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Hydraulic Laboratory Report HL-2009-05

Hydraulic Model Study of Folsom Dam Joint Federal Project Auxiliary Spillway Confluence Area

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Hydraulic Investigations and Laboratory Services
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Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Acknowledgments

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LIST OF SYMBOLS AND ACRONYMS

2D	two-dimensional
3D	three-dimensional
CFD	computational fluid dynamics
COE	U.S. Army Corps of Engineers
JFP	Joint Federal Project
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
PMF	probable maximum flood
Reclamation	Bureau of Reclamation
SAFCA	Sacramento Area Flood Control Agency
SAFL	University of Minnesota's St. Anthony Falls Laboratory

Executive Summary

A 1:48-scale physical hydraulic model of the principal features of the Joint Federal Project at Folsom Dam was constructed and tested by the Bureau of Reclamation's Hydraulic Investigations and Laboratory Services Group. The model included the main dam spillways, the auxiliary spillway and stilling basin, the confluence area of the two exit channels, and a section of the downstream river channel. The primary objective of the physical modeling was to evaluate the three-dimensional flow characteristics in the vicinity of the confluence between the main dam exit channel and the auxiliary spillway channel in order to assess potential design and operational issues.

Water surface profiles measured along the permanent and temporary sections of the construction cofferdam were used to design the prototype cofferdam heights for a main dam discharge of 50,000 ft^3/s. To allow for 1 ft of freeboard along the concrete cofferdam section that follows the construction haul road, the wall was designed to have 4 sections at various elevations (elevation 178.5 for 153 ft, elevation 177.5 for 269 ft, elevation 173.0 for 36 ft, and elevation 168.5 for 135 ft). The temporary cofferdam downstream of the auxiliary stilling basin was recommended to be a continuation of the concrete wall at crest elevation 168.5 ft.

Water surface elevations were measured throughout the model when various flows were released from the main dam, the auxiliary spillway, or a combination of the two structures. Flow overtops the right auxiliary stilling basin wall when the main dam releases 300,000 ft^3/s or when the main dam releases 140,000 ft^3/s with an auxiliary spillway release of 160,000 ft^3/s. During all emergency spillway releases from the main dam including the probable maximum flood (PMF), there is insufficient tailwater to cushion the impact of the flip bucket flow on the downstream concrete pad.

Vertical wave heights along the right and left banks in the confluence area were 2 to 8 ft above the normal water surface for total discharges less than 300,000 ft^3/s, except for a discharge of 160,000 ft^3/s from the main dam and no flow from the auxiliary spillway where wave heights reached 10-12 ft along the right bank. Wave heights above 8 ft were observed for total discharges greater than 300,000 ft^3/s with wave heights of up to 28 ft along both banklines during the PMF.

Velocities in the American River channel were in the downstream direction with transverse flow toward the right bank at most locations in the confluence area. A dead zone or recirculation zone existed for most test conditions in the powerplant tailrace. When 25,000 ft^3/s was released from the auxiliary spillway without a release from the main dam, turbulent boiling against the right bank was observed in the confluence area. For flows less than 518,000 ft^3/s, the highest velocity component recorded perpendicular to the right bank was 5.96 ft/s in the

confluence area during an auxiliary spillway release of 60,000 ft^3/s. During a PMF flood of 830,000 ft^3/s, the highest velocity component recorded against the right bank was 13.30 ft/s. Two-dimensional velocities in the river channel can be analyzed by a team of geotechnical and hydraulic engineers to determine if protective measures may be necessary.

In the auxiliary spillway exit channel, downstream velocities increase with increasing discharge from the auxiliary spillway up to 21.4 ft/s for a total discharge of 115,000 ft^3/s with no flow from the main dam. Velocities along the left bankline were generally directed away from the bankline and into the river channel. The greatest velocity impinging on the left bankline was 3.5 ft/s prototype at an auxiliary spillway discharge of 160,000 ft^3/s. Main dam releases, both independently and in conjunction with auxiliary spillway releases, produced lower transverse and downstream velocities in the auxiliary spillway exit channel.

The knob topographic feature remaining from the haul road excavation encounters direct flow impact from main dam discharges of up to 60,000 ft^3/s and is submerged at 90,000 ft^3/s. The maximum measured velocity directly impacting the knob is 21.9 ft/s at a main dam discharge of 25,000 ft^3/s and no flow from the auxiliary structure. When flows are split between the main dam structure and the auxiliary spillway structure, the velocities at the knob are reduced.

The water surface reaches the right bridge pier of the Folsom Lake Crossing Bridge at a river flow rate of about 250,000 ft^3/s. Velocities measured near the right bridge pier can be analyzed by a team of geotechnical and hydraulic engineers to determine if bridge pier scour may be problematic. Flow velocities measured at a point 48 ft upstream and 48 ft to the left of the right bridge pier increased with river flow rate. The downstream velocity was 9.2 ft/s with a 2.8 ft/s transverse velocity toward the right bank at 200,000 ft^3/s and the downstream velocity was 15.6 ft/s with a 6.2 ft/s transverse velocity toward the right bank at 570,000 ft^3/s. Downstream velocities near the toe of the bank by the right bridge pier ranged from 19.0 to 21.1 ft/s and transverse velocities toward the right bank ranged from 1.5 to 4.7 ft/s. Near the toe of the bank, there was little change in downstream and transverse velocities with increased discharge.

Differential loadings on the right auxiliary stilling basin wall were determined from pressure and water surface measurements taken in the model for a range of flow conditions. Data collected with flush-mount pressure transducers and a wave probe arrangement show that the typical design approach of accounting for a full height static differential across the 66-ft-high right auxiliary stilling basin wall appears conservative as the maximum mean values of pressure differentials were of the order of slightly more than one-half of the wall height (approximately 41 ft of water). The highest pressure differential loadings occur near the beginning of the stilling basin at a condition where the flow is still supercritical inside the basin yielding a minimum water level inside the basin and full tailwater on the outside of the basin. Frequency analysis of the differential time series denoted no periodic forcing at any flow condition tested.

With the original baffle blocks installed, the auxiliary stilling basin performance is acceptable without supplemental flows from the main dam for discharges up to 160,000 ft^3/s as long as the tailwater elevation at section 28.6555 is equal to or greater than the elevation predicted by the HEC-RAS study (184.02 ft). Basin performance quickly deteriorates for tailwater elevations only 1 or 2 ft lower at a discharge of 160,000 ft^3/s. During 115,000 ft^3/s with the original baffle blocks installed, the basin performance remains acceptable even for tailwater elevations up to 7 ft less than the HEC-RAS prediction. The performance of the auxiliary spillway stilling basin should be monitored during flood operations. If the stilling basin performance becomes unacceptable, main dam releases should be increased with a corresponding decrease in auxiliary spillway releases.

The stepped spillway chute is necessary to obtain acceptable performance of the stilling basin for the current stilling basin geometry. A smooth chute cannot be used unless structural modifications are made to the stilling basin design. With a smooth chute in place and with no flow from the main dam, performance of the auxiliary stilling basin was progressively worse with increasing discharge. At 160,000 ft^3/s, a very large rooster tail was produced as the flow directly impacted the baffle blocks and deflected upward. When the tailwater elevation was raised by 10 ft to elevation 194.0 ft, the stilling basin began to perform properly.

If the auxiliary baffle blocks were to fail and be carried out of the stilling basin, the auxiliary stilling basin performance would be acceptable during a maximum auxiliary discharge as long as at least 120,000 ft^3/s is also released from the main dam. If the discharge from the main dam is less than 120,000 ft^3/s, the tailwater is too low for the auxiliary stilling basin to contain the hydraulic jump.

Velocity profiles were measured along the auxiliary spillway stepped chute at steps 1, 30, and 64 during the design discharge of 135,000 ft^3/s. The stepped chute dissipated 52.4% of the energy available from the top of the chute to the stilling basin floor. This value compares favorably to the average energy dissipation value of 53% (range 48-58%) measured in the SAFL 1:26-scale physical model (Lueker et al., 2008) and the predicted energy dissipation value of 60.6% from the FLOW-3D numerical model (Kubitschek, 2008).

A new baffle block design comprised of seven supercavitating blocks and an upstream ramp was installed in the model after tests in a low ambient pressure chamber showed low cavitation potential. The baffle blocks performed adequately, but the ramp projected the flow upward in the basin. Four alternate ramp configurations were tested in the model. A 4-ft-high by 12-ft-long ramp placed between the blocks produced good basin performance with a mild change in water surface elevation and a small standing wave at the end sill. Energy dissipation was adequate and velocities measured downstream from the end sill in the exit channel were similar to velocities measured with the original baffle blocks. It appears that the performance of the supercavitating baffle blocks and ramp configuration is less dependent on tailwater depth than the original baffle block configuration. This finding warrants further investigation in a future study.

Elevation Datum

Folsom Dam was originally designed and constructed using the National Geodetic Vertical Datum of 1929 (NGVD29) as an elevation reference. Design and construction documents for the current Joint Federal Project (JFP) at Folsom Dam are being prepared using the North American Vertical Datum of 1988 (NAVD88) as an elevation reference. In the vicinity of Folsom Dam, the difference in numerical value between the two elevation references is approximately 2.34 ft (i.e., 0 ft NGVD29 equals 2.34 ft NAVD88). This difference in reference elevation between the original project drawings and the JFP drawings presents a significant potential for confusion. At the request of the U.S. Army Corps of Engineers (COE), all hydraulic modeling and reporting activities related to the JFP are to be done using the original NGVD29 elevation reference. Thus, all elevations in this document, unless otherwise noted, are referenced to the NGVD29 as used in the original project design documents and drawings.

Introduction

Project Background

Folsom Dam is located on the American River about 20 miles upstream from Sacramento, California (figure 1). The dam was designed and built by the COE and transferred to the Bureau of Reclamation (Reclamation) for operation and maintenance in 1956. The existing dam and spillway are comprised of a 340-ft high and 1,400-ft long concrete gravity section flanked on each side by earthfill wing dams that extend from the gravity section to the abutments. In addition to the main section and wing dams, there is one auxiliary dam and eight smaller earthfill dikes that impound a reservoir of 1,010,000 acre-feet. The dam is operated for municipal and agricultural water supply purposes and to provide flood control protection for the city of Sacramento.

Figure 1. Location map of Folsom Dam and Lake upstream from Sacramento, California.

The gravity section of the dam includes an ogee crest at elevation 418 ft for both the service and emergency spillways (figure 2). Releases are controlled using five 50-ft-tall by 42-ft-wide radial gates for the service spillway and three 53-ft-tall by 42-ft-wide radial gates for the adjacent emergency spillway. The service spillway discharges into a 242-ft-wide stilling basin at invert elevation 115 ft while the emergency spillway discharges from a flip bucket into a plunge-pool energy dissipator. A powerplant is located along the right side of the gravity section to which flow is delivered via three 15-ft diameter penstocks. The dam is also equipped with eight outlet conduits through the gravity section, four outlets at elevation 280 ft (upper level) and four outlets at 210 ft (lower level), each having 5-ft by 9-ft slide gates. The downstream ends of the conduits daylight on the service spillway face, but during large floods that produce spillway operation, releases through the outlets are limited. The primary contribution to overall release capacity during flood routing is from the service and emergency spillways.

Figure 2. Overview of Folsom Dam.

Dam safety responsibility for Folsom Dam rests with the Bureau of Reclamation. In 2006, Reclamation made a new assessment of the probable maximum flood (PMF) at the dam site (Reclamation, 2006) that accounts for changes in upstream land use in the past 60 years and uses flood records obtained since the completion of the dam. Subsequent routing studies show that the existing discharge facilities at Folsom Dam are not capable of safely passing the new PMF. The design and construction of a new spillway or outlet system is thus needed to address this dam safety deficiency. One such alternative for a fuse plug controlled spillway was studied in a physical hydraulic model by Reclamation's Hydraulic Investigations and Laboratory Services Group in Denver, Colorado in 2007.

Separately, beginning in about 1999, the COE and the Sacramento Area Flood Control Agency (SAFCA) studied modifications to Folsom Dam that could increase flood control protection along the American River. Current release capacity of the eight outlet gates through the dam is significantly less than the channel capacity of the American River downstream from Folsom Dam, within the constraints of existing levees along the river. Additional release capacity at reservoir levels below the spillway crest would allow releases during the rising limb of a flood event to approach the river channel capacity, thereby allowing the early release of a larger percentage of the volume of an incoming flood. This would increase the size of the flood that could be successfully accommodated through controlled releases and flood control storage in the reservoir. The objective has been to add facilities capable of routing a 200-year flood event through the reservoir while keeping the reservoir elevation below the crest of the service and emergency spillways and not releasing flows that would overtop levees along the downstream river channel. One proposal to achieve this objective was to increase the size, number, and capacity of the upper and lower level outlets through the dam (Frizell, 2004).

The Joint Federal Project (JPF) combines these independent efforts into one project that meets both Reclamation's probable maximum flood criteria and the COE's flood damage reduction goals. Under the JFP, the maximum pool elevation during passage of the PMF was set at elevation 477.5 ft. To achieve at least 3 ft of freeboard during the PMF, the total discharge requirement was 818,000 ft^3/s from the Folsom Dam outlet structures. This discharge value was based on flood routings similar to those published in the latest flood routing study (Reclamation, 2006).

To obtain the required discharge capacity, the JFP includes the construction of a new auxiliary spillway near the left abutment of the main dam embankment (figure 3). The auxiliary spillway is comprised of a control structure that houses six 23-ft-wide by 34-ft-high submerged tainter gates (top-seal radial gates) at invert elevation 368.0 ft, an approach channel from the reservoir to the control structure, a 169-ft-wide rectangular, concrete lined chute, a stilling basin, and an exit channel to return flood discharges to the American River. The downstream section of the spillway chute from Station 32+00 to Station 38+82 was designed as a stepped chute to dissipate some energy before flow entered the stilling basin.

Figure 3. Artist's rendering of the new auxiliary spillway structure to the left of the main dam spillway structure. The new Folsom Lake Crossing Bridge across the American River is shown just downstream from the confluence area.

Overview of Model Studies

This report documents studies of a 1:48-scale physical model of the main dam spillway, the new auxiliary spillway and stilling basin, the confluence area of the two exit channels, and a portion of the downstream river. This model was constructed and tested by Reclamation's Hydraulic Investigations and Laboratory Services Group at the Technical Service Center in Denver, Colorado from 2007 to 2009.

In addition to the 1:48-scale model that encompasses most of the project area, several other physical and numerical model studies have been conducted in Reclamation's hydraulics laboratory to study specific components of the project. Improving estimates of the maximum gate-controlled discharge capacity of the existing service and emergency spillway structure was critical to confirming the required release capacity from the new auxiliary spillway. A 1:36-scale physical sectional model was installed in a 4-ft-wide laboratory flume with two full spillway gates and two partial gates. In addition, a three-dimensional (3D) computational fluid dynamics (CFD) model was developed to simulate a main dam spillway release using the commercial computer model FLOW-3D. Results showed that a maximum vertical gate opening of 40 ft maintains gate control at a reservoir water surface elevation 477.5 ft. At this gate opening, the main dam discharge capacity is 518,000 ft^3/s based on physical model measurements (Frizell et al., 2008).

The 1:48-scale physical model described in this report was also used for discharge verification tests in conjunction with these studies. A temporary symmetry wall was installed within the headbox of the main dam spillway to split the spillway in half (4 gates). This was done in part to provide a direct comparison to the numerical modeling which had considered a similar half-section of the spillway and also because the discharge capacity in the physical model was not large enough to supply water to all 8 gates at that time. Later installation of a large capacity portable pump allowed for operation of all 8 gates. This model test provided discharge curve information and verification of when the particular gate openings were in the transition region between free flow and gated flow.

Several studies focusing on details of the new auxiliary spillway stepped chute and stilling basin were also conducted by Reclamation. A two-dimensional (2D) CFD numerical model of the auxiliary spillway stepped chute was developed to obtain prototype hydraulic performance assessments of velocity profiles, energy dissipation, cavitation potential, and abrasion damage potential on the stepped chute (Kubitschek, 2008). A physical model of the auxiliary stilling basin baffle blocks was also tested in Reclamation's low ambient pressure chamber to study the cavitation performance of the blocks. A new supercavitating baffle block and ramp were designed from the cavitation studies (Frizell, 2009a). An evaluation of cavitation on the spillway chute steps is currently underway in the low ambient pressure chamber (Frizell, 2009b).

Two other larger-scale studies of auxiliary spillway components were carried out at other laboratories. First, a 1:30-scale physical model of the auxiliary spillway approach channel and gated control structure was tested at Utah State University's Utah Water Research Laboratory (Rahmeyer et al., 2009) in Logan, Utah. The model study showed that the final geometry of the auxiliary spillway approach channel and gated structure has a discharge capacity of approximately 312,000 ft^3/s at the specified maximum reservoir water surface elevation of 477.5 ft (Rahmeyer et al., 2009). Second, a 1:26-scale physical model of the auxiliary spillway chute (downstream from the control structure) and auxiliary stilling basin was tested at the University of Minnesota's St. Anthony Falls Laboratory (SAFL) in Minneapolis, Minnesota (Lueker et al., 2008). The model was used to examine the design and performance of the stepped spillway chute and stilling basin under a range of operating conditions. This model was unable to consider the influence of three-dimensional flow conditions in the stilling basin area produced by the flow from the existing service and emergency spillways.

Model Objectives

The primary purpose of the 1:48-scale Folsom confluence model study was to evaluate the three-dimensional characteristics of the flow in the vicinity of the confluence of the main dam exit channel and the auxiliary spillway channel, particularly with regard to energy dissipation and the interaction between flow from the primary and auxiliary spillways. Both design and operational issues were addressed with the model study.

The following study objectives were identified:

1.) Observe and document overall flow conditions in the American River for an array of discharge scenarios, including flow releases from the main dam only, auxiliary spillway only, and combined operations. Note whether main dam flows passing over the right stilling basin wall affect basin performance.

2.) Determine water surface elevations for an array of flow conditions.

3.) Measure velocities at various points in the American River, auxiliary spillway exit channel, and near the topographic knob feature from the haul road excavation. Note the location of eddies or turbulent conditions in the converging flow zones or in the main dam spillway or auxiliary spillway exit channels.

4.) Evaluate flow conditions and potential erosion areas at the new Folsom Lake Crossing Bridge downstream from Folsom Dam on the American River.

5.) Determine hydraulic loadings on the auxiliary spillway stilling basin walls due to flows in the auxiliary spillway and flow impingement on the right stilling basin wall from main dam spillway releases.

Additional items were added to the scope of work as the study progressed:

1.) Collect dynamic pressure data on the right auxiliary stilling basin wall to ensure that the wall design can withstand the hydrodynamic loadings (using pressure transducers rather than the wave probe instrument used in Study Objective 5 above).

2.) Examine influence of the permanent cofferdam along the haul road and the temporary cofferdam downstream from the auxiliary stilling basin end sill.

3.) Determine the flow condition at which the auxiliary stilling basin no longer performs satisfactorily. Determine the sensitivity of the auxiliary stilling basin performance to changes in tailwater elevation.

4.) Compare the auxiliary stilling basin performance with a stepped spillway chute and a smooth spillway chute.

5.) Remove baffle blocks from the auxiliary stilling basin to examine stilling basin performance during a hypothetical situation where the baffle blocks have failed and are assumed to be washed downstream.

6.) Measure velocity profiles along the stepped portion of the auxiliary spillway chute and determine total energy dissipation along the stepped chute. Compare results to the SAFL 1:26-scale physical model and Reclamation's FLOW-3D numerical model results.

7.) Examine energy dissipation produced by several supercavitating baffle block and ramp combinations. Determine which block/ramp configuration tested in Reclamation's low ambient pressure chamber produces good energy dissipation while maintaining minimal cavitation damage potential.

Model Description

Model Scale

A physical hydraulic model of Folsom Dam including the main dam spillway structure, the auxiliary spillway structure, and the confluence area where the spillway releases converge was constructed in Reclamation's hydraulics laboratory in Denver, Colorado in 2007. In order to include all of the desired model features in the floor space available, the physical hydraulic model was built at a 1:48 geometric scale. Similitude between the model and the prototype is achieved when the ratios of the major forces controlling the physical processes are kept equal in the model and prototype. Since gravitational and inertial forces dominate open channel flow, Froude-scale similitude was used to establish a kinematic relationship between the model and the prototype. The Froude number is

$$F_r = \frac{v}{\sqrt{gd}}$$

where v = velocity, g = gravitational acceleration, and d = flow depth. When Froude-scale modeling is used, the following relationships exist between the model and prototype where the r subscript refers to the ratio of the prototype to the model:

Length ratio:	L_r = 1:48
Pressure ratio:	P_r = 1:48
Velocity ratio:	$V_r = L_r^{1/2} = (48)^{1/2} = 6.93$
Time ratio:	$T_r = L_r^{1/2} = (48)^{1/2} = 6.93$
Discharge ratio:	$Q_r = L_r^{5/2} = (48)^{5/2} = 15{,}962.58$

Model Features

The 1:48-scale physical model included a 25-ft-wide by 22-ft-long by 7-ft-high rectangular headbox for the main dam and a 12-ft-wide by 11-ft-long by 7-ft-high rectangular headbox for the auxiliary spillway structure, both elevated 3.7 ft from the laboratory floor. A large nonrectangular tailbox was constructed to follow the course of the American River channel with a return channel to carry flows back to the laboratory sump (figures 4 and 5).

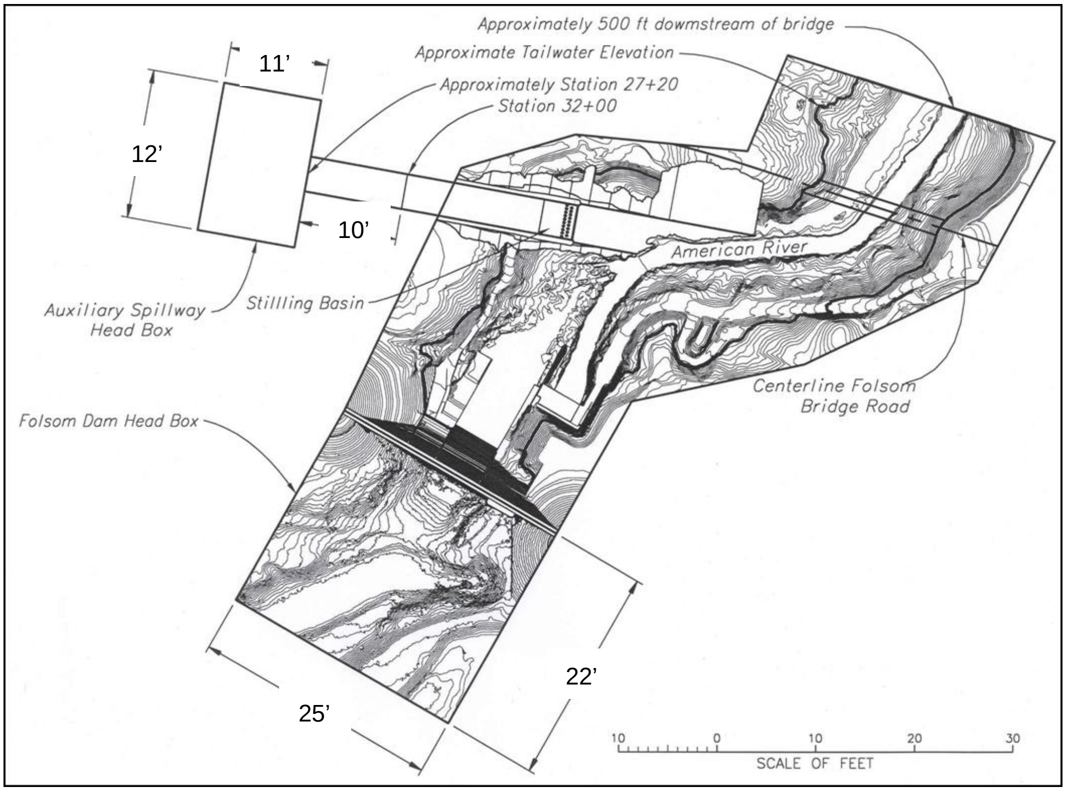

Figure 4. General layout of the 1:48-scale Folsom Dam and auxiliary spillway confluence physical hydraulic model.

Figure 5. 1:48-scale physical hydraulic model of the main Folsom Dam, auxiliary spillway, and confluence area where flows combine in the American River.

Main Dam

Two 12-inch PVC inlet pipes delivered water from the laboratory flow distribution system to the bottom of the main dam headbox while one 16-inch steel inlet pipe delivered water from the portable pump over the top of the main dam headbox. A vertical rock baffle was constructed to calm the energetic flow entering the headbox. Approximately 1,000 ft of approach flow in Folsom Lake was modeled upstream from the main Folsom Dam. Bathymetry in the headbox was derived from acoustic soundings of Folsom Lake in 2007.

Model features on the main dam structure included the roadway bridge, a high density foam ogee crest with 5 adjustable metal radial gates on the service spillway and 3 adjustable metal radial gates at the emergency spillway at crest elevation 418 ft. The spillway face was constructed of ¾-inch marine-grade plywood. Flow from the service spillway entered a marine-grade plywood sloped stilling basin with an end sill while flow from the emergency spillway entered a flip bucket with a plunge-pool energy dissipator. Low flow discharge facilities including the tiered outlet works conduits and the powerplant turbines were not modeled. The general outline of the powerplant building was built from marine-grade plywood in the model to determine how release flows affect the powerplant submergence. Model construction drawings and photographs of the main dam are shown in figures 6-10.

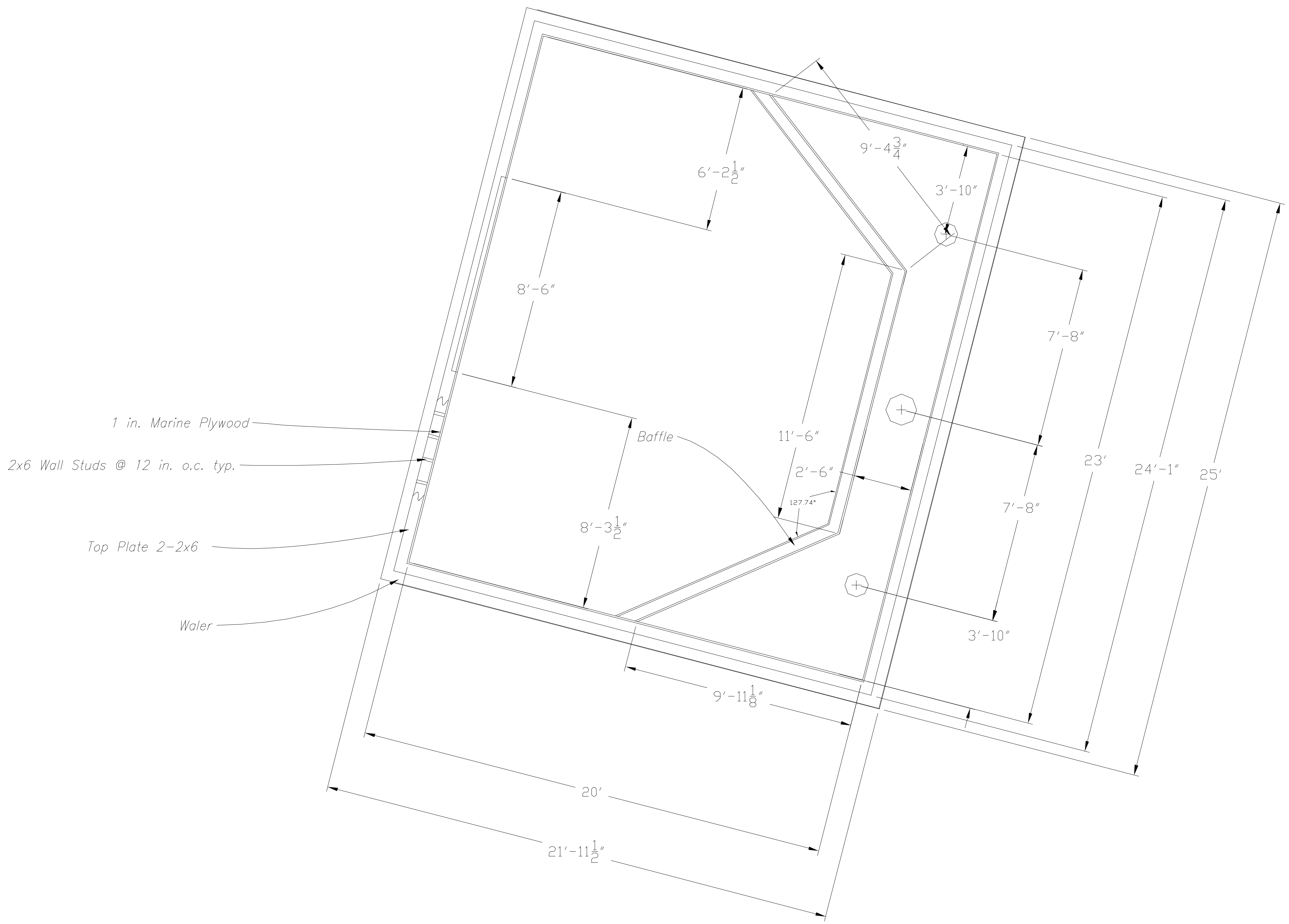

Figure 6. Model construction drawing: Plan view of main dam headbox with inlet pipes, rock baffle, and spillway cut-out.

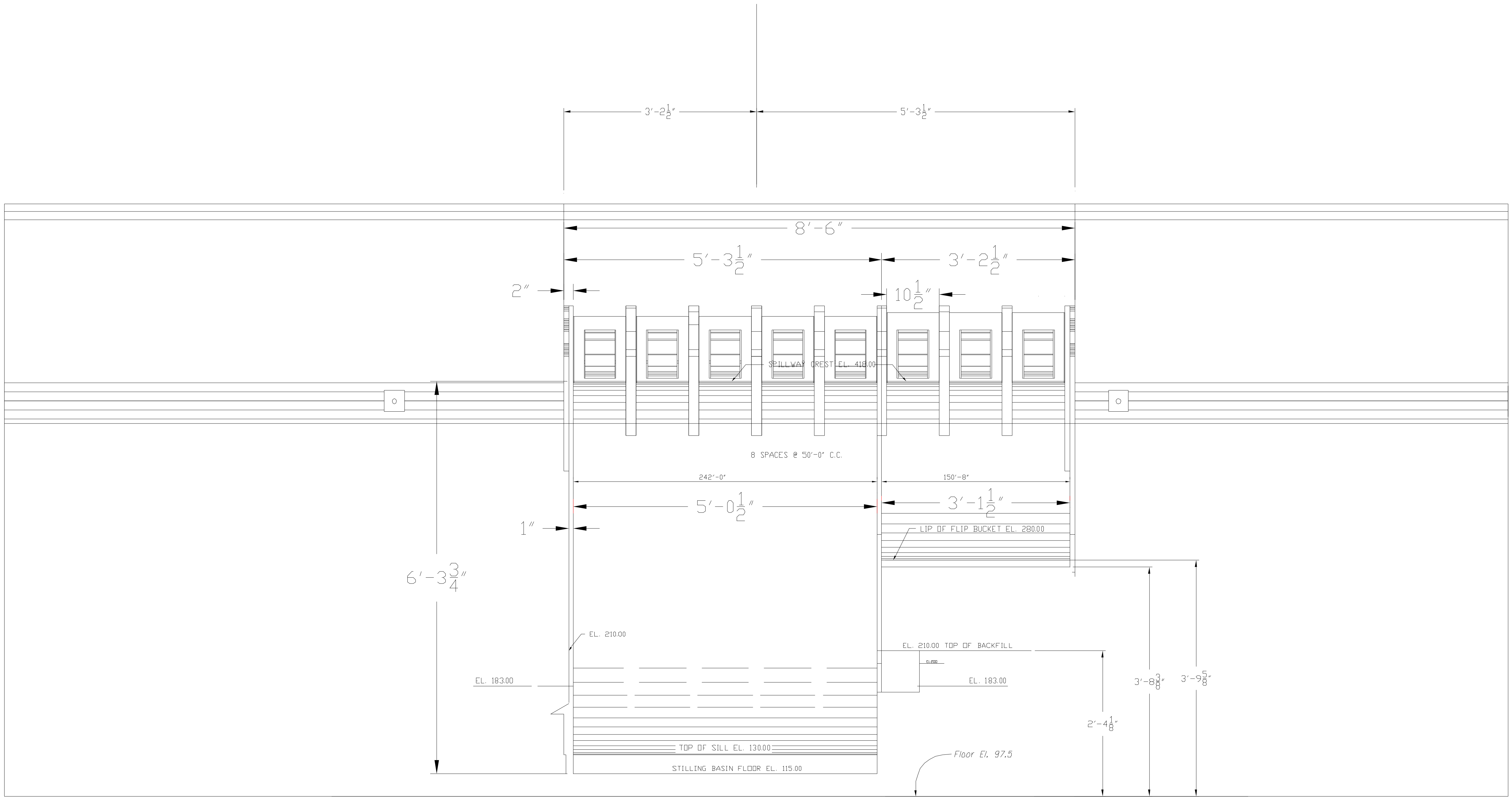

Figure 7. Model construction drawing: Profile view of main dam spillway looking upstream.

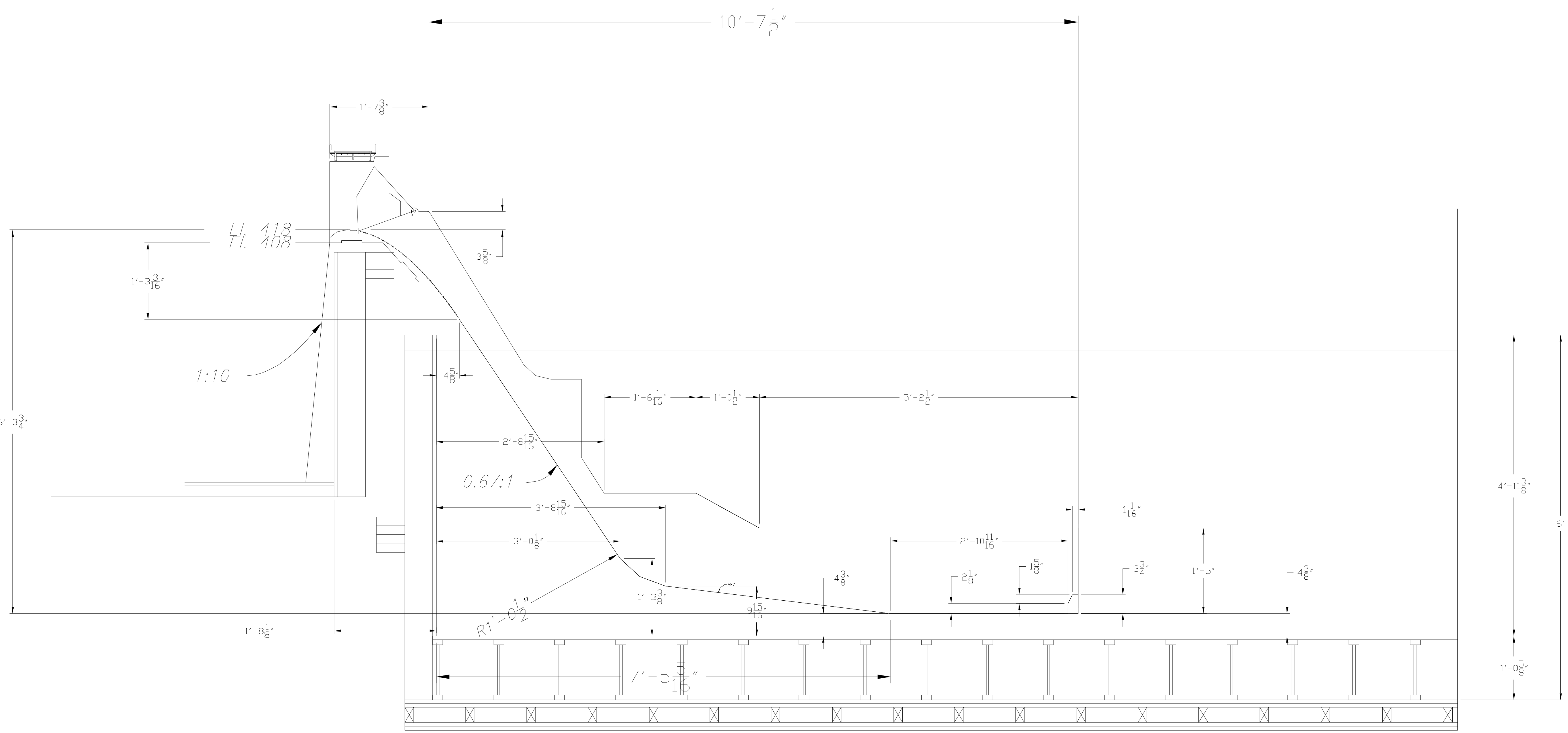

Figure 8. Model construction drawing: Cross-sectional profile view of main dam spillway and stilling basin.

Figure 9. Main dam headbox with vertical rock baffle and topographic contours during model construction.

Figure 10. Main dam bridge deck, spillway radial gates, spillway, stilling basin, and flip bucket energy dissipator.

Auxiliary Spillway and Stilling Basin

Two 12-inch PVC inlet pipes delivered water to the top of the auxiliary spillway headbox from the laboratory system. Flow passed into the headbox through a vertical rock baffle. Elliptical curves made of sheet metal were used to transition flow into the spillway chute. Horizontally curved inlet structures were installed on both sides of the headbox and a vertically curved false floor was installed from the bottom of the box. Since the auxiliary control structure was not modeled, a vertical slide gate was used to control the flow release from the auxiliary spillway headbox. Approximately 500 ft of smooth chute were modeled upstream from the start of the stepped spillway portion of the chute at station 32+00. The flow depth on the spillway chute at station 32+00 was measured with a vertical point gage to match the flow depth measured in the physical hydraulic model of the spillway chute at SAFL during specific flow releases (Lueker et al., 2008).

Structural features of the main dam and auxiliary spillway were constructed from as-built and design drawings. Critical elevations were surveyed with a level to ensure proper placement. The auxiliary spillway chute and stilling basin were constructed of ¾-inch marine-grade plywood. On the smooth section of the spillway chute, junctions between plywood sheets were smoothly transitioned with silicone to avoid flow separation. The stepped portion of the spillway was carefully constructed and surveyed with a level to ensure that steps were installed at the proper elevations.

Results from the hydraulic model at SAFL showed that the initial stilling basin design could not contain the hydraulic jump at the design discharge of 135,000 ft^3/s. The stilling basin was lengthened by 80 ft, the two rows of 9-ft tall baffle blocks were replaced with 16-ft-high baffle blocks, and the 4.5-ft-high solid end sill was replaced with a 15-ft high solid end sill (Lueker et al., 2008). In the 1:48-scale model, seven baffle blocks (16 ft high by 12 ft wide by 19 ft deep with a 1:1 sloping back face and 3 ft flat top) were installed in the stilling basin in one row at Station 39+71 with the 15-ft-high end sill installed from Station 41+00 to 41+32. Model construction drawings and photographs of the auxiliary spillway structure are shown in figures 11-15.

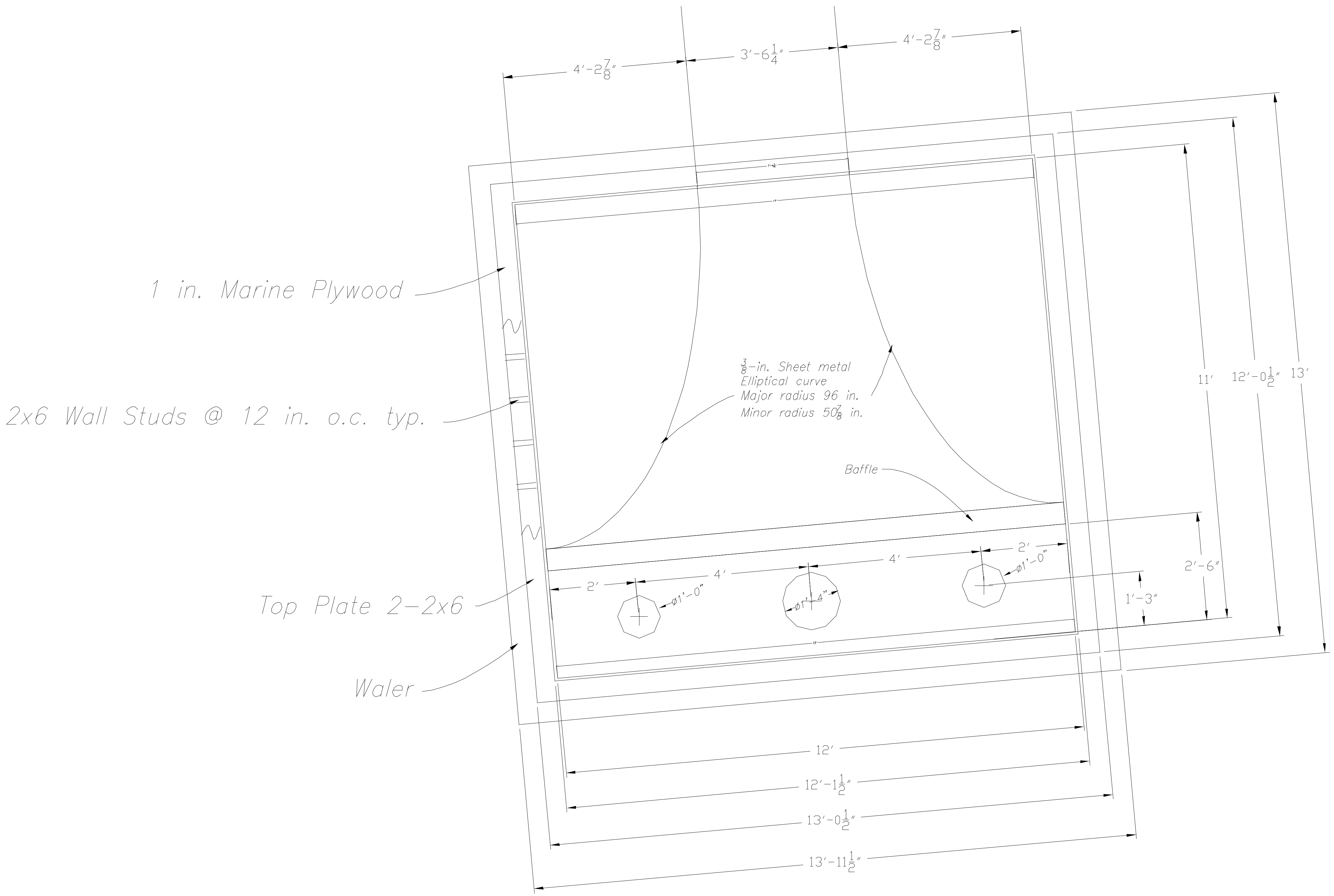

Figure 11. Model construction drawing: Plan view of auxiliary spillway headbox with inlet pipes and curved transition walls.

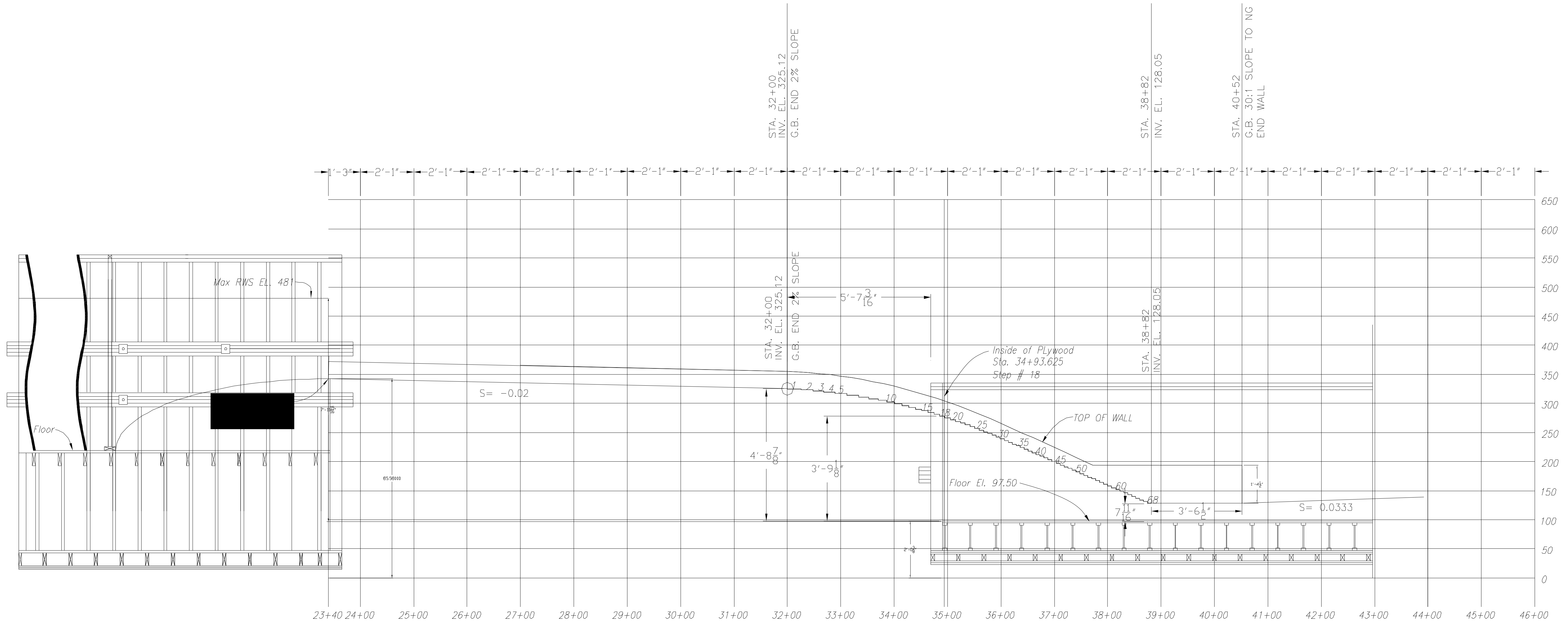

Figure 12. Model construction drawing: Cross-sectional profile view of auxiliary headbox, spillway chute, and stilling basin.

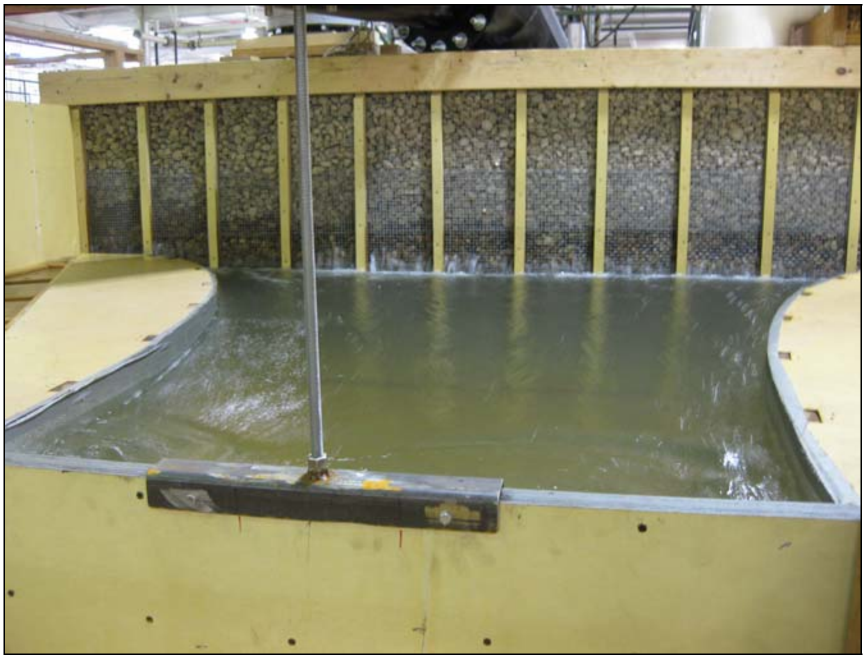

Figure 13. Looking upstream at the auxiliary spillway headbox with rock baffle, curved transition walls, and slide gate.

Figure 14. Auxiliary spillway stepped chute and stilling basin.

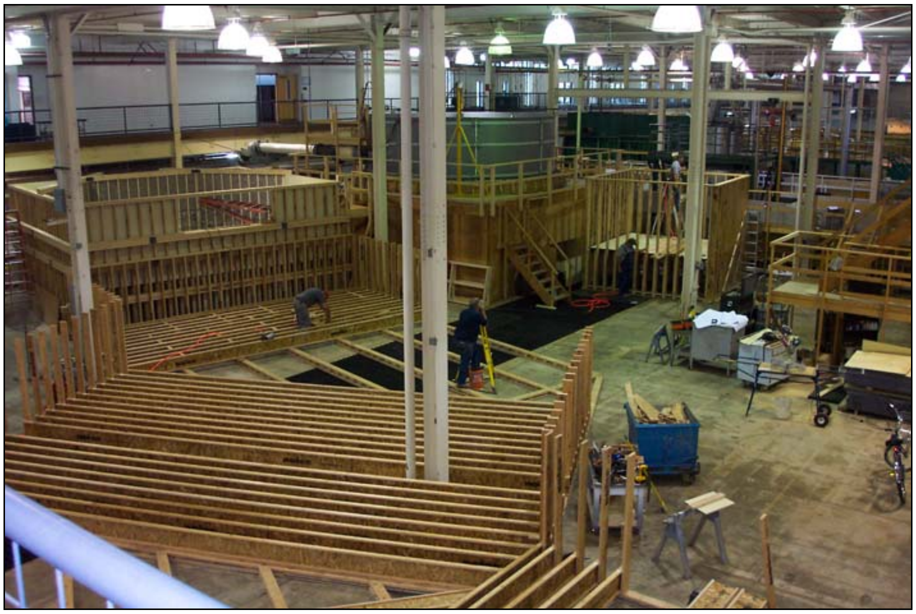

Figure 15. Surveying critical elevations in the auxiliary spillway headbox.

Confluence Area and American River Channel

The model tailbox contained topography of the main dam and auxiliary spillway exit channels, the confluence area, and the American River channel. Topography near Folsom Dam was collected by Reclamation's Mid-Pacific Region in 2007 via aerial photography. The orthophotos were converted to AutoCAD drawings and re-contoured to NGVD29 datum for use in the hydraulic model. Excavation required for the auxiliary spillway and stilling basin was generated by Reclamation designers and confirmed by the COE design team. The topography in the model was also modified to incorporate the 66-ft-wide construction haul road extending from the left bank downstream from the main dam to the auxiliary stilling basin exit channel. Excavation for the haul road leaves a knob-like topographic feature near the confluence of the two hydraulic structures (figure 16).

Topographic contours were cut with a CNC router and installed at specified model elevations. Wire mesh was placed over the contours to support concrete placement (figure 17). Since high operational discharges were the primary focus in this model study, the topography of the river channel, including topographic features such as the drop in elevation from the main dam and auxiliary spillway exit channels to the river, the channel shape, and river bend downstream from the confluence area were deemed to be dominant over the roughness of the surface by the model designers. Therefore, the prototype channel roughness was not directly replicated in the model.

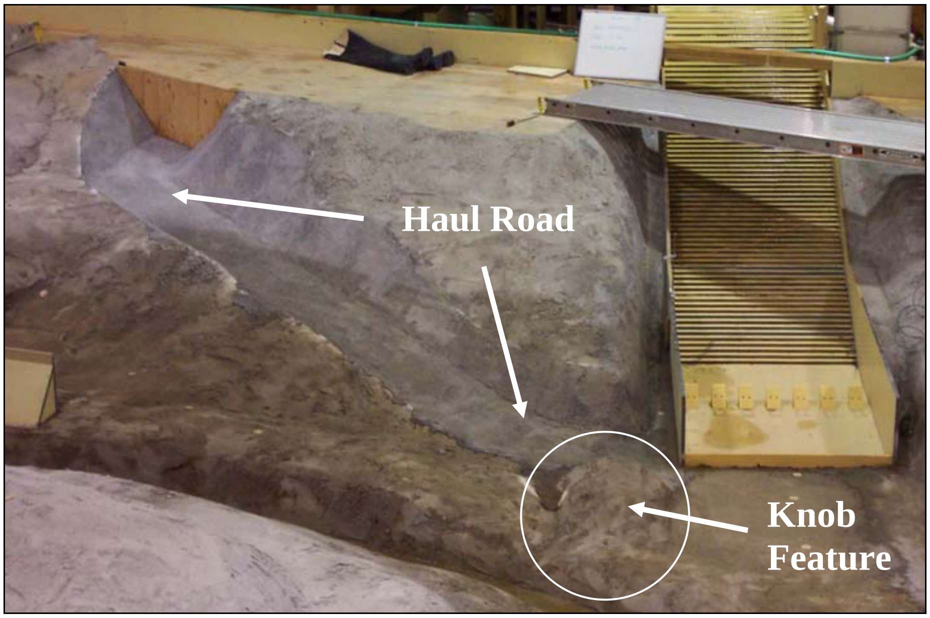

Figure 16. Topography modified to include the construction haul road. A knob-like topographic feature remains after excavation.

Figure 17. Installing contours for the tailbox topography.

The cofferdam for the construction of the auxiliary spillway consists of a government-designed reinforced concrete wall section along the construction haul road and a contractor-designed temporary cofferdam section downstream from the auxiliary stilling basin end sill. During model testing, the design team decided that the reinforced concrete wall section will be left in place after the JFP construction is completed. The wall will deflect normal discharges from the main dam spillway and outlet works away from the auxiliary stilling basin wall and may minimize material transport along the access road. Leaving the concrete wall in place will also reduce the project cost associated with removal and disposal of the reinforced concrete. The entire portion of the contractor-designed cofferdam will be removed after the auxiliary spillway is constructed.

Model testing was performed to optimize the cofferdam height. A marine-grade plywood wall was installed in the model to represent the final alignment and height (figure 18). The cofferdam wall was left in place for all model tests except for the wave probe and dynamic pressure tests on the right stilling basin wall. For these tests, it was desired to simulate a worst case scenario where the cofferdam wall failed at some future time, allowing increased flow impact against the auxiliary stilling basin wall.

Figure 18. Permanent reinforced concrete cofferdam wall along the haul road.

The bridge piers of the new Folsom Lake Crossing Bridge across the American River were installed in the downstream section of the model (figures 19 and 20). Topography extended for approximately 500 ft downstream from the new bridge. The tailwater was controlled with a tailgate and tailboards at the downstream end of the model.

Figure 19. Bridge piers of Folsom Lake Crossing Bridge installed in the physical model.

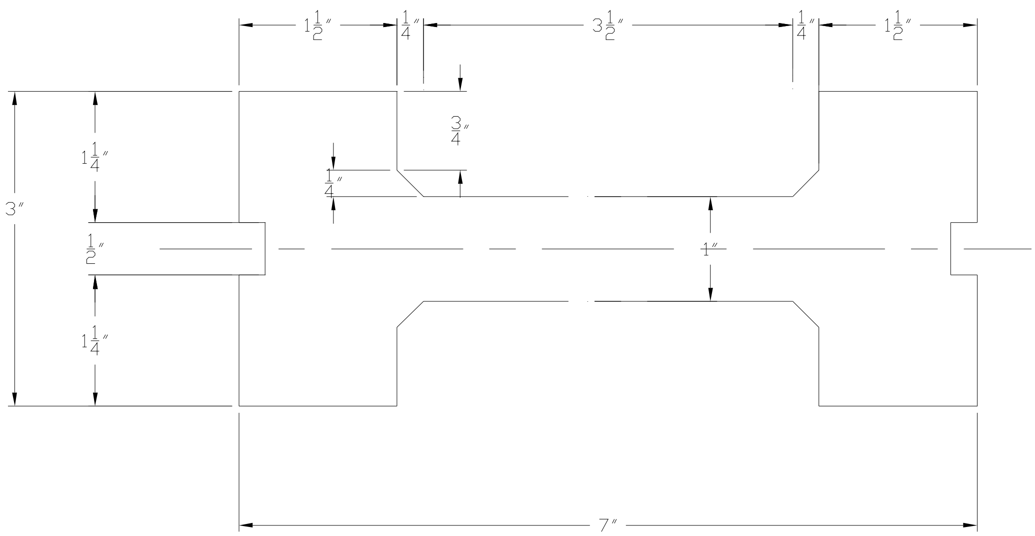

Figure 20. Model construction drawing of bridge pier in plan view.

Instrumentation

Flow Measurement

A 240,000-gallon storage reservoir under the laboratory floor supplied water for the hydraulic model through an automated flow delivery and measurement system. Four 100-150 hp variable-speed centrifugal pumps located in the pump pits at the north and south ends of the storage reservoir delivered water to the 12-inch supply line that runs around the perimeter of the laboratory. The permanent laboratory pumps can operate singly, in series, or in parallel to deliver up to about 45 ft^3/s.

Laboratory venturi meters from 3 to 14 inches in diameter provided flow measurement between 0.1 and 20 ft^3/s. A 44,000 pound volumetric/weigh tank facility is used to calibrate the laboratory venturi meters in place at regular intervals to an accuracy of $\pm 0.25\%$. A state-of-the-art laboratory control and data acquisition system outputs flow measurement data on a LCD screen. A low-head portable vertical turbine pump with approximately 25 ft^3/s output capacity was purchased to obtain additional flow capacity for this model. In conjunction with the permanent laboratory pumps, the PMF could be simulated. There was no direct measurement of discharge from the portable pump, so the gated stage-discharge relationship for the main dam (Frizell et al., 2008) was used to set the appropriate supplemental discharge from the portable pump. The reservoir elevation in the model was read with a stilling well and hook gage equipped with a vernier scale, allowing the water level to be read to the nearest 0.001 ft. An error of ± 0.0005 ft model (± 0.024 ft prototype) in the water level corresponds to a change of ± 344.5 ft^3/s in the prototype discharge rating or $\pm 0.07\%$ of the 518,000 ft^3/s PMF discharge.

Water Surface Elevations

Piezometer taps were used to measure water surface elevations at 53 locations in the model. All water surface elevations and water depths in this report are listed in prototype units unless otherwise specified. Tap locations are shown in figure 21 and tap spatial coordinates and invert elevations are reported in table 1.

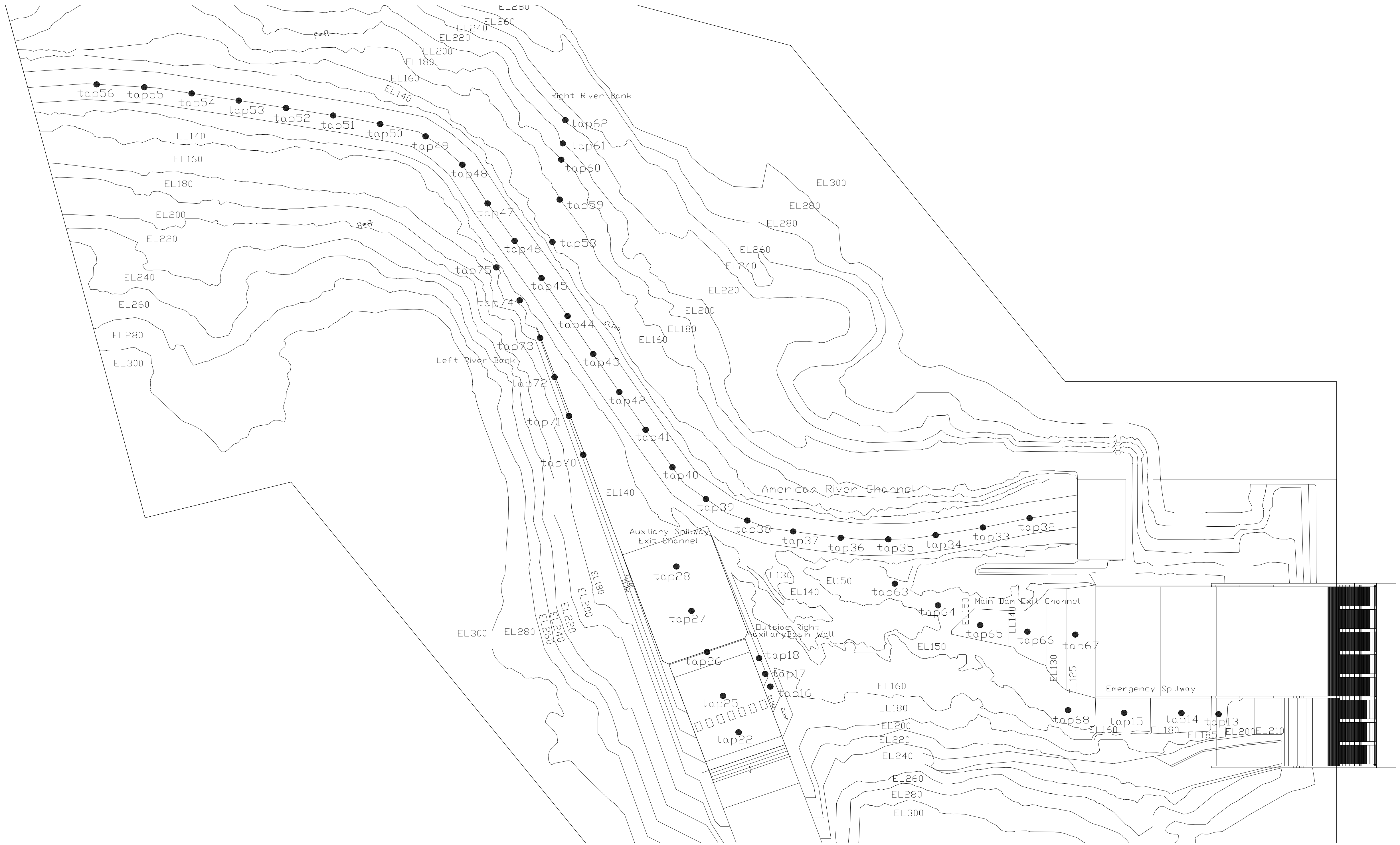

Figure 21. Locations of piezometer taps in the model.

Table 1. Spatial coordinates and invert elevations of model piezometer taps.

Tap No.	Northing	Easting	Elevation (ft)
13	2,020,320.38	6,802,295.81	192.5
14	2,020,253.88	6,802,257.09	180.0
15	2,020,151.16	6,802,195.32	170.0
16	2,019,523.35	6,801,841.22	131.0
17	2,019,528.68	6,801,811.69	124.5
18	2,019,534.00	6,801,782.17	124.0
22	2,019,427.67	6,801,828.05	128.0
25	2,019,443.00	6,801,743.02	128.0
26	2,019,460.71	6,801,644.80	131.0
27	2,019,478.46	6,801,546.38	133.5
28	2,019,496.20	6,801,447.97	137.0
32	2,020,193.24	6,801,724.19	115
33	2,020,098.10	6,801,693.44	115
34	2,020,004.24	6,801,658.93	115
35	2,019,913.38	6,801,617.74	115
36	2,019,829.13	6,801,564.30	114.9
37	2,019,749.61	6,801,503.68	114.9
38	2,019,678.12	6,801,434.63	114.8
39	2,019,627.13	6,801,349.63	114.8
40	2,019,601.00	6,801,254.59	114.7
41	2,019,593.66	6,801,154.86	114.7
42	2,019,587.51	6,801,055.05	114.6
43	2,019,582.11	6,800,955.19	114.5
44	2,019,577.19	6,800,855.32	114.5
45	2,019,571.64	6,800,755.48	114.4
46	2,019,563.78	6,800,655.79	114.4
47	2,019,556.10	6,800,556.09	114.3
48	2,019,552.88	6,800,456.14	114.3
49	2,019,517.47	6,800,362.94	114.2
50	2,019,448.47	6,800,292.02	114.1
51	2,019,372.67	6,800,226.83	114.1
52	2,019,295.40	6,800,163.35	114.0
53	2,019,218.00	6,800,100.02	114.0
54	2,019,140.22	6,800,037.17	113.9
55	2,019,061.19	6,799,976.03	113.9
56	2,018,977.79	6,799,920.86	113.8
58	2,019,631.52	6,800,697.46	173.0
59	2,019,693.97	6,800,643.85	192.5
60	2,019,737.55	6,800,547.14	213.0
61	2,019,756.95	6,800,510.29	231.5
62	2,019,788.82	6,800,475.40	250.0
63	2,019,876.39	6,801,709.55	159.0
64	2,019,931.04	6,801,793.77	156.0
65	2,019,985.68	6,801,877.99	146.0
66	2,020,064.03	6,801,938.41	133.5
67	2,020,148.47	6,801,992.71	125.5

68	2,020,051.68	6,802,132.66	147.0
70	2,019,453.30	6,801,137.62	157.5
71	2,019,469.84	6,801,048.00	157.5
72	2,019,486.71	6,800,958.45	157.5
73	2,019,503.56	6,800,868.89	157.5
74	2,019,509.27	6,800,778.36	155
75	2,019,501.18	6,800,687.63	152

Care was taken to ensure that the piezometer taps were flush with the model surface. Clear Poly-Flow tubing was run from a metal fitting at the model surface to a manometer board where readings of water surface elevation could be taken. Water levels were visually averaged to the nearest 0.001 ft model (0.048 ft prototype). Due to the large number of piezometer taps, a pressurized manifold system was devised to bleed air from all taps on a manometer board at one time before data were collected (figures 22 and 23).

A piezometer tap was placed in the sidewall of the main dam headbox to measure the reservoir water surface elevation (figure 24). Clear Poly-Flow tubing connected the tap to a stilling well on the outside of the headbox. The stilling well could be read to the nearest 0.001 ft in the model (0.048 ft prototype).

Figure 22. Piezometer taps were used to measure water surface elevations throughout the model. The piezometer taps shown in this photograph are attached to the flat discs downstream from the stilling basin end sill.

Figure 23. Pressurized manifold bleed system installed on manometer boards.

Figure 24. Stilling well used to measure reservoir water surface elevation.

Tailwater Elevations

Tailwater elevations for specific river flows were provided by a HEC-RAS numerical river model run by the COE. Tailwater curves were provided for two cases: one in which the remnants of an old stone dam downstream from the confluence area were left in place and one in which the old dam remnants were removed from the river banks (figure 25). The old dam is located approximately 1,500 ft downstream of the new Folsom Lake Crossing Bridge. Since the remnants of the old dam are high on the banklines, the influence of the old dam on the tailwater elevation would not be seen until the American River discharge was approximately 450,000 ft^3/s (figure 26).

The cross-section 28.6555 in the HEC-RAS model equated to a river location midway between piezometric taps 54 and 55 in the physical model. The tailwater elevation was set by visually averaging the water surface elevations on the manometer board for taps 54 and 55 to ± 0.001 ft, corresponding to ± 0.048 ft in the prototype. The tailgate was raised or lowered as needed to set the appropriate tailwater elevation for the flow condition.

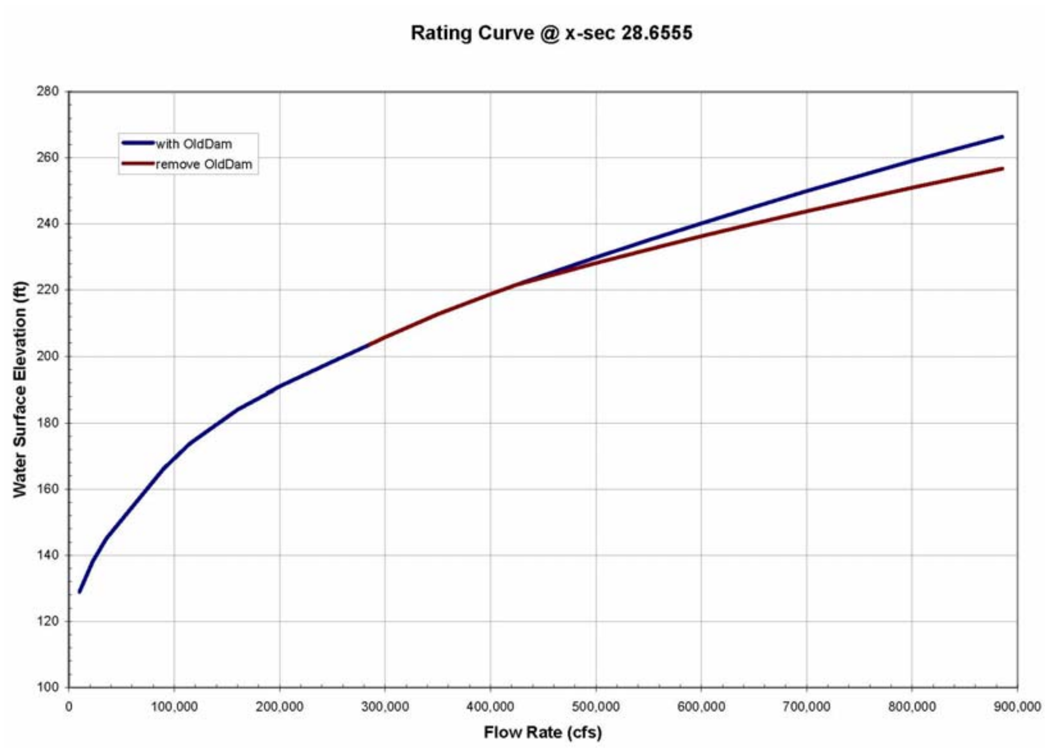

Figure 25. HEC-RAS data from the American River was used to set the tailwater elevation in the physical model.

Figure 26. The remnants of an old stone dam sit high on the banks.

Velocity Measurements

Velocity data at various locations in the model were collected with a handheld SonTek 2D FlowTracker acoustic Doppler velocimeter mounted on a 6 ft wading rod at a sample rate of 1 Hz (figure 27). The instrument measures two-dimensional velocity vectors in a small remote sampling volume (about 0.1 in^3) by emitting sound pulses (pings) at a specific frequency that reflect off of particles present in the water. The FlowTracker is a side-looking instrument with a 10 cm (3.94 inch) sample distance. The FlowTracker has an accuracy of $\pm$ 1% of the measured velocity with a velocity range from $\pm$ 0.003 to 13 ft/s. All velocity measurements in this report are referred to in prototype units unless noted otherwise.

Figure 27. A SonTek 2D FlowTracker acoustic Doppler velocimeter mounted on a 6 ft wading rod was used to measure velocities in the model.

High velocities along the stepped chute of the auxiliary spillway were measured with a pitot-static tube. The pitot-static tube was connected to a Sensotec model KZ differential pressure transducer with 0-5 psid range and model GM signal conditioner. The voltage output of the signal conditioner was calibrated using shunt resistors and the output voltage was read with an IOTech Wavebook 516 and a laptop computer. Each point of the profile was an average of 100 seconds of data collected at 20 Hz (2000 points). The pitot-static tube was mounted on a vernier-type adjustable rod and the measurements were all taken perpendicular to the slope (figure 28).

Figure 28. A pitot-static tube measured high velocities on the stepped spillway portion of the auxiliary spillway.

Wave Probe Measurements

Water levels were measured on both sides of the right auxiliary stilling basin wall to estimate the resulting total load on the wall due to flow conditions. Initial plans were to simultaneously monitor water surface levels on either side of the right wall of the auxiliary stilling basin using capacitance-type water level sensors. These sensors are typically a metal rod, coated with a dielectric material. The probe is excited with signal conditioning hardware and using the water in the model as the return path, lengths of the sensor rod immersed in the water can be calibrated to be proportional to an output voltage. The sensors used in this study were Delevan Cap Analog 410 (figure 29) with probes of 1/8 inch diameter and about 14 inches long (about 10.5 inches active length).

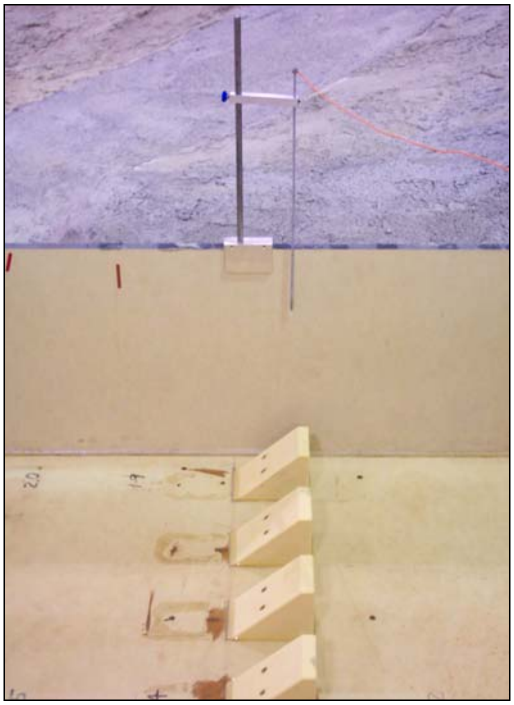

Figure 29. Wave probe was positioned 100 ft downstream from the upstream end of the basin.

The two probes were first calibrated on the bench where zeros were set and spans were adjusted to give a read-out of 1 V/in. When the probes were set up in the model across the stilling basin wall (separated by about 2 inches) there was apparent interference between the probes. With one probe energized they reacted as they had in calibration but when both were energized, the zero point drifted and there was substantial cross-probe interference. Due to this problem, measurements were collected on one side of the wall at a time and a statistical approach was used to determine the differential loading on the right stilling basin wall. After results were submitted to the COE structural designers, it was determined that more information was needed to design the right stilling basin wall. Therefore, additional tests were conducted with the dynamic pressure transducers described in the following section.

Data for the tests using the wave probe were collected using a laptop computer and an IOTech Wavebook 516 data acquisition system (figure 30). This system performs analog-digital conversions of 16 bit accuracy at up to 1 MHz. The software package, EZAnalyst, was chosen as the platform for the acquisition. An analysis rate of 100 Hz was selected, and a Nyquist frequency multiplier of 10.24 was selected resulting in a Δt of 0.00098 seconds in the model. The effective data collection rate was 1024 Hz, however the computer program takes into account the Nyquist frequency multiplier and the actual frequency range analyzed is 0-100 Hz.

Figure 30 – Wave probe data acquisition system.

To estimate the proper time series length needed to approximate the sample variance, tests of various lengths were conducted. The probe was located within the stilling basin in an area most affected by the hydraulic jump. Statistical results showed that the standard error became approximately constant after about 50,000 points; the variance required about 100,000 points. A data record of approximately 2 minutes (144 seconds) was chosen for each location to yield an n=147,456 for each measurement. One interesting note was that the interrange value (maximum-minimum) changed very little after about 10,000 points were collected.

Flush-Mount Pressure Transducers

In addition to collecting differential water surface data using capacitance-type wave probes at simulated discharges of 160,000 ft^3/s and lower, total pressures at five positions along the length of the wall were collected at several elevations. Using simultaneous sampling of these pressures across the wall, true pressure differentials could be calculated.

Eight flush-mount transducers were used to measure the differential pressure across the right auxiliary stilling basin wall. These transducers were mounted at four elevations on a u-shaped fitting that was placed over the top of the stilling basin wall, four on the inside of the basin and four on the outside of the basin (figures 31 and 32). The fitting was constructed of ¼-inch-thick aluminum plate and the transducers were located in machined recesses so that they would be flush with the plate surface (figure 33). The transducers were placed at elevations of 143.05 ft, 158.05 ft, 173.05 ft, and 188.05 ft (figure 34). They were glued in place with RTV silicone adhesive and the wires were routed along the basin wall to an area in the model where signal conditioning and data acquisition equipment were located.

Figure 31. Outside view of auxiliary stilling basin right wall with fitting holding a total of 8 pressure transducers.

Figure 32. Inside view of auxiliary stilling basin right wall with fitting holding a total of 8 pressure transducers.

Figure 33. Fitting showing the recessed area with the transducer in place.

Figure 34. Close-up view showing the flush-mounted subminiature pressure transducers attached to aluminum U-shaped fitting at the four elevations specified (outside of basin, Sta. 40+75).

The transducers were Honeywell-Sensotec subminiature, flush-diaphragm Model F with a 10 psi full scale range. They use a bonded 4 arm strain gage configured in a Wheatstone bridge on a thin unitized stainless steel diaphragm. They were connected to Honeywell Universal in-line amplifiers, providing the 5 VDC excitation and an amplified output of ±5V. The in-line amplifier allowed for checking and adjustment of zero and span using a shunt calibration procedure that was followed preceding each data collection run. The output voltage was read and recorded using an IOTech Wavebook 516 portable data acquisition system powered with a laptop computer. Data were collected in the model at a rate of 200 Hz. Several data samples of 45 seconds in length were collected and analyzed for each position.

Results

Cofferdam Testing

The top elevations of the government-designed and contractor-designed portions of the cofferdam were optimized by analyzing water surface elevation profiles in the 1:48-scale physical model. A marine-grade plywood wall was initially installed at a constant elevation along the alignment of the government-designed concrete cofferdam. A break in the wall alignment was designed to keep the wall close to the haul road and to avoid environmental restrictions associated with a disturbance to the low flow channel. The wall height for the concrete cofferdam was set higher than the expected maximum elevation in order to observe water surface elevations for a range of flow conditions.

The wall was surveyed with a level at several locations to ensure that the wall was installed at an elevation of 197 ft. A second marine-grade plywood wall was added downstream from the auxiliary stilling basin at a constant elevation of 161.86 ft to model the location of the temporary contractor-designed cofferdam (figure 35).

Figure 35. Installation of plywood cofferdam sections in the model. Model topography was modified to incorporate the construction access road.

Ten staff gages were attached to the upstream concrete cofferdam as measured from the top-of-wall elevation and one staff gage was attached to the temporary downstream cofferdam at the centerline. Staff gages on the concrete wall were placed at 50 ft increments (prototype) along the cofferdam wall as referenced from the break in alignment. Discharges of 30,000, 50,000, 75,000 and 100,000 ft^3/s were released from the main dam with the cofferdam in place. Water surface elevations were measured by observing the staff gages. Visual averages were collected over a 2 minute period along with the maximum water heights observed during this time period. The maximum water heights may not be the absolute maximum values. Water surface profiles and data tables are shown in figures 36-39 and tables 2-5.

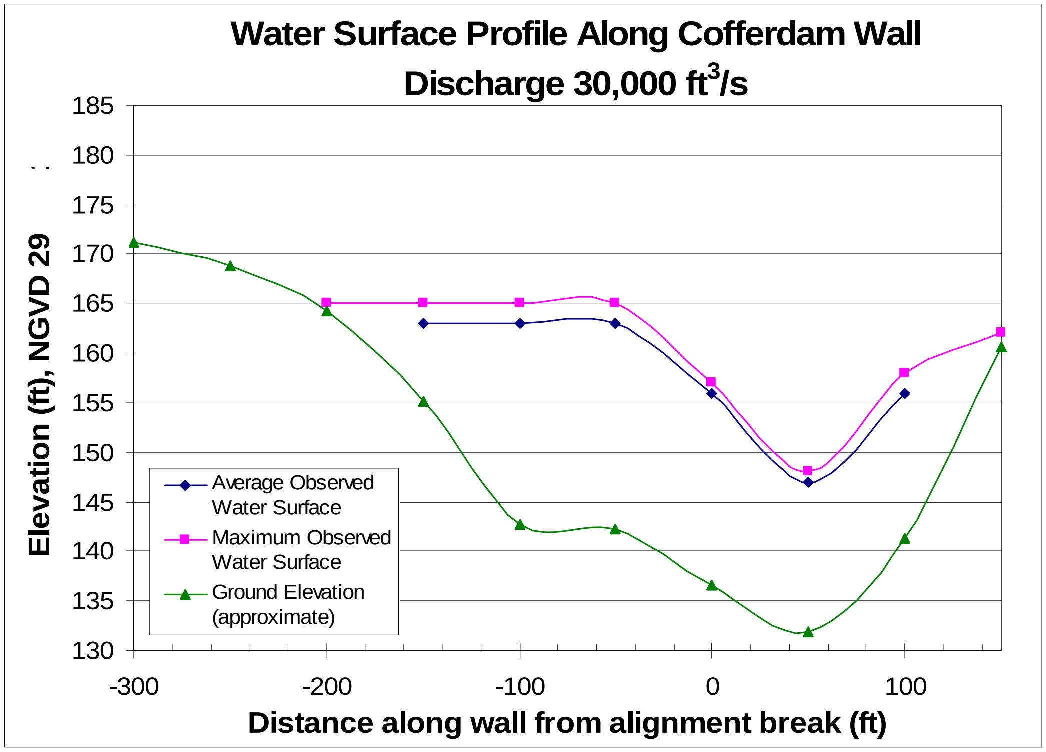

Figure 36. Water surface profile along the reinforced concrete cofferdam at a main dam discharge of 30,000 ft^3/s. Station 0 is at the break in alignment.

Table 2. Table of water surface profile data along the two cofferdam sections for a main dam discharge of 30,000 ft^3/s.

Location	Station	Average Elevation (ft)	Maximum Elevation (ft)
300 ft upstream from break	-300	Below Toe	Below Toe
250 ft upstream from break	-250	Below Toe	Below Toe
200 ft upstream from break	-200	Below Toe	165
150 ft upstream from break	-150	163	165
100 ft upstream from break	-100	163	165
50 ft upstream from break	-50	163	165
Alignment break	0	156	157
50 ft downstream from break	50	147	148
100 ft downstream from break	100	156	158
150 ft downstream from break	150	Below Toe	162
Auxiliary end sill wall	Centerline	145	145

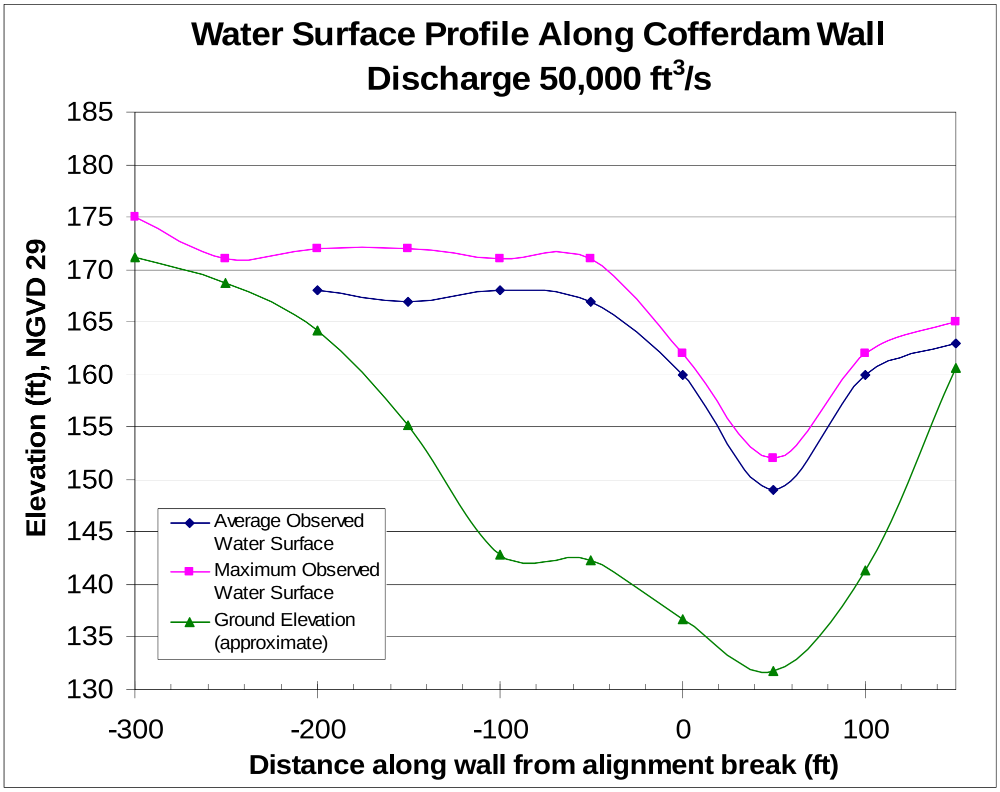

Figure 37. Water surface profile along the reinforced concrete cofferdam at a main dam discharge of 50,000 ft^3/s. Station 0 is at the break in alignment.

Table 3. Table of water surface profile data along the two cofferdam sections for a main dam discharge of 50,000 ft^3/s.

Location	Station	Average Elevation (ft)	Maximum Elevation (ft)
300 ft upstream from break	-300	Below Toe	175
250 ft upstream from break	-250	Below Toe	171
200 ft upstream from break	-200	168	172
150 ft upstream from break	-150	167	172
100 ft upstream from break	-100	168	171
50 ft upstream from break	-50	167	171
Alignment break	0	160	162
50 ft downstream from break	50	149	152
100 ft downstream from break	100	160	162
150 ft downstream from break	150	163	165
Auxiliary end sill wall	Centerline	153.5	155

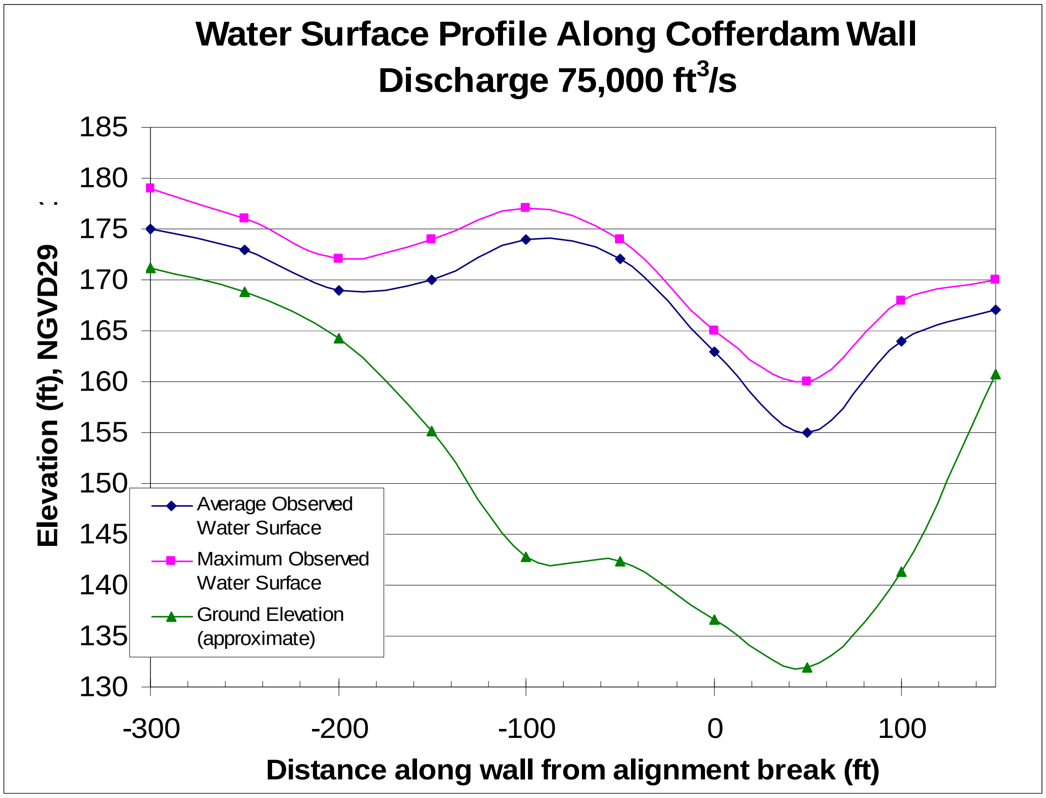

Figure 38. Water surface profile along the reinforced concrete cofferdam at a main dam discharge of 75,000 ft^3/s. Station 0 is at the break in alignment.

Table 4. Table of water surface profile data along the two cofferdam sections for a main dam discharge of 75,000 ft^3/s.

Location	Station	Average Elevation (ft)	Maximum Elevation (ft)
300 ft upstream from break	-300	175	179
250 ft upstream from break	-250	173	176
200 ft upstream from break	-200	169	172
150 ft upstream from break	-150	170	174
100 ft upstream from break	-100	174	177
50 ft upstream from break	-50	172	174
Alignment break	0	163	165
50 ft downstream from break	50	155	160
100 ft downstream from break	100	164	168
150 ft downstream from break	150	167	170
Auxiliary end sill wall	Centerline	Submerged	Submerged

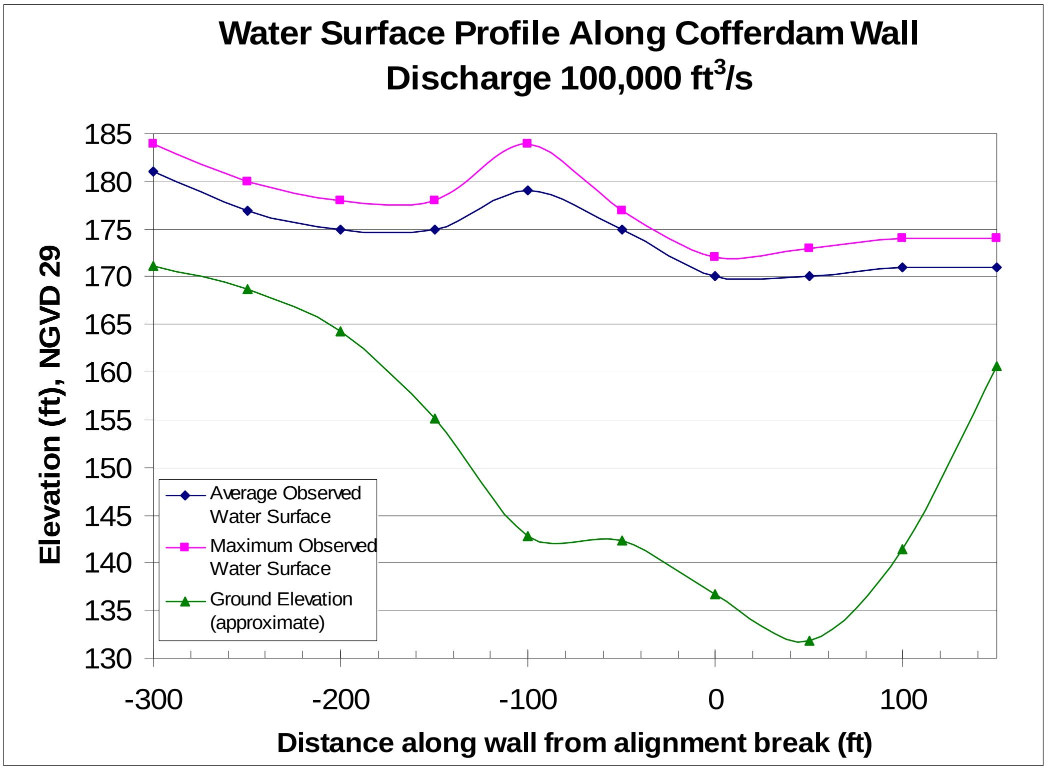

Figure 39. Water surface profile along the reinforced concrete cofferdam at a main dam discharge of 100,000 ft^3/s. Station 0 is at the break in alignment.

Table 5. Table of water surface profile data along the two cofferdam sections for a main dam discharge of 100,000 ft^3/s.

Location	Station	Average Elevation (ft)	Maximum Elevation (ft)
300 ft upstream from break	-300	181	184
250 ft upstream from break	-250	177	180
200 ft upstream from break	-200	175	178
150 ft upstream from break	-150	175	178
100 ft upstream from break	-100	179	184
50 ft upstream from break	-50	175	177
Alignment break	0	170	172
50 ft downstream from break	50	170	173
100 ft downstream from break	100	171	174
150 ft downstream from break	150	171	174
Auxiliary end sill wall	Centerline	Submerged	Submerged

Following discussions between the COE and Reclamation designers, 50,000 ft^3/s was chosen as the design discharge for the cofferdams. Flows larger than 50,000 ft^3/s will overtop the cofferdams and place water to nearly an equal elevation on both sides. The cofferdam top-of-wall elevations were designed to allow for a minimum of 1 ft of freeboard for the water surface resulting from 50,000 ft^3/s discharging from the main dam service spillway. Using the water surface profiles collected in the 1:48-scale physical model, the final design of the concrete cofferdam includes 4 wall sections at various elevations along the length of the wall (elevation 178.5 for 153 ft, elevation 177.5 for 269 ft, elevation 173.0 for 36 ft, and elevation 168.5 for 135 ft as shown in figure 40). The differences in elevation are due to the invert of the channel changing, lowering the water surface in the downstream direction. In some areas the freeboard is more than 1 ft to minimize the amount of elevation changes in the cofferdam. The contractor-designed cofferdam is to be a continuation of the wall at crest elevation 168.5 and designed for flowing water to elevation 167.5.

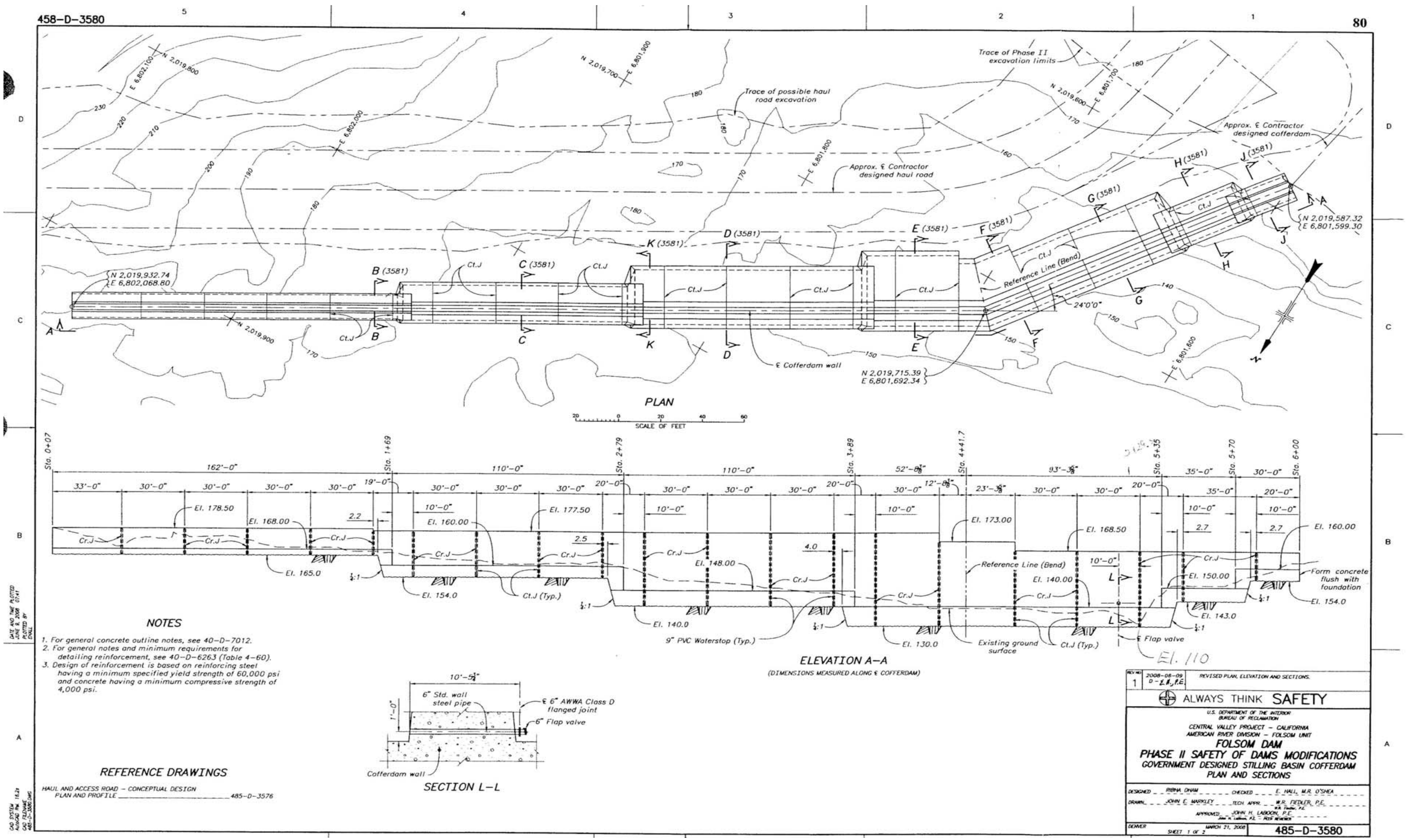

Figure 40. Construction drawing of government-designed reinforced concrete cofferdam section.

Flow Observations and Water Surface Elevations

Water surface elevations were measured throughout the model with piezometer taps on the bed of the model. Water levels on the manometer boards were visually averaged and recorded. Water surface elevations were measured in the main dam exit channel, downstream from the emergency spillway, in the auxiliary stilling basin and exit channel, outside of the right auxiliary stilling basin wall, in the American River channel, and along the left and right banks looking downstream. Data were collected during 20 flow combinations with release flows divided between the auxiliary spillway and the main dam (table 6).

Table 6. Water surface elevations were collected for 20 flow conditions.

Test No.	Auxiliary Spillway Flow Rate (ft^3/s)	Main Dam Spillway Flow Rate (ft^3/s)	Total Flow Rate (ft^3/s)	Tailwater Elevation (ft)
1	0	25,000	25,000	140.00
2	25,000	0	25,000	140.00
3	0	60,000	60,000	156.50
4	60,000	0	60,000	156.50
5	35,000	25,000	60,000	156.50
6	0	90,000	90,000	166.23
7	90,000	0	90,000	166.23
8	65,000	25,000	90,000	166.23
9	0	115,000	115,000	173.86
10	90,000	25,000	115,000	173.86
11	115,000	0	115,000	173.86
12	0	160,000	160,000	184.02
13	160,000	0	160,000	184.02
14	135,000	25,000	160,000	184.02
15	80,000	80,000	160,000	184.02
16	160,000	140,000	300,000	205.75
17	0	300,000	300,000	205.75
18	312,000	518,000	830,000	253.50
19	0	518,000	518,000	230.00
20	312,000	206,000	518,000	230.00

All tap locations at the centerline of the American River channel were 100 ft apart. Taps located along the banklines and below the main dam (not on the axis of the American River channel) were labeled according to their location along the American River channel perpendicular to the channel axis. Taps located in and just outside of the auxiliary stilling basin were labeled according to their distance along the axis of the auxiliary spillway channel, with the coordinate systems for the auxiliary spillway channel and main dam exit channel coinciding at the intersection of the channel axes. Location 0 near the emergency spillway was the most upstream tap location in the model. The color and symbol at each tap location corresponds to the color and symbol depicted on the water surface elevation graphs (figure 41).

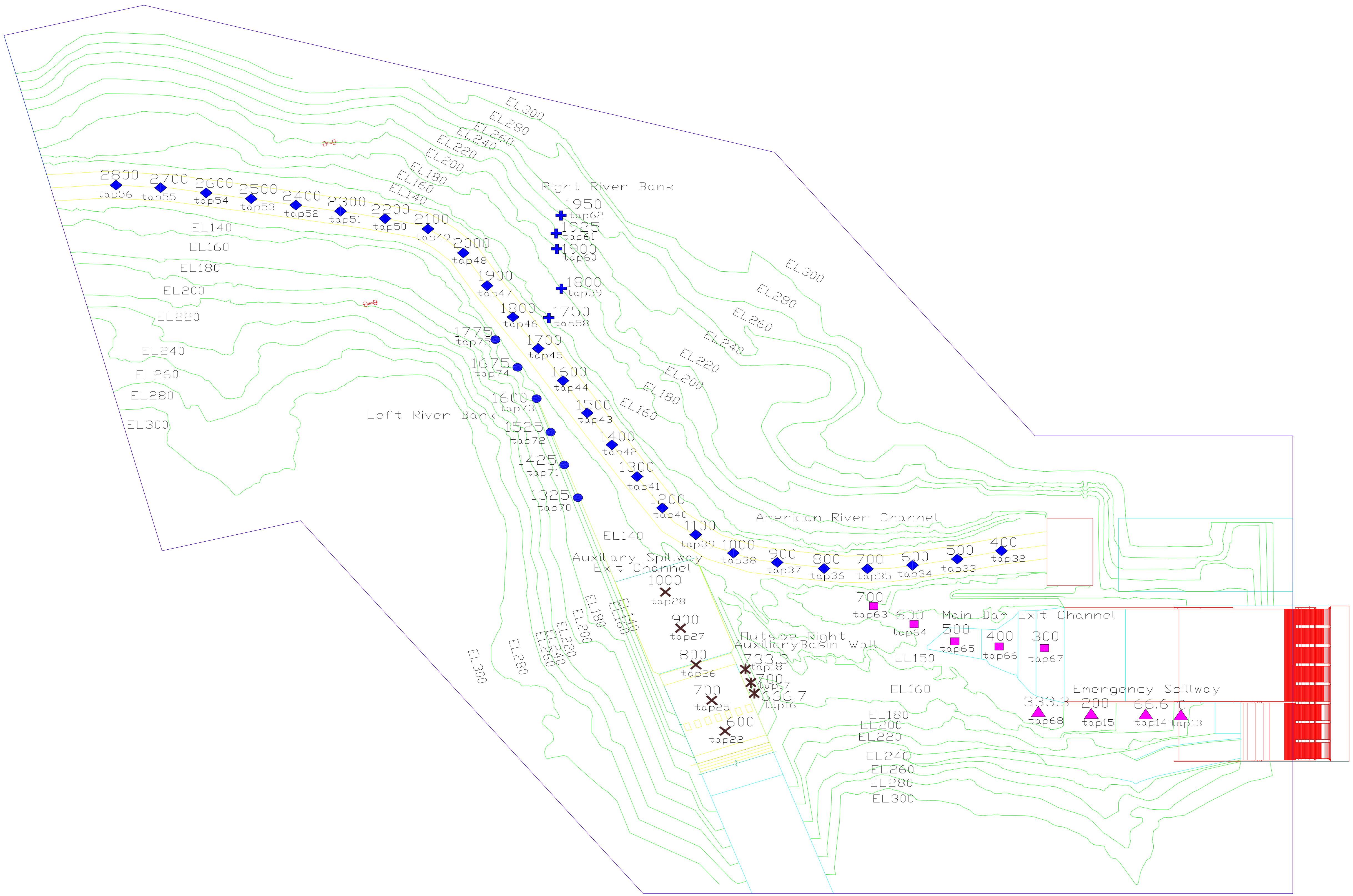

Figure 41. Drawing of locations where water surface elevations were collected. The color and symbol at each tap location corresponds to the color and symbol depicted on the water surface elevation graphs: American River Channel = blue diamond, Left River Bank = blue circle, Right River Bank = blue plus sign, Main Dam Exit Channel = pink square, Emergency Spillway = pink triangle, Auxiliary Stilling Basin and Exit Channel = brown X, Outside Right Auxiliary Stilling Basin Wall = brown star. The two red symbols at the downstream end of the model are the bridge piers of the Folsom Lake Crossing Bridge.

For each of the 20 flow conditions, there is a photograph showing the flow conditions viewed in the model and a graph depicting all of the measured water surface elevations (figures 42-81). Notable model observations are also provided.

For a given total discharge, flows released concurrently from the two structures appear visually to have better energy dissipation characteristics. This is largely due to the reduced velocity (lower Froude number) entering each structure versus what would be present had all flow been diverted into a single spillway and stilling basin. This same reasoning results in lower wave heights along the banklines for the concurrent operations. However, the total flow amounts from a single structure are still fully acceptable up to and including 160,000 ft^3/s.

Test 1: Auxiliary spillway flow rate 0 ft^3/s and main dam spillway flow rate 25,000 ft^3/s with a tailwater elevation of 140.00 ft.

Figure 42. Test 1: Overview of flow condition with 0 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** Main dam flow redirected away from the right auxiliary stilling basin sidewall by the cofferdam. No auxiliary spillway flow. Auxiliary stilling basin sidewalls not submerged by tailwater.

Main Dam Stilling Basin: Hydraulic jump fully contained within main dam stilling basin. No turbulence downstream from the end sill. Basin sidewalls not submerged. Flow is contained by the main dam spillway chute walls.

***Right River Bank*: Mild wave action against right bank from main dam flow with wave run-up of about 4 to 6 ft prototype (approximately 1 to 1.5 inches model) along right bankline.

***Left River Bank*: Tailwater remains in river channel without entering auxiliary spillway exit channel. Left bank wave run-up is about 4 ft prototype (approximately 1 inch model).

***Knob Topographic Feature*: Knob is not submerged by tailwater. Water impacts knob and is deflected to the right into the main river channel. Higher velocities are observed along the cofferdam alignment break.

***Downstream Bridge Pier*: Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

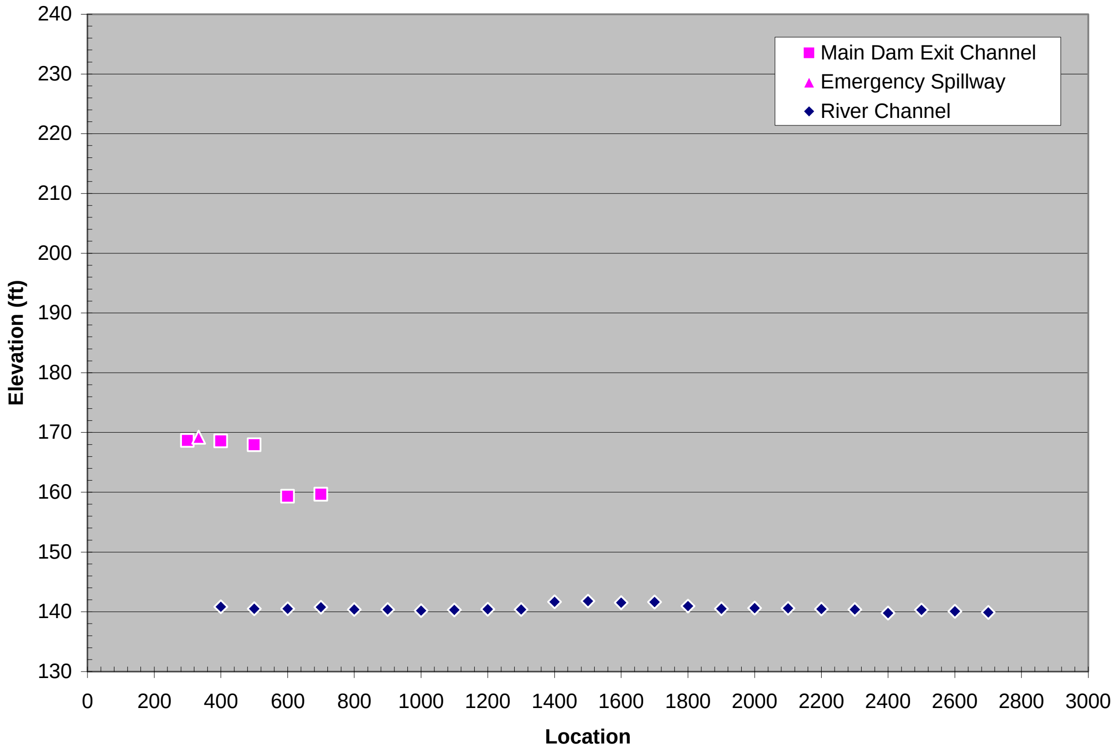

Figure 43. Test 1: Water surface profile with 0 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway.

Test 2: Auxiliary spillway flow rate 25,000 ft^3/s and main dam spillway flow rate 0 ft^3/s with a tailwater elevation of 140.00 ft.

Figure 44. Test 2: Overview of flow condition with 25,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** Uniform water surface over the length of the auxiliary stilling basin. Hydraulic jump contained within the stilling basin. Water surface drops off at the end sill and produces surface undulations downstream from the end sill.

Main Dam Stilling Basin: No main dam flow. Tailwater does not extend up to the main dam exit channel.

***Right River Bank*:** Boiling occurs against the right bankline from auxiliary spillway flow entering the river channel. Boiling occurs between Stations 1000 and 1200 with mild wave action of about 4 ft prototype (approximately 1 inch model).

***Left River Bank*:** Mild wave action against left bank from the auxiliary spillway discharge. Wave run-up of about 2 ft prototype (approximately 0.5 inches model) occurs along the left bankline.

***Knob Topographic Feature*:** Knob is not submerged by tailwater. Water flows past knob into river channel

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

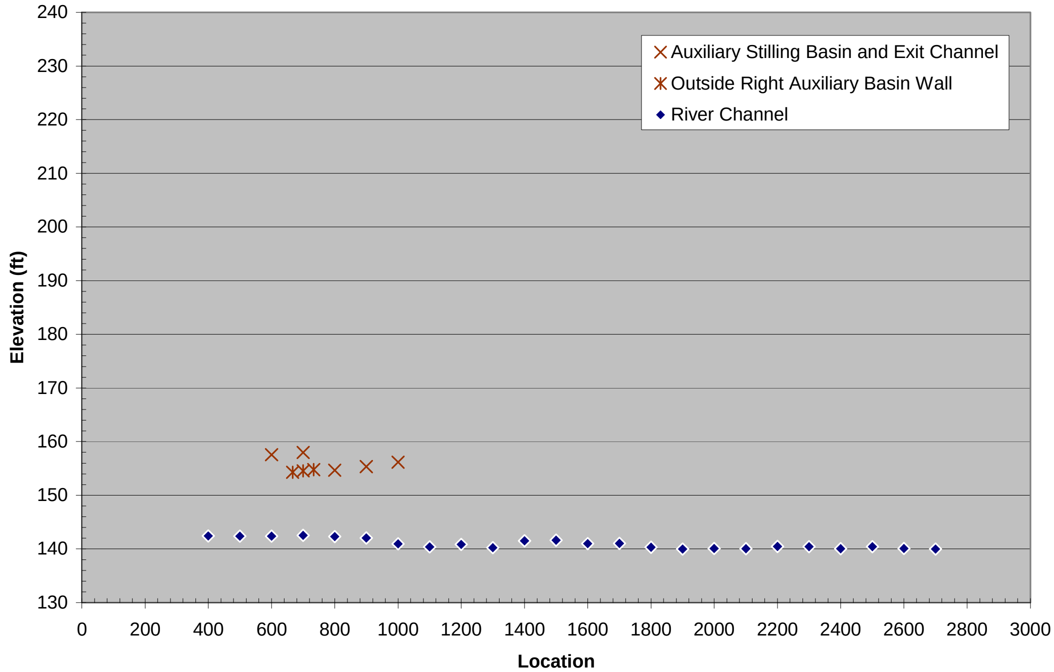

Figure 45. Test 2: Water surface profile with 25,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway.

Test 3: Auxiliary spillway flow rate 0 ft^3/s and main dam spillway flow rate 60,000 ft^3/s with a tailwater elevation of 156.50 ft.

Figure 46. Test 3: Overview of flow condition with 0 ft^3/s from the auxiliary spillway and 60,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** Main dam flow redirected away from the right auxiliary stilling basin sidewall by the cofferdam. No auxiliary spillway flow. Auxiliary stilling basin sidewalls not submerged by tailwater.

Main Dam Stilling Basin: Hydraulic jump fully contained within the basin. No turbulence downstream from the end sill. Basin sidewalls are not submerged. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Mild wave action against right bank from main dam flow with wave run-up of about 6 ft prototype (approximately 1.5 inches model) along right bankline.

***Left River Bank*:** Mild wave action against left bank with wave run-up of about 4 ft prototype (approximately 1 inch model) along the bankline.

***Knob Topographic Feature*:** Knob is not submerged. A thin sheet of water splashes over the top of the knob. A hydraulic jump occurs upstream of the knob and as water drops into the main river channel. Higher velocities are observed at the narrowest point between the cofferdam and right bank (adjacent to the alignment break in the cofferdam).

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

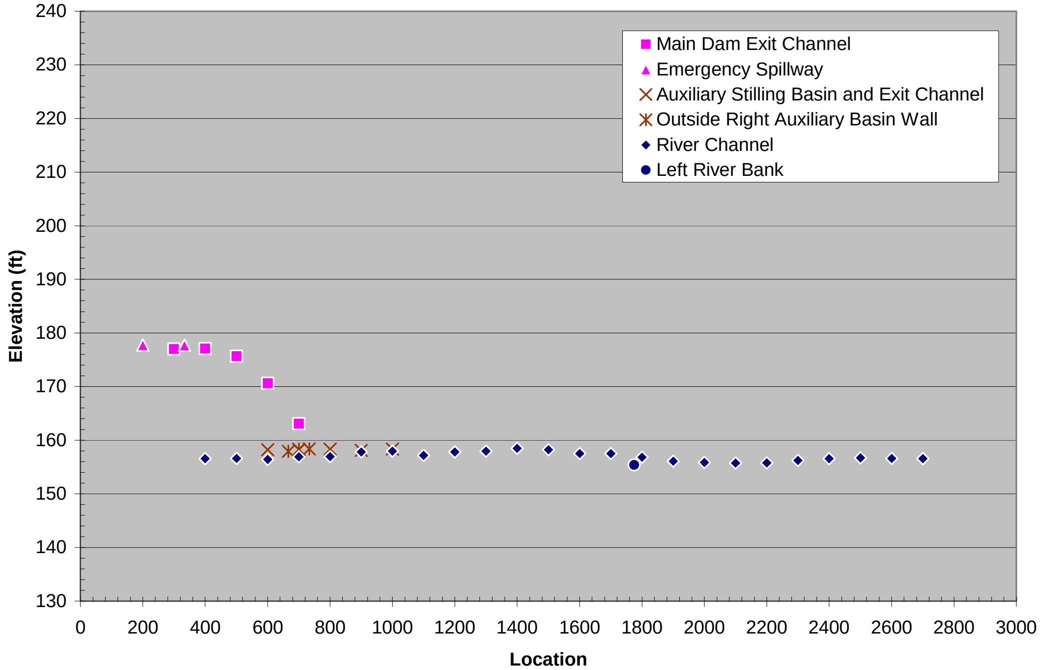

Figure 47. Test 3: Water surface profile with 0 ft^3/s from the auxiliary spillway and 60,000 ft^3/s from the main dam service spillway.

Test 4: Auxiliary spillway flow rate 60,000 ft^3/s and main dam spillway flow rate 0 ft^3/s with a tailwater elevation of 156.50 ft.

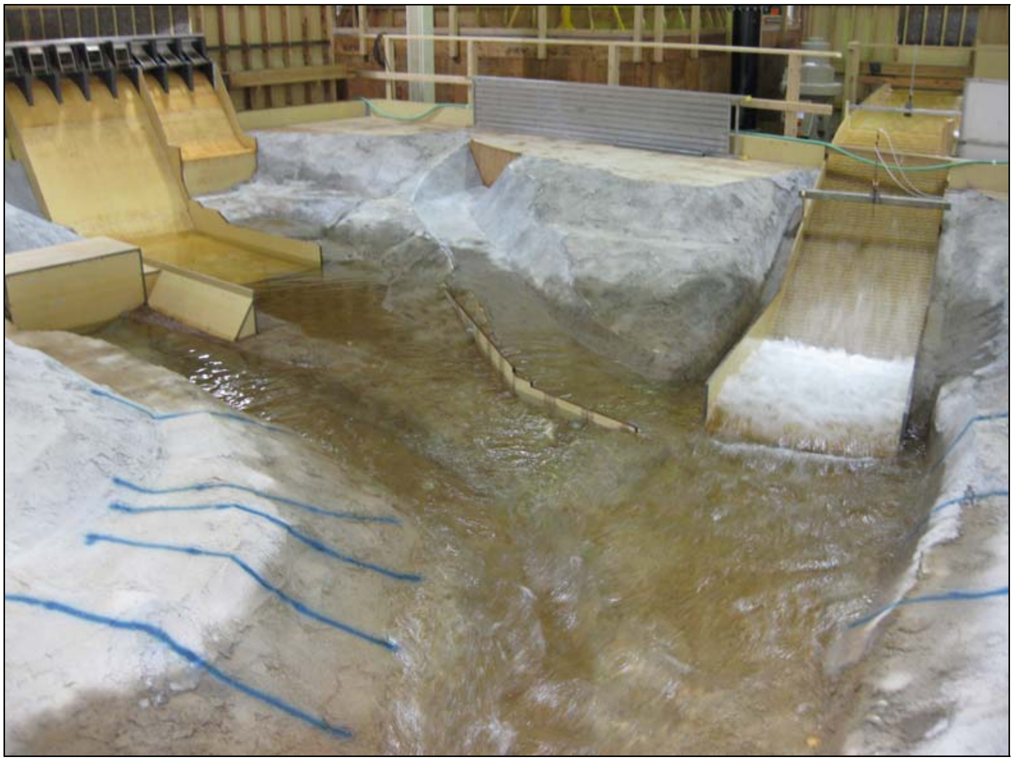

Figure 48. Test 4: Overview of flow condition with 60,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway.

Auxiliary Stilling Basin**:** Fairly uniform water surface over the length of the auxiliary stilling basin. Hydraulic jump contained within the basin. Water surface drop off at end sill produces standing wave downstream from the end sill.

Main Dam Stilling Basin: No main dam flow. Main dam stilling basin walls not submerged by tailwater.

Right River Bank**:** Mild wave action against right bank from auxiliary spillway flow. Wave run-up of about 6 ft prototype (approximately 1.5 inches model) greatest between stations 1100 and 1200.

Left River Bank**:** Mild wave action against left bank from auxiliary spillway discharge. Wave run-up of about 4 ft prototype (approximately 1 inch model) along the left bankline.

Knob Topographic Feature**:** Knob is not submerged by tailwater. Some splashing over the knob, but velocities against the knob are relatively low.

Downstream Bridge Pier**:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

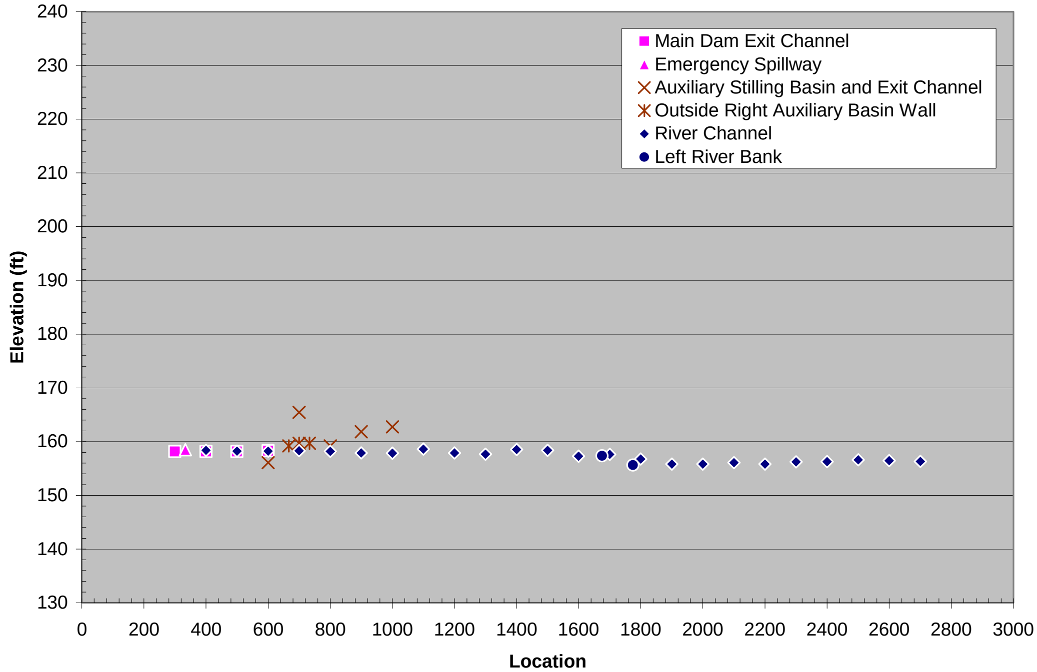

Figure 49. Test 4: Water surface profile with 60,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway.

Test 5: Auxiliary spillway flow rate 35,000 ft^3/s and main dam spillway flow rate 25,000 ft^3/s with a tailwater elevation of 156.50 ft.

Figure 50. Test 5: Overview of flow condition with 35,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** No wave action against the right basin sidewall from the main dam release. Hydraulic jump contained within the auxiliary stilling basin. Water drops off slightly at the end sill.

Main Dam Stilling Basin: Hydraulic jump fully contained within basin and no turbulence extends downstream from the end sill. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Minimal wave action against the right bank. Wave run-up of about 4 ft prototype (approximately 1 inch model) greatest from stations 1100 to 1200.

***Left River Bank*:** Minimal wave action against the left bank. Wave run-up of about 2 ft prototype (approximately 0.5 inches model) along the extent of the left bank.

***Knob Topographic Feature*:** Knob is not submerged by tailwater. Impact from the main dam flow is minimal with most water turning into the channel before reaching the knob. The cofferdam is not submerged.

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

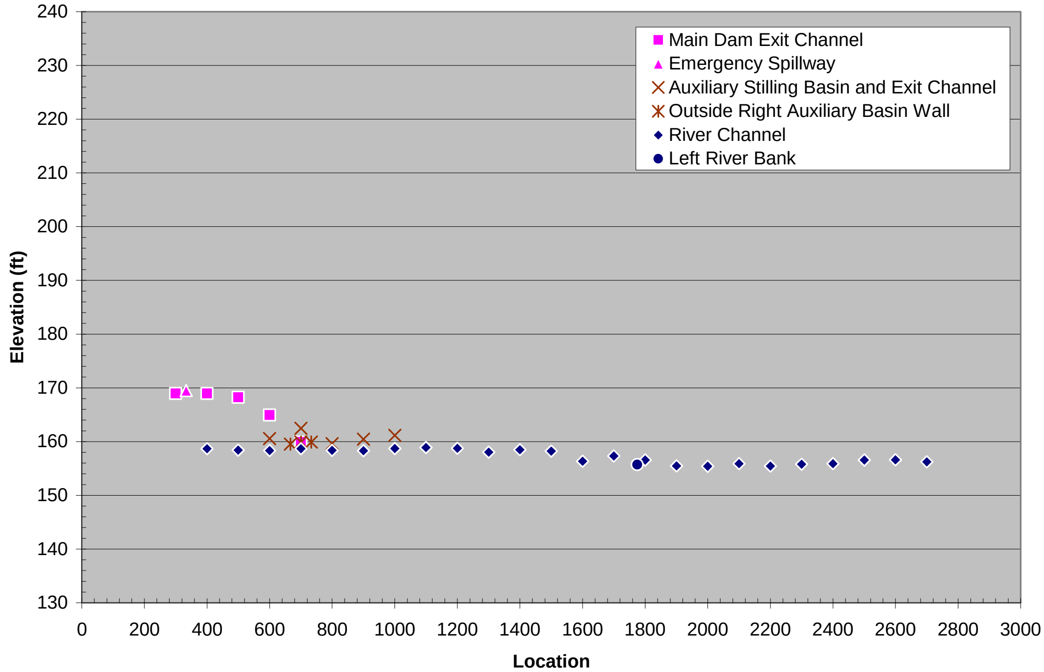

Figure 51. Test 5: Water surface profile with 35,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway.

Test 6: Auxiliary spillway flow rate 0 ft^3/s and main dam spillway flow rate 90,000 ft^3/s with a tailwater elevation of 166.23 ft.

Figure 52. Test 6: Overview of flow condition with 0 ft^3/s from the auxiliary spillway and 90,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** No auxiliary spillway flow. Auxiliary stilling basin sidewalls are not submerged by tailwater. Flow is redirected away from the right basin sidewall by the cofferdam.

Main Dam Stilling Basin: Hydraulic jump fully contained within the main dam stilling basin. No turbulence downstream from the end sill. Basin sidewalls in the downstream portion of the basin are almost completely submerged. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Moderate wave action against right bank from the main dam flow. Wave run-up of about 8 ft prototype (approximately 2 inches model) occurs at station 800. Wave run-ups of about 6 to 8 ft prototype (approximately 1.5 to 2 inches model) continue down the right bankline.

***Left River Bank*:** Mild wave action against the left bank. Wave run-up of about 4 ft prototype (approximately 1 inch model) is greatest near station 1300.

***Knob Topographic Feature*:** Cofferdam is not submerged, but water splashes over the cofferdam. Standing waves and higher velocities observed at the narrowest point between the cofferdam and the right bank (adjacent to the alignment break in the cofferdam). The knob is overtopped and fully submerged. A weak hydraulic jump occurs upstream from the knob and as water drops into the main river channel.

***Downstream Bridge Pier*: ** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

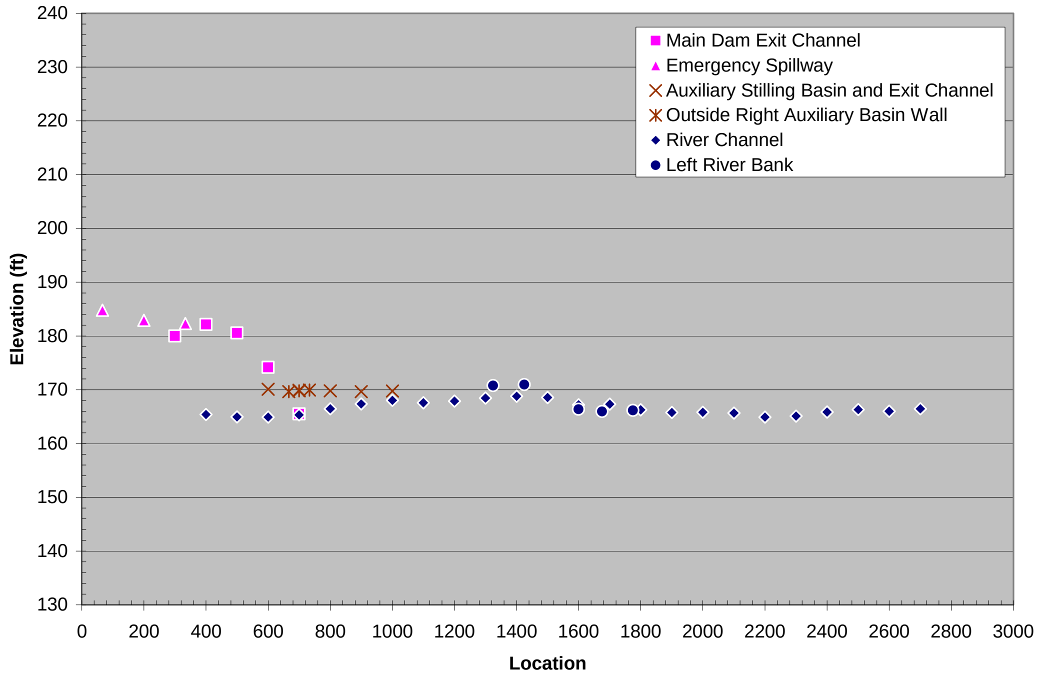

Figure 53. Test 6: Water surface profile with 0 ft^3/s from the auxiliary spillway and 90,000 ft^3/s from the main dam service spillway.

Test 7: Auxiliary spillway flow rate 90,000 ft^3/s and main dam spillway flow rate 0 ft^3/s with a tailwater elevation of 166.23 ft.

Figure 54. Test 7: Overview of flow condition with 90,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway.

Auxiliary Stilling Basin**:** Near full static head differential in upstream portion of the auxiliary stilling basin and little water surface differential at the downstream end of the basin. Hydraulic jump contained within the basin. Water surface drop off at end sill produces a standing wave downstream from the end sill.

Main Dam Stilling Basin: No main dam flow. Main dam stilling basin walls not submerged by tailwater.

Right River Bank**:** Moderate wave action against the right bank from auxiliary spillway flow. Wave run-up of about 6 to 8 ft prototype (approximately 1.5 to 2 inches model) greatest between stations 1000 and 1200.

Left River Bank**:** Moderate wave action against the left bank from the auxiliary spillway discharge. Wave run-up of about 6 to 8 ft prototype (approximately 1.5 to 2 inches model) along the left bankline.

Knob Topographic Feature**:** Knob submerged by tailwater. Cofferdam not submerged.

Downstream Bridge Pier**:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

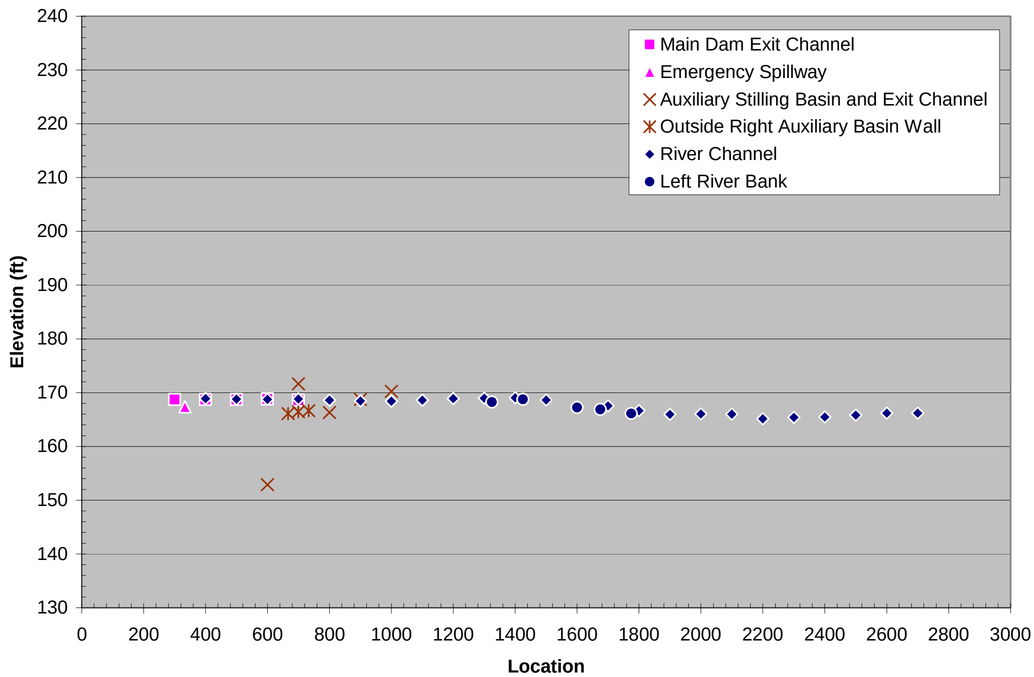

Figure 55. Test 7: Water surface profile with 90,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway. Location 600 in the auxiliary stilling basin is in the supercritical portion of the hydraulic jump.

Test 8: Auxiliary spillway flow rate 65,000 ft^3/s and main dam spillway flow rate 25,000 ft^3/s with a tailwater elevation of 166.23 ft.

Figure 56. Test 8: Overview of flow condition with 65,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** No wave action against the right auxiliary stilling basin sidewall. Some eddying behind the downstream edge of the auxiliary basin sidewalls. Hydraulic jump contained within basin with some mild turbulence downstream from the end sill.

Main Dam Stilling Basin: Hydraulic jump fully contained within the basin. No turbulence downstream from the end sill. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Moderate wave action against the right bank from the auxiliary spillway flow. Wave run-up of about 6 to 8 ft prototype (approximately 1.5 to 2 inches model) is greatest between stations 1100 and 1200.

***Left River Bank*:** Moderate wave action against the left bank from the auxiliary spillway flow. Wave run-up of about 6 to 8 ft prototype (approximately 1.5 to 2 inches model) along the left bankline.

***Knob Topographic Feature*:** Knob submerged by tailwater. Cofferdam partially submerged.

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

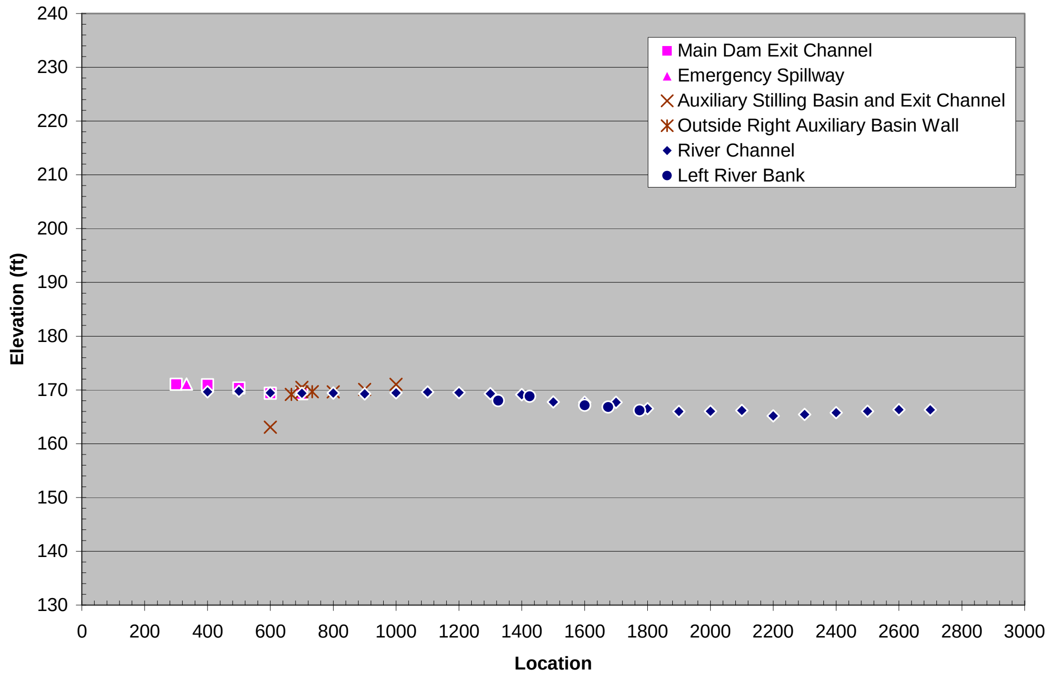

Figure 57. Test 8: Water surface profile with 65,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway. Location 600 in the auxiliary stilling basin is in the supercritical portion of the hydraulic jump.

Test 9: Auxiliary spillway flow rate 0 ft^3/s and main dam spillway flow rate 115,000 ft^3/s with a tailwater elevation of 173.86 ft.

Figure 58. Test 9: Overview of flow condition with 0 ft^3/s from the auxiliary spillway and 115,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** Some mild wave action against the right auxiliary stilling basin sidewall, but no splashing or overtopping. No auxiliary spillway flow. Stilling basin sidewalls not submerged by tailwater.

Main Dam Stilling Basin: Hydraulic jump fully contained within the basin. Tailwater occasionally splashes back into the stilling basin over the upstream portion of the basin sidewalls. Water submerges the left and right basin sidewalls in the downstream portion of the basin. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Moderate wave action against the right bank. Wave run-up of about 8 ft prototype (approximately 2 inches model) occurs along the right bankline.

***Left River Bank*:** Mild wave action against the left bank. Wave run-up of about 4 ft prototype (approximately 1 inch model) greatest near station 1300.

***Knob Topographic Feature*:** Knob is fully submerged by tailwater. Cofferdam is not submerged, but water splashes over the cofferdam. Standing waves and higher velocities observed at the narrowest point between the cofferdam and right bank (adjacent to the alignment break in the cofferdam).

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

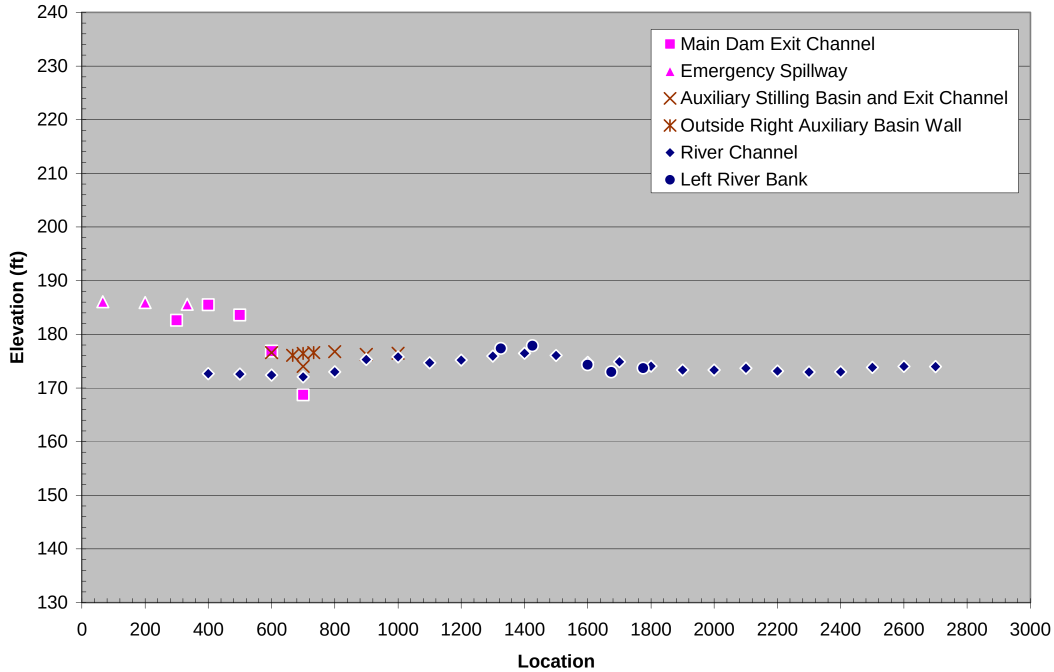

Figure 59. Test 9: Water surface profile with 0 ft^3/s from the auxiliary spillway and 115,000 ft^3/s from the main dam service spillway.

Test 10: Auxiliary spillway flow rate 90,000 ft^3/s and main dam spillway flow rate 25,000 ft^3/s with a tailwater elevation of 173.86 ft.

Figure 60. Test 10: Overview of flow condition with 90,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*: ** No wave action against right auxiliary stilling basin sidewall. Some eddying behind the downstream edge of the auxiliary basin sidewalls. Hydraulic jump contained within the basin. Some mild turbulence continues about 48 ft prototype (approximately 1 ft model) downstream from the end sill.

Main Dam Stilling Basin: Hydraulic jump fully contained within the main dam stilling basin. No turbulence downstream from the end sill. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Moderate wave action against the right bank from the auxiliary spillway flow. Wave run-up of about 6 to 8 ft prototype (approximately 1.5 to 2 inches model) greatest at stations 1000 to 1200.

***Left River Bank*:** Moderate wave action against the left bank from the auxiliary spillway flow. Wave run-up of about 6 ft prototype (approximately 1.5 inches model) along the left bankline.

***Knob Topographic Feature*:** Knob is submerged by tailwater. Cofferdam is partially submerged.

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

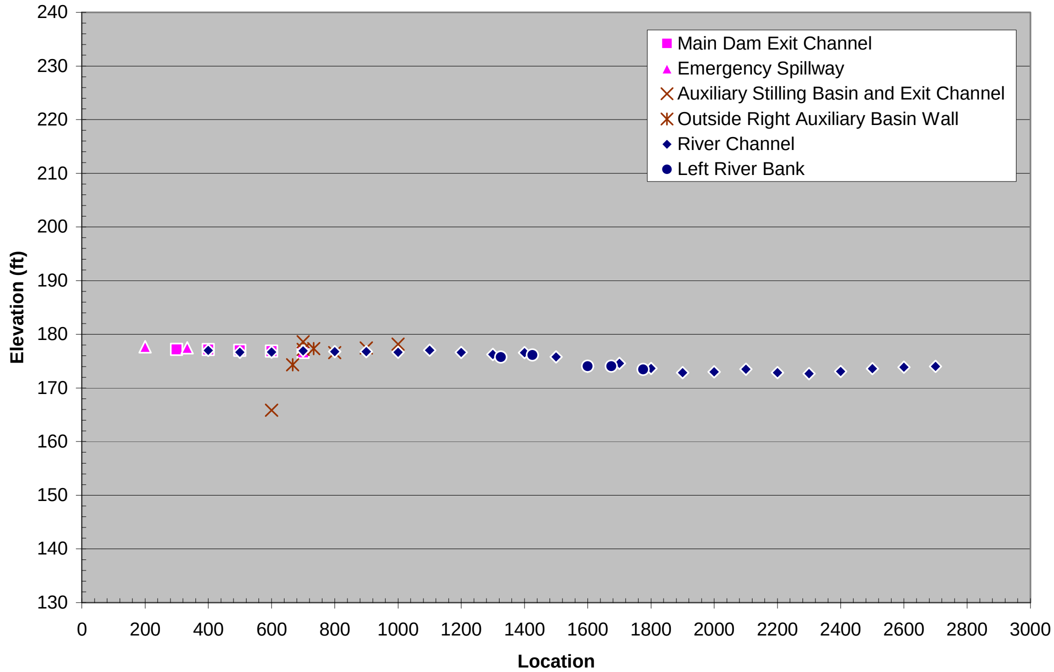

Figure 61. Test 10: Water surface profile with 90,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway. Location 600 in the auxiliary stilling basin is in the supercritical portion of the hydraulic jump.

Test 11: Auxiliary spillway flow rate 115,000 ft^3/s and main dam spillway flow rate 0 ft^3/s with a tailwater elevation of 173.86 ft.

Figure 62. Test 11: Overview of flow condition with 115,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway.

Auxiliary Stilling Basin**:** Water occasionally splashes over the auxiliary stilling basin sidewalls from the inside of the basin. Near full static head differential in the upstream portion of the basin and little water surface differential at the downstream end of the basin. Hydraulic jump contained within basin. Some standing waves and white water extend about 48 ft prototype (approximately 1 ft model) downstream from the end sill.

Main Dam Stilling Basin: No main dam flow. Basin walls not submerged by tailwater.

Right River Bank**:** Moderate wave action against the right bank from the auxiliary spillway flow. Wave run-up of about 8 ft prototype (approximately 2 inches model) greatest between stations 1000 to 1300.

Left River Bank**:** Moderate wave action against the left bank from the auxiliary spillway flow. Wave run-up of about 6 to 8 ft (approximately 1.5 to 2 inches model) along the left bankline.

Knob Topographic Feature**:** Knob submerged by tailwater. Cofferdam submerged on downstream end, but not on upstream portion.

Downstream Bridge Pier**:** Water surface not up to bridge pier. Wave action minimal at about 2 ft prototype (approximately 0.5 inches model).

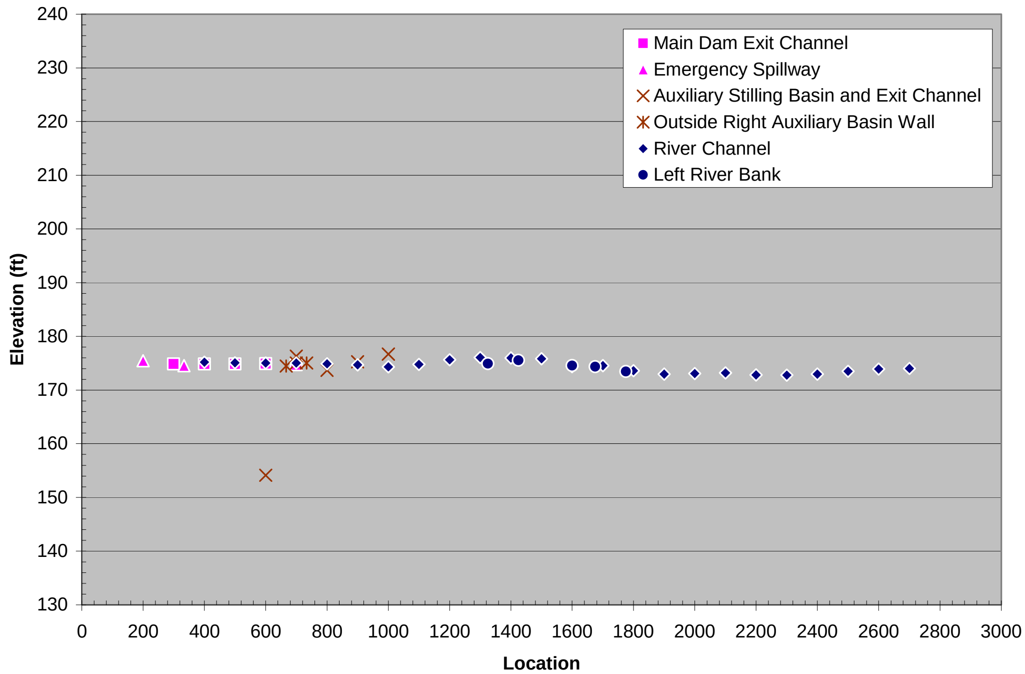

Figure 63. Test 11: Water surface profile with 115,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway. Location 600 in the auxiliary stilling basin is in the supercritical portion of the hydraulic jump.

Test 12: Auxiliary spillway flow rate 0 ft^3/s and main dam spillway flow rate 160,000 ft^3/s with a tailwater elevation of 184.02 ft.

Figure 64. Test 12: Overview of flow condition with 0 ft^3/s from the auxiliary spillway and 160,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** No auxiliary spillway flow. Auxiliary stilling basin sidewalls not submerged by tailwater. Some mild wave action against the right stilling basin sidewall, but no splashing or overtopping.

Main Dam Stilling Basin: Hydraulic jump not fully contained within the main dam stilling basin. Boiling occurs about 70 to 95 ft prototype (approximately 1.5 to 2 ft model) downstream from the end sill. Tailwater pours back into the stilling basin over the upstream portion of the left and right basin sidewalls. Water submerges the basin sidewalls in the downstream portion of the basin. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Moderate wave action against the right bank. Wave run-up of about 10 to 12 ft prototype (approximately 2.5 to 3 inches model) along the right bankline. Velocities are higher along the right bank than the left bank.

***Left River Bank*:** Moderate wave action against the left bank. Wave run-up of about 8 ft prototype (approximately 2 inches model) greatest near station 1300.

***Knob Topographic Feature*:** Knob and cofferdam submerged by tailwater.

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 to 4 ft prototype (approximately 0.5 to 1 inch model).

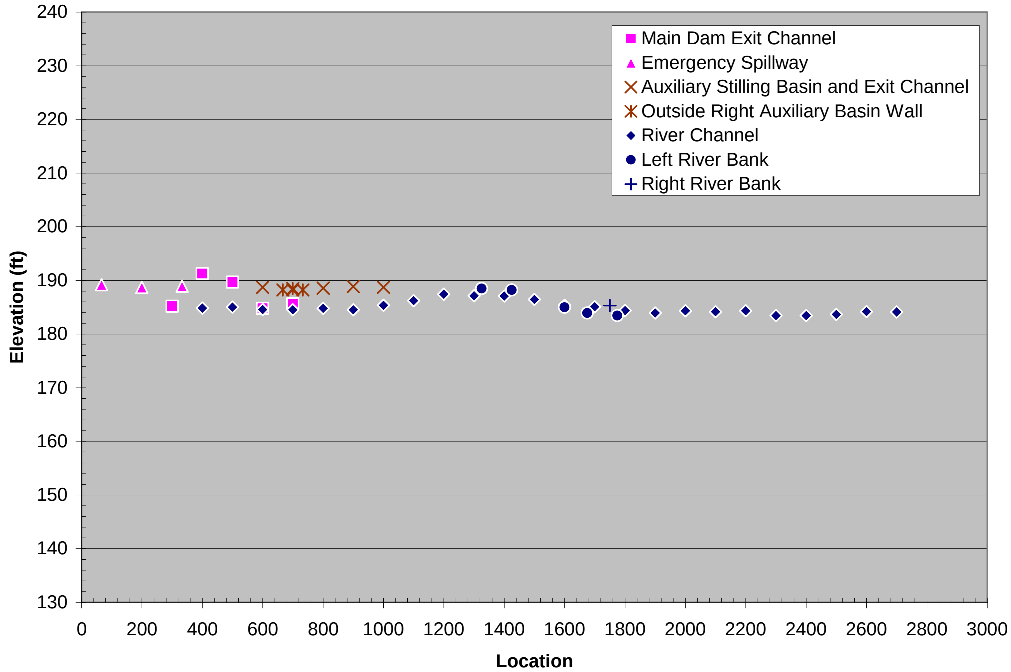

Figure 65. Test 12: Water surface profile with 0 ft^3/s from the auxiliary spillway and 160,000 ft^3/s from the main dam service spillway.

Test 13: Auxiliary spillway flow rate 160,000 ft^3/s and main dam spillway flow rate 0 ft^3/s with a tailwater elevation of 184.02 ft.

Figure 66. Test 13: Overview of flow condition with 160,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** Hydraulic jump barely contained within basin. Basin is supercritical for a distance of about 24 ft prototype (approximately 6 inches model) from last spillway step. Previous studies showed that a slight drop of 1.2 ft in tailwater produces full sweep-out of the basin. Moderate turbulence extends about 70 ft prototype (approximately 1.5 ft model) downstream from the end sill. Water regularly splashes over the auxiliary stilling basin sidewalls from the inside of the basin. Near full static head differential in the upstream portion of the basin and little water surface differential at the downstream end of the basin.

Main Dam Stilling Basin: No main dam flow. Main dam stilling basin walls submerged by tailwater.

***Right River Bank*:** Moderate wave action against right bank from the auxiliary spillway flow. Wave run-up of about 8 ft prototype (approximately 2 inches model) greatest between stations 1000 and 1300.

***Left River Bank*:** Moderate wave action against the left bank from the auxiliary spillway flow. Wave run-up of about 6 to 8 ft prototype (approximately 1.5 to 2 inches model) along the left bankline.

***Knob Topographic Feature*:** Knob and cofferdam are submerged by tailwater.

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 to 4 ft prototype (approximately 0.5 to 1 inch model).

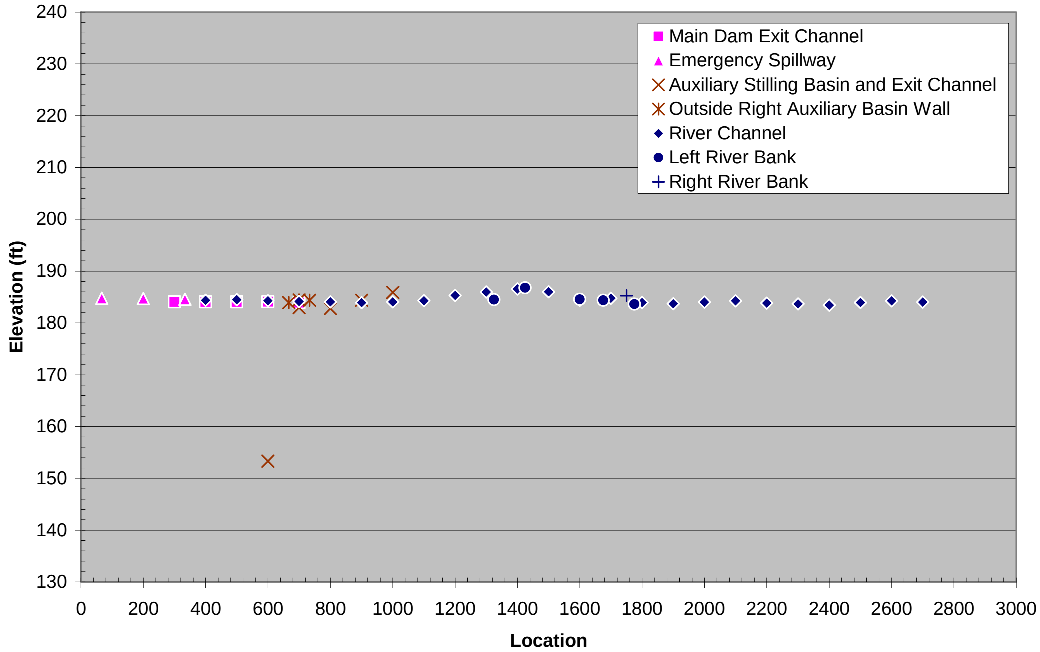

Figure 67. Test 13: Water surface profile with 160,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam service spillway. Location 600 in the auxiliary stilling basin is in the supercritical portion of the hydraulic jump.

Test 14: Auxiliary spillway flow rate 135,000 ft^3/s and main dam spillway flow rate 25,000 ft^3/s with a tailwater elevation of 184.02 ft.

Figure 68. Test 14: Overview of flow condition with 135,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** No wave action against the right auxiliary stilling basin sidewall. Water occasionally splashes over the basin sidewalls from the inside of the basin. Water surface differential at the upstream end of the basin is about 32 to 36 ft prototype (approximately 8 to 9 inches model) with little water surface differential at the downstream end of the basin. Hydraulic jump contained within the basin. Some mild turbulence extends 48 ft prototype (approximately 1 ft model) downstream from the end sill.

Main Dam Stilling Basin: Hydraulic jump fully contained within the basin. No turbulence downstream from the end sill. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Moderate wave action against the right bank from the auxiliary spillway flow. Wave run-up of about 6 to 8 ft prototype (approximately 1.5 to 2 inches model) greatest between stations 1000 and 1200.

***Left River Bank*:** Moderate wave action against the left bank. Wave run-up of about 8 ft prototype (approximately 2 inches model) greatest near station 1300.

***Knob Topographic Feature*:** Knob and cofferdam are submerged by tailwater.

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 to 4 ft prototype (approximately 0.5 to 1 inch model).

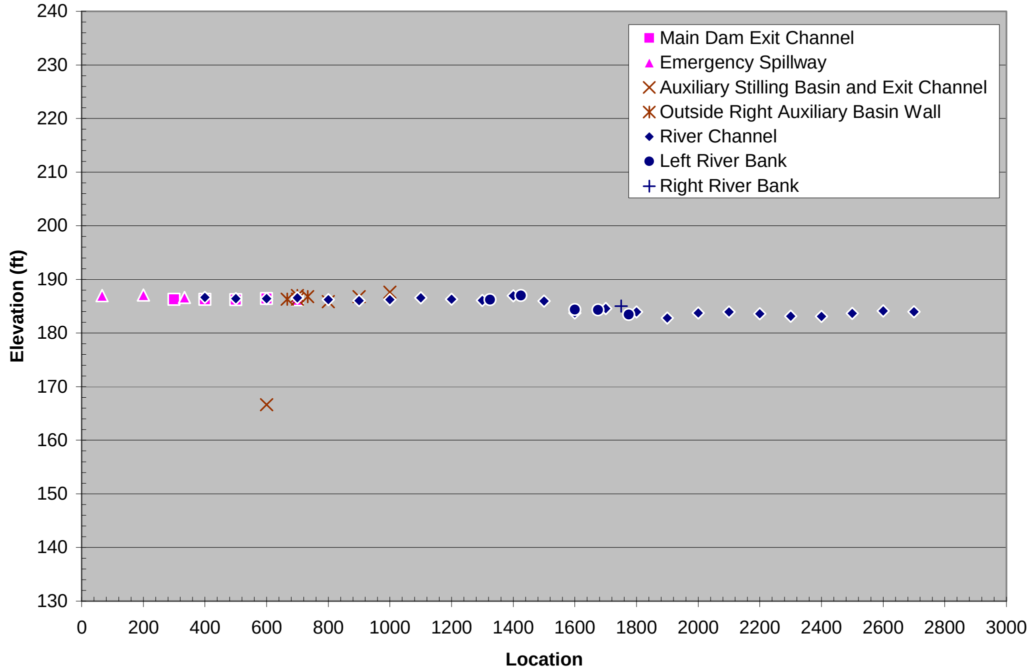

Figure 69. Test 14: Water surface profile with 135,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam service spillway. Location 600 in the auxiliary stilling basin is in the supercritical portion of the hydraulic jump.

Test 15: Auxiliary spillway flow rate 80,000 ft^3/s and main dam spillway flow rate 80,000 ft^3/s with a tailwater elevation of 184.02 ft.

Figure 70. Test 15: Overview of flow condition with 80,000 ft^3/s from the auxiliary spillway and 80,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** Water does not spill over the auxiliary stilling basin sidewalls. Water surface differential at the upstream end of the basin is 16 to 20 ft prototype (approximately 4 to 5 inches model). There is little water surface differential at the downstream end of the basin. Hydraulic jump contained within the basin.

Main Dam Stilling Basin: Hydraulic jump contained within the main dam stilling basin. Water submerges the downstream portion of the basin sidewalls. Some water pours over the upstream portion of the basin sidewalls. Minimal turbulence downstream from the end sill. Flow is contained by the main dam spillway chute walls.

***Right River Bank*:** Minimal wave action against the right bank. Wave run-up of about 4 to 6 ft prototype (approximately 1 to 1.5 inches model) along the right bankline.

***Left River Bank*:** Minimal wave action against the left bank. Wave run-up of about 6 ft prototype (approximately 1.5 inches model) greatest near station 1300.

***Knob Topographic Feature*:** Knob and cofferdam are submerged by tailwater.

***Downstream Bridge Pier*:** Water surface not up to bridge pier. Wave action minimal at about 2 to 4 ft prototype (approximately 0.5 to 1 inches model).

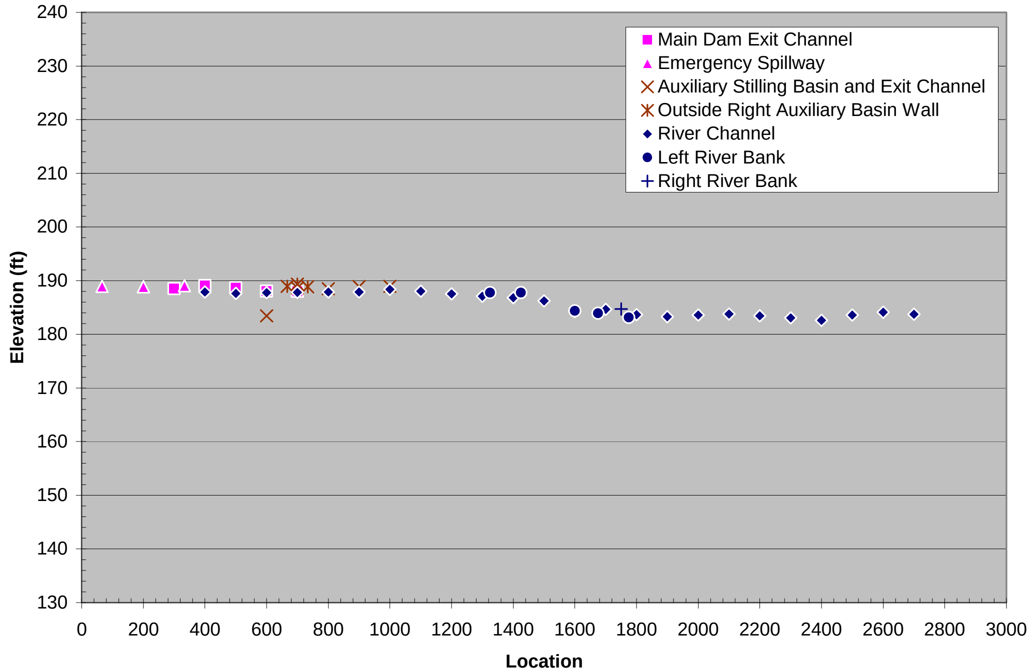

Figure 71. Test 15: Water surface profile with 80,000 ft^3/s from the auxiliary spillway and 80,000 ft^3/s from the main dam service spillway. Location 600 in the auxiliary stilling basin is in the supercritical portion of the hydraulic jump.

Test 16: Auxiliary spillway flow rate 160,000 ft^3/s and main dam spillway flow rate 140,000 ft^3/s with a tailwater elevation of 205.75 ft.

Figure 72. Test 16: Overview of flow condition with 160,000 ft^3/s from the auxiliary spillway and 140,000 ft^3/s from the main dam service spillway.

Auxiliary Stilling Basin**:** Water pours over the right auxiliary stilling basin sidewall from the main dam with a water surface differential of about 8 ft prototype (approximately 2 inches model). No water surface differential on the left auxiliary stilling basin sidewall. Hydraulic jump contained within the basin.

Main Dam Stilling Basin: Hydraulic jump contained within the basin. Turbulence extends about 48 ft prototype (approximately 1 ft model) downstream from the end sill on the right and left sides of the basin and recirculates in the center section of basin. Water submerges the basin sidewalls in the downstream portion of the basin and tailwater pours back into the basin over the upstream sidewalls. Flow is contained by the main dam spillway chute walls.

Right River Bank**:** Moderate wave action against the right bank. Wave run-up of about 8 to 10 ft prototype (approximately 2 to 2.5 inches model) along the right bankline.

Left River Bank**:** Moderate wave action against the left bank. Wave run-up of about 8 ft prototype (approximately 2 inches model) greatest near station 1300.

Knob Topographic Feature**:** Knob and cofferdam are submerged by tailwater.

Downstream Bridge Pier**:** Water surface up to the right bridge pier. Wave action minimal at about 2 to 4 ft prototype (approximately 0.5 to 1 inch model).

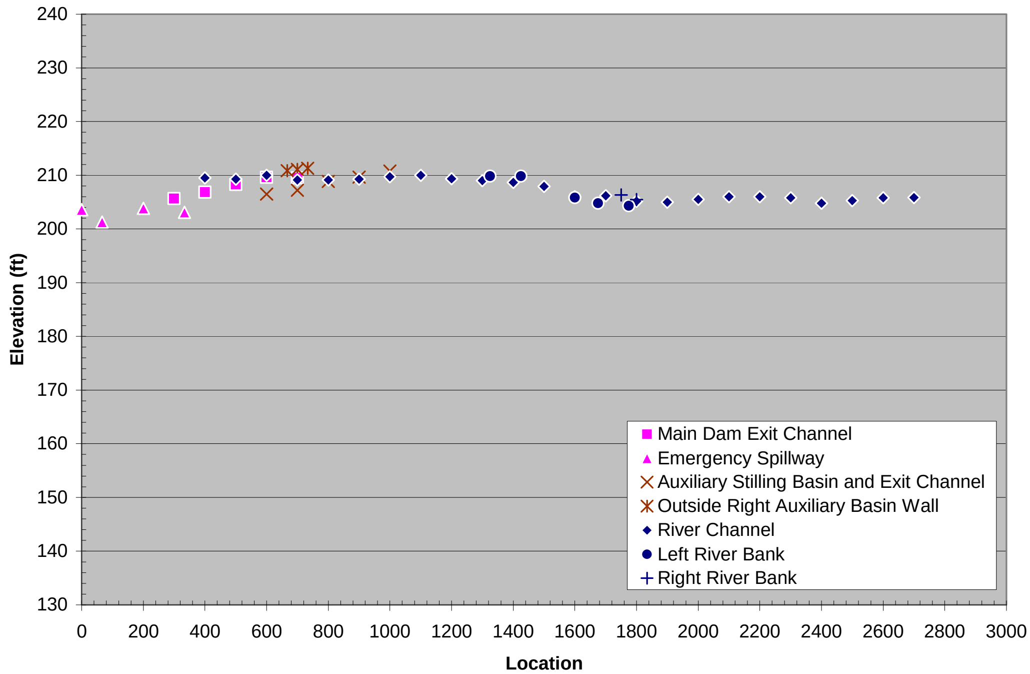

Figure 73. Test 16: Water surface profile with 160,000 ft^3/s from the auxiliary spillway and 140,000 ft^3/s from the main dam service spillway.

Test 17: Auxiliary spillway flow rate 0 ft^3/s and main dam spillway flow rate 300,000 ft^3/s with a tailwater elevation of 205.75 ft.

Figure 74. Test 17: Overview of flow condition with 0 ft^3/s from the auxiliary spillway and 300,000 ft^3/s from the main dam service spillway.

***Auxiliary Stilling Basin*:** No auxiliary spillway flow, but stilling basin is completely submerged by tailwater. Water flows over both auxiliary stilling basin walls from the main dam across to the left bankline. Wave action exists at both walls, but there is no water surface differential.

Main Dam Stilling Basin: Toe of the hydraulic jump sweeps downstream by 48 ft prototype (approximately 1 ft model). Hydraulic jump pushed out of basin with boiling 170 ft prototype (approximately 3.5 ft model) downstream from the end sill and 24 ft prototype (approximately 6 inches model) above the normal water surface. Heavy turbulence downstream from the end sill. Tailwater pours back into the stilling basin over upstream portion of the basin sidewalls. Water submerges the basin sidewalls in the downstream portion of the basin. Water splashes over the chute walls on right side and impinges on the ground outside of basin sidewall. Water occasionally splashes over left chute wall and impacts the emergency flip bucket.

***Right River Bank*:** Significant wave action against the right bank. Wave run-up of about 10 to 12 ft prototype (approximately 2.5 to 3 inches model) greatest between stations 800 and 1000.

***Left River Bank*:** Significant wave action against the left bank. Wave run-up of about 10 ft prototype (approximately 2.5 inches model) greatest near station 1300.

Knob Topographic Feature**:** Knob and cofferdam are submerged by tailwater.
Downstream Bridge Pier**:** Water surface up to the right bridge pier. Wave action minimal at about 2 to 4 ft prototype (approximately 0.5 to 1 inch model).

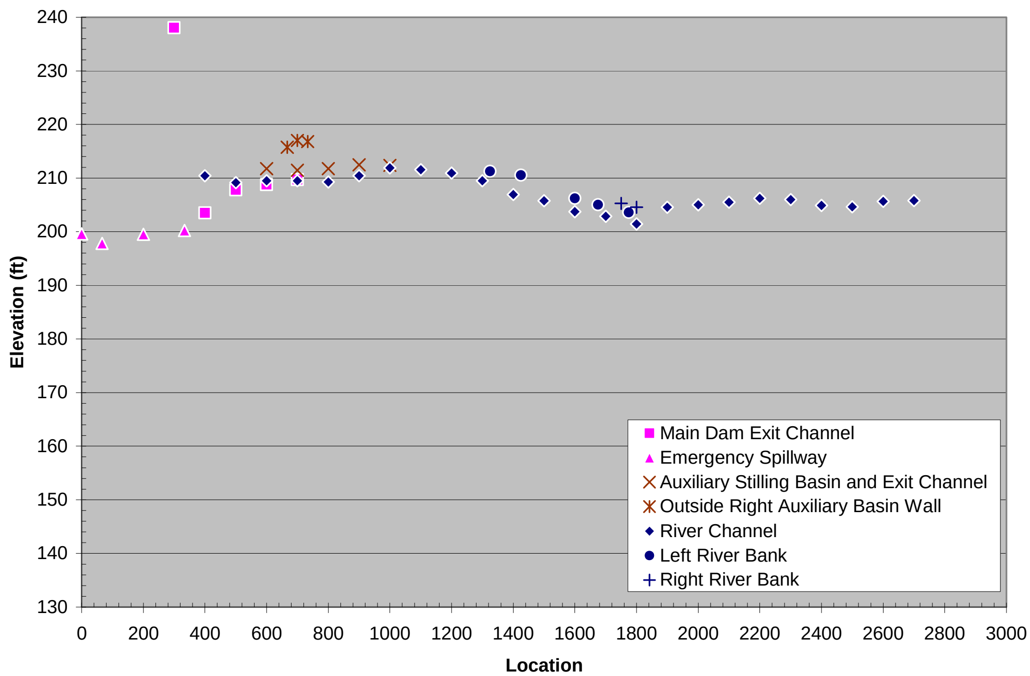

Figure 75. Test 17: Water surface profile with 0 ft^3/s from the auxiliary spillway and 300,000 ft^3/s from the main dam service spillway.

Test 18: Auxiliary spillway flow rate 312,000 ft^3/s and main dam spillway flow rate 518,000 ft^3/s with a tailwater elevation of 253.50 ft.

Figure 76. Test 18: Overview of flow condition with 312,000 ft^3/s from the auxiliary spillway and 518,000 ft^3/s from the main dam service spillway and emergency spillway.

***Auxiliary Stilling Basin*:** Flow from the main dam overtops the right auxiliary basin sidewall and continues across the width of the basin to the left bank. Significant wave action occurs above the stilling basin sidewalls with little water surface differential across the walls. Hydraulic jump not contained within the basin. Heavy turbulence observed about 430 to 480 ft prototype (approximately 9 to 10 ft model) downstream from the end sill. Auxiliary spillway chute walls are close to overtopping at the change of slope in the chute.

Main Dam Stilling Basin: Emergency gates open. There is no tailwater plunge pool for the center and left emergency spillway gates, resulting in direct impact on the slab. Tailwater slightly cushions direct impact from rightmost emergency gate. Hydraulic jump pushed out of the basin with boiling about 290 to 340 ft prototype (approximately 6 to 7 ft model) downstream from the end sill and about 36 ft prototype (approximately 9 inches model) above the normal water surface. Turbulence extends across the auxiliary stilling basin to the left bank. Power plant is completely submerged. Basin sidewalls are submerged with no water surface differential between the outside and inside of the walls. Water is on top of the spillway radial gates at a 40 ft gate opening. The rightmost gate experiences impact on the gate support. Upstream portion of the bridge piers are submerged and the downstream portion of the bridge piers are impacted by splashing water. Both right and left spillway chute walls are overtopped with impact on the adjacent banks.

Right River Bank: Significant wave action against the right bank. Wave run-up of up to 28 ft prototype (approximately 7 inches model) greatest just downstream from the powerplant to station 800. Wave run-up is 12 to 16 ft prototype (approximately 3 to 4 inches model) along the right bankline. Some eddying flow near stations 1000 to 1300 due to cove-like topographic feature.

Left River Bank: Significant wave action against the left bank. Wave run-up of 20 to 28 ft prototype (approximately 5 to 7 inches model) along the left bankline until the river curves. Water from the main dam flows past the auxiliary stilling basin and impacts the left bank.

Knob Topographic Feature: Knob and cofferdam submerged by tailwater.

Downstream Bridge Pier: Water surface up to both bridge piers. On the right side, water is about 95 ft prototype (approximately 2 ft model) up the bank from the centerline of the bridge pier. Wave run-up on the right bank by the bridge pier is about 8 ft prototype (approximately 2 inches model). Flow separation occurs downstream from the right bridge pier. On the left side, water is 70 ft prototype (approximately 1.5 ft model) up the bank from the centerline of the bridge pier. Wave run-up on the left bank is about 10 to 12 ft prototype (approximately 2.5 to 3 inches model).

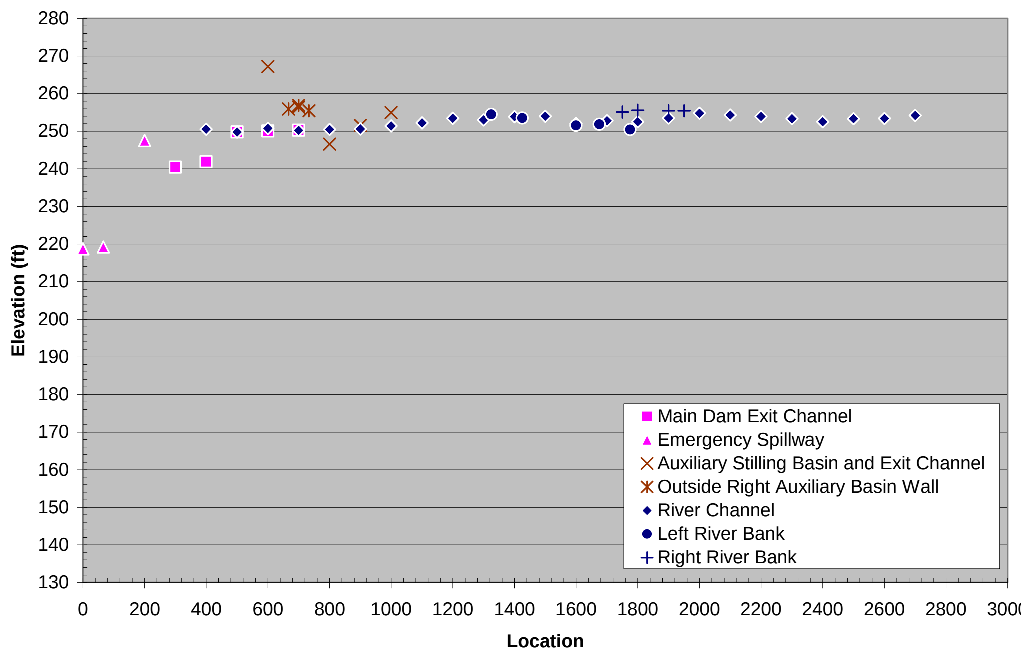

Figure 77. Test 18: Water surface profile with 312,000 ft^3/s from the auxiliary spillway and 518,000 ft^3/s from the main dam service spillway and emergency spillway.

Test 19: Auxiliary spillway flow rate 0 ft^3/s and main dam spillway flow rate 518,000 ft^3/s with a tailwater elevation of 230.00 ft.

Figure 78. Test 19: Overview of flow condition with 0 ft^3/s from the auxiliary spillway and 518,000 ft^3/s from the main dam service spillway and emergency spillway.

***Auxiliary Stilling Basin*:** Flow from the main dam overtops the right stilling basin sidewall and continues across the width of the basin to the left bank. There is significant wave action above the stilling basin sidewalls, but no water surface differential across the walls. No auxiliary spillway flow. Auxiliary stilling basin is submerged by tailwater. A counterclockwise eddy exists from left bankline back into the auxiliary stilling basin with standing water near the left bank.

Main Dam Stilling Basin: Emergency gates open. No tailwater to cushion direct impact of the jet from the emergency spillway on the downstream slab. Hydraulic jump pushed out of the stilling basin with boiling about 190 to 240 ft prototype (approximately 4 to 5 ft model) downstream from the end sill. Turbulence extends all the way to the left bank. Powerplant is almost completely submerged. Basin sidewalls are submerged in the downstream section and water flows back into the basin in the upstream section. Water is on top of the spillway radial gates at a 40 ft gate opening. The rightmost gate experiences impact on the gate support. Upstream portion of the bridge piers are submerged and the downstream portion of the bridge piers are impacted by splashing water. Both right and left spillway chute walls are overtopped with impact on the adjacent banks.

***Right River Bank*:** Significant wave action against the right bank. Wave run-up of up to 28 ft prototype (approximately 7 inches model) greatest just downstream from the powerplant to station 800. Wave run-up is about 16 to 24 ft prototype (approximately 4 to 6 inches model) along the right bankline.

***Left River Bank*:** Significant wave action against the left bank. Wave run-up of about 16 to 24 ft prototype (approximately 4 to 6 inches model) along left bank to station 1300. Near the auxiliary stilling basin, water moves laterally upstream along left bankline, then moves counterclockwise into auxiliary stilling basin.

***Knob Topographic Feature*:** Knob and cofferdam submerged by tailwater.

***Downstream Bridge Pier*:** Water surface up to both bridge piers. On the right side, water is 70 ft prototype (approximately 1.5 ft model) up the bank from the centerline of the bridge pier. Wave run-up on the right bank by the bridge pier is about 4 ft prototype (approximately 2 inches model). On the left side, water just barely covers the bridge pier with wave run-up of about 10 to 12 ft prototype (approximately 2.5 to 3 inches model).

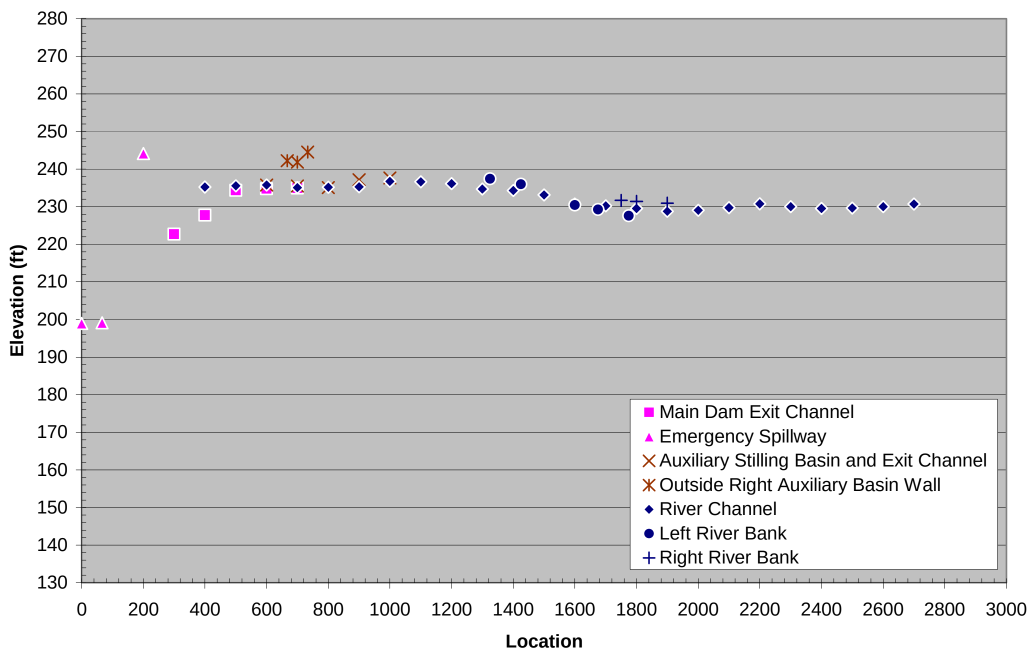

Figure 79. Test 19: Water surface profile with 0 ft^3/s from the auxiliary spillway and 518,000 ft^3/s from the main dam service spillway and emergency spillway.

Test 20: Auxiliary spillway flow rate 312,000 ft^3/s and main dam spillway flow rate 206,000 ft^3/s with a tailwater elevation of 230.00 ft.

Figure 80. Test 20: Overview of flow condition with 312,000 ft^3/s from the auxiliary spillway and 206,000 ft^3/s from the main dam service spillway with the emergency spillway gates closed.

Auxiliary Stilling Basin: Flow from main dam overtops the right auxiliary stilling basin sidewall and continues across the basin to the left bank. Significant wave action above the stilling basin walls. Hydraulic jump not contained within the auxiliary stilling basin. Heavy turbulence is observed about 290 ft prototype (approximately 6 ft model) downstream from the end sill. Auxiliary spillway chute walls are close to overtopping at the change of slope in the chute.

Main Dam Stilling Basin: Emergency gates closed. Hydraulic jump pushed out of the main dam stilling basin with boiling about 190 ft prototype (approximately 4 ft model) downstream from the end sill. Powerplant is almost completely submerged. Water pours back into the stilling basin over the upstream portion of sidewalls. Water submerges the basin sidewalls in the downstream portion of the basin. Flow is contained by the main dam spillway chute walls.

Right River Bank: Significant wave action against the right bank. Wave run-up of up to 24 ft prototype (approximately 6 inches model) greatest near station 800. Wave run-up is about 12 to 16 ft (approximately 3 to 4 inches model) along the right bankline. Some eddying flow near stations 1000 to 1300 due to a cove-like topographic feature.

***Left River Bank*:** Significant wave action against the left bank. Wave run-up of up to 24 ft prototype (approximately 6 inches model) along the left bankline.

***Knob Topographic Feature*:** Knob and cofferdam submerged by tailwater.

***Downstream Bridge Pier*:** Water surface up to both bridge piers. On the right side, water is about 70 ft prototype (approximately 1.5 ft model) up the bank from the centerline of the bridge pier. Wave run-up on the right bank by the bridge pier is about 4 ft prototype (approximately 2 inches model). On the left side, water just barely covers the bridge pier with wave run-up of about 10 to 12 ft prototype (approximately 2.5 to 3 inches model).

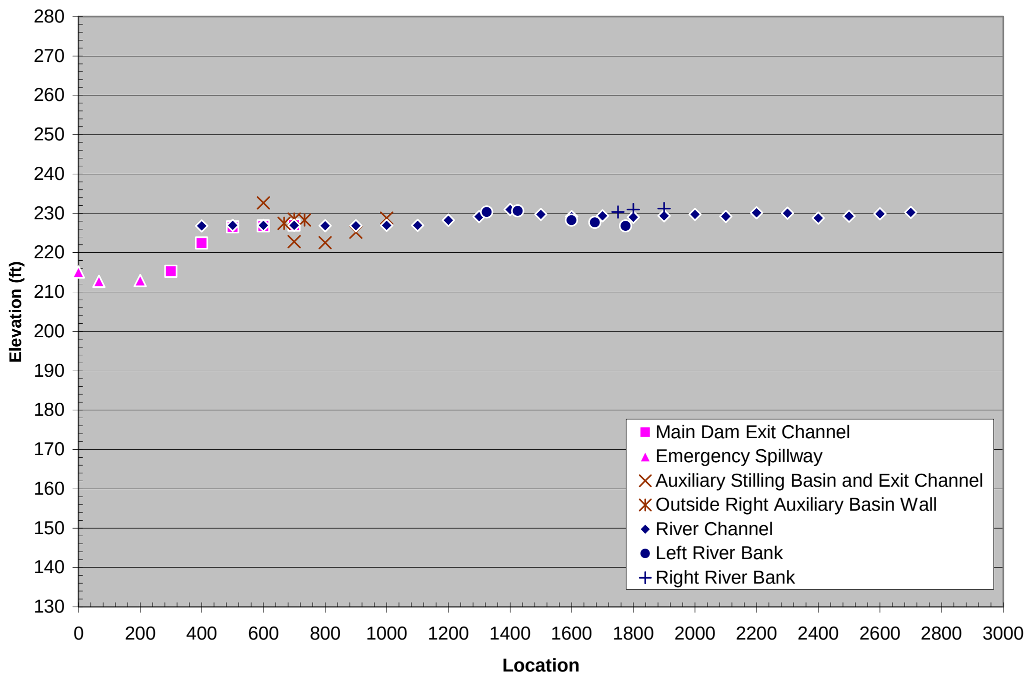

Figure 81. Test 20: Water surface profile with 312,000 ft^3/s from the auxiliary spillway and 206,000 ft^3/s from the main dam service spillway with the emergency spillway gates closed.

Channel Velocity Data

Flow velocities were measured in various locations in the American River channel and the auxiliary spillway exit channel. These data will be used to evaluate bank stability and erosion concerns in the confluence area and downstream in the river channel. Velocity data were collected with a handheld SonTek 2-D FlowTracker acoustic velocimeter during the 20 discharge combinations shown in table 6.

Velocities were measured at 15 locations in the physical model at 0.6 times the total depth from the water surface at a sample rate of 1 Hz for 40 seconds (4.6 minutes prototype). Figure 82 shows the measurement stations and velocity orientations. Measurement stations 1-10 were located in the American River channel. Due to the 10 cm (3.94 inch) offset of the sampling volume from the probe position for the

FlowTracker instrument, velocity data were collected approximately 5 inches from the right bankline in the model, corresponding to 20 ft prototype. The velocimeter was oriented so that the positive X velocity vector pointed downstream and the positive Y velocity vector pointed toward the left bank (figure 82).

Stations 11-15 were located in the auxiliary spillway stilling basin and exit channel as shown in figure 82. The velocimeter was aligned with the axis of the stilling basin with the positive X velocity vector pointed downstream and the positive Y velocity vector pointed toward the left bankline.

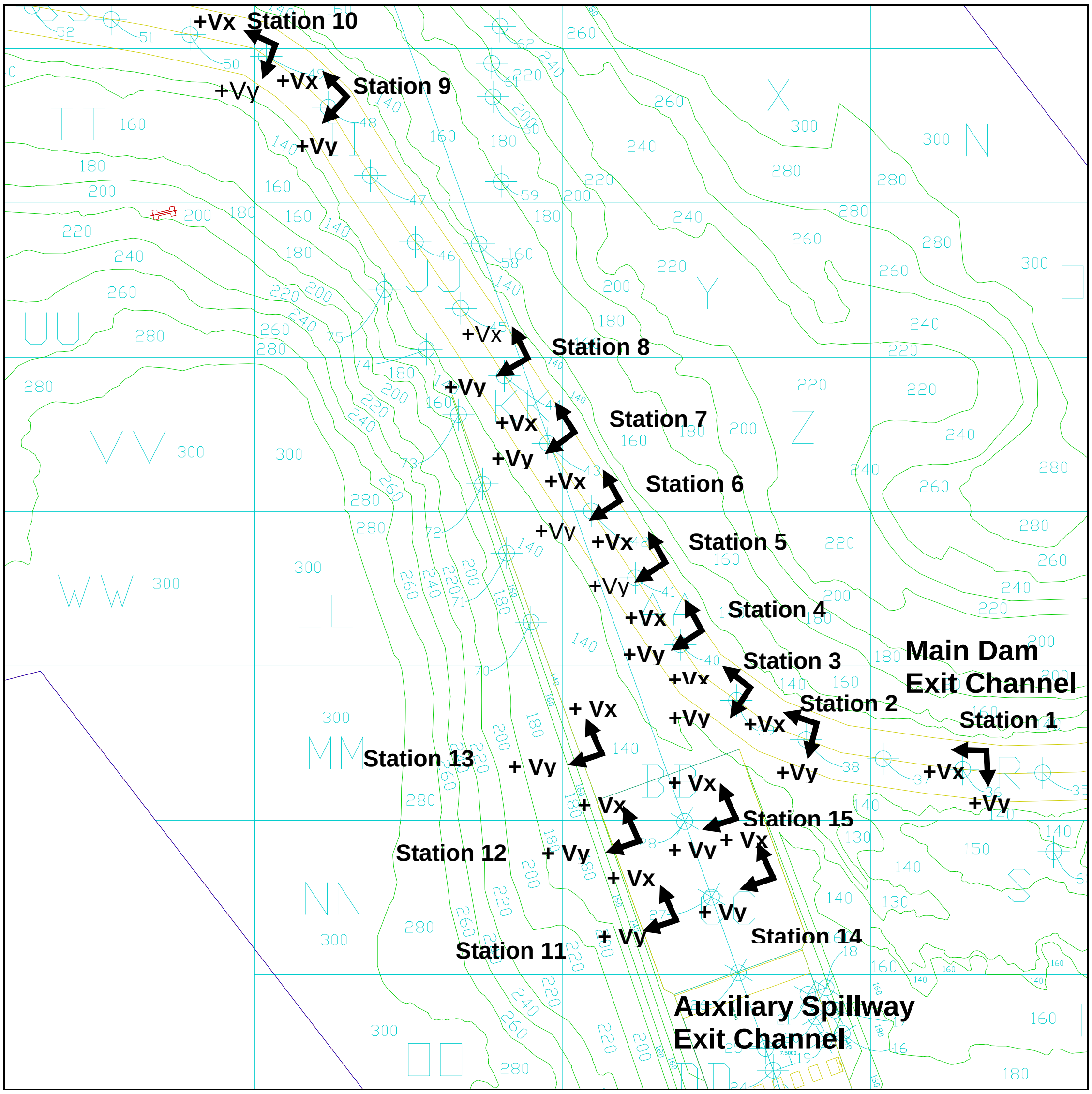

Figure 82. Velocity measurement locations and instrument orientation.

Results for Stations 1-10 in the river channel are presented in tables 7-27. In general, channel flow was in the downstream direction with transverse flow toward the right bank at most stations. A dead zone or recirculation zone existed for most test conditions at Station 1 since it is located in the powerplant tailrace. When 25,000 ft^3/s was released from the auxiliary spillway without a release from the main dam (Test 2), turbulent boiling against the right bank in the confluence was observed at Stations 2-4. With the exception of very large floods of 518,000 ft^3/s and above, the highest velocity recorded against the right bank was 5.96 ft/s at station 3 during an auxiliary spillway release of 60,000 ft^3/s. It is interesting to note that highest resultant velocities observed in the river channel during a combined flow rate of 160,000 ft^3/s were around 19.4 ft/s. Increasing the total discharge by 419 percent from 160,000 ft^3/s to 830,000 ft^3/s increases the highest resultant velocity to about 26.6 ft/s, only a 37 percent increase.

The graphs in figures 83-87 show velocities in the American River channel under different operational scenarios for the same total discharge. The flow release was either from the main dam only, the auxiliary spillway only, or a combination of the two structures. Differences in the velocity magnitude and direction between the operational scenarios were most pronounced in the confluence area (stations 2-4). Farther downstream, the velocities are similar in magnitude and direction when the total discharge is the same. In general, flows released from only the auxiliary spillway produced higher components of velocity perpendicular to the right bank at stations 2-4 in the confluence area. Flows released from the main dam produced higher components of velocity sweeping past the bank in the confluence area. Combined flow releases tended to produce the lowest velocities both into the bank and sweeping past the bank. Erosion potential or bank instability along the right bank during different operational scenarios should be analyzed by project geotechnical engineers.

Stations 11-15, downstream from the auxiliary stilling basin, were in highly turbulent, highly aerated locations. Although signal quality for the FlowTracker instrument was good, standard errors in the data were high which appeared to be caused by turbulent velocity fluctuations for flow conditions of 160,000 ft^3/s or less. For releases above 160,000 ft^3/s, collected data did not match physical observations. High levels of aeration in the flow were likely the cause. Data for tests 16-20 were discarded.

Downstream velocity vectors in the auxiliary spillway exit channel increased with increasing discharge from the auxiliary spillway. The highest measured downstream velocity vector of 21.4 ft/s was collected during an auxiliary spillway discharge of 115,000 ft^3/s. Velocities along the left bankline were generally directed away from the bankline and into the river channel. The greatest magnitude velocity vector into the left bankline was 3.5 ft/s prototype at an auxiliary spillway discharge of 160,000 ft^3/s. Main dam releases, both independently and in conjunction with auxiliary spillway releases, produced lower transverse and streamwise velocities in the auxiliary spillway exit channel.

Table 7. Test 1: Total discharge 25,000 ft^3/s with main dam spillway flow 25,000 ft^3/s and auxiliary spillway flow 0 ft^3/s. The positive X velocity vector is pointed downstream and the positive Y velocity vector is pointed toward the left bank. No flow at stations 11-15.

		MODEL				PROTOTYPE				
		Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	
Station	**Tap**									**NOTES**
1	36	0.6	0.16	0.03	0.16	28.8	1.11	0.21	1.13	Dead zone
2	38	0.6	0.93	-0.49	1.05	28.8	6.44	-3.39	7.28	
3	39	0.6	1.42	0.14	1.43	28.8	9.84	0.97	9.89	
4	40	0.65	1.35	0.02	1.35	31.2	9.35	0.14	9.35	
5	41	0.65	1.01	-0.05	1.01	31.2	7.00	-0.35	7.01	
6	42	0.65	1.11	-0.04	1.11	31.2	7.69	-0.28	7.70	
7	43	0.65	1.05	0.00	1.05	31.2	7.27	0.00	7.27	
8	44	0.7	1.14	-0.06	1.14	33.6	7.90	-0.42	7.91	
9	48	0.7	1.07	-0.02	1.07	33.6	7.41	-0.14	7.41	
10	49	0.7	1.05	0.02	1.05	33.6	7.27	0.14	7.28	

Table 8. Test 2: Total discharge 25,000 ft^3/s with main dam spillway flow 0 ft^3/s and auxiliary spillway flow 25,000 ft^3/s.

		MODEL				PROTOTYPE				
		Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	
Station	**Tap**									**NOTES**
1	36	0.58	0.00	-0.08	0.08	27.8	0.00	-0.55	0.55	Dead zone
2	38	0.58	0.74	-0.32	0.81	27.8	5.13	-2.22	5.59	Turbulent boiling
3	39	0.55	0.86	-0.44	0.97	26.4	5.96	-3.05	6.69	Turbulent boiling
4	40	0.6	1.44	-0.31	1.47	28.8	9.98	-2.15	10.2	Turbulent boiling
5	41	0.6	1.45	0.02	1.45	28.8	10.0	0.14	10.0	
6	42	0.55	1.30	-0.02	1.30	26.4	9.01	-0.14	9.01	
7	43	0.55	1.35	-0.14	1.36	26.4	9.35	-0.97	9.40	
8	44	0.55	1.67	-0.05	1.67	26.4	11.6	-0.35	11.6	
9	48	0.55	1.63	-0.09	1.63	26.4	11.3	-0.62	11.3	
10	49	0.55	1.59	-0.03	1.59	26.4	11.0	-0.21	11.0	
11	27A	0.4	0.07	-0.04	0.08	19.2	-0.28	-0.28	0.4	Located in eddy
11CL	27CL	0.4	1.67	-0.29	1.69	19.2	-2.01	-2.01	2.8	Centerline value
12	28A	0.35	0.63	0.28	0.69	16.8	1.94	1.94	2.7	
13	Left bank	0	0.00	0	0.00	0	0	0.00	0	No flow
14	27B	0.45	1.32	-0.02	1.32	21.6	-0.14	-0.14	0.2	
15	28B	0.3	1.42	-0.42	1.48	14.4	-2.91	-2.91	4.1	

Table 9. Test 3: Total discharge 60,000 ft^3/s with main dam spillway flow 60,000 ft^3/s and auxiliary spillway flow 0 ft^3/s. No flow at stations 11-15.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	0.9	0.17	0.28	0.33	43.2	1.18	1.94	2.27	Dead zone
2	38	0.9	1.28	-0.34	1.32	43.2	8.87	-2.36	9.18	
3	39	0.9	1.59	-0.10	1.59	43.2	11.02	-0.69	11.04	
4	40	0.9	1.74	0.01	1.74	43.2	12.06	0.07	12.06	
5	41	0.9	1.56	-0.15	1.57	43.2	10.81	-1.04	10.86	
6	42	0.9	1.69	-0.25	1.71	43.2	11.71	-1.73	11.84	
7	43	0.9	1.85	-0.19	1.86	43.2	12.82	-1.32	12.88	
8	44	0.9	2.19	-0.32	2.21	43.2	15.17	-2.22	15.33	
9	48	0.9	2.39	-0.51	2.44	43.2	16.56	-3.53	16.93	
10	49	0.9	2.36	-0.47	2.41	43.2	16.35	-3.26	16.67	

Table 10. Test 4: Total discharge 60,000 ft^3/s with main dam spillway flow 0 ft^3/s and auxiliary spillway flow 60,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	0.98	-0.07	0.17	0.18	47.0	-0.48	1.18	1.27	Dead zone
2	38	0.95	0.2	-0.06	0.21	45.6	1.39	-0.42	1.45	
3	39	1	1.21	-0.86	1.48	48.0	8.38	-5.96	10.3	
4	40	1	1.41	-0.21	1.43	48.0	9.77	-1.45	9.88	
5	41	1	1.77	-0.38	1.81	48.0	12.3	-2.63	12.5	
6	42	1	2.11	0.07	2.11	48.0	14.6	0.48	14.6	
7	43	1	1.58	0.08	1.58	48.0	10.9	0.55	11.0	
8	44	1	2.04	0.05	2.04	48.0	14.1	0.35	14.1	
9	48	1	2.13	-0.08	2.14	48.0	14.8	-0.58	14.8	
10	49	1	2.08	0.01	2.08	48.0	14.4	0.07	14.4	
11	27A	0.55	1.83	-0.44	1.88	26.4	12.68	-3.05	13.0	
12	28A	0.5	1.11	0.21	1.13	24.0	7.69	1.45	7.8	
13	Left Bank	0.22	0	0	0	10.6	0	0	0	Too shallow
14	27B	0.65	2.09	-0.27	2.11	31.2	14.48	-1.87	14.6	
15	28B	0.55	1.91	-0.59	2.00	26.4	13.23	-4.09	13.8	

Table 11. Test 5: Total discharge 60,000 ft^3/s with main dam spillway flow 25,000 ft^3/s and auxiliary spillway flow 35,000 ft^3/s.

		MODEL				PROTOTYPE				
		Flow Depth	Vx	Vy	Resultant	Flow Depth	Vx	Vy	Resultant	
Station	Tap	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	NOTES
1	36	1	0.08	0.00	0.08	48.0	0.56	0.00	0.56	Dead zone
2	38	1	0.00	0.10	0.10	48.0	0.03	0.71	0.71	Dead zone
3	39	1	0.79	-0.33	0.86	48.0	5.49	-2.30	5.96	
4	40	1	1.28	-0.34	1.32	48.0	8.87	-2.35	9.18	
5	41	1	1.35	-0.10	1.35	48.0	9.33	-0.66	9.35	
6	42	1.05	1.48	-0.06	1.48	50.4	10.3	-0.41	10.3	
7	43	1	1.60	-0.12	1.60	48.0	11.1	-0.80	11.1	
8	44	1.1	1.71	-0.29	1.73	52.8	11.8	-1.98	12.0	
9	48	1.1	1.74	-0.30	1.77	52.8	12.1	-2.08	12.2	
10	49	1.05	1.65	-0.38	1.69	50.4	11.4	-2.65	11.7	
11	27A	0.65	0.86	0.12	0.87	31.2	5.94	0.80	6.0	
12	28A	0.55	0.67	0.04	0.67	26.4	4.62	0.26	4.6	
13	Left Bank	0.2	1.34	-0.34	1.38	9.6	9.29	-2.33	9.6	
14	27B	0.7	1.17	0.00	1.17	33.6	8.10	0.02	8.1	
15	28B	0.6	0.68	-0.12	0.69	28.8	4.68	-0.83	4.8	

Table 12. Test 6: Total discharge 90,000 ft^3/s with main dam spillway flow 90,000 ft^3/s and auxiliary spillway flow 0 ft^3/s.

		MODEL				PROTOTYPE				
		Flow Depth	Vx	Vy	Resultant	Flow Depth	Vx	Vy	Resultant	
Station	Tap	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	NOTES
1	36	1.1	0.34	-0.10	0.35	52.8	2.36	-0.69	2.46	
2	38	1.15	1.19	-0.17	1.20	55.2	8.24	-1.18	8.33	
3	39	1.15	1.42	0.02	1.42	55.2	9.84	0.14	9.84	
4	40	1.15	1.44	-0.02	1.44	55.2	9.98	-0.14	9.98	
5	41	1.1	1.57	0.00	1.57	52.8	10.9	0.00	10.9	
6	42	1.1	1.83	-0.01	1.83	52.8	12.7	-0.07	12.7	
7	43	1.1	2.01	-0.20	2.02	52.8	13.9	-1.39	14.0	
8	44	1.1	2.33	-0.44	2.37	52.8	16.1	-3.05	16.4	
9	48	1.1	2.52	-0.40	2.55	52.8	17.5	-2.77	17.7	
10	49	1.1	2.60	-0.54	2.66	52.8	18.0	-3.74	18.4	
11	27A	0.7	-0.38	-0.27	0.47	33.6	-2.63	-1.87	3.2	
12	28A	0.6	-0.70	0.18	0.72	28.8	-4.85	1.25	5.0	
13	Left Bank	0.3	0.51	0.35	0.62	14.4	3.53	2.42	4.3	
14	27B	0.7	0.38	-0.13	0.40	33.6	2.63	-0.90	2.8	
15	28B	0.6	0.56	0.41	0.69	28.8	3.88	2.84	4.8	

Table 13. Test 7: Total discharge 90,000 ft^3/s with main dam spillway flow 0 ft^3/s and auxiliary spillway flow 90,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	1.15	-0.06	-0.02	0.06	55.2	-0.42	-0.14	0.44	Dead zone
2	38	1.1	0.18	0.1	0.21	52.8	1.25	0.69	1.43	
3	39	1.2	0.45	-0.53	0.70	57.6	3.12	-3.67	4.82	
4	40	1.2	1.15	-0.42	1.22	57.6	7.97	-2.91	8.48	
5	41	1.2	1.71	-0.35	1.75	57.6	11.8	-2.42	12.1	
6	42	1.2	1.96	-0.04	1.96	57.6	13.6	-0.28	13.6	
7	43	1.25	2.16	-0.12	2.16	60.0	15.0	-0.83	15.0	
8	44	1.15	2.31	-0.17	2.32	55.2	16.0	-1.18	16.0	
9	48	1.15	2.53	-0.11	2.53	55.2	17.5	-0.76	17.5	
10	49	1.15	2.46	-0.21	2.47	55.2	17.0	-1.45	17.1	
11	27A	0.7	2.05	-0.38	2.08	33.6	14.20	-2.63	14.4	
12	28A	0.65	1.59	-0.47	1.66	31.2	11.02	-3.26	11.5	
13	Left Bank	0.35	2.58	-0.81	2.70	16.8	17.87	-5.61	18.7	
14	27B	0.8	2.04	-0.22	2.05	38.4	14.13	-1.52	14.2	
15	28B	0.65	2.01	-0.83	2.17	31.2	13.93	-5.75	15.1	

Table 14. Test 8: Total discharge 90,000 ft^3/s with main dam spillway flow 25,000 ft^3/s and auxiliary spillway flow 65,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	1.15	-0.04	0.06	0.07	55.2	-0.31	0.39	0.50	Dead zone
2	38	1.15	0.04	-0.04	0.06	55.2	0.28	-0.29	0.40	Dead zone
3	39	1.15	0.65	-0.15	0.67	55.2	4.54	-1.01	4.65	
4	40	1.15	1.44	-0.13	1.44	55.2	9.95	-0.92	9.99	
5	41	1.15	1.83	-0.22	1.84	55.2	12.7	-1.53	12.8	
6	42	1.15	2.15	-0.21	2.16	55.2	14.9	-1.44	15.0	
7	43	1.1	2.24	-0.48	2.29	52.8	15.5	-3.30	15.9	
8	44	1.15	2.68	-0.24	2.70	55.2	18.6	-1.63	18.7	
9	48	1.15	2.75	-0.67	2.83	55.2	19.1	-4.68	19.6	
10	49	1.1	2.73	-0.67	2.81	52.8	18.9	-4.65	19.5	
11	27A	0.7	1.91	-0.07	1.91	33.6	13.21	-0.52	13.2	
12	28A	0.6	1.32	0.00	1.32	28.8	9.12	0.02	9.1	
13	Left Bank	0.2	2.05	-0.84	2.22	9.6	14.23	-5.79	15.4	
14	27B	0.8	2.11	-0.05	2.11	38.4	14.64	-0.37	14.6	
15	28B	0.65	1.45	-0.12	1.45	31.2	10.01	-0.83	10.0	

Table 15. Test 9: Total discharge 115,000 ft^3/s with main dam spillway flow 115,000 ft^3/s and auxiliary spillway flow 0 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	1.2	0.19	-0.26	0.32	57.6	1.32	-1.80	2.23	
2	38	1.3	1.46	-0.31	1.49	62.4	10.1	-2.15	10.3	
3	39	1.3	1.79	-0.09	1.79	62.4	12.4	-0.62	12.4	
4	40	1.3	2.18	0.05	2.18	62.4	15.1	0.35	15.1	
5	41	1.3	2.06	0.00	2.06	62.4	14.3	0.00	14.3	
6	42	1.3	1.96	-0.12	1.96	62.4	13.6	-0.83	13.6	
7	43	1.3	2.12	-0.37	2.15	62.4	14.7	-2.56	14.9	
8	44	1.3	2.49	-0.34	2.51	62.4	17.3	-2.36	17.4	
9	48	1.3	2.54	-0.42	2.57	62.4	17.6	-2.91	17.8	
10	49	1.3	2.57	-0.43	2.61	62.4	17.8	-2.98	18.1	
11	27A	0.85	-0.19	-0.05	0.20	40.8	-1.32	-0.35	1.4	
12	28A	0.8	-0.25	-0.02	0.25	38.4	-1.73	-0.14	1.7	
13	Left Bank	0.45	-0.03	0.05	0.06	21.6	-0.21	0.35	0.4	
14	27B	0.85	0.07	-0.11	0.13	40.8	0.48	-0.76	0.9	
15	28B	0.75	0.14	-0.05	0.15	36.0	0.97	-0.35	1.0	

Table 16. Test 10: Total discharge 115,000 ft^3/s with main dam spillway flow 25,000 ft^3/s and auxiliary spillway flow 90,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	1.35	-0.11	0.01	0.11	64.8	-0.79	0.05	0.79	Dead zone
2	38	1.35	0.16	-0.16	0.22	64.8	1.08	-1.12	1.55	
3	39	1.3	0.44	0.02	0.45	62.4	3.08	0.15	3.08	
4	40	1.3	1.36	-0.39	1.41	62.4	9.41	-2.67	9.78	
5	41	1.3	1.78	-0.19	1.79	62.4	12.3	-1.31	12.4	
6	42	1.35	2.04	0.03	2.04	64.8	14.2	0.20	14.2	
7	43	1.35	2.26	-0.29	2.28	64.8	15.7	-2.04	15.8	
8	44	1.35	2.55	-0.28	2.56	64.8	17.7	-1.91	17.8	
9	48	1.35	2.77	-0.45	2.81	64.8	19.2	-3.13	19.5	
10	49	1.3	2.45	-0.56	2.51	62.4	17.0	-3.86	17.4	
11	27A	0.9	1.82	0.11	1.82	43.2	12.61	0.78	12.6	
12	28A	0.8	1.45	0.00	1.45	38.4	10.03	0.00	10.0	
13	Left Bank	0.45	2.30	-0.49	2.35	21.6	15.93	-3.38	16.3	
14	27B	0.95	2.08	0.05	2.08	45.6	14.43	0.37	14.4	
15	28B	0.85	1.96	-0.56	2.04	40.8	13.61	-3.88	14.2	

Table 17. Test 11: Total discharge 115,000 ft^3/s with main dam spillway flow 0 ft^3/s and auxiliary spillway flow 115,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	1.25	-0.26	0.19	0.32	60.0	-1.80	1.32	2.23	Dead zone
2	38	1.25	0.06	-0.02	0.06	60.0	0.42	-0.14	0.44	Dead zone
3	39	1.3	0.74	-0.66	0.99	62.4	5.13	-4.57	6.87	
4	40	1.35	1.24	-0.54	1.35	64.8	8.59	-3.74	9.37	
5	41	1.35	1.9	-0.25	1.92	64.8	13.2	-1.73	13.3	
6	42	1.35	2.25	0	2.25	64.8	15.6	0.00	15.6	
7	43	1.3	2.2	-0.05	2.20	62.4	15.2	-0.35	15.2	
8	44	1.3	2.6	-0.12	2.60	62.4	18.0	-0.83	18.0	
9	48	1.3	2.72	-0.16	2.72	62.4	18.8	-1.11	18.9	
10	49	1.3	2.47	-0.45	2.51	62.4	17.1	-3.12	17.4	
11	27A	0.9	2.84	0.04	2.84	43.2	19.68	0.28	19.7	
12	28A	0.8	1.64	-0.06	1.64	38.4	11.36	-0.42	11.4	
13	Left Bank	0.5	2.7	-0.51	2.75	24.0	18.71	-3.53	19.0	
14	27B	0.9	3.09	0.17	3.09	43.2	21.41	1.18	21.4	
15	28B	0.85	2.48	-0.6	2.55	40.8	17.18	-4.16	17.7	

Table 18. Test 12: Total discharge 160,000 ft^3/s with main dam spillway flow 160,000 ft^3/s and auxiliary spillway flow 0 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	1.5	-0.4	0.05	0.40	72.0	-2.77	0.35	2.79	Recirculation zone
2	38	1.5	1.08	-0.53	1.20	72.0	7.48	-3.67	8.33	
3	39	1.55	1.6	-0.19	1.61	74.4	11.1	-1.32	11.2	
4	40	1.5	2.18	-0.18	2.19	72.0	15.1	-1.25	15.2	
5	41	1.55	2.23	0.01	2.23	74.4	15.4	0.07	15.5	
6	42	1.5	2.33	0.01	2.33	72.0	16.1	0.07	16.1	
7	43	1.5	2.42	-0.3	2.44	72.0	16.8	-2.08	16.9	
8	44	1.55	2.77	-0.4	2.80	74.4	19.2	-2.77	19.4	
9	48	1.55	2.63	-0.38	2.66	74.4	18.2	-2.63	18.4	
10	49	1.55	2.53	-0.6	2.60	74.4	17.5	-4.16	18.0	
11	27A	1.1	-0.14	-0.05	0.15	52.8	-0.97	-0.35	1.0	
12	28A	1	-0.09	0.25	0.27	48.0	-0.62	1.73	1.8	
13	Left Bank	0.55	0.83	0.05	0.83	26.4	5.75	0.35	5.8	
14	27B	1.15	0.87	0.55	1.03	55.2	6.03	3.81	7.1	
15	28B	1	0.49	0.37	0.61	48.0	3.39	2.56	4.3	

Table 19. Test 13: Total discharge 160,000 ft^3/s with main dam spillway flow 0 ft^3/s and auxiliary spillway flow 160,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	1.45	-0.09	0.03	0.09	69.6	-0.62	0.21	0.66	Recirculation zone
2	38	1.45	-0.1	0.1	0.14	69.6	-0.69	0.69	0.98	Recirculation zone
3	39	1.5	0.22	-0.13	0.26	72.0	1.52	-0.90	1.77	
4	40	1.6	1.41	-0.54	1.51	76.8	9.77	-3.74	10.5	
5	41	1.55	1.36	-0.12	1.37	74.4	9.42	-0.83	9.46	
6	42	1.55	1.85	0	1.85	74.4	12.8	0.00	12.8	
7	43	1.55	2.24	0.1	2.24	74.4	15.5	0.69	15.5	
8	44	1.5	2.67	0.05	2.67	72.0	18.5	0.35	18.5	
9	48	1.55	2.63	0.1	2.63	74.4	18.2	0.69	18.2	
10	49	1.55	2.59	-0.27	2.60	74.4	17.9	-1.87	18.0	
11	27A	1.1	2.85	0.51	2.90	52.8	19.75	3.53	20.1	
12	28A	1.05	1.67	-0.04	1.67	50.4	11.57	-0.28	11.6	
13	Left Bank	0.7	2.63	-0.35	2.65	33.6	18.22	-2.42	18.4	
14	27B	1.1	3.06	0.48	3.10	52.8	21.20	3.33	21.5	
15	28B	1	2.94	-0.37	2.96	48.0	20.37	-2.56	20.5	

Table 20. Test 14: Total discharge 160,000 ft^3/s with main dam spillway flow 25,000 ft^3/s and auxiliary spillway flow 135,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	1.5	0.01	0.01	0.01	72.0	0.04	0.07	0.08	Dead zone
2	38	1.5	0.06	0.04	0.08	72.0	0.44	0.29	0.53	Dead zone
3	39	1.5	0.19	0.04	0.20	72.0	1.33	0.31	1.36	
4	40	1.55	1.15	-0.27	1.18	74.4	7.93	-1.85	8.15	
5	41	1.55	1.82	-0.33	1.85	74.4	12.6	-2.29	12.8	
6	42	1.55	2.29	-0.12	2.29	74.4	15.8	-0.80	15.9	
7	43	1.55	2.47	-0.07	2.47	74.4	17.1	-0.51	17.1	
8	44	1.55	2.70	0.02	2.70	74.4	18.7	0.11	18.7	
9	48	1.5	2.80	-0.14	2.80	72.0	19.4	-0.94	19.4	
10	49	1.5	2.59	-0.44	2.63	72.0	17.9	-3.01	18.2	
11	27A	1.1	2.24	-0.09	2.25	52.8	15.55	-0.63	15.6	
12	28A	1	1.54	-0.22	1.56	48.0	10.68	-1.51	10.8	
13	Left Bank	0.55	2.32	-0.52	2.38	26.4	16.06	-3.63	16.5	
14	27B	1.1	2.40	0.33	2.42	52.8	16.63	2.28	16.8	
15	28B	1.05	2.35	-0.54	2.41	50.4	16.27	-3.75	16.7	

Table 21. Test 15: Total discharge 160,000 ft^3/s with main dam spillway flow 80,000 ft^3/s and auxiliary spillway flow 80,000 ft^3/s.

		MODEL				PROTOTYPE				
		Flow Depth	Vx	Vy	Resultant	Flow Depth	Vx	Vy	Resultant	
Station	Tap	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	NOTES
1	36	1.6	-0.11	0.11	0.15	76.8	-0.73	0.75	1.05	Recirculation zone
2	38	1.55	0.83	-0.34	0.90	74.4	5.76	-2.37	6.23	
3	39	1.6	1.22	-0.10	1.23	76.8	8.48	-0.72	8.52	
4	40	1.6	1.41	-0.04	1.41	76.8	9.78	-0.30	9.79	
5	41	1.6	1.71	0.05	1.71	76.8	11.8	0.38	11.9	
6	42	1.55	2.03	0.12	2.03	74.4	14.0	0.83	14.1	
7	43	1.5	2.31	-0.09	2.31	72.0	16.0	-0.61	16.0	
8	44	1.55	2.64	-0.27	2.66	74.4	18.3	-1.90	18.4	
9	48	1.55	2.51	-0.14	2.51	74.4	17.4	-0.97	17.4	
10	49	1.55	2.41	-0.37	2.44	74.4	16.7	-2.53	16.9	
11	27A	1.1	0.91	-0.10	0.92	52.8	6.31	-0.71	6.3	
12	28A	1	0.78	-0.08	0.78	48.0	5.39	-0.53	5.4	
13	Left Bank	0.65	1.53	-0.29	1.56	31.2	10.60	-2.01	10.8	
14	27B	1.2	1.41	0.10	1.41	57.6	9.76	0.67	9.8	
15	28B	1.05	1.40	-0.02	1.40	50.4	9.70	-0.17	9.7	

Table 22. Test 16: Total discharge 300,000 ft^3/s with main dam spillway flow 140,000 ft^3/s and auxiliary spillway flow 160,000 ft^3/s.

		MODEL				PROTOTYPE				
		Flow Depth	Vx	Vy	Resultant	Flow Depth	Vx	Vy	Resultant	
Station	Tap	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	NOTES
1	36	2	-0.25	0.14	0.28	96.0	-1.70	0.97	1.96	Recirculation zone
2	38	2	0.49	-0.34	0.60	96.0	3.37	-2.38	4.13	
3	39	2	0.90	-0.27	0.94	96.0	6.26	-1.88	6.53	
4	40	2	1.77	-0.25	1.79	96.0	12.3	-1.73	12.4	
5	41	2	1.69	-0.07	1.69	96.0	11.7	-0.45	11.7	
6	42	2	2.26	-0.12	2.27	96.0	15.7	-0.84	15.7	
7	43	1.9	2.69	-0.45	2.72	91.2	18.6	-3.09	18.9	
8	44	1.95	2.84	-0.21	2.84	93.6	19.7	-1.46	19.7	
9	48	1.95	3.04	-0.11	3.04	93.6	21.0	-0.78	21.1	
10	49	1.95	2.94	-0.54	2.99	93.6	20.4	-3.71	20.7	

Table 23. Test 17: Total discharge 300,000 ft^3/s with main dam spillway flow 300,000 ft^3/s and auxiliary spillway flow 0 ft^3/s.

		MODEL				PROTOTYPE				
		Flow Depth	Vx	Vy	Resultant	Flow Depth	Vx	Vy	Resultant	
Station	Tap	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	NOTES
1	36	2	-0.39	0.26	0.47	96.0	-2.68	1.80	3.23	Recirculation zone
2	38	2	1.26	-0.67	1.43	96.0	8.73	-4.62	9.88	
3	39	2	1.56	-0.60	1.67	96.0	10.8	-4.13	11.6	
4	40	2	2.13	-0.52	2.20	96.0	14.8	-3.63	15.2	
5	41	2	2.34	-0.44	2.38	96.0	16.2	-3.02	16.5	
6	42	2	2.39	-0.38	2.42	96.0	16.6	-2.66	16.8	
7	43	2	2.71	-0.74	2.81	96.0	18.8	-5.10	19.5	
8	44	2	3.22	-0.31	3.23	96.0	22.3	-2.15	22.4	
9	48	2	3.00	-0.13	3.01	96.0	20.8	-0.92	20.8	
10	49	1.95	2.78	-0.88	2.92	93.6	19.3	-6.11	20.2	

Table 24. Test 18: Total discharge 830,000 ft^3/s with main dam spillway flow 518,000 ft^3/s and auxiliary spillway flow 312,000 ft^3/s.

		MODEL				PROTOTYPE				
		Flow Depth	Vx	Vy	Resultant	Flow Depth	Vx	Vy	Resultant	
Station	Tap	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	(ft)	(ft/s)	(ft/s)	Velocity (ft/s)	NOTES
1	36	2.85	-0.25	0.07	0.26	136.8	-1.70	0.49	1.77	Recirculation zone
2	38	2.85	0.26	-0.45	0.52	136.8	1.82	-3.11	3.61	
3	39	2.85	1.22	-0.67	1.39	136.8	8.48	-4.63	9.66	
4	40	2.9	2.34	-1.92	3.03	139.2	16.2	-13.3	21.0	
5	41	2.9	3.48	-1.31	3.72	139.2	24.1	-9.11	25.8	
6	42	2.9	3.28	-0.89	3.40	139.2	22.8	-6.18	23.6	
7	43	2.9	3.44	-0.96	3.57	139.2	23.8	-6.68	24.8	
8	44	2.9	3.61	-1.31	3.84	139.2	25.0	-9.04	26.6	
9	48	2.95	3.27	0.39	3.29	141.6	22.6	2.67	22.8	
10	49	2.95	3.10	-0.78	3.20	141.6	21.5	-5.39	22.2	

Table 25. Test 19: Total discharge 518,000 ft^3/s with main dam spillway flow 518,000 ft^3/s and auxiliary spillway flow 0 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	2.55	-0.31	0.19	0.37	122.4	-2.16	1.34	2.55	Recirculation zone
2	38	2.55	0.76	-0.42	0.87	122.4	5.24	-2.92	6.00	
3	39	2.55	1.54	-0.58	1.65	122.4	10.7	-3.99	11.4	
4	40	2.55	2.11	-0.98	2.33	122.4	14.6	-6.82	16.1	
5	41	2.55	2.15	-0.67	2.25	122.4	14.9	-4.62	15.6	
6	42	2.5	2.56	-0.36	2.58	120.0	17.7	-2.47	17.9	
7	43	2.5	2.82	-0.58	2.88	120.0	19.6	-4.00	20.0	
8	44	2.5	3.34	-0.90	3.46	120.0	23.2	-6.26	24.0	
9	48	2.5	3.35	0.34	3.36	120.0	23.2	2.35	23.3	
10	49	2.45	3.40	-0.35	3.42	117.6	23.6	-2.41	23.7	

Table 26. Test 20 with emergency gates closed: Total discharge 518,000 ft^3/s with main dam spillway flow 206,000 ft^3/s and auxiliary spillway flow 312,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	2.35	-0.36	0.14	0.39	112.8	-2.52	0.95	2.69	Recirculation zone
2	38	2.35	-0.19	0.00	0.19	112.8	-1.28	0.00	1.28	Recirculation zone
3	39	2.35	-0.25	0.36	0.44	112.8	-1.76	2.47	3.04	Recirculation zone
4	40	2.35	0.53	-0.34	0.63	112.8	3.67	-2.33	4.35	
5	41	2.4	1.72	-1.12	2.05	115.2	11.9	-7.74	14.2	
6	42	2.4	2.60	-1.17	2.85	115.2	18.0	-8.13	19.7	
7	43	2.4	2.88	-0.94	3.03	115.2	20.0	-6.51	21.0	
8	44	2.4	3.29	-0.15	3.30	115.2	22.8	-1.05	22.8	
9	48	2.45	3.22	0.27	3.23	117.6	22.3	1.90	22.4	
10	49	2.45	3.33	-0.11	3.34	117.6	23.1	-0.73	23.1	

Table 27. Test 20 with emergency gates open: Total discharge 518,000 ft^3/s with main dam spillway flow 206,000 ft^3/s and auxiliary spillway flow 312,000 ft^3/s.

		MODEL				PROTOTYPE				
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	NOTES
1	36	2.35	-0.41	0.07	0.41	112.8	-2.83	0.47	2.87	Recirculation zone
2	38	2.35	-0.18	-0.04	0.19	112.8	-1.25	-0.30	1.28	Recirculation zone
3	39	2.3	-0.14	0.44	0.46	110.4	-0.94	3.02	3.16	Recirculation zone
4	40	2.35	0.34	-0.23	0.41	112.8	2.37	-1.61	2.86	
5	41	2.45	2.17	-1.08	2.43	117.6	15.0	-7.48	16.8	
6	42	2.45	2.46	-0.84	2.60	117.6	17.1	-5.82	18.0	
7	43	2.4	2.78	-0.23	2.79	115.2	19.3	-1.59	19.3	
8	44	2.45	3.34	-0.34	3.36	117.6	23.1	-2.34	23.3	
9	48	2.45	3.40	-0.35	3.42	117.6	23.6	-2.42	23.7	
10	49	2.45	3.00	-0.46	3.03	117.6	20.8	-3.18	21.0	

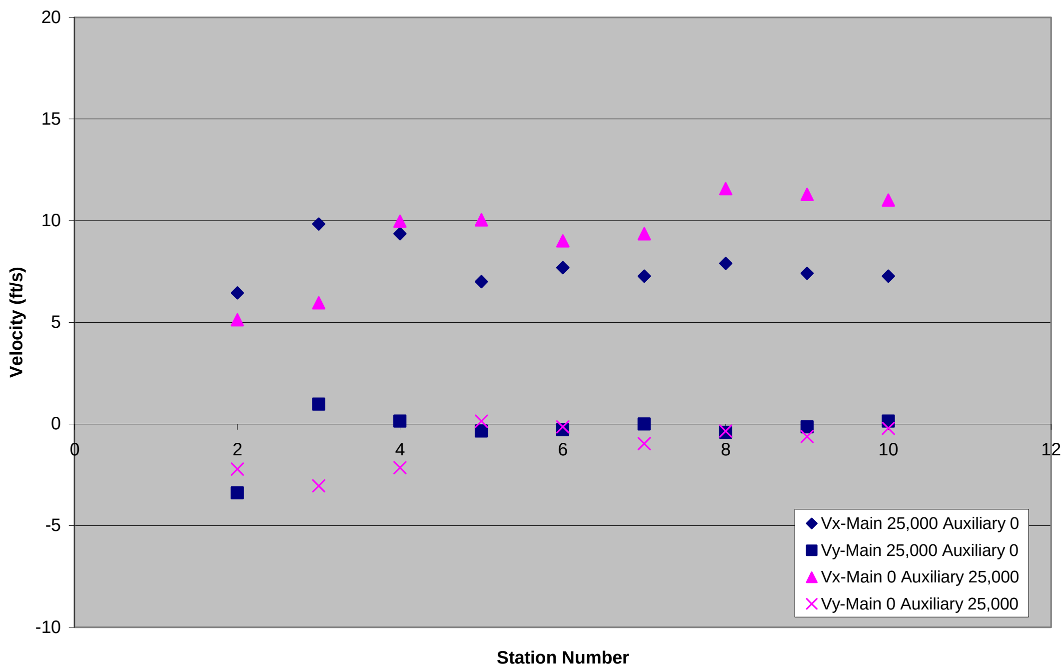

Figure 83. Prototype velocities in the American River channel for different operational scenarios with a total discharge of 25,000 ft^3/s.

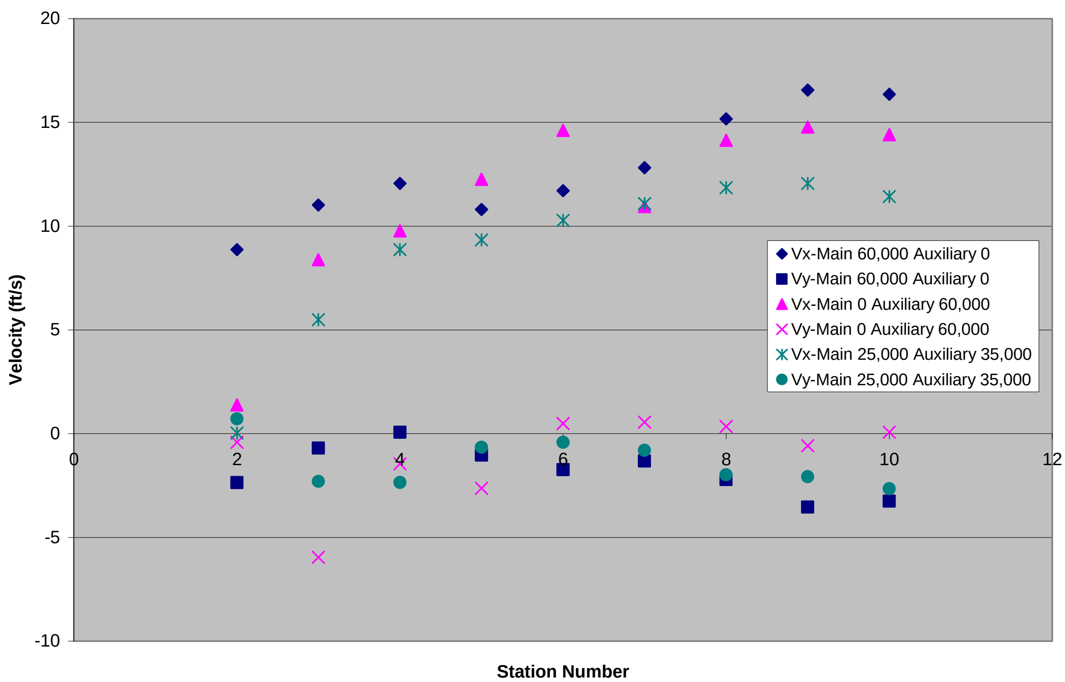

Figure 84. Prototype velocities in the American River channel for different operational scenarios with a total discharge of 60,000 ft^3/s.

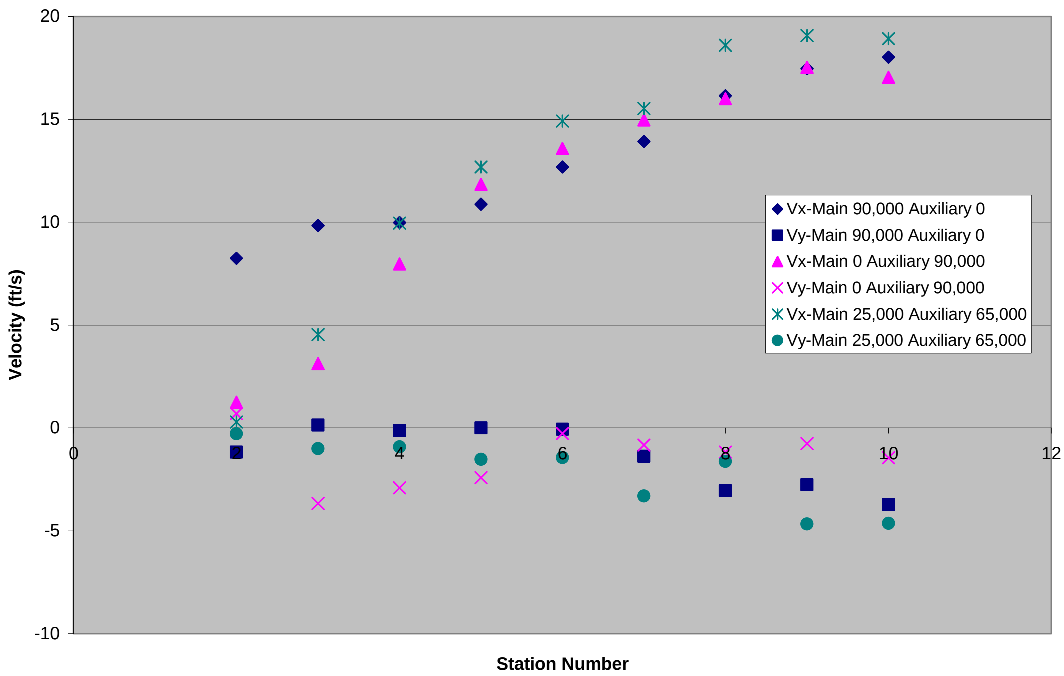

Figure 85. Prototype velocities in the American River channel for different operational scenarios with a total discharge of 90,000 ft^3/s.

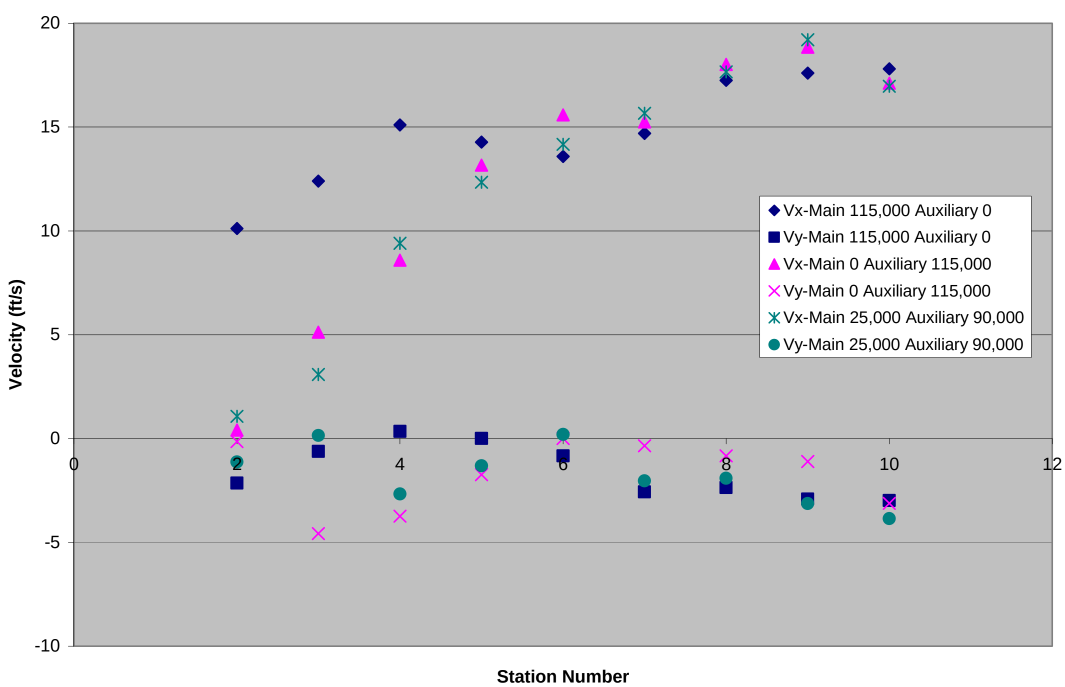

Figure 86. Prototype velocities in the American River channel for different operational scenarios with a total discharge of 115,000 ft^3/s.

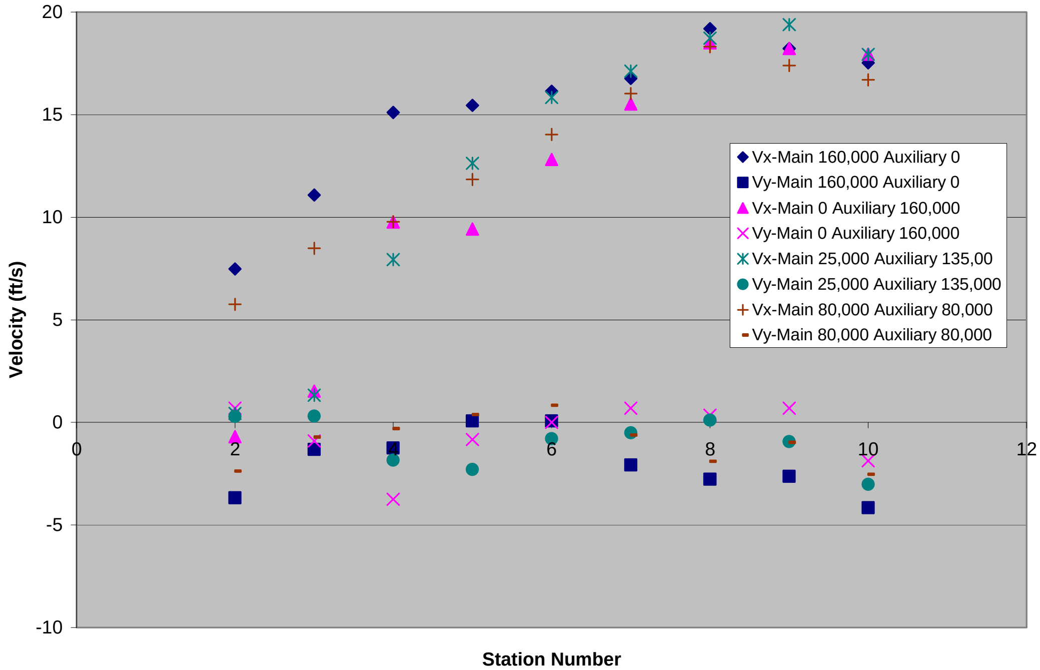

Figure 87. Prototype velocities in the American River channel for different operational scenarios with a total discharge of 160,000 ft^3/s.

Knob Topographic Feature Velocity Data

Flow velocities were also measured at three locations (stations 16, 17, 18) just upstream from the knob topographic feature remaining from the haul road excavation (figure 88). The two-dimensional FlowTracker acoustic velocity meter could not be used to collect data at the knob due to aeration produced from turbulence at the knob. A one-dimensional Swoffer propeller meter was used instead of an acoustic velocity meter. To ensure that the propeller meter was providing accurate readings, both the acoustic and propeller velocity meters were used to measure the downstream velocity vector in a non-aerated region of the river channel. On average, velocity readings with the two instruments were within 3.7% (table 28).

The velocity meter was oriented into the dominant flow direction to record the velocity magnitude. The velocity direction was measured with a protractor with reference to the knob face. The velocity vectors perpendicular to the knob face (Vx) and parallel to the knob face (Vy) were calculated from the velocity magnitude and direction. Three velocities were collected and averaged at each measurement location. Velocities were measured 4 inches upstream of the knob (16 ft prototype) due to the geometry of the instrument at 0.6 times the total water depth from the water surface.

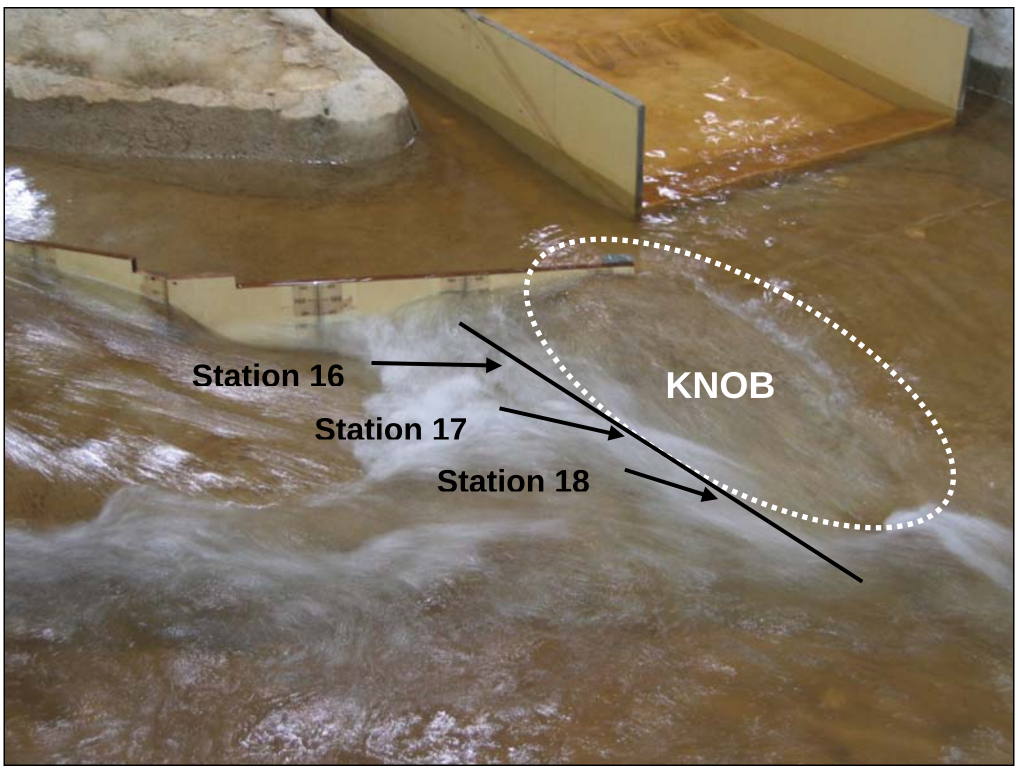

Figure 88. Main dam discharge of 60,000 ft^3/s impacts the knob remaining from the haul road excavation. Three velocity measurement locations are labeled on the photograph.

Direct impact on the knob occurred when discharges of up to 60,000 ft^3/s were released from the main dam with no release from the auxiliary spillway. Prototype velocities of 21.9 and 19.4 ft/s were measured perpendicular to the knob face near the juncture with the cofferdam wall for 25,000 and 60,000 ft^3/s, respectively. The highest velocities sweeping parallel to the knob were measured on the right side of the knob as the water drops into the river channel. These prototype velocity components were 28.1 and 21.5 ft/s for main dam releases of 25,000 and 60,000 ft^3/s, respectively.

Velocities at the knob decreased significantly during a discharge of 90,000 ft^3/s because the knob was submerged with a weak hydraulic jump at the knob. Velocities at the knob were also considerably lower when flows were split between the main dam structure and the auxiliary spillway structure. Model data is presented in table 28 and prototype scale conversions are presented in table 29.

Table 28. Three sets of velocities were measured at three locations just upstream from the knob topographic feature. Velocity magnitudes (Vmag) and angles are shown in the table along with a comparison between the propeller and acoustic velocity meters.

						MODEL SCALE						
	Auxiliary	Main Dam	CHANNEL CENTERLINE			LEFT SIDE		CENTER		RIGHT SIDE		
Test	Spillway Flow Rate (ft^3/s)	Spillway Flow Rate (ft^3/s)	Propellor Meter	Acoustic Meter	Percent Difference	Vmag (ft/s)	Angle (deg)	Vmag (ft/s)	Angle (ft/s)	Vmag (ft/s)	Angle (ft/s)	
1	0	25,000	1.42	1.36		3.61		2.59		4.05		
			1.49	n/a		3.67		2.66		4.18		
			1.39	n/a		3.67		2.51		4.13		
			1.43	**1.36**	**5.4**	**3.65**	**60**	**2.59**	**25**	**4.12**	**10**	**Average**
3	0	60,000	1.97	1.92		2.89		2.09		3.18		
			2	2.12		3.09		2.10		3.07		
			2.02	2.08		2.95		2.22		3.20		
			2.00	2.04	2.1	2.98	70	2.14	40	3.15	10	Average
5	35,000	25,000	2.31	2.25		0.57		1.07		1.72		
			2.26	2.21		0.58		1.14		1.66		
			2.29	2.23		0.52		1.15		1.58		
			2.29	2.23	2.5	0.56	60	1.12	60	1.65	30	Average
6	0	90,000	1.96	1.74		1.05		1.40		2.33		
			1.87	1.89		1.10		1.45		2.34		
			1.89	1.76		1.10		1.22		2.51		
			1.91	**1.80**	**6.1**	1.08	70	1.36	60	2.39	40	**Average**
8	65,000	25,000	2.2	2.15		0.72		0.94		0.81		
			2.22	2.24		0.74		0.90		0.86		
			2.29	2.16		0.71		0.97		0.90		
			2.24	2.18	2.4	0.72	70	0.94	65	0.86	45	Average

Table 29. Measured resultant velocities (Vres), calculated velocities perpendicular to the knob face (Vx), and calculated velocities parallel to the knob face (Vy) are displayed in prototype units.

			PROTOTYPE SCALE											
	Auxiliary Spillway Flow Rate (ft^3/s)	Main Dam Spillway Flow Rate (ft^3/s)	LEFT SIDE				CENTER				RIGHT SIDE			
Test			Vres (ft/s)	Angle (deg)	Vx (ft/s)	Vy (ft/s)	Vres (ft/s)	Angle (deg)	Vx (ft/s)	Vy (ft/s)	Vres (ft/s)	Angle (deg)	Vx (ft/s)	Vy (ft/s)
1	0	25,000	25.29	60	**21.90**	**12.64**	17.92	25	**7.57**	**16.24**	28.54	10	**4.96**	**28.11**
3	0	60,000	20.62	70	**19.38**	**7.05**	14.80	40	**9.52**	**11.34**	21.82	10	**3.79**	**21.49**
5	35,000	25,000	3.86	60	**3.34**	**1.93**	7.76	60	**6.72**	**3.88**	11.45	30	**5.73**	**9.92**
6	0	90,000	7.51	70	**7.05**	**2.57**	9.40	60	**8.14**	**4.70**	16.58	40	**10.66**	**12.70**
8	65,000	25,000	5.01	70	**4.71**	**1.71**	6.49	65	**5.88**	**2.74**	5.94	45	**4.20**	**4.20**

Bridge Pier Velocity Data

Overbank flow submerges the right bridge pier of the Folsom Lake Crossing Bridge at a discharge of about 250,000 ft^3/s. The objective of these tests was to measure velocities near the right bridge pier for discharges from 200,000 to 570,000 ft^3/s. These data will be analyzed by project geotechnical engineers to determine whether areas of erosion or instability can be expected near the bridge pier during high discharges.

Velocity data were collected with a SonTek 2D FlowTracker acoustic velocimeter at a sample rate of 1 Hz for 40 seconds (4.6 minutes prototype). Data were collected at four discharges with release flows coming from the auxiliary spillway and the main dam spillway in varying proportions as shown in table 30. The preferred operational scenario for the structures had not yet been determined at the time of testing. After discussions with the Folsom Design Team, it was decided that a release of 160,000 ft^3/s from the auxiliary structure and the remaining flow from the main dam would be a reasonable operational scenario for these tests. Photographs of the right bridge pier during the four flow conditions are shown in figures 89-92.

Table 30. Flow conditions during which right bridge pier velocity data was collected (BP = bridge pier).

Test No.	Auxiliary Spillway Flow Rate (ft^3/s)	Main Dam Spillway Flow Rate (ft^3/s)	Total Flow Rate (ft^3/s)
BP-1	160,000	40,000	200,000
BP-2	160,000	90,000	250,000
BP-3	160,000	140,000	300,000
BP-4	320,000	250,000	570,000

Figure 89. Right bridge pier at a channel discharge of 200,000 ft^3/s (test BP-1).

Figure 90. Right bridge pier at a channel discharge of 250,000 ft^3/s (test BP-2).

Figure 91. Right bridge pier at a channel discharge of 300,000 ft^3/s (test BP-3).

Figure 92. Right bridge pier at a channel discharge of 570,000 ft^3/s (test BP-4).

Four locations near the right bridge pier were chosen for measurement of flow velocities (depicted in figure 93):

1. **Location A: 24 ft prototype upstream from the right bridge pier.** The coordinate system was oriented with the pier so that Vx is positive downstream and Vy is positive away from the pier. At 200,000 and 250,000 ft^3/s, this location was not submerged. At 300,000 ft^3/s, the flow was deep enough for only 1 mid-depth reading. At 570,000 ft^3/s, near-surface and mid-depth velocities were collected.
2. **Location B: 48 ft prototype upstream and 48 ft prototype to the left of the right bridge pier.** The coordinate system is oriented with the pier so that Vx is positive downstream and Vy is positive away from the pier. At 200,000 and 250,000 ft^3/s, the flow was deep enough for only 1 mid-depth reading. At 300,000 and 570,000 ft^3/s, near-surface and mid-depth velocities were collected.
3. **Location C: Near the toe of the right bank.** Due to the 4-inch offset from the velocimeter probe to the sampling volume, data were collected at 16 ft prototype from the toe of the right bank. The coordinate system was oriented with the channel so that Vx is positive downstream and Vy is positive away from the bank. Velocity data were collected at 0.2, 0.6, and 0.8 times the flow depth.
4. **Location D: At the channel centerline.** The coordinate system was oriented with the channel so that Vx is positive downstream and Vy is positive away from the bank. Velocity data were collected at 0.2, 0.6, and 0.8 times the flow depth.

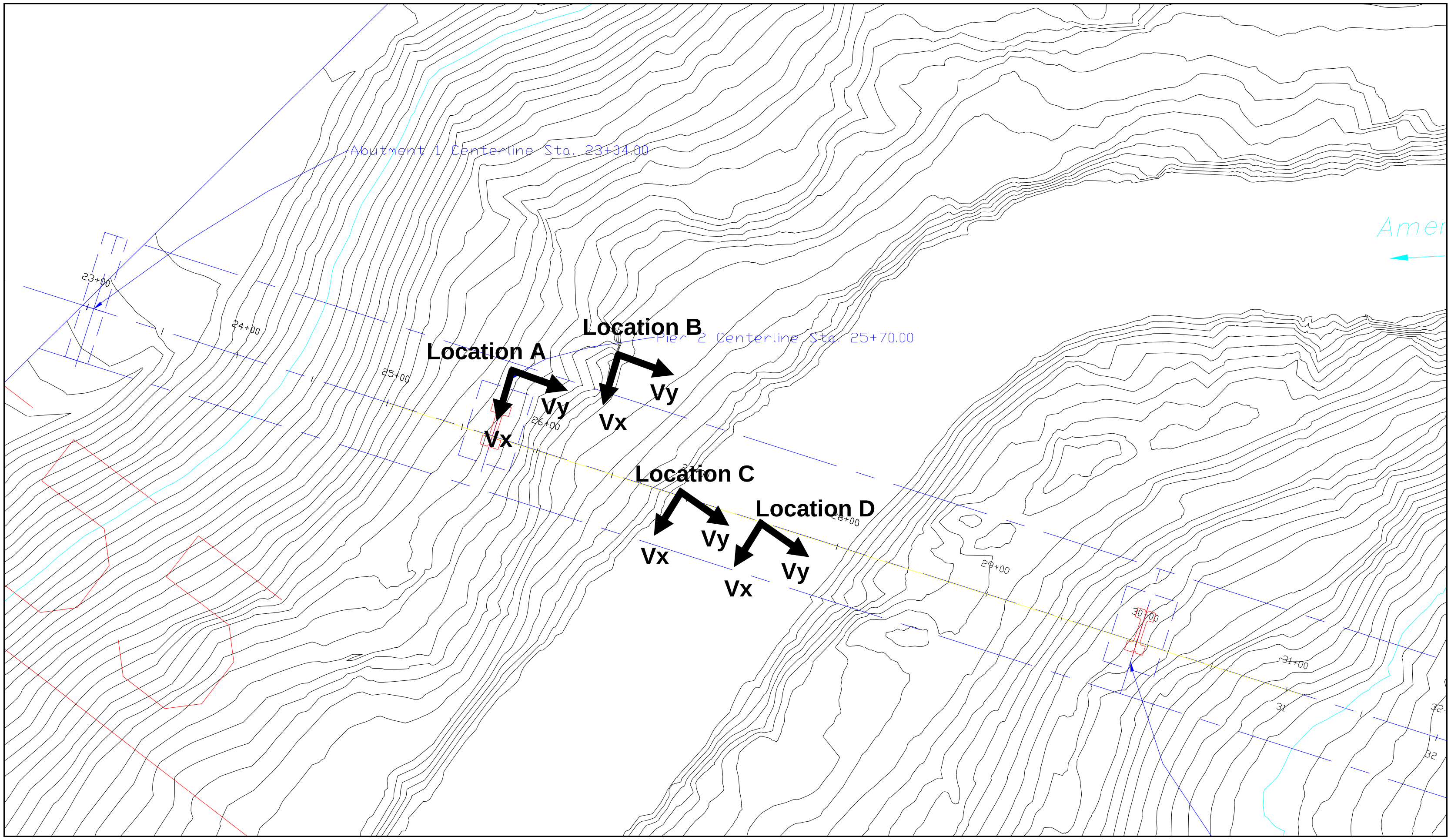

Figure 93. Plan view of model velocity measurement locations near the right bridge pier. Location A is 24 ft prototype upstream from the right bridge pier, Location B is 48 ft prototype upstream and 48 ft prototype to the left of the right bridge pier, Location C is near the toe of the right bank, and Location D is at the channel centerline.

Velocities approaching the right bridge pier were greater at location B (48 ft upstream from the bridge pier and 48 ft to the left of the bridge pier) than location A (24 ft upstream from the bridge pier). At location B for a discharge of 200,000 ft^3/s, Vx was 9.2 ft/s and Vy was 2.8 ft/s. At 250,000 ft^3/s, Vx was 8.9 ft/s and Vy was 4.0 ft/s. At 300,000 ft^3/s, the mid-depth Vx and Vy velocities were 9.8 ft/s and 4.2 ft/s, respectively. At 570,000 ft^3/s, the mid-depth Vx velocity was 15.6 ft/s and the mid-depth Vy velocity was 6.2 ft/s.

Near the toe of the right bank (location C) at 0.6 times the flow depth, the streamwise downstream velocity ranged from 19.0 ft/s to 21.1 ft/s. The transverse velocity toward the right bank ranged from 1.5 to 4.7 ft/s. There was little change in streamwise and transverse velocities with increased discharge. Velocities at the centerline of the channel (location D) were typically slightly higher than the velocities near the toe of the right bank (location C). Tables 31-34 show all velocity data collected during these tests.

Table 31. Velocity and depth data are shown for a total discharge of 200,000 ft^3/s (test BP-1). The auxiliary spillway released 160,000 ft^3/s and the main dam spillway released 40,000 ft^3/s. The coordinate systems for locations A and B were oriented with the pier so that positive Vx was downstream and positive Vy was away from the pier. The coordinate systems for locations C and D were oriented with the channel so that positive Vx was downstream and positive Vy was away from the bank.

		MODEL DATA				PROTOTYPE DATA			
	Measurement Location	**Flow Depth (ft)**	**Distance from Bottom (ft)**	**Vx (ft/s)**	**Vy (ft/s)**	**Flow Depth (ft)**	**Distance from Bottom (ft)**	**Vx (ft/s)**	**Vy (ft/s)**
Location A: 24 ft upstream from pier	Near Surface	Not submerged	--	--	--	--	--	--	--
	0.5 depth	Not submerged	--	--	--	--	--	--	--
Location B: 48 ft upstream and 48 ft to the left of pier	Near Surface	Too shallow	--	--	--	--	--	--	--
	0.5 depth	0.20	0.10	1.33	-0.41	9.6	4.8	9.2	-2.8
Location C: Near toe of right bank	0.2 depth	1.70	1.36	2.58	-0.91	81.6	65.3	17.9	-6.3
	0.6 depth	1.70	0.68	2.74	-0.51	81.6	32.6	19.0	-3.5
	0.8 depth	1.70	0.34	2.78	-0.23	81.6	16.3	19.3	-1.6
Location D: Centerline of channel	0.2 depth	1.70	1.36	2.82	-0.97	81.6	65.3	19.5	-6.7
	0.6 depth	1.70	0.68	2.71	-0.38	81.6	32.6	18.8	-2.6
	0.8 depth	1.70	0.34	2.76	-0.03	81.6	16.3	19.1	-0.2

Table 32. Velocity and depth data shown for a total discharge of 250,000 ft^3/s (test BP-2). The auxiliary spillway released 160,000 ft^3/s and the main dam spillway released 90,000 ft^3/s.

		MODEL DATA				PROTOTYPE DATA			
	Measurement Location	**Flow Depth (ft)**	**Distance from Bottom (ft)**	**Vx (ft/s)**	**Vy (ft/s)**	**Flow Depth (ft)**	**Distance from Bottom (ft)**	**Vx (ft/s)**	**Vy (ft/s)**
Location A: 24 ft upstream from pier	Near Surface	Not submerged	--	--	--	--	--	--	--
	0.5 depth	Not submerged	--	--	--	--	--	--	--
Location B: 48 ft upstream and 48 ft to left of pier	Near Surface	Too shallow	--	--	--	--	--	--	--
	0.5 depth	0.35	0.18	1.29	-0.58	16.8	8.4	8.9	-4.0
Location C: Near toe of right bank	0.2 depth	1.80	1.44	2.84	-0.74	86.4	69.1	19.7	-5.1
	0.6 depth	1.80	0.72	2.86	-0.22	86.4	34.6	19.8	-1.5
	0.8 depth	1.80	0.36	2.68	0.19	86.4	17.3	18.6	1.3
Location D: Centerline of channel	0.2 depth	1.80	1.44	2.94	-0.73	86.4	69.1	20.4	-5.1
	0.6 depth	1.80	0.72	2.77	-0.21	86.4	34.6	19.2	-1.5
	0.8 depth	1.80	0.36	2.82	0.05	86.4	17.3	19.5	0.3

Table 33. Velocity and depth data shown for a total discharge of 300,000 ft^3/s (test BP-3). The auxiliary spillway released 160,000 ft^3/s and the main dam spillway released 140,000 ft^3/s.

		MODEL DATA				PROTOTYPE DATA			
	Measurement Location	**Flow Depth (ft)**	**Distance from Bottom (ft)**	**Vx (ft/s)**	**Vy (ft/s)**	**Flow Depth (ft)**	**Distance from Bottom (ft)**	**Vx (ft/s)**	**Vy (ft/s)**
Location A: 24 ft upstream from pier	Near Surface	Too shallow	--	--	--	--	--	--	--
	0.5 depth	0.15	0.08	1.36	-0.28	7.2	3.6	9.4	-1.9
Location B: 48 ft upstream and 48 ft to left of pier	Near Surface	0.50	0.40	1.33	-0.39	24.0	19.2	9.2	-2.7
	0.5 depth	0.50	0.25	1.42	-0.60	24.0	12.0	9.8	-4.2
Location C: Near toe of right bank	0.2 depth	2.00	1.60	2.87	-0.92	96.0	76.8	19.9	-6.4
	0.6 depth	2.00	0.80	2.86	-0.68	96.0	38.4	19.8	-4.7
	0.8 depth	2.00	0.40	2.82	-0.23	96.0	19.2	19.5	-1.6
Location D: Centerline of channel	0.2 depth	2.00	1.60	3.04	-1.21	96.0	76.8	21.1	-8.4
	0.6 depth	2.00	0.80	2.89	-0.60	96.0	38.4	20.0	-4.2
	0.8 depth	2.00	0.40	2.91	-0.11	96.0	19.2	20.2	-0.8

Table 34. Velocity and depth data shown for a total discharge of 570,000 ft^3/s (test BP-4). The auxiliary spillway released 320,000 ft^3/s and the main dam spillway released 250,000 ft^3/s.

		MODEL DATA				PROTOTYPE DATA			
	Measurement Location	**Flow Depth (ft)**	**Distance from Bottom (ft)**	**Vx (ft/s)**	**Vy (ft/s)**	**Flow Depth (ft)**	**Distance from Bottom (ft)**	**Vx (ft/s)**	**Vy (ft/s)**
Location A: 24 ft upstream from pier	Near Surface	0.80	0.70	1.68	-0.38	38.4	33.6	11.6	-2.6
	0.5 depth	0.80	0.40	2.15	-0.57	38.4	19.2	14.9	-3.9
Location B: 48 ft upstream and 48 ft to left of pier	Near Surface	1.00	0.90	2.29	-0.98	48.0	43.2	15.9	-6.8
	0.5 depth	1.00	0.50	2.25	-0.90	48.0	24.0	15.6	-6.2
Location C: Near toe of right bank	0.2 depth	2.50	2.00	3.15	-1.34	120.0	96.0	21.8	-9.3
	0.6 depth	2.50	1.00	3.05	-0.41	120.0	48.0	21.1	-2.8
	0.8 depth	2.50	0.50	2.74	0.72	120.0	24.0	19.0	5.0
Location D: Centerline of channel	0.2 depth	2.50	2.00	3.60	-0.35	120.0	96.0	24.9	-2.4
	0.6 depth	2.50	1.00	2.74	-1.08	120.0	48.0	19.0	-7.5
	0.8 depth	2.50	0.50	2.78	-0.04	120.0	24.0	19.3	-0.3

Water Surface Differential Measurements on Right Auxiliary Stilling Basin Wall

Differential water level data were collected along the right wall of the auxiliary stilling basin to further examine the possible effects of flow impingement on the un-backfilled wall with the haul road constructed (figures 94-96). At the time of testing, the cofferdam wall along the access road had not been designed and the decision to make the cofferdam a permanent structure had not yet been made. When installed, the cofferdam deflects flows from the main dam of up to 50,000 ft^3/s away from the right auxiliary stilling basin wall. For flows above 50,000 ft^3/s from the main dam, water will pass over the cofferdam wall toward the right stilling basin wall. For these flows, mean water levels at the stilling basin wall would be similar, but fluctuations in the water surface may be affected by the cofferdam wall. Subsequent tests to evaluate pressure fluctuations on the right stilling basin wall were specifically requested to be done without the cofferdam in place.

Observed water levels partially represent the total loading on the wall due to flow conditions. As flow impacts the wall, velocity is at least partially converted to pressure which produces a higher water surface elevation in the proximity of the impingement zone. The velocity is not entirely converted to a vertical rise in water surface as the pressure is dissipated in all directions away from the impact point; however the resulting general flow patterns reflect the underlying conditions that are occurring.

Wave probe measurements were collected on both sides of the right auxiliary stilling basin wall. Figures 97-98 show the probe located within the model in a typical measuring position with wave action. Table 35 lists the six flow conditions that were set up in the model for this round of tests.

Figure 94. Overview of the haul road.

Figure 95. Looking downstream toward the auxiliary spillway stilling basin.

Figure 96. Backfill removed behind the stilling basin wall.

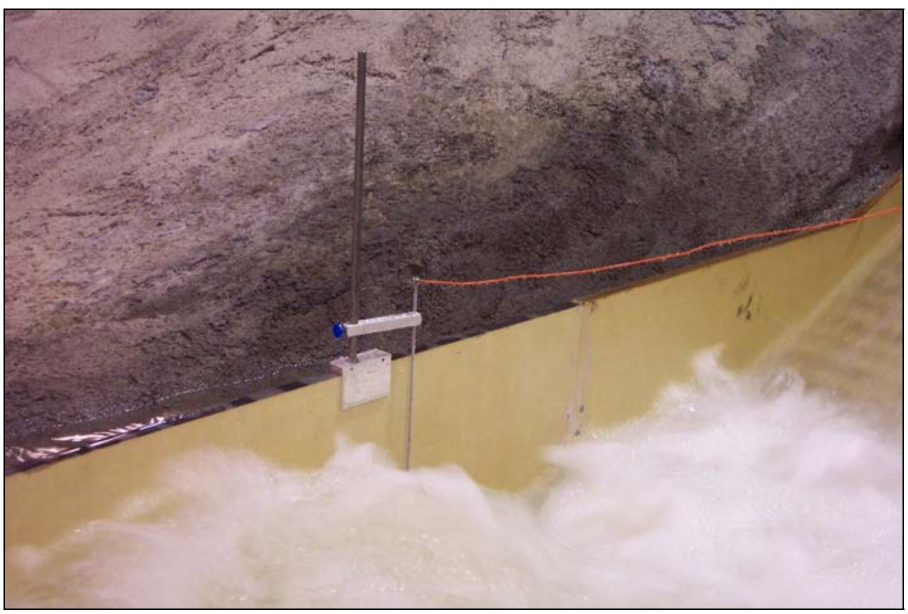

Figure 97. Wave probe measuring water depth during passage of a wave crest.

Figure 98. Wave probe measuring water depth in a wave trough.

Table 35. Six test conditions for which water surface differentials were measured across the right auxiliary stilling basin wall (WSD = water surface differential).

Test No.	Auxiliary Spillway Flow Rate (ft^3/s)	Main Dam Spillway Flow Rate (ft^3/s)	Total Flow Rate (ft^3/s)
WSD-1	22,625	0	22,625
WSD-2	115,000	0	115,000
WSD-3	90,000	25,000	115,000
WSD-4	160,000	0	160,000
WSD-5	0	160,000	160,000
WSD-6	135,000	25,000	160,000

Tailwater elevations were set near the end of the physical model, about 300 ft downstream from the new bridge location, according to HEC-RAS data provided by COE. In all 6 test cases, the tailwater is lower than the top of the auxiliary basin wall (194.05 ft), producing the potential for the largest amount of differential loading. In the first flow condition with a discharge of 22,625 ft^3/s from the auxiliary spillway, the differential loading across the right wall was minimal (figures 99-100) so data were not recorded. Photographs of flow conditions 3 and 5 are shown in figures 101 and 102, respectively.

Figure 99. Water surface differential across the right stilling basin wall with 22,625 ft^3/s from the auxiliary spillway and no flow from the main dam (test WSD-1).

Figure 100. Water passing over the right side of the end sill passes to the right of the knob in the topography produced by haul road excavation (test WSD-1).

Figure 101. Total discharge of 115,000 ft^3/s with 90,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam spillway (test WSD-3).

Figure 102. Total discharge of 160,000 ft^3/s with all flow released from the main dam spillway (test WSD-5).

The data are presented in graphical form using 6 locations along each side of the right auxiliary stilling basin wall along the horizontal x-axis. The “0” position is even with the vertical face of the last step entering the basin. The graphs show the sample mean with error bars representing $\pm 2\sigma$ (2 times the sample standard deviation). This error band represents the 95% confidence interval for a Gaussian distribution. The maximum differential at the same location along the right basin wall was developed using the -2σ of the lower mean level and the $+2\sigma$ of the higher mean level at each position.

Figures 103-107 show results for these test flow conditions. As expected, the highest differential loadings occur near the beginning of the stilling basin at a condition where the flow is still supercritical in the basin yielding a minimum water level inside the basin and full tailwater on the outside of the basin. When the water level inside the basin was too shallow to measure with the wave probe, the water depth was determined with a scale, measuring from the top of the stilling basin wall down to the water surface.

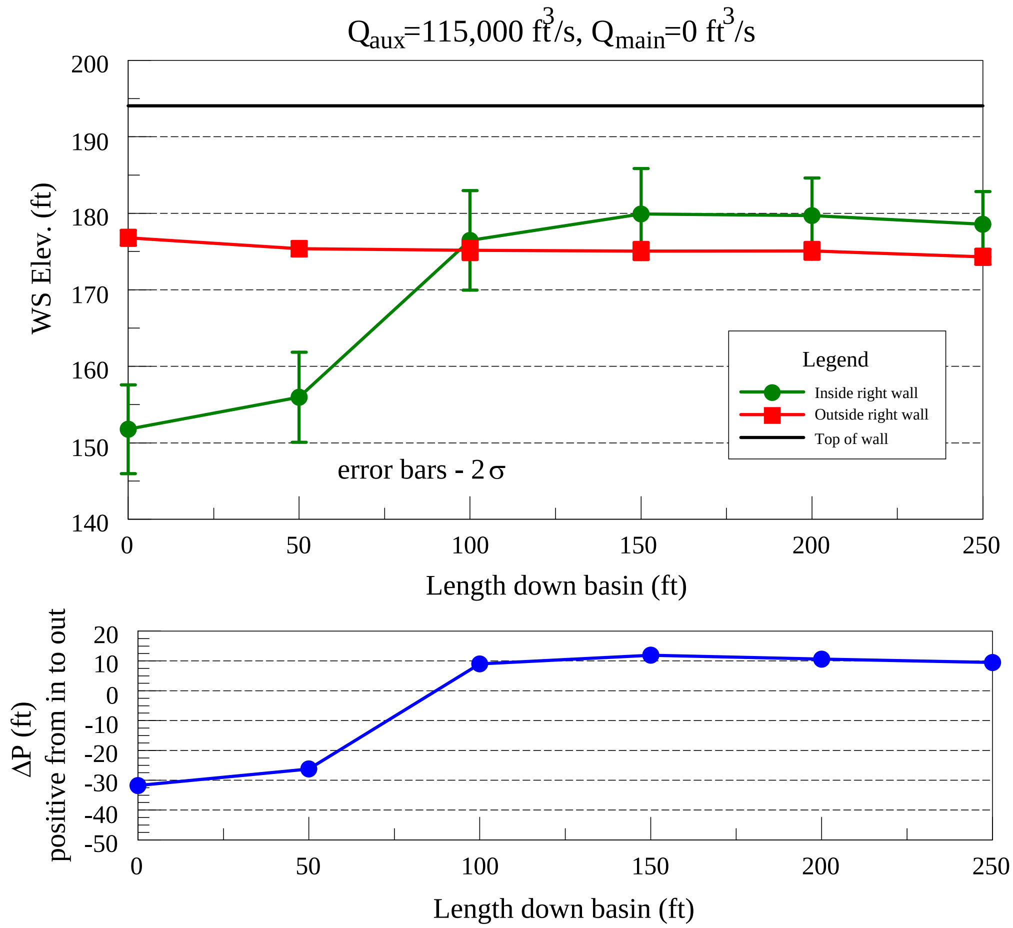

Figure 103. Water surface profiles along right stilling basin wall. Total discharge is 115,000 ft^3/s with an auxiliary flow rate (Q_{aux}) of 115,000 ft^3/s and a main dam flow rate (Q_{main}) of 0 ft^3/s (test WSD-2).

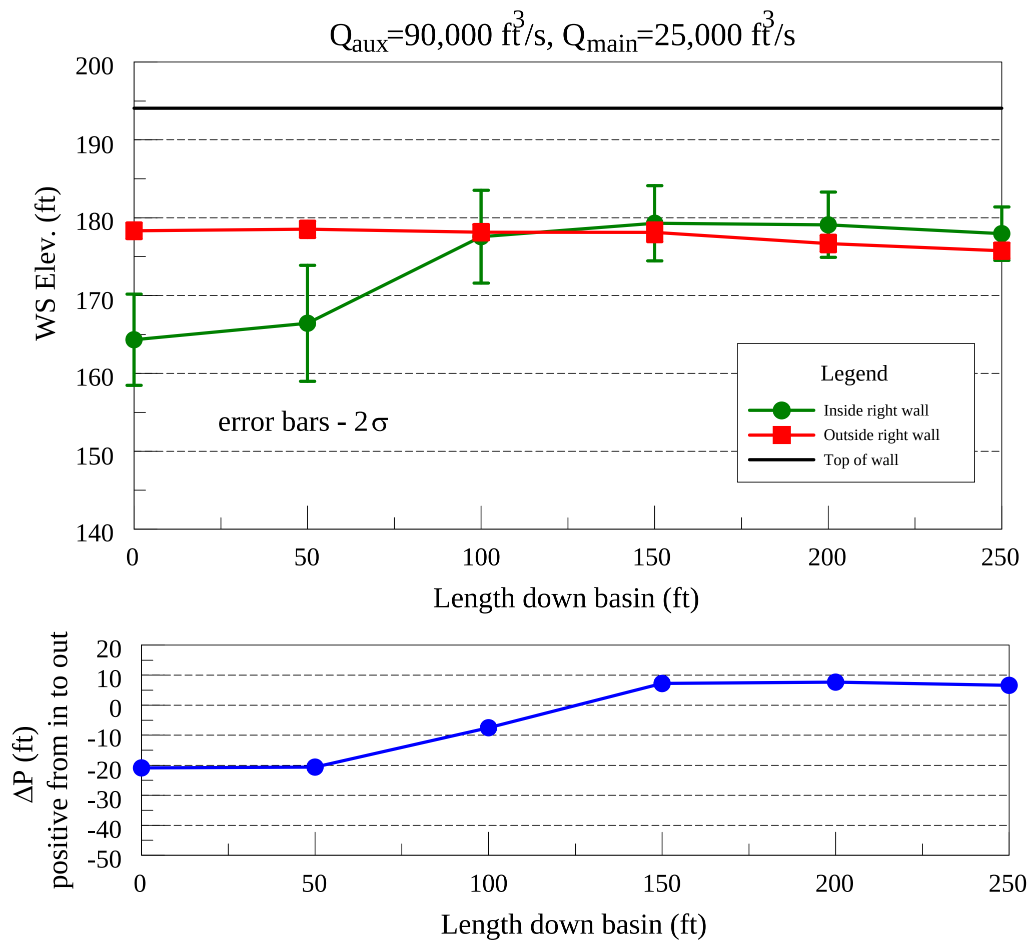

Figure 104. Water surface profiles along right stilling basin wall. Total discharge is 115,000 ft^3/s with an auxiliary flow rate (Q_{aux}) of 90,000 ft^3/s and a main dam flow rate (Q_{main}) of 25,000 ft^3/s (test WSD-3).

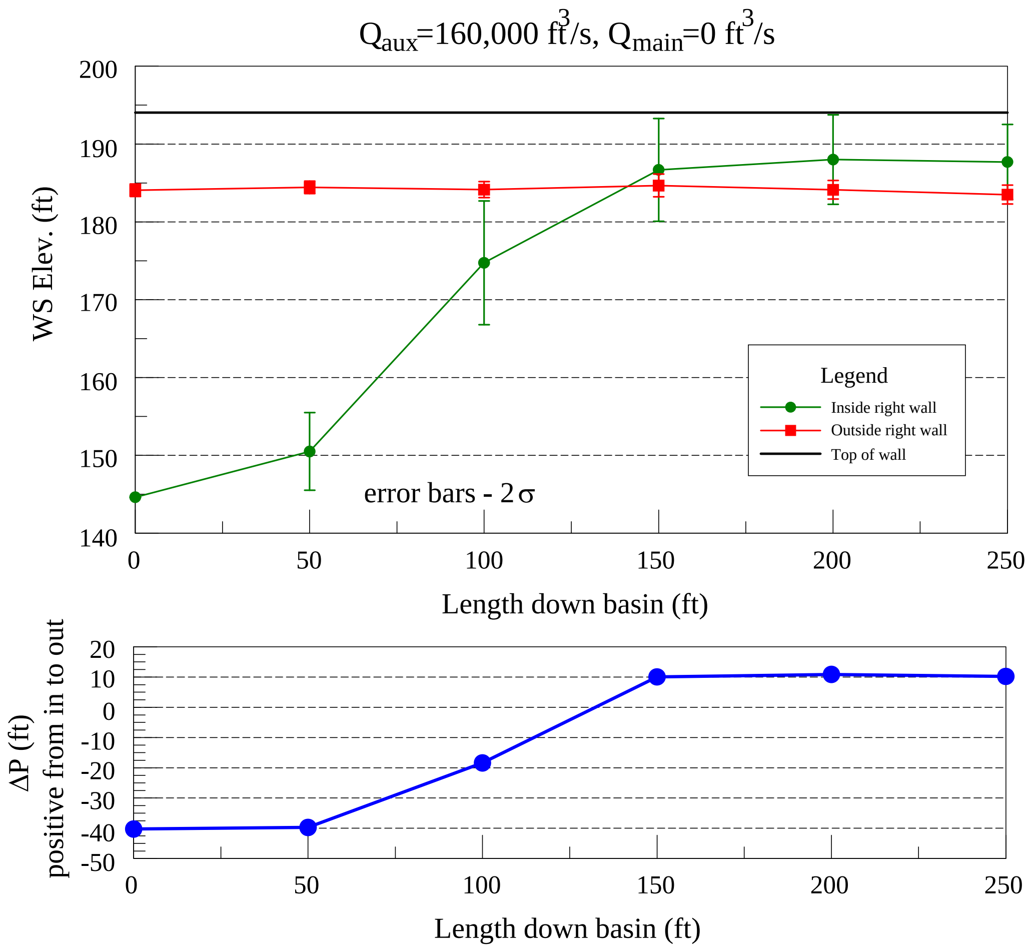

Figure 105. Water surface profiles along right stilling basin wall. Total discharge is 160,000 ft^3/s with an auxiliary flow rate (Q_{aux}) of 160,000 ft^3/s and a main dam flow rate (Q_{main}) of 0 ft^3/s (test WSD-4).

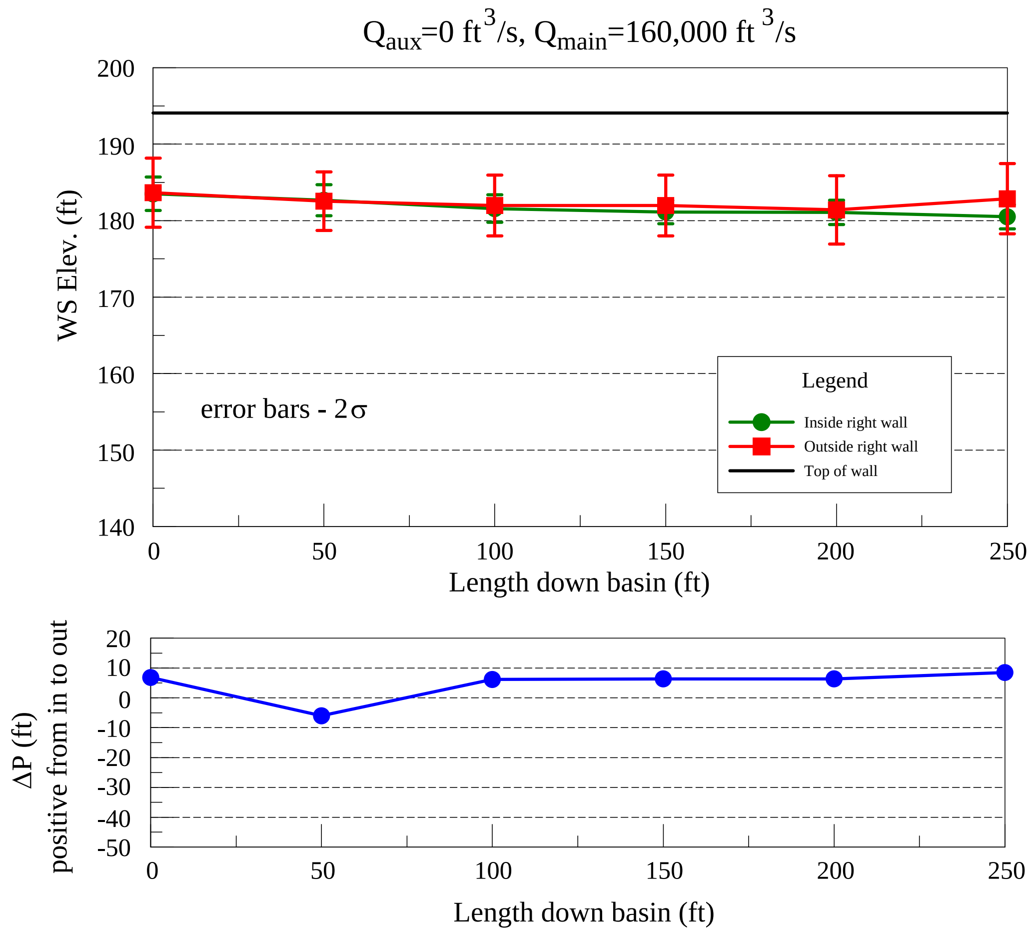

Figure 106. Water surface profiles along right stilling basin wall. Total discharge is 160,000 ft^3/s with an auxiliary flow rate (Q_{aux}) of 0 ft^3/s and a main dam flow rate (Q_{main}) of 160,000 ft^3/s (test WSD-5).

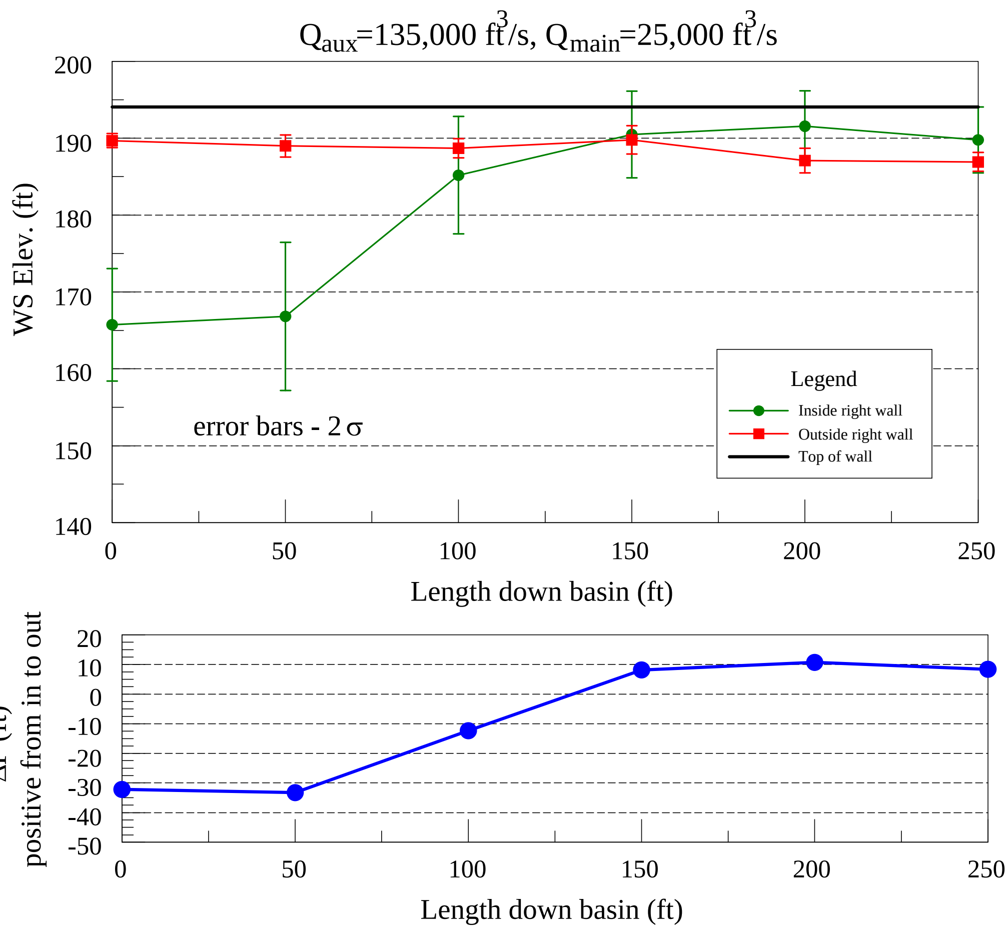

Figure 107. Water surface profiles along right stilling basin wall. Total discharge is 160,000 ft^3/s with an auxiliary flow rate (Q_{aux}) of 135,000 ft^3/s and a main dam flow rate (Q_{main}) of 25,000 ft^3/s (test WSD-6).

At flow rates too low to submerge the auxiliary stilling basin wall, the differential loading was principally associated with the different water levels on each side of the wall, and the maximum observed water surface differential was about 40 ft. Once the auxiliary spillway basin wall was submerged, the hydrostatic load equalized and the differential loading became primarily a function of the momentum imparted by main dam flows impacting the downstream end of the right auxiliary stilling basin wall. Using the concept of stagnation pressure, the pressure can be predicted using $P_d = \rho V^2 / 2$. To generate a differential load comparable to the observed 40 ft hydrostatic loading, a perpendicular velocity of about 50 ft/s would be needed.

To determine if such a loading could be produced, 250,000 ft^3/s was released from the main dam through the 5 service spillway gates with no flow from the auxiliary spillway. With water flowing well over the top of the right stilling basin wall, differential water level measurements were not meaningful. Water surface wave measurements on the outside of the right wall are shown in figure 108.

A velocity profile was collected 200 ft prototype downstream from the beginning of the basin and approximately 20 ft prototype away from the right stilling basin wall towards the main dam. A SonTek Flow Tracker was used to measure the approach velocity component perpendicular to the wall. Velocity data were averaged over a period of 60 seconds in the model (7 minutes prototype). The probe was oriented so that the x-component of velocity was perpendicular and towards the wall. A velocity profile was also collected during the same flow condition with all 8 spillway gates open.

Figure 109 shows the velocity profiles at this location approaching the wall for the 5-gate and 8-gate flow releases from the main dam at 250,000 ft^3/s. Maximum velocities are about 14 ft/s in the 5-gate case and 25 ft/s in the 8-gate case. If these maximum velocities were decelerated at the wall to stagnation, the pressure increase would be about 3 ft for the 5-gate case and 9.75 ft for the 8-gate case.

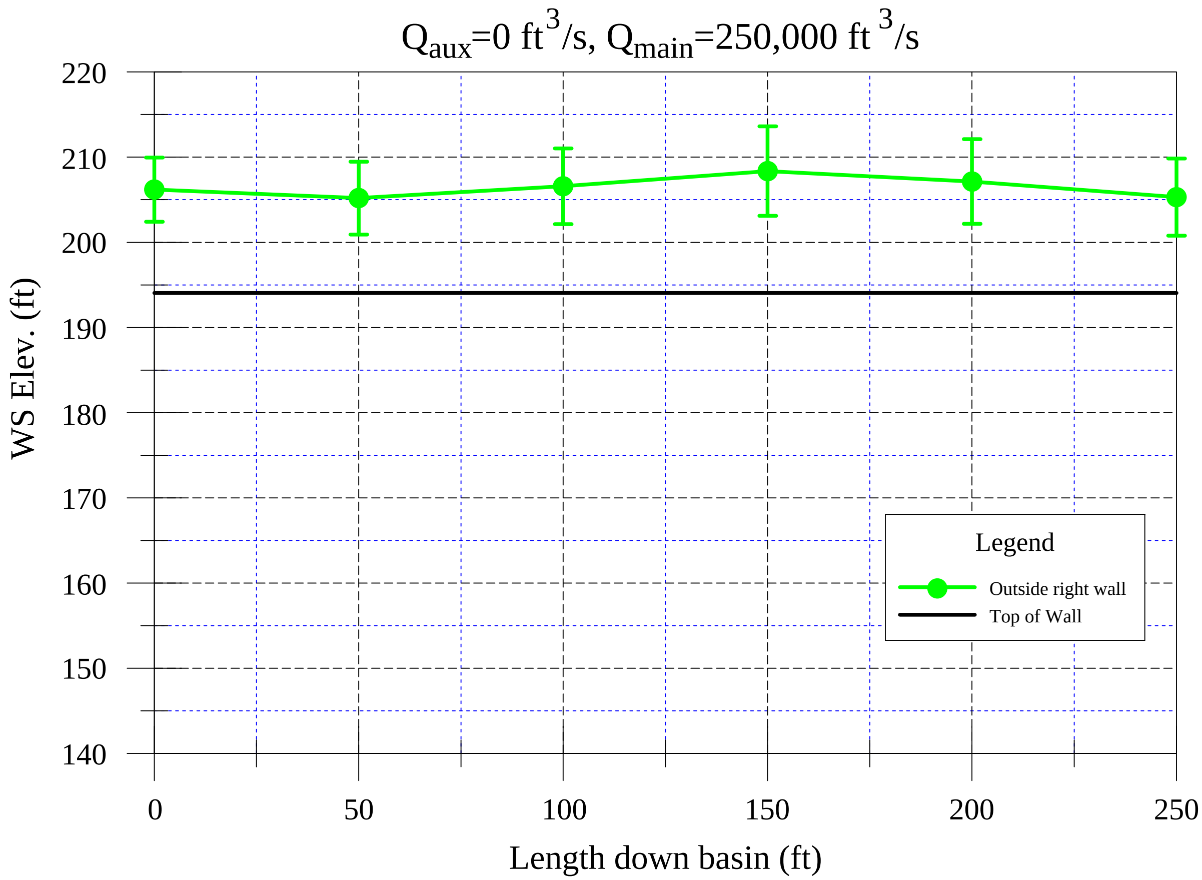

Figure 108. Water surface wave measurements outside of the right auxiliary stilling basin wall under a total discharge of 250,000 ft^3/s with an auxiliary flow rate (Q_{aux}) of 0 ft^3/s and a main dam flow rate (Q_{main}) of 250,000 ft^3/s.

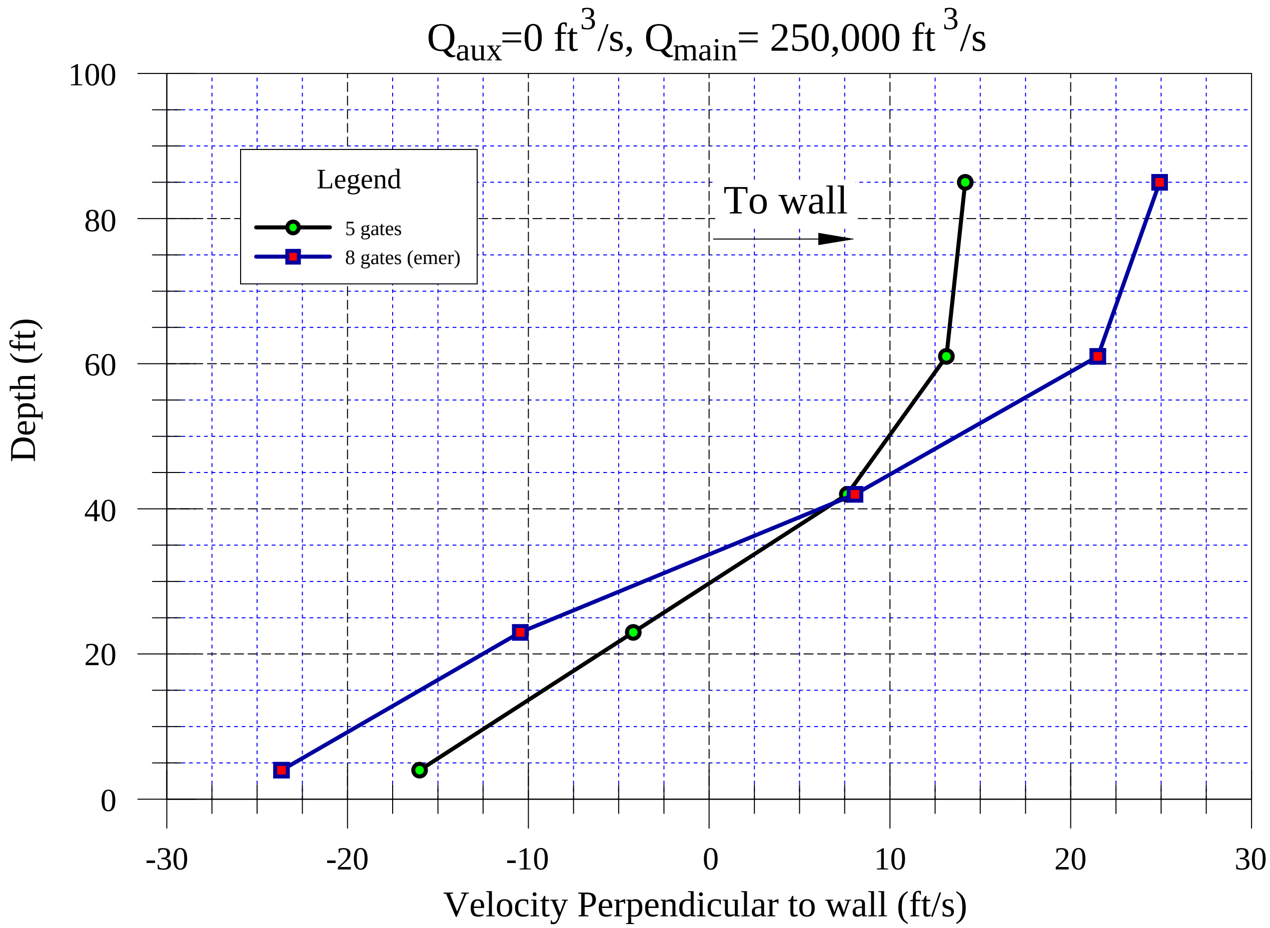

Figure 109. Velocity profile of flow component approaching the right auxiliary stilling basin wall. The location is 200 ft downstream from beginning of the basin and about 20 ft away (upstream) from the wall toward the main dam.

Total Pressure Differential Measurements on Right Auxiliary Stilling Basin Wall

In addition to the differential water surface data collected using capacitance-type wave probes at simulated discharges of 160,000 ft^3/s and lower, total pressure data were collected at five locations along the right auxiliary stilling basin wall at several elevations. Using simultaneous sampling of these pressures, true pressure differentials were calculated.

Table 36 shows the eight flow conditions that were tested in the model. A couple of the requested tests were not run as they provide no additional data regarding the final determination of maximum pressure differentials.

Table 36. Eight flow conditions for which total pressure differentials were measured across the right auxiliary stilling basin wall (TPD = total pressure differential).

Test No.	Auxiliary Spillway Flow Rate (ft^3/s)	Main Dam Spillway Flow Rate (ft^3/s)	Total Flow Rate (ft^3/s)
TPD-1	0	115,000	115,000
TPD-2	0	160,000	160,000
TPD-3	90,000	0	90,000
TPD-4	115,000	0	115,000
TPD-5	160,000	0	160,000
TPD-6	135,000	25,000	160,000
TPD-7	312,000	323,750	635,750
TPD-8	312,000	518,000	830,000

Data were collected at five lateral stations along the stilling basin wall: 39+26.5, 39+85, 40+30, 40+75, and 41+22. These locations were used for all except the two largest flow conditions (tests TPD-8 and TPD-9). Due to difficulties in moving the fitting along the wall during the tests for tests TPD-8 and TPD-9, only Sta. 40+75 ft was used for data collection. This position yielded the maximum differentials up to the point that movement of the fitting was not possible. Topographic conditions within the model were similar to when the wave probe water surface differential study was completed as the haul road cut was in place and the cofferdam wall was removed for these tests. Additionally, the newly-designed supercavitating baffle blocks were in place (as discussed below in the section “New Baffle Block Installation”.)

Each test run resulted in a time series of voltages that were proportional to pressure. The voltages were converted to pressures and then to water surface elevations for tabular presentation. Each time series was treated statistically and a mean and standard deviation were computed. Results shown in tables 37- 44 are in prototype feet with the pressure differential taken from inside to out, i.e. lower water surface in the basin would represent a negative difference.

Table 37. Total pressure differential across the right auxiliary stilling basin wall. Total flow rate is 115,000 ft^3/s with an auxiliary flow rate 0 ft^3/s and main dam flow rate 115,000 ft^3/s (test TPD-1).

Lateral Position	Transducer Elevation (ft)	Mean Difference (ft across wall)	Std. Deviation (ft)
Sta. 39+26.5	143.05	1.29	3.81
	158.05	2.21	3.85
	173.05	4.38	3.82
	188.05	-0-	-0-
Sta. 39+85	143.05	1.88	2.59
	158.05	2.22	2.49
	173.05	4.60	3.17
	188.05	-0-	-0-
Sta. 40+30	143.05	1.96	2.74
	158.05	1.99	2.62
	173.05	4.25	3.21
	188.05	-0-	-0-
Sta. 40+75	143.05	0.04	2.90
	158.05	0.50	2.22
	173.05	2.94	3.08
	188.05	-0-	-0-
Sta. 41+22	143.05	-0.38	2.69
	158.05	0.69	2.33
	173.05	2.40	2.72
	188.05	-0-	-0-

Table 38. Total pressure differential across the right auxiliary stilling basin wall. Total flow rate is 160,000 ft^3/s with an auxiliary flow rate 0 ft^3/s and main dam flow rate 160,000 ft^3/s (test TPD-2).

Lateral Position	Transducer Elevation (ft)	Mean Difference (ft across wall)	Std. Deviation (ft)
Sta. 39+26.5	143.05	-8.07	2.04
	158.05	-8.54	1.75
	173.05	-9.13	2.59
	188.05	-0-	-0-
Sta. 39+85	143.05	-7.46	2.37
	158.05	-4.21	2.15
	173.05	-5.72	2.66
	188.05	-0-	-0-
Sta. 40+30	143.05	1.96	2.74
	158.05	3.21	2.03
	173.05	1.95	2.66
	188.05	-0-	-0-
Sta. 40+75	143.05	4.93	3.99
	158.05	1.56	2.02
	173.05	2.59	2.31
	188.05	-0-	-0-
Sta. 41+22	143.05	2.70	2.42
	158.05	2.78	2.63
	173.05	1.14	2.59
	188.05	-0-	-0-

Table 39. Total pressure differential across the right auxiliary stilling basin wall. Total flow rate is 90,000 ft^3/s with an auxiliary flow rate 90,000 ft^3/s and main dam flow rate 0 ft^3/s (test TPD-3).

Lateral Position	Transducer Elevation (ft)	Mean Difference (ft across wall)	Std. Deviation (ft)
Sta. 39+26.5	143.05	-26.55	8.39
	158.05	-21.13	3.24
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 39+85	143.05	-9.85	6.27
	158.05	-6.68	4.28
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 40+30	143.05	8.09	1.92
	158.05	9.84	2.09
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 40+75	143.05	4.72	1.70
	158.05	1.13	1.78
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 41+22	143.05	5.57	1.45
	158.05	2.48	1.63
	173.05	-0-	-0-
	188.05	-0-	-0-

Table 40. Total pressure differential across the right auxiliary stilling basin wall. Total flow rate is 115,000 ft^3/s with an auxiliary flow rate 115,000 ft^3/s and main dam flow rate 0 ft^3/s (test TPD-4).

Lateral Position	Transducer Elevation (ft)	Mean Difference (ft across wall)	Std. Deviation (ft)
Sta. 39+26.5	143.05	-34.97	10.83
	158.05	-35.10	2.34
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 39+85	143.05	5.63	8.58
	158.05	4.68	6.88
	173.05	0.57	8.58
	188.05	-0-	-0-
Sta. 40+30	143.05	11.42	2.76
	158.05	14.88	2.11
	173.05	-2.24	2.76
	188.05	-0-	-0-
Sta. 40+75	143.05	6.80	1.88
	158.05	11.24	2.35
	173.05	1.87	3.26
	188.05	-0-	-0-
Sta. 41+22	143.05	7.79	1.55
	158.05	12.47	1.65
	173.05	3.15	2.45
	188.05	-0-	-0-

Table 41. Total pressure differential across the right auxiliary stilling basin wall. Total flow rate is 160,000 ft^3/s with an auxiliary flow rate 160,000 ft^3/s and main dam flow rate 0 ft^3/s (test TPD-5).

Lateral Position	Transducer Elevation (ft)	Mean Difference (ft across wall)	Std. Deviation (ft)
Sta. 39+26.5	143.05	-36.55	8.39
	158.05	-41.13	2.24
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 39+85	143.05	-19.85	6.27
	158.05	-16.68	4.28
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 40+30	143.05	4.09	1.92
	158.05	5.84	2.09
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 40+75	143.05	4.72	1.70
	158.05	1.13	1.78
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 41+22	143.05	5.57	1.45
	158.05	2.48	1.63
	173.05	-0-	-0-
	188.05	-0-	-0-

Table 42. Total pressure differential across the right auxiliary stilling basin wall. Total flow rate is 160,000 ft^3/s with an auxiliary flow rate 135,000 ft^3/s and main dam flow rate 25,000 ft^3/s (test TPD-6).

Lateral Position	Transducer Elevation (ft)	Mean Difference (ft across wall)	Std. Deviation (ft)
Sta. 39+26.5	143.05	-26.55	8.39
	158.05	-27.13	8.24
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 39+85	143.05	-9.85	6.27
	158.05	-6.68	4.28
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 40+30	143.05	8.09	1.92
	158.05	9.84	2.09
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 40+75	143.05	4.72	1.70
	158.05	1.13	1.78
	173.05	-0-	-0-
	188.05	-0-	-0-
Sta. 41+22	143.05	5.57	1.45
	158.05	2.48	1.63
	173.05	-0-	-0-
	188.05	-0-	-0-

Table 43. Total pressure differential across the right auxiliary stilling basin wall. Total flow rate is 635,750 ft^3/s with an auxiliary flow rate 312,000 ft^3/s and main dam flow rate 323,750 ft^3/s (test TPD-7).

Lateral Position	Transducer Elevation (ft)	Mean Difference (ft across wall)	Std. Deviation (ft)
Sta. 40+75	143.05	4.72	1.70
	158.05	1.13	1.78
	173.05	-2.55	2.10
	188.05	-0-	-0-

Table 44. Total pressure differential across the right auxiliary stilling basin wall. Total flow rate is 830,000 ft^3/s with an auxiliary flow rate 312,000 ft^3/s and main dam flow rate 518,000 ft^3/s (test TPD-8).

Lateral Position	Transducer Elevation (ft)	Mean Difference (ft across wall)	Std. Deviation (ft)
Sta. 40+75	143.05	4.72	1.70
	158.05	1.13	1.78
	173.05	-5.66	1.98
	188.05	-0-	-0-

Results from the pressure transducer differential measurements are quite similar to the water surface differentials measured with the wave probes. There are occasions where there is evidence of impact of flow on the wall from the main dam; however, this is reflected in changes of the differential pressure values of only a few feet of water. In order to generate 5 ft of pressure head from a stagnation of velocity, a velocity of roughly 18 ft/s directly impacting the wall is needed. This range of velocities is only likely at main dam flows of 250,000 ft^3/s and greater.

From the data collected with the pressure transducers and wave probe, a typical design approach of accounting for a full height static differential across the 66-ft-high wall appears conservative as the maximum mean values of pressure differentials were of the order of slightly more than one-half of the wall height (about 41 ft of water). Frequency analysis of the differential time series denoted no periodic forcing at any flow condition tested. Sloshing waves along the outside of the right wall were noticeable for many of the flow conditions tested when tailwater elevations were below the top of the wall. These waves at times had peak-to-peak amplitudes of 12 to 15 ft; however, their frequency was less than

0.1 Hz prototype. There was no significant frequency content in any of the differential pressure signals recorded above about 0.3 Hz prototype.

Auxiliary Spillway Tailwater Sensitivity Tests

Tailwater sensitivity tests were conducted at auxiliary spillway flows of 115,000 ft^3/s and 160,000 ft^3/s to determine the degree to which acceptable stilling basin performance was sensitive to the tailwater setting. These tests were conducted with the seven original, standard-shaped baffle blocks (16 ft high by 12 ft wide by 19 ft deep with a 1:1 sloping back face and 3 ft flat top) in one row at Station 39+71 with the 15-ft-high end sill installed from Station 41+00 to 41+32.

For each flow rate, the initial tailwater elevation in the model was set to the value approximated by the HEC-RAS prediction at cross section 28.6555. Flow conditions in the auxiliary stilling basin and exit channel were observed and photographed. The tailwater elevation was then lowered in the model. Flow conditions were allowed to stabilize and the process was repeated until the auxiliary stilling basin performance was no longer acceptable.

At a discharge of 115,000 ft^3/s, the water surface elevation at cross section 28.6555 in the HEC-RAS model was 173.86 ft. After this flow condition was observed, the model tailwater was reduced as follows:

Model Tailwater = 173.4 ft	(0.5 ft below HEC-RAS prediction)
Model Tailwater = 171.2 ft	(2.7 ft below HEC-RAS prediction)
Model Tailwater = 169.0 ft	(4.9 ft below HEC-RAS prediction)
Model Tailwater = 166.8 ft	(7.1 ft below HEC-RAS prediction)

For a discharge of 115,000 ft^3/s, the auxiliary stilling basin performance was reasonable for the entire range of tested tailwater conditions. The general trend was that as the tailwater was lowered, the toe of the jump in the basin moved downstream from the toe of the steps. The splashing in the basin increased and some splashing over the sidewalls occurred. Undulations in the water surface downstream from the basin increased. None of the conditions observed, however, appeared to be problematic or would prevent operation of the spillway and stilling basin for the given flow rate (figures 110-111).

Figure 110. View of the stilling basin from downstream at an auxiliary spillway flow of 115,000 ft^3/s and tailwater elevation of 173.86 ft as predicted by the HEC-RAS river model. Stilling basin performance is acceptable.

Figure 111. View of the stilling basin from downstream at an auxiliary spillway flow of 115,000 ft^3/s and tailwater lowered by 7.1 ft to elevation 166.8 ft. The toe of the hydraulic jump moved slightly downstream from the toe of the steps, but the basin performance is still acceptable.

At a river flow rate of 160,000 ft^3/s, the water surface elevation at cross section 28.6555 in the HEC-RAS model was 184.02 ft. After this flow condition was observed, the model tailwater was reduced as follows:

Model Tailwater = 183.9 ft	(0.1 ft below HEC-RAS prediction)
Model Tailwater = 183.0 ft	(1.0 ft below HEC-RAS prediction)
Model Tailwater = 182.8 ft	(1.2 ft below HEC-RAS prediction)
Model Tailwater = 182.0 ft	(2.0 ft below HEC-RAS prediction)

For the 160,000 ft^3/s condition, the basin performance was much less robust. At a tailwater elevation of 183.9 ft (0.1 ft below the HEC-RAS prediction) the basin performance was acceptable although periodic splashing over the sidewalls was observed. At a tailwater elevation of 183.0 ft (1.0 ft below the HEC-RAS prediction) the basin performance began to deteriorate. Surging began to develop in which the toe of the jump was pushed toward the baffle blocks creating a significant uplift of the water surface which then collapsed back on itself and pushed the jump back upstream. This process repeated itself in a cyclic fashion, with significant overtopping of the basin walls during the upswell periods. As the tailwater was lowered further, the magnitude of the surging became worse until, at a tailwater elevation of 182.0 ft (only 2 ft below the HEC-RAS predicted level) the upstream side of the baffle blocks could be occasionally seen through the flow in the stilling basin (figures 112-119).

Figure 112. View of the stilling basin from downstream at an auxiliary spillway flow of 160,000 ft^3/s and tailwater lowered by 0.1 ft to elevation 183.9 ft. The toe of the hydraulic jump moved slightly downstream from the toe of the steps, but the basin performance is still acceptable.

Figure 113. View of the stilling basin from upstream at an auxiliary spillway flow of 160,000 ft^3/s and tailwater lowered by 0.1 ft to elevation 183.9 ft. The toe of the hydraulic jump moved slightly downstream from the toe of the steps, but the basin performance is still acceptable.

Figure 114. View of the stilling basin from downstream at an auxiliary spillway flow of 160,000 ft^3/s and tailwater lowered by 1.0 ft to elevation 183.0 ft. The stilling basin has begun to sweep out and the basin performance is not acceptable.

Figure 115. View of the stilling basin from upstream at an auxiliary spillway flow of 160,000 ft^3/s and tailwater lowered by 1.0 ft to elevation 183.0 ft. The stilling basin has begun to sweep out and the basin performance is not acceptable.

Figure 116. View of the stilling basin from downstream at an auxiliary spillway flow of 160,000 ft^3/s and tailwater lowered by 1.2 ft to elevation 182.8 ft. The stilling basin sweeps out and the basin performance is not acceptable.

Figure 117. View of the stilling basin from upstream at an auxiliary spillway flow of 160,000 ft^3/s and tailwater lowered by 1.2 ft to elevation 182.8 ft. The stilling basin sweeps out and basin performance is not acceptable.

Figure 118. View of the stilling basin from downstream at an auxiliary spillway flow of 160,000 ft^3/s and tailwater lowered by 2.0 ft to elevation 182.0 ft. The stilling basin sweeps out and baffle blocks can be seen occasionally through the flow. Basin performance is not acceptable.

Figure 119. View of the stilling basin from upstream at an auxiliary spillway flow of 160,000 ft^3/s and tailwater lowered by 2.0 ft to elevation 182.0 ft. The stilling basin sweeps out and baffle blocks can be seen occasionally through the flow. Basin performance is not acceptable.

From these observations, it appears that the auxiliary stilling basin performance is acceptable without supplemental flows from the main dam for discharges up to 160,000 ft^3/s provided that the tailwater elevation at section 28.6555 is equal to or greater than the elevation predicted by the HEC-RAS study. At a discharge of 115,000 ft^3/s, the basin performance remains acceptable even for significantly lower tailwater elevations. At a discharge of 160,000 ft^3/s, however, the basin performance quickly deteriorates for tailwater elevations only 1 or 2 ft lower than the prediction in the HEC-RAS study, so supplemental flow from the main dam may be needed to ensure acceptable performance.

Auxiliary Stilling Basin Performance: Smooth Chute Comparison

The stepped spillway portion of the auxiliary spillway was covered by sheet metal to provide a qualitative comparison of stilling basin performance between a stepped spillway chute (figure 120) and a smooth spillway chute (figure 121). The seven flow conditions tested are listed in table 45 (SC = spillway chute). Photographs and video were collected during the tests. Figures 122-141 show some comparative photos between the stepped and smooth chutes.

Figure 120. Stepped spillway chute in auxiliary spillway structure.

Figure 121. Smooth spillway chute in auxiliary spillway structure.

Table 45. Flow conditions at which smooth chute and stepped chute performance were compared (SC = spillway chute).

Test No.	Auxiliary Spillway Flow Rate (ft^3/s)	Main Dam Spillway Flow Rate (ft^3/s)	Total Flow Rate (ft^3/s)
SC-1	60,000	0	60,000
SC-2	115,000	0	115,000
SC-3	160,000	0	160,000
SC-4	90,000	25,000	115,000
SC-5	135,000	25,000	160,000
SC-6	320,000	230,000	550,000
SC-7	320,000	515,000	835,000

Figure 122. Smooth spillway chute with 60,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-1).

Figure 123. Smooth spillway chute with 60,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-1).

Figure 124. Stepped chute spillway with 60,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-1).

Figure 125. Stepped chute spillway with 60,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-1).

Figure 126. Smooth spillway chute with 115,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-2).

Figure 127. Smooth spillway chute with 115,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-2).

Figure 128. Stepped spillway chute with 115,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-2).

Figure 129. Stepped spillway chute with 115,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-2).

Figure 130. Smooth spillway chute with 160,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-3).

Figure 132. Stepped spillway chute with 160,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-3).

Figure 131. Smooth spillway chute with 160,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-3).

Figure 133. Stepped spillway chute with 160,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam (test SC-3).

Figure 134. View from upstream of smooth spillway chute with 160,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam. The tailwater elevation was raised by 10 ft and the stilling basin began to perform properly (compare to test SC-3 in figures 126-129).

Figure 135. View from downstream of smooth spillway chute with 160,000 ft^3/s from the auxiliary spillway and 0 ft^3/s from the main dam. The tailwater elevation was raised by 10 ft and the stilling basin began to perform properly (compare to test SC-3 in figures 126-129).

Figure 136. Smooth spillway chute with 135,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam (test SC-5).

Figure 137. Smooth spillway chute with 135,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam (test SC-5).

Figure 138. Stepped spillway chute with 135,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam (test SC-5).

Figure 139. Stepped spillway chute with 135,000 ft^3/s from the auxiliary spillway and 25,000 ft^3/s from the main dam (test SC-5).

Figure 140. Smooth spillway chute with 320,000 ft^3/s from the auxiliary spillway and 515,000 ft^3/s from the main dam (test SC-7). When the auxiliary stilling basin is completely submerged at high discharges, the performance of the smooth chute and stepped chute are similar.

Figure 141. Stepped spillway chute with 312,000 ft^3/s from the auxiliary spillway and 518,000 ft^3/s from the main dam (similar to test SC-7).

The stepped chute is necessary to obtain acceptable performance in the stilling basin for the current stilling basin geometry. A smooth chute cannot be used unless structural modifications are made to the stilling basin design. Performance of the auxiliary stilling basin was progressively worse with increasing discharge. At 160,000 ft^3/s, a very large rooster tail was produced as the supercritical flow directly impacted the baffle blocks and deflected upward. When the tailwater elevation was raised by 10 ft to elevation 194.0 ft, the stilling basin began to perform properly. Similar results would be expected by lowering the stilling basin invert by 10 ft or more.

Splitting the discharge between the auxiliary spillway structure and the main dam structure (i.e., using main dam releases to provide supplemental tailwater) notably improved flow conditions, but the hydraulic jump location in the auxiliary spillway was shifted somewhat downstream with the smooth chute, indicating less robust performance. During the PMF, the high tailwater level completely submerged the auxiliary stilling basin. The flow conditions with a smooth chute and a stepped chute looked similar under this submerged condition.

Auxiliary Stilling Basin Performance: Baffle Blocks Removed from Stilling Basin

The baffle blocks were completely removed from the auxiliary stilling basin in the model to examine stilling basin performance during an unlikely scenario where the baffle blocks fail and are carried out of the stilling basin. Three flow conditions were initially tested with the auxiliary spillway operating near maximum capacity (table 46, BR = blocks removed).

Table 46. Initial test conditions with basin blocks completely removed from the auxiliary stilling basin (BR = blocks removed).

Test No.	Auxiliary Spillway Flow Rate (ft^3/s)	Main Dam Spillway Flow Rate (ft^3/s)	Total Flow Rate (ft^3/s)	Tailwater Elevation (ft)
BR-1	320,000	515,000	835,000	253.9
BR-2	320,000	250,000	570,000	234.0
BR-3	312,000	206,000	518,000	230.0

During the PMF release in test BR-1, the tailwater completely submerged the auxiliary stilling basin. The tailwater prevented the toe of the jump from sweeping downstream and the turbulence downstream from the stilling basin was submerged. The flow condition with no baffle blocks looked equivalent to the flow condition with baffle blocks installed (see test 18 in figure 76). In test BR-2, stilling basin performance with a release of 320,000 ft^3/s from the auxiliary spillway and 250,000 ft^3/s from the main dam also appeared to be the same with and without baffle blocks installed.

During a release of 312,000 ft^3/s from the auxiliary spillway and 206,000 ft^3/s from the main dam in test BR-3, basin performance was still similar to the case in which the baffle blocks were installed (see figures 142-143 and compare to test 20 in figure 80). With no baffle blocks in the basin, there was slightly more turbulence in the exit channel with higher velocities in the downstream channel. Velocities approaching the right bank at the curve in the river channel just upstream from the bridge pier appeared to be greater. The hydraulic jump, however, was contained inside the stilling basin near the end sill as it was during the condition with baffle blocks installed.

Following test BR-3, the discharge from the main dam was then lowered in stages to reduce the performance of the auxiliary stilling basin. This produced three additional test conditions summarized in table 47.

Table 47. Subsequent 3 test conditions with basin blocks completely removed from the auxiliary stilling basin (BR = blocks removed).

Test No.	Auxiliary Spillway Flow Rate (ft^3/s)	Main Dam Spillway Flow Rate (ft^3/s)	Total Flow Rate (ft^3/s)	Tailwater Elevation (ft)
BR-4	312,000	160,000	472,000	226.9
BR-5	312,000	120,000	432,000	222.9
BR-6	312,000	90,000	402,000	219.3

At a discharge of 160,000 ft^3/s from the main dam, the hydraulic jump was contained within the basin, but turbulence within the basin was noticeably increased. At a discharge of 120,000 ft^3/s from the main dam with a tailwater elevation of 222.9 ft, the hydraulic jump was located at the end of the basin and appeared to be on the verge of sweeping out of the basin (figures 144 and 145). At a discharge of 90,000 ft^3/s from the main dam with a tailwater elevation of 219.3 ft, the hydraulic jump completely swept out of the basin (figures 146 and 147).

Figure 142. Model overview with no baffle blocks in the auxiliary stilling basin with 312,000 ft^3/s from the auxiliary spillway and 206,000 ft^3/s from the main dam. Basin performance was similar to the condition with the baffle blocks installed.

Figure 143. Closer view with no baffle blocks in the auxiliary stilling basin with 312,000 ft^3/s from the auxiliary spillway and 206,000 ft^3/s from the main dam. Basin performance was similar to the condition with the baffle blocks installed.

Figure 144. Model overview with no baffle blocks in the auxiliary stilling basin with 312,000 ft^3/s from the auxiliary spillway and 120,000 ft^3/s from the main dam. During this condition, the hydraulic jump was located near the end sill and appeared to be on the verge of sweeping out of the basin.

Figure 145. Closer view of no baffle blocks in the auxiliary stilling basin with 312,000 ft^3/s from the auxiliary spillway and 120,000 ft^3/s from the main dam. During this condition, the hydraulic jump was located near the end sill and appeared to be on the verge of sweeping out of the basin.

Figure 146. Model overview of no baffle blocks in the auxiliary stilling basin with 312,000 ft^3/s from the auxiliary spillway and 90,000 ft^3/s from the main dam. During this condition, the hydraulic jump completely swept out of the basin.

Figure 147. Closer view of no baffle blocks in the auxiliary stilling basin with 312,000 ft^3/s from the auxiliary spillway and 90,000 ft^3/s from the main dam. During this condition, the hydraulic jump completely swept out of the basin.

Auxiliary Stilling Basin Performance: Energy Dissipation of Auxiliary Spillway Steps

Velocity data were collected along the auxiliary spillway stepped chute and the energy dissipation due to the stepped section was calculated. The stepped chute contains 68 steps and terminates in the auxiliary stilling basin at elevation 128.05 ft. The step heights of the first five steps increase to the maximum vertical offset of 3 ft. Steps 6 through 68 have a step height of 3 ft. The final step 68 is at elevation 128.05 ft on the stilling basin floor. Velocity profiles at the spillway centerline were collected at step 1, step 30, and step 64. The measurements were performed with a pitot-static tube oriented perpendicular to the slope, deployed from a point gage equipped with a vernier scale. The data were collected at the auxiliary spillway design discharge of 135,000 ft^3/s. Figure 148 shows velocity profiles at the three locations in prototype units.

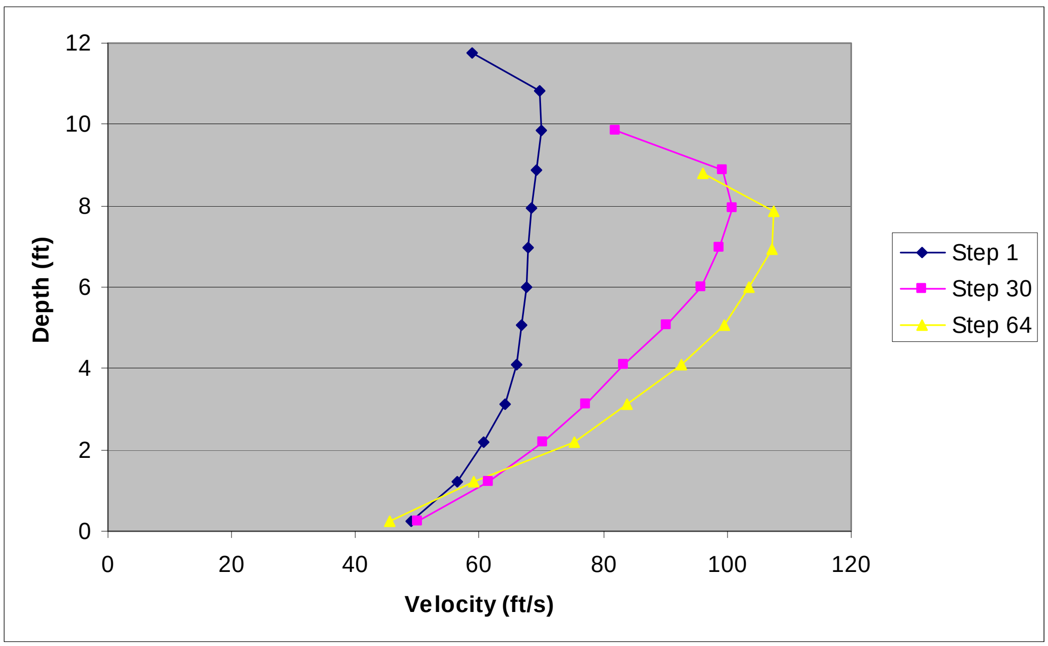

Figure 148: Velocity profiles taken at auxiliary spillway centerline for the design discharge of 135,000 ft^3/s.

The mean velocity at each step location was calculated by numerical integration of the measured velocity profiles using the trapezoidal rule. This value was used to calculate the velocity head which was added to the depth and the elevation to give the total energy at each location. In this way, energy dissipation along the steps can be calculated. Table 48 shows the energy values for the measured profiles.

Table 48. Prototype total energy values from measurements on the model auxiliary spillway at 135,000 ft^3/s.

	Mean velocity (ft/s)	Velocity Head (ft)	Flow Depth (ft)	Elevation (ft)	Total Energy (ft)
Step 1	64.19	63.98	11.77	196.32	272.07
Step 30	82.80	106.46	9.85	117.12	233.43
Step 64	86.72	116.78	8.79	14.88	140.45

To estimate energy dissipated by the last four steps, steps 65-68, of the stepped spillway, the rate of energy dissipation from step 30 to 64 (the constant-sloped section) was assumed to apply down to step 68. Total energy dissipation was calculated by taking the difference between the total energy at step 1 and the total energy estimated at step 68 divided by the energy at step 1, plus the incremental estimated rate for the final four steps. The energy dissipation from the start of the steps to the stilling basin floor was calculated as 48.4% + 4.0 % for a total of 52.4%.

A comparison between model study results is shown in table 49. The energy dissipation value from the 1:48-scale physical model compares favorably to the values measured in the 1:26-scale physical model at St. Anthony Falls Laboratory. Results varied from 48 to 58% with an average energy dissipation value of 53% (Lueker et al., 2008). The results can also be compared to a 2D representation of the stepped spillway modeled with the FLOW-3D program, a commercially available numerical code. Velocity profiles extracted from FLOW-3D predicts energy dissipation of 60.6% for the stepped portion of the chute during the design discharge (Kubitschek, 2008). Figure 149 shows the computed velocity profiles from FLOW-3D for the 135,000 ft^3/s design flow which are similar to the measured profiles shown in figure 148. The numerical model was also used to evaluate the losses on a totally smooth chute with the same geometry. The portion of the chute that previously had been stepped showed an 18.8% energy reduction over that length of chute. The difference between the stepped and smooth chutes was then 60.6% - 18.8% = 41.8%. This difference is the amount of additional energy dissipation afforded to the stepped chute over a smooth chute of the same profile and width.

Table 49. Comparison of stepped spillway energy dissipation between model studies.

	Energy Dissipation (%) in Stepped Portion of Auxiliary Spillway
Reclamation 1:48-Scale Physical Model	52.4%
SAFL 1:26-Scale Physical Model	53% (range 48 – 58%)
Reclamation FLOW-3D CFD Model	60.6%

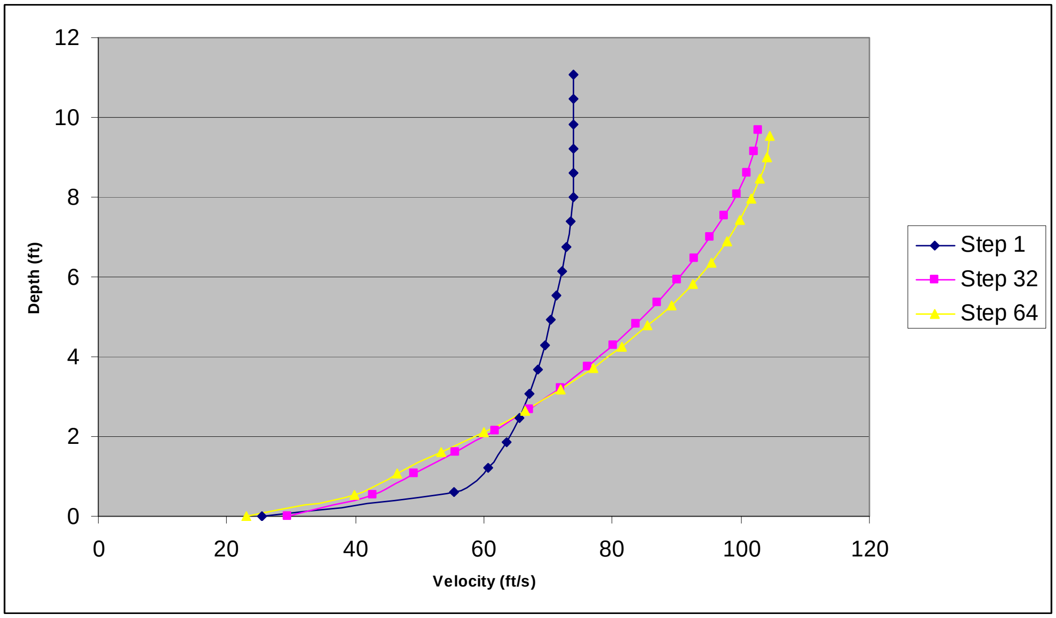

Figure 149: Computed velocity profiles for 135,000 ft^3/s from FLOW-3D numerical code.

New Baffle Block Installation

Results from a CFD model of the auxiliary spillway stepped chute show that velocities entering the stilling basin approach 100 ft/s at the design discharge of 135,000 ft^3/s and 130 ft/s for a high flow release of 300,000 ft^3/s (Kubitschek, 2008). Due to the high velocities entering the stilling basin, concerns were raised that there would be high potential for cavitation damage to the standard-shaped auxiliary baffle blocks and the stilling basin floor. A physical model study was conducted in the low ambient pressure chamber in Reclamation's hydraulics laboratory. The most effective baffle block design for reducing cavitation potential on both the blocks and the floor was a design that allowed the block to operate in the supercaviting regime with a fully ventilated cavity around the block (Frizell, 2009a). Figure 150 shows a prototype drawing of the supercavitating block with has the same frontal shape and area as the standard block, but with tapered tails. Testing in the low ambient pressure chamber also showed that the addition of a ramp upstream from the block face reduced cavitation potential on the basin floor.

After testing for cavitation damage potential in the low ambient pressure chamber was completed, seven supercavitating baffle blocks and the upstream ramp were installed in the 1:48-scale physical model to ensure that the baffle blocks provided adequate energy dissipation in the auxiliary basin. In the initial supercavitating block configuration, the ramp extended across the entire basin width just upstream from the baffle blocks and was 4 ft high and 12 ft long (prototype) on a 3:1 slope (figures 151-153).

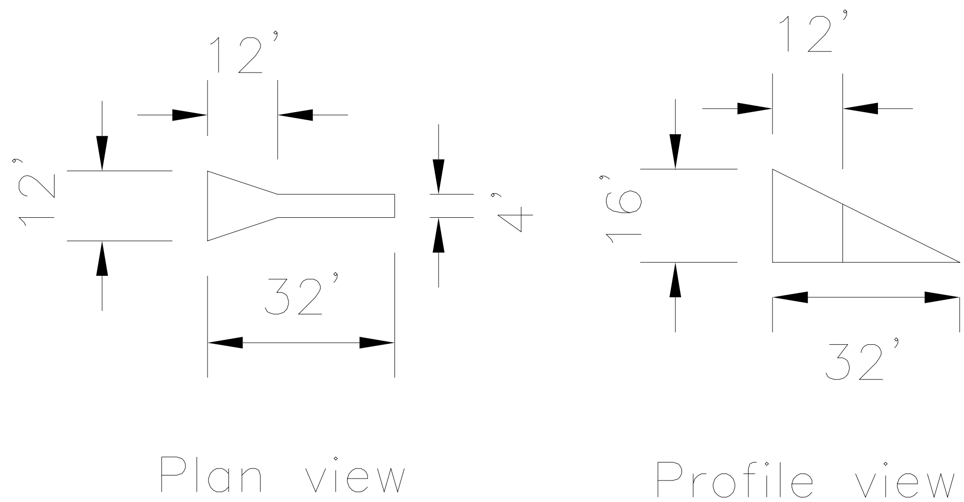

Figure 150. Prototype drawing of the supercavitating baffle block.

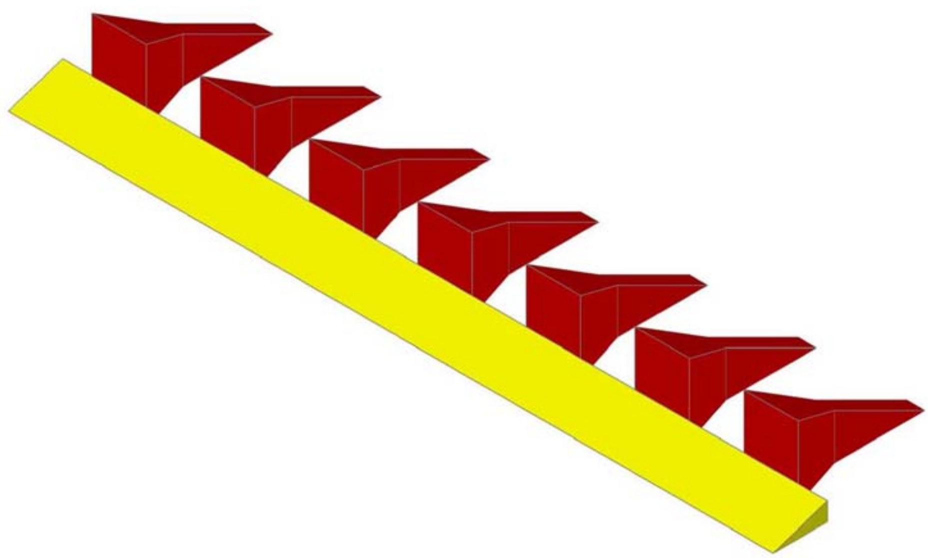

Figure 151. Schematic of supercavitating baffle block and 4-ft-high by 12-ft-long ramp.

Figure 152. View from downstream of the supercavitating baffle blocks with a 4-ft-high by 12-ft-long ramp.

Figure 153. Side view of supercavitating baffle blocks with a 4-ft-high by 12-ft-long ramp.

Tests at the design discharge showed that the ramp in front of the baffle projected the flow upward in the basin so that the water surface was elevated at the location of the baffle blocks and the flow replunged in the basin near the end sill (figures 154-155). More water splashed over the basin sidewalls with this baffle configuration than with the original baffle blocks. Turbulence in the exit channel was slightly greater with the new baffle block configuration.

Figure 154. Basin performance with supercavitating baffle blocks and a 4-ft-high by 12-ft-long ramp. Note the undulation in the water surface. Auxiliary spillway flow is 135,000 ft^3/s and main dam flow is 25,000 ft^3/s.

Figure 155. Side view of basin performance with supercavitating baffle blocks and a 4-ft-high by 12-ft-long ramp. Note the undulation in the water surface. Auxiliary spillway flow is 135,000 ft^3/s and main dam flow is 25,000 ft^3/s.

Observations of basin performance in the 1:48-scale model suggested that the ramp affected the flow in two ways. In addition to projecting the flow upward, the ramp seemed to make the portion of the baffle block below the ramp crest ineffective, which reduced energy dissipation. To confirm this concept, the same flow condition was run with the ramp completely removed to ensure that basin performance was affected by the ramp only and not the change in block shape.

Results showed that baffle block performance was acceptable when the ramp was removed. To preserve the cavitation benefits of the ramp and to obtain better basin performance, four alternate ramp configurations were then tested in the model to determine the best ramp size and location. In Ramp 01, the ramp retained a 3:1 slope, but the dimensions were reduced to 2 ft high by 6 ft long. In Ramp 02, the ramp slope was increased to 1:1 with dimensions of 4 ft high by 4 ft long. Ramp 03 included the initial dimensions of the 4-ft-high by 12-ft-long ramp, but the ramp was moved downstream between the baffle blocks. Ramp 04 also retained the 4-ft-high by 12-ft-long ramp size, but was placed in front of the baffle blocks in section so that the ramp did not block the front face of the baffle blocks. Figures 156-159 show schematics of the 4 alternate ramp configurations.

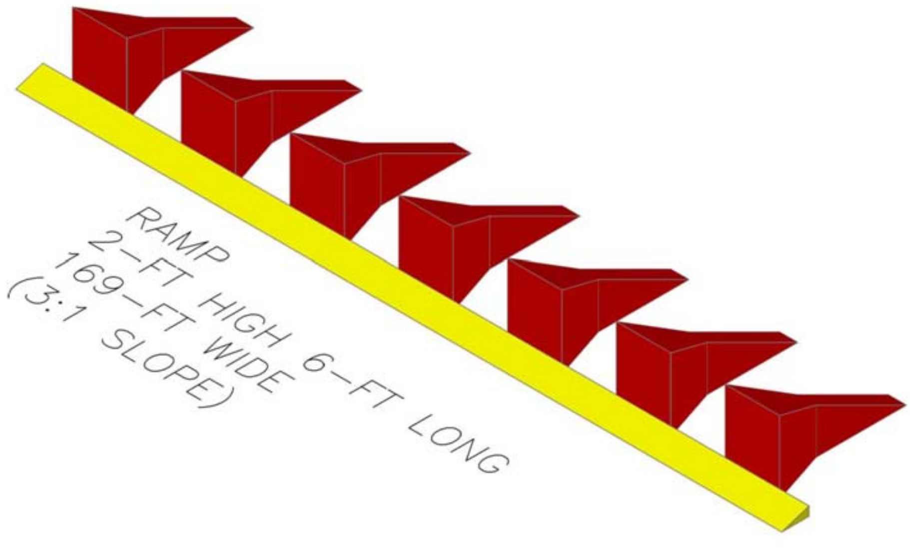

Figure 156. Ramp 01 with a 2-ft-high by 6-ft-long ramp on a 3:1 slope.

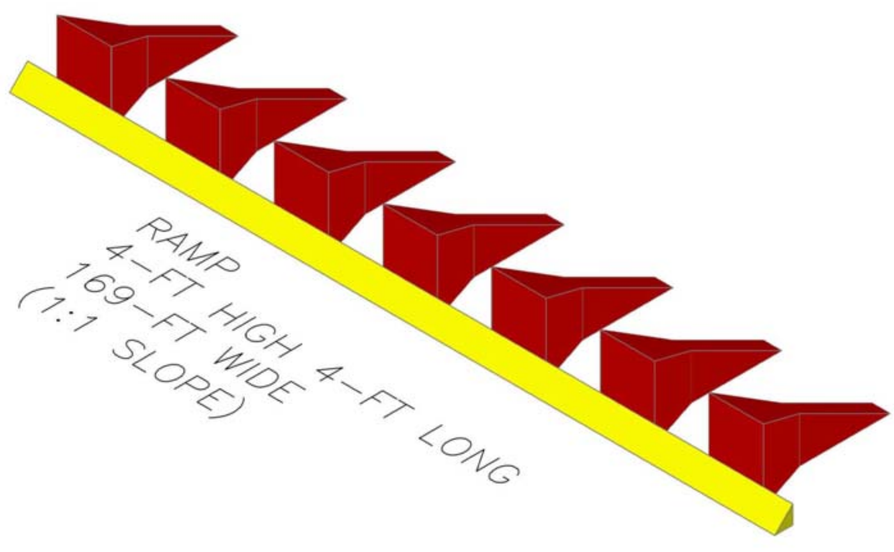

Figure 157. Ramp 02 with a 4-ft-high by 4-ft-long ramp on a 1:1 slope.

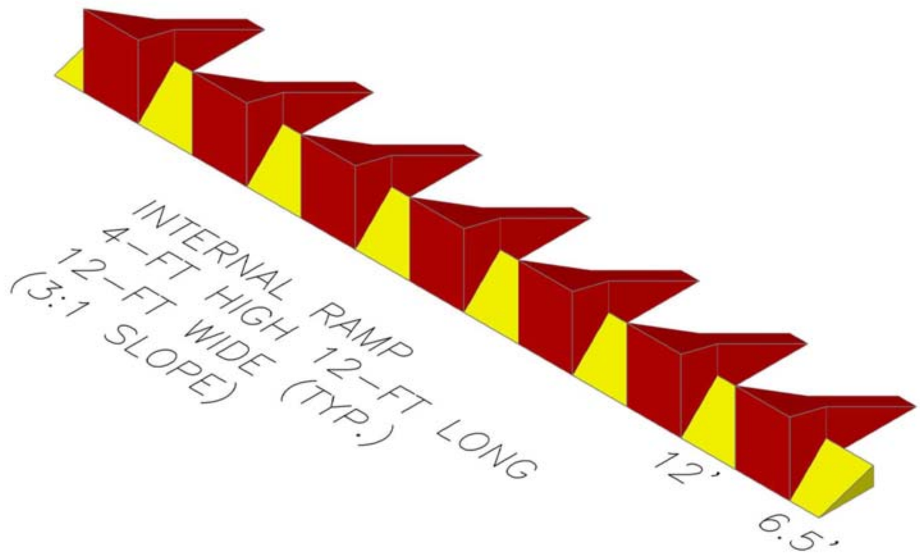

Figure 158. Ramp 03 with a 4-ft-high by 12-ft-long ramp on a 3:1 slope moved downstream between the baffle blocks.

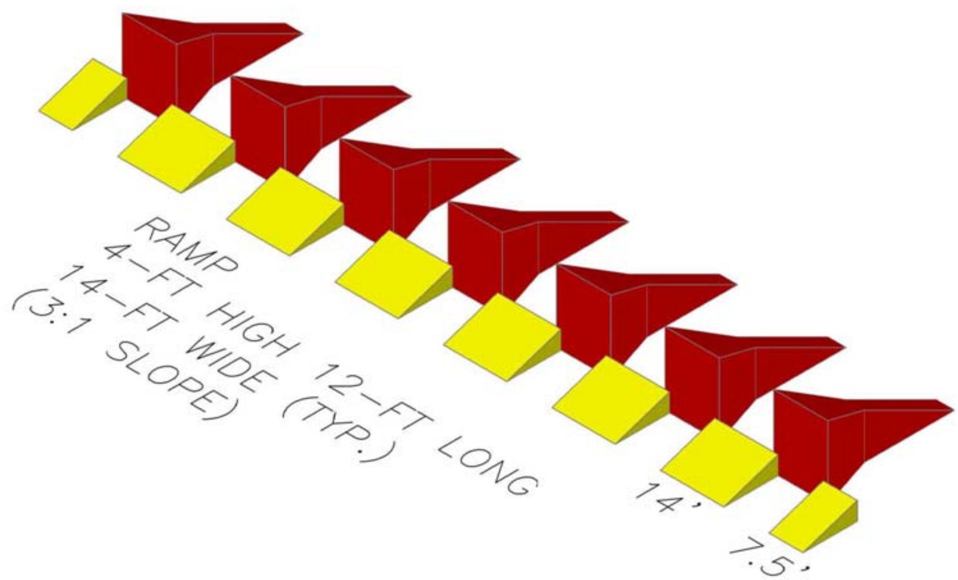

Figure 159. Ramp 04 with a 4-ft-high by 12-ft-long ramp on a 3:1 slope placed in front of the baffle blocks in sections.

The installation of Ramp 01, Ramp 02, and Ramp 04 did not improve flow patterns in the basin. Photographs of basin performance are shown in figures 160-165 for these ramp configurations.

Figure 160. Ramp 01 with a 2-ft-high by 6-ft-long ramp on a 3:1 slope. Auxiliary spillway flow is 135,000 ft^3/s and main dam flow is 25,000 ft^3/s. Flow patterns are not improved over the initial ramp design.

Figure 161. Side view of Ramp 01 with a 2-ft-high by 6-ft-long ramp on a 3:1 slope. Auxiliary spillway flow is 135,000 ft^3/s and main dam flow is 25,000 ft^3/s. Flow patterns are not improved over the initial ramp design.

Figure 162. Ramp 02 with a 4-ft-high by 4-ft-long ramp on a 1:1 slope. Auxiliary spillway flow is 135,000 ft^3/s and main dam flow is 25,000 ft^3/s. Flow patterns are not improved over the initial ramp design.

Figure 163. Side view of Ramp 02 with a 4-ft-high by 4-ft-long ramp on a 1:1 slope. Auxiliary spillway flow is 135,000 ft^3/s and main dam flow is 25,000 ft^3/s. Flow patterns are not improved over the initial ramp design.

Figure 164. Ramp 04 with a 4-ft-high by 12-ft-long ramp on a 3:1 slope placed in front of the baffle blocks in sections. Auxiliary spillway flow is 135,000 ft^3/s and main dam flow is 25,000 ft^3/s. Flow patterns are not improved over the initial ramp design.

Figure 165. Side view of Ramp 04 with a 4-ft-high by 12-ft-long ramp on a 3:1 slope placed in front of the baffle blocks in sections. Auxiliary spillway flow is 135,000 ft^3/s and main dam flow is 25,000 ft^3/s. Flow patterns are not improved over the initial ramp design.

The best basin performance was observed with Ramp 03 installed (figures 166-168). During the design discharge, the water surface was elevated at the baffle blocks with a standing wave at the end sill. However, energy dissipation appeared to be adequate and splashing over the sidewalls was reduced. The objectives of minimizing cavitation damage potential while maintaining sufficient energy dissipation at the design discharge were balanced so that both goals were achieved. Basin performance during various flow conditions is shown in figures 169-176. It appears that the performance of the supercavitating baffle blocks and ramp configuration is less dependent on tailwater depth than the original baffle block configuration. This finding warrants further investigation in a future study.

Figure 166. Looking upstream at Ramp 03 with a 4-ft-high by 12-ft-long ramp on a 3:1 slope between the baffle blocks.

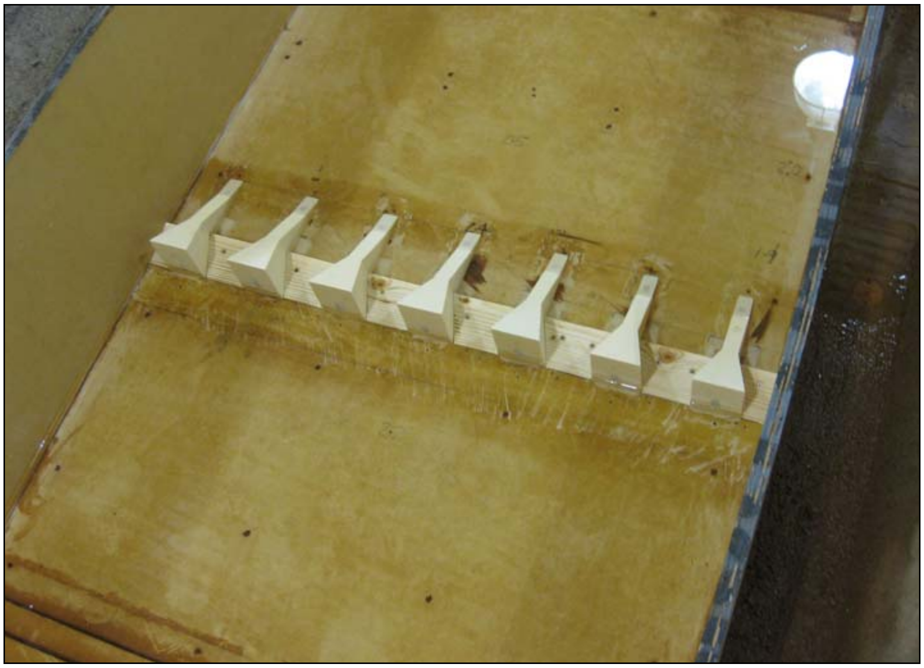

Figure 167. Looking downstream at Ramp 03 with 4-ft-high by 12-ft-long ramp on a 3:1 slope between the baffle blocks.

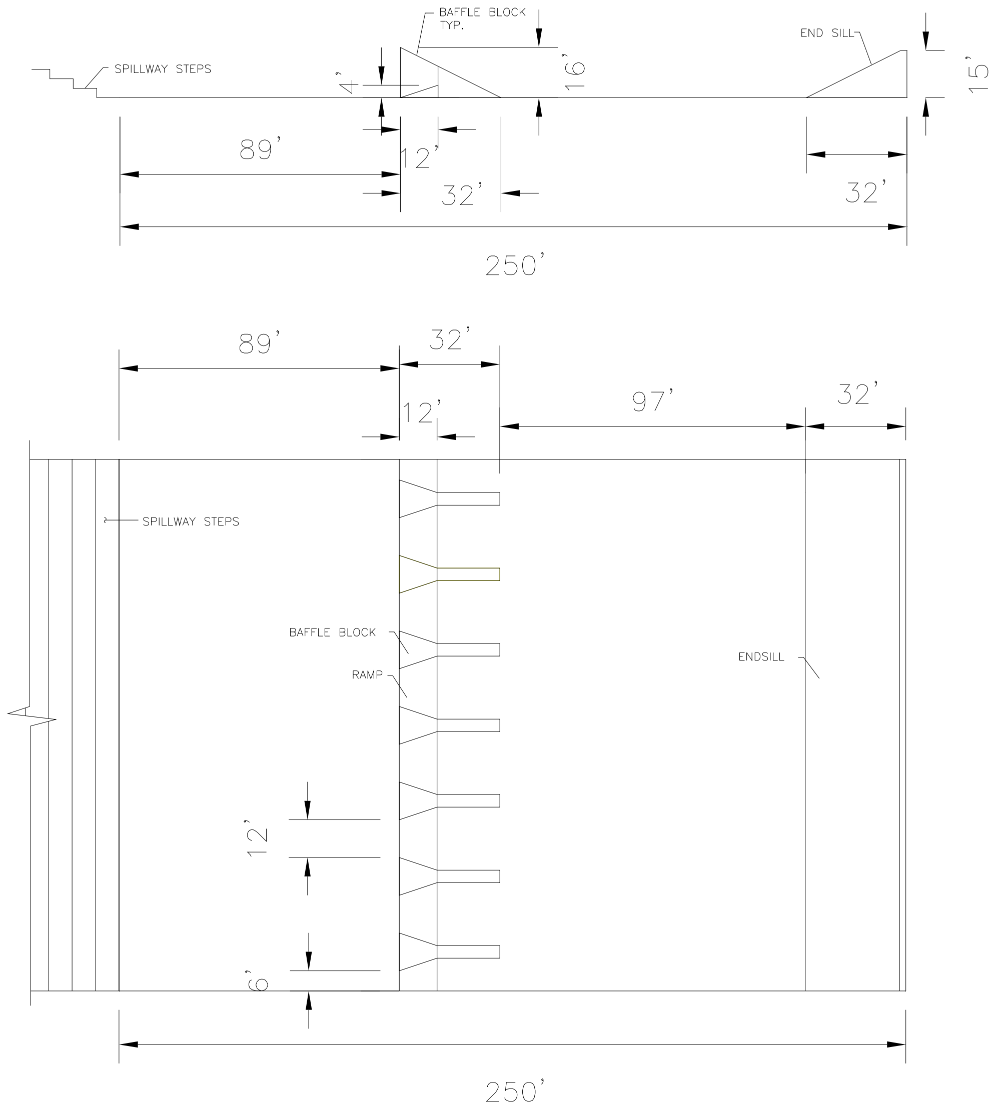

Figure 168. Prototype drawing of stilling basin with supercavitating baffle blocks and a 4-ft-high by 12-ft-long ramp between the blocks.

Figure 169. Preferred ramp alternative (Ramp 03) with auxiliary spillway flow of 90,000 ft^3/s and main dam flow of 25,000 ft^3/s.

Figure 170. Side view of flow conditions with preferred ramp alternative (Ramp 03) with auxiliary spillway flow of 90,000 ft^3/s and main dam flow of 25,000 ft^3/s.

Figure 171. Preferred ramp alternative (Ramp 03) with auxiliary spillway flow of 135,000 ft^3/s and main dam flow of 25,000 ft^3/s.

Figure 172. Side view of flow conditions with preferred ramp alternative (Ramp 03) with auxiliary spillway flow of 135,000 ft^3/s and main dam flow of 25,000 ft^3/s.

Figure 173. Preferred ramp alternative (Ramp 03) with auxiliary spillway flow of 160,000 ft^3/s and main dam flow of 0 ft^3/s.

Figure 174. Side view of flow conditions with preferred ramp alternative (Ramp 03) with auxiliary spillway flow of 160,000 ft^3/s and main dam flow of 0 ft^3/s.

Figure 175. Preferred ramp alternative (Ramp 03) with auxiliary spillway flow of 312,000 ft^3/s and main dam flow of 518,000 ft^3/s.

Figure 176. Side view of flow conditions with preferred ramp alternative (Ramp 03) with auxiliary spillway flow of 312,000 ft^3/s and main dam flow of 518,000 ft^3/s.

To compare energy dissipation between the new baffle block and ramp configuration and the original baffle blocks, two-dimensional velocities were measured downstream from the end sill in the exit channel. Figure 82 in the

"Channel Velocity Data" section shows the data measurement locations for stations 11 through 15 and the instrument orientation at those locations. All velocity data are shown in tables 50 and 51. Results showed that the downstream component of the velocities in the exit channel were similar to the downstream component of the velocities measured with the original baffle blocks installed. A summary of the data is as follows:

- Auxiliary spillway flow rate 135,000 ft^3/s, main dam flow rate 25,000 ft^3/s
    - Downstream component of velocity ranges from 10.7 to 16.6 ft/s with original baffle blocks.
    - Downstream component of velocity ranges from 10.7 to 17.7 ft/s with new baffle blocks and ramp.
- Auxiliary spillway flow rate 160,000 ft^3/s, main dam flow rate of 0 ft^3/s
    - Downstream component of velocity ranges from 11.6 to 21.2 ft/s with original baffle blocks.
    - Downstream component of velocity ranges from 13.7 to 18.9 ft/s with new baffle blocks and ramp.

Table 50. Comparison of velocities in auxiliary stilling basin exit channel between the original baffle blocks and the new supercavitating baffle blocks and ramp configuration. Auxiliary spillway flow rate 135,000 ft^3/s and main dam flow rate 25,000 ft^3/s.

ORIGINAL BAFFLE BLOCKS		MODEL DATA				PROTOTYPE DATA			
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)
11	27A	1.1	2.24	-0.09	2.25	52.8	15.55	-0.63	15.6
12	28A	1	1.54	-0.22	1.56	48.0	10.68	-1.51	10.8
13	Left Bank	0.55	2.32	-0.52	2.38	26.4	16.06	-3.63	16.5
14	27B	1.1	2.40	0.33	2.42	52.8	16.63	2.28	16.8
15	28B	1.05	2.35	-0.54	2.41	50.4	16.27	-3.75	16.7

NEW BAFFLE BLOCKS & RAMP		MODEL DATA				PROTOTYPE DATA			
Station	Tap	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)
11	27A	1.1	1.99	0.35	2.02	52.8	13.79	2.42	14.0
12	28A	1	1.54	0.14	1.55	48.0	10.67	0.97	10.7
13	Left Bank	0.65	2.56	-0.94	2.73	31.2	17.74	-6.51	18.9
14	27B	1.2	2.34	-0.17	2.35	57.6	16.21	-1.18	16.3
15	28B	1	2.02	-1.10	2.30	48.0	13.99	-7.62	15.9

Table 51. Comparison of velocities in auxiliary stilling basin exit channel between the original baffle blocks and the new supercavitating baffle blocks and ramp configuration. Auxiliary spillway flow rate 160,000 ft^3/s and main dam flow rate 0 ft^3/s.

		MODEL DATA				PROTOTYPE DATA			
ORIGINAL BAFFLE BLOCKS		Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)
Station	Tap								
11	27A	1.1	2.85	0.51	2.90	52.8	19.75	3.53	20.1
12	28A	1.05	1.67	-0.04	1.67	50.4	11.57	-0.28	11.6
13	Left Bank	0.7	2.63	-0.35	2.65	33.6	18.22	-2.42	18.4
14	27B	1.1	3.06	0.48	3.10	52.8	21.20	3.33	21.5
15	28B	1	2.94	-0.37	2.96	48.0	20.37	-2.56	20.5

		MODEL DATA				PROTOTYPE DATA			
NEW BAFFLE BLOCKS & RAMP		Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)	Flow Depth (ft)	Vx (ft/s)	Vy (ft/s)	Resultant Velocity (ft/s)
Station	Tap								
11	27A	1.15	2.16	0.34	2.19	55.2	14.96	2.36	15.1
12	28A	1.1	1.97	0.23	1.98	52.8	13.65	1.59	13.7
13	Left Bank	0.75	2.73	-0.10	2.73	36.0	18.91	-0.69	18.9
14	27B	1.2	2.48	-0.38	2.51	57.6	17.18	-2.63	17.4
15	28B	1	1.97	-0.52	2.04	48.0	13.65	-3.60	14.1

Conclusions

A 1:48-scale physical hydraulic model of the principal features of the Joint Federal Project at Folsom Dam was constructed and tested in Reclamation's Hydraulics Laboratory. The model included the main dam spillways, the auxiliary spillway and stilling basin, the confluence area of the two exit channels, and a section of the downstream river channel. The primary objective of the physical modeling was to evaluate the three-dimensional flow characteristics in the vicinity of the confluence between the main dam exit channel and the auxiliary spillway channel. Design and operational issues were also addressed with this model study. Important results of this model study are as follows:

- Water surface profiles measured along the two cofferdam sections in the model were used to design the prototype cofferdam elevations. To allow for 1 ft of freeboard along the government-designed reinforced concrete section of the cofferdam at a main dam discharge of 50,000 ft^3/s, the wall was designed to have 4 sections at various elevations (elevation 178.5 for 153 ft, elevation 177.5 for 269 ft, elevation 173.0 for 36 ft, and elevation 168.5 for 135 ft).

- To allow for 1 ft of freeboard at a main dam discharge of 50,000 ft^3/s along the temporary contractor-designed cofferdam downstream of the auxiliary stilling basin, the cofferdam crest elevation should be set to 168.5 ft.

- Water surface elevations were collected throughout the model. Flow overtops the right auxiliary stilling basin wall when the main dam releases 300,000 ft^3/s or when the main dam releases 140,000 ft^3/s with an auxiliary spillway release of 160,000 ft^3/s. The water level reaches the right bridge pier of the Folsom Lake Crossing Bridge at a river flow rate of about 250,000 ft^3/s.

- During all emergency spillway releases from the main dam including the PMF, there is insufficient tailwater to cushion the impact of the flip bucket flow on the downstream concrete pad.

- In the confluence area, vertical wave heights along the right and left banks were 2 to 8 ft above the normal water surface for total discharges less than 300,000 ft^3/s. The only exception to this was the flow condition with 160,000 ft^3/s from the main dam and no flow from the auxiliary spillway where wave heights of 10-12 ft were observed along the right bank.

- Wave heights above 8 ft were observed during total discharges greater than 300,000 ft^3/s. The greatest wave heights were observed during the PMF flow condition with vertical wave heights of up to 28 ft along both banklines.

- Velocities in the American River channel were in the downstream direction with transverse flow toward the right bank at most locations in the confluence area. A dead zone or recirculation zone existed for most test conditions in the powerplant tailrace.

- When 25,000 ft^3/s was released from the auxiliary spillway without a release from the main dam, turbulent boiling against the right bank was observed in the confluence area.

- With the exception of very large floods of 518,000 ft^3/s and above, the highest velocity recorded perpendicular to the right bank was 5.96 ft/s in the confluence area during an auxiliary spillway release of 60,000 ft^3/s.

- During total discharges of 160,000 and 830,000 ft^3/s, the maximum measured resultant velocities in the American River channel were 19.4 ft/s and 26.6 ft/s, respectively.

- Two-dimensional velocities in the river channel can be analyzed by a team of geotechnical and hydraulic engineers to determine if protective measures may be necessary.

- In the auxiliary spillway exit channel, downstream velocities increase with increasing discharge from the auxiliary spillway. The highest downstream velocity recorded was 21.4 ft/s prototype during a discharge of 115,000 ft^3/s from the auxiliary spillway and no flow from the main dam.

- Velocities along the left bankline downstream of the auxiliary stilling basin were generally directed away from the bankline and into the river channel. The greatest velocity impinging (perpendicular to) the left bankline was 3.5 ft/s prototype at an auxiliary spillway discharge of 160,000 ft^3/s.

- Main dam releases, both independently and in conjunction with auxiliary spillway releases, produced lower transverse and streamwise velocities in the auxiliary spillway exit channel.

- Direct impact on the knob topographic feature occurred for main dam discharges of up to 60,000 ft^3/s with no discharge from the auxiliary spillway. Prototype velocities of 21.9 and 19.4 ft/s were measured perpendicular to the knob face near the juncture with the cofferdam wall for 25,000 and 60,000 ft^3/s, respectively. The highest velocity components sweeping parallel to the knob face and into the river channel were 28.1 and 21.5 ft/s for 25,000 and 60,000 ft^3/s, respectively. Shallow overtopping of the knob occurs at 60,000 ft^3/s.

- Velocities at the knob decreased significantly during a discharge of 90,000 ft^3/s because the knob was submerged with a weak hydraulic jump.

Velocities at the knob were also considerably lower when flows were split between the main dam structure and the auxiliary spillway structure.

- Velocities near to the right bridge pier on the Folsom Lake Crossing Bridge were collected at 200,000, 250,000, 300,000, and 570,000 ft^3/s. At a point 48 ft upstream and 48 ft to the left of the bridge pier, longitudinal and transverse velocity components increased with river flow rate. The longitudinal downstream velocity was 9.2 ft/s with a 2.8 ft/s transverse velocity toward the right bank at 200,000 ft^3/s and the longitudinal downstream velocity was 15.6 ft/s with a 6.2 ft/s transverse velocity toward the right bank at 570,000 ft^3/s.

- Streamwise downstream velocities near the toe of the bank by the right bridge pier ranged from 19.0 to 21.1 ft/s and transverse velocities toward the right bank ranged from 1.5 to 4.7 ft/s. There was little change in velocities measured near the toe of the bank when the discharge was increased.

- Velocities collected near the right bridge pier can be analyzed by a team of geotechnical and hydraulic engineers to determine if bridge pier scour may be problematic. Measured velocities can be used in the design of protective measures as applicable.

- Differential loadings on the right auxiliary stilling basin wall were determined from pressure and water surface measurements taken in the model for a range of flow conditions. Flush-mount pressure transducers and a wave probe arrangement show that the typical design approach of accounting for a full height static differential across the wall appears conservative as the maximum mean values of pressure differentials were on the order of slightly more than one-half of the wall height (maximum differential load of about 41 ft of water on a 66-ft-high wall).

- The highest pressure differential loadings on the right auxiliary stilling basin wall occur near the beginning of the stilling basin at a condition where the flow is still supercritical inside the basin yielding a minimum water level inside the basin and full tailwater on the outside of the basin. Frequency analysis of the differential time series denoted no periodic forcing at any flow condition tested.

- With the original baffle blocks installed, the auxiliary stilling basin performance is acceptable without supplemental main dam flows for discharges up to 160,000 ft^3/s as long as the tailwater elevation at section 28.6555 is equal to or greater than the elevation predicted by the HEC-RAS study (184.02 ft). Basin performance quickly deteriorates for tailwater elevations only 1 or 2 ft lower than the prediction in the HEC-RAS study.

- At an auxiliary spillway discharge of 115,000 ft^3/s with the original baffle blocks installed, the auxiliary basin performance remains acceptable even for tailwater elevations up to 7 ft less than the HEC-RAS prediction.

- The auxiliary stepped spillway chute is necessary to obtain acceptable performance of the auxiliary stilling basin for the current stilling basin geometry. A smooth chute cannot be used unless structural modifications are made to the stilling basin design.

- With a smooth chute replacing the stepped portion of the auxiliary chute, performance of the auxiliary stilling basin was progressively worse with increasing discharge with no flow from the main dam. At 160,000 ft^3/s, a very large rooster tail was produced as the flow directly impacted the baffle blocks and deflected upward. When the tailwater elevation was raised by 10 ft to elevation 194.0 ft, the auxiliary stilling basin began to perform properly.

- If the auxiliary baffle blocks were to fail and be carried out of the stilling basin, the auxiliary stilling basin performance would be acceptable during a maximum release of 312,000 ft^3/s as long as at least 120,000 ft^3/s is also released from the main dam. If the discharge from the main dam is less than 120,000 ft^3/s, the tailwater is too low for the auxiliary stilling basin to contain the hydraulic jump.

- Since it may be difficult to tell whether failure of the auxiliary stilling basin baffle blocks has occurred during a large flood, main dam releases should be increased with a corresponding decrease in the auxiliary spillway releases if the auxiliary stilling basin performance appears unacceptable during flood operation.

- Velocity profiles were measured along the stepped portion of the auxiliary spillway at steps 1, 30, and 64 during the design discharge of 135,000 ft^3/s. The stepped chute dissipated 52.4% of the energy available from the top of the chute to the stilling basin floor. This value compares favorably to the average energy dissipation value of 53% (range 48-58%) measured in the SAFL 1:26-scale physical model (Lueker et al., 2008) and the predicted energy dissipation value of 60.6% from the FLOW-3D numerical model (Kubitschek, 2008).

- A new baffle block design comprised of seven supercavitating blocks and an upstream ramp was installed in the model after testing in a low ambient pressure chamber showed low cavitation potential. The supercavitating baffle blocks performed adequately, but the ramp in front of the baffle blocks projected the flow upward in the basin so that the water surface was elevated at the location of the baffle blocks and the flow replunged near the end sill.

- Four alternate ramp configurations were tested in the model. A 4-ft-high by 12-ft-long ramp placed between the blocks produced good basin performance with a mild change in water surface elevation and a standing wave at the end sill. Although the basin performance was slightly altered, energy dissipation was adequate and velocities measured downstream from the end sill in the exit channel were similar to velocities measured with the original baffle blocks.

- It appears that the performance of the supercavitating baffle blocks and ramp configuration is less dependent on tailwater depth than the original baffle block configuration. This finding warrants further investigation in a future study.

- During flood operations, the performance of the auxiliary spillway stilling basin should be monitored. If the performance of the auxiliary stilling basin becomes unacceptable, performance can be improved by increasing main dam releases with a corresponding decrease in the auxiliary spillway release.

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