UNIVERSITY OF MINNESOTA
**ST. ANTHONY FALLS LABORATORY**
**Engineering, Environmental and Geophysical Fluid Dynamics**

**PROJECT REPORT 512**

# The Physical Model Study of the Bond Falls Emergency Spillway

By

Richard Christopher and Omid Mohseni

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Downstream view of the 1:25 scale model of the Bond Falls existing service spillway

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## Executive Summary

The Bond Falls Main Dam located in the Upper Peninsula, MI, is approximately 50 feet high with a crest length of 820 feet. Its service spillway system is currently rated at 6,000 cfs. To pass the probable maximum flood (PMF) through the reservoir, an additional spillway or a new spillway will be required. The conditions downstream of the spillway, primarily a low automotive bridge, the shallow channel between the bridge and the toe of the spillway, and a contraction in the river channel downstream of the bridge were found likely to cause overtopping of the channel side walls downstream of the spillway, thus undermining the embankment toe.

A physical model study of the Bond Falls spillway was conducted at the St. Anthony Falls Laboratory to test two new spillway designs, proposed by Ayres Associates. The primary objective of the testing was to determine if the proposed designs could discharge the PMF out of the reservoir without impacting the embankment toe. The model was built at a scale of 1:25. The initial test series was to verify the model. A total of five test series were then conducted on two spillway designs and some modified configuration of each design. All tests were documented using a digital video camera. The velocities of the sheet flow over the toe of the embankment resulting from the flow overtopping the west side wall were measured using the Particle Imagery Velocimetry to determine any potential of erosion at the toe of the embankment.

The spillway Design-1 was comprised of a broad-crested weir equipped with two tainter gates and positioned to the east side of the existing service spillway. The results of the test conducted on the spillway Design-1 showed that this design could not discharge the PMF without undermining the embankment toe. The spillway Design-2 was also comprised of a broad-crested weir but with two larger tainter gates replacing the existing spillway. This design could potentially discharge the PMF without undermining the embankment toe. The constraint for the spillway Design-2 was the county bridge pier and its low chord elevation. If the county bridge is rebuilt without any piers and the low chord height is set approximately 6.3 ft higher than the existing one, the PMF can be discharged through the system without undermining the embankment toe. With the modified spillway Design-2, the county bridge should be built with no piers and a low chord only 3 ft higher to pass the PMF.

## Acknowledgements

The work reported herein was supported by the Upper Peninsula Power Company (UPPCO). Mr. Ben Trotter from the Wisconsin Public Service Corporation was the project manager. We would like to thank Mr. Dean Steines from Ayres Associates for providing the surveyed data, and reviewing and providing feedback on the physical model layouts. We also thank Ben Erickson, Chris Ellis, Mike Plante, and Matthew Lueker of St. Anthony Falls Laboratory for their contribution to construction, instrumentation and data collection.

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# 1. Introduction

The Bond Falls Main Dam is located in Ontonagon County, in the Upper Peninsula, MI. The Bond Falls Dam is approximately 50 feet high with a crest length of 820 feet. The surface area of the lake is about 2,160 acres. The spillway crest is 26 feet long. In June 2004, STS Consultants estimated the probable maximum flood (PMF) with a peak inflow of 16,500 cfs. In 2005, the PMF peak outflow through the Bond Falls Spillway was estimated to be 13,500 cfs. The existing spillway was found to be insufficient to pass the PMF. The Upper Peninsula Power Company (UPPCO) retained Ayres Associates to design a new emergency spillway for the Bond Falls Dam to meet the new requirement.

The Bond Falls Spillway is currently rated for 4,600 cfs but according to the estimates given by Ayres Associates it can be upgraded to approximately 6,000 cfs with minor modifications to the tainter gate system. To pass the PMF through the reservoir, an additional spillway or a new spillway configuration will be required. The conditions downstream of the spillway, primarily a low automotive bridge, the shallow channel between the bridge and the toe of the spillway, and a contraction in the river channel downstream of the bridge were found likely to cause overtopping of the channel side walls downstream of the spillway thus impacting the embankment toe. The topographic map of the region with the location of the spillway, road and the stream channel is given in Figure 1.1.

The complex geometry and the flow path between the bridge and spillway[1] on one hand and the concrete retaining wall downstream of the bridge on the other hand do not allow one-dimensional computer models to simulate the flow patterns accurately, i.e. it is not a one-dimensional flow problem. Other more complex computer models require extensive observed data for calibration. Therefore, UPPCO decided to have a physical model study done on the exiting spillway system and the new proposed designs to determine the best alternative to safely pass the PMF (13,500 cfs).

The scope of this project was to conduct a physical model study on the spillway system of the Bond Falls Main Dam to assess the effects of two proposed spillway designs on the potential

[1] It is a steep channel with an asymmetrical expansion where standing waves can form.

spillage of water over the channel walls and subsequently possible erosion at the toe of the embankment, and eventually to help with the design and the selection of the best emergency spillway design for the Bond Falls Main Dam.

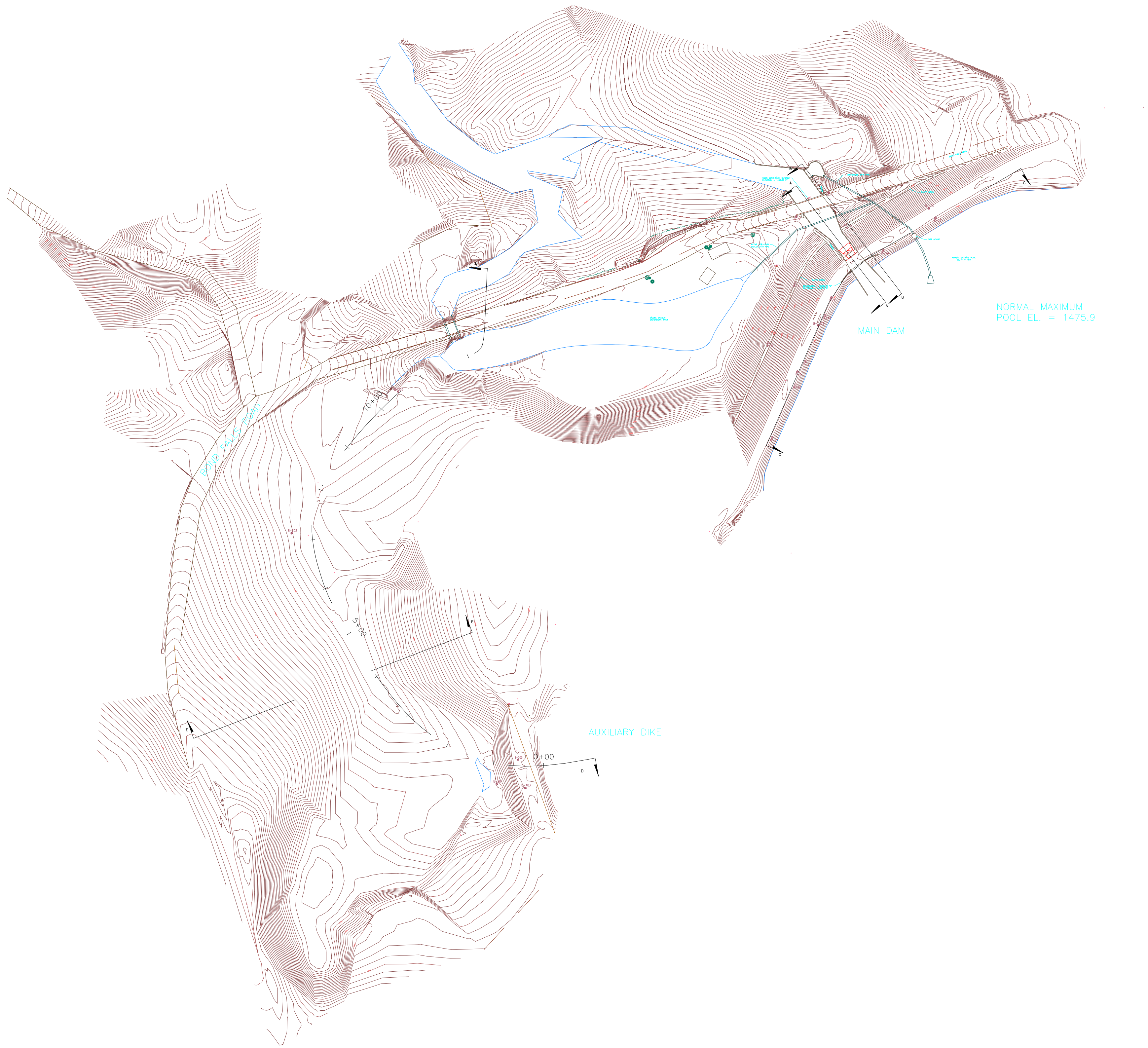



**Figure 1.1**. Topographic map of the region developed by the STS Consultants from 1-ft LIDAR data.

## 2. Physical Model Construction

### *2.1. Model Scale*

The goal of the physical model study was to accurately simulate the flow conditions associated with the spillway, the concrete and bedrock channel, the bridge and road, and also the toe of the earth dam when the flow overtopped the channel walls.

For this model study, Froude similarity was sufficient to accurately simulate the flow patterns downstream of the spillway, through the bridge and over the side walls. In order to fit the physical model study into the space available on the Model Floor of the Saint Anthony Falls Laboratory (SAFL) a scale of 1:25 was selected. The model width was the primary factor in choosing the scale. At a scale of 1:25, the maximum discharge through the model, i.e. 13,500 cfs, becomes 4.32 cfs. Table 1.1 gives the geometric and dynamic scales used in this model study.

**Table 1.1.** Geometric and dynamic scaling ratios

| Parameter | Relationship | Scale |
|---|---|---|
| Length | $r$ | 1:25 |
| Area | $r^2$ | 1:625 |
| Velocity | $r^{1/2}$ | 1:5 |
| Discharge | $r^{5/2}$ | 1:3125 |
| Manning's coefficient of roughness | $r^{1/6}$ | 1:1.71 |

### *2.2. Physical Model Extent*

To determine the downstream end of the model, it was necessary to investigate the effects of tailwater levels on the flow patterns upstream of the bridge. It was not completely clear whether the channel downstream of the bridge, or the concrete retaining wall on the east side of the channel or only the bridge was a control section. Therefore, it was decided to run a HEC-RAS model of the river downstream of the bridge. The UPPCO provided 12 cross-sections of the river from the downstream point of the bridge to the location where there was a free fall. The cross-sections and the resulting contour lines of the surveyed area are shown in Figure 2.1. The HEC-

RAS model was developed using the data shown in Figure 2.1 and the observations made during the site visit in October 2007. The channel bed was all bed rock with a very steep slope, while the floodplain was a densely wooded area (Figure 2.2). The downstream boundary condition was set at the critical depth because of the free fall at the downstream end of the stream. The upstream boundary condition was set at the normal depth because the upstream end of the surveyed area was relatively flat, and a normal depth at the upstream could potentially pronounce the effect of tailwater on the flow patterns at the bridge and upstream of the bridge. Assuming a Manning's n-value[2] of 0.15 for the floodplains and 0.03 for the channel, the HEC-RAS model was run under the 13,500 flow condition, i.e. the PMF. The model was run under "mixed flow conditions" to allow both subcritical and supercritical regimes in the stream.

From the site visit and the topographic maps, the flatness of the floodplain was not readily evident. Therefore, the extent of the surveyed cross-sections was set between 100 and 150 ft. The HEC-RAS model, however, showed that the 150 ft width for a cross-section was too short under the PMF and by default assumed a vertical wall at both ends of all cross-sections, which caused an unrealistic high water depth in the stream. Despite this unrealistic projection of the topography along the floodplain, the simulation results showed that supercritical flow would occur in some sections of the stream primarily because of the very steep slope of the channel as shown in Figure 2.3. The red line in Figure 2.3 represents the critical depth along the channel. Whenever the water depth drops below the critical depth, the flow regime becomes supercritical. The presence of supercritical regime shows that the control section is upstream and most likely at the bridge or the concrete retaining wall downstream of the bridge, and there is no tailwater effect from the channel downstream. Nevertheless, another scenario was run using the HEC-RAS model and it was assumed that no supercritical flow could occur along the stream downstream of the bridge. The simulation result is shown in Figure 2.4. Since no-supercritical condition was allowed, the stretches which exhibited supercritical condition in the first hydraulic simulation (Figure 2.3) only dropped to the critical depth (an unrealistic condition) and subsequently the water depth at the upstream end of the HEC-RAS model (Figure 2.4) was simulated to be the same as in under the mixed flow condition (Figure 2.3).

Due to uncertainties in the HEC-RAS model regarding the floodplain topography and the

[2] Coefficient of roughness

coefficient of roughness of the wooded area, the downstream end of the physical model was set at approximately 310 feet (at the prototype scale) downstream of the spillway. The model extended about 180 feet (at the prototype scale) upstream of the existing outlet structure into the reservoir, to accurately simulate the approach flow in the reservoir. The model extent is shown in Figure 2.5.

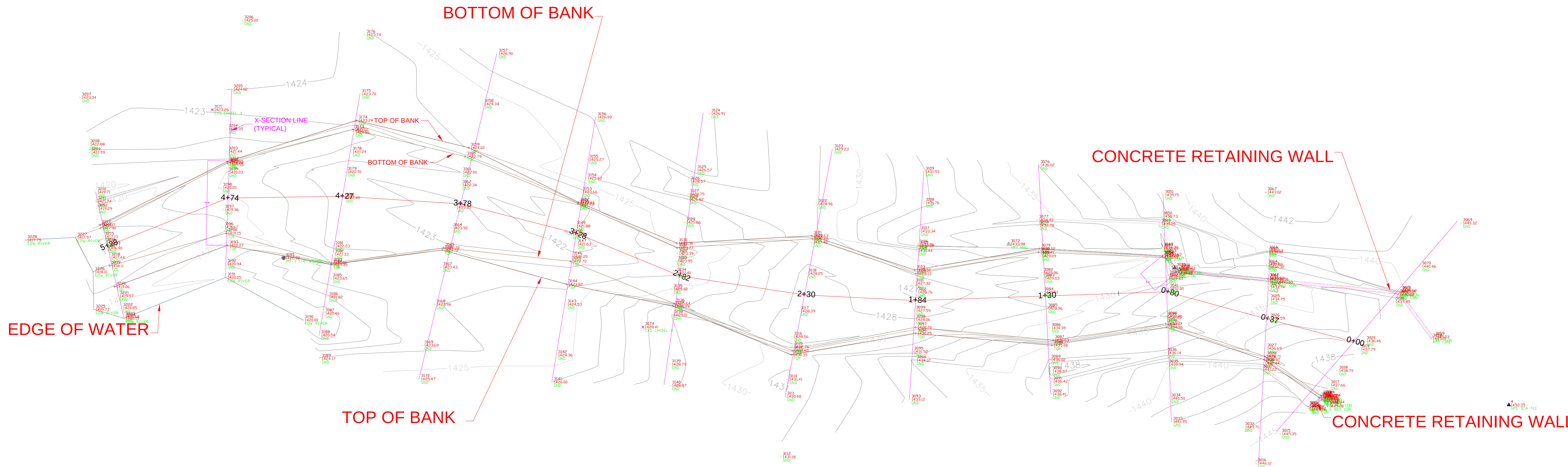



**Figure 2.1.** Surveyed cross-sections and the bathymetry along the stream downstream of the bridge.

**Figure 2.2.** A photo of the channel bed rock and the wooded area of the floodplain downstream of the retaining concrete wall, looking downstream.

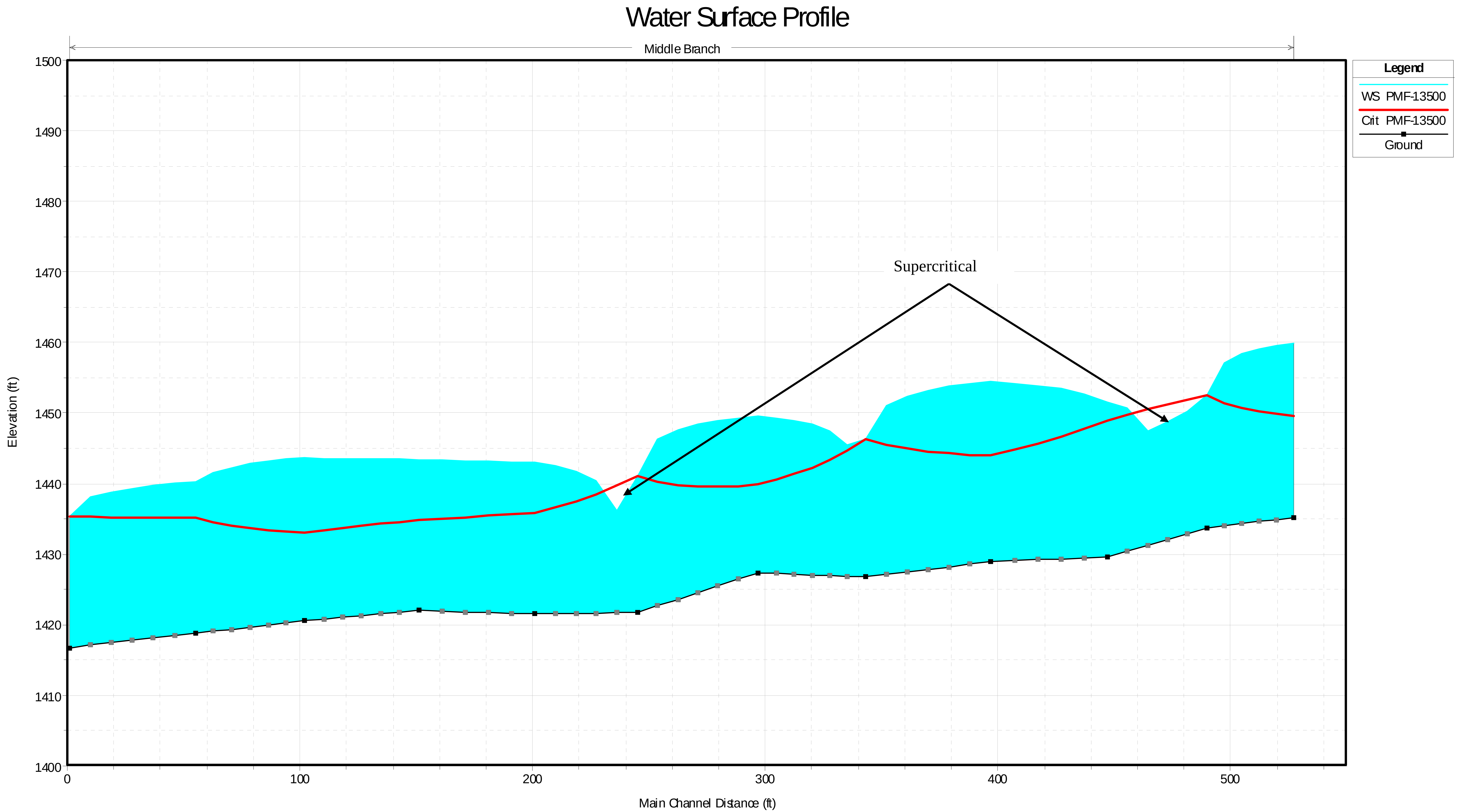



**Figure 2.3.** Water surface profile simulated along the channel downstream of the bridge using HEC-RAS assuming a mixed flow regime, i.e. allowing both subcritical and supercritical flow in the channel.

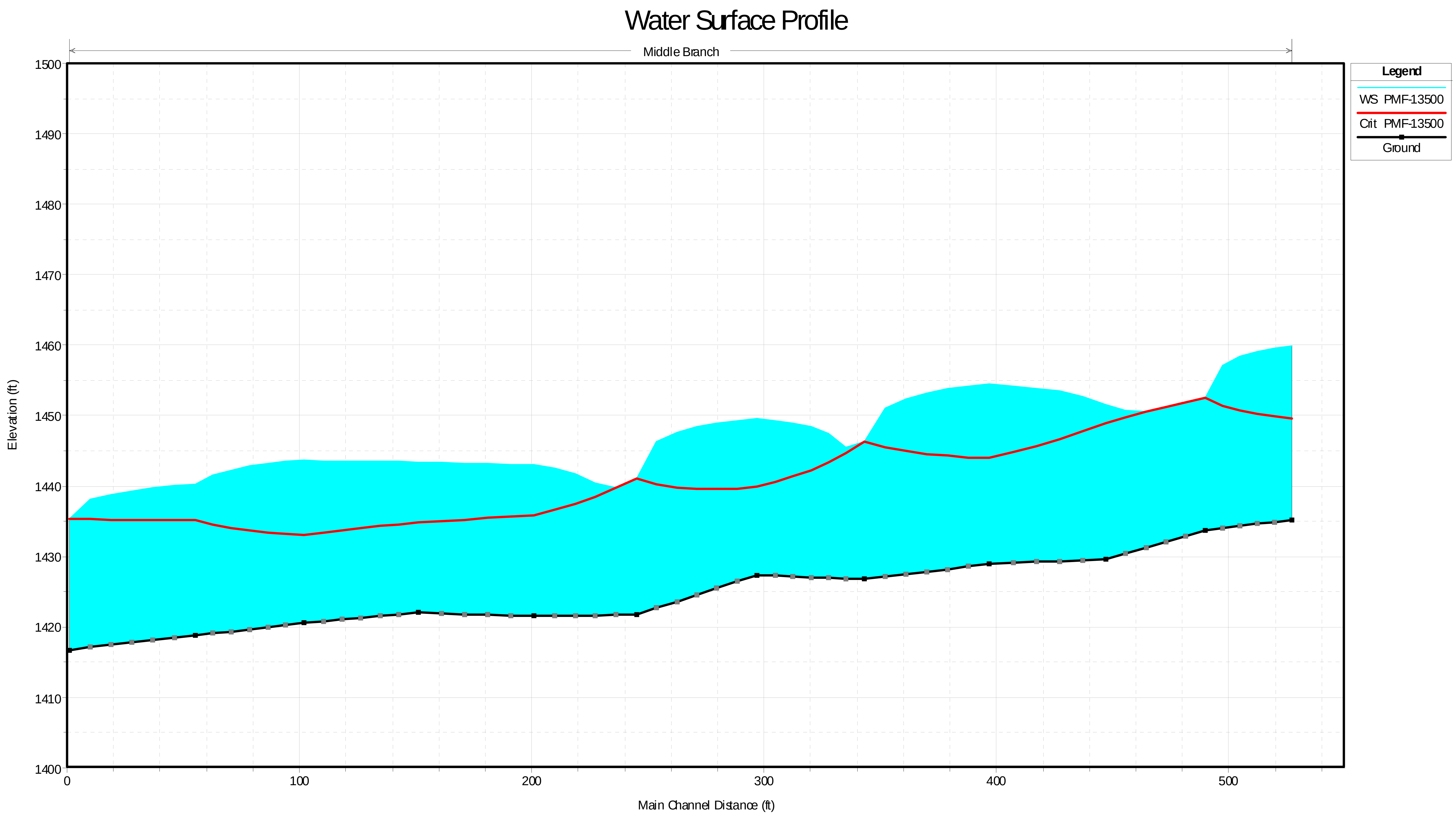



**Figure 2.4.** Water surface profile simulated along the channel downstream of the bridge using HEC-RAS assuming subcritical flow regime, i.e. not allowing supercritical flow in the channel.

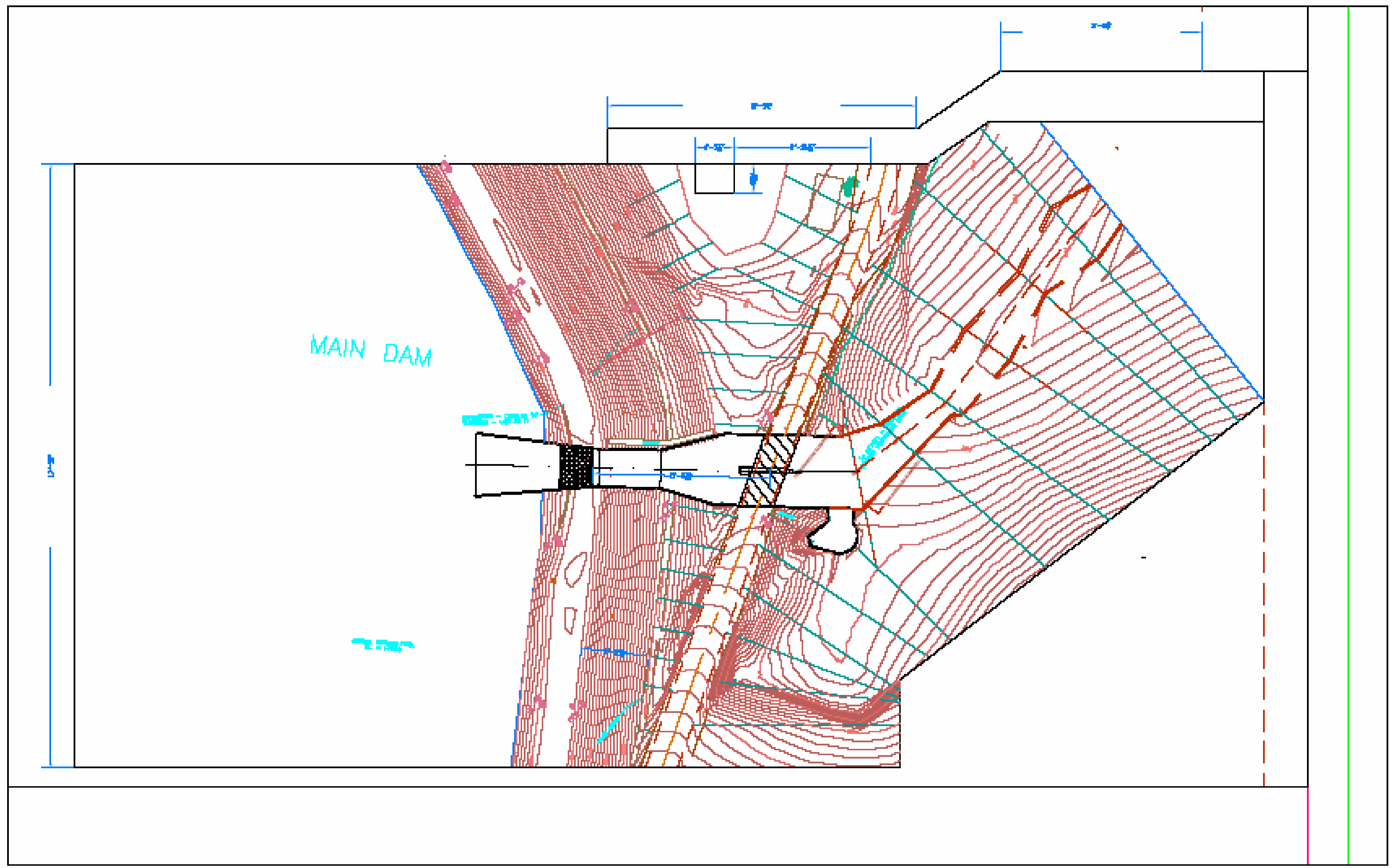



**Figure 2.5.** The layout of the physical model built on the Model Floor of the St. Anthony Falls Laboratory. The blue lines show the locations of templates for the construction of the floodplain topography of the area at the toe of the embankment.

### *2.3. Model Construction and Materials*

A head tank was constructed at the upstream end along the entire width of the model (Figure 2.6). Because the Bond Falls reservoir is very wide in comparison to its spillway structure, it is very likely that the spillway exhibits a uniform approach flow in the reservoir. To maintain a uniform approach flow, downstream of the head tank a rock crib wall was built. The model reservoir only needed to be wide enough to accurately capture the flow patterns along the contraction upstream of the spillway structure. The model was built 18 ft wide and 32 ft long.

The model was constructed using three different types of materials. The structural components, reservoir, dam or embankment, roadway and the portion of the channel chute at the downstream end of the spillway were all built from wood (Figures 2.6, 2.7, 2.8 and 2.9). The rest of the model was built using topographic templates, packed with filler materials and finished with a thin layer of concrete (Figure 2.10). The spillway, gates, and the bridge deck and pier were constructed out of acrylic and sheet metal (Figures 2.8 and 2.11). The gate actuators were fabricated using ACME threads and nuts because the model gates required a high degree of adjustability for opening and closing of the gates. The entire structure was painted and sealed to create a water tight basin.

The construction accuracy of all woodwork was 1/8″ (about 3 inches for the prototype). The construction accuracy of the concrete work was 1/4″, and the accuracy of those elements built from acrylic and sheet metal, such as the spillways, gates, and bridge elements were held to 1/16″ (about 1.5 inches for the prototype).

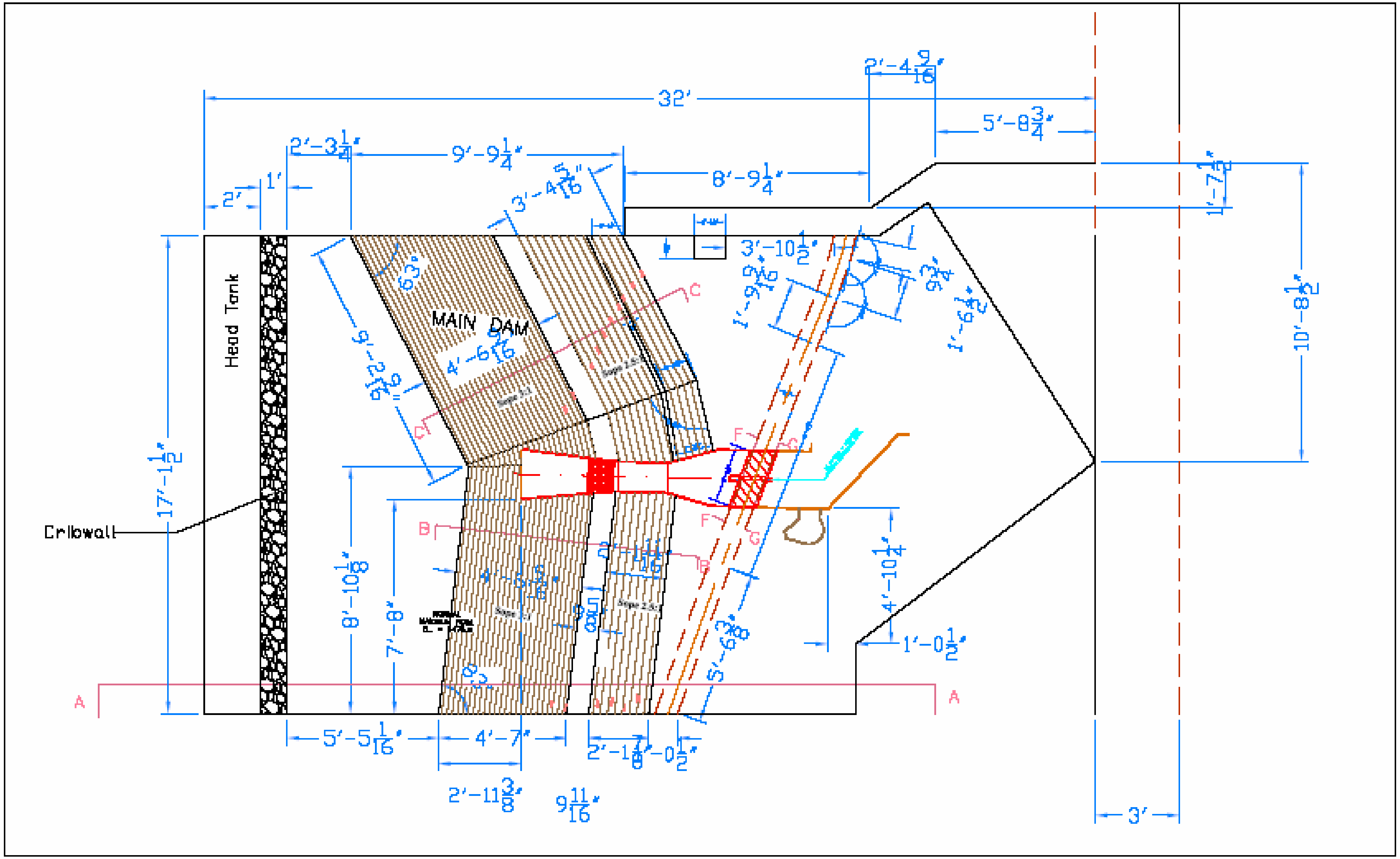



**Figure 2.6.** The layout of the physical model and the components which were built from lumber and plywood.

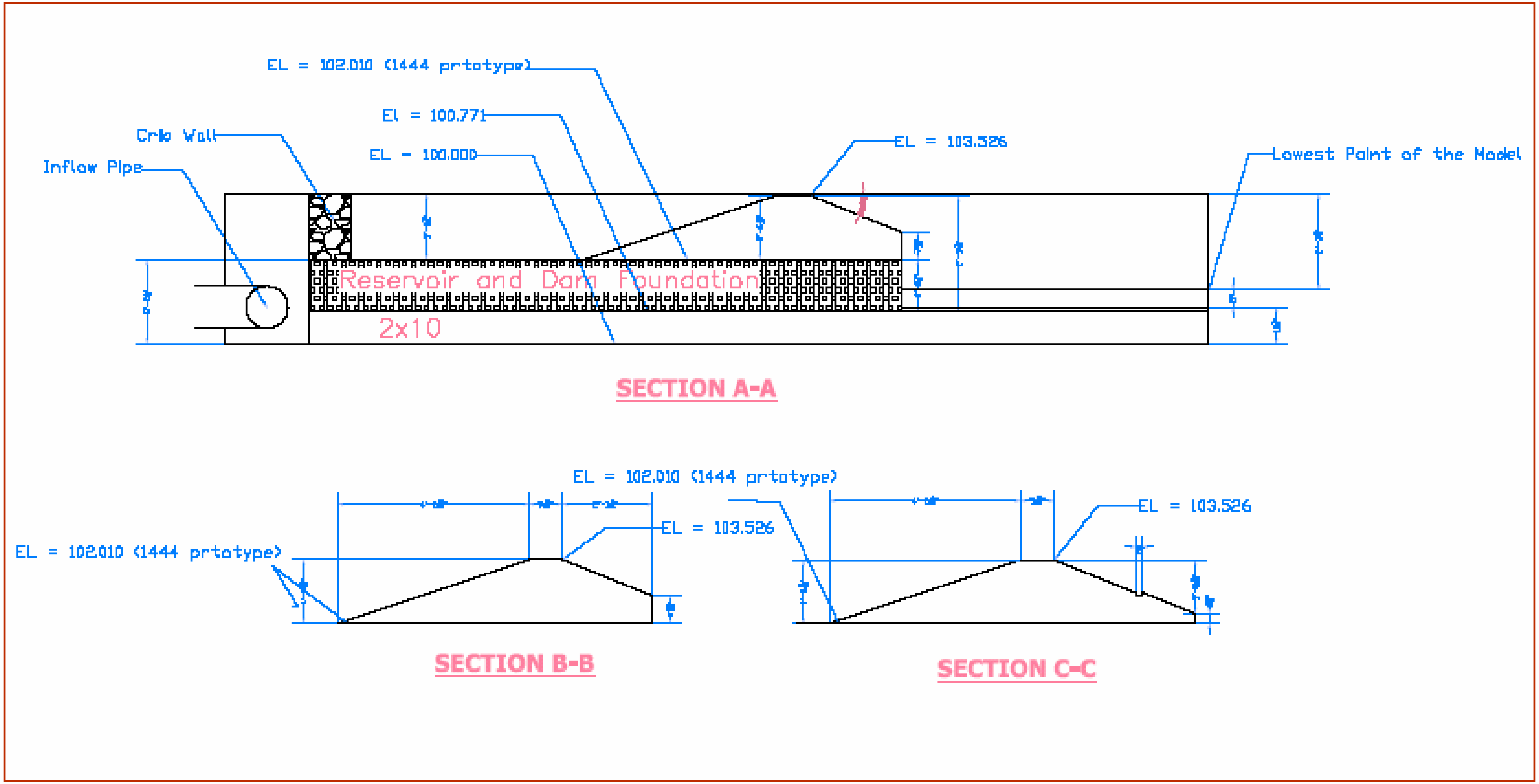
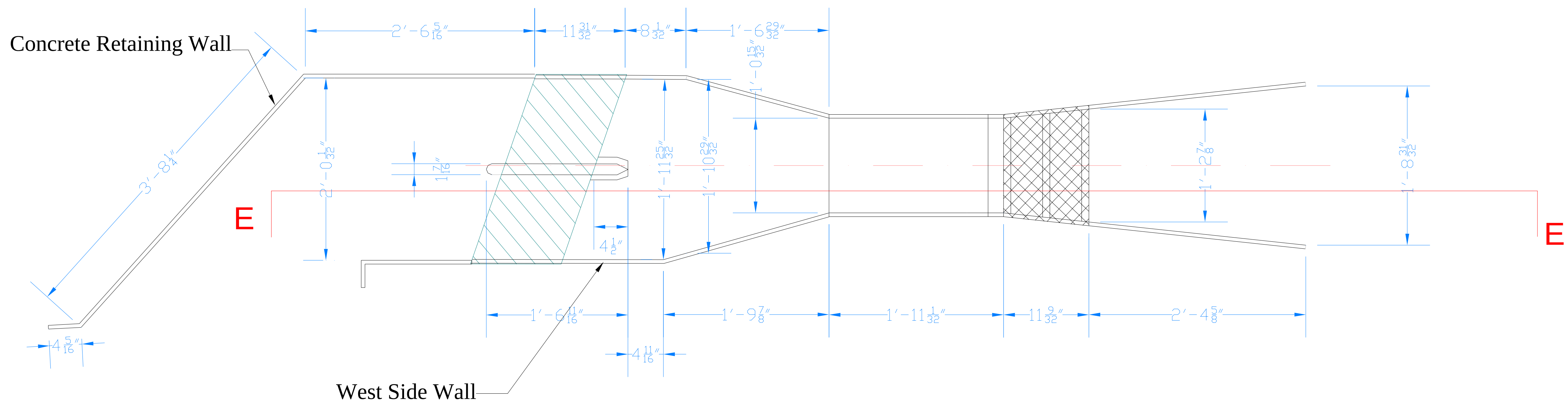
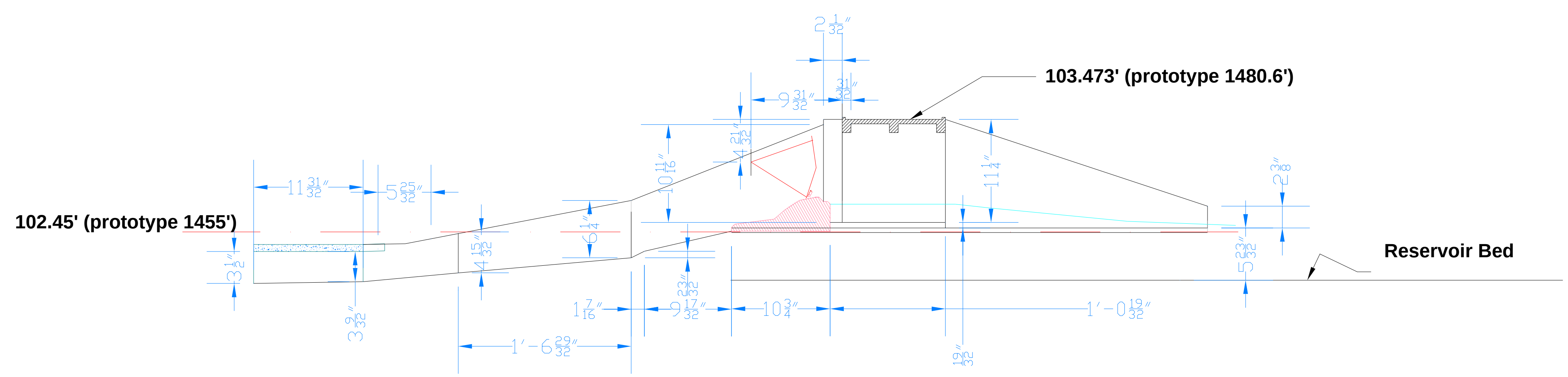



**Figure 2.7.** The cross-sections of the embankment, which was made of lumber and plywood.




## Plan View




## E-E

**Figure 2.8.** Plan view and longitudinal cross-section of the existing service spillway at 1:25 scale.


**Figure 2.9.** The physical model of the Bond Falls spillway system under construction. The embankment, road, topographic templates and structural elements were built from lumber and plywood.

**Figure 2.10.** The topography of the stream channel and its floodplain was built using formed concrete.

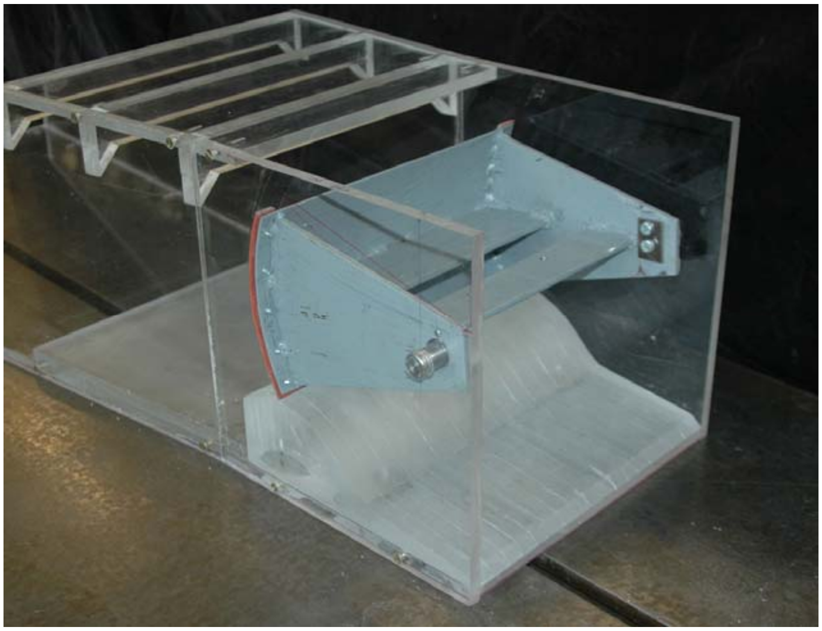


**Figure 2.11.** The original geometry of the existing spillway and the tainter gate which were made of acrylics and sheet metal, respectively.

### *2.4. Instrumentation*

To measure the discharge through the model, an orifice plate was installed in the influent plumbing of the model. Pressure taps were then installed at appropriate locations upstream and downstream of the orifice plate to accurately measure the pressure differential across the orifice plate. The taps were connected to a Rosemount pressure transducer via 1/16″ plastic tubes. The orifice flow meter was then calibrated against the SAFL weighing tanks to develop a calibration curve for the entire range of flow in this study (Figure 2.12). The SAFL weighing tanks have an accuracy of 0.2%.

Time averaged dynamic pressure at the nose and the side of the bridge pier, and on the west abutment of the bridge were measured using pressure taps connected to a second pressure transducer via 1/16″ plastic tubes. The position of pressure taps were surveyed to give the forces acting on the pier and nose of the bridge.

To determine the model pool elevation another pressure tap consisting of a quarter inch flush cut vinyl tube was placed near the floor in the reservoir and connected to a stilling well with a point gauge with a precision of 0.001 ft.

Flow velocities at the toe of the earthen dam to the west of the spillway were measured using Particle Imagery Velocimetry (PIV). Flow at the toe of the embankment was a sheet flow wherein no velocity meter could be used. For PIV, a high speed video camera (with a speed of sixty frames per second) was mounted on the ceiling and adjusted to capture the area of interest. A grid was initially placed over the area and photographed. The grid overlying the area is given in red in Figure 2.13. The calibration grid spacing was set at 6 inches at the model scale which corresponded to 12.5 feet for the prototype. Consequently, the xy-coordinates of any particle photographed in the area could be determined. To measure flow velocities, confetti was sprinkled near the west wall of the channel and videotaped for several minutes. Figure 2.14 shows two frames of one run over a section of the area. This process was repeated a number of times until enough information was collected over the entire region. Using the PIV software, the positions of all particles were estimated in each frame through a correlation analysis among all particles of two consecutive frames. The velocity vectors were then developed from the displacement of particles over time.

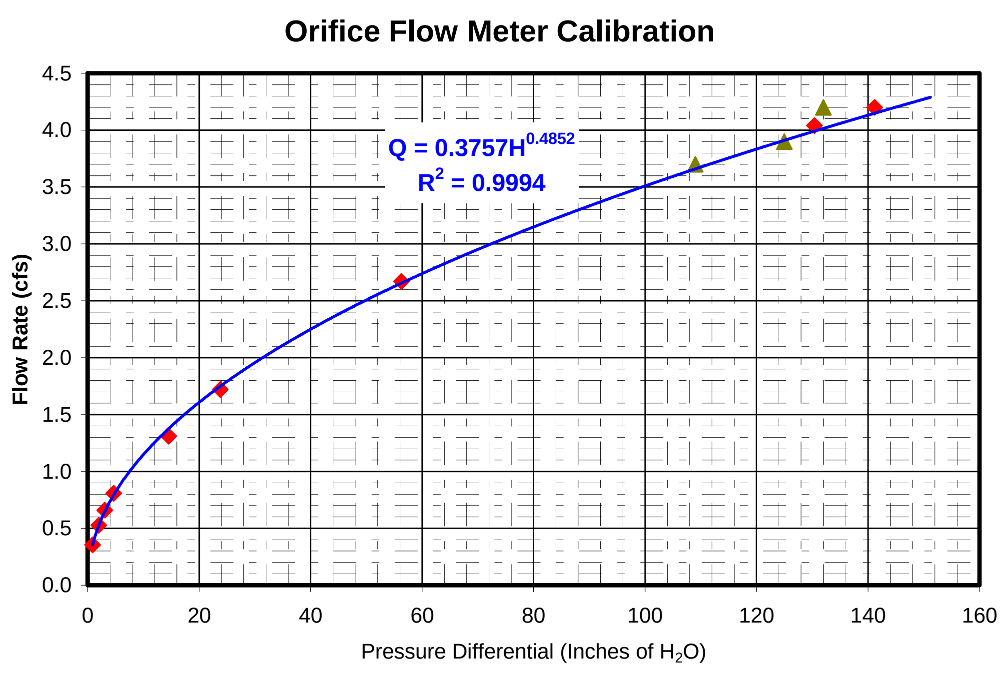



**Figure 2.12.** Measured flows versus measured pressure differentials across the orifice plate and the function fitted to the data for the orifice flow meter calibration. The red diamonds were the initial data collected for calibration, and the green triangles were collected later to verify the calibration function for high flows.

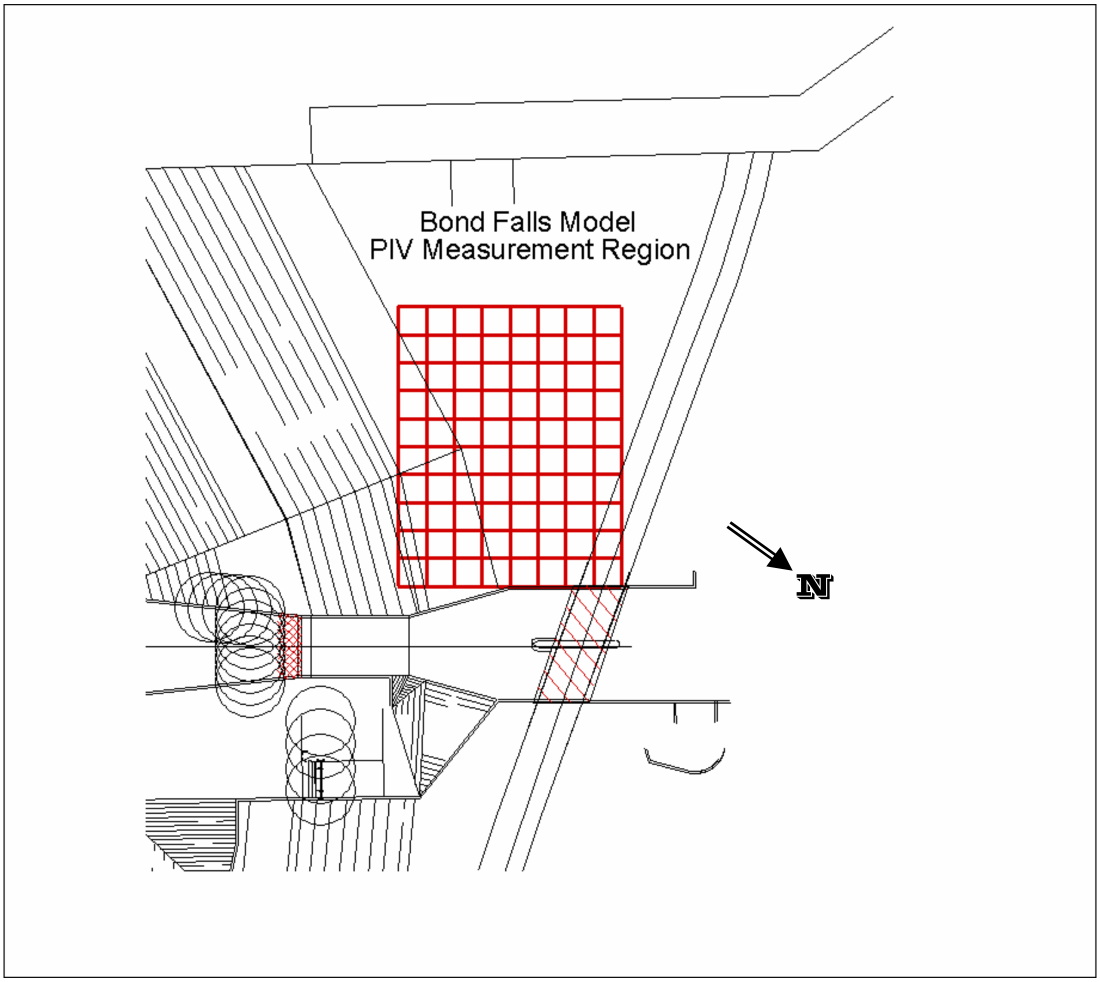



**Figure 2.13.** The grid and domain at the toe of the west embankment used for the PIV analysis.

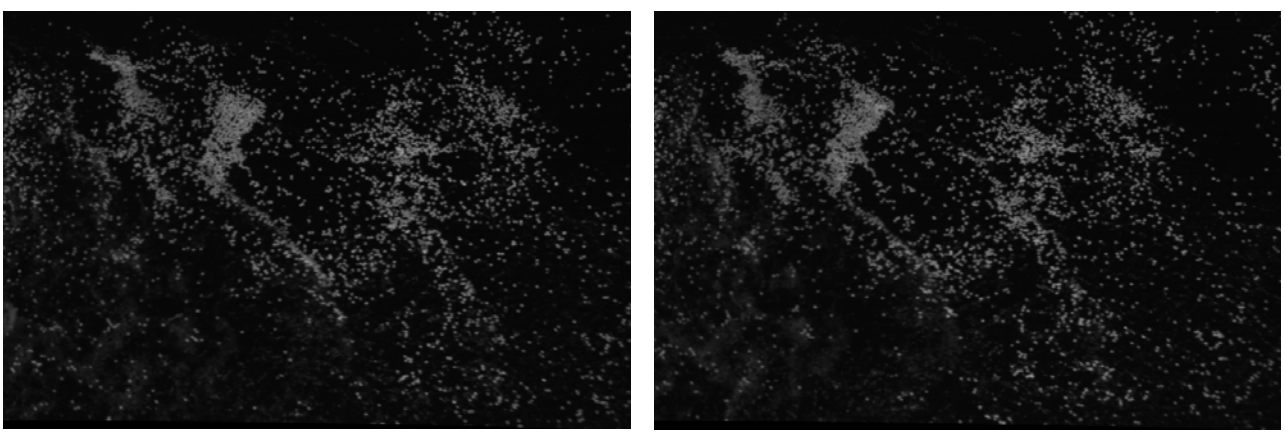


**Figure 2.14.** Two consecutive frames taken of the confetti flowing over the toe of the embankment for the PIV analysis.

# 3. Tests

## *3.1. Testing Overview*

Upon the completion of the model construction, the scaled drop in control structure and spillway, which was fabricated from acrylic and sheet metal (Figure 2.11), was inserted into the model. The model was then validated using the personal observations of the spillway operator as well as the photos of the flood event which occurred on April 19, 2002. The discharge was measured to be 1948 cfs.

After the model validation, the construction of the spillway Design-1 with a double gate system on the east of the existing spillway started. Again a pre-fabricated structure and spillway were retrofitted next to the existing model. The spillway Design-1 was modified when it was found that this configuration did not provide desirable flow conditions and velocities at the toe of the embankment.

The spillway system of the model was then modified to retrofit the spillway Design-2, which was a two-gate system in place of the existing single gate spillway.  The spillway Design-2 was also modified to improve the performance of the emergency spillway system to eliminate the possibility of any damages on the toe of embankment. In this section of the report, each spillway design and the subsequent modification are described and the test results are presented.

For each design or modification, the model was run for the 100-year flood, 500-year flood, the flood of record and the PMF (Table 3.1). For each test, the flow through the system was documented using a digital video camera. Velocities were measured using the PIV system under the PMF condition, i.e. when flow overtopped the west side wall (Figure 2.8) of the channel downstream of the spillway onto the toe of the embankment.

**Table 3.1.** The discharges associated with the major events at the Bond Falls Main Dam

| Flood Event | Discharge |
|---|---:|
| Flood of record (April 1951) | 2,300 cfs |
| 100-year flood | 2,050 cfs |
| 500-year flood | 3,250 cfs |
| Probable maximum flood (approved by FERC on May 16, 2005) | 13,500 cfs |

### 3.2. Initial Test Series: Existing Condition

Initial testing was conducted on the existing spillway (Figure A.1) for verifying the model for the flood of 2002 (Figures 3.1 and 3.2) and gauging the gate opening required for the flows given in Table 3.2.

To verify the model under the flood of 2002, artificial roughness was added to the channel simulating the natural roughness of bedrock (Figure 3.3). The model showed that under a discharge of 1948 cfs, the concrete retaining wall (Figure 2.8) downstream of the bridge would be overtopped (Figure 3.4). However, the white water shown in the photos taken from the flood of 2002 (Figures 3.1 and 3.2) could not be reproduced in the model testing. The white water is due to air entrainment which is surface tension dependent and could not be modeled in a 1:25 scale model.

During the model visit by the representatives from the UPPCO and the Wisconsin Public Service Corporation, two problems were identified with the model. One was the inaccuracy of the topography of the area to the east of the concrete retaining wall, which was obtained from the topographic map developed by the STS Consultants (Figure 1.1). The elevations seemed to be too low in comparison to the experience of the staff at the site, therefore, the topographic map was compared to the recent surveyed data from the area. Therefore, the topography of that area was modified to reflect the new surveyed data (Figure 3.5). Correcting the topography behind the retaining wall resulted in diverting the flow overtopping the retaining wall back into the main channel. With this additional flow back into the channel, tailwater did not increase and it was further reinforced that there was no tailwater effect. Nevertheless, the floodplain of the stream reach downstream of the bridge was covered with layers of chicken wire to model the roughness of the wooded area (Figure 3.5). This significant change in the roughness of the floodplain pushed more water back into the main channel of the stream and increased the water depth. However, the flow regime stayed supercritical along certain stretches and did not give any implications that the flow patterns upstream of the bridge could be affected by the tailwater levels in the stream, i.e. the extent of the model was selected correctly.

The other problem was the flow at the toe of the spillway, which rebounded off of the spillway and formed a “rooster tail”. This condition was never observed by the spillway operator at mid-range flows or higher (the operator had only observed this phenomenon at very low flows with the gate slightly open). By comparing the original geometry of the ogee spillway given in the construction drawings of the spillway to the surveyed data along the spillway collected by Ayres Associates, it was concluded that the spillway had not been built according to the original construction drawings and the model spillway geometry did not reflect the prototype; therefore, the spillway was rebuilt using the Ayres Associates surveyed data as shown in Figure 3.6. After modifying the spillway geometry, under mid-range flows or higher the rooster tail at the toe of the spillway disappeared.

Within the initial test series, the maximum capacity of the existing structure with a modified 10-ft gate opening was tested. It was found that the maximum discharge for the existing control structure was 5,800 cfs when the pool elevation was at 1480.9 ft (Table 3.2).

**Table 3.2.** Discharges, gate openings, and pool elevations during the initial test series

| Flood Condition | Tested Discharge (cfs) | Target Discharge (cfs) | Gate Opening (ft) | Pool Elevation (ft) |
|---|---|---|---|---|
| Observed flow on 4/19/2002 | 1,938 | 1,948 | 3.25 | 1475.9 |
| 100-year Flood | 2,062 | 2,050 | 4.00 | 1475.9 |
| Flood of Record (April 1951) | 2,312 | 2,300 | 4.75 | 1475.9 |
| Maximum discharge with a fully open gate | 4,500 | 4,300 | 10.00 | 1475.9 |
| Maximum discharge through the bridge opening without overtopping the side walls | 5,812 | - | 10.00 | 1480.9 |

**Figure 3.1.** A photo of the flood of April 19, 2002 through the channel downstream of the spillway and the bridge.

**Figure 3.2.** A photo of the flood of April 19, 2002 near the concrete retaining wall downstream of the bridge.

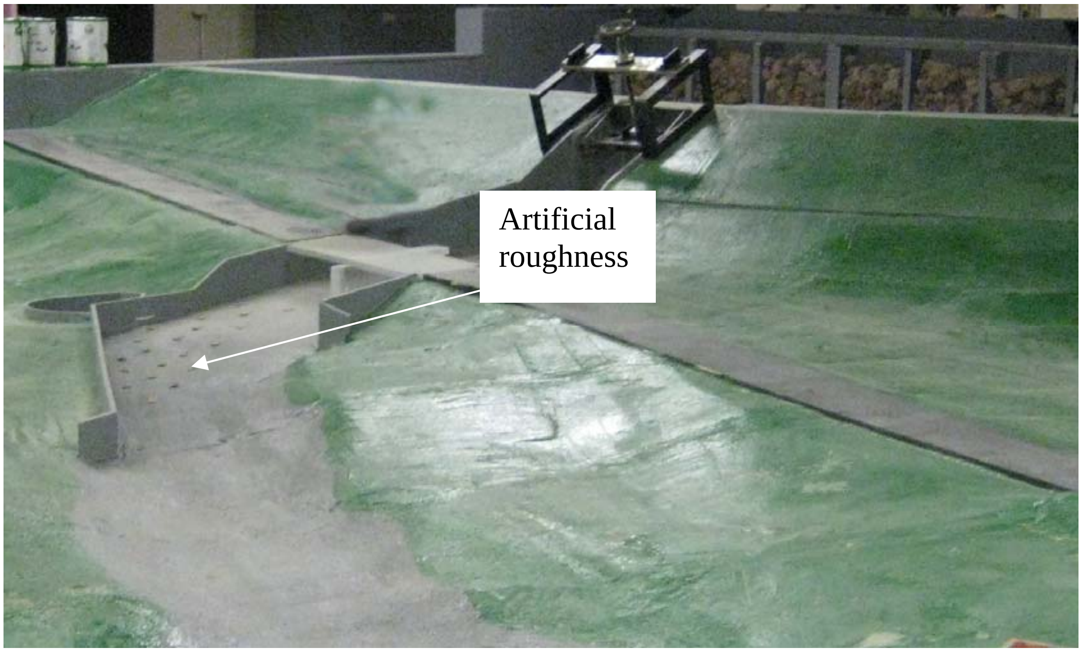



**Figure 3.3.** Artificial roughness added to the channel bed to model the roughness of the channel bed rock.

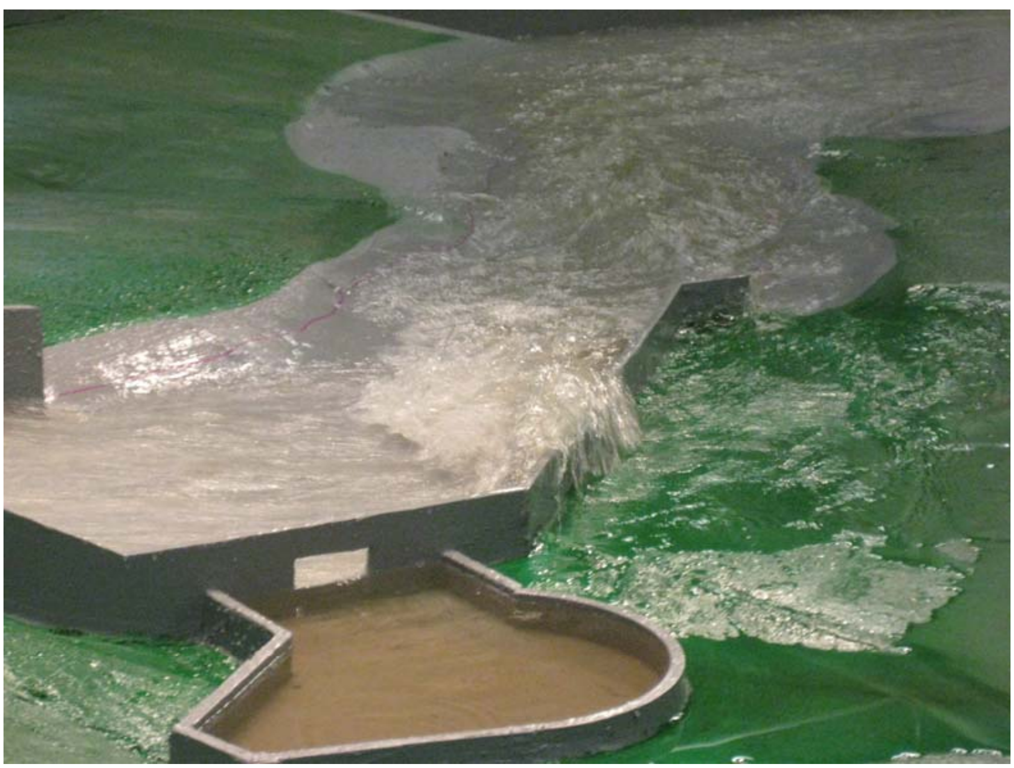


**Figure 3.4.** Flood of April 2002 (1,948 cfs) modeled in a 1:25 scale model at SAFL with water

overtopping the concrete retaining wall downstream of the bridge.

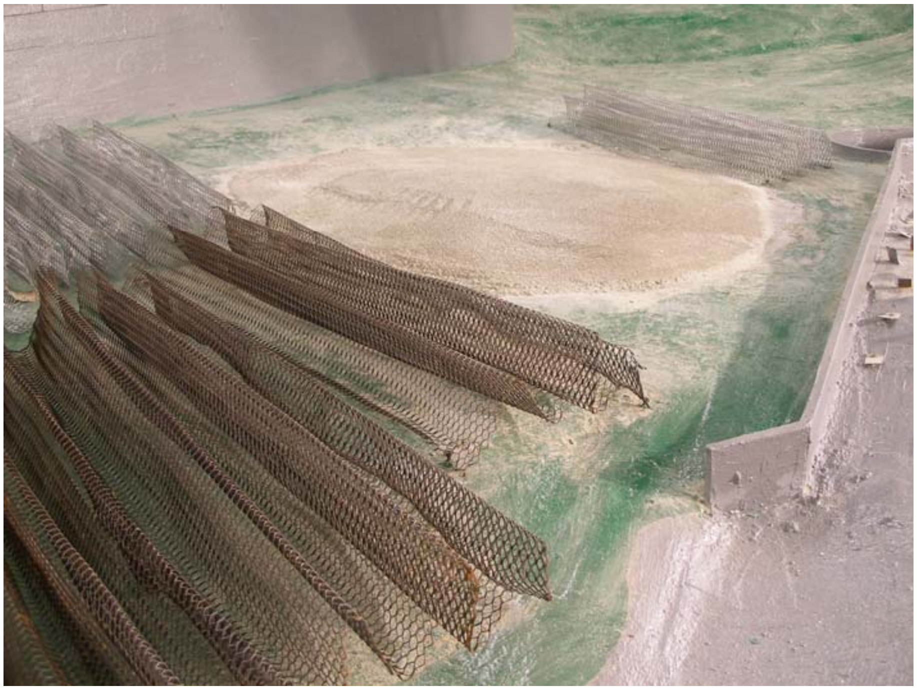


**Figure 3.5.** The elevation of the area behind the retaining wall was increased to more accurately reflect the topography of that area. In addition, chicken wire was added to the channel floodplain to model the roughness of the wooded areas in the prototype.

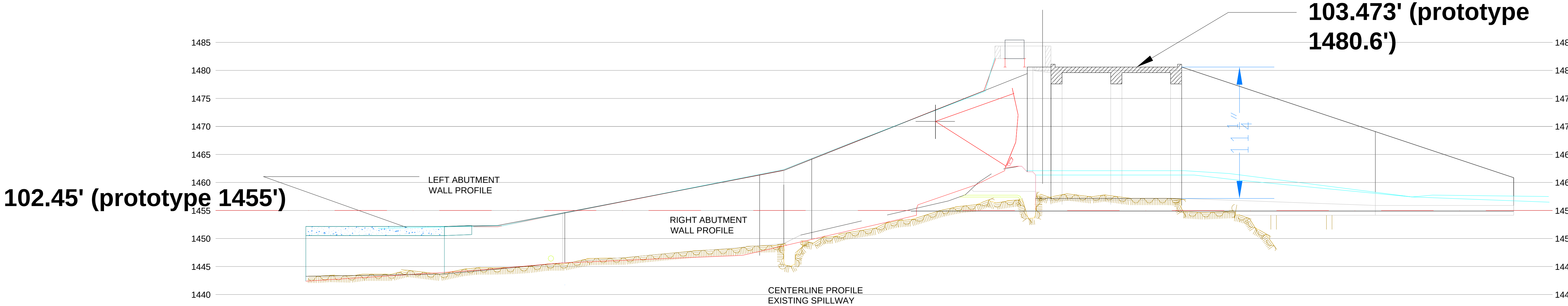



**Figure 3.6.** Longitudinal cross-section of the existing service spillway with the geometry obtained from the original construction drawings (dashed black line) and the geometry developed from the surveyed data (red line).

### *3.3. First Test Series: Spillway Design-1*

After the completion of the initial test series and the validation of the model, the construction of a second control structure cavity east of the existing spillway was commenced and the spillway Design-1 was fabricated and installed. The drawing provided by Ayres Associates (Figure B.1) and the model layout (Figures B.2 and B.3) are given in Appendix B. This control structure consisted of a double gate system with a central pier and a broad-crested weir. Each new gate was 16 feet wide (Figures B.1, 3.7 and 3.8). The retaining wall immediately downstream of the spillway Design-1 was set at a $50^o$ angle counterclockwise, which redirected the flow to the existing channel upstream of the bridge. Based on the computations done for this design, the key issues were the potential hydraulic jump next to the west side wall (Figure 2.8) upstream of the bridge, and the potential for erosion at the left or right embankment toe.

The tests were conducted for flows and gate opening scenarios listed in the Table 3.3. In order to identify the gates in this test series, the gates were referenced left to right looking downstream, i.e. gate 1 was the existing gate, gate 2 was the center gate (new), and gate 3 was the right gate (new). The model was used to evaluate the following scenarios proposed by Ayres Associates (Appendix C).

**Test A.** A discharge of 4,300 cfs with gate 1 fully open, gates 2 and 3 closed and the reservoir at the normal maximum pool level (1475.9'). This scenario represented the planned operation for opening the existing gate fully before opening the auxiliary gates (new).
Observations:

- Small spray over the west side of bridge.
- Flow overtopped the concrete retaining wall by approximately 6.25 ft.
- Bridge was not affected.

**Conclusions**:

- Under a discharge of 4,300 cfs, the outlet system works fine with only the existing gate fully open.

**Test B.** A discharge of 7,100 cfs with gate 1 fully open, gates 2 and 3 open by 6.5 feet and the reservoir at the normal maximum pool level (1475.9'). The purpose of this scenario was to determine whether it would be more favorable to open gates 2 and 3 equally vs. opening gate 2 fully before opening gate 3.
Observations:

- Flow overtopped the $50^o$ wall downstream of new gates by 6.25 ft.
- Bridge deck was overtopped.
- Water flowed west of the bridge over the toe of the embankment.
- To bring the pool elevation to 1475.9, gates 2 and 3 were opened by 5.5 ft.

**Conclusions**:
- Under Test B, flow will overtop the 50$^{o}$ wall downstream of new gates as well as the west side wall of the channel downstream of the existing spillway (Figure 2.8).
- Erosion will be possible at the toes of embankment.

**Test C.** A discharge of 6,500 cfs with gate 1 fully open, gate 2 fully open, gate 3 closed and the reservoir at the normal maximum pool level (1475.9'). The purpose of this scenario was to determine whether it would be more favorable opening gate 2 before opening gate 3 vs. opening gates 2 and 3 equally.
Observations:
- Water flowed west of bridge over the toe of the embankment.
- Flow overtopped the 50$^{o}$ wall downstream of new gates by 4.2 ft.
- Bridge deck was overtopped.

**Conclusions**:
- Under this scenario, the west side wall will be overtopped by two feet less in comparison to Test B, but flow overtopped both walls.
- Erosion was possible at the toes of embankment.

**Test D.** A discharge of 13,500 cfs i.e., the PMF, with all gates fully open and the reservoir at the maximum pool level (1480.9')
Observations:
- Flow overtopped the 50$^{o}$ wall downstream of new gates by 16 ft (Figure 3.9).
- Flow overtopped the west side wall upstream of the bridge by 8 ft.
- Water flowed west of the bridge over the toe of the embankment.
- Flow through all gates overtopped the gate pivot axels.
- Bridge was submerged.

**Conclusions**:
- The bridge and its pier will most likely be washed out.
- The roadway and the east embankment toe will be subject to severe erosion.
- Erosion is possible at the toe of the west embankment.

**Test E.** A discharge of 2,580 cfs with gate 1 open by 6.7 feet, gates 2 and 3 closed and the reservoir at the normal maximum pool level (1475.9'). This scenario represented the 100-year flood.
Observations:
- Gate 1 was set open by 6.0 ft rather than 6.7 ft to set the reservoir at the normal maximum pool level.
- Bridge was not affected

**Conclusions**:
- The 100-year flood will pass through the system with no problem.

**Test F.** A discharge of 3,250 cfs with gate 1 open by 8.9 feet, gates 2 and 3 closed and the reservoir at the normal maximum pool level (1475.9'). This scenario represented the 500-year.
Observations:

- Gate 1 was set open by 8.0 ft rather than 8.9 ft to set the reservoir at the normal maximum pool level.
- Small spray upstream of the bridge.

**Conclusions**:

- The flood will pass through the system with no problem.

The PIV (particle imagery velocimetry) analysis was conducted under the PMF condition for the spillway Design-1. Two realizations from the PIV analysis are shown in Figure 3.10. Even though the roughness at the toe of the west embankment was not correctly modeled, it is evident that for the spillway Design-1 the velocities right downstream of the west side wall (Figure 2.8) would exceed 12 fps and could reach 18 fps. The flow velocities right downstream of the west side wall were independent of the surface roughness because of their proximity to the side wall. With a velocity more than 7 fps, severe erosion at the toe of the embankment is very likely, unless protective measures are considered at the toe.

**Table 3.3.** Flow scenarios for the first test series with the spillway Design-1

| Tests | Gate 1 Existing Left | Gate 2 (New) Center | Gate 3 (New) Right | Pool Elevation (ft) | Discharge Prototype |
|---|---|---|---|---|---|
| A | Open Full | Closed | Closed | 1475.9 | 4300 cfs 500-year |
| B | Open Full | Open 6.5 ft. | Open 6.5 ft. | 1475.9 | 7100 cfs |
| C | Open Full | Open Full | Closed | 1475.9 | 6500 cfs |
| D | Open Full | Open Full | Open Full | 1475.9 | 13500 cfs PMF |
| E | Open 6.7 ft. | Closed | Closed | 1475.9 | 2580 cfs |
| F | Open 8.9 ft. | Closed | Closed | 1475.9 | 3250 cfs |

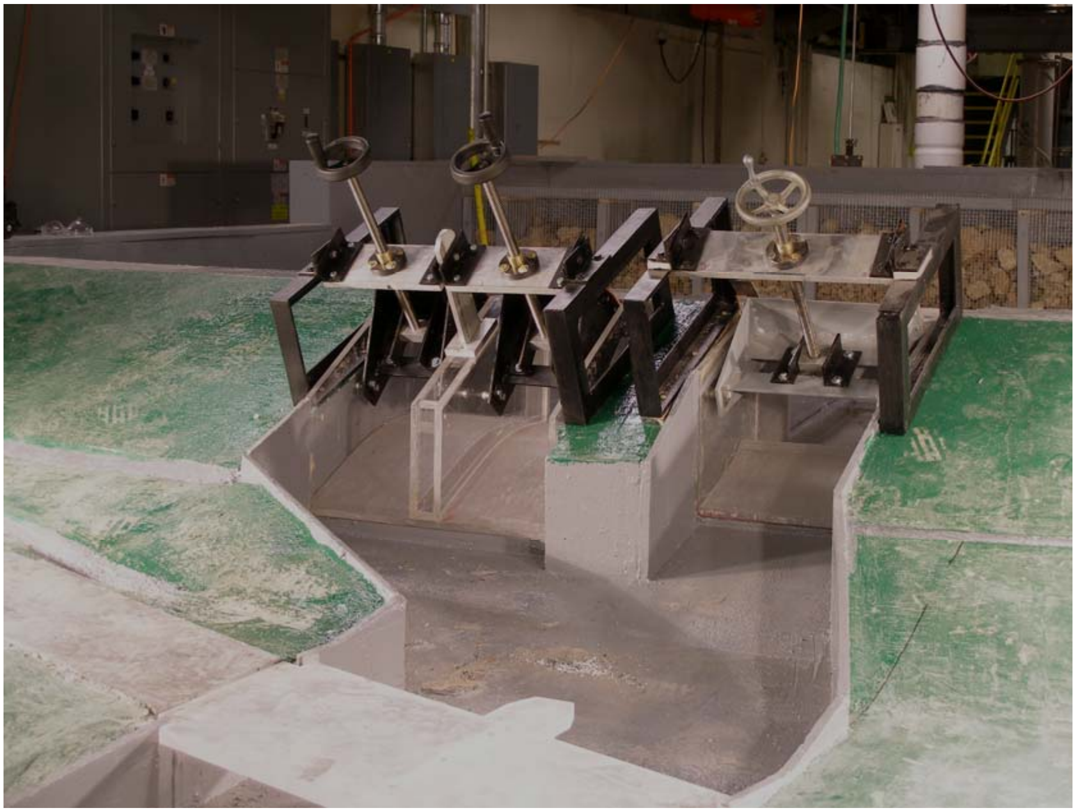


**Figure 3.7.** Downstream view of the spillway Design-1 which included two additional tainter gates to the east of the existing spillway.

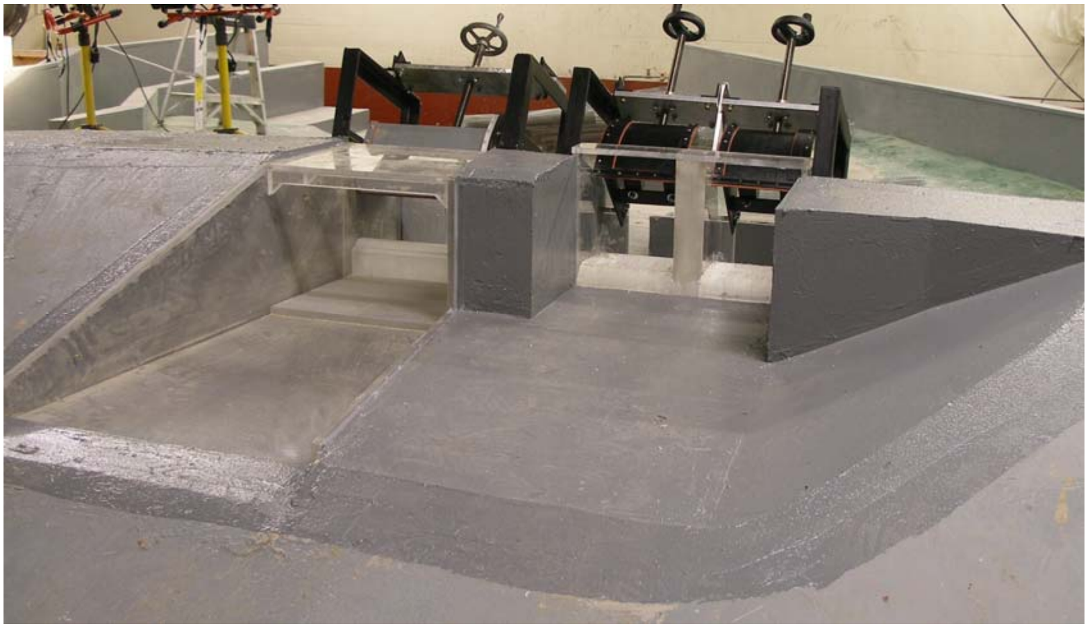


**Figure 3.8.** Upstream view of the spillway Design-1.

**Figure 3.9.** Water flowing through the spillway Design-1 under the PMF (flow). All three gates are fully open and the reservoir is at the maximum pool level (1480.9’). Under this condition, flow overtopped the $50^{o}$ wall downstream of the new gates (on the left side of the picture) by 16 ft and the west side wall upstream of the bridge (on the right side of the picture) by 8 ft.

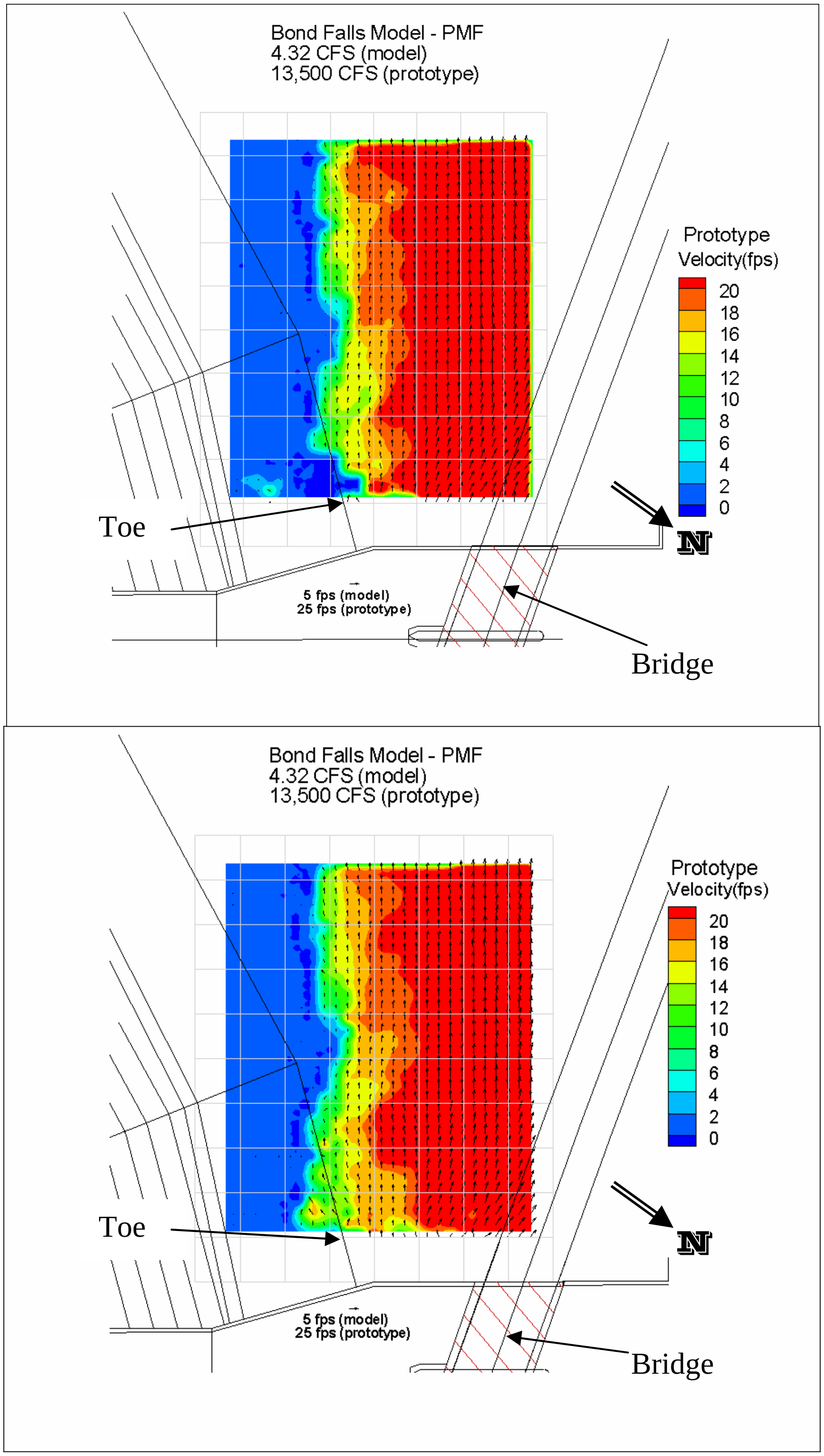



**Figure 3.10.** Measured velocities and velocity vectors of two realizations at the toe of the west embankment under the PMF (13,500 cfs).

## *3.4. Second Test Series: Spillway Design-1*

The second test series was conducted to determine the performance of the two new gates and the entire system overall as well as the capacity of the new design if gate 1 is not in operation. The dynamic pressure heads on the bridge nose are summarized in Table 3.4.

**Test A.** A discharge of 2,410 cfs with gate 1 closed, gate 2 open, gate 3 closed and the reservoir at the normal maximum pool level (1475.9')

Observations:

- Flow overtopped the $50^o$ wall downstream of new gates by 5.0 ft.
- Flow overtopped the west side wall by 1.25 ft.
- Flow overtopped the bridge by 1.25 ft.

**Conclusions:**

- Flow overtopped the bridge

**Test B.** A discharge of 4,770 cfs gate 1 closed, gate 2 open, gate 3 closed and the reservoir at the maximum pool level (1480.9')

Observations:

- Flow overtopped the $50^o$ wall downstream of new gates by 12.5 ft.
- Flow overtopped the west side wall by 2.5 ft.
- Flow overtopped the bridge by 2.5 ft.

**Conclusions:**

- With the discharge roughly doubled in comparison to Test A, the water height over the side walls and the bridge doubled.

**Test C.** A discharge of 2,410 cfs with gate 1 closed, gate 2 closed, gate 3 open and the reservoir at the normal maximum pool level (1475.9')

Observations:

- Flow overtopped the $50^o$ wall downstream of new gates by 10.0 ft.
- Flow overtopped the west side wall by 5.0 ft.
- Flow overtopped the bridge by 5.0 ft.

**Conclusions:**

- With the same discharge as in Test A but with gate 3 open, the water height over the side walls and the bridge deck increased twice as much or more.

**Test D.** A discharge of 4,640 cfs with gate 1 closed, gate 2 closed, gate 3 open and the reservoir at the maximum pool level (1480.9')

Observations:

- Flow overtopped the $50^o$ wall downstream of new gates by 12.5 ft.
- Flow overtopped the west side wall by 2.5 ft.
- Flow overtopped the bridge by 7.5 ft.

**Conclusions:**

- With the same gate configuration as in Test C and approximately a doubled flow, it appears that the flow over the $50^o$ side wall was slightly greater due to the increased flow

rate.
- The flow over the west channel wall was reduced due to a decrease in the diversion of the flow.

**Test E.** A discharge of 3,020 cfs with gate 1 closed, gates 2 and 3 open by 6.5 ft and the reservoir at the normal maximum pool level (1475.9').

Observations:
- Flow overtopped the 50° side wall downstream of new gates by 7.5 ft.
- Flow overtopped the west side wall by 5.0 ft.
- Flow overtopped the bridge by 2.5 ft.

**Conclusions:**
- With the bulk of the flow dispersed over the channel downstream of the new spillway and a decrease in the flow velocity, the momentum decreased, therefore the 50° side wall was overtopped less than in Test C. However, the bridge was overtopped more due to more diversion of the flow downstream of the new spillway.

**Test F.** A discharge of 4,700 cfs with gate 1 closed, gate 2 and 3 open by 6.5 ft and the reservoir at the maximum pool level (1480.9').

Observations:
- Flow overtopped the 50° wall downstream of new gates by 15.0 ft.
- Flow overtopped the west side wall by 5.0 ft.
- Flow overtopped the bridge by 2.5 ft.

**Conclusions:**
- With an increase in the flow due to a higher pool level elevation in comparison to Test E, only the 50° side wall was subject to more overtopping because the overtopping of the west side wall and the bridge stayed the same.

**Table 3.4.** Pressure heads on the pier nose during the second test series.

| Test | Gate 1 Condition | Gate 2 Condition | Gate 3 Condition | Pressure head on the bridge pier nose (ft) | Discharge (cfs) | Pool Elevation (ft) |
|---|---|---|---|---|---|---|
| A | Closed | Open | Closed | 2.3 | 2410 | 1475.9 |
| B | Closed | Open | Closed | 4.17 | 4772 | 1480.9 |
| C | Closed | Closed | Open | 1.5 | 2410 | 1475.9 |
| D | Closed | Closed | Open | 4.75 | 4640 | 1480.9 |
| E | Closed | 6.5 ft. open | 6.5 ft. open | 3.1 | 3019 | 1475.9 |
| F | Closed | 6.5 ft. open | 6.5 ft. open | 3.6 | 4693 | 1480.9 |

The results of the first test series showed that with the spillway Design-1 in place, as the discharge surpasses the 500-year flood (3,250 cfs) with one of the two new gates open, erosion is possible at the toes of the embankment. The flows that overtop the 50° side wall downstream of the new gates will impact the east embankment under higher flow conditions and will undermine the roadway and ultimately the embankment.

The second and first test series with the spillway Design-1 in place showed that as either gate 1 or 2 is open, the supercritical condition downstream of the new spillway and the $50^o$ side wall downstream of the new gates causes a standing wave along the $50^o$ side wall. Under higher flow conditions, the standing wave overtops that wall, which will impact the stability of the road and the east embankment. In addition, the standing wave enters the main channel downstream of the existing spillway with a velocity towards the west side wall and pushes water to overtop that wall and the bridge, and thus undermines the west embankment. Subsequently, there is a potential for erosion on the west toe of the embankment with the spillway Design-1 in place.

### *3.5. Third Test Series: Spillway Design-1 Modified-1*

During the site visit in late January, it was decided to modify the spillway Design-1 by moving the new control structure 21.5 feet back into the reservoir. Subsequently, the angle of the wall downstream of the two new gates became milder ($30^o$) and provided a gradual transition into the main channel. Additionally wall height was adjusted to contain the resulting standing wave inside the channel (Figures B.4 and B.5). The photos of the modified spillway tested are shown in Figures 3.11 and 3.12.

Further modifications were made to the model regarding instrumentation by installing one more pressure tap on the side of the bridge pier as well as two sets of taps (a total of four) on the west abutment of the bridge. The pressure taps on the abutment were installed at one third and two thirds of the bridge height as well as one third and two thirds of the bridge width (Figure 3.13).

The spillway Design-1 Modified-1 was only tested under the PMF. However, two scenarios were considered for testing: One with the bridge pier in-place and another with the bridge pier out. The second scenario was based on the possible future design of the bridge without any piers. For this test series, the pool elevation was set at 1480.9 ft, dynamic pressure heads were measured and a PIV analysis was conducted on the toe of the west embankment.

The results of this test series showed a great improvement along the $30^o$ wall downstream of the new gates (Figure B.5), i.e. no overtopping occurred over that wall. However, overtopping of the west side wall (Figure 2.8) did not disappear and the height of water over that wall under both scenarios was measured to be about 7.5 feet, which was not significantly different from the results of the first test series under the PMF. The dynamic pressure heads on the bridge pier and the west abutment are given in Tables 3.5 and 3.6. The dynamic pressure heads with the bridge pier in and bridge pier out were consistent because as the bridge pier was removed, the dynamic pressure on the west abutment increased. The results of the PIV analysis for both scenarios are given in Figures 3.14 to 3.15 for ensemble averages of two realizations. With the bridge pier in place, the flow velocities at the west embankment toe exceed 6 fps and at only 15 ft away from the toe, the flow velocities exceed 14 fps. With the bridge pier out, the flow velocities are slightly less. Nevertheless, the modified spillway showed an improvement.

**Table 3.5.** Dynamic pressure heads at the bridge pier and west abutment under the PMF for the spillway Design-1 Modified-1 with the bridge pier in place.

| **Position** | **Pressure head (Feet H20)** |
|---|---|
| **Pier nose** | 12.6 |
| **Pier side** | 20.4 |
| **West abutment, 1/3 upstream bottom** | 4.4 |
| **Abutment, 1/3 upstream top** | 4.6 |
| **Abutment, 1/3 downstream bottom** | 2.9 |
| **Abutment, 1/3 downstream top** | 2.7 |

**Table 3.6.** Dynamic pressure heads at the bridge pier and west abutment under the PMF for the spillway Design-1 Modified-1 with the bridge pier out.

| **Position** | **Pressure head (Feet H20)** |
|---|---|
| **Pier nose** | NA |
| **Pier side** | NA |
| **Abutment, 1/3 upstream bottom** | 9.38 |
| **Abutment, 1/3 upstream top** | 9.78 |
| **Abutment, 1/3 downstream bottom** | 7.08 |
| **Abutment, 1/3 downstream top** | 7.29 |

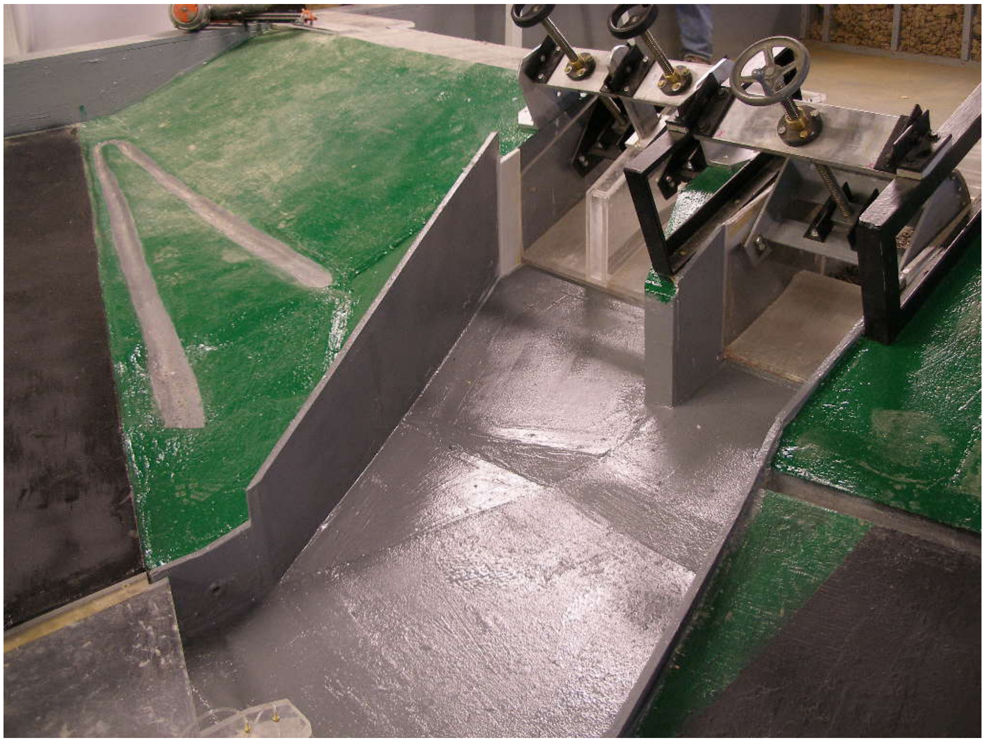


**Figure 3.11.** Downstream view of the spillway Design-1 Modified-1.

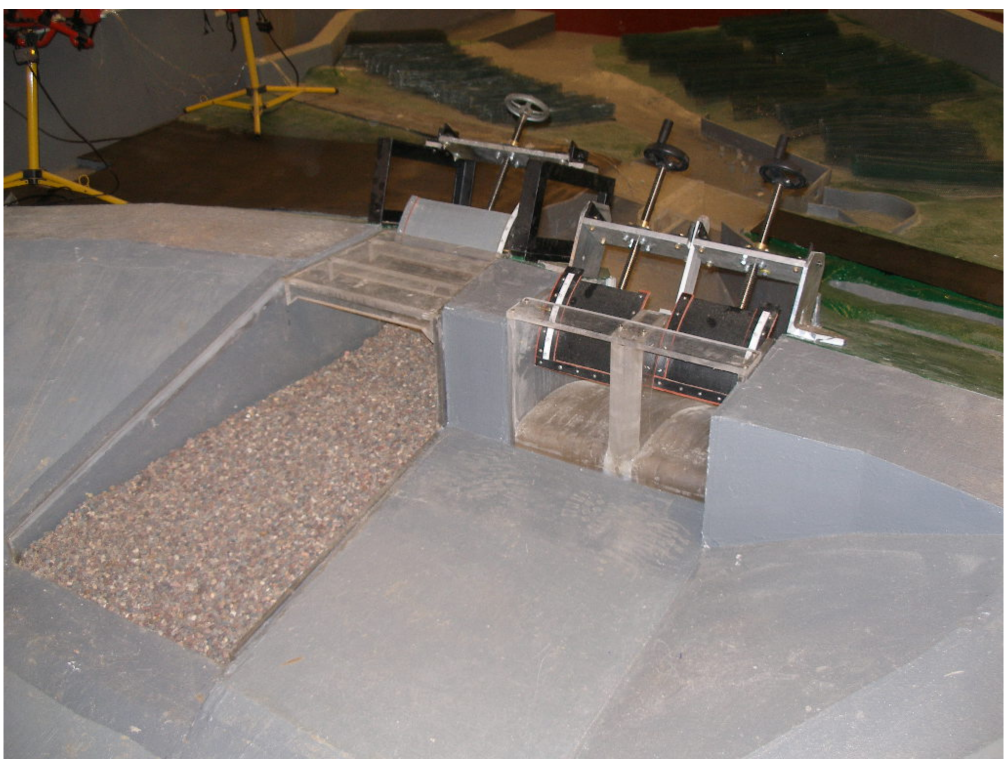


**Figure 3.12.** Upstream view of the spillway Design-1 Modified-1.

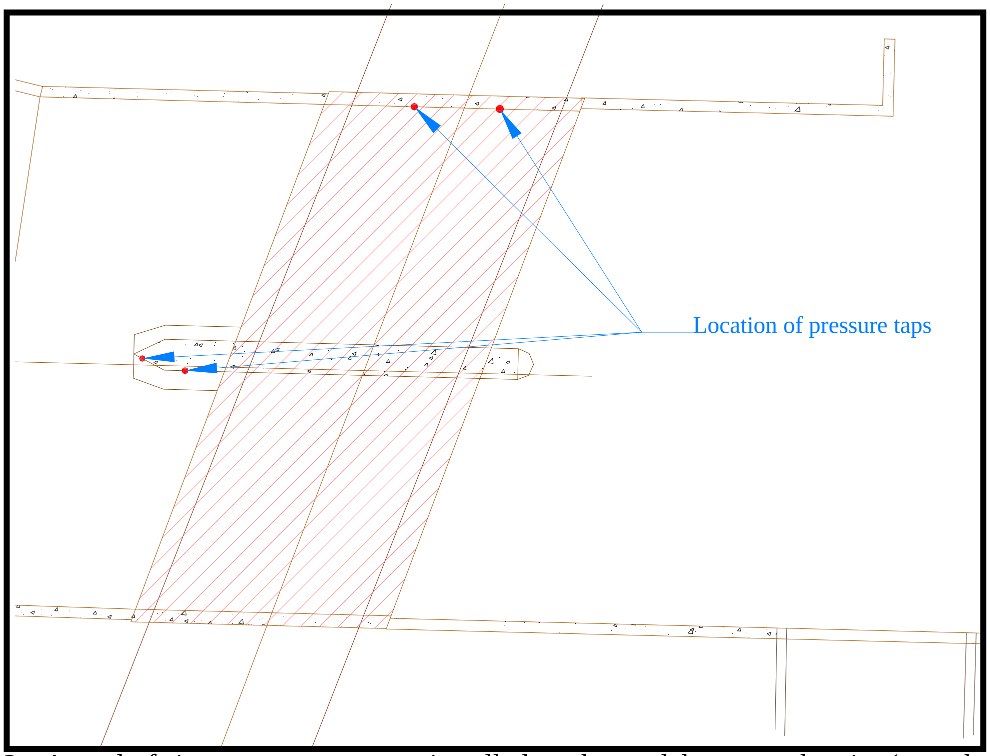



**Figure 3.13.** A total of six pressure taps were installed on the model, two on the pier (near the nose and on the side), and 4 on the west abutment of the bridge (at 1/3 and 2/3 of the width of the bridge at two different heights).

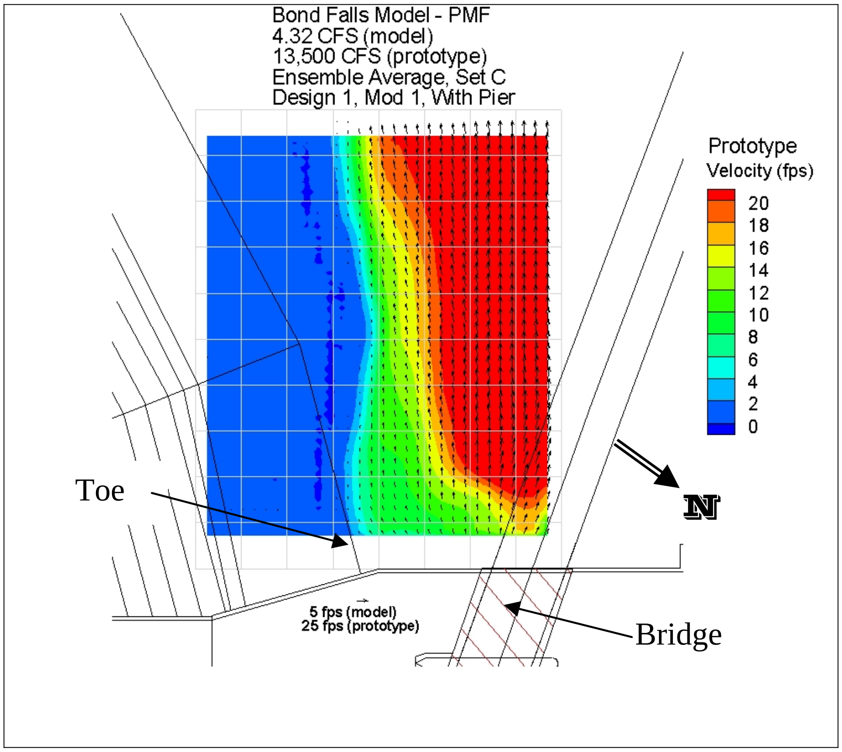



**Figure 3.14.** Measured velocities and velocity vectors of ensemble average of two realizations (Set C) at the toe of the west embankment under the PMF. Under this scenario, the bridge pier was in place.

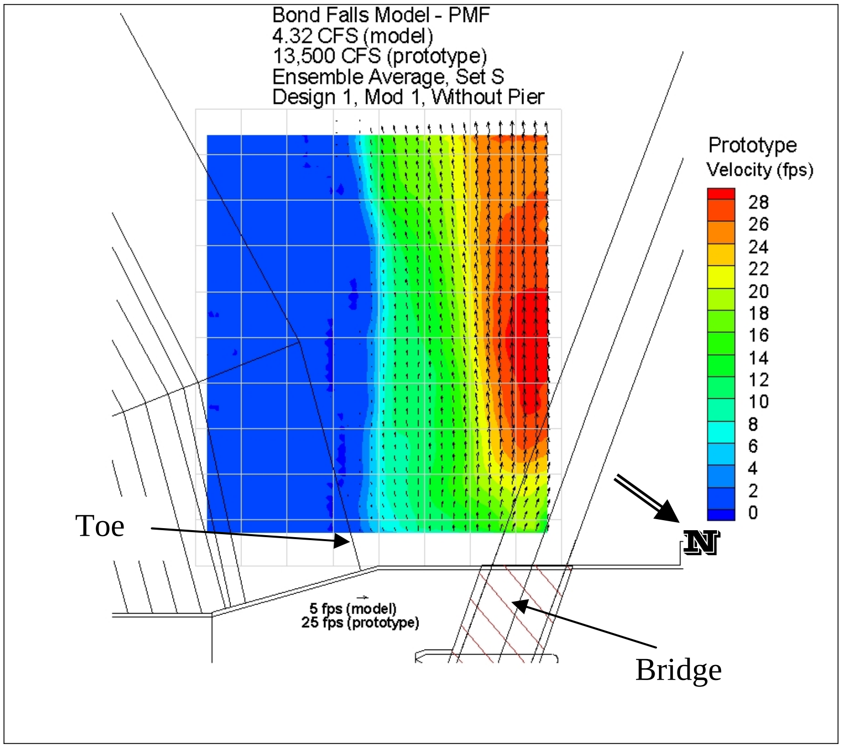



**Figure 3.15.** Measured velocities and velocity vectors of ensemble average of two realizations (Set S) at the toe of the west embankment under the PMF. Under this scenario, the bridge pier was out.

### *3.6. Fourth Test Series: Spillway Design-2*

After concluding that the spillway Design-1 and the subsequent modification would not satisfactorily pass the PMF, the construction of the spillway Design-2 began (Figure D.1). The model layout and profile of the spillway Design-2 is given in Appendix D (Figures D.2, and D.3). As shown in Figures 3.16 and 3.17, the spillway Design-2 was a double gated spillway replacing the existing structure. The gates were 25 feet wide with a central pier between them. At the toe of the spillway, the wall on the west side was notched by 10 feet into the west embankment. The wing wall then ran parallel to the channel for 24 feet and then tapered back to the existing channel side wall at the bridge. This notch was designed for the installation of a low level outlet system for maintaining the minimum flow requirements through the channel downstream. The wing wall on the east side of the spillway ran parallel to the channel and tied back into the existing channel wall with no taper.

Ayres Associates provided the matrix of the fourth test series for specific flow conditions, gate openings and pool levels (Appendix E). Dynamic pressure heads on the bridge pier and its west abutment (Figure 3.13) were measured, flow conditions through the channel and at the toe of embankment were documented using a digital video camera and a PIV analysis was conducted at the toe of the west embankment under the PMF. The test matrix and dynamic pressure heads resulting from this test series are given in Table 3.7. The dynamic pressure heads as expected were very small and did not seem to be an issue. The negative pressures in Table 3.7 are due to the separation of a high flow velocity from its boundary.

In the test matrix, the gates under Tests D and E were expected to be open by 3.1 and 3.9 ft, respectively. However, in order to maintain the reservoir at the maximum normal pool level and pass 2,580 and 3,250 cfs through the gates, the two gates opening had to be set at 2.5 and 3.5 ft, respectively (Table 3.7).

With the spillway Design-2 in place, under all flow conditions, water did not overtop the channel side walls upstream of the bridge or the bridge deck except under the PMF condition, i.e. Test C. However, overtopping the west side wall (Figure 2.8) under the PMF was relatively small in comparison to the spillway Design-1 and the Design-1 Modified-1 under the PMF. The PIV analysis of the flow over the toe of the west embankment under the PMF condition indicated very small flow velocities (Figures 3.18 and 3.19). The flow velocities at the toe did not exceed 2 fps and at 15 ft away from the toe the flow velocities did not exceed 6 fps. However, the flow velocities on the shoulder of the road exceeded 18 fps.

The reasons for overtopping the west side wall were the bridge pier itself and the merger of two standing waves developed at the toe of the spillway. The abrupt supercritical expansion at the toe of the spillway produced a standing wave diverging towards the walls, and, the notch on the west side of the channel caused another standing wave to develop along the west side wall. The two standing waves intersected each other at the upstream end of the bridge and caused a large rooster tail. In addition, the change in momentum at the bridge pier caused another large splash over the bridge deck which eventually flowed towards the west bank.

Therefore, Test C' was added to the test matrix (Table 3.7) to determine the potential for overtopping the side wall if a new bridge is designed with no pier. The difference in test results between Tests C and C' was not that significant. The dynamic pressures on the bridge pier nose and the abutment are good indicators of a small difference between the two conditions. Therefore, it was concluded that the splash against the bridge chord and the standing wave developed due to the left notch were probably playing more important roles in flow overtopping the west side wall than the bridge pier nose. To verify the effect of the bridge chord, the entire bridge was removed and the flow, gate and pool level under Test C were resumed in the model. With no bridge in, water flowed through the channel without overtopping the side walls except downstream of the bridge at the concrete retaining wall which was immaterial to the safety of the dam. By placing a board over the channel walls the standing wave hit the board and splashed over the west side wall (Figure 3.20).

Subsequently, it was decided to determine the minimum low chord elevation in the event a new bridge is built. To determine the minimum low chord elevation, the PMF condition was run through the model and the bridge deck (with no pier) was slowly shimmed up until the flow under the bridge was no longer touching the upstream low chord of the bridge. This elevation was found to be 1455.3 ± 0.5 ft. The existing bench mark on the downstream end over the bridge nose pier is 1451.86 and the low chord elevation as shown in Ayres drawings is at 1450.54. This corresponds to an elevation increase by 4.8 ± 0.5 ft to pass the PMF without overtopping the bridge deck or the west side wall. The ± 0.5 ft is the uncertainty due to the accuracy of the model construction. To determine this range more accurately, an uncertainty analysis should be conducted on all model parameters affecting the water depth underneath the bridge. Using the momentum equation for calculating the depth of the standing wave, and assuming a 3% error in measuring discharge in the model, the error in water depth for the prototype can be as high as 1 ft. Therefore, the elevation change to pass the PMF without overtopping the bridge deck will be 4.8 ± 1.5 ft. In this estimate, the bulking of the flow due to aeration is not considered.

**Table 3.7.** The matrix of the fourth test series conducted on the spillway Design-2 and the dynamic pressure heads recorded on the bridge pier nose and its west abutment. The locations of the pressure taps are shown in Figure 3.13. Left and right gates are referenced looking downstream.

| Test | Gate 1 Left | Gate 2 Right | Pier | Flow (cfs) | Pool Elev. | Pier Side (ft) | Pier Nose (ft) | Abut. U/S Low (ft) | Abut. U/S High (ft) | Abut. D/S Low (ft) | Abut. D/S High (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **A** | Open | Closed | In | 4,300 | 1475.9 | 6.3 | 9.6 | -0.6 | -0.9 | -1.0 | 1.6 |
| **B** | Closed | Open | In | 4,300 | 1475.9 | 1.6 | 1.7 | 1.6 | 1.6 | 1.5 | 1.3 |
| **C** | Open | Open | In | 13,500 | 1480.9 | 5.7 | 3.6 | 4.0 | 3.4 | 3.1 | 3.3 |
| **D** | 2.5' Open | 2.5' Open | In | 2,580 | 1475.9 | -0.6 | -1.5 | -2.4 | -2.6 | -2.5 | -2.7 |
| **E** | 3.5' Open | 3.5' Open | In | 3,250 | 1475.9 | 0.4 | 0.4 | -0.9 | -0.7 | -0.5 | -0.1 |
| **C’** | Open | Open | Out | 13,500 | 1480.9 | NA | NA | 3.35 | 3.17 | 2.92 | 3.08 |

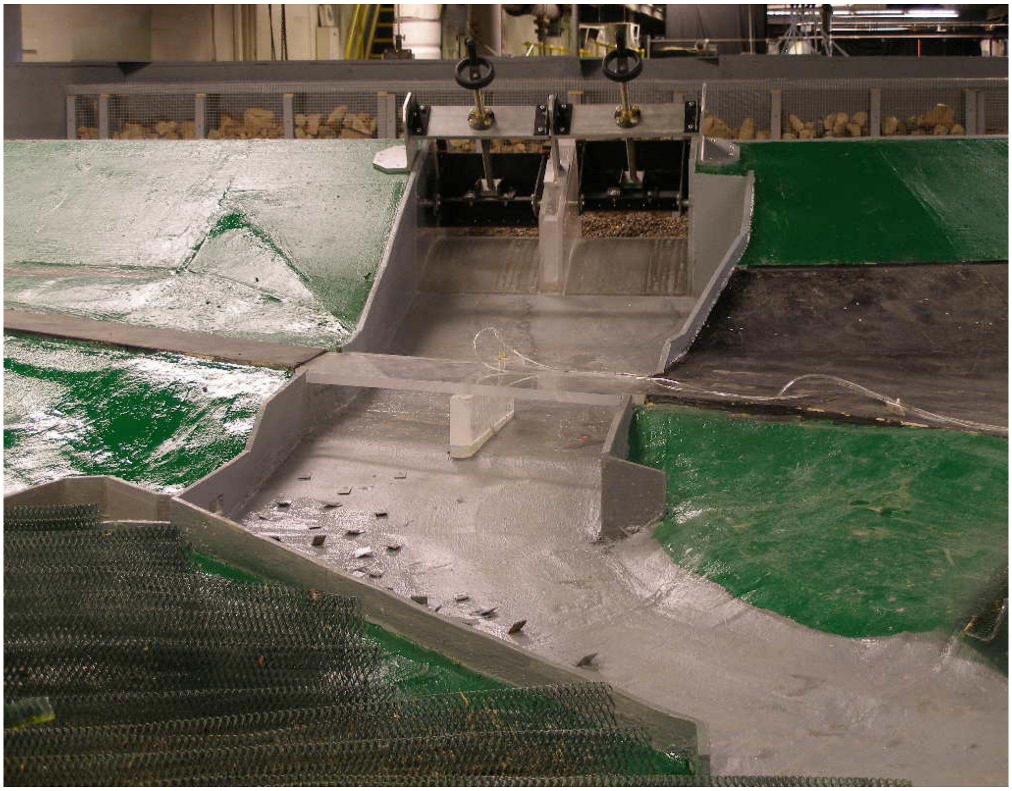


**Figure 3.16.** Downstream view of the spillway Design-2.


**Figure 3.17.** Upstream view of the spillway Design 2

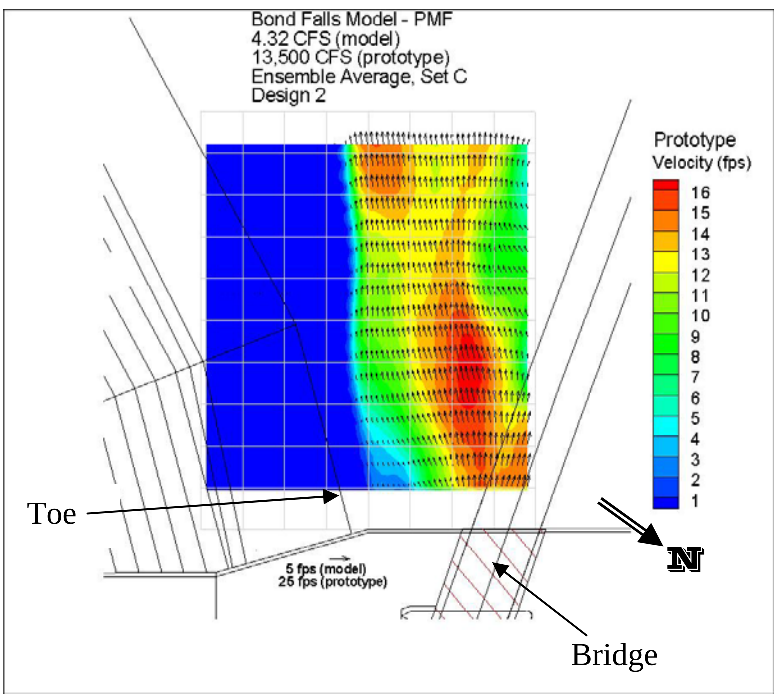



**Figure 3.18.** Measured velocities and velocity vectors of ensemble average of two realizations (Set C) at the toe of the west embankment under the PMF with the spillway Design-2 in place.

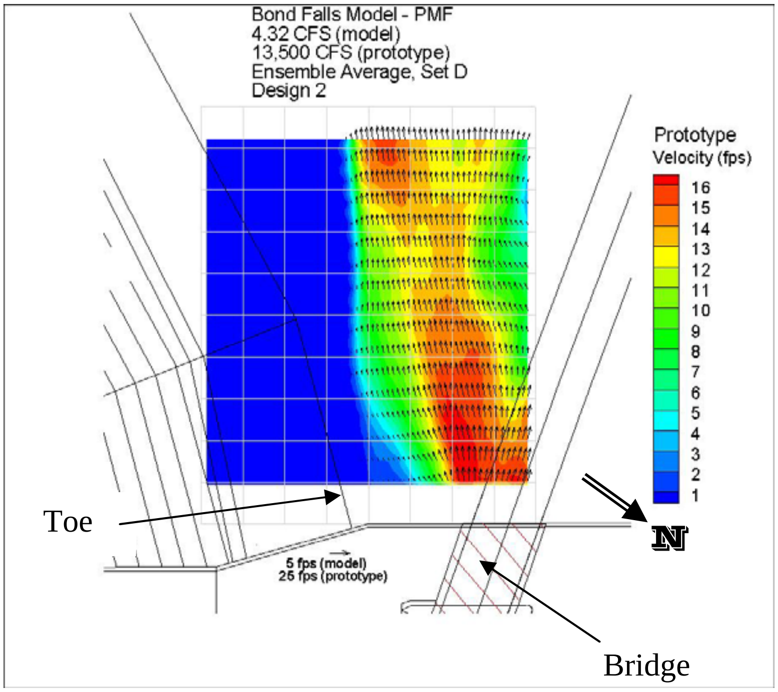



**Figure 3.19.** Measured velocities and velocity vectors of ensemble average of two realizations (Set D) at the toe of the west embankment under the PMF with the spillway Design-2 in place.

**Figure 3.20.** Under the spillway Design-2, but with no bridge in, water flowed through the channel without overtopping the side walls. By placing a plastic board over the channel walls the standing wave hit the board and splashed over the west side wall

### 3.7. Fifth Test Series: Spillway Design-2 Modified-1

In order to prevent the flow overtopping the west side wall of the channel upstream of the bridge, it was decided to eliminate the standing wave along the west side wall. Therefore, the spillway Design-2 was modified by removing the low level outlet system on the west wall and extending the spillway side wall to the wall upstream of the bridge (Figure 3.21).

The fifth test series was conducted with the spillway Design-2 Modified-1 in place. The fifth test series was conducted only under the PMF condition. However, three scenarios were tested: (1) the existing bridge in place, (2) the bridge pier out, and (3) the bridge deck and pier out. The dynamic pressure heads measured on the bridge pier nose and its west abutment are given in Table 3.8.

By removing the abrupt expansion on the west side of the channel, the standing wave along that wall disappeared. Subsequently, more water stayed in the middle of the channel and the pressure on the pier nose increased (Table 3.8). In addition, water splashed over the bridge deck due to a larger flow over the width of the bridge pier in comparison to the fourth test series. The PIV analysis of the flow over the toe of the embankment (Figures 3.22 and 3.23) showed that the flow velocities at the toe did not exceed 2 fps and 15 ft away from the toe, the flow velocities did not exceed 4 fps. On the shoulder of the road, the flow velocities exceeded 18 fps.

By removing the bridge pier, water did not touch the low chord except at the very west corner of the channel. The flow velocities at the toe of the west embankment and 15 ft away from the toe were practically the same as in the one with the bridge pier in (Figures 3.24 and 2.25). However, the velocities on the shoulder of the road did not exceed 10 fps, which was the result of a significantly smaller discharge over the bridge deck and the west side wall.

Similar to the fourth test series, it was decided to determine the minimum low chord elevation if a new bridge is built. To determine the minimum low chord elevation, the PMF condition was run through the model and the bridge deck with no pier was slowly shimmed up until the flow under the bridge was no longer touching the upstream low chord of the bridge. This elevation was found to be 1452.0 ft. This corresponds to an elevation increase by 1.5 ± 1.5 ft to pass the PMF without overtopping the bridge deck or the west side wall.

**Table 3.8.** The matrix of the fifth test series conducted on the spillway Design-2 Modified-1 and the dynamic pressure heads recorded on the bridge pier nose and its west abutment (Figure 3.13). Left and right gates are referenced looking downstream.

| Gate 1 Left | Gate 2 Right | Pier | Deck | Flow (cfs) | Pool Elev. | Pier Side (ft) | Pier Nose (ft) | Wall U/S Low (ft) | Wall U/S High (ft) | Wall D/S Low (ft) | Wall D/S High (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Open | Open | In | In | PMF | 1480.9 | 3.0 | 10.7 | 6.04 | 6.46 | 5.10 | 6.45 |
| Open | Open | Out | In | PMF | 1480.9 | NA | NA | 5.67 | 5.31 | 3.83 | 3.25 |
| Open | Open | Out | Out | PMF | 1480.9 | NA | NA | 5.21 | 4.79 | 4.38 | 4.63 |

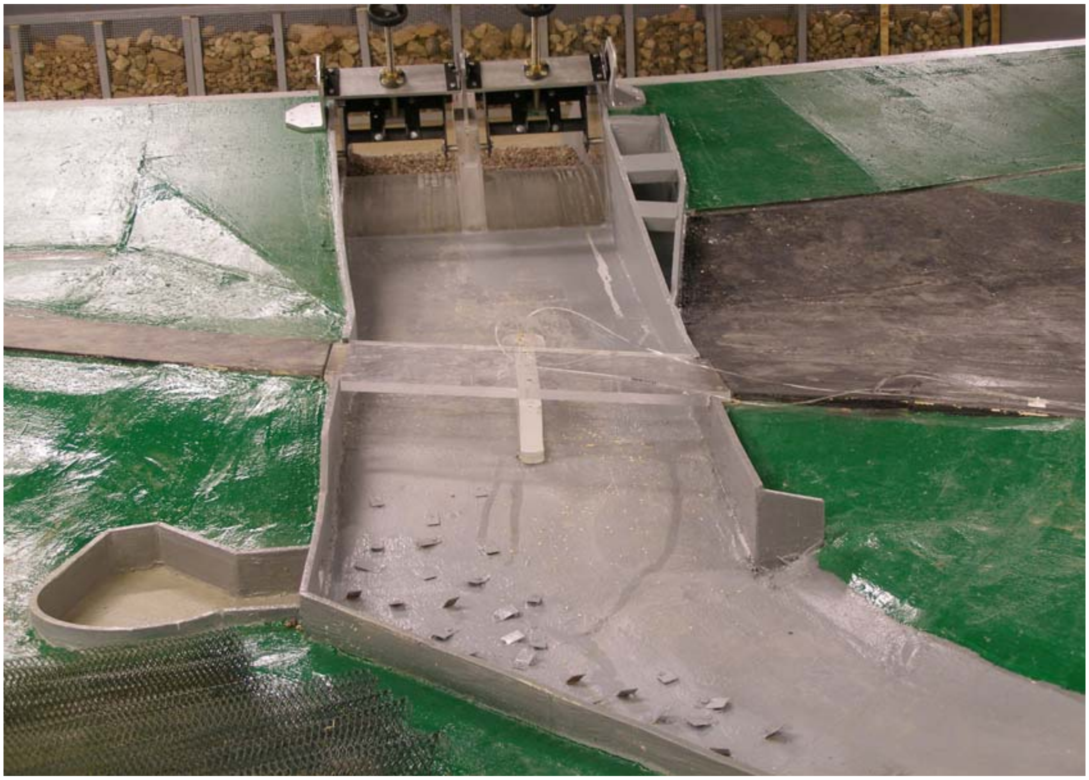


**Figure 3.21.** Downstream view of the spillway Design-2 Modified-1

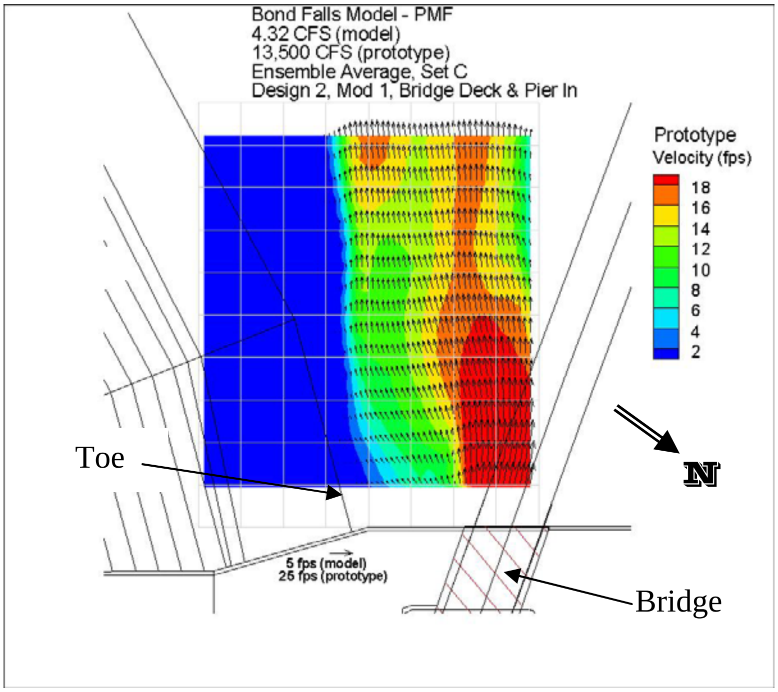



**Figure 3.22.** Measured velocities and velocity vectors of ensemble average of two realizations (Set C) at the toe of the west embankment under the PMF with the spillway Design-2 Modified-1 in place.

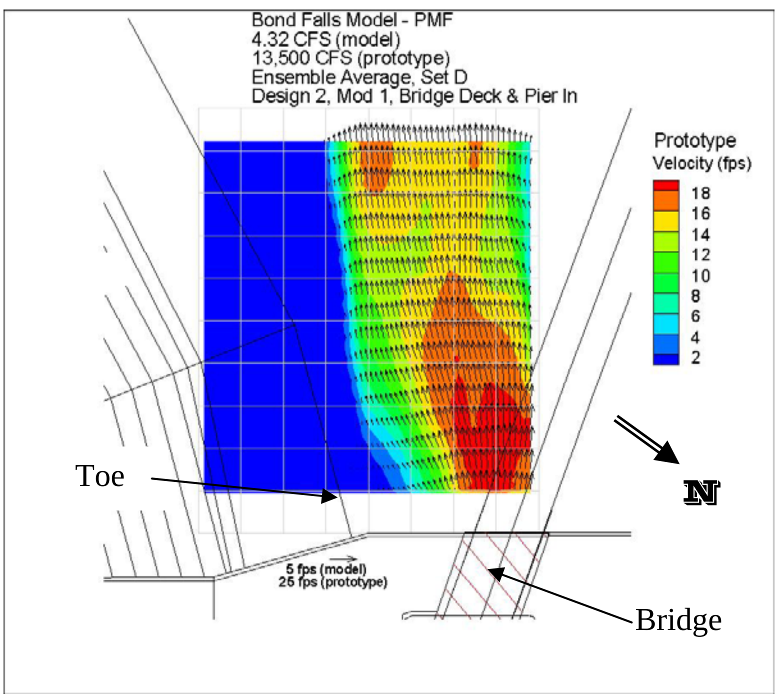



**Figure 3.23.** Measured velocities and velocity vectors of ensemble average of two realizations (Set D) at the toe of the west embankment under the PMF with the spillway Design-2 Modified-1 in place.

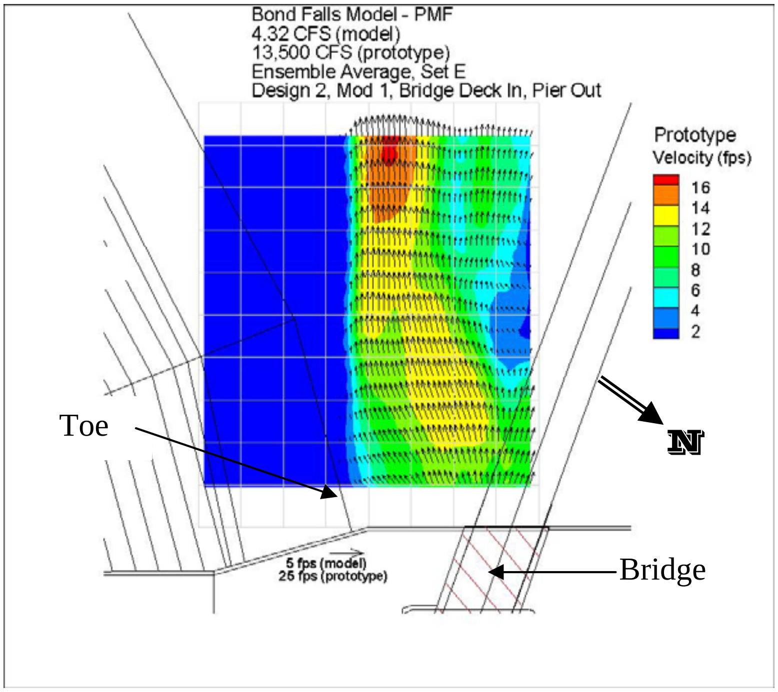



**Figure 3.24.** Measured velocities and velocity vectors of ensemble average of two realizations (Set E) at the toe of the west embankment under the PMF with the spillway Design-2 Modified-1 in place, without the bridge pier in.

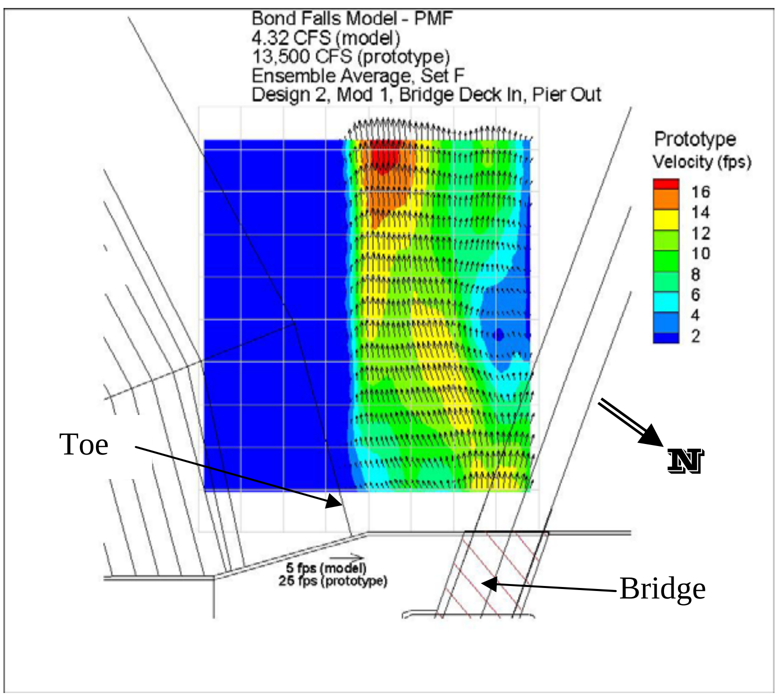



**Figure 3.25.** Measured velocities and velocity vectors of ensemble average of two realizations (Set F) at the toe of the west embankment under the PMF with the spillway Design-2 Modified-1 in place, without the bridge pier in.

## 4. Conclusion

A physical model study was conducted on the Bond Falls Spillway system. The scope of the study was to help with the design of a new spillway system which could pass the probable maximum flood (PMF) without undermining the embankment toe. The existing spillway has a maximum capacity of approximately 6,000 cfs. The PMF outflow from the reservoir is estimated to be 13,500 cfs.

A 1:25 scale model of part of the reservoir, the entire spillway system, the county bridge and about 300 ft of the stream downstream of the spillway was constructed at the St. Anthony Falls Laboratory. The model was slightly modified after the site visit to verify the geometry of the spillway and the topography of the floodplain more accurately. After verifying the model against the flood event on April 19, 2002, two spillway designs and two modifications to those designs were evaluated. A total of five test series were conducted on the new designs and their modifications.

The results of the spillway Design-1 which was comprised of a new broad-crested spillway equipped with two 16-ft wide tainter gates and a center pier, located to the east of the existing spillway showed that the standing wave generated along the wall downstream of the gates would force the flow over the west side wall which would ultimately undermine the safety of the embankment toe. The modification to the spillway Design-1 improved the flow conditions in the channel, but the flow velocities at the toe of the west embankment did not change significantly.

The results of the spillway Design-2, which was comprised of a new broad-crested spillway equipped with two 25-ft tainter gates and a center pier replacing the existing spillway showed that the PMF can be discharged through the system. However, the low elevation of the existing bridge low chord and its pier will splash enough water over the west side wall of the channel that it could erode the embankment toe. With the spillway Design-2 Modified-1, the bridge pier and deck have less of an adverse effect. If the existing bridge is replaced with a new bridge with no piers and with a low chord 3-ft higher, the embankment toe will not be undermined under the PMF condition.

Since it is not clear when a new bridge will be redesigned and rebuilt, it is recommended to protect the toe of the west embankment with riprap or other appropriate means against erosion. The velocity measurements conducted in this model study provide conservative estimates of the flow velocity for the design of an erosion control system at the embankment toe.

## Appendix A. Existing Condition

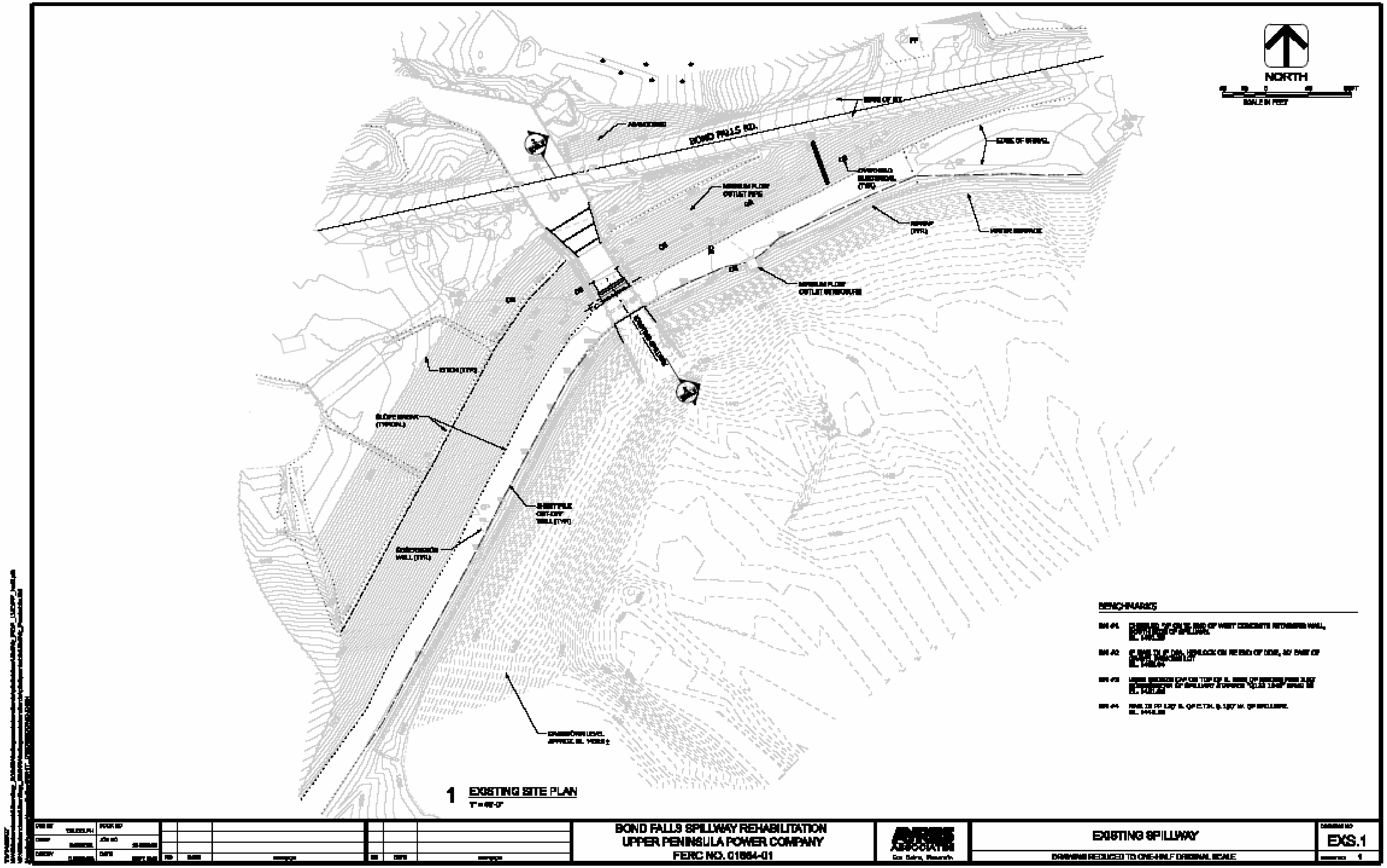



**Figure A.1.** The prototype plan view of the existing spillway.

## Appendix B: Spillway Design-1

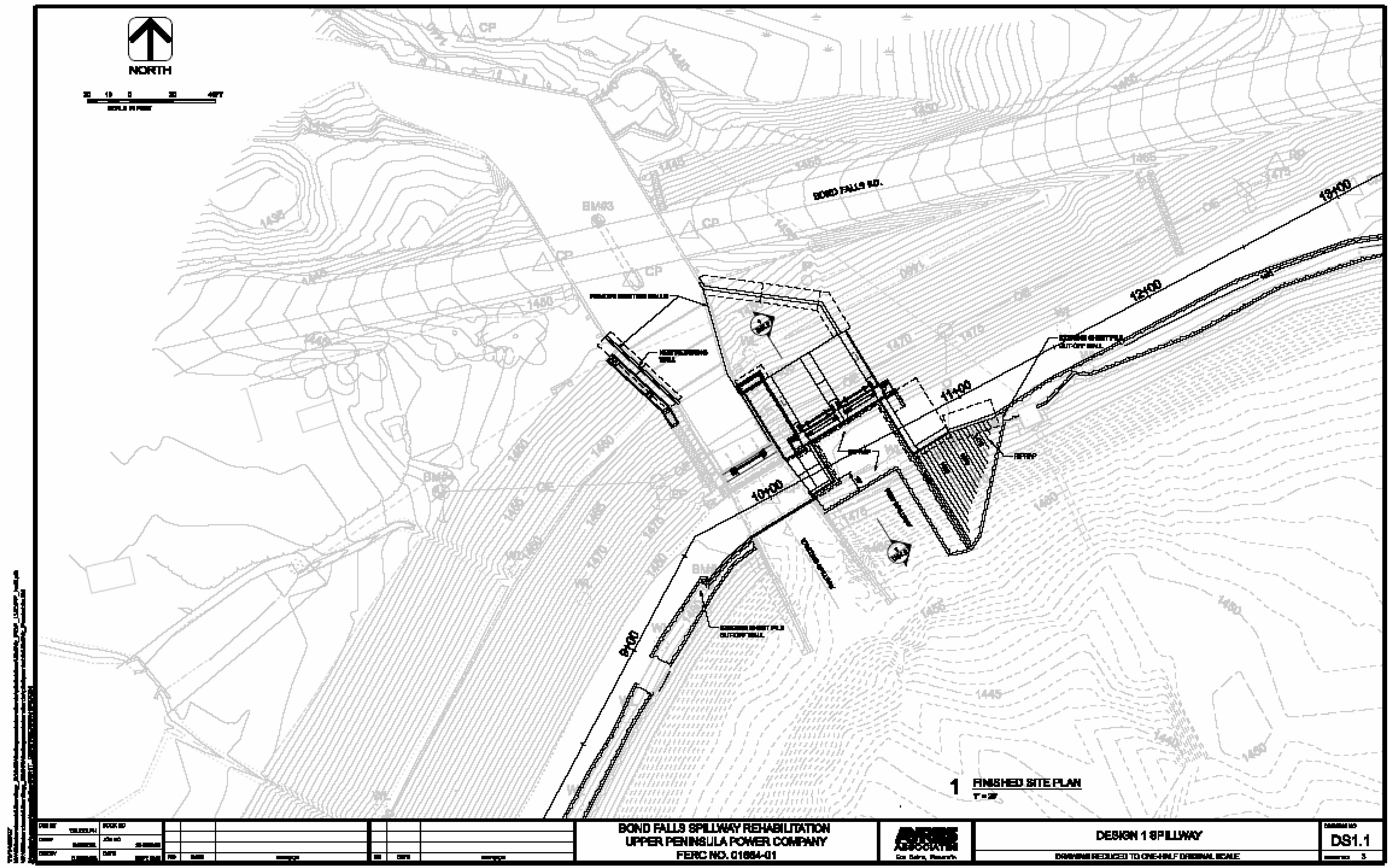



**Figure B.1.** The prototype plan view of the spillway Design-1

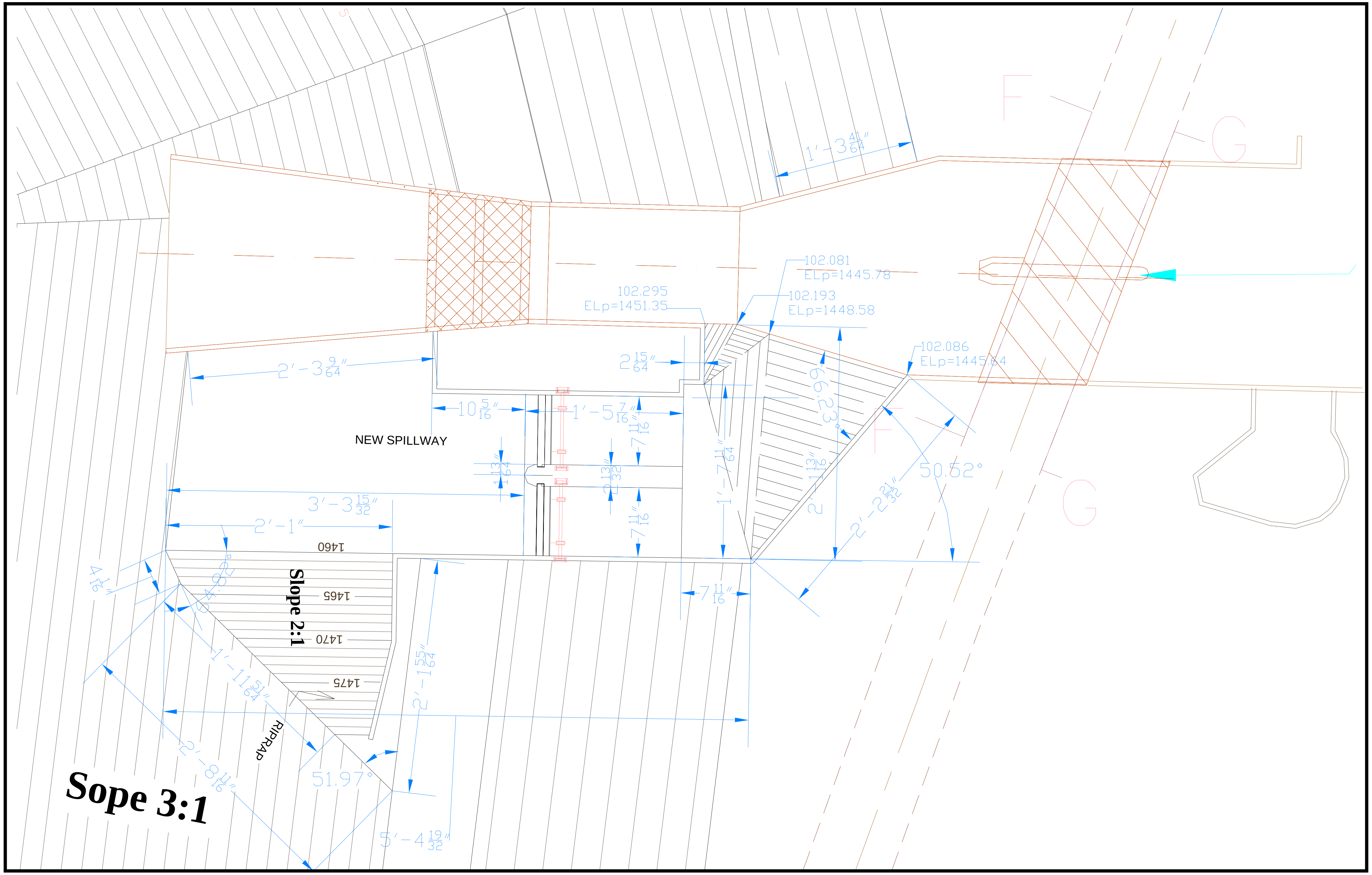



**Figure B.2.** The layout of the physical model of the Spillway Design-1

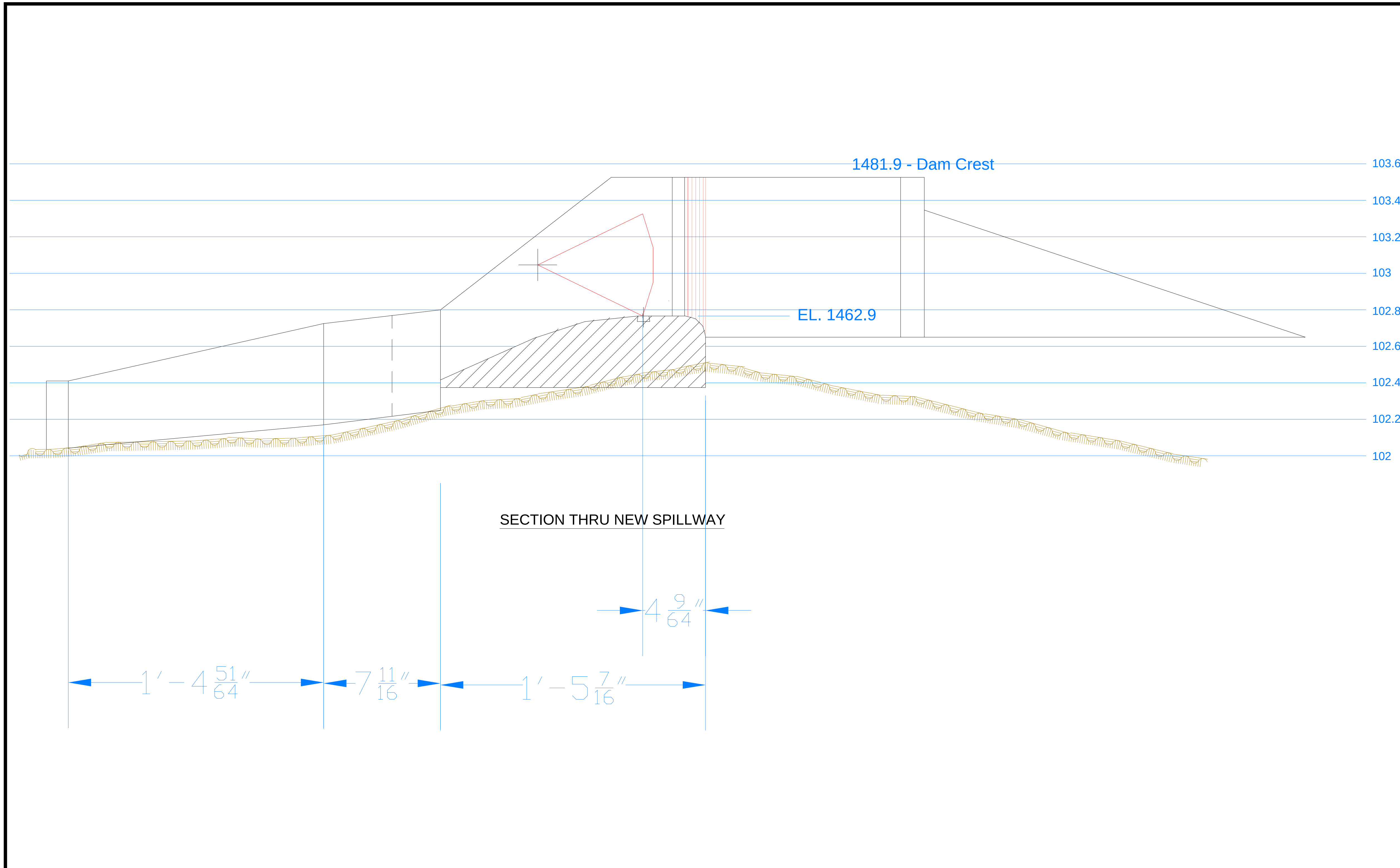



**Figure B.3.** The profile of the physical model of the Spillway Design-1

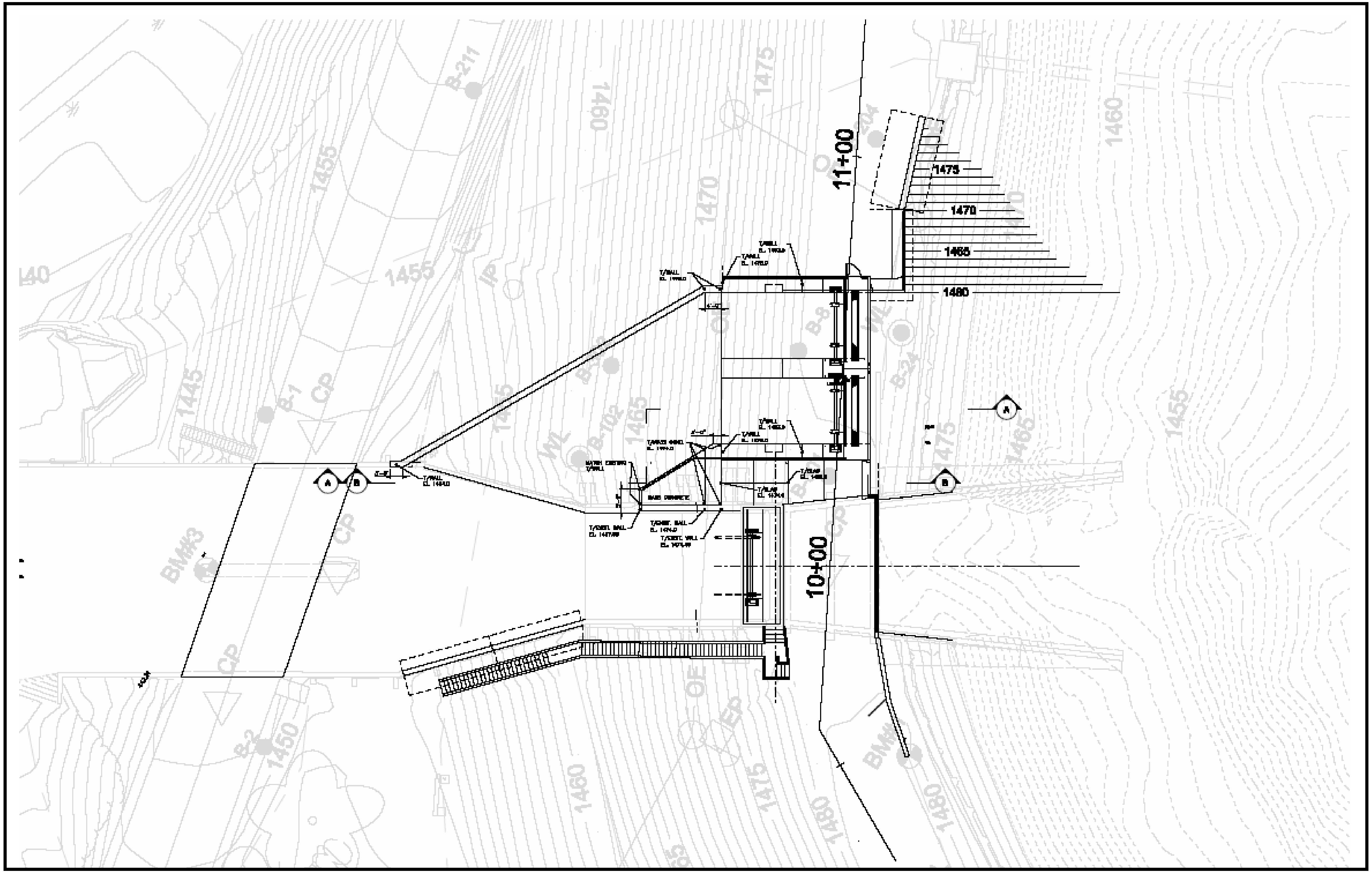


**Figure B.4**. The prototype plan view of the spillway Design-1 Modified-1.

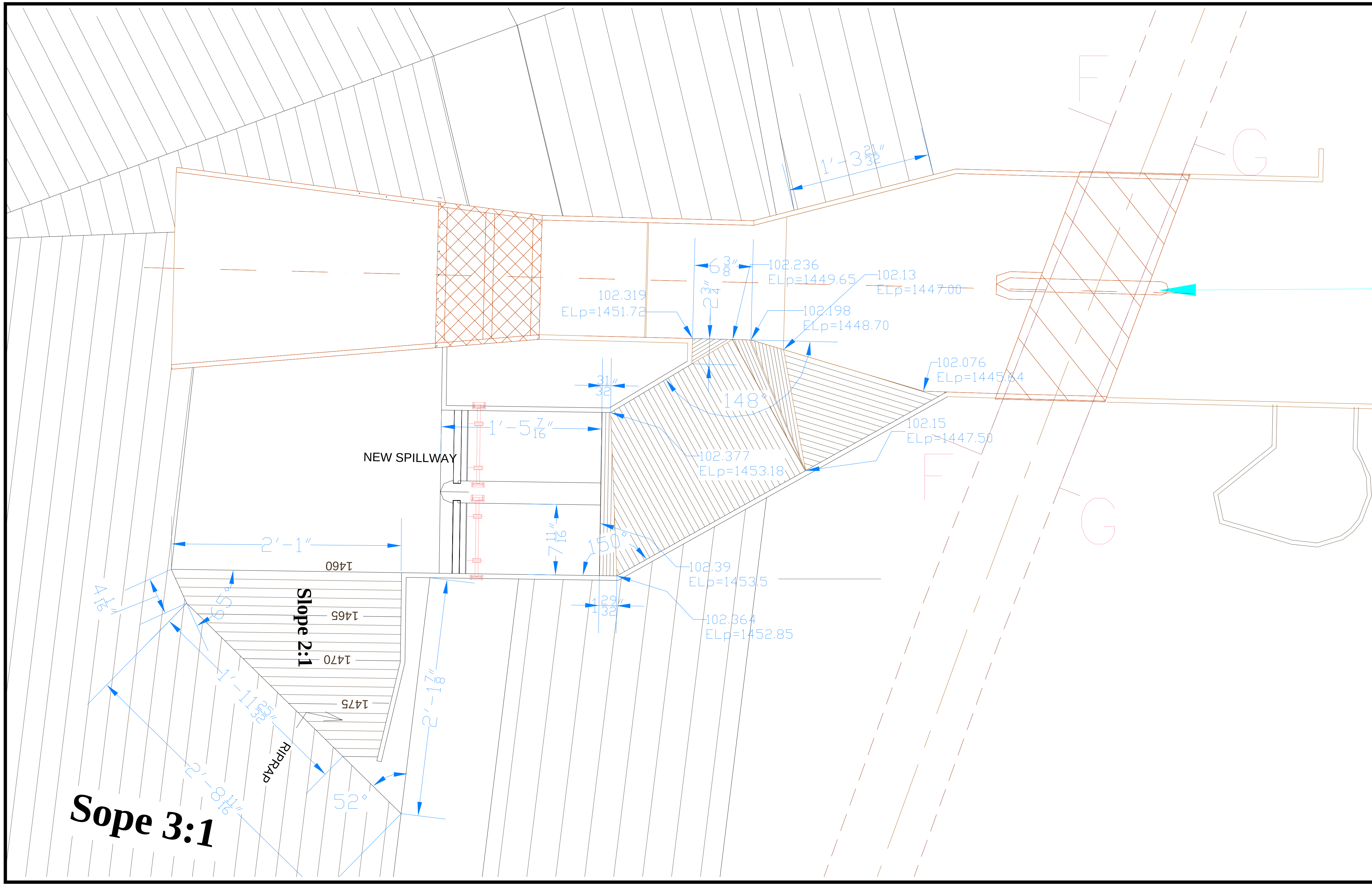



**Figure B.5.** The layout of the physical model of the Spillway Design-1 Modified-1

## Appendix C. Test Matrix for the Spillway Design 1

For the purpose of identifying the gates for the model test, the gates are numbered as follows (left to right referenced looking downstream):

1. Existing (main) gate

2. Center (new) gate

3. Right (new) gate

The model will be used to evaluate the following scenarios. The key issues are the hydraulic jump at the downstream left spillway retaining wall upstream of the bridge and the potential for erosion at the left or right embankment toe.

A. Gate 1 fully open, Gates 2 and 3 closed with normal maximum pool (1475.9). This scenario represents the planned operation to open the main gate fully before opening the auxiliary (new) gates. Computed discharge is 4,300 cfs.

B. Gate 1 fully open, Gates 2 and 3 open 6.5 feet with normal maximum pool. The purpose of this scenario is to determine whether it will be more favorable to open gates 2 and 3 equally vs. opening Gate 2 fully before opening Gate 3. Computed discharge is 7,100 cfs.

C. Gate 1 fully open, Gate 2 fully open, and Gate 3 closed with normal maximum pool. The purpose of this scenario is to determine whether it will be more favorable to open Gate 2 fully before opening Gate 3 vs. opening gates 2 and 3 equally. Computed discharge is 6,500 cfs.

D. All gates fully open with full pool/PMF (1480.9). This scenario represents the probable maximum flood. Computed discharge is 13,500 cfs (based on routings).

E. Gate 1 open 6.7 feet, Gates 2 and 3 closed with normal maximum pool. This scenario represents the 100-year flood with a discharge of 2,580 cfs.

F. Gate 1 open 8.9 feet, Gates 2 and 3 closed with normal maximum pool. This scenario represents the 500-year flood with a discharge of 3,250 cfs.

## Appendix D: Spillway Design-2

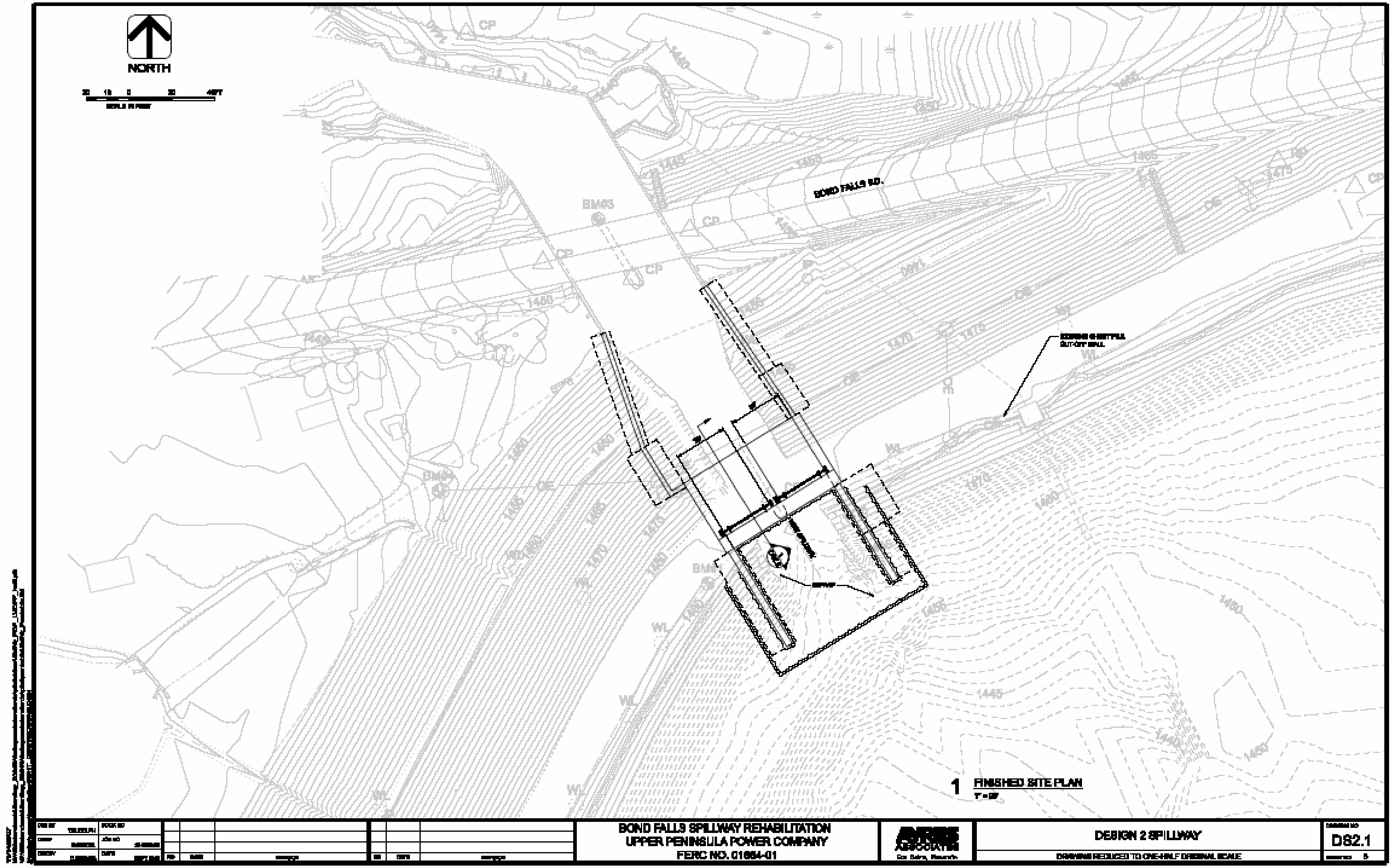



**Figure D.1.** The prototype plan view of the spillway Design-2

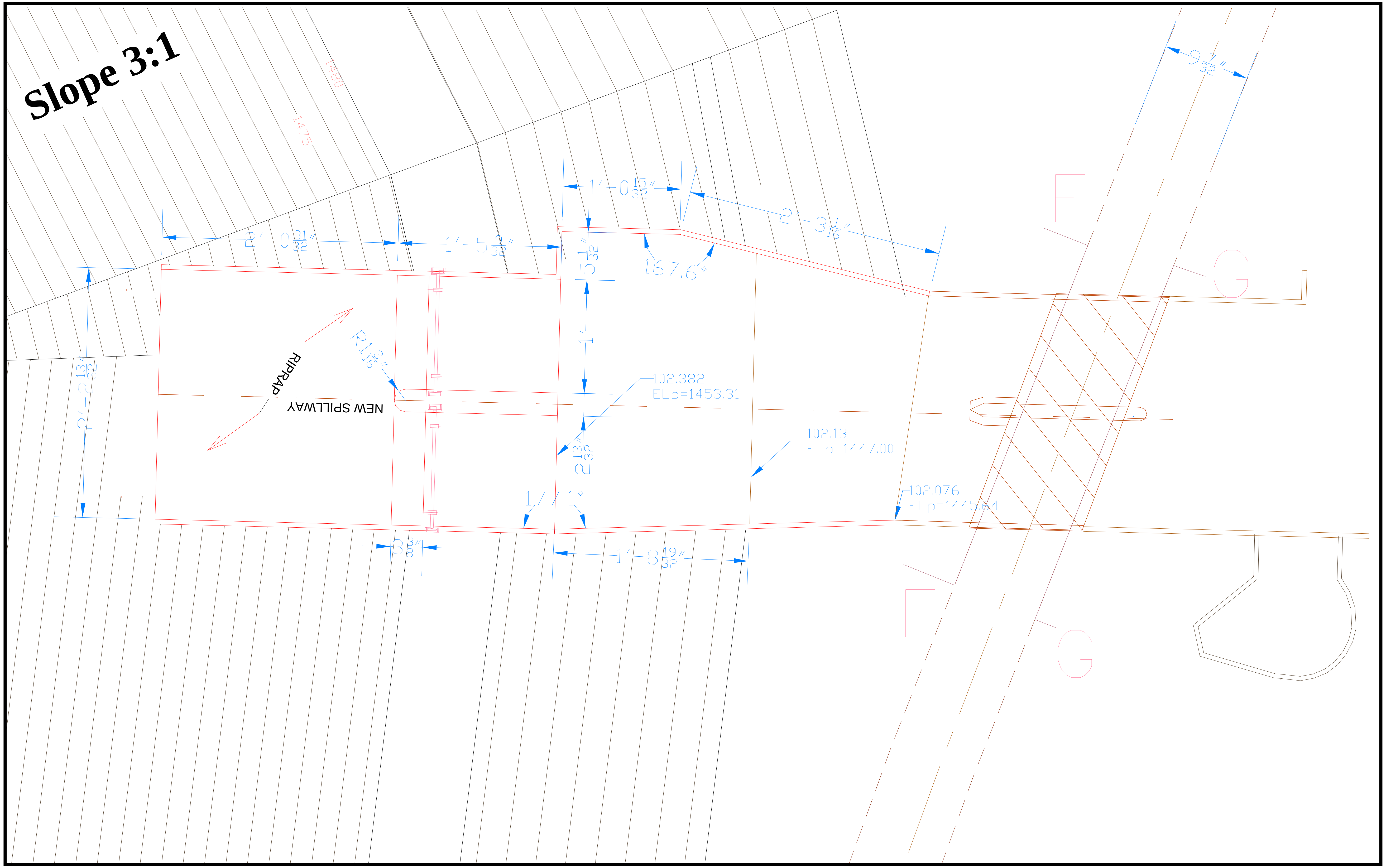



**Figure D.2.** The layout of the physical model of the Spillway Design-2

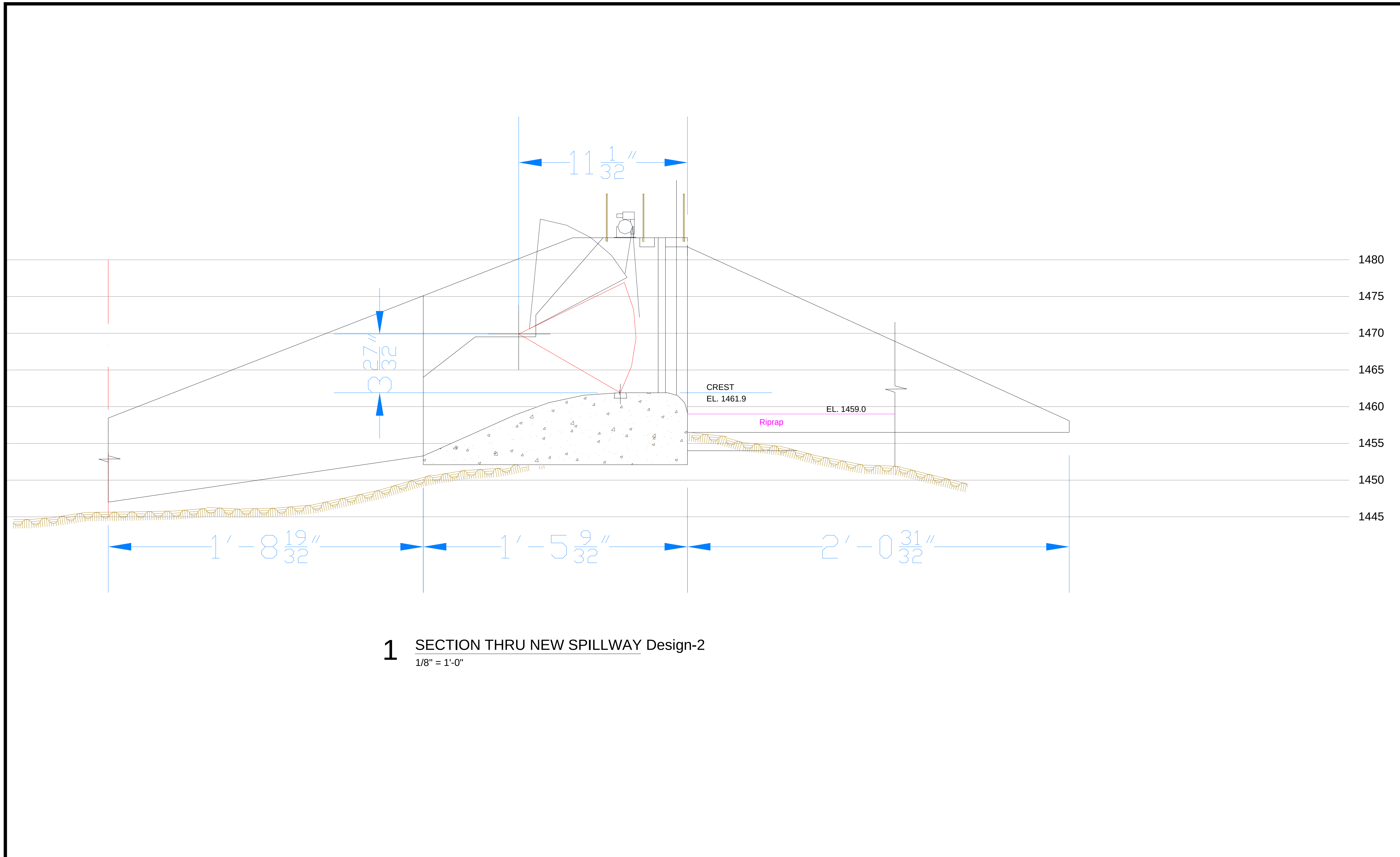



**Figure D.3.** The profile of the physical model of the Spillway Design-2

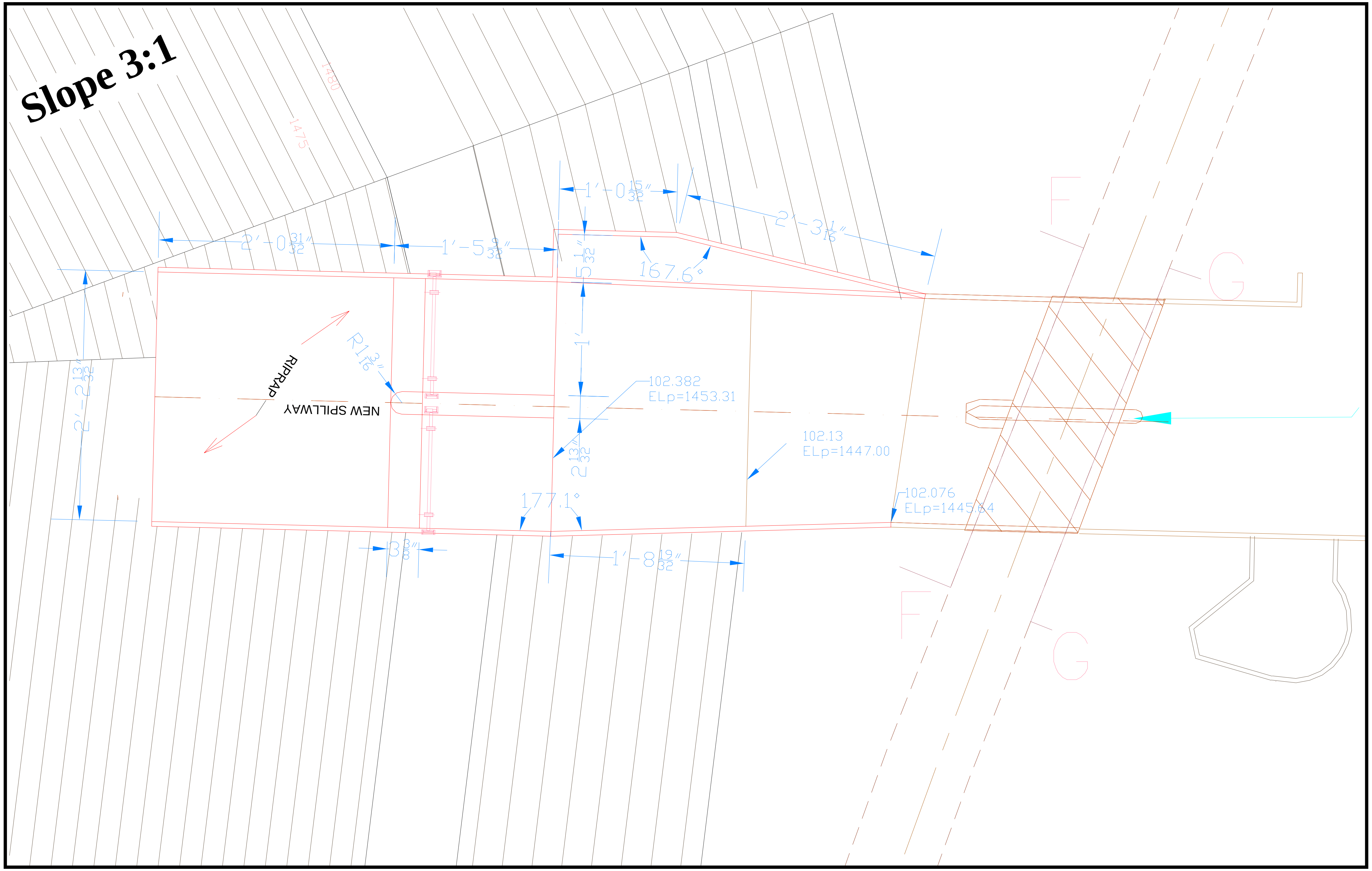



**Figure D.4.** The layout of the physical model of the spillway Design-2-Modified-1

## Appendix E. Test Matrix for the Spillway Design 2

For the purpose of identifying the gates for the model test, the gates are numbered as follows (left to right referenced looking downstream):

1. Left gate

2. Right gate

The model will be used to evaluate the following scenarios. The key issue is the potential for erosion at the left or right embankment toe due to overtopping of the downstream abutment walls between the spillway and bridge.

A. Gate 1 fully open, Gate 2 closed with normal maximum pool (1475.9). This scenario evaluates whether there are adverse impacts of fully opening one gate while the other remains closed. Computed discharge is 4,300 cfs.

B. Gate 2 fully open Gate 1 closed with normal maximum pool. This scenario evaluates whether there are adverse impacts of fully opening one gate while the other remains closed. Computed discharge is 4,300 cfs.

C. Both gates fully open with full pool (1480.9). This scenario represents the probable maximum flood. Computed discharge is 13,600 cfs.

D. Both gates open 3.1 feet with normal maximum pool. This scenario represents the 100-year flood with a discharge of 2,580 cfs.

E. Both gates open 3.9 feet with normal maximum pool. This scenario represents the 500-year flood with a discharge of 3,250 cfs.