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Hydraulic Model Studies of Drop 2 Storage Structures Colorado River Front Work and Levee System, CA

All American Canal Drop 2 Storage Reservoirs
Lower Colorado Region



U.S. Department of the Interior
Bureau of Reclamation

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14. ABSTRACT This report documents the hydraulic model study performed to assist with the design of the hydraulic structures of the Drop 2 Storage Structures for the Colorado River Front Work and Levee System, CA. The reservoir cells were constructed to provide storage of water from the Coachella canal during times when requested water is no longer needed, thus providing for water conservation. The model study included investigation of the flow conditions of the inlet canal to the forebay/afterbay basin, flow into the reservoir with a sandy soil cover, then draining of the reservoir back to an inlet that will take the water through pipelines back to the canal. A unique aspect of the study was the criteria to limit velocities on the soil cement and sand soil bottom of the reservoirs during initial filling. Verifying the time to fill and drain the reservoirs was also aided by use of the model.					
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**All American Canal Drop 2 Storage Reservoirs
Lower Colorado Region**

Kathleen H. Frizell



**U.S. Department of the Interior
Bureau of Reclamation**

March 2008

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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.

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Executive Summary

This summary briefly describes the results of the hydraulic model studies performed to investigate the designs for the Drop 2 Storage Structures Colorado River Front Work and Levee System, near Yuma, Arizona. The entire study results are documented in Hydraulic Laboratory Report HL-2008-01, "Hydraulic Model Studies of Drop 2 Storage Structures Colorado River Front Work and Levee System, CA, All American Canal Drop 2 Storage Reservoirs, Lower Colorado Region" by Kathleen H. Frizell. The report is available electronically on the Hydraulic Investigations and Laboratory Services Group website at: http://www.usbr.gov/pmts/hydraulics_lab/pubs/HL-2008-01.pdf.

The purpose of the Drop 2 Storage Reservoir project is to conserve water by providing operational flexibility to the Lower Colorado River System by construction of an off-stream reservoir and associated facilities. The project includes a turnout located upstream from All American Canal Drop Structure No. 1, a 6½-mile-long new canal section, a gated canal inlet to a forebay/afterbay basin with two gated inlet/outlet structures for filling or draining two 4000 AF cells of the Drop 2 Reservoir. When draining, the reservoir cells empty through the forebay/afterbay basin into a gated siphon inlet, then into two 9-ft-diameter pipelines, finally returning to the All American Canal through a short canal and a newly constructed confluence.

Design criteria for the project require filling or draining the reservoir cells with the design flow of 1800 ft³/s in 3 days while not eroding the soil cement or sandy cover placed over the constructed embankments and geomembrane liners. To investigate these criteria, a physical and numerical modeling effort was undertaken by the Hydraulic Investigations and Laboratory Services Group. The physical model was to investigate the adequacy of hydraulic structure performance in steady-state situations with the numerical model providing information on the filling and draining requirements.

The three-dimensional physical model, with 1:18 Froude scale, was initially constructed to include a small portion of reservoir cell No. 1 with the inlet/outlet structure, the full 135-ft-width and 345 ft of the 438 ft design length of the forebay/afterbay basin with 2:1 stepped sloping side walls for a height of 28 ft and the siphon inlet. The forebay/afterbay basin floor at El. 133 ft was rectangular in shape. Water was supplied into the reservoir. The siphon inlet was located at the southwest end of the forebay/afterbay basin. The 40-ft-wide inlet/outlet structure and siphon inlet were both constructed with vertical side walls following the 3:1 upstream reservoir slope and/or 2:1 forebay/afterbay basin slopes. The inlet/outlet structure included three 10-ft-wide gates with piers separating the bays. The gate seats and apron were at the same elevation of 133 ft as the forebay/afterbay basin and reservoir floor. Aprons extended out to the forebay/afterbay basin floor or the reservoir floor to meet the toe of the 2:1 or 3:1 embankment slopes. The siphon inlet was constructed with a flat floor and boards

to control flow exiting the model without modeling the remaining siphon inlet geometry including the pipelines and outlet canal.

Both the physical and numerical models were utilized during the initial testing of the Drop 2 reservoirs. Operations were discussed at length during the initial testing phase. It was determined that the initial filling phase was the most critical and would be the focus of the studies. Therefore, testing of flow into the forebay/afterbay basin was mostly accomplished with as low a depth in the basin as possible. In reality, the forebay/afterbay basin will most likely be full and the reservoirs empty when the canal inlet opens to fill the reservoirs. This will mean high velocity flow will enter the reservoirs initially. It was determined during these discussions and initial operation of both models that filling and draining should probably be accomplished with all gates fully open as much as possible to reduce losses.

Initial testing of the concept designs showed that target velocities to minimize erosion would be exceeded and that flow conditions needed to be improved. As a result, major modifications were made to the designs and these were incorporated into the physical model. The numerical modeling effort was abandoned at this point and final observations and results were obtained only from the physical model. Important modifications included relocating the forebay/afterbay basin so that the siphon inlet was centered between the two inlet/outlet structures, flaring structure walls, and lowering apron inverts. At this time, an operational canal inlet was added for model investigation.

The design criteria during initial filling produced difficult flow conditions to deal with for flow through all the structures. The final design geometries were obtained from the model study, consultations with the design team and the use of engineering judgment. Final designs of the siphon inlet, inlet/outlet structure, and canal inlet greatly improved flow conditions. All flow patterns will improve as the forebay/afterbay basin or reservoirs fill. Filling will be fast in the forebay/afterbay basin and, this result, along with results of soil cement jet testing [Bartojay, 2007], contributed to the decision regarding the final design geometries.

It is recommended that the initial operation of the facilities be monitored to ensure appropriate flow conditions are achieved. After an initial filling and draining of the reservoirs it is recommended that they be inspected to assess whether the constructed inlet geometry performed adequately.

Background

The purpose of the Drop 2 Storage Reservoir project is to conserve water by providing operational flexibility to the Lower Colorado River System by construction of an off-stream reservoir and facilities. For the All American Canal, the water priorities are irrigation, domestic use, and power production. Water that is provided to water users on the Lower Colorado River is ordered in advance for scheduled release from Parker Dam. It takes approximately three days travel time for water released from Parker Dam to arrive at Imperial Dam for diversion into the All American Canal. During this time, changes in demand may occur resulting in demand/delivery mismatches.

The Drop 2 Storage Reservoir will support conservation of water by temporarily storing water resulting from operational mismatches that occur within the Lower Colorado River System. Instead of allowing the water to continue in the Colorado River past Imperial Dam, the excess water will be diverted into the All American Canal. At Drop Structure No. 1, the excess water will be delivered by gravity to the Drop 2 Storage Reservoir. The diverted water will be stored in the new Drop 2 Storage Reservoir until it can be used at a later date by the Imperial Irrigation District. To match the water travel time, the two 4000 acre-foot reservoirs must be capable of being completely filled or drained with a maximum inflow of 1800 ft³/s in a three day period. Water temporarily stored in the reservoirs will then be delivered by gravity back to the All American Canal at the newly constructed confluence immediately downstream from Drop No. 2.

A general plan showing all of the included features of the Drop 2 Storage Reservoir is shown in Figure 1.

Water will be diverted to the Drop 2 Storage Reservoir through the existing Coachella Canal Turnout Structure located on the All American Canal immediately upstream from All American Canal Drop Structure No. 1 (Drop 1). The existing turnout structure will be modified to allow a portion of the structure to deliver water to the existing Coachella Canal with the remainder of the structure delivering water to the Drop 2 Storage Reservoir via a Reservoir Inlet Canal that will be approximately 6½ miles long, skirting a privately held section.

The inlet canal will flow into a reservoir forebay/afterbay basin which is essentially a deepened and widened canal section utilizing a 2:1 sloping compacted embankment with a geomembrane under a full lining of soil cement placed in layers to form steps. The reservoir forebay/afterbay basin serves as a transfer basin for water either entering or leaving the two reservoir cells to the siphon inlet and outlet canal back to the All American Canal. The reservoir forebay/afterbay basin is located to the south of the two reservoir cells. The reservoir cells will be filled and drained with a gated inlet/outlet structure for each cell through the forebay/afterbay basin. A ramp will be provided for vehicular

access to the bottom of the forebay/afterbay basin for operation and maintenance activities.

The two reservoir cells have a total capacity of 8000 AF and will have a depth of 21 ft at reservoir El. 154 ft. The bottom of the reservoir is at El. 133 ft after the geomembrane lining is covered with sandy soil. The 3:1 reservoir embankment slopes will be covered with soil cement slope protection using the plating method. The reservoir cells can operate independently using their respective inlet/outlet structures located near the southwest and southeast corners of reservoir cells No. 1 and 2, respectively. Maintenance can be performed on one cell while water can be stored in the other cell.

Drop 2 Storage Reservoir water will be returned to the All American Canal system through a siphon inlet to a 2300-ft-long pair of pipelines and canal outlet system crossing under Evan Hewes Highway and Interstate 8. The I-8 road crossing and the outlet canal siphon entrance originate near the center forebay/afterbay basin south wall. The outlet canal terminates at the confluence with the newly lined All American Canal immediately downstream of All American Canal Drop Structure No. 2.

Introduction

The new canal will have a gated outlet structure that flows into a forebay/afterbay basin, then into two large storage reservoir cells through separate gated inlet/outlet structures for filling (inlet) and draining (outlet) the two reservoir cells. The gated structures between the forebay/afterbay basin and the two reservoir cells must pass 8000 AF into and out of the reservoir in 72 hours. When the water is needed the reservoir cells are then emptied back into the All American Canal through the forebay/afterbay basin, siphon inlet, pipelines, and outlet canal at a newly constructed confluence. A closer plan view of the forebay/afterbay basin with the location of the structures is shown on figure 2. The canal inlet and the two inlet/outlet structures have the potential to produce high velocity flow from discharges up to 1800 ft³/s onto either the soil-cement lining in the forebay/afterbay basin or the unprotected sandy soil material on the bottom of the two storage reservoir cells. The reinforced concrete inlet/outlet structures consist of three gated bays with aprons that end a short distance downstream from the gates on the bottom of the forebay/afterbay basin or reservoir at the same elevation as the gate seat. The normal water surface when the reservoir cells and forebay/afterbay basin are filled is El. 154 ft and is 21 ft above the structures, basin, and reservoir cell inverts at El. 133 ft.

The operation of the structures could require release of water from a full forebay/afterbay basin to an empty reservoir that could lead to erosion of the sandy reservoir material covering the membrane liner. In addition, the forebay/afterbay basin footprint is very near a state highway; therefore, flow conditions must be appropriate to reduce loss, minimize sediment deposition, and ensure timely filling and draining while limiting its footprint size.

Purpose

The overall purpose of the hydraulic modeling was to ensure adequate performance of all the hydraulic structures. The flow conditions associated with flow into and out of the forebay/afterbay basin given the location and geometry of the structures and the size of the basin was studied. The following operational conditions were investigated:

- Flow from the reservoir through the outlet structure to the forebay/afterbay basin (reservoir draining condition),
- Flow from the forebay/afterbay basin through the inlet structure to the reservoir (reservoir filling condition),
- Flow from the gated inlet canal through the forebay/afterbay basin and inlet structure (direct reservoir filling condition).

Assessment of energy dissipation, flow patterns, and potential zones of sediment deposition during flow into and out of the forebay/afterbay basin may lead to reducing the size of the basin.

The modeling was proposed in several phases that included modifications to the hydraulic model each time to investigate a different operational scheme with the components of the Drop 2 structures.

Results from the physical model were to be input into a computational fluid dynamics (CFD) numerical model, FLOW-3D®, to verify the filling and draining times for the reservoirs.

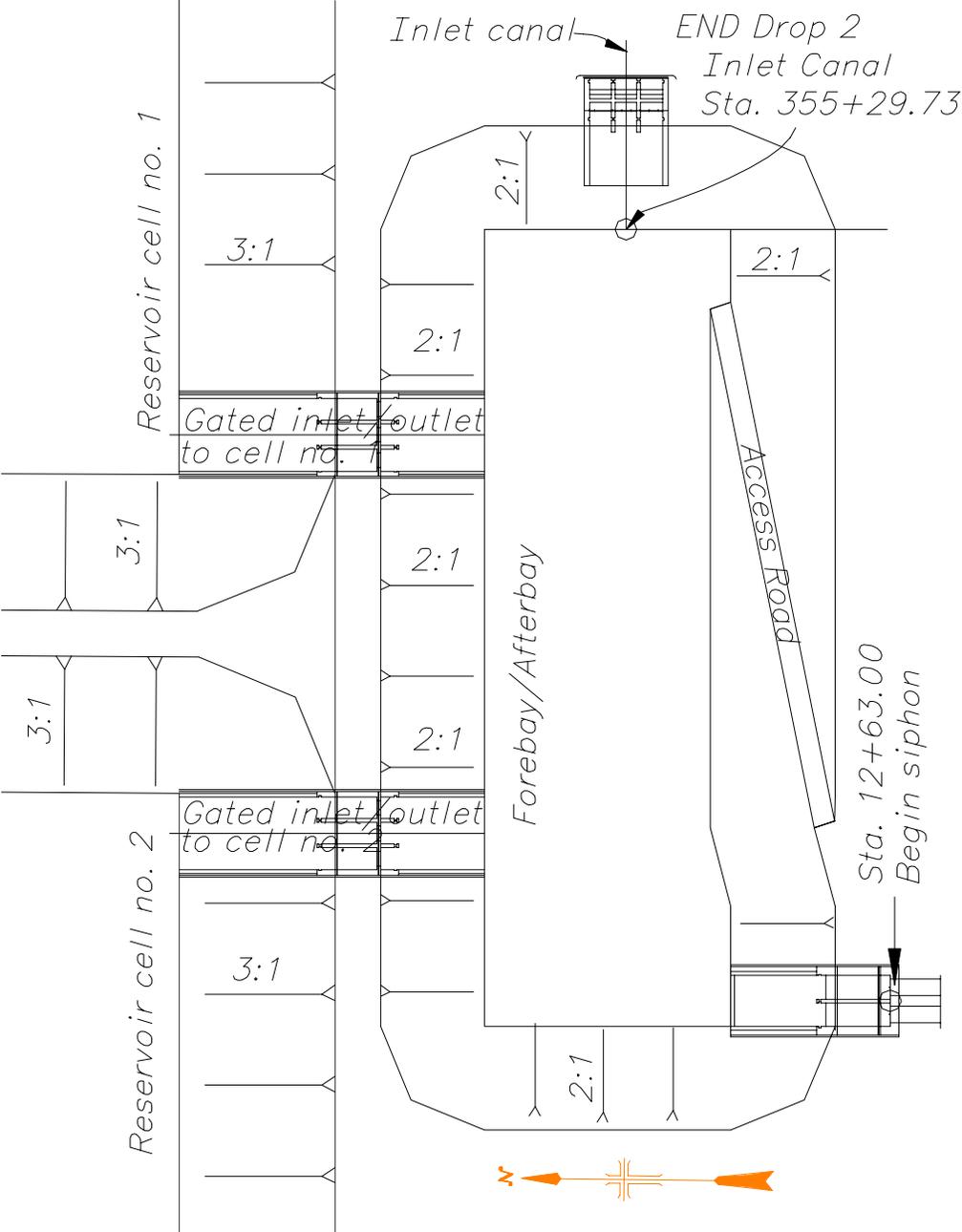


Figure 2. - Close up plan view of the canal inlet and the forebay/afterbay basin with the inlet/outlet structures to the two reservoir cells and the siphon inlet leading to the confluence with the newly-lined canal portion.

Design Criteria

The maximum flow into the forebay/afterbay basin and through the inlet/outlet structures and the siphon inlet is 1800 ft³/s. The normal operational water surface elevation is El. 154 ft in the forebay/afterbay basin, the canal, and the reservoirs. With unexpected operation, a reservoir elevation of 157 ft could be attained and would be a worst case scenario from an energy dissipation standpoint.

The following are the design criteria for the system:

- Reservoir filling and draining must occur within 3 days. This is related to the travel time from Parker Dam for flow changes,
- Velocities on the invert of the reservoir must be 1.5 ft/s or less to prevent soil erosion and exposure of the geomembrane lining. This criterion is of concern during initial filling of the reservoir cells,
- Limit the velocities over the soil cement in the forebay/afterbay basin to 10 ft/s, and
- Minimize the size of the forebay/afterbay basin and deposition of sediment by improving flow patterns.

The concern for meeting these criteria is during initial filling or draining of the reservoir through the inlet/outlet structure or from initial filling of the forebay/afterbay basin with the canal inlet. The operation of the forebay/afterbay basin and the reservoirs provides for unusual flow conditions during startup compared to operation for most Reclamation structures. The reservoirs will be empty upon initial opening of the inlet structure gates. The forebay/afterbay basin may be empty or full when the canal inlet gates are open. If maintenance has been performed in the forebay/afterbay basin it might be empty during initial start up of the canal flow during filling or the release of stored reservoir water into the forebay/afterbay basin during draining. Either of these cases would cause high velocity flow to exit onto the soil cement surfaces with no tailwater available for energy dissipation until filling has progressed for some length of time.

The other important design criterion is the ability of the system of both reservoirs to be completely filled or drained within 3 days. This would apply for flow rates up to 1800 ft³/s. If only one reservoir cell is needed then the cell would need to be filled or drained within 36 hours.

Model Description

The physical hydraulic model was constructed in an existing model box that is 28-ft-wide by 40-ft-long by 4-ft-deep in the Hydraulic Investigations and Laboratory Services Group laboratory. The selected Froude model scale of 1:18 maximized the discharge into the model while still being able to use the existing laboratory space. It was decided to model the majority of the length of the forebay/afterbay basin, a portion of reservoir cell No. 1, with the inlet/outlet structure, and a simplified siphon inlet because the geometry of the other inlet/outlet and reservoir were identical. Judgment could be made about potential dissimilarities that might exist when filling or draining

reservoir cell No. 2. Therefore, flow only entered the model into reservoir cell No. 1 to first investigate reservoir draining and then the model would be changed to test other configurations in phases.

The three-dimensional physical model, with 1:18 Froude scale, was initially constructed to include a small portion of reservoir cell No. 1 with the inlet/outlet structure, the full 135-ft-width and 345 ft of the 438 ft design length of the forebay/afterbay basin with 2:1 stepped sloping side walls for a height of 28 ft and the siphon inlet, figure 3. The forebay/afterbay basin floor at El. 133 ft was rectangular in shape. The siphon inlet was located at the southwest end of the forebay/afterbay basin. The 40-ft-wide inlet/outlet structure and siphon inlet were both constructed with vertical side walls following the 3:1 upstream reservoir slope and/or 2:1 forebay/afterbay basin slopes. The inlet/outlet structure included three 10-ft-wide gates with piers separating the bays. The gate seats and apron were at the same elevation (133 ft) as the forebay/afterbay basin and reservoir floor. Aprons extended out to the forebay/afterbay basin floor or the reservoir floor to meet the toe of the 2:1 or 3:1 embankment slopes. The siphon inlet was constructed with a flat floor and boards to control flow exiting the model without modeling the remaining siphon inlet geometry.

The maximum discharge of 1800 ft³/s was delivered to the model and measured using the laboratory's permanent system of pumps and Venturi meters.

The physical model study investigations were proposed to be conducted in three phases. The initial geometry under investigation in phase 1 was just discussed. Planned phase 2 modifications included constructing the mirror image of the topography and reversing the inlet/outlet structure to investigate reservoir filling. Planned phase 3 modifications included removing the reservoir inlet/outlet structure, making small modifications to the geometry, and reinstalling it as the canal inlet structure to investigate filling the forebay/afterbay basin. Phases 2 and 3 were to be performed in the manner described because flow was to only enter the model through the one reservoir location.

A plan view of the initially constructed geometry is shown on figure 3 with only the structures that were modeled shown. In addition, the locations for velocity measurements during reservoir draining are labeled.

The physical model did not include the capability to dynamically simulate gate operations, nor did it include the entire siphon inlet geometry and downstream outlet canal system. During concept design, a HEC-RAS one-dimensional model was used by Ken Sayer, Water Conveyance Group, to determine that the filling and draining rate could be obtained. The results of that model were highly dependent on a number of assumptions. Therefore, CFD modeling was performed to investigate the filling and draining times for the system.

The CFD model for this investigation used a commercially available computational code. The CFD program FLOW-3D® Version 9.2 by Flow Science Inc. is a finite difference/volume, free surface, unsteady flow modeling system, developed to solve the Navier-Stokes equations in three spatial dimensions. The finite difference equations are based on an Eulerian mesh of non-uniform hexahedral-shaped control volumes using the Fractional Area/Volume (FAVOR) method. Free surfaces and material interfaces are defined by a fractional volume-of-fluid (VOF) function. FLOW-3D® uses an orthogonal coordinate system as opposed to a body-fitted system and can have a single

nested mesh block, adjacent linked mesh blocks, or a combination of nested and linked mesh blocks.

The geometry of the Drop 2 reservoirs including the gate structure of the inlet canal, the forebay/afterbay basin, the two inlet/outlet structures, a small portion of each reservoir cell (40,000 square feet each), and the siphon inlet structure was created using AutoCAD to produce a three-dimensional solids model from which a stereolithography (STL) input file was rendered and imported into FLOW-3D®. The model will use mass particles that will help indicate zones of sediment deposition and transport into and out of the forebay/afterbay basin and each reservoir.

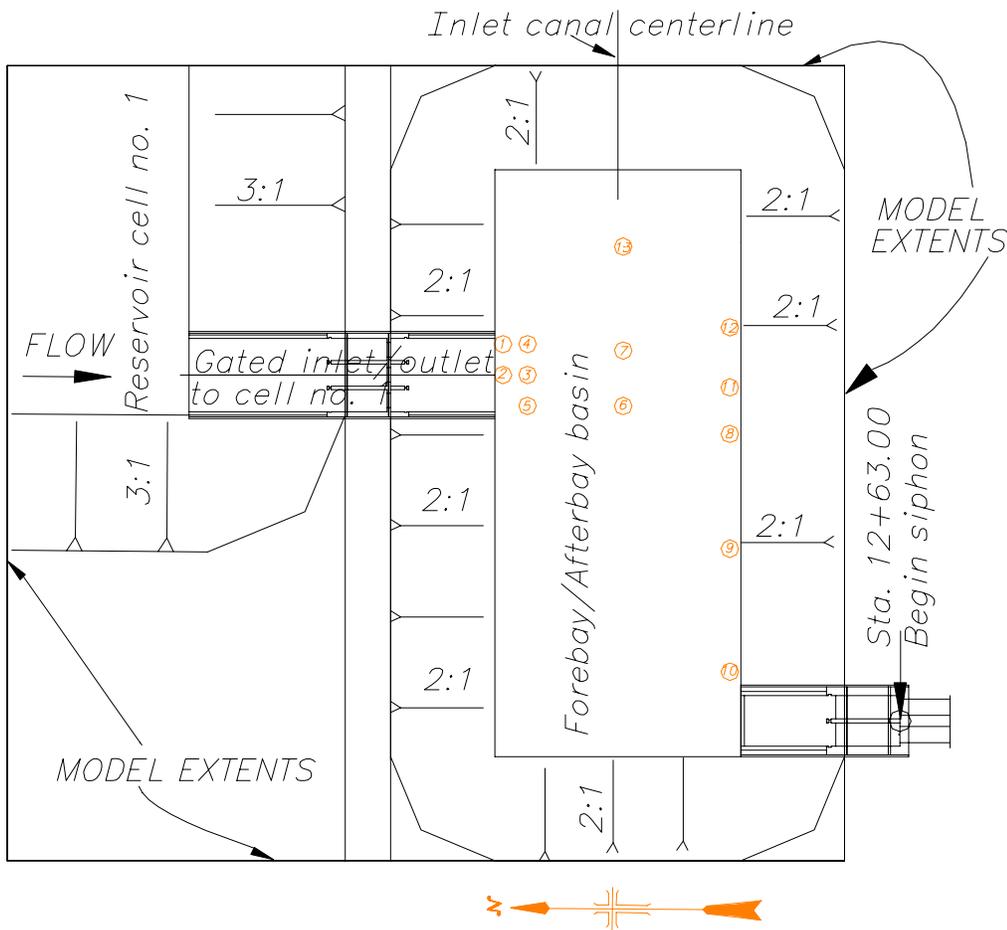


Figure 3. - Plan view of the originally modeled structures with the forebay/afterbay basin and small portion of reservoir cell No. 1. Flow entered the model through the reservoir. Velocity measurements were taken at the numbered locations in the forebay/afterbay basin.

Test Plan

The test plan included investigations with both a physical and numerical model.

Physical Model

The physical model testing included an initial phase that would determine whether following phases were necessary. The initial modeling, phase 1, and two modifications, phases 2 and 3, to the physical model were proposed. Phase 1 of the project was to design and construct the physical model and perform the following tests in the model:

- Observe flow conditions and document velocities for the initial outlet structure geometry during reservoir draining,
- Determine a configuration of blocks or sills on the reinforced concrete apron below the gates to dissipate the energy in the high velocity release so that:
 - Velocities are at an acceptable level for the soil cement floor in the forebay/afterbay basin,
 - Evaluate if protection would be needed on the bottom of the two sandy earth-lined reservoir cells and, if so, the type and extent, and
 - Determine if the footprint of the forebay/afterbay basin may be reduced.

Phase 2 would include turning the inlet/outlet structure around and constructing the mirror image of the topography to investigate inflow to the reservoir from the forebay/afterbay basin through the inlet structure, because the flow in the model cannot be reversed. The flow conditions into the reservoir including along the toe of the slope adjacent to the end of the inlet structure would be evaluated. Potentially, the inlet/outlet structure may need to be moved away from the toe of the embankment slope.

Phase 3 would include removing the reservoir inlet/outlet structure, making small modifications to the geometry and reinstalling it as the canal inlet structure. Flow conditions from the canal inlet structure into the forebay/afterbay basin would then be investigated.

At the conclusion of phase 1, the results were presented to the design team and it was decided that several major modifications to all the structures were needed. Therefore, phases 2 and 3 were not completed, but incorporated into a major modification of the physical model. The modifications included adding flow to the model through the construction of the canal inlet at the east end of the forebay/afterbay basin, modifying the invert and walls of the inlet/outlet structure and siphon inlet and centering the siphon inlet in the forebay/afterbay basin where it would be between the two inlet/outlet structures in the prototype.

Numerical Model

Numerical modeling of the forebay/afterbay basin was to be performed in a separate phase following the physical modeling if deemed necessary after results from the physical model were reviewed. Initial operation of the physical model pointed out that the CFD model could be useful in

determining operations. Therefore, the operational criteria were investigated using the physical and numerical models concurrently.

The objective of the numerical modeling was to determine if flow conditions in the forebay/afterbay basin are reasonable and will allow expected operation of the structures and if the filling and draining times meet the criteria. The objectives would be accomplished by running 1800 ft³/s through the system under the following four scenarios until a steady state is achieved:

- Inlet canal to reservoir cell No. 1,
- Inlet canal to reservoir cell No. 2,
- Reservoir cell No. 1 to the siphon inlet, and
- Reservoir cell No. 2 to the siphon inlet.

The results of the numerical modeling would provide velocity vector maps showing the expected locations of low velocity or eddy zones where sediment would be expected to deposit. Head loss through the system would be provided that would allow the designers the ability to determine if the filling and draining time criteria would be met.

Instrumentation

Discharges into the model were controlled by the central laboratory system and measured using the permanently installed Venturi system. Water levels in the reservoir were measured using a stilling well and hook gage. Water levels in the forebay/afterbay basin were measured manually by inserting a measuring tape into the basin or by reading the staff gage installed on the floor. A target water level of El. 154 ft was recorded upstream from the canal inlet gates and used to set gate openings for various discharges investigated.

Velocities were measured using a SonTec/YSI, Inc. FlowTracker Handheld Acoustic Doppler Velocimeter (ADV) with a two-dimensional probe and firmware version 3.3 with software version 2.20 (figure 4). The FlowTracker operates using separate acoustic transducers to transmit and receive signals that are mounted on the probe and intersect in a controlled volume of water. The generated signal is reflected back from the water sampling volume to the receivers where the signal is stored in the handheld unit. The velocity can be determined by the change in frequency or Doppler shift and knowledge of the orientation of the probe axes.

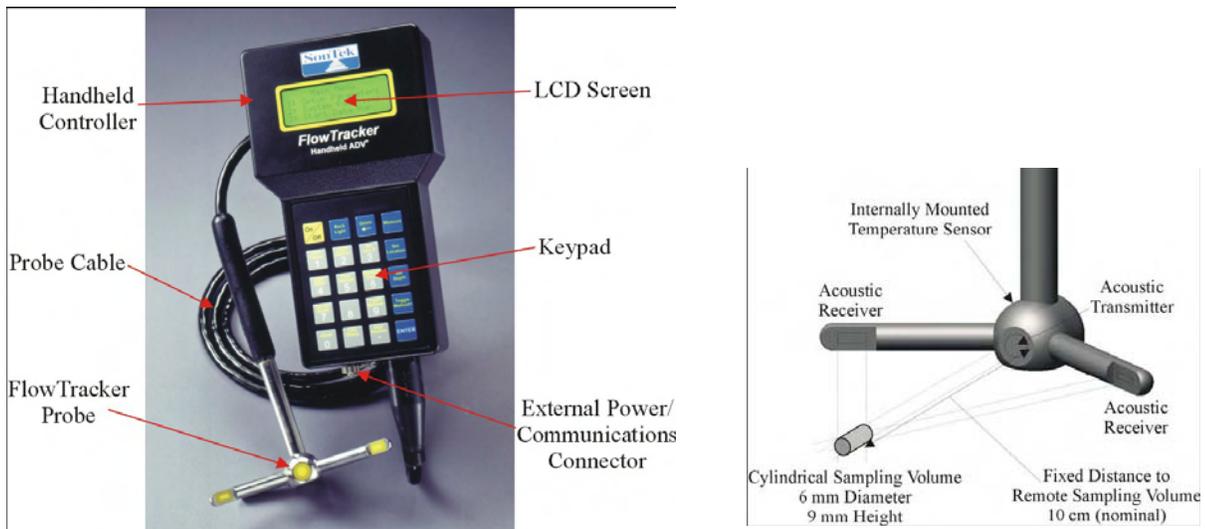


Figure 4. - FlowTracker handheld controller with 2D probe on the left and close up of the 2D probe on the right showing the location of the sampling volume. (Photos courtesy of Sontek, Inc.)

Model Investigations

Initial testing of the hydraulic structures began with investigating draining reservoir cell number 1 through the outlet structure to the forebay/afterbay basin. Several operations were evaluated as worst case scenarios for the draining of the reservoir:

- A 1-ft gate opening with reservoir El. 157 ft and a discharge of about 630 ft³/s with the forebay/afterbay basin water level at El. 136 ft.
- Gate control with reservoir El. 157 ft and a discharge of 1800 ft³/s with the forebay/afterbay basin water level at El. 140 ft.
- Fully open gates with reservoir El. 142.72 ft and a discharge of 1800 ft³/s with forebay/afterbay basin water level at El. 140 ft.

Initial observations of the flow conditions were made with forebay/afterbay basin water surface elevation controlled by the size of the siphon inlet opening exiting the basin. The geometry of the siphon inlet and siphon were not replicated in the model, so the CFD model was set up and run to verify if the water surface elevations being used during testing were appropriate. Because the forebay/afterbay basin water surface elevation will be constantly changing as the reservoir empties, the water surface used for testing was determined adequate. For each operation, flow conditions were observed and documented and velocities at six-tenths depth recorded at the locations shown on figure 3 in the forebay/afterbay basin.

Initial Testing during Reservoir Draining

Gate Opening of 1 ft

Initial observations of the flow conditions were made with 1 ft gate openings under maximum reservoir El. 157 ft and minimum tailwater. The flow exited the apron prior to formation of the hydraulic jump under this tailwater condition. Flow then traveled to the right across the basin due to the backwater area forming to the left of the outlet and critical flow at the siphon inlet. The flow slightly ran up the 2:1 slope on the opposite bank of the forebay/afterbay basin. The flow then split, with the majority headed along the toe of the slope toward the siphon inlet. Upon reaching the inlet, the flow turned 90 degrees forming a large contraction on the left side before entering the inlet and exiting the model. Figure 5 shows the flow conditions in the forebay/afterbay basin for 1 ft gate openings as would be the case when the reservoir initially begins draining into an empty forebay/afterbay basin.

Figure 5 shows that the hydraulic jump fluctuated near the end of the apron. In addition, the flow leaves the basin traveling towards the opposite bank of the forebay/afterbay basin and then along the toe of the slope until it enters the siphon inlet. The instability in the location of the jump is caused by the flow recirculation in the forebay/afterbay basin and the shear zones near the edges of the high velocity jet. Table 1 shows the highest velocities across the basin floor, 7.5 and 8.9 ft/s, were recorded at locations 3 and 6 in the middle of the jet about 18 and 70 ft, respectively, downstream from the end of the apron. After reaching the other side of the forebay/afterbay basin, a velocity of 8.5 ft/s was recorded along the toe of the embankment slope leading to the siphon inlet.



Figure 5. - Flow conditions in the forebay/afterbay basin for 1 ft gate openings as reservoir draining begins with shallow water in the forebay/afterbay basin.

Table 1. - Velocities measured in the forebay/afterbay basin downstream from the outlet structure for three flow conditions with the original geometry.

Location	Average velocity magnitude (ft/s) 1 ft gate openings≈630 ft ³ /s	Average velocity magnitude (ft/s) Gate control 1800 ft ³ /s	Average velocity magnitude (ft/s) Gates fully open 1800 ft ³ /s
1	4.48	0.60	6.39
2	1.72	0.45	6.25
3	7.52	8.69	6.15
4	1.27	10.40	3.86
5	6.70	10.32	7.67
6	8.88	10.98	6.02
7	1.45	3.83	1.43
8	3.02	6.10	1.59
9	8.51	11.37	1.20
10	3.41	11.05	8.79
11	3.49	3.86	1.33
12	4.02	6.37	0.64
13	0.25	0.82	0.11

Gate Control Passing 1800 ft³/s

Figure 6 shows the flow conditions with the gates set to control 1800 ft³/s through the outlet structure under reservoir El. 157 ft and shallow forebay/afterbay basin water depth. A gate opening of 4.31 ft was needed in the model to pass 1800 ft³/s under reservoir El. 157 ft. This may be different than the prototype because the gates were not modeled exactly.

Flow conditions were similar to those with the 1-ft-gate opening; however, the higher flow rate produced much more turbulence in the forebay/afterbay basin. Again the hydraulic jump extended beyond the end of the apron into the forebay/afterbay basin. The high-velocity release into the forebay/afterbay basin actually impacted upon the far 2:1 slope of the stepped soil cement producing run up and splitting the flow. After impacting on the far slope, a small portion of the flow recirculated in the area to the left of the outlet while the majority turned and traveled along the toe of the slope toward the siphon inlet.

Velocities are shown in table 1 and are about 11 ft/s downstream from the apron and along the toe of the slope heading toward the siphon inlet. The velocity magnitudes and vectors showing the flow direction are shown on figure 7. This overall flow condition was not acceptable due to the high velocities, the unstable hydraulic jump location, and the recirculation pattern.



Figure 6. - Flow conditions during initial reservoir draining to the forebay/afterbay basin under $1800 \text{ ft}^3/\text{s}$ with the gates controlling the reservoir to El. 157 with the original geometry. The left photo shows the high-velocity release extending across the forebay/afterbay basin. The right photo shows the hydraulic jump exiting the basin.

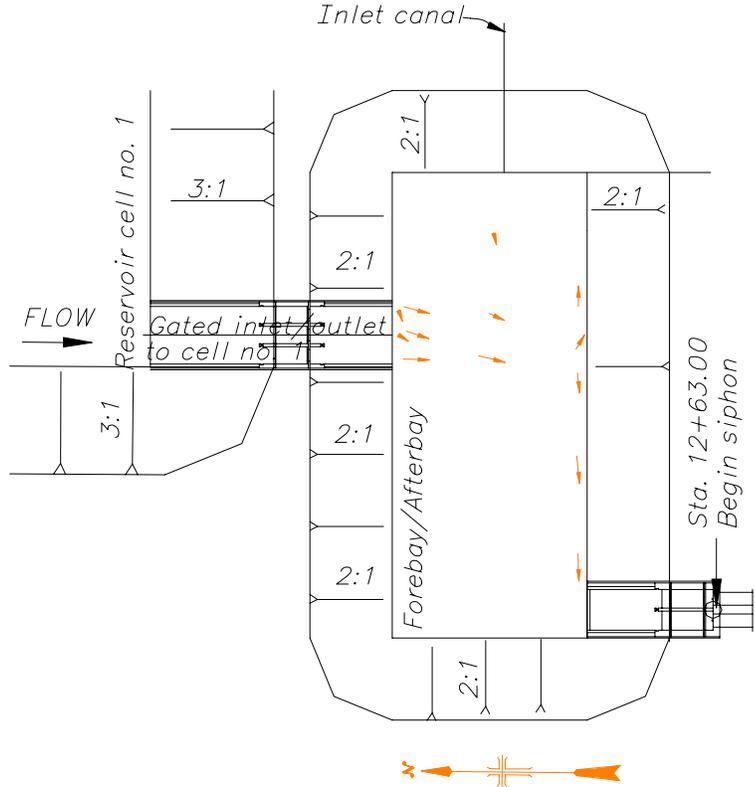


Figure 7. - Plan view of the velocity vectors showing the general direction of the flow when draining reservoir cell number 1. The velocity vector magnitudes are for $1800 \text{ ft}^3/\text{s}$ releasing under gate control and maximum reservoir El. 157 ft into the forebay/afterbay basin with water surface El. 140.

Gates Fully Open Passing 1800 ft³/s

The flow conditions were similar to the gated condition releasing 1800 ft³/s, except much less turbulent with the lower head driving the flow. During initial filling, the water level was controlled by the gate openings, but there was not enough energy for a hydraulic jump to form. Figure 8 shows the flow conditions with 1800 ft³/s through the outlet structure with the gates fully open and reservoir El. 142.72 ft. Velocities were again gathered, but as may be seen in table 1, they are much less than with gate control under maximum head in the reservoir. Velocities and flow conditions under this operation met the velocity criterion and were considered acceptable.



Figure 8. - Original geometry with 1800 ft³/s draining from the reservoir to the forebay/afterbay basin with gates fully open under initial filling.

Physical model testing indicated that when the reservoir water surface elevation dropped below El. 142.7 ft, the maximum discharge out of one reservoir cell was less than 1800 ft³/s. The HEC-RAS model used to initially evaluate filling and draining times showed a similar decrease in discharge at water surface El. 136 ft. Thus, the physical model data suggests that the actual drain time may be longer than expected. To resolve these differences, the timing of the filling and draining will yet be determined by the CFD modeling effort.

Reservoir Approach Conditions during Draining

The reservoir approach flow conditions were also observed during reservoir draining at 1800 ft³/s both with and without gate control. Figure 9 shows 1800 ft³/s flowing freely through the outlet structure looking down on the reservoir side. Upstream approach conditions from the reservoir were asymmetric due to the topography and the location of the structure. Flow directly entered the structure along the right embankment but swept across the left 3:1 embankment slope then flowed over the structure wall and turned to enter the gates. These flow conditions may cause additional head loss that could affect the reservoir draining time.

The change in flow direction so close to the gates, very noticeable under free flow conditions, produced twin vortices in the center bay of the gate structure near both piers, figure 10. The

vortices were strong enough to form an air core and draw lightweight paper confetti through the gates. More flow appeared to be drawn through the right bay looking downstream. The effect of these vortices should be considered in the operation and design of the gate structure.



Figure 9. - Top view of the reservoir draining approach conditions for 1800 ft³/s without gate control. Note the turbulent water surface caused by flow turning to enter the structure after flowing across the left embankment.



Figure 10. - Looking down into the center bay of the outlet structure showing vortices formed during reservoir draining with 1800 ft³/s under gate control. (Flow is from top to bottom).

Siphon Inlet

The siphon inlet was located in the model at the end of the abbreviated forebay/afterbay basin. The 40-ft basin width with side walls following the 2:1 stepped soil cement slopes was modeled with the floor at El. 133 ft. Tailboards were used for depth control. Flow from the outlet structure during reservoir draining swept across the forebay/afterbay basin and flowed along the basin slope toward the siphon inlet. As the flow reached the inlet it turned 90 degrees before flowing into the structure. As a result, a large contraction formed near the left sidewall and flow built up in the right side of the inlet, figure 11, producing undesirable flow conditions.

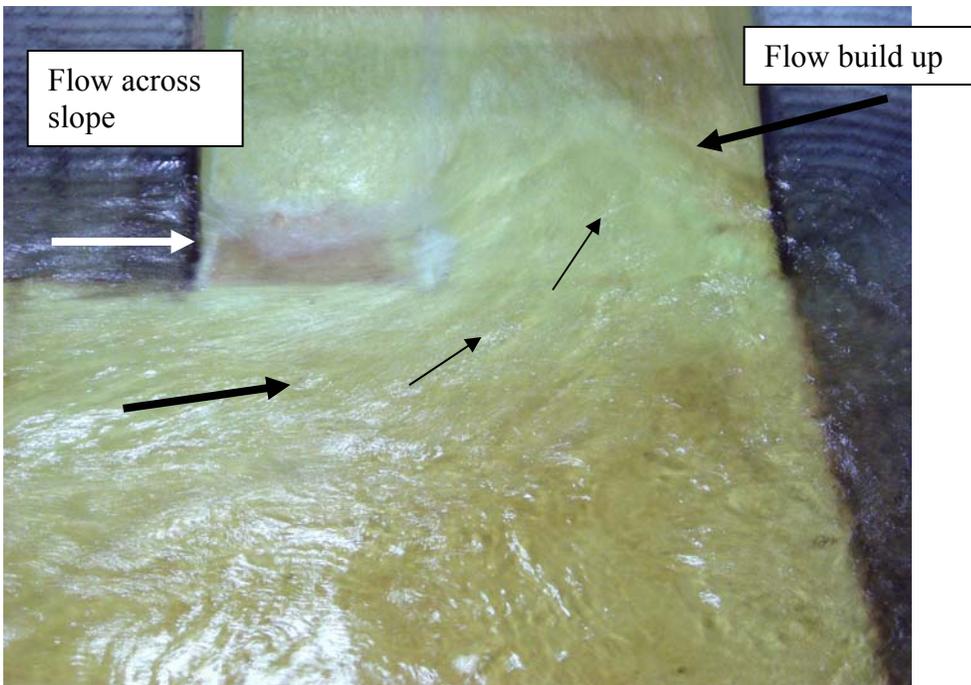


Figure 11. - Large contraction formed at the entrance to the siphon inlet under all initial flow conditions with flow from reservoir draining.

The siphon inlet geometry was not modeled; therefore, the CFD model was used to determine if the water surface in the forebay/afterbay basin used during this initial testing phase was correct. The numerical model indicated that the shallow water level in the physical model was probably deeper than would be expected under initial filling with the designed siphon inlet geometry. Therefore, the conditions observed in the physical model could be even more severe in the prototype unless the velocities can be significantly reduced from the outlet structures. Even if significant improvements can be made to the flow conditions, flow must travel a long way from reservoir cell No. 1 and turn sharply to enter the siphon inlet. It was felt that it would be better to realign the siphon inlet closer to the center of the two inlet/outlet structures to reduce head loss and improve flow conditions.

Modifications to the Original Outlet Structure for Reservoir Draining

Velocity magnitudes and flow conditions observed during the initial testing meant that improvements needed to be made to dissipate energy on the apron of the outlet structure during

reservoir draining. Although velocities were acceptable for some operating conditions over the soil cement on the forebay/afterbay basin floor, similar velocities would not be acceptable on the unprotected reservoir floor. Also, the high velocity flow along the toe of forebay/afterbay basin embankment adjacent to the siphon inlet structure produced turbulent flow into the siphon inlet structure.

Various floor blocks and sills were tested on the outlet structure apron in an attempt to dissipate energy. The modifications were evaluated for the 1-ft-gate opening operation with maximum reservoir El. 157 ft and minimum forebay/afterbay basin water surface El. 136 ft or 3 ft of depth.

The first attempt included installing a set of 1-ft-high by 1-ft-wide floor blocks evenly spaced across the three bays about 13.5 ft downstream from the end of the piers with a 1.3-ft-high sloping solid sill at the end of the apron. The flow impacted on the face of the blocks and shot up vertically without being submerged by the water depth created by the end sill. Therefore, the height of the first row of floor blocks was increased to 1.4 ft and moved to 32 ft downstream from the piers and various end sill configurations tested. End sill geometries evaluated included increasing the height of the sill using individually spaced blocks instead of a full width sill, and trying various sill locations on the apron.

Figure 12 shows the final attempt at energy dissipation with the final geometry obtained for the outlet structure operating under a 1-ft-gate opening during initial draining of the reservoir into a shallow forebay/afterbay basin. The end sill geometry consisted of a 0.84-ft-high sloping continuous sill across the width of the apron with 2.25-ft-high by 4-ft-wide dentates evenly spaced across the width. The downstream side of the end sill was located about 2.5 ft upstream from the end of the apron. In addition to the floor blocks and end sill, a ramped fillet was added to the floor in the center of each bay to spread the flow exiting the gates. The flow-spreading fillet is shown in gray in figure 12. The gates were narrower than the width of each bay producing a gap near each wall and/or pier. The flow spread to the walls then rebounded toward the middle of each bay concentrating the flow. The ramped fillet helped spread the flow more evenly across the apron.

Flow conditions leaving the basin were greatly improved with this geometry, but there was still quite a bit of turbulence exiting from the basin. Table 2 shows the velocity magnitudes measured for the 1-ft-gate opening during reservoir draining. Comparing the velocities obtained after the modifications to those in table 1 for the small gate opening, the maximum velocity was reduced from about 9 ft/s to just over 6 ft/s. The highest velocity occurred near the apron and velocities were reduced adjacent to the embankment across the forebay/afterbay basin so there was overall improvement in the flow condition.



Figure 12. - Final attempt at energy dissipation with the blocks and sill added to the original geometry for the outlet structure operating under a 1-ft-gate opening during initial draining of the reservoir into a shallow forebay/afterbay basin.

Table 2. - Velocities measured for 1-ft-gate openings for the final modifications to the original outlet structure geometry during reservoir draining.

Location	Average velocity magnitude (ft/s) 1 ft gate openings, 630 ft ³ /s
1	3.53
2	6.32
3	4.93
4	3.34
5	3.12
6	4.60
7	0.94
8	1.51
9	4.72
10	5.47
11	1.75
12	0.78
13	0.34

Unfortunately, an unexpected problem with dissipating energy on the apron at the same elevation as the gate seat was then discovered. Successful energy dissipation meant forcing the jump up onto the apron. However, if the toe of the jump reached the downstream nose of the pier it traveled upstream along either side of the pier or wall to the gate. The width of the opening downstream

from the gate is larger than the gate, causing space for the jet to attach to one side or the other of the pier or wall, figure 13.

The placement of flow spreaders on the floor in the center of each bay (dark rectangular pieces in figure 12) helped prevent the inception of the problem. However, once the jet attached to the side of the pier or wall it would not release and produced very non-uniform flow conditions on the apron that cancelled out the benefit previously gained with the addition of the blocks or sills to dissipate energy.



Figure 13. - Uneven flow distribution downstream from the outlet structure gates during reservoir draining prior to installing flow spreaders in each bay. Flow conditions are shown under intermediate testing of floor blocks to aid energy dissipation with a 1-ft-gate opening under reservoir El. 157 ft.

Summary of Modeling Results with the Original Geometry

The physical model showed that flow from the outlet structure during reservoir draining with a shallow water depth in the forebay/afterbay basin would produce velocities up to 11 ft/s and generally poor flow conditions. Flow exiting the outlet structure apron traveled across the 135-ft-wide basin and ran slightly up the 2:1 slope opposite the outlet structure. The flow then turned 90 degrees and headed toward the siphon inlet. A large contraction, producing head loss, formed where the jet turned to enter the siphon inlet. The hydraulic jump was formed downstream from the

end of the reinforced concrete apron until floor blocks and an end sill were added. The toe of the jump was unstable and combined with the instability of the jet issuing from the gates produced generally undesirable flow conditions.

Upstream approach conditions to the outlet structure during reservoir draining were also asymmetric. Vortices formed primarily in the middle bay of the gate structure as a result of the flow conditions entering the structure. Flow was aligned with the structure on the right side but turned sharply after flowing across the left embankment slope to enter the structure on that side. Vortex formation and asymmetric flow tend to increase head loss through the outlet.

The physical model of flow conditions during reservoir draining showed that velocities during reservoir filling would significantly exceed the target velocity of 1.5 ft/s for the sandy soil floor of the reservoir cells.

Operations were discussed at length during the initial testing phase. It was initially assumed in the design phase that to achieve the 1800 ft³/s discharge from the forebay/afterbay basin to the reservoir cells the reservoir gates would remain closed until the forebay/afterbay basin was full. The inlet gates would then be opened to discharge into the reservoirs under full head. Those operations would cause higher entrance velocities to the reservoir which would make energy dissipation much harder. Therefore, it is preferable to fill and drain with all gates fully open as much as possible. This conclusion was also supported by the initial runs with the CFD model FLOW-3D®.

An initial FLOW-3D® simulation of the canal inlet discharging 1800 ft³/s with the gates fully open showed velocities reaching 35 ft/s on the soil cement. It was felt that a baffled apron drop structure might improve flow conditions, and should be added to the physical model during modifications.

The physical model is only operated in a steady state. The CFD model is run with continuous time steps. Each modeling technique has its own advantages and was utilized during the initial testing of the Drop 2 reservoirs. The siphon inlet was not physically modeled due to the use of the CFD model. The results from both the physical and CFD modeling led to design team discussions of what design modifications to the original structures would be necessary to improve hydraulic performance.

Model Investigations of the Modified Structure Geometry

The physical model was revised based upon the results from the modeling of the original geometry for forebay/afterbay basin, a portion of the reservoir cell No. 1 with the inlet/outlet structure, and the siphon inlet. With the revision of the physical model it was decided to not continue the effort with the CFD model. Numerous modifications were necessary to update the CFD model and the computational times to reach stable flow conditions would be too long to be able to provide data in a reasonable time frame.

Description of Model Modifications

The following geometry changes were agreed upon by the design team and are shown on figure 14:

- Modify the geometry and location of the forebay/afterbay basin so that the inlet/outlet structures and siphon inlet would be repositioned within the basin without moving their locations. The siphon pipelines have many restrictions on their location given the existing highway; therefore, the location of the forebay/afterbay basin and length of canal were repositioned until the siphon inlet was centered between the two inlet/outlet structures in the forebay/afterbay basin. (The exact location of the canal inlet in the model could not be replicated because of constraints on the model extents.) The location of the dividing embankment between the two reservoir cells was also modified but did not impact the model.
- Flare the inlet/outlet and siphon inlet structure walls approximately 22 degrees through the embankment slopes to improve flow conditions through the entrances.
- The siphon inlet was centered between the two inlet/outlet structures and the approach invert dropped 10 ft to El. 123 ft. This then led to a gate bulkhead representing two fully-open 10 by 7 ft gates. (The siphon inlet approach was later changed to slope from El. 133 to 132 ft at the gate seat. In the final design, the gate seat was changed to El. 131 ft, but was not modeled.)
- Add an operational canal inlet structure to the hydraulic model to test the design of a baffled apron drop energy dissipator on the chute of the canal inlet structure and flow through the forebay/afterbay basin to the reservoir during filling.
- Reconstruct the inlet/outlet structures to add a lowered basin of 3-5 ft depth with baffle blocks on each side of the gate structure. The gate seat will remain at El. 133 ft. The structural modifications will remain internal to both inlet/outlet structures, however, the lowered portion may extend beyond the structure on the reservoir side and be protected with soil cement.

Once modifications to the physical model were complete, the original test plan was repeated with the intent of meeting the same design criteria. The revised design inlet/outlet structure geometries, siphon and inlet canal geometries, and their orientations are shown on figure 14. Of course, of the inlet/outlet structures, only the inlet/outlet structure for reservoir cell No. 1 was modeled.

Figure 15 shows the model as reconstructed. The basin remained rectangular and somewhat shorter than the design basin in length, but the canal structure was now operational and the siphon inlet was positioned correctly with the outlet from reservoir cell No. 1.

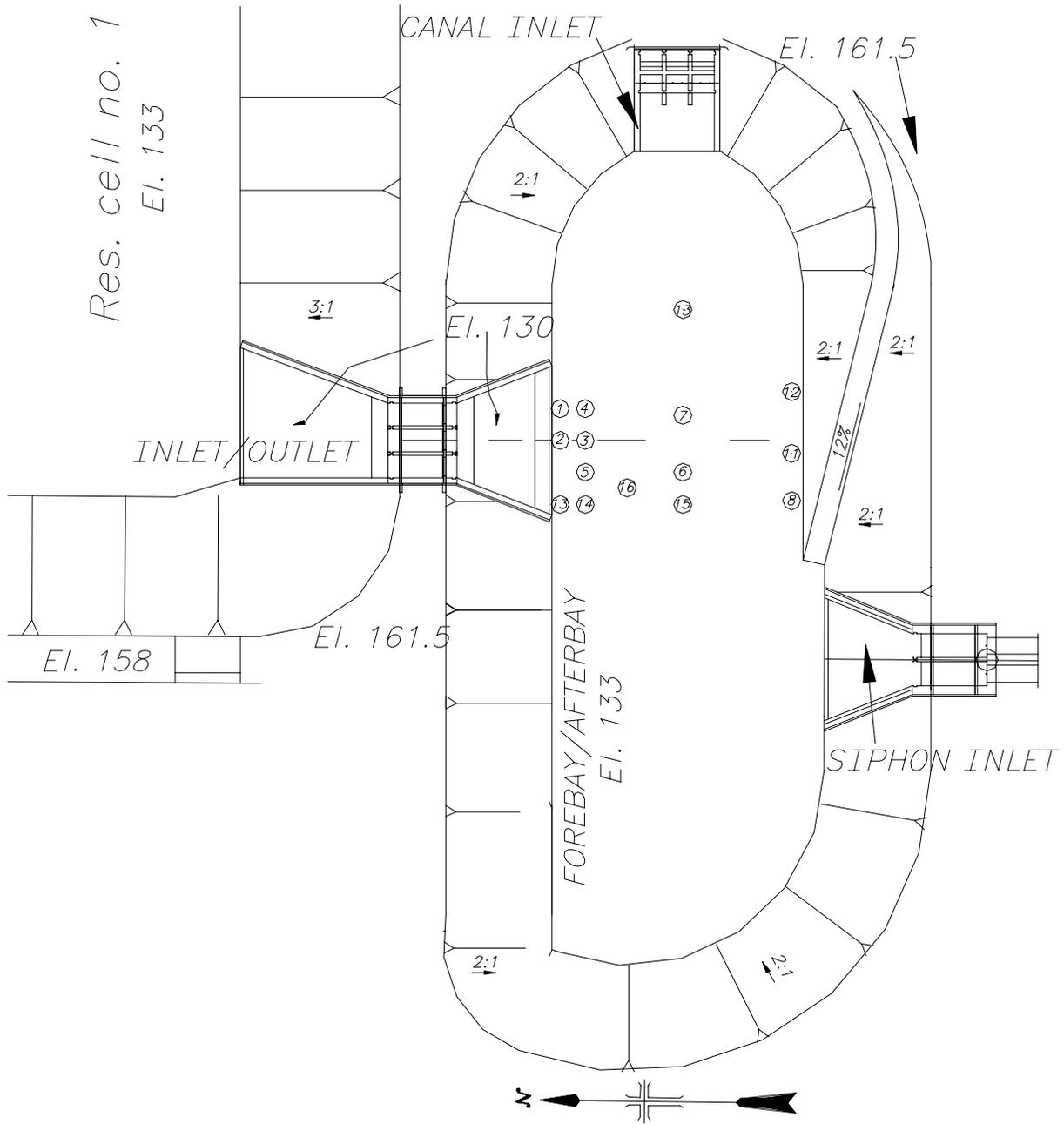


Figure 14. - Plan view of the design modifications made to the original forebay/afterbay basin and structures. Reservoir cell number 2 was again not modeled. The velocity locations where model data were taken during reservoir draining are also included.

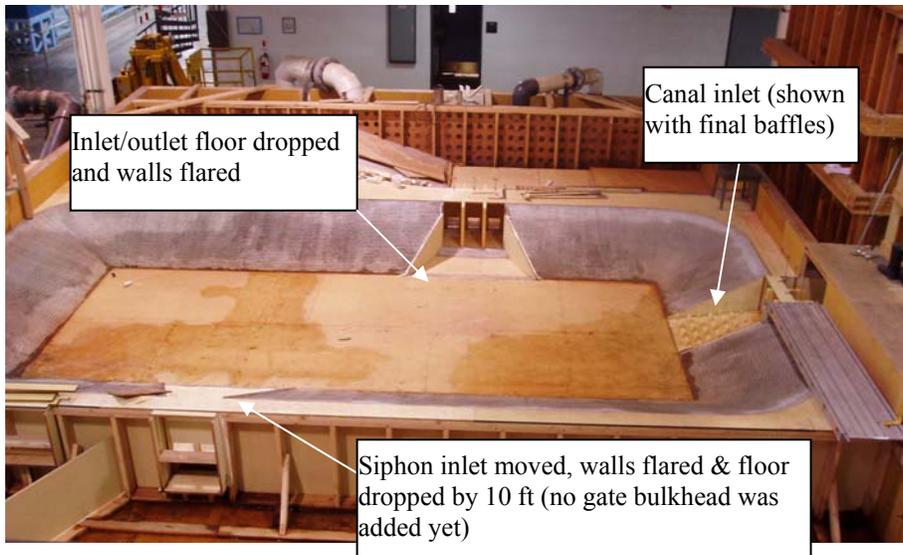


Figure 15. - Overall view of the physical model modifications including the canal inlet with flow capability, centering the siphon inlet, flaring the inlet/outlet and siphon inlet structure walls, and lowering the apron of the inlet/outlet structure.

Reservoir Draining Investigations

The first testing involved the flow conditions during reservoir draining through the modified outlet structure and new siphon inlet location and geometry. The following discussion is focused on evaluating various options to improve energy dissipation in the outlet structure on the forebay/afterbay basin side. The outlet structure geometry was modified to flare from the original 40-ft-width at the gates to 80-ft-wide at the end of the apron. The floor of the apron was dropped by 3 ft and a 3:1 sloping end sill was added to ramp up to the floor of the forebay/afterbay basin at El. 133 ft. Velocities for the basin modifications were taken primarily at the critical locations from the initial testing, or those locations in front of the jet exiting the basin. In addition, locations 13-16 were added to be downstream from the flared wall. Therefore, locations 3, 4, 5, 6, 14, and 15 as shown on figure 14, were documented while changes were being made to the outlet apron geometry. Velocities were measured with a water depth of about 4.7 ft in the forebay/afterbay basin for a discharge of $1800 \text{ ft}^3/\text{s}$ as a result of the siphon inlet floor constructed to El. 123 and no gate bulkhead.

The siphon inlet geometry was modified throughout the study period as the design progressed from concept to final design. Therefore, initial velocity data and final data were taken with various initial depths in the forebay/afterbay basin.

The flow case used for comparison of the various baffle block configurations was $1800 \text{ ft}^3/\text{s}$ under gate control up to reservoir El. 154 ft as this condition produced the worst flow conditions from the previous test program. Table 3 and figures 16-21 show the geometries in the outlet basin that were tested and the resulting flow conditions. Photographs of the baffle block arrangements tested with a description of the geometries and flow conditions follow.

Table 3. - Tests completed to determine appropriate geometry for the inlet/outlet structure after the major modifications.

Test Name	Geometry
	All outlet geometries had a 3 foot drop in the floor and flared walls producing an 80 ft width at the downstream end of a 3:1 sill ramping up to the forebay/afterbay basin El. 133 ft. Siphon inlet with 10 ft drop to El. 123 and no gate bulkhead
Rest1aa	No baffles, – figure 16.
Resm1	2 rows of 3-ft-high baffle blocks - figure 17.
Rest2a	3 rows of 3-ft-high baffle blocks - figure 18.
Rest3a	3 rows of 3-ft-high baffle blocks with the end blocks removed from both sides of row 2 and the blocks equally spaced. The height of two middle blocks in row 2 and 1 middle block in row 3 was increased to 4.5 ft – figure 19.
Rest4a	3 rows of blocks with first 2 located the same as test rest3a and the third row moved to the floor of the forebay/afterbay basin just downstream from the end sill – figure 20.
Rest5a	No baffles with the siphon modified producing higher water level in the forebay/afterbay basin of about 9.75 ft. – Figure 21.

Figure 16 shows the initial test of reservoir draining with no baffles on the floor of the outlet basin floor and minimal forebay/afterbay basin depth. The flared walls cause an asymmetrical jump to form in the basin with flow recirculation on the left side, looking downstream, and the jet exiting the basin on the right side toward the siphon inlet. Velocities were measured at all the locations previously used, plus the additional locations added for these tests. The highest velocity was measured on the right side of the basin where the jet exited. Velocities up to 19 ft/s were observed in this area as shown in table 4, test Rest1aa.



Figure 16. - Initial tests with $1800 \text{ ft}^3/\text{s}$ draining from the reservoir at El. 154 ft with a 3 ft drop in the inlet/outlet basin floor and 3:1 downstream sill transitioning back to the forebay/afterbay basin El. 133 ft. Left photo is looking down on the basin and right photo is looking into the basin from the forebay/afterbay basin. Test Rest1aa.

Figure 17 shows the first modification to the outlet basin with 2 rows of 3-ft-high by 5-ft-wide blocks centered on the floor (test ResM1). The addition of the baffle blocks produced a more uniform jump in the basin, but the same general flow pattern existed. Velocities were measured at the additional locations only and reached a maximum of 13 ft/s (table 4).



Figure 17. - First modification with $1800 \text{ ft}^3/\text{s}$ draining from the reservoir at El. 154 ft with two rows of 3-ft-high baffle blocks in the dropped floor of the inlet/outlet basin floor. Left photo is looking down on the basin and right photo is looking into the basin from the forebay/afterbay basin. Test ResM1.

Table 4. - Velocities for initial draining of the reservoir into the forebay/afterbay basin with various stilling basin baffle geometries and siphon inlet geometries producing different tailwater levels. All data gathered under gate control with 1800 ft³/s and reservoir El. 154 ft. The x axis is the location of the velocity measurement, and the values in the table are the velocity magnitudes represented by the bars for each test condition.

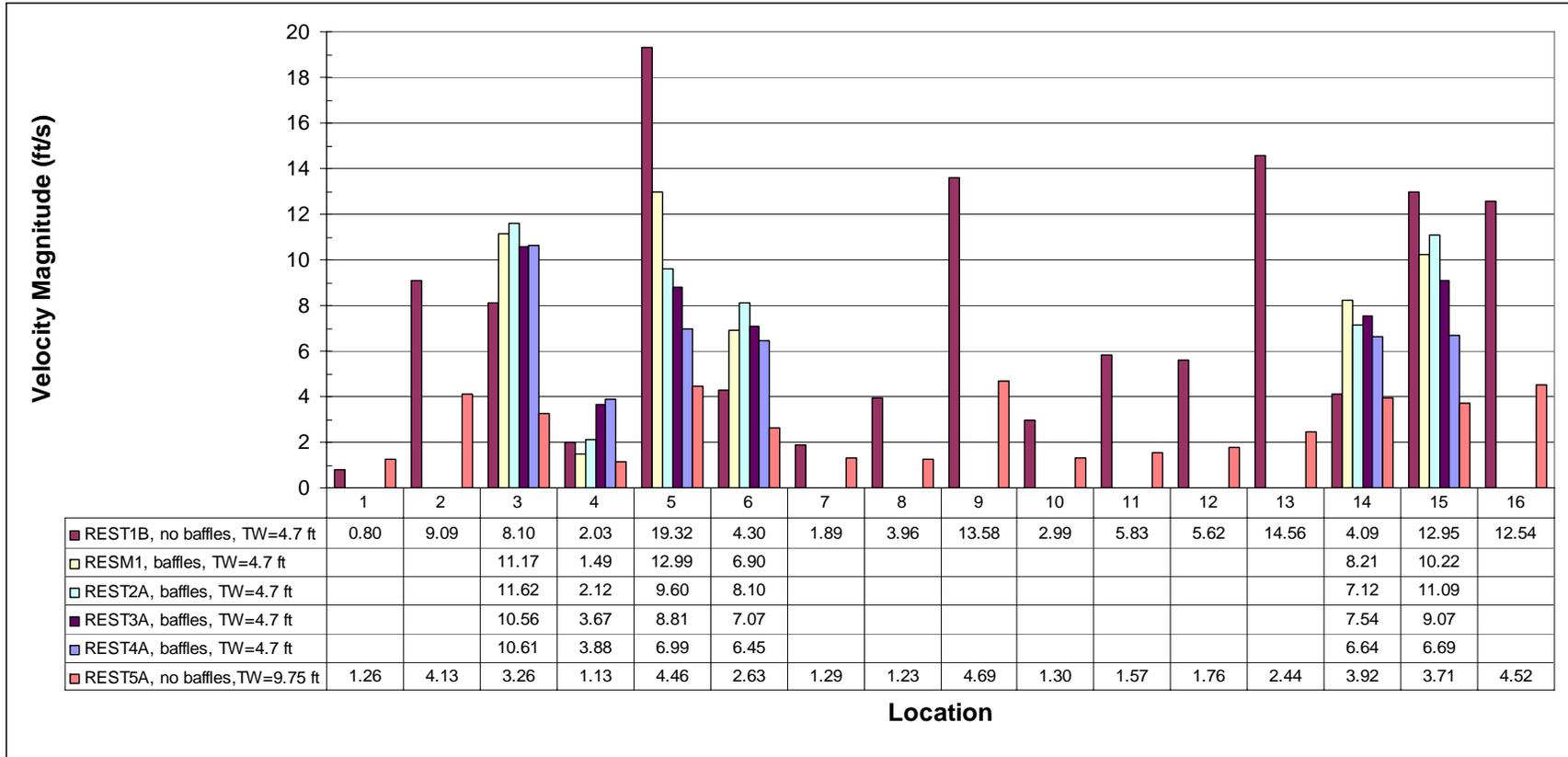


Figure 18 shows the next modification to the outlet basin floor, Rest2a, where a third row of the same dimension blocks were added downstream. Table 4 shows the location of the maximum velocity changed and the magnitude decreased to about 11.6 ft/s. However, the same general flow pattern existed and the target velocity of 10 ft/s for the soil cement was still exceeded.



Figure 18. - Second modification with 1800 ft³/s draining from the reservoir at El. 154 ft with three rows of 3-ft-high baffle blocks in the dropped floor of the inlet/outlet basin floor. Left photo is looking down on the basin and right photo is looking into the basin from the forebay/afterbay basin. Test Rest2a.

Figure 19 shows modification Rest3a with the end blocks of row 2 removed and blocks made higher in the main flow path of the jet. This modification was attempting to break up the jet caused by the recirculation zone on the left side of the basin. The maximum velocity, table 4, was reduced to 10.6 ft/s but no appreciable difference was seen in the flow pattern.



Figure 19. - Third modification with 1800 ft³/s draining from the reservoir at El. 154 ft with three rows of 3-ft-high baffle blocks with the end blocks removed from both sides of row 2 and the blocks equally spaced. The height of two middle blocks in row 2 and 1 middle block in row 3 was increased to 4.5 ft. Left photo is looking down on the basin and right photo is looking into the basin from the forebay/afterbay basin. Test Rest3a.

Figure 20 shows the final attempt at modifying the basin with baffle blocks. A fourth row of same dimensioned blocks was added at the end of the sill of the inlet/outlet basin. These blocks actually protruded above the elevation of the forebay/afterbay basin floor and would therefore, cause additional loss during reservoir draining. The flow pattern was slightly improved and the velocities reduced overall, but the maximum velocity remained at about 10.6 ft/s, table 4. This velocity was right at the target velocity, but the complexity of the block geometry was unacceptable.



Figure 20. - Fourth modification with 1800 ft³/s draining from the reservoir at El. 154 ft with three rows of baffle blocks. The first three rows are the same as the previous test, Rest3a, with the third row moved downstream to the top of the end sill in the forebay/afterbay basin. Left photo is looking down on the basin and right photo is looking into the basin from the forebay/afterbay basin. Test Rest4a.

In general, it was difficult to say that much improvement was accomplished with continued addition of baffles. Table 4 showed the velocities had a decreasing trend, but the flow conditions, figures 16-20, did not indicate as much improvement as was expected.

While the next tactic for improving flow conditions was being determined, the siphon inlet geometry was modified by the design team to reflect structural design considerations. The modified siphon inlet structure affected the water level in the forebay/afterbay basin by producing a tailwater for both reservoir draining and canal inlet flows. Meaningful velocity data could not be gathered while the water level was changing, so the data was gathered after the gate openings were set for each test flow condition and the water level had stabilized in the forebay/afterbay basin. The water level in the forebay/afterbay basin affected the ability of the structures to dissipate energy and reduce the velocity to which the soil cement is subjected to, particularly during initial filling.

A gate bulkhead was installed in the siphon inlet to match the two-10-ft-wide by 7-ft-tall gate openings and the siphon inlet floor was modified to slope from the forebay/afterbay basin El. 133 ft to El 132 ft at the gate bulkhead, figure 21. Figure 21 also shows the flow conditions into the final geometry for the siphon inlet. Siphon inlet flow conditions were greatly improved by repositioning the inlet, flaring the walls to the gate structure, and improving energy dissipation of the flow entering the forebay/afterbay basin. Comparison of flow conditions between figures 11 and 21 confirms this observation.

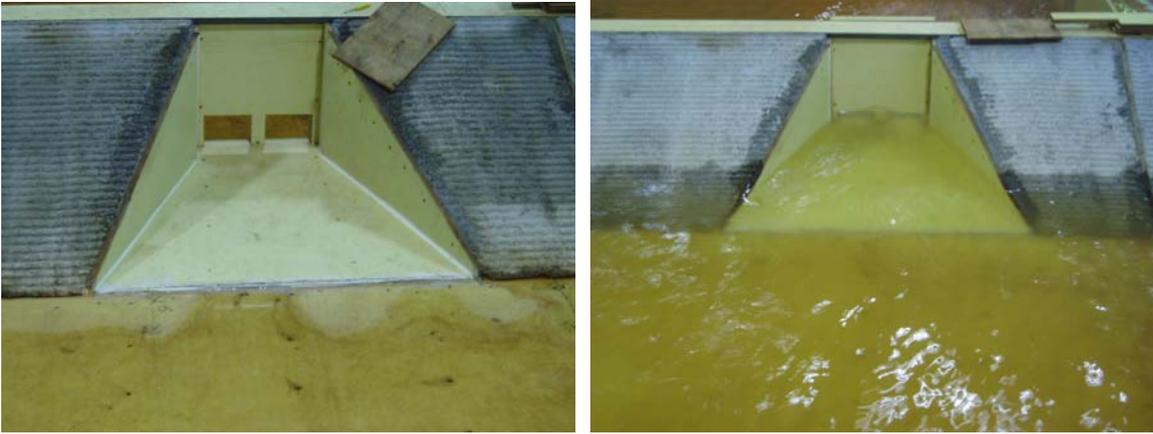


Figure 21. - Siphon inlet as modeled with 66-ft-wide opening sloping in to the gate bulkhead with 10-ft-wide by 7-ft-high gate openings and invert sloping from El. 133 to 132 ft. Right photo shows operation with 1800 ft³/s and gate control from reservoir draining.

Discussions were also held at this time with the design team about the sustainable velocity limit for the soil cement, the time it would take to fill the forebay/afterbay basin to various depths, exposing the soil cement to high velocities for a potentially short time period, and about the complexity of the baffle designs.

Two different initial water levels in the forebay/afterbay basin of 9.75 ft and 7.8 ft were investigated under 1800 ft³/s with the gates controlling to reservoir El. 154 ft. These water levels were both greater than had been previously investigated; therefore, it was decided to remove all the baffles and perform the basic test again.

Velocities were taken with 1800 ft³/s under reservoir El. 154 ft and the initial forebay/afterbay basin water level of 9.75 ft (test Rest5a). Velocities may be compared to those from the initial tests and lower initial basin water level by looking at the maroon and pink bars on table 4. The difference in the siphon geometry, initially modeled with no bulkhead (maroon) and a depth of 4.7 ft in the forebay/afterbay basin, and the higher water level in basin with the bulkhead (pink) and a depth of 9.75 ft produced a significant reduction in the velocities. The higher initial water level in the forebay/afterbay basin produced velocities significantly beneath the target value of 10 ft/s with no baffles in the outlet basin.

Recommended Design

The final design of the outlet basin during reservoir draining was delayed until:

- Computations could be made to determine the filling time of the forebay/afterbay basin.
- Testing could be accomplished with soil cement samples to determine if the soil cement would withstand higher velocities than originally assumed.
- Canal inlet testing could be accomplished to compare velocities.

Computations were made to determine the time to fill the forebay/afterbay basin to various levels. Velocity criteria will be exceeded while filling up to a water depth in the forebay/afterbay basin that produces enough energy dissipation. The projected time to fill the forebay/afterbay basin to 4.7 ft,

where most of the testing was performed, would be about 3 minutes. The forebay/afterbay basin depth was measured to be 7.8 ft during operation under 1800 ft³/s and reservoir El. 154 ft. Filling time would be about 5.6 minutes from an empty basin or an additional 2.6 minutes above where the previous testing had been conducted.

Fast filling of the forebay/afterbay basin, subsequent testing of the canal inlet flow conditions, and results from the soil cement erosion testing [Bartojay, 2007], led to the decision to remove all baffle blocks from the outlet basin floor for the final design.

Figure 22 shows the final design of the outlet structure under 1800 ft³/s with reservoir El. 154 ft during reservoir draining under a forebay/afterbay basin water level of 7.8 ft. Comparison with the initial tests in figure 16 shows the greater energy dissipation accomplished with additional depth in the forebay/afterbay basin. Velocities were not measured for this intermediate water level, but because the flow conditions were very similar to those with the 9.75 ft water level, they were assumed to be acceptable.



Figure 22. - Final design of the inlet/outlet structure operating with 1800 ft³/s draining from the reservoir at El. 154 ft with no baffle blocks on the floor. Water level in the forebay/afterbay basin of 7.8 ft was produced by 10-ft-wide by 7-ft-high gate openings in the bulkhead. Left photo is looking down on the basin and right photo is looking into the basin from the forebay/afterbay basin.

The water level in the forebay/afterbay basin was recorded at about El. 141.5 ft or 8.5 ft under 1800 ft³/s with no outlet gate control, but control by the siphon inlet geometry. This information would later be utilized to determine whether or not adequate filling and draining times were attained. Lowering the apron below the invert of the gate seat eliminated the instability in the toe of the jump that caused the jet to attach to the side of the pier or walls.

Observations were also made of the approach to the structure on the reservoir side. Whereas in testing with the previous straight walls on the outlet structure showed the formation of strong vortices, the flared walls improved approach flow conditions and the vortices disappeared.

Additional discussion about the canal inlet tests and soil cement erosion testing follow.

Canal Inlet Investigations

The canal inlet structure with the capability to pass flow from the gate structure down the sloping floor to the invert of the forebay/afterbay basin was a new feature constructed in the physical model. The canal inlet consisted of the three gates seated at invert El. 140.5 ft discharging onto a 0.747 ft/ft sloping chute to the invert of the forebay/afterbay basin at El. 133 ft. During the reorientation of the forebay/afterbay basin, the station for the canal inlet also changed and the chute slope lengthened as much as possible to increase energy dissipation. All initial testing was performed with a forebay/afterbay basin depth of 4.7 ft. The canal inlet energy dissipation was investigated first by flowing into the forebay/afterbay basin and out the siphon inlet. In reality, the flow would go through the inlet/outlet structures and into the reservoir cells. Flow from the canal is used to fill the reservoir and this condition will be reported in a later section.

Figure 23 shows initial operation of the canal inlet with highly fluctuating flow conditions in the forebay/afterbay basin. The toe of the hydraulic jump intermittently swept well downstream from the end of the inlet slope then moved back to the toe of the slope. The jet also moved from side to side producing erratic recirculating and/or sweeping out of the flow adjacent to the canal inlet. This flow condition was unacceptable regardless of the velocity magnitudes measured.

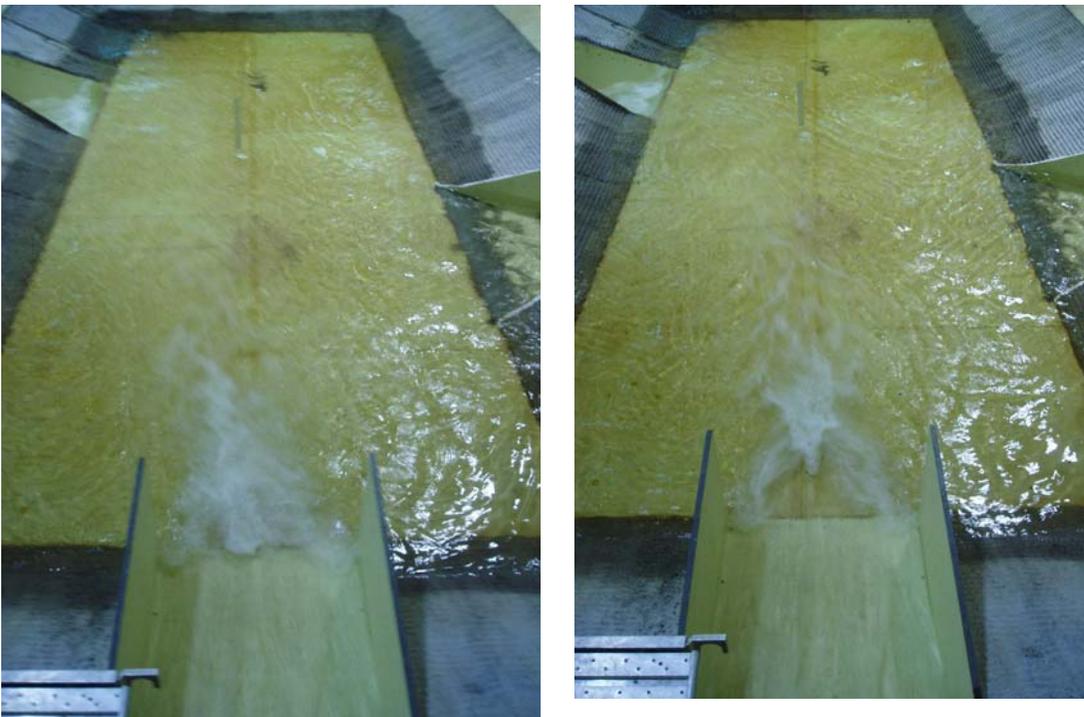


Figure 23. - Original design of the canal inlet with 1800 ft³/s under gate control flowing into a shallow forebay/afterbay basin with a depth of 4.7 ft.

It had been expected that baffles would need to be added to improve performance of the system and reduce velocities on the soil cement. Baffles were designed for the sloping apron of the canal inlet, even though the design was not typical. Baffled apron drops are usually utilized downstream from a free-flow ogee-type crest instead of the gated structure controlling the canal releases [Peterka, 1978]. The design included a row of transitional blocks at the end of the gate seat invert at

El. 140.5 ft in an arrangement referred to as the Fujimoto entrance [Rhone, 1977]. These transitional blocks formed a continuous width across the chute with 5-ft-wide wedges alternating between 4.5 and 2.25-ft-long sections. Five rows of baffle blocks were added to the slope starting 20.5 ft downstream from the break in slope. The blocks were 3-ft-high and 5-ft-wide spaced at 6 ft intervals down the slope. The blocks were alternately spaced across the width with every other row starting with a ½ block at the wall.

The flow conditions for this geometry are shown in figure 24 operating under a discharge of 1800 ft³/s and gate control to water surface El. 154 ft in the canal. Flow conditions in the forebay/afterbay basin under operation with the initial baffle geometry were greatly improved, with the jet stabilized and the hydraulic jump formed near the end of the chute. However, the flow impinged heavily on the first row of blocks causing undesirable conditions on the baffled chute.

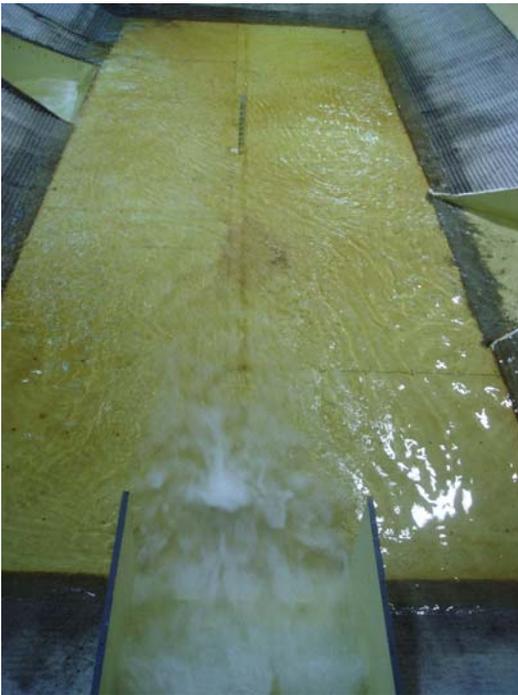


Figure 24. - Initial baffles added to slope with 1800 ft³/s under gate control flowing into the forebay/afterbay basin with a 4.7 ft depth.

As a result, an additional row of shorter baffles and another row of the same baffles were added upstream from the initial baffles on the slope, with the top of the baffles flush with the elevation of the gate seat at El. 140.5 ft. This geometry, which became the recommended geometry, is shown in figure 25. Figure 26 shows the flow condition in the forebay/afterbay basin and on the chute with the full set of baffle blocks installed. The photos both show operation under the minimal forebay/afterbay basin water level of 4.7 ft. The photo on the left is looking down on the chute under fully open gates and shows decreased impingement on the blocks and good flow conditions in the chute. The photo on the right shows that with 1800 ft³/s under gate control flow conditions were greatly improved entering the forebay/afterbay basin.



Figure 25. - Looking upstream on the slope of the final baffled apron drop geometry for the canal inlet.

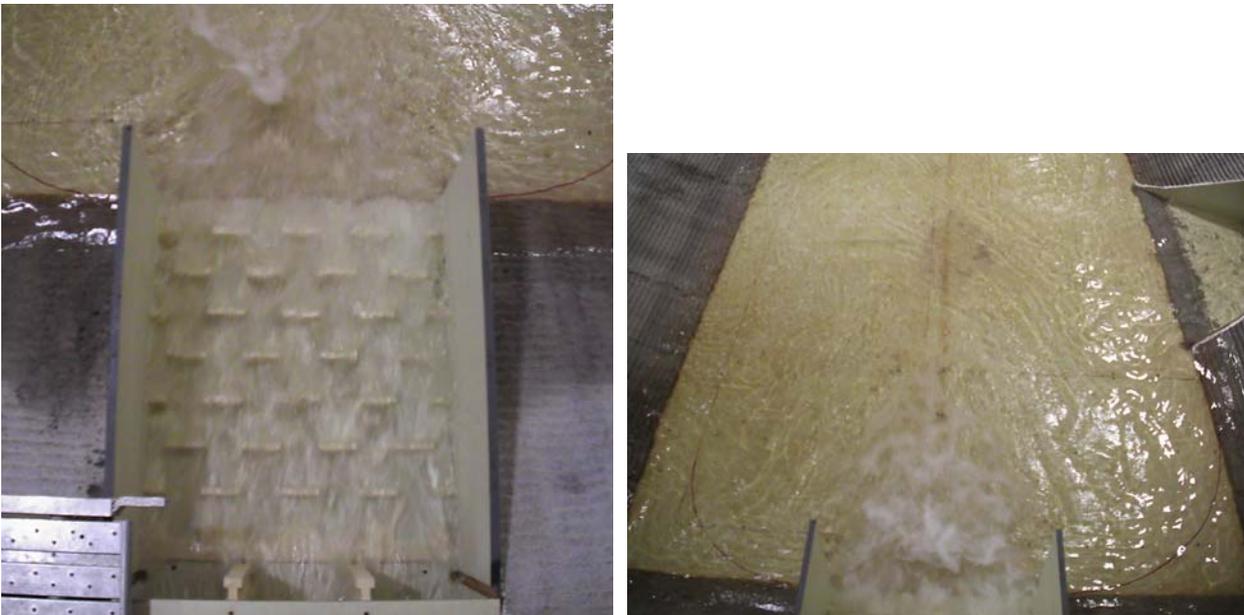


Figure 26. - Final canal inlet geometry with 1800 ft³/s flowing into the forebay/afterbay basin with the gates fully open (left) and gates controlling (right).

Figure 27 shows the final baffled apron drop geometry operating under a forebay/afterbay basin depth of 9.95 ft produced by a siphon modification and canal inlet gate settings. The hydraulic jump moves further up onto the chute producing better energy dissipation and reducing the velocities to which the soil cement would be subjected. (The side walls of the chute where eventually trimmed to match the profile of the 2:1 stepped soil cement slope).



Figure 27. - Final canal inlet chute design with higher forebay/afterbay basin water level of 9.95 ft under a discharge of 1800 ft³/s with gate control under canal water surface El. 154 ft.

Canal Inlet Velocities

Velocities were measured for the initial design with no baffles and each of the subsequent modifications as baffles were added. All velocities were measured at the locations shown on figure 28. Locations A-D are along the centerline of the canal inlet, E-G are located about 15 ft to the left of centerline, looking downstream. Locations H and I are along the toe of the 2:1 embankment slope and location J in front of the siphon inlet. Three flow conditions were investigated under a forebay/afterbay basin water depth of 4.7 ft under 1800 ft³/s and about 3 ft with 550 ft³/s. All velocities are shown in table 5 for the various modifications, including the final geometry with all baffles.

The velocities measured under the smaller discharge were, as expected, less than the target velocity and of no concern. Under 1800 ft³/s the velocities, in general, exceeded the target whether with or without canal inlet gate control. The velocities measured with the initial chute and no baffle blocks were actually lower than some measured after the addition of some of the blocks. This was due to the transient nature of the flow exiting the chute without the baffles which resulted in periods where the jet was not at the location of the instrument. When the average velocity over the same time period (with and without baffles) was computed, the values ended up less than what would be expected without the baffles given observations of the flow conditions. Intermediate baffle arrangements did not produce improvement because high impact on the chute baffles produced poor flow conditions both in the chute and downstream.

The fully baffled chute produced greatly improved flow conditions with the toe of the jump consistently formed at the base of the chute with a water depth of 4.7 ft in the forebay/afterbay basin. The 10 ft/s soil cement erosion target velocity was still exceeded; however, reaching 14.4 ft/s at the centerline location 60 ft downstream from the end of the chute.

Table 6 shows the result of the velocities measured for the final baffle geometry for the canal inlet for the three flow conditions reported in table 5 with the addition of the velocity data under the

siphon modification that produced a 9.95 ft water level in the forebay/afterbay basin. Table 6 clearly shows the advantage of operating with a higher initial water level in the forebay/afterbay basin as all the velocities were reduced. The higher initial water level produced velocities that met the target value of 10 ft/s with the maximum recorded velocity of about 7 ft/s.

Final modifications to the siphon inlet produced a forebay/afterbay basin water level between the two tested values. In addition, the filling time for the forebay/afterbay basin is expected to be the same as that computed for reservoir draining.

At this point in the study, decision regarding time to fill the forebay/afterbay basin and the ability of the soil cement to resist erosion needed to be made. It would be difficult to perform additional meaningful changes to the canal inlet chute that would significantly reduce the velocities.

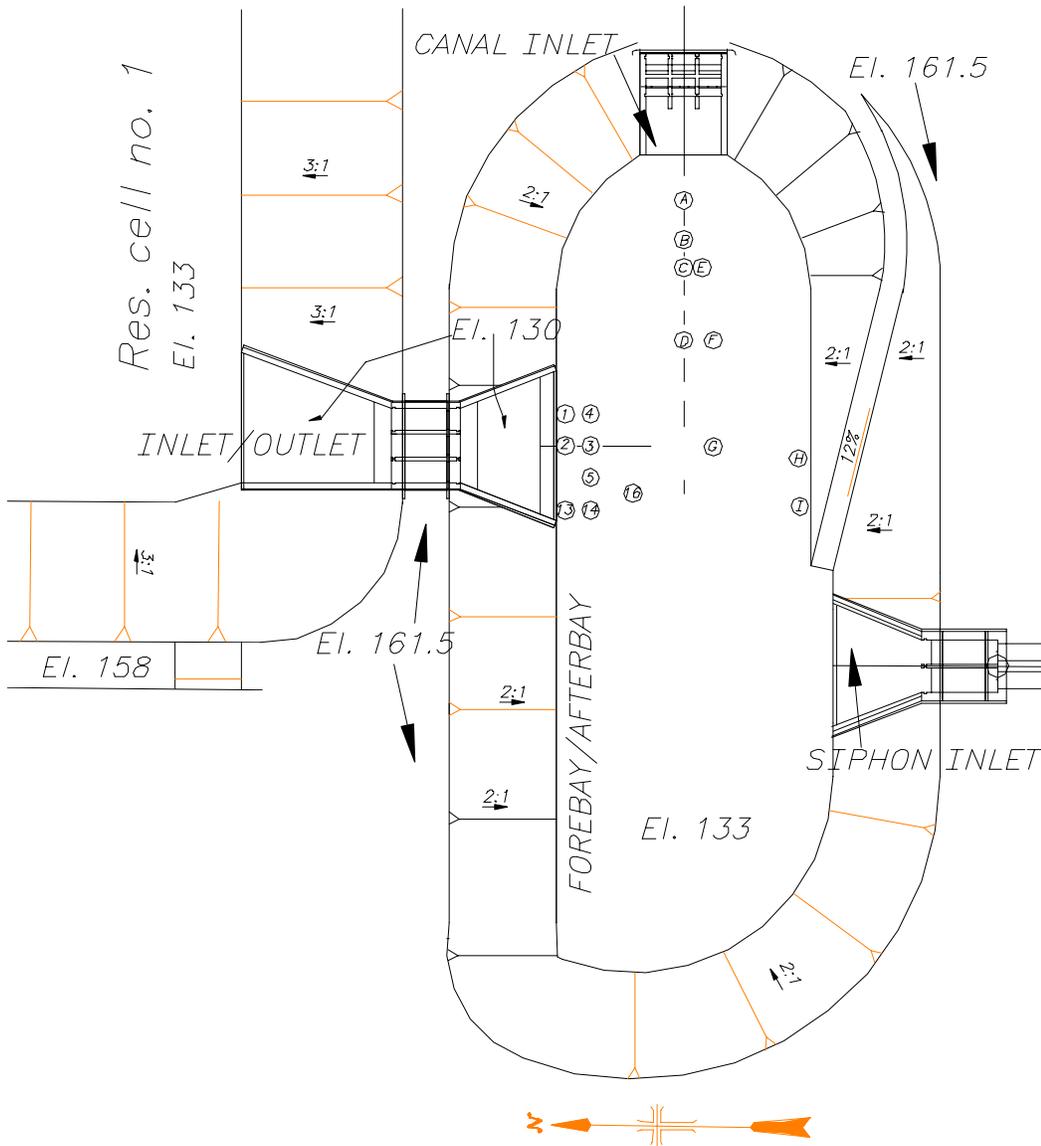


Figure 28. - Plan view of the locations where velocity measurements were taken for the canal inlet and final inlet/outlet structure geometries.

Table 5. - Velocities measured in the forebay/afterbay basin below the canal inlet during testing of the initial geometry with no baffles, through to the final baffle design, under the shallow basin depth of 4.7 ft produced by no gate bulkhead in the siphon inlet.

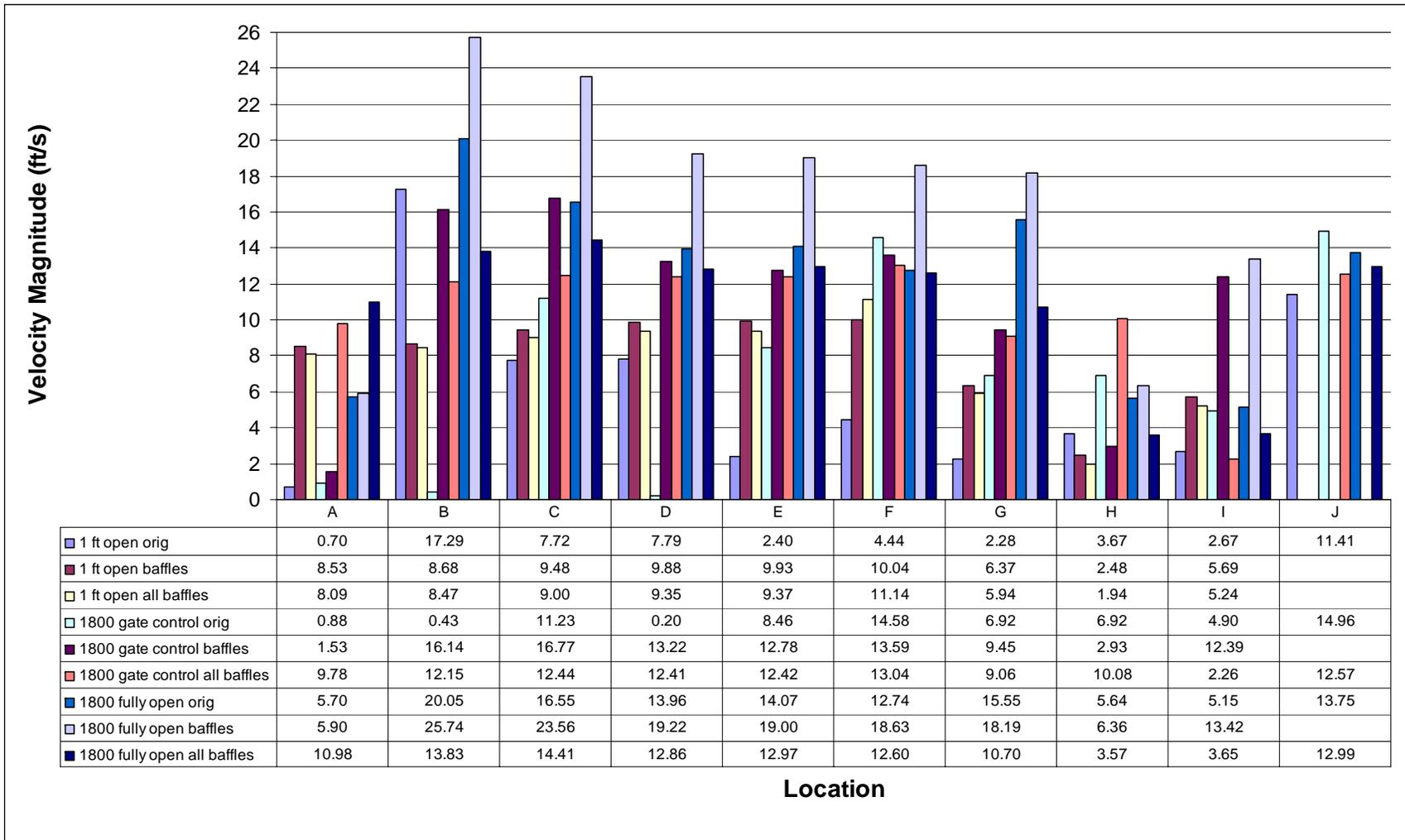
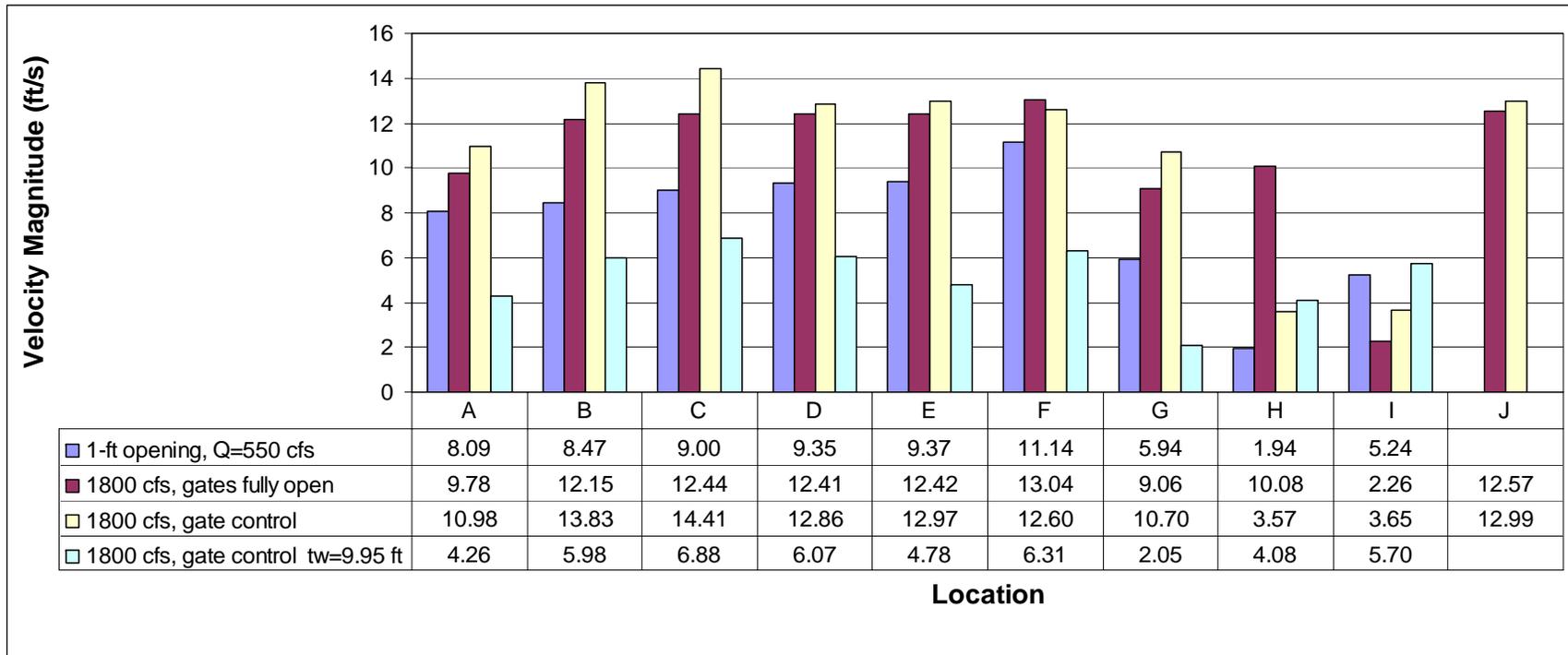


Table 6. - Comparison of velocities under various operations with two different forebay/afterbay basin depths with the final baffle geometry on the canal inlet slope.



Final Canal Inlet Test with Reservoir Filling

The final test with the canal inlet was to investigate flow conditions under reservoir filling with 1800 ft³/s and a canal water surface El. 154 ft. In this test, the siphon inlet was closed. The objective was to test the worst case scenario of the minimum forebay/afterbay basin water level of 9.95 ft with the inlet/outlet gates fully open and flow into an empty reservoir. This flow condition is shown in figure 29. Velocities were measured in the flow path from the canal to the reservoir inlet structure and the results are shown in table 7. Locations for the measurements are shown in figure 28 and were a combination of those used for canal inlet flow and reservoir draining flow.

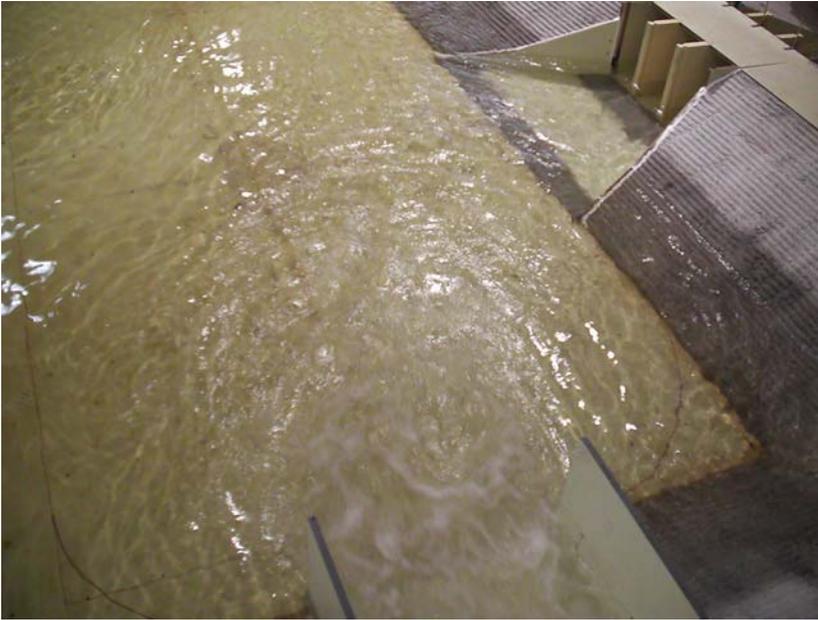
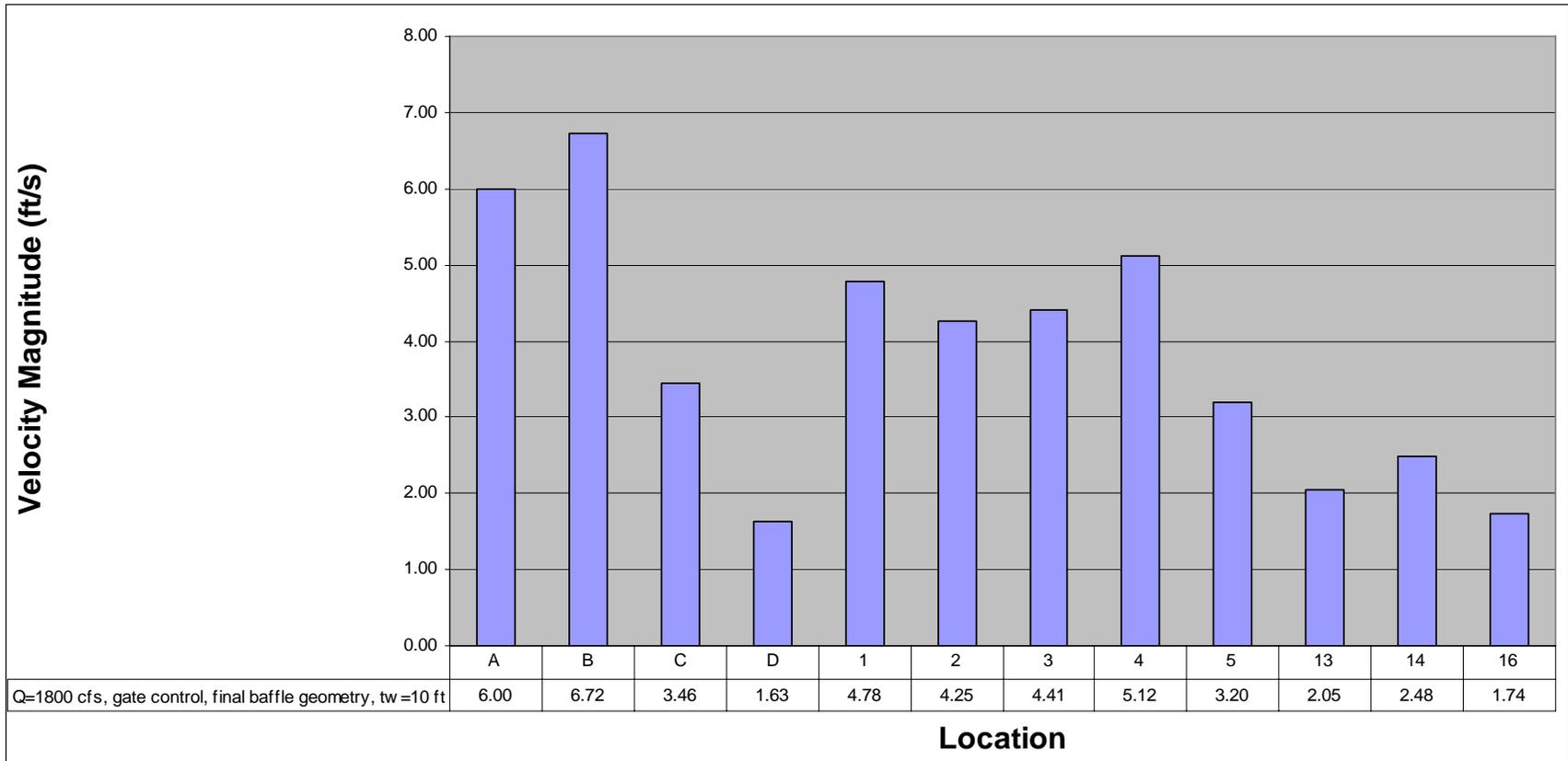


Figure 29. - Flow of 1800 ft³/s with gate control from the final canal inlet geometry through the forebay/afterbay basin and through the reservoir inlet structure to an empty reservoir.

Flow conditions were good with no velocities exceeding the 10 ft/s initial soil cement erosion criteria. Table 7 shows the trend of decreasing velocity with distance from the canal chute, then an increase in front of the reservoir inlet structure prior to entering the basin, and a decrease again near the outside flared wall. This flow pattern improved with higher water levels in the forebay/afterbay basin.

This test also confirmed that the location proposed for a water level sensor in the forebay/afterbay basin, centered between the two inlet/outlet structures, should be acceptable with minimal turbulence.

Table 7. - Flow velocities for 1800 ft³/s under canal water surface El. 154 from the canal into the forebay/afterbay basin and in front of the inlet/outlet structure under reservoir filling with a water level of 10 ft in the forebay/afterbay basin.



Soil Cement Erosion Testing

The objective of the hydraulic model study was to investigate the flow conditions into and out of the Drop 2 Storage reservoirs. The investigation included measuring velocities as a means to compare modifications to the structures that would improve energy dissipation and reduce the potential for erosion in the forebay/afterbay basin and the reservoir.

Model study results were discussed during team meetings. The target maximum velocity of 10 ft/s for the soil cement in the forebay/afterbay basin invert and along the embankment slopes came from the literature.

Improvements were made to the canal inlet and the reservoir outlet structure design. Despite these improvements, with shallow depths in the forebay/afterbay basin, velocities exceeded the 10 ft/s target for short durations while the forebay/afterbay basin filled. The forebay/afterbay basin would fill to a depth of 4.7 ft in about 3 minutes and 9.75 ft in about 7.5 minutes under the conditions tested.

Key factors in determining whether or not the soil cement covered forebay/afterbay basin would erode are:

- The length of time that the soil cement would be subjected to the initial maximum velocities.
- The ability of the soil cement to withstand the initial velocity.

The length of time to fill the forebay/afterbay basin with 1800 ft³/s entering it is relatively very short. Assuming a rectangular bottom with 2:1 side wall with a footprint of 135 by 438 ft the estimated time to entirely fill the basin to El. 154 ft would be approximately 22 minutes.

Due to uncertainty regarding the ability of the soil cement to withstand the initial velocity, some samples were prepared and cured for 7 days and jet tested in a test apparatus constructed and used by Tony Wahl, Hydraulic Engineer, USBR, Hydraulic Investigations and Laboratory Services, 86-68460, Denver, CO. Even though this test directed flow vertically onto the samples, the resulting energy and shear stress imparted would be expected to be the same as with a parallel jet such that as expected from the Drop 2 structures.

Jet tests were run on two specimens, ER1 and ER2. There was no measurable erosion on either sample. Sample ER1 was tested for one hour at pressure heads increasing from 6.5 ft up to 55 ft. Sample ER2 was tested for 30 minutes at 55 ft of head. Jet velocities directed at the specimen during these tests ranged from 20.5 to 59.7 ft/s. Peak shear stresses applied to the surface of the specimens varied from 3.4 to 29.3 lbs/ft². The testing at higher pressures was done in the hydraulics lab using tap water pressure, in hopes that some measurable erosion could be produced. (The drain system in the soils lab could not handle the flow corresponding to such high pressures).

These velocities and the resulting shear stresses far exceeded any measured in the physical model. Therefore, the soil cement was considered competent to withstand the initial velocities and the geometry of the canal inlet and the outlet from the reservoir to the forebay/afterbay basin could be

finalized based on the model study observations. The design team discussed the possibility that even though the soil cement may withstand very high velocities, construction techniques and an abrupt change in slope at the end of the structures, might require that a reinforced concrete pad extend 10-12 ft out from the end of the structures.

Further information regarding the results of this testing and other soil cement investigations may be found in “Laboratory Studies of Soil-Cement Materials for Drop 2 Storage Structures Colorado River Front Work and Levee System, CA All American Canal Drop 2 Storage Reservoirs, Lower Colorado Region” [Bartojay, 2007].

Siphon Inlet and Pipeline Concerns

As mentioned previously, the siphon inlet design evolved throughout the study period and only the portion from the forebay/afterbay basin to the gate bulkhead was modeled. The design team still had some questions about the size of the drop inlet basin where the flow discharged from the gates, the flow conditions into the two 9-ft-diameter pipelines, and air demand or blowback possibilities in the siphon pipes. The pipelines will pass under the existing Evan Hewes highway and discharge into a short open channel canal section forming the confluence with the existing All American canal. These paragraphs address consultation given separate from the hydraulic modeling effort.

The gate seat elevation was set based upon the elevation needed to control reservoir draining time. It is assumed that the backwater computation through the pipelines was correct and that the gate seat will not submerge. Given this assumption, the length of the basin was investigated. The basin width seemed adequate. The jet trajectory from the gates was computed and would impact on the basin floor prior to the pipeline entrance. The basin was then treated as a drop structure with infinite run out. Design guidelines from Chow, 1959 and Natural Resources Conservation Service, Gulp, 1968 recommended about a 24-ft-long basin with a depth of 12.5 ft. Therefore, the basin depth of 15 ft and length of 30 ft, respectively, should be adequate as designed, assuming similar performance.

The jet entering the relatively small basin will create enough turbulence that air will be drawn into the pipelines. Using the geometry of the pipelines and discharge, the direction the air would move and the location of an air vent were determined using Falvey, 1980. The very mild slope and figure 29 indicated that the air should move downstream and not blowback into the basin. A distance to the air vent of 150 to 300 ft would be needed to capture the air assuming a bubble rise time ranging from 0.41 to 0.82 ft/s and the 9 ft diameter pipe flowing with a velocity of 14 ft/s. The design calls for a vent at 30 ft downstream from the inlet of the pipelines and another 60 ft upstream from where the pipelines will daylight into the open canal section. The downstream design location for the air vent is also upstream from the flow meter to ensure that air will be evacuated before the meter location. Some air will be released at the vent 30 ft downstream; however, it would be anticipated that there will still be air in the pipelines to be vented near the end of the pipelines and upstream from the flow meter.

Reservoir Draining Time

Draining times for the reservoir cells were initially going to be determined using FLOW-3D®, a CFD model that would track the flow from the reservoirs through the forebay/afterbay basin and out the siphon inlet. The geometry for the model was developed for the initial design, but because of computational time required and changes to various structure geometries, the numerical modeling could not be completed within the design schedule. Therefore, the physical model was used to provide data for comparison with the designer's HEC-RAS model to determine if the reservoir cells could be drained within the required 72 hours.

Ratings were developed using the physical model for both the original model geometry and the final design including the flared inlet/outlet structures, the flared siphon inlet with the sill at El. 132 ft, and two 10-ft-wide by 7-ft-high gate bulkhead openings simulating fully open gates. Figure 30, shows the rating developed with no baffles on the floor of the basins and the outlet structure gates fully open. The water depth was measured in the reservoir with a stilling well and point gage and in the forebay/afterbay basin by a tape measurement. Flow was measured by the laboratory Venturi system. As shown in figure 30, the ratings were quite similar for the original and final designs, with divergence appearing at the lower and upper ends of the curve. Table 8 shows the initial elevations and discharges predicted by the HEC-RAS program and those determined by the physical model.

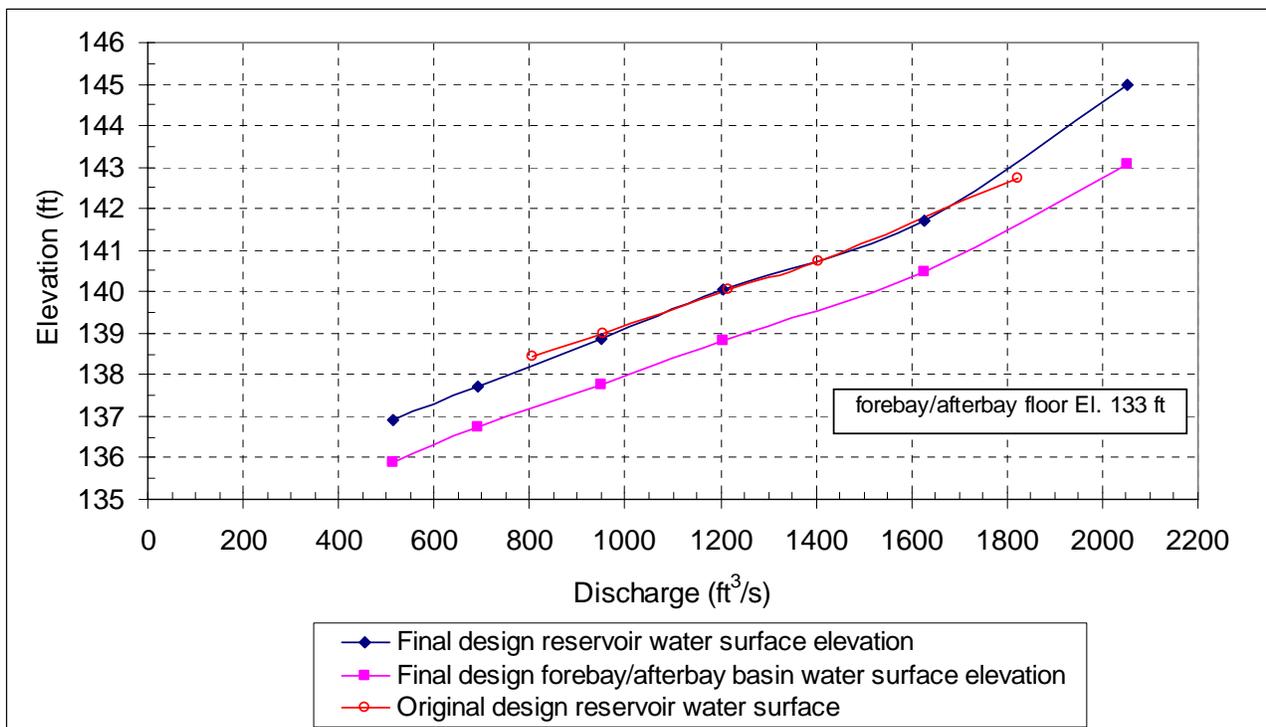


Figure 30. - Measured water surface elevations in the forebay/afterbay basin and reservoir with the outlet structure gates and siphon inlet gates wide open. During the final testing, the siphon inlet gate seat elevation was 132 ft and the structure side walls were flared.

Table 8. - Results for draining the reservoir for both the initial and final design geometries.

Initial Design		Final Design			
Discharge (ft ³ /s)	Reservoir El. (ft)	Discharge (ft ³ /s)	Reservoir El. (ft)	Forebay/afterbay Basin El. (ft)	Forebay/afterbay Basin Depth (ft)
806.50	138.44	513.66	136.91	135.91	2.91
953.31	139.01	691.53	137.72	136.75	3.75
1214.86	140.04	948.49	138.87	137.76	4.76
1406.38	140.74	1206.53	140.04	138.82	5.82
1821.94	142.72	1625.40	141.73	140.50	7.50
		2051.84	144.97	143.05	10.05

As a result of the ratings, the design team adjusted the gate seat on the siphon inlet down 1 ft to El. 131 ft to attain the needed draining time for the system based on the HEC-RAS model.

Reservoir Filling Investigations

The final testing involved initial filling of the reservoir. This testing was performed last after completion of the soil cement testing and decisions on design of the other structures. The goal was to have a design that would produce velocities less than 1.5 ft/s exiting into the reservoir. This velocity was assumed based upon the sand layer that will be covering the geomembrane liner on the invert of the reservoir. Previous testing with filling the forebay/afterbay basin from the canal and from draining the reservoir provided insight into how difficult it would be to accomplish the criteria. Flow filling the reservoir exits onto the inlet apron that is 25 ft longer on the reservoir side than on the outlet forebay/afterbay basin side.

Various types of blocks, vanes, and sill arrangements were utilized in an attempt to reduce velocities exiting the inlet structure basin by forcing a hydraulic jump and spreading the flow as much as possible. All tests were performed with 1800 ft³/s and forebay/afterbay basin water surface El. 154 ft. Methods used to reduce velocities entering the reservoir during initial filling were:

- Lowering the apron invert of the inlet structure by 3 ft to El. 130 ft with no end sill.
- Adding a 6:1 end sill on the inlet structure apron sloping up to the reservoir invert at El. 133 ft.
- Adding a 3-ft-high curved downstream sill at the end of the structure extending into the reservoir as much as the model extents would permit. The area upstream of the curved sill could be protected by reinforced concrete or soil cement producing additional space for energy dissipation. Floor blocks were then added on the reinforced concrete apron in the following geometries:
 - Three rows of 3-ft-high spaced floor blocks on the apron of the inlet structure apron located normal to the flow.

- Removing the first set of blocks then adding 1 row of 3-ft-high spaced floor blocks angled to the flow about 125 degrees.
 - Two rows of 3-ft-high spaced floor blocks angled to the flow about 125 degrees.
 - Three rows of 3-ft-high spaced floor blocks angled to the flow about 125 degrees.
 - Evaluating performance and reducing the number of blocks in each row as much as possible.
- Investigating non-uniform gate operations to assist with spreading of the flow.

These investigations were carried out in the model with the objective of improving flow conditions, although there was not a great deal of confidence that the 1.5 ft/s velocity could be attained after previous testing with the reservoir draining. The following discussion shows the progression of the testing and the flow conditions observed.

The model was initially modified on the reservoir side with the apron extended out at El 130 ft to the extent allowed in the model. Prior to testing, a pipe was added to the model to allow the reservoir to drain back out into the laboratory channel. Figure 31 shows the initial testing with supercritical flow exiting the gates onto the apron and traveling across the apron to the baffle wall in the model. This allowed testing and observations of initial reservoir filling. Velocities could not be measured; however, due to the very shallow depths present during initial filling and space restrictions in the model.



Figure 31. - Flow conditions with initial reservoir filling with an extended flat apron at El. 130 ft.

The concept design had a 6:1 sloping end sill from the inlet apron at El. 130 ft to the reservoir invert at El. 133 ft. This is shown on figure 32 with a short section of flat representing the reservoir invert at El. 133 ft at the downstream end of the sill. The left photo shows the supercritical flow sweeping out the end of the basin and the model. The right photo shows the influence of the flared wall with

a jump formed on the left side of the basin caused by the eddy on the right side that pushed the jet to the left. The left embankment slope confined the flow even more.



Figure 32. - Initial reservoir filling and sweep out of the flow on the left with the floor at El. 130 ft and 6:1 downstream sill up to reservoir invert El. 133 ft. The photo on the right shows a slightly filled reservoir and a non-uniform hydraulic jump formed along the left side of the inlet structure apron.

Figure 33 shows a sequence of flow conditions with a 3-ft-high ramped sill extended up to El. 133 ft that utilized the space available in the model reservoir area. The sill would extend on a 70 ft radius from the intersection of the left structure wall and the toe of the 3:1 embankment slope in the prototype. The three flow conditions in figure 33 show the basin sweeping out, and the jump forming still on the left side of the basin and progressively moving upstream as the reservoir fills. The longer apron area formed by the sill extension into the reservoir did show promise, but performance was still not adequate. Vanes were temporarily added to the apron floor to investigate redirecting and spreading large volumes of the flow. This led to the use of blocks to spread the flow exiting the basin, thereby reducing both flow concentrations and velocities on the left side.

Several rows of blocks were investigated, first oriented normal to the flow, then at the subsequently final angle to the flow. The blocks were all 2.25-ft-wide by 3 ft high with the spacing the same as the block width and offset from each other in each row. The number of blocks was reduced until the minimum that would spread the flow was attained. The block installation was confined to the extents of the reinforced concrete apron area in the prototype, or the downstream end of the sloping side walls of the basin.



Figure 33. - A sequence of flow conditions are shown with a 3-ft-high curved end sill with a 3:1 ramp formed in the model across the 70-ft-width at the end of the inlet/outlet basin. From left to right the flow conditions are shown progressively filling the reservoir. The flow progressed from sweeping out of the basin, to the beginning of the jump formation on the left of the apron after a slight buildup of water in the reservoir (looking down), to a side view showing the jump still located on the left side of the basin but moved upstream.

Initial impingement on the floor blocks caused the flow to shoot up vertically for a short period of time before the area upstream from the sill began to fill with water and submerge the blocks, figure 34 (left photograph). This condition caused excessive turbulence and splash until the blocks were submerged by water filling the apron area and the jump moving upstream, as shown in the right photograph of figure 34. The blocks were angled at 125 degrees from the left side wall of the basin. The three rows of a minimal number of blocks did an adequate job in spreading the flow over the end sill in the model based upon visual observations. This geometry was the final geometry determined and tested in the hydraulic model. The plan view of the geometry as submitted to the design team is shown on figure 35.



Figure 34. - Flow conditions during initial reservoir filling with the final block pattern and simulated 70-ft-radius end sill extending into the reservoir under uniform gate operations.

The flow conditions appeared to be quite uniform over the modeled end sill, thus achieving as much spread and energy dissipation as seemed possible. The velocities would; however, still exceed the target value for initial exposure of the sandy reservoir floor. The difference between the modeled sill length and the length of 110 ft produced by an arc with a 70-ft radius is actually quite significant. The flow conditions over the true length of the radius that could not be modeled due to model space constraints would need to be extrapolated and were expected to be better than what was observed in the model.

Gate Operations

The model gates were not motorized so the model was operated by setting the gates to predetermined openings for the desired water levels and flow rates. Uniform gate openings were used to make observations and gather data. All worst case scenarios dealt with initial filling or draining with no or minimal initial depth in the basins. Non-uniform gate operations were investigated as a method to assist in spreading the flow because of the great potential for erodibility during reservoir filling.

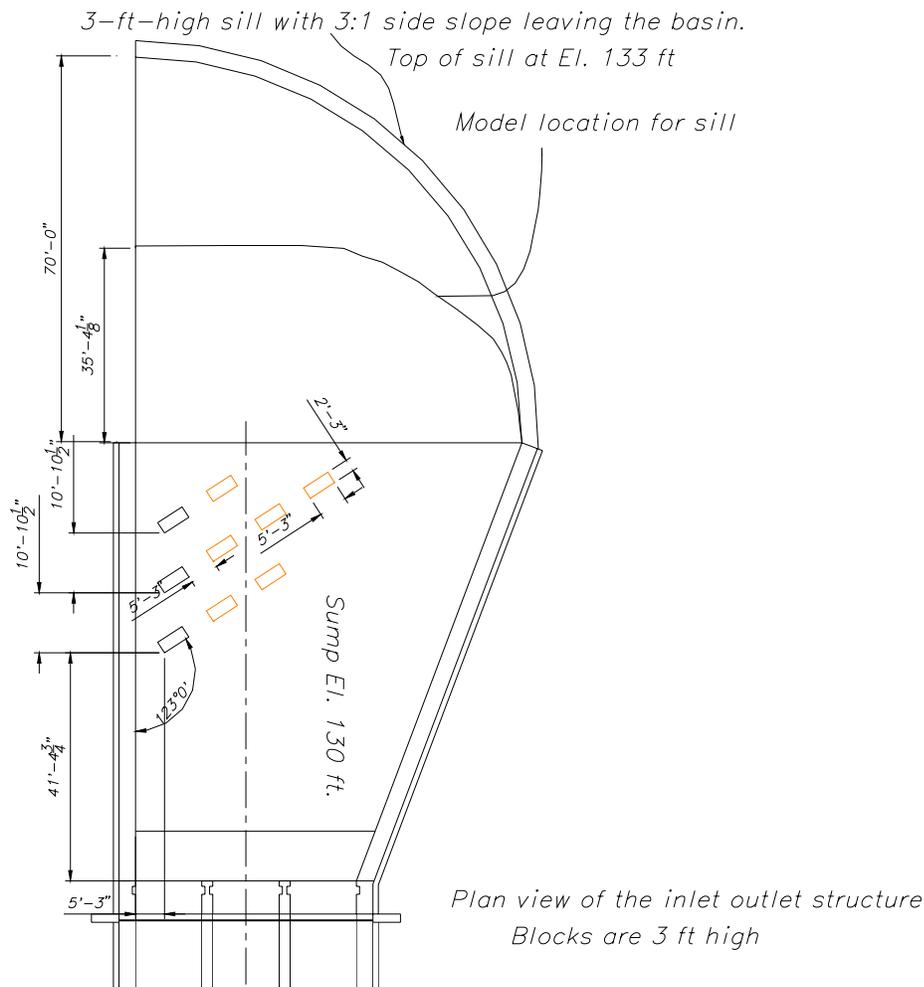


Figure 35. - Plan view of the model and prototype replication of the final block and sill geometries for the inlet structure during reservoir filling. The final geometry investigated in the hydraulic model is shown with the floor blocks arranged at a 125 degree angle to the flow to spread the flow over the shortened 70-ft-radius sill. Flow is from bottom to top.

The gate operation that spread the flow the most evenly was to open the right then middle then left gates looking from the forebay/afterbay basin into the reservoir. The gates will open about 1 ft per minute in the prototype so the gates will quickly open in this sequence. The model was operated with uniform openings and then individual gates opened or closed incrementally from that point while keeping the forebay/afterbay basin water surface elevation as close as possible to El 154 ft. The eventual settings were 2.2, 4.5, and 4.5 ft open from right to left, respectively, looking downstream, to pass 1800 ft³/s through the gates under forebay/afterbay basin El. 154 ft. During continued filling, the gates could continue to be brought up non-uniformly until they are fully open. This sequencing produced the best distribution in flow exiting over the sill in the model as may be seen in figure 36. The exact opening may vary in the prototype but the opening order could be replicated with the middle and left gates opened more than the right gate. The reservoirs are too big for tailwater to build up prior to the gates fully opening and the gates must continue to open to meet

filling criteria. The right gate was opened first, but to a smaller opening which reduced flow along the flared wall where water recirculated and aided in the flow concentration on the left. Flow conditions seemed to be somewhat improved with non-uniform gate openings as may be seen by comparing figures 34 and 36.



Figure 36. - Flow conditions under the final geometry modeled with the end sill and 3 rows of a minimal number of blocks angled at 125 degrees across the apron under 1800 ft³/s and El. 154 ft with non-uniform gate operation.

Recommended Design for Reservoir Filling

The possibility that the velocity could not be adequately reduced for the sand invert of the reservoir using structural modifications developed from the model investigations became clear. The final block and sill configuration was presented to the design team and discussions ensued to determine the best way to proceed from both technical and economical standpoints.

Using continuity, the target velocity of 1.5 ft/s would mean an exceptionally long sill would be needed to reduce the depth over the sill for a release of 1800 ft³/s. This would not be economical or practical. Computation of the required sequent depth indicated that there was no way for an adequate jump to form in the basin under initial filling. Computations, made to look at forcing a jump with an abrupt rise or a sloping floor also indicated that it would be too high to construct and would hamper reservoir draining [Chow, 1959].

The flow depth and velocity over the sill were assumed to be critical and calculated from:

$$d_c = \sqrt[3]{\frac{q^2}{g}} \quad \text{And} \quad v_c = \sqrt{gd_c}$$

Using critical depth and various radius sills produced the results shown in table 9 for a discharge of 1800 ft³/s during initial reservoir filling.

Table 9. - Table of possible sill radii with lengths produced and hydraulic parameters for a total discharge of 1800 ft³/s.

Sill radius (ft)	¼ Arc ($2\pi r/4$) length into reservoir (ft)	Unit discharge (ft ³ /s/ft)	Critical depth (ft)	Critical velocity (ft/s)	Assumed velocity for conservative design (ft/s)
70	109.95	16.37	2.01	8.1	10
125	196.34	9.17	1.37	6.7	*
250	392.69	4.58	0.87	5.3	7
500	785.38	2.29	0.55	4.2	*
750	1178.06	1.53	0.42	3.6	*

*Only the 70 and 250 ft sill radii were actually considered further in design.

The design team discussed the cost of the various length sills and the potential benefit in reducing the velocity to the target of 1.5 ft/s. The cost of soil cement versus availability and effectiveness of riprap was debated. In addition, a design using flow through the rock was investigated [Frizell, 1998, Lorenz, et.al, 2000]. Unfortunately, most designs include rock on a slope and no direct conclusion could be made, only the use of engineering judgment regarding the adequacy of flow through the rock in reducing velocities.

The final design was based upon the above described process and was a combination of the physical model results, computations, engineering judgment, and projected cost effectiveness. The final geometry for the energy dissipation structure is shown on figure 38. It includes the initial reinforced concrete apron at El. 130 ft with the floor block geometry developed in the hydraulic model. This then leads to a soil cement area extending out to the 250-ft-radius soil cement sloping sill that ramps up to the reservoir invert at El. 133 ft. Out from the sill is a 100-ft-wide section of riprap that will then lead to the sandy protection over the geomembrane throughout the rest of the reservoir. It was decided that flow through the rock would decrease velocities to an acceptable level and help distribute the flow as well. Limitations in the model space did not allow for the full sills to be modeled. Therefore, the performance of the final design is based on engineering judgment and is undocumented in the model study.

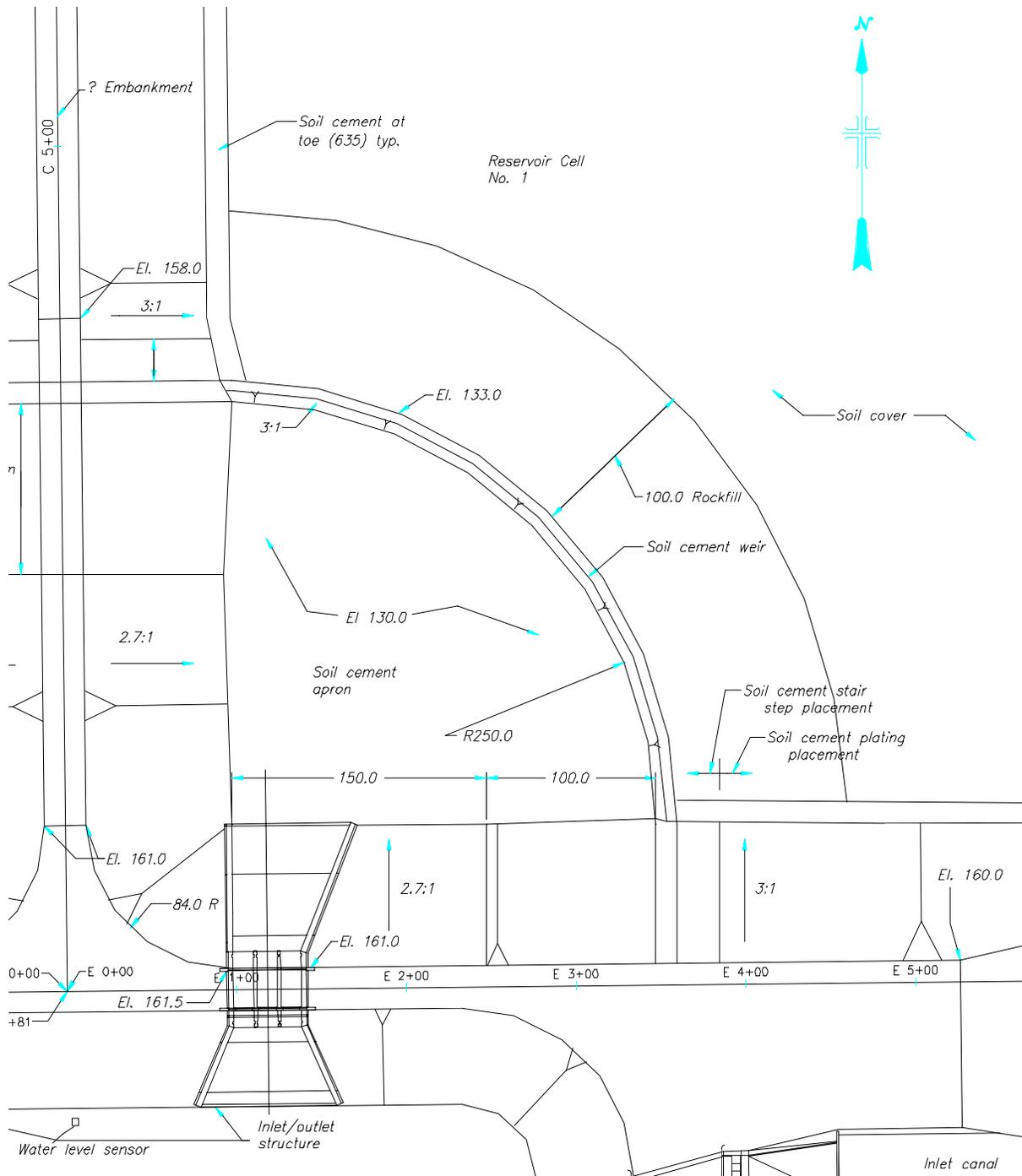


Figure 37. - Plan view of the final design of reservoir cell No. 1 area near the exit of the structure showing the 250 ft radius arc from the corner of the reinforced concrete apron forming the recommended sill. Soil cement will be placed upstream from the sill with a 100-ft-wide placement of rock riprap on the downstream side leading into the sandy soil cover.

Conclusions

The design criteria during initial filling for the forebay/afterbay basin and reservoir produced difficult flow conditions to deal with for all the structures. The forebay/afterbay basin will be covered with soil cement; therefore, reducing the need to dissipate quite as much energy as originally expected. In addition, the size of the basin allows for it to quickly fill. The reservoir filling situation could not be fully addressed technically in the model as the velocity over the sand geomembrane cover could not be attained during the physical model testing. Engineering judgment was utilized to determine the final geometry for the reservoir inlet during filling. Filling and draining times were computed after use of the physical model results.

The following conclusions were obtained from the model study and consultations with the design team:

- Forebay/afterbay basin geometry:
 - The siphon inlet should be centered between the two inlet/outlet structures to improve flow conditions.
 - The overall size of the basin was minimized to reduce eddies and minimize sediment deposition.
 - Time to fill the forebay/afterbay basin was quite short, with filling to 4.7 ft of depth accomplished in about 3 minutes, 9.75 ft in about 7.5 minutes, and final filling to 21 ft in about 22 minutes.
 - Velocities on the soil cement floor will be acceptable for all flow conditions given the result from the soil cement jet testing which demonstrated that the material is exceptionally strong.
 - A reinforced concrete pad should be placed at the end of the canal inlet and outlet structure to prevent erosion and provide additional support for maintenance equipment.
 - Water level instrumentation should be centered between the two outlet structures in a zone of fairly low turbulence.
 - It is assumed that performance of the basin will be acceptable when operating the structures under draining of reservoir cell No. 2, based on evaluations of reservoir cell No. 1 in the model.
- Reservoir draining and outlet structure geometry on the forebay/afterbay basin side:
 - The flared walls helped reduce headloss through the system. This did seem to help with losses on the approach side but produced an eddy zone during releases.

- The basin apron is at El. 130 ft with 3:1 upstream and downstream ramps or sills to allow access into and out of the basin. Lowering the apron below the invert of the gate seat eliminated the instability in the toe of the jump that caused the jet to attach to the side of the pier or walls. The lowered floor also produced some initial depth for energy dissipation.
- No baffles need to be used on the floor of the outlet structure on the forebay/afterbay basin side.
- A discharge rating for reservoir draining was provided for use with a HEC-RAS model to determine if the draining time criteria would be met.
- Siphon inlet:
 - Flared walls produced better approach flow conditions into the siphon inlet structure from the forebay/afterbay basin.
 - The same type of reinforced concrete pad as placed at the end of the inlet/outlet structures should probably also be placed at the entrance to the siphon inlet.
 - The siphon inlet geometry and gates controlled draining rate of the reservoirs. The model geometry included the approach floor and side walls and a gate bulkhead representing fully open gates that discharged out the end of the model. A rating curve for reservoir draining with fully open outlet structure gates was provided. The gate seat was then lowered by 1 ft based upon HEC-RAS modeling results completed by the designers.
- Reservoir filling and inlet geometry on the reservoir side:
 - The final configuration was a combination of the geometry from the hydraulic model and engineering judgment regarding technically and economically feasible solutions to obtaining the target velocity to prevent erosion of the sandy reservoir floor.
 - The right side wall was flared to reduce headloss in the system during draining. This also eliminated vortex formation during reservoir draining.
 - The inlet apron on the reservoir side was lowered to El. 130 ft and will be constructed of reinforced concrete to the end of the sloping left side wall and toe of the embankment slope.
 - Three rows of 2.25-wide by 3-ft-high blocks should be constructed on the reinforced floor at a 125 degree angle to the flow. The first row of blocks should be placed about 41.3 ft downstream from the end of the piers and spaced at the same width as the blocks are wide.
 - The apron floor will then extend with a soil cement cover outward into the reservoir on a 250 ft radius to a 3:1 sloping sill finishing at El. 133 ft.

- A 100-ft-wide section of riprap will be placed downstream from the soil cement sill to further reduce the flow velocity prior to reaching the sandy floor of the reservoir.
 - Initial opening of the inlet structure gates might be performed from right to left with the middle and left gates opened more quickly to the fully open setting. A video clip of the dynamic opening of the gates filling the reservoir with no initial depth in the reservoir and an approximate water level of El. 154 set in the forebay/afterbay basin is available upon request.
- Canal inlet geometry:
 - The chute slope design of 0.747 ft/ft accommodated as many baffle blocks as possible in the 40-ft-width.
 - A Fujimoto entrance was designed for the chute followed by 6 rows of baffles producing good flow conditions into the forebay/afterbay basin.

Recommendations

It is recommended that the initial operation of the facilities be monitored to ensure appropriate flow conditions are achieved. After an initial filling and draining of the reservoirs it is recommended that they be inspected to assess whether the constructed inlet geometry performed adequately.

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