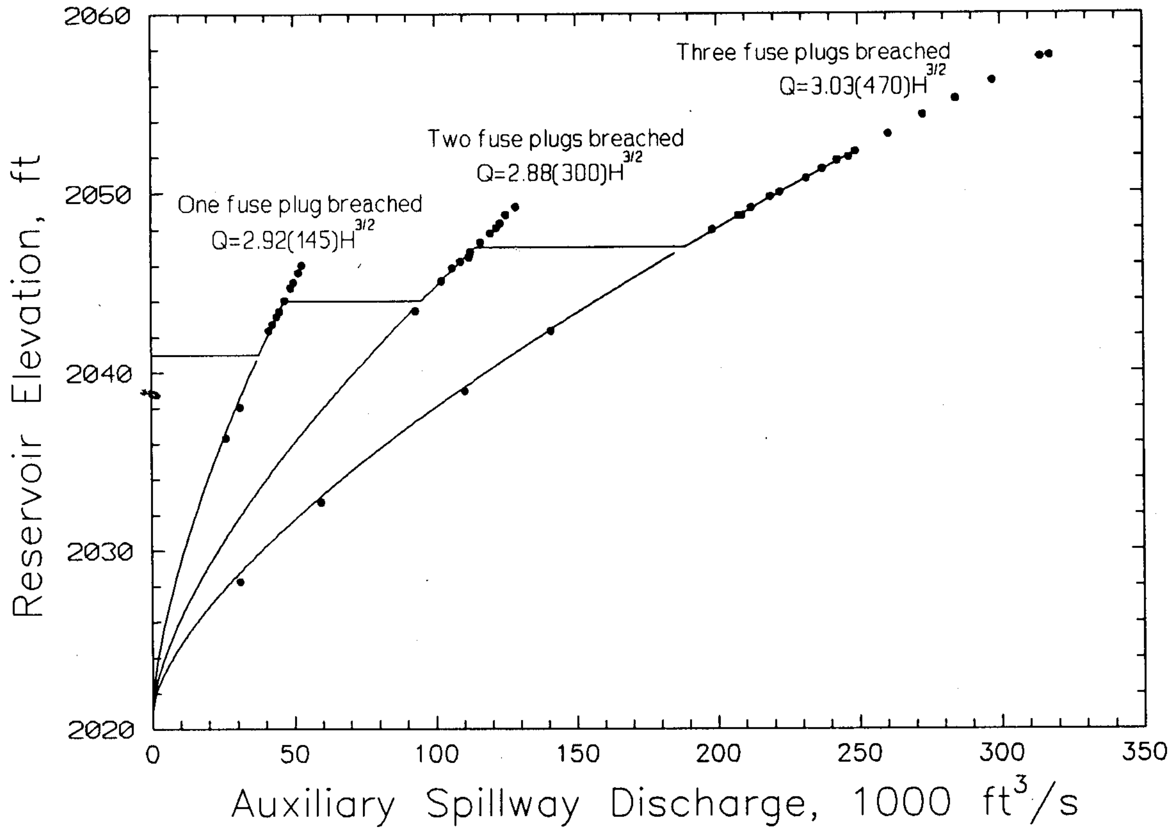

Figure 7. – Prototype auxiliary spillway discharge curves constructed from hydraulic model data (470-ft modified spillway).

The variation of the discharge coefficients is caused by the channel geometry and the losses associated with entrance contractions and flow obstructions. To illustrate these factors, an upstream plan view of the spillway crest structure is shown on figure 8. The discharge coefficient for the first section is 2.92, based on a nominal crest length of 145 ft. Factors reducing the discharge through the first section are the contractions from the left entrance and the first splitter wall, and the flow area occupied by the splitter wall, which fills a portion of the nominal crest length. One factor increasing the discharge coefficient is the presence of additional flow area contained within the 0.25 to 1 wedge (fig. 8) on the left side of the opening.

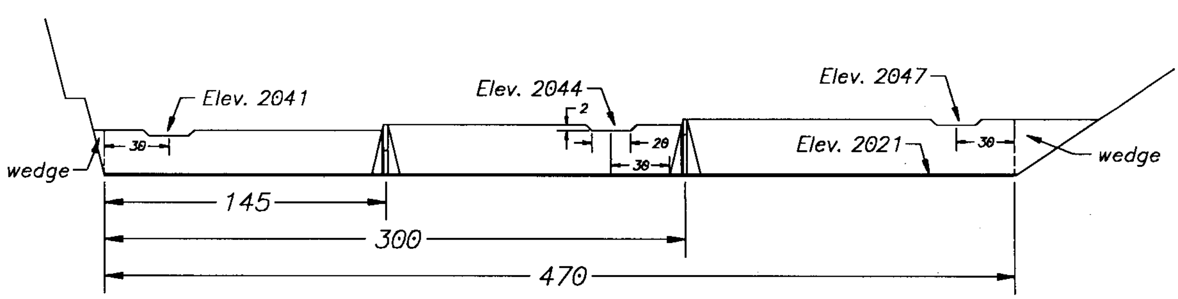

Figure 8. – Upstream elevation view of the auxiliary spillway control structure with fuse plug embankments and concrete splitter walls. Wedges referred to in the text are also indicated.

With the second section open, the coefficient drops to 2.88, based on a nominal crest length of 300 ft. The reduction of the discharge coefficient is caused by the presence of the first splitter wall in the middle of the crest; it now occupies a portion of the nominal crest length. The approach length upstream of the crest is also greater for the second section. Losses from the contraction off the left entrance and the second splitter wall are similar to those for the first section.

With the third section open the discharge coefficient increased to 3.03, based on a nominal crest length of 470 ft. The second splitter wall now occupies a portion of the nominal crest length. However, the contraction off the splitter wall on the right side of the section has been replaced by the relatively flat right side slope. The contraction associated with this side slope is extremely small, and an additional component of flow area is contained in the wedge; this area is not accounted for in the nominal crest length.

Variation of discharge coefficient with head.—The discharge coefficients given above were fitted to the data points collected at reservoir elevations above the pilot channel invert elevation of each fuse plug section. This yields the greatest flood routing accuracy during the critical rising limb of a large flood event. However, the discharge coefficient does vary significantly over the full range of spillway operating conditions.

Figure 9 shows the observed discharge coefficients plotted against reservoir elevation. At high reservoir levels the crest and sill block function as a broad-crested weir, with little variation of the discharge coefficient. At low reservoir levels, the sill block height, P (fig. 2), is larger in proportion to the depth of flow and begins to perform similarly to an ogee crest shape; this causes the discharge coefficient to increase. The lowest data points shown on the figure correspond to model flow depths of about 1 inch over the sill block. Below this level, surface tension effects in the model begin to distort the results.

Generalized discharge coefficient.—To assist in the design of future spillways using similar crest sill block designs, a generalized discharge coefficient was computed for the crest. The nominal crest length was converted to an effective crest length by subtracting the crest length occupied by the splitter walls and adding the crest length contained in the side slope sections. These calculations were based on the flow area contained in or occupied by these features at the critical section. The computed effective crest length also included a reduction of $0.1NH$ due to contractions at the left and right sides of the spillway, where N is the number of contractions and H is the head on the crest. The relatively flat, right-side slope was assumed to cause no contraction loss.

Using the effective crest lengths, generalized discharge coefficients were computed for each data point. Figure 10 shows the variation of this coefficient as a function of nondimensionalized head, H/P, obtained by dividing the head (H) by the height of the sill block (1 ft). Again, the coefficient tends to increase as the head is reduced. Above a nondimensional head of about 20, the generalized discharge coefficient is in the range of 2.95 to 3.05. The variation of the coefficients among one-, two-, and three-section operation is reduced from that shown on figure 9. The variation that still remains is due to factors not considered in the analysis, such as differences in the length of the approach channel for individual sections.

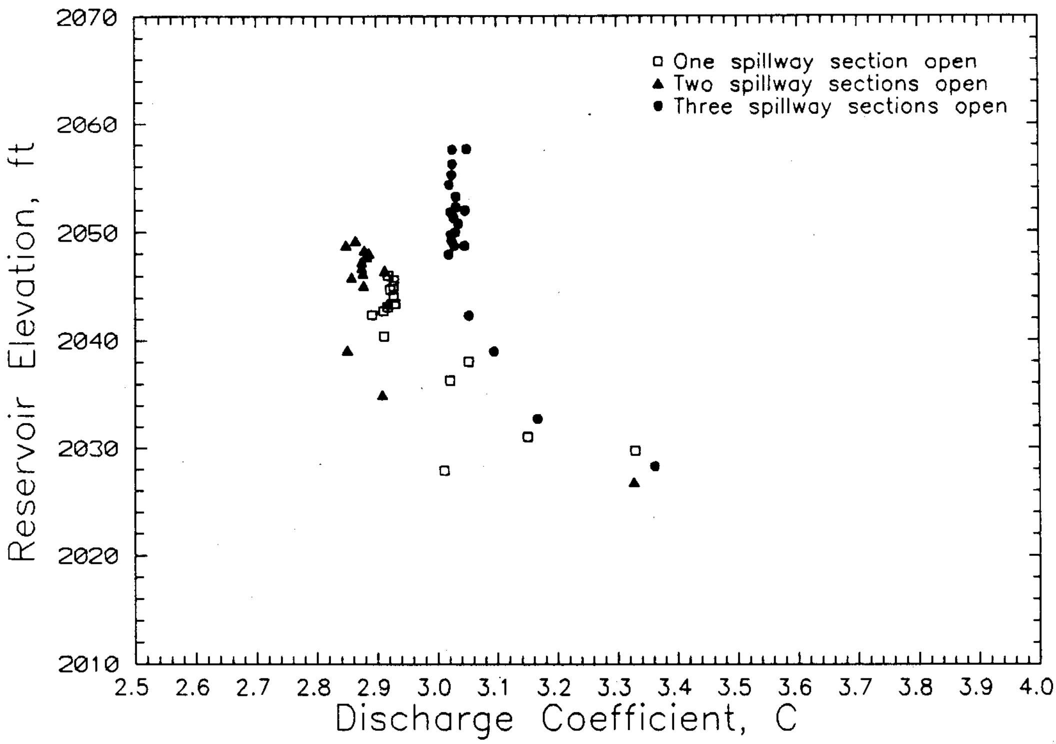

Figure 9. – Variation of spillway discharge coefficient as a function of reservoir elevation.

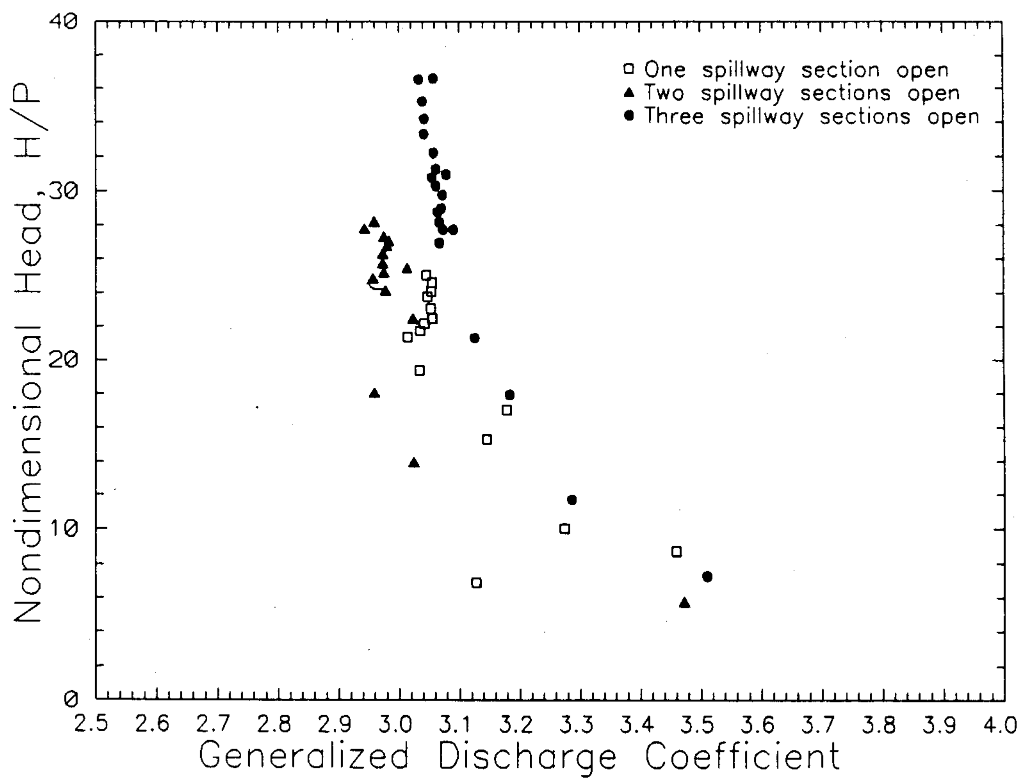

Figure 10. – Generalized discharge coefficients for sill block crest shape with adjustments for pier losses and non-rectangular channel cross section. The nondimensional head is the ratio of head, H, to sill block height, P.

Splitter Walls

Two splitter walls will be constructed in the auxiliary spillway to separate the three fuse plug embankments. Each splitter wall has an upstream and downstream training wall. These training walls protect the second and third fuse plug embankments during the operation of the preceding spillway sections. Figure 11 shows one of the two splitter walls.

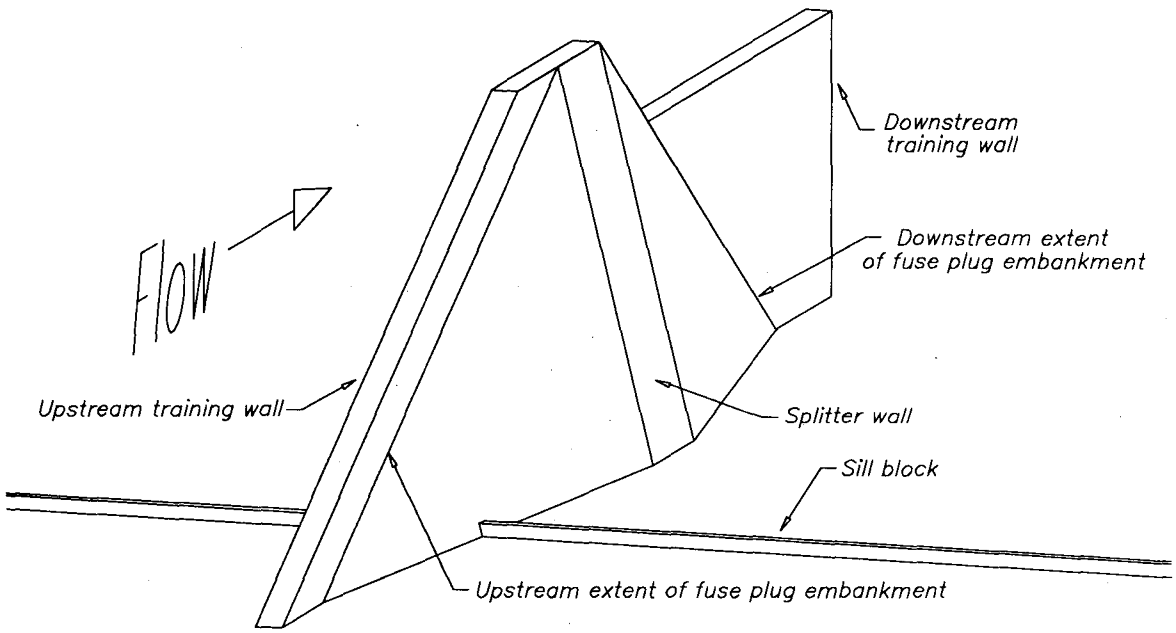

Figure 11. – Three-dimensional view showing the basic configuration of the concrete splitter walls with attached upstream and downstream training walls.

The core section of each splitter wall will be constructed flush with the upstream and downstream faces of the highest adjacent embankment. The upstream training wall rises an additional 5 ft vertically above the upstream embankment face. During operation of the spillway section to the left of the splitter wall, the upstream training wall reduces the velocity of the flow sweeping across the embankment toward the open spillway section. High velocities in this area could cause damage to the embankment that might lead to premature operation of the next fuse plug section.

Each downstream training wall extends horizontally downstream from the core of the splitter wall and prevents flow through one spillway section from attacking the downstream face of the adjacent embankment. This also prevents damage to the embankment that could cause premature breaching of the adjacent fuse plug.

Upstream training walls.—As successive fuse plug sections are breached, flow enters the approach channel and converges toward the open spillway sections. With only one or two sections open, significant velocities occur along the upstream face of the adjacent fuse plug embankment that has not yet breached. To provide data required for the design of slope protection on the upstream face of the fuse plug embankments, velocities were measured along the upstream faces of the embankments with the adjacent spillway section open and the reservoir just below the level at which breaching of the next fuse plug would occur. The velocity measurements were made with a small propeller meter held about 1/2 inch above the face of the embankments. Figures 12 and 13 show the prototype velocity readings and isovels for the second and third fuse plugs, respectively. The 5-ft vertical height upstream training wall effectively reduces the velocities within 10 ft of the open section. The maximum velocity along the second fuse plug was 6.6 ft/s, and the maximum velocity along the third fuse plug was 7.5 ft/s. The maximum velocities occurred about 10 to 15 ft away from the left edge of each embankment, near the upstream toe. Without the upstream training walls, velocities would continue to increase up to the edge of the opening.

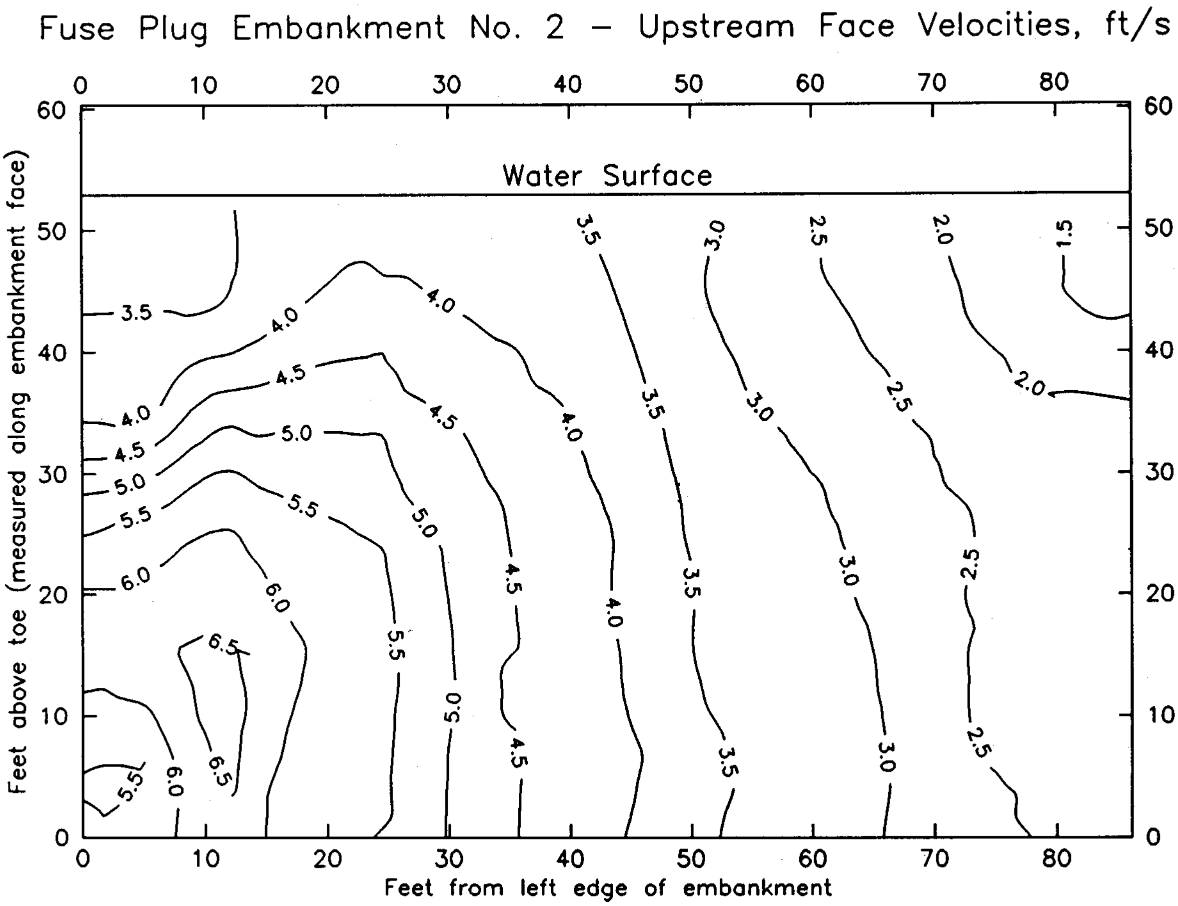

Figure 12. – Prototype isovels on the upstream face of the second fuse plug embankment with the first section open and the reservoir at elevation 2044.0 ft. This view is perpendicular to the face of the embankment.

Downstream training walls.—The water surface profile along the first splitter wall's downstream training wall was recorded with flow through the first section at reservoir elevation 2044 ft, the highest level for which flow will be limited to only the first section. Similarly, the water surface profile along the second downstream training wall was recorded with flow through the first two sections at reservoir elevation 2047 ft. The maximum elevations of these profiles were 2029 and 2031 ft, respectively. The training walls should be constructed to provide additional freeboard above these elevations to prevent overtopping of the walls by waves. A recommended freeboard level can be estimated from the following empirical equation, based on Reclamation experience in the design of spillway chutes (Reclamation, 1987):

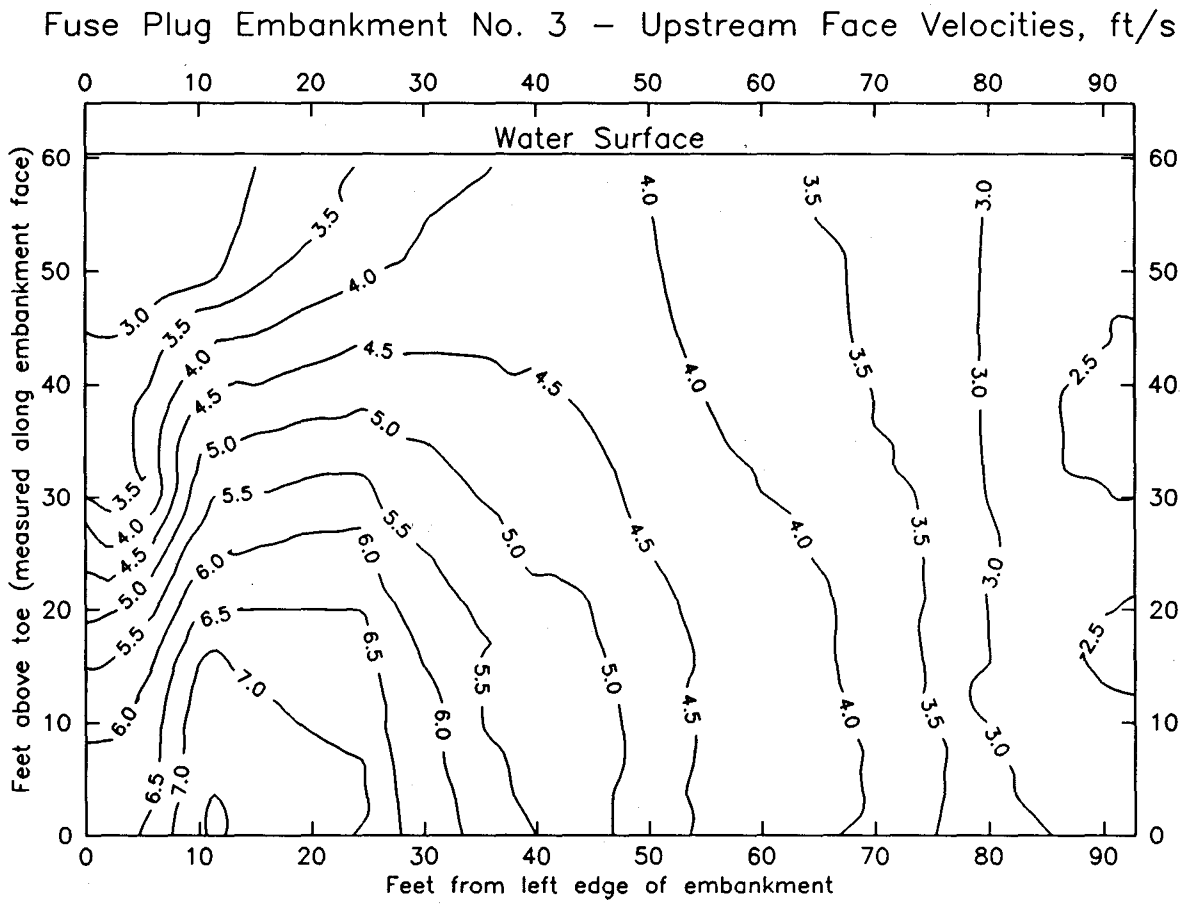

Figure 13. – Prototype isovels on the upstream face of the third fuse plug embankment with the first two sections open and the reservoir at elevation 2047.0 ft. This view is perpendicular to the face of the embankment.

$$f = 2 + 0.025\, vd^{1/3}$$

where:

f = recommended freeboard, ft
v = velocity through chute, ft/s
d = depth of flow in chute, ft

Using depth and velocity data collected from the model, the equation yields a recommended freeboard of 4 ft for each wall, producing wall elevations of 2033 and 2035 ft.

The flow entering the spillway refracts slightly to the right (about 5°) as it crosses from the deep reservoir into the relatively shallow approach channel. This effect was most noticeable on the left side of the spillway. The refraction of the flow is caused by the angle at which the approach channel daylights into the reservoir. When the flow reaches the splitter walls it tends to ride up slightly on the left side of each wall. However, this does not cause a significant increase in the water surface profile along the downstream training walls; the thickened midsections of the splitter walls deflect the flow away before it reaches the downstream training walls.

Approach Channel

Understanding the flow conditions in the approach channel is critical for properly locating the pilot channels for the second and third fuse plug sections. If a significant drawdown of the water surface occurs in the approach channel, the initiation of flow through the pilot channel will be delayed to a higher reservoir elevation. This could lead to a delayed breaching of the second and third fuse plug sections. Because drawdown is likely to be the greatest near the opening of the section that is operating, the pilot channels for the second and third sections were located on the right ends of the embankments. Measurements were made in the model to determine whether drawdown was still significant at these locations.

The spillway was placed in operation with the first section open, and the reservoir was adjusted to elevation 2044.0 ft, the level at which flow begins through the pilot channel of the second fuse plug. Point gage measurements were then taken to determine the water surface elevation in front of the pilot channel of the second fuse plug. Significant drawdown of the water surface could not be detected. Similarly, with flow through the first two sections at reservoir elevation 2047.0 ft, there was no significant drawdown of the water surface in front of the third fuse plug pilot channel. The lack of drawdown was primarily due to the relatively short length of the approach channel. Based on these results, adjustment of the pilot channel invert elevations is not required.

To aid in the design of slope protection at the right side of the spillway crest, the maximum water surface along the right side of the crest section was marked and surveyed with all three spillway sections operating at reservoir elevation 2052.0 ft (fig. 14). These data were collected with the right side excavated at a 1.5h:1v slope, rather than the 1 to 1 slope used for final design. The change in water surface profile due to this difference should be minimal. The reduced water surface between stations 1+50 and 2+25 (feet) is due to a slight separation of the flow off the corner of the 40° beveled entrance. This slight separation was not deemed to be a problem. The corner will be slightly rounded by blasting during construction, which will reduce the separation and cause the water surface profile to be more uniform.

Velocities were measured across the full width of the approach channel from the upstream daylight point of the spillway excavation to 60 ft downstream of the crest location (fig. 15). The velocity in the approach channel varies from 15 ft/s at the right side of the channel entrance to 30 ft/s at the crest. The Froude number in the approach channel varies from about 0.5 to 1.0 at the crest. Figure 15 also shows the area of reduced velocity on the centerline of the spillway caused by the boiling action discussed earlier.

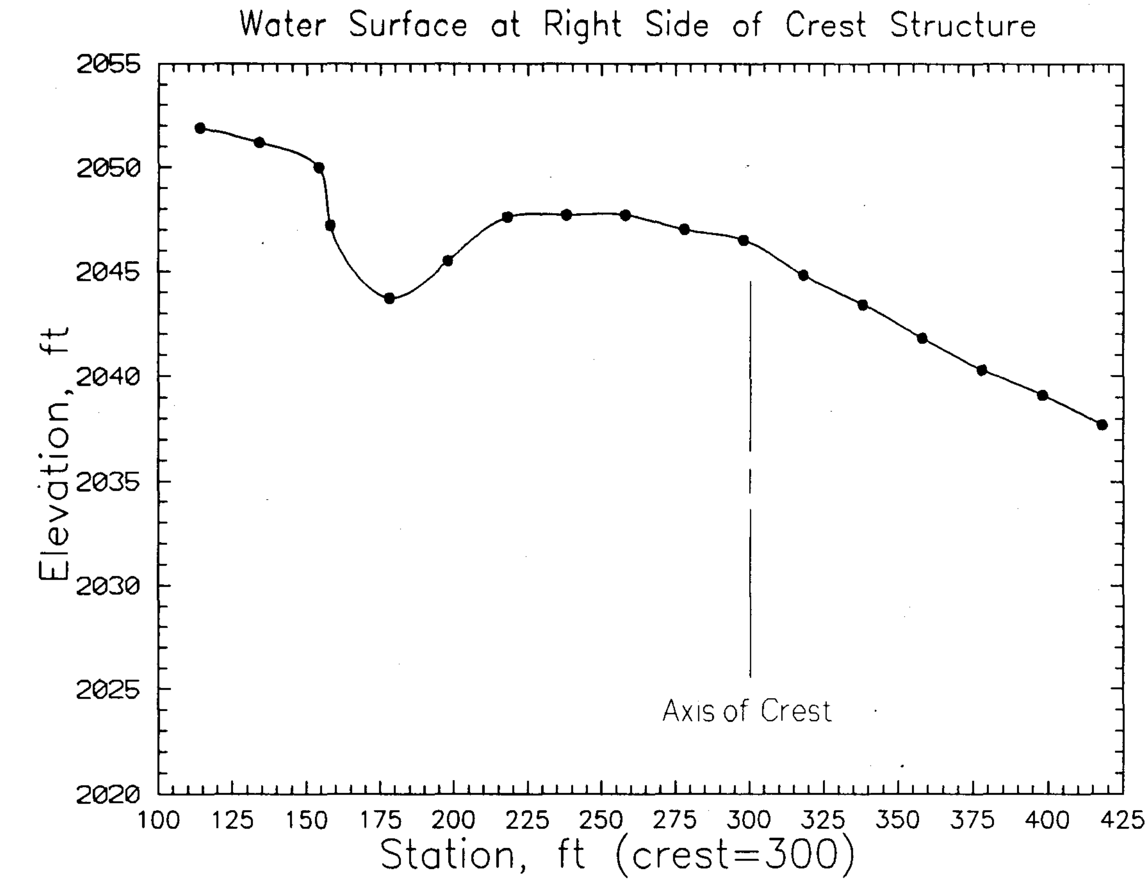

Figure 14. – Maximum water surface along the right side of the spillway channel, upstream and downstream of the crest location. The reduced water surface between stations 150 and 225 ft is due to slight flow separation off the 40° beveled entrance.

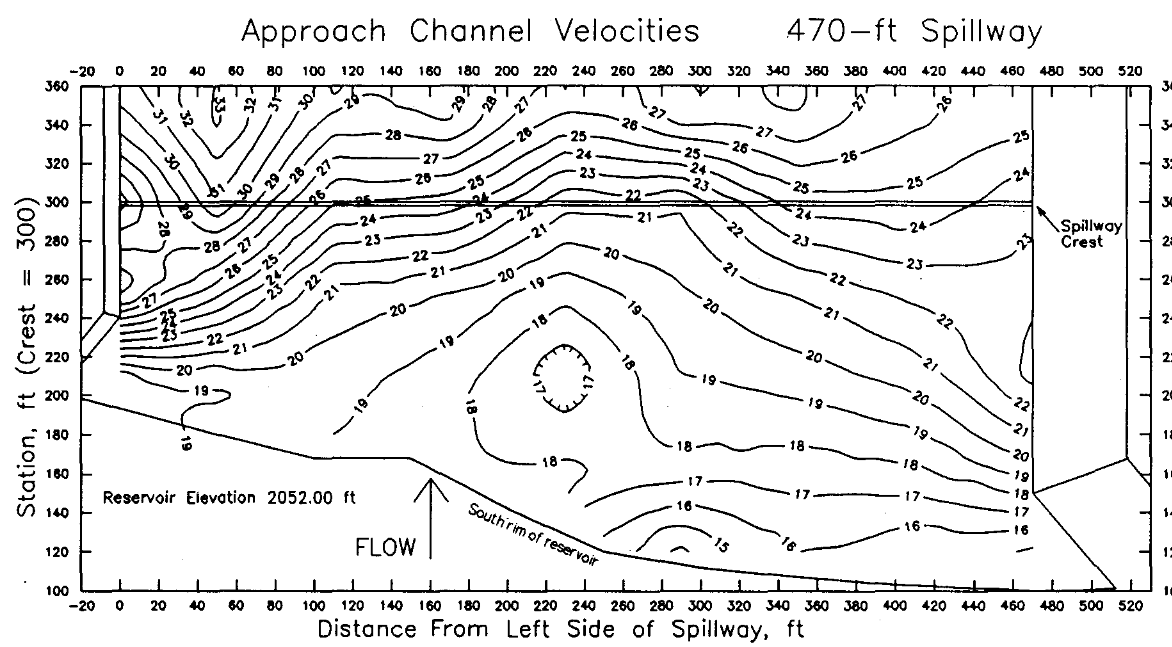

Figure 15. – Prototype approach channel isovels based upon velocity measurements made in the hydraulic model.

SPILLWAY CHANNEL EROSION INVESTIGATION

Test No. 1 — 575-ft Crest Length

The first dynamic erosion test was conducted with the 575-ft crest length spillway, with successive fuse plug sections of 150, 175, and 250 ft in length. The PMF was routed through the reservoir using the auxiliary spillway discharge coefficients determined in the first phase of testing. The service spillway gates were assumed to have a maximum opening of 30.4 ft, yielding a maximum discharge of 316,000 ft^3/s at reservoir elevation 2052.0 ft. This routing produced a maximum auxiliary spillway discharge of 253,100 ft^3/s. The elapsed time from initiation of breach of the first fuse plug to the peak flow was 15.8 hours—equivalent to 2.04 hours in the model. The test ran for a total of 3 hours; at the end of the test the flow had decreased to about 75 percent of the peak discharge, and erosion of the channel had essentially ended.

The pattern for erosion in the spillway channel was established early in the test. The critical feature was the *transition line* where the channel bottom crosses from the rock excavation into the alluvial deposits in the ancient river channel. (The hydraulic jump, visible on figure 21, is just downstream of the transition line. The transition line in the model is also visible on figure 22.) This line crosses the spillway from west-northwest to east-southeast at about a 60 to 70° angle to the spillway centerline. Along this line the flow quickly began eroding the bed and established a hydraulic jump in the channel. This eroded additional material, some of which was dropped immediately downstream of the hydraulic jump; this created an obstruction in the channel that deflected the flow to the left, into the training dike on the east side of the spillway channel. This flow pattern persisted throughout the operation of the first spillway section.

Following the breach of the second section, about 12 minutes into the test, the same pattern of erosion, deposition, and deflection of the flow quickly developed. Discharge through the first and second spillway sections continued to rise for about 20 minutes, with little change in the flow patterns in the spillway. At 32 minutes into the test the flow through the first two sections had scoured out a deep hole in the ancient river channel. Lateral erosion into the training dike had slowed significantly.

The third fuse plug breached between 32 and 35 minutes into the test. Shortly following the completion of the breach, the flow began overtopping the training dike about 1200 ft downstream of the crest structure, through a low saddle provided for the crossing of an access road. The dike elevation in the model at this point (surveyed following the sand placement) was approximately 2027.5 ft. The overtopping flow quickly cut down through the dike and began laterally eroding the dike back upstream. At the peak discharge, 2 hours into the test, about 50 percent of the flow was being diverted back toward the dam and service spillway area.

Erosion of the right side of the spillway channel was confined to the upstream reach of the channel, between the crest structure and the uplifted lakebed deposits at the downstream right side of the spillway channel (the lakebed deposits were nonerodible in the model). The high velocity flow along the right side after the breaching of the third fuse plug quickly eroded laterally into the right side of the spillway excavation. A back eddy formed with a recirculating flow that continued to erode further into the right side of the excavation. The areal limits of the model prevented determination of the ultimate extent of this erosion. However, as erosion in this eddy progresses further upstream (outside of the model boundaries) the elevation and quality of the andesite rock will increase quickly, limiting the extent of erosion. Also, velocity measurements in this eddy showed the flow to be unsteady. Average velocities over a 30-second period at the north side of the eddy (the leading edge of erosion) were in the range of 7 ft/s (prototype). Maximum instantaneous velocities were in the range of 10 ft/s. Occasionally the flow in the eddy was nearly tranquil. The maximum water surface elevation observed in the eddy was about 2030 ft. Based on the geologic conditions and the observations made in the model, the erosion is expected to stop before progressing far enough to threaten the closure dike, just west of the auxiliary spillway.

Figure 16 shows the extent of erosion at the end of the test. Toeboards attached to the edge of the model topography during sand placement exaggerated the extent of erosion at the downstream left corner of the box. Flow through this area should not threaten the relocated dam tender's facilities. (The training dike shown on figure 16 is an early conceptual design, and is much narrower than the dike tested in the model. The streamwise length of the dike shown on figure 16 is similar to the dike that was tested in the model; the breadth of the tested dike was similar to that shown on figure 4.)

Test No. 2 — 470-ft Crest Length

Following the first phase of testing of both the auxiliary and service spillways, the auxiliary spillway was narrowed to a total crest length of 470 ft, with successive section lengths of 145, 155, and 170 ft. The downstream width of the spillway channel was maintained at 290 ft. At the same time, the crest sill block was moved about 27 ft downstream to provide increased area for construction access. To prevent overtopping of the training dike, the access road was relocated and the saddle eliminated. The 18-in thick concrete contact pad was modified at the left side of the crest structure.

For the second erosion test, the PMF was routed assuming a service spillway gate opening of 30.68 ft (maximum gate opening, maximum discharge of 318,000 ft^3/s). This yielded a peak discharge through the auxiliary spillway of 241,300 ft^3/s. This peak occurred about 15.5 hours after the breach of the first fuse plug—equivalent to 2.0 hours in the model. Figure 17 shows the PMF inflow and the routed outflows through both spillways. Figure 18 shows the routed auxiliary spillway hydrograph for the second erosion test.

In the second test, erosion of the spillway channel developed in a similar fashion to the first test. As alluvial material eroded at the transition line, a hydraulic jump was established that formed a scour hole and redeposited material downstream of the jump. This encouraged lateral erosion into the training dike. During operation of the first section, erosion of the dike was minor and limited to the most upstream 200 ft of the dike (fig. 19). With the second section in operation, lateral erosion began occurring along the full length of the dike, with the most pronounced erosion taking place about 800 to 1400 ft downstream of the crest (fig. 20). Erosion of the training dike had nearly ceased just prior to the breaching of the third embankment.

When the third embankment section breached, the flow did not overtop the training dike (fig. 21). Lateral erosion into the dike, deepening and lengthening of the scour hole in the ancient river channel, and degradation of the full length of the channel continued until about 1 hour into the test (7.75 prototype hours). By this time, the scour hole in the ancient river channel had been extended downstream several hundred feet from the transition line. The flow plunging into the scour hole was accelerated to the right, due to the angle at which the transition line crosses the spillway channel. This established a weak back eddy on the left side of the channel along the training dike, and lateral erosion of the dike essentially ceased. Degradation of the channel bed continued until all erodible materials were washed out of the channel invert. The test was stopped about 3.5 hours after the breach of the first fuse plug (27.5 hours, prototype).

Figure 22 shows the spillway operation near the peak discharge. Figure 23 shows the configuration of the channel at the end of the test. Following the test, the eroded channel was surveyed, and a contour map was constructed (fig. 24). Areas where erosion reached the nonerodible topography are shown on figure 24. The figure shows that the channel was eroded down to the nonerodible bed over the majority of the area of the original channel excavation.

To simplify access to the site, it may be advantageous to cross the training dike with an access road that enters the spillway channel at the downstream left corner. The road could traverse up the west side of the training dike and cross over the crest of the dike near its upstream end. The grade of the road would be minimized if a lowered section could be provided near the upstream end of the dike.

Figure 25 shows the high-water level recorded during the second erosion test along the left side of the spillway channel. The height of the training dike at the edge of the eroded area is also shown as it was placed and surveyed in the model; where erosion did not reach the midsection of the dike, the dike crest is higher than the line shown. Figure 25 also shows that the high-water line increases in elevation in the downstream direction. This is confirmation of the back eddy and reverse flow along the dike at the peak discharge. Based upon these data, the crest elevation of the upstream section of the dike can be reduced to 2035 ft.

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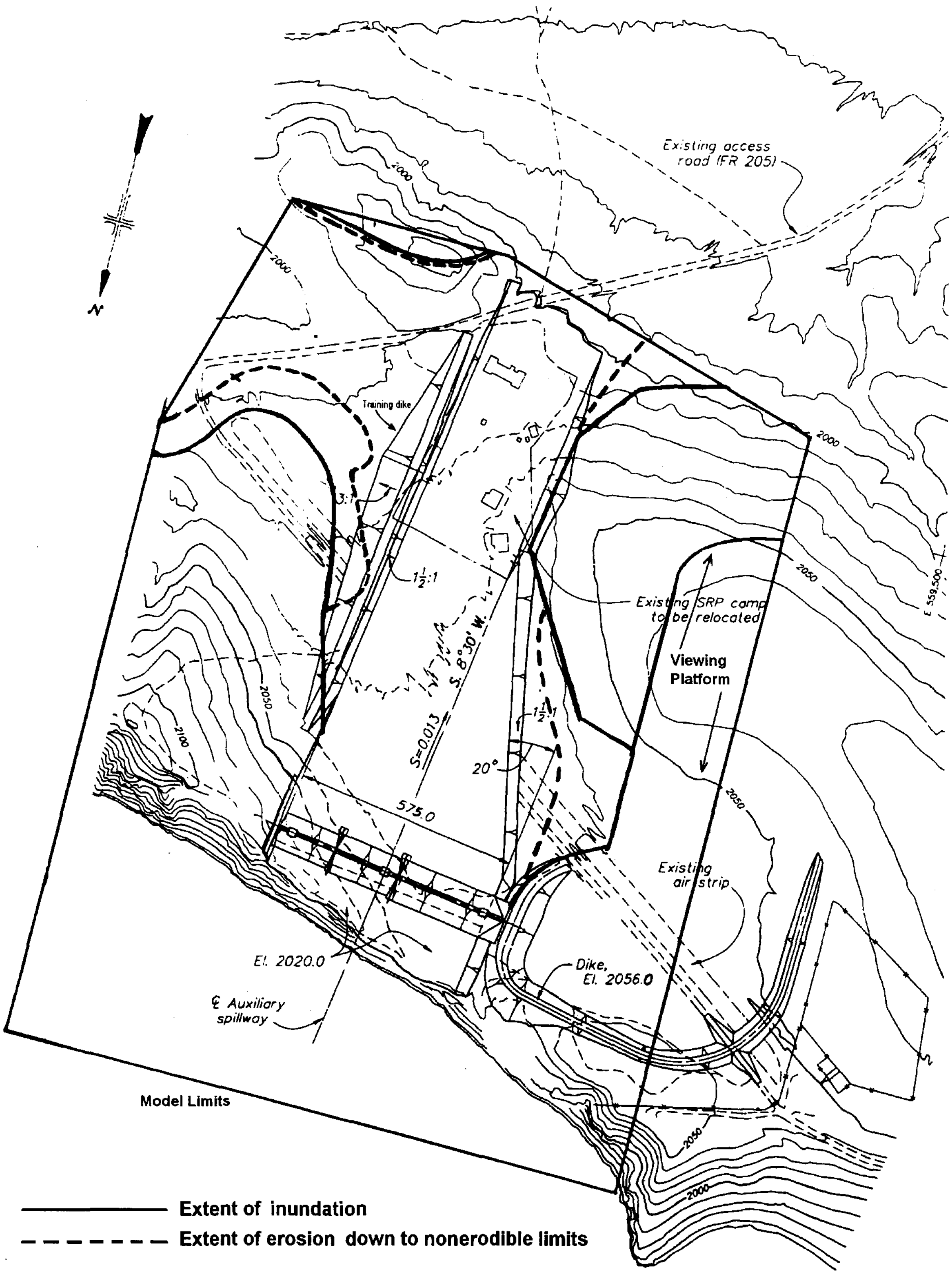

Figure 16. – Extent of erosion and inundated area after the first dynamic erosion test. Solid lines mark the edge of the inundated area at the maximum discharge. Between the dashed lines, erosion removed all of the erodible materials from the model.

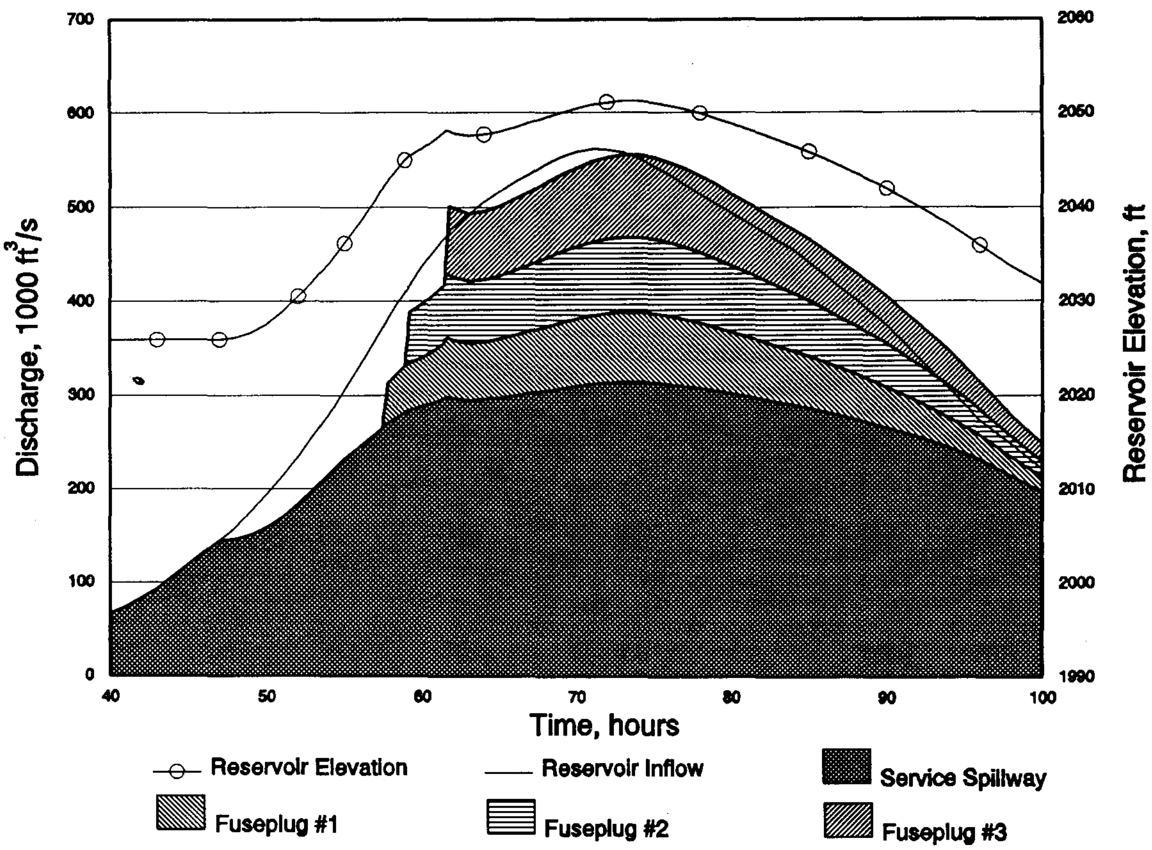

Figure 17. – The results of routing the PMF through Horseshoe Reservoir for use in the second dynamic erosion test.

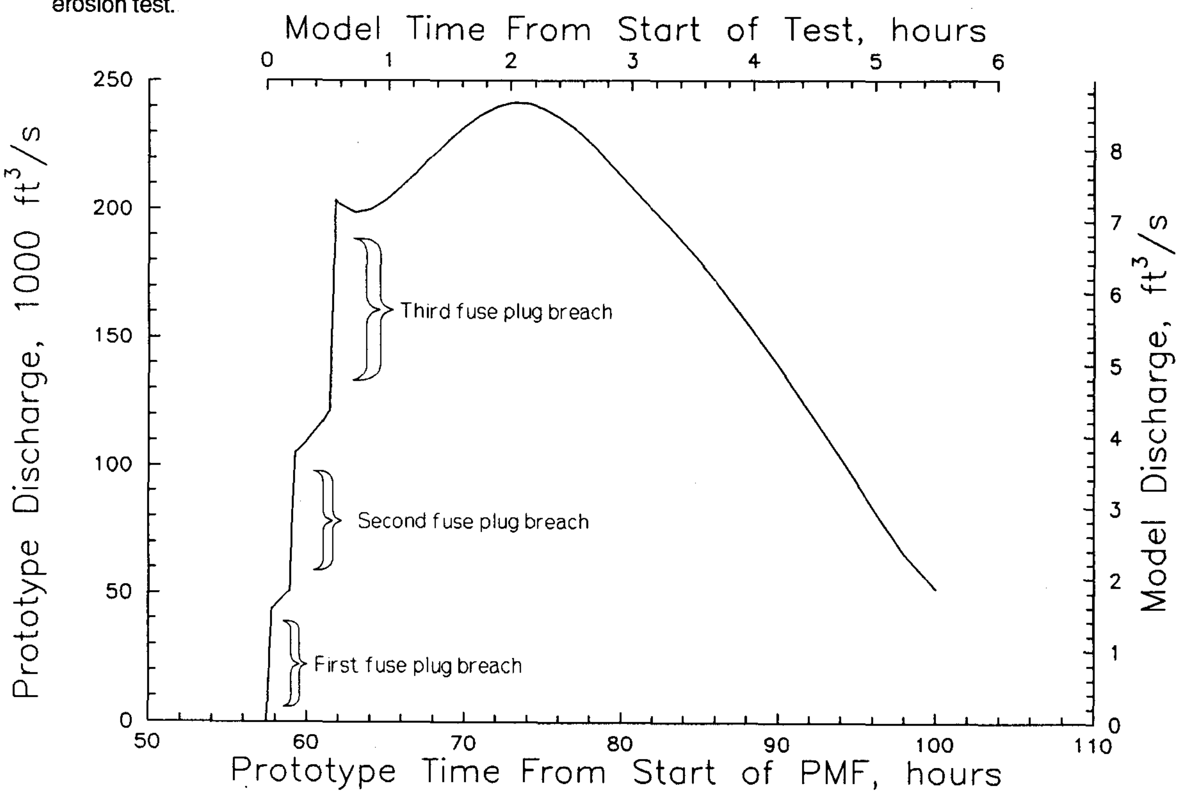

Figure 18. – Auxiliary spillway discharge during second dynamic erosion test.

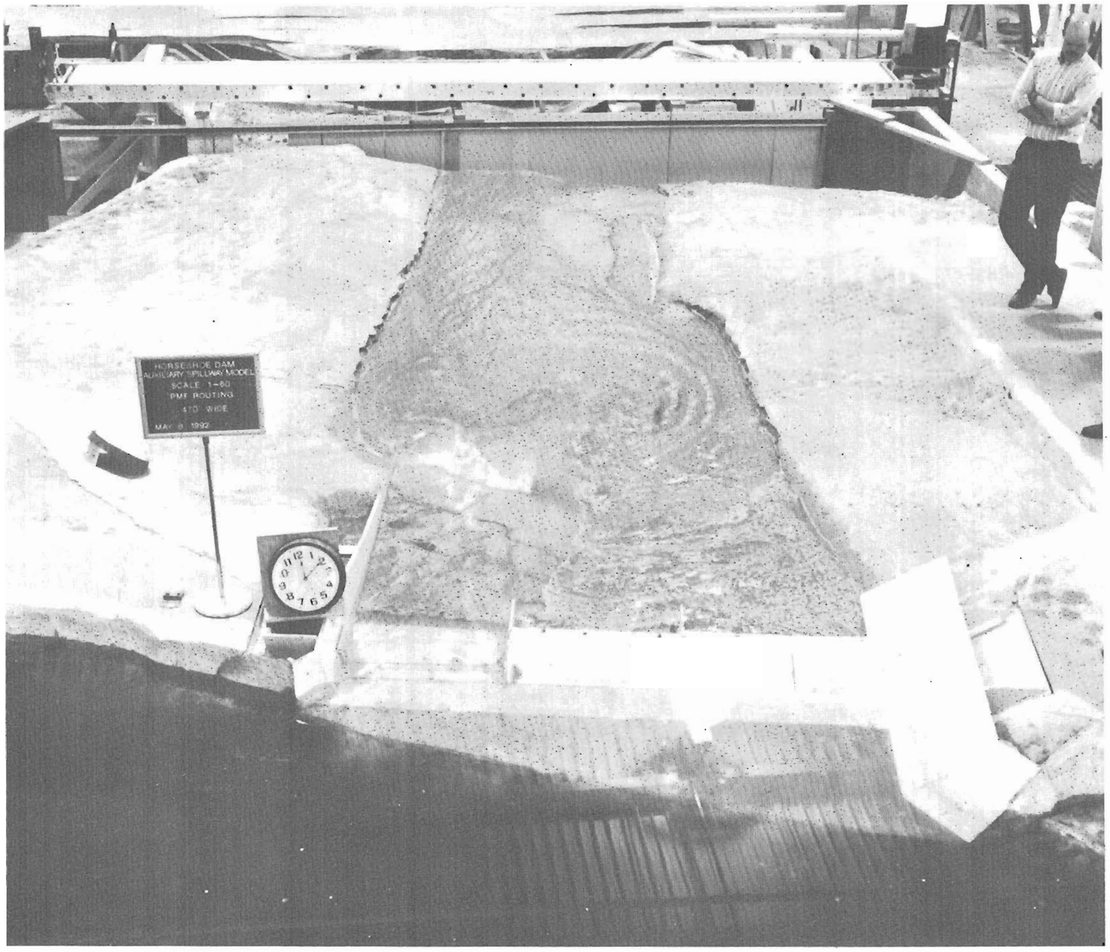

Figure 19. – Flow of 45,000 ft^3/s through the first section of the auxiliary spillway during the second dynamic erosion test. The flow is deflected into the training dike by erodible materials redeposited in the tailwater of the hydraulic jump.

Figure 20. – Flow of 115,000 ft^3/s through the first two sections of the auxiliary spillway during the second dynamic erosion test. The flow continues to erode the training dike at the left side of the channel.

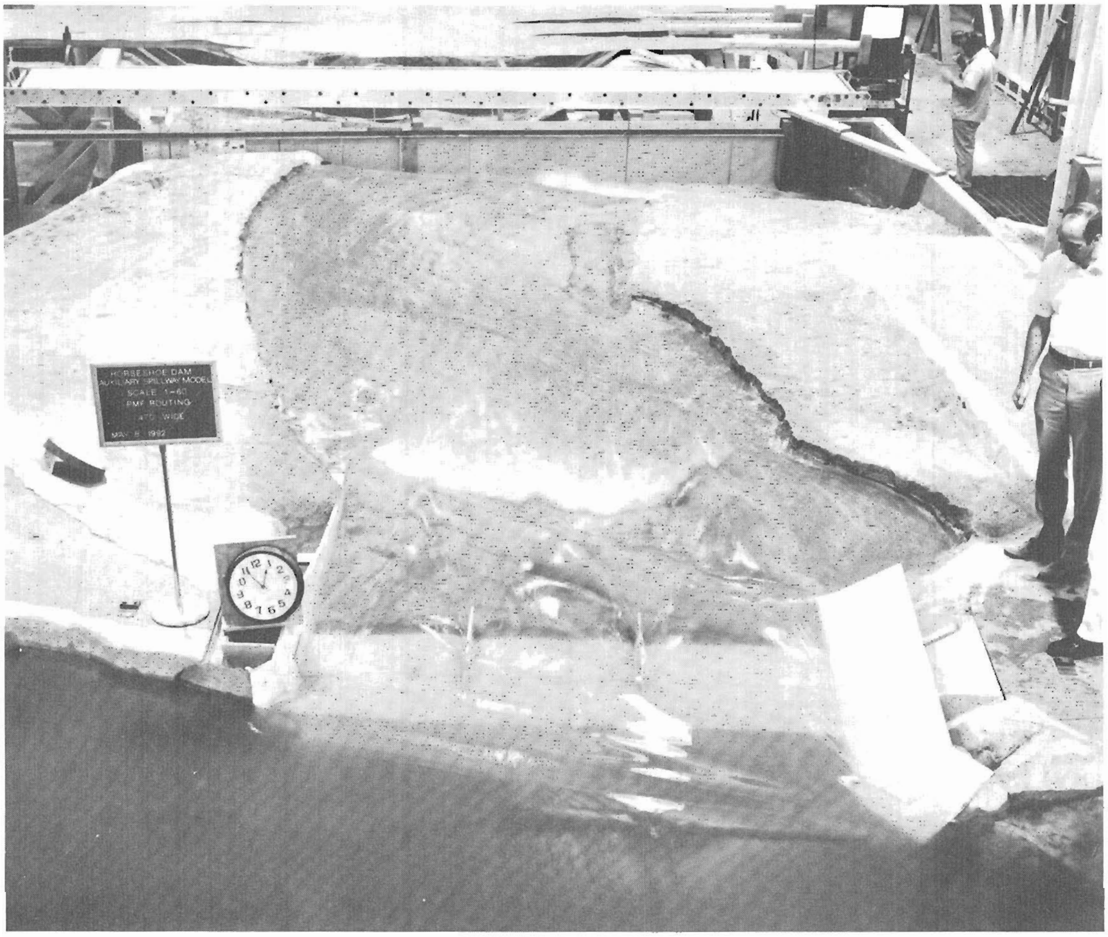

Figure 21. – Flow of 200,000 ft^3/s through all three sections of the auxiliary spillway during the second dynamic erosion test. This photograph was taken just after the breach of the third fuse plug section.

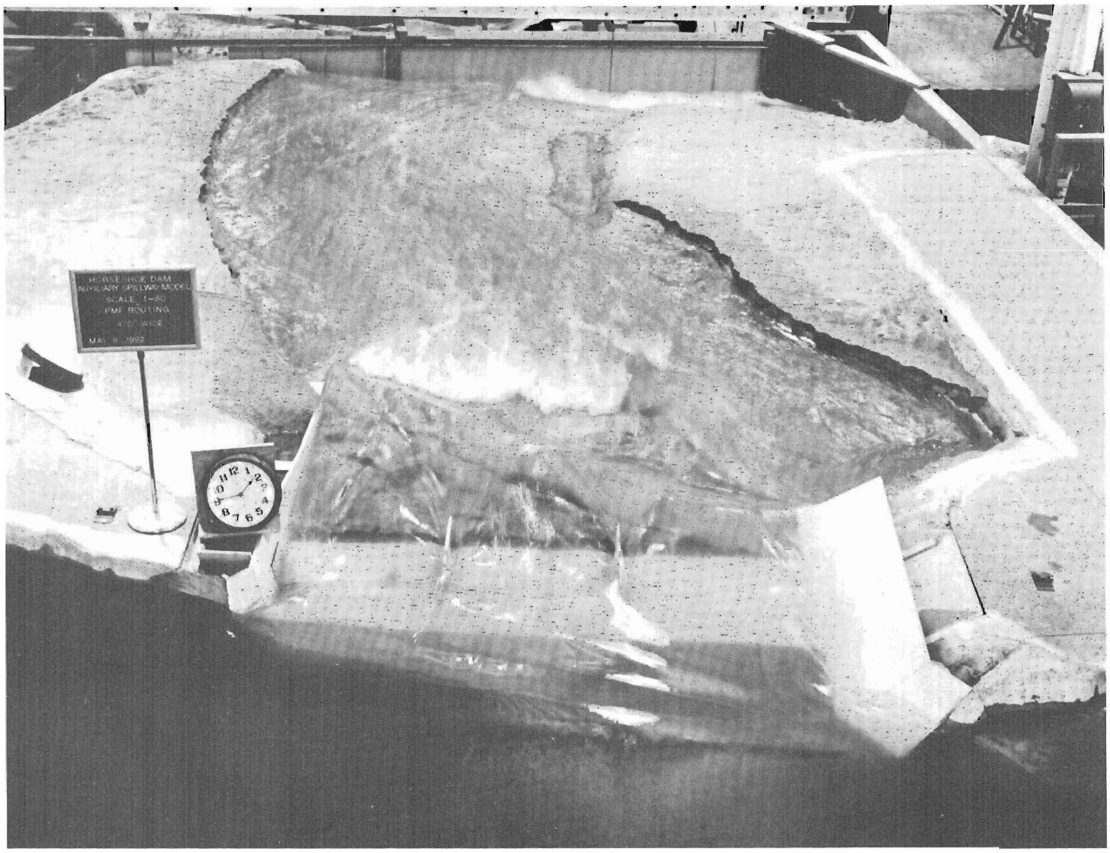

Figure 22. – Peak discharge of 242,000 ft^3/s during the second dynamic erosion test. The *transition line* discussed in the text is just upstream of the hydraulic jump at the center of the photo.

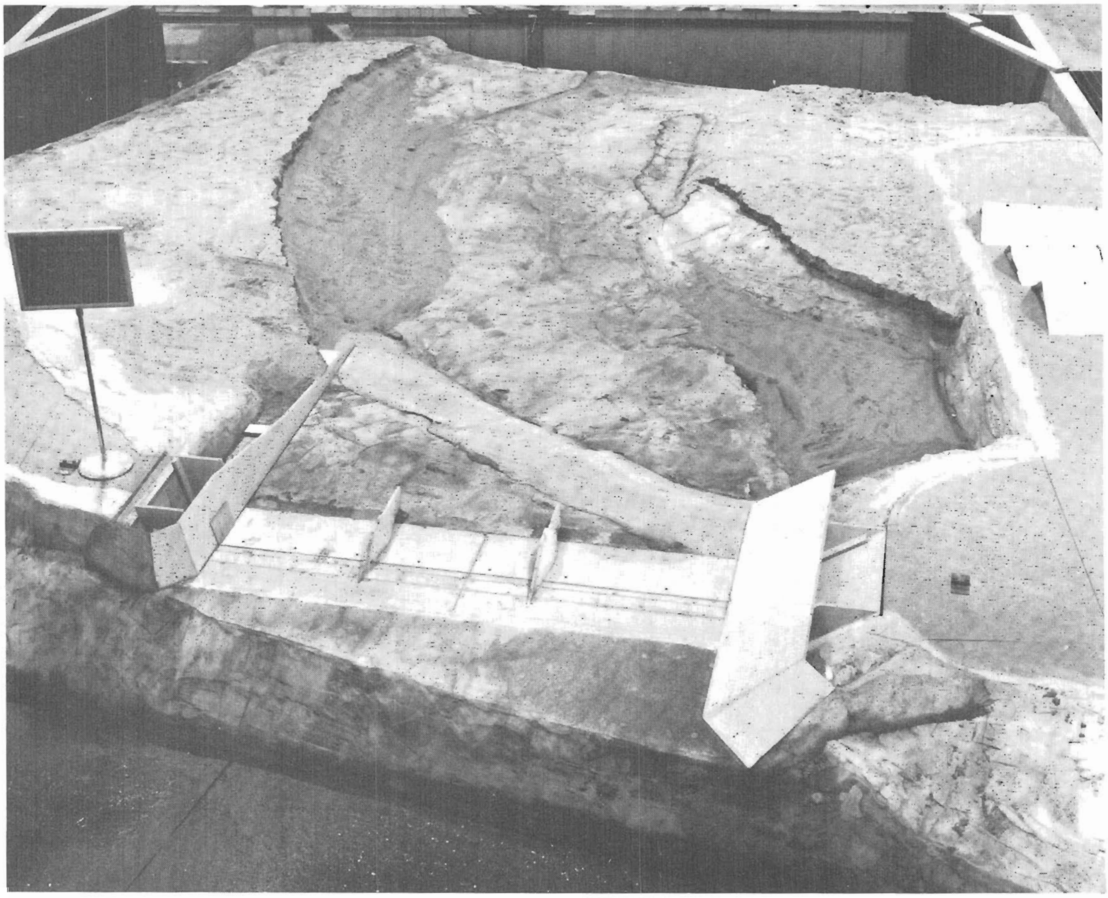

Figure 23. – Final eroded condition of the model auxiliary spillway channel following the second dynamic erosion test.

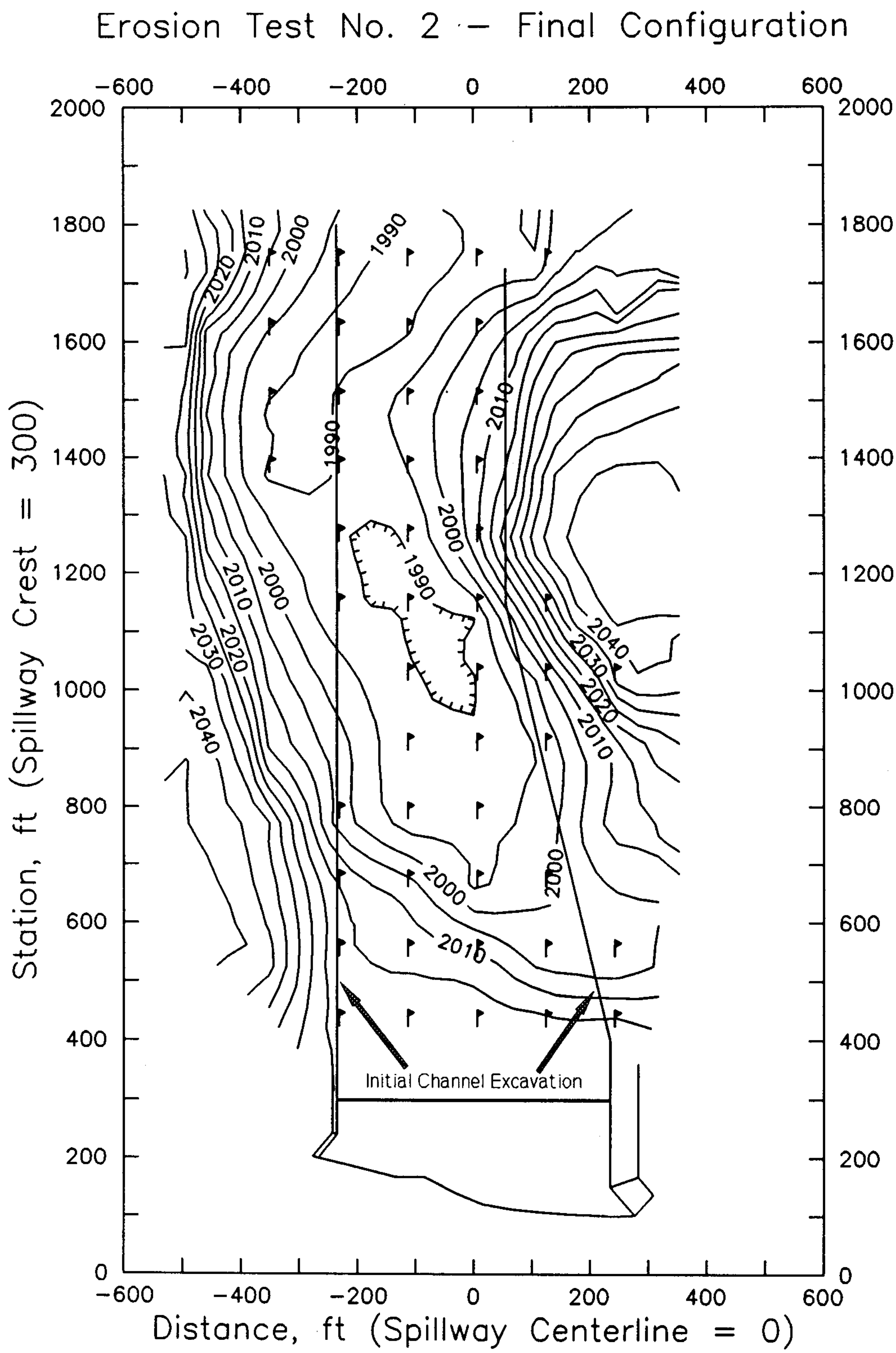

Figure 24. – Final configuration of the spillway channel after the second dynamic erosion test. Flags mark the survey points where erosion reached the nonerodible bed of the model.

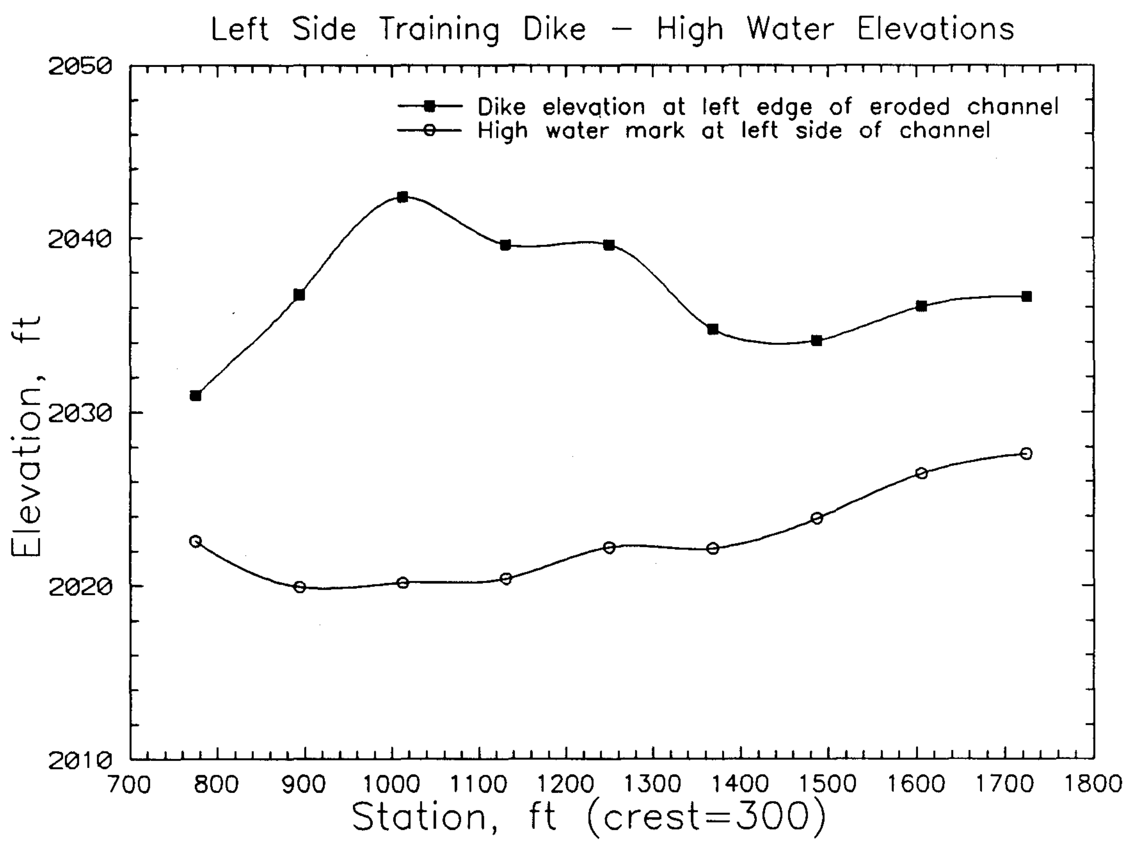

Figure 25. – Elevation of high-water line on the left side of the spillway channel during the second dynamic erosion test. Dike elevations shown are at the edge of the inundated area; the crest elevation of the dike may be higher.

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