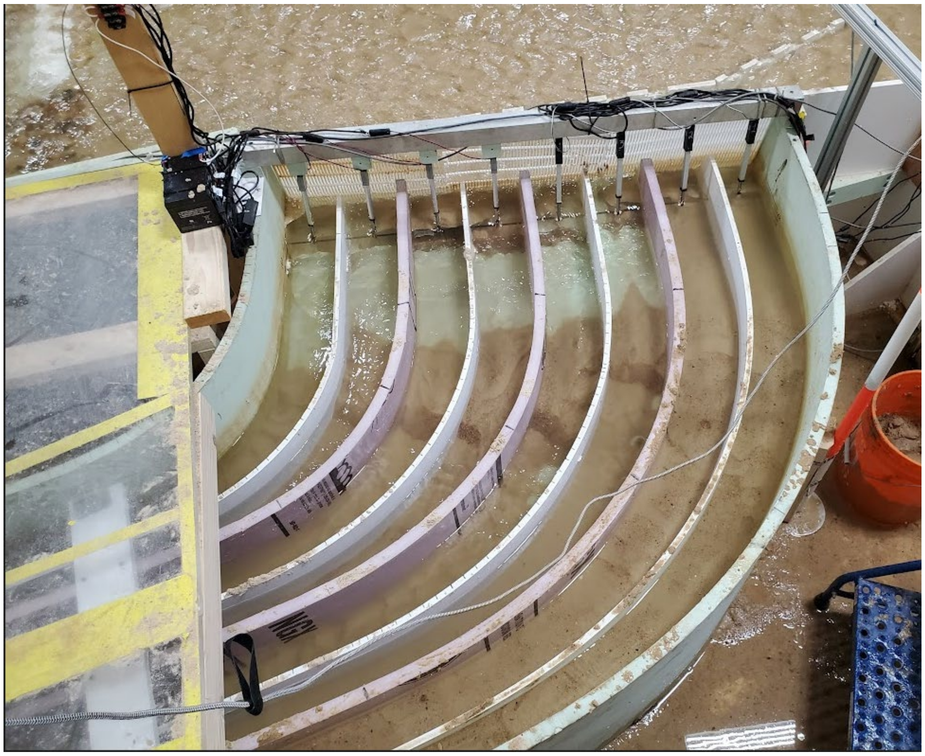
Hydraulic Laboratory Report HL-2023-01

Physical Hydraulic Modeling of Operational and Stress Testing of the Freeman Diversion Hardened Ramp Fish Passage Alternative

Freeman Diversion
United Water Conservation District, Ventura County, California



U.S. Department of the Interior May 2023

REPORT DOCUMENTATION PAGE	*Form Approved* *OMB No. 0704-0188*

1. REPORT DATE *(DD-MM-YYYY)* May 2023	**2. REPORT TYPE** Technical	**3. DATES COVERED** *(From - To)* November -December 2022

4. TITLE AND SUBTITLE Physical Hydraulic Modeling of Operational and Stress Testing of the Freeman Diversion Hardened Ramp Fish Passage Alternative	**5a. CONTRACT NUMBER**
	5b. GRANT NUMBER
	5c. PROGRAM ELEMENT NUMBER
6. AUTHOR(S) Melissa Shinbein, P.E.	**5d. PROJECT NUMBER**
	5e. TASK NUMBER
	5f. WORK UNIT NUMBER
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) Bureau of Reclamation, Technical Service Center Hydraulic Investigations and Laboratory Services	**8. PERFORMING ORGANIZATION REPORT NUMBER** HL-2023-01
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) United Water Conservation District Ventura County, California	**10. SPONSOR/MONITOR'S ACRONYM(S)**
	11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION/AVAILABILITY STATEMENT

13. SUPPLEMENTARY NOTES

14. ABSTRACT

Operational and stress testing was conducted in two movable bed physical hydraulic models of the Freeman Diversion hardened ramp fish passage alterative in support of design development. Testing focused on the training wall ("castle wall"), desander, debris passage, variation in the upstream approach channel orientation, and low-flow diversion capacity. The castle wall and flushing channel configuration was able to move a large amount of sediment in a short amount of time in front of the apron. Stress testing for the latest desander geometry proved effective at clearing sediment from within the desander channels with operational variability. Debris rarely accumulated in the baffled section of the hardened ramp, but when debris did accumulate, it was flushed out when river flow rates increased or decreased. When the thalweg of the river was skewed to the right, disconnecting the river from the diversion intake, a pilot channel paired with flushing channel operations was shown to successfully train flow back into the diversion. Maximum diversion flow rates were obtained for a range of river discharges, sediment depths on the apron in front of the intake, and the presence of an operational bulkhead in the form of a lowered notch at the downstream end of the training wall.

15. SUBJECT TERMS

hardened ramp, fish passage, steelhead, sediment management, desander

16. SECURITY CLASSIFICATION OF:			**17. LIMITATION OF ABSTRACT**	**18. NUMBER OF PAGES** 72	**19a. NAME OF RESPONSIBLE PERSON** Connie D. Svoboda
a. REPORT	**b. ABSTRACT**	**a. THIS PAGE**			**19b. TELEPHONE NUMBER** *(Include area code)* 303-445-2152

Standard Form 298 (Rev. 8/98)
Prescribed by ANSI Std. Z39.18

Hydraulic Laboratory Report HL-2023-01

Physical Hydraulic Modeling of Operational and Stress Testing of the Freeman Diversion Hardened Ramp Fish Passage Alternative

Freeman Diversion
United Water Conservation District, Ventura County, California

prepared by MELISSA SHINBEIN Digitally signed by MELISSA SHINBEIN Date: 2023.05.19 17:16:16 -06'00'

Melissa Shinbein, P.E.
Hydraulic Engineer, Hydraulic Investigations and Laboratory Services, 86-68560

reviewed by

TONY WAHL Digitally signed by TONY WAHL Date: 2023.05.19 20:26:12 -06'00'

Peer Review: Tony Wahl, P.E.
Hydraulic Engineer, Hydraulic Investigations and Laboratory Services, 86-68560

CONNIE SVOBODA Digitally signed by CONNIE SVOBODA Date: 2023.05.21 12:27:01 -06'00'

Technical Approval: Connie D. Svoboda, P.E.
Manager, Hydraulic Investigations and Laboratory Services, 86-68560

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Hydraulic Investigations and Laboratory Services
Denver, Colorado

January 2023

Acknowledgments

This study was funded by United Water Conservation District through a Contributed Funds Agreement with the Bureau of Reclamation's California Great Basin Region - Southern California Area Office. The author greatly appreciates United Water Conservation District and its consultants for providing information and support throughout the modeling effort. The author would also like to thank Josh Mortensen for his help with all aspects of this project, Jacob Carter-Gibb and Kit Shupe for their help with model data collection and analysis, and Tony Wahl and Bryan Heiner for their efforts with model design. Connie Svoboda provided project management and overall coordination for the project. Special thanks are due to Jeff Falkenstine, Jason Black, Miles Van Zweden, Dane Cheek, and Robert Borrego for their exceptional craftsmanship in fabricating and constructing two movable bed physical hydraulic models and being flexible on completing numerous modifications and changes.

Hydraulic Laboratory Reports

The Hydraulic Laboratory Report series is produced by the Bureau of Reclamation's Hydraulic Investigations and Laboratory Services Group (Mail Code 86-68560), PO Box 25007, Denver, Colorado 80225-0007.

Disclaimer

The information provided in this report is believed to be appropriate and accurate for the specific purposes described herein, but users bear all responsibility for exercising sound engineering judgment in its application, especially to situations different from those studied. References to commercial products do not imply endorsement by the Bureau of Reclamation and may not be used for advertising or promotional purposes.

Cover Photo: 1:24 scale physical model of Freeman Diversion desander system.

Contents

Tables

Figures

Executive Summary

Physical hydraulic modeling of the Freeman Diversion hardened ramp fish passage alternative was conducted at the Bureau of Reclamation Hydraulics Laboratory in Denver, Colorado. The Freeman Diversion is located on the Santa Clara River in Ventura County, California. Movable bed models were constructed and tested at a 1:24 model scale and a 1:12 model scale. Two physical model scales were needed to study flow interaction among the diversion and fish passage structures within the system at higher flow ranges (1:24 scale) and to study detailed hydraulic conditions at fish passage features on and adjacent to the hardened ramp at lower operational flow ranges (1:12 scale). The physical models were used to provide results and observations over a discharge range of 100 to 6,000 ft^3/s at a 1:12 scale and 1,500 to 30,000 ft^3/s at a 1:24 scale. Reclamation's role and purpose in this study was to provide modeling data to assist with decision-making on design features and improvements to support fish passage and sediment management goals. Physical hydraulic modeling of the hardened ramp fish passage alternative is detailed in Mortensen and Shinbein (2022). This report presents operational and stress testing in support of design development efforts.

Key findings from both physical models are summarized below.

- Training Wall Stress Testing - The flushing channel and castle training wall configuration was effective at removing large amounts of sediment, especially as river flow increased. Additionally, downstream of the hardened ramp, flow from the sluiceway and flushing channel exit were streamlined with little disturbance between them.
- Desander Stress Testing - Accumulated sediment was removed from the desander by operating one gate at a time or two gates, simultaneously. Elevating the tailwater had the greatest impact on sluicing efficiency, with tailwater elevations of 151 ft and higher reducing the sluicing ratio from 1% to 0.2%. However, sediment was cleared from the desander channels at total sluice flows as low as 200 ft^3/s.
- Debris Passage - Modeled debris representing Arundo did not accumulate in the hardened ramp at flows higher than 6,000 ft^3/s due to baffle overtopping, nor lower than 1,500 ft^3/s due to baffle inactivity. In the model, debris accumulated at river flows between 1,500 and 6,000 ft^3/s, though accumulation was still infrequent within this flow range and would clear when flows increased or decreased. Woody debris only accumulated in the baffled area if there was already accumulation of "Arundo"-like debris. Debris often clogged the trash rack, making it difficult to obtain desired diversion rates.
- Variations in Upstream Approach Channel Orientation - When the thalweg of the river was skewed such that the river was disconnected from the diversion intake, a pilot channel was be excavated and used in conjunction with the flushing channel to train flow back into the diversion.
- Low Flow Diversion Capacity - Maximum diversion rates for a given discharge depended on the sediment depths on the apron in front of the intake and the presence of an operational bulkhead in the form of a lowered notch at the downstream end of the castle training wall. For all tested sediment conditions, 72-100% of the river flow could be diverted into the intake. With a 250 ft^3/s river flow rate and normal sediment accumulation on the intake apron, more flow could be diverted with an unblocked downstream notch than with a blocked downstream notch. When the apron was full of sediment at river flow rates of 250 and 500 ft^3/s, less water was diverted than under a normal sediment condition. Results with

sediment removed from the apron showed slightly lower maximum diversion capacity for river flows from 250 to 750 ft^3/s with all 8 gates operating in comparison to only gates 5-8 operating. With no sediment present on the apron, flows could not overtop the lowered notch of the training wall to activate the hardened ramp, even when the bulkhead was open.

Introduction

United Water Conservation District (United Water) owns and operates the Freeman Diversion on the Santa Clara River in Ventura County, California (Figure 1). Freeman Diversion diverts water from the Santa Clara River to spreading basins for groundwater recharge at an average rate of 60,000 acre-ft per year. Freeman is a 28-ft-high, 1,200-ft-long roller compacted concrete gravity structure with a Denil fish ladder and diversion facilities. United Water currently diverts up to 375 ft^3/s but plans to file a water right change petition to divert up to 750 ft^3/s from the Santa Clara River.

Figure 1. Location of Freeman Dam (circled) on the Santa Clara River in Ventura County, California.

United Water contracted the Bureau of Reclamation (Reclamation) Hydraulics Laboratory in Denver, Colorado to accomplish physical hydraulic modeling of the proposed hardened ramp fish passage alternative. Two movable bed physical hydraulic models were constructed at a 1:24 and 1:12 scale to support the modeling effort. The overall purpose of the physical modeling was to provide modeling data to assist with decision-making on design features and improvements in support of fish passage and sediment management goals. This testing was completed in October 2022 (Mortensen and Shinbein, 2022). Additional testing of the modified 30% design from this study was conducted from November through December 2022 to provide information about operational and stress testing. The results from these tests are included within this document.

Experimental Approach

Sediment transport models were constructed and tested at a 1:24 model scale and a 1:12 model scale. Two physical model scales were needed to look at flow interaction among all the structures within the system (1:24 scale) and to resolve the details of hydraulic conditions at fish passage features on the hardened ramp and intake gates at the canal diversion (1:12 scale). The physical models were used to provide results and observations over a range of 100 to 6,000 ft^3/s at the 1:12 scale and 1,500 to 30,000 ft^3/s at the 1:24 scale. For more information on model design and operation, sediment scaling, and shakedown testing, please see Mortensen and Shinbein, 2022.

Model Operation

Both models were constructed as an integrated facility with a common return channel and large pumps capable of passing larger materials and providing sediment laden flow to both model boxes. The model boxes were laid out perpendicular to each other within the allowable floor space in the lab as shown in Figure 2. The piping was arranged so that the two 8-inch horizontal pumps (recirculation pumps) with variable frequency drives operated either individually or in parallel to provide recirculated flow to both models (Figure 3). Additional pipes from the primary lab system were also included to slowly fill the model boxes in preparation for testing and for model flows either greater than the combined capacity of the sediment pumps or lower than the flow range of the flow meters on the recirculation pipes.

Tests with unsteady flows to represent dynamic hydrographs of the Santa Clara River were not considered as part of this study. This was primarily due to the different time scales associated with water and sediment transport processes. Since sediment transport times do not necessarily scale with Froude similitude for hydraulics, it could be possible to produce sediment patterns that are not fully developed or are significantly different from reality by simulating a hydrograph. Instead, a range of sequential steady state flows (always either rising or falling) was used to allow the mobile bed to fully develop and give insight into trends and patterns.

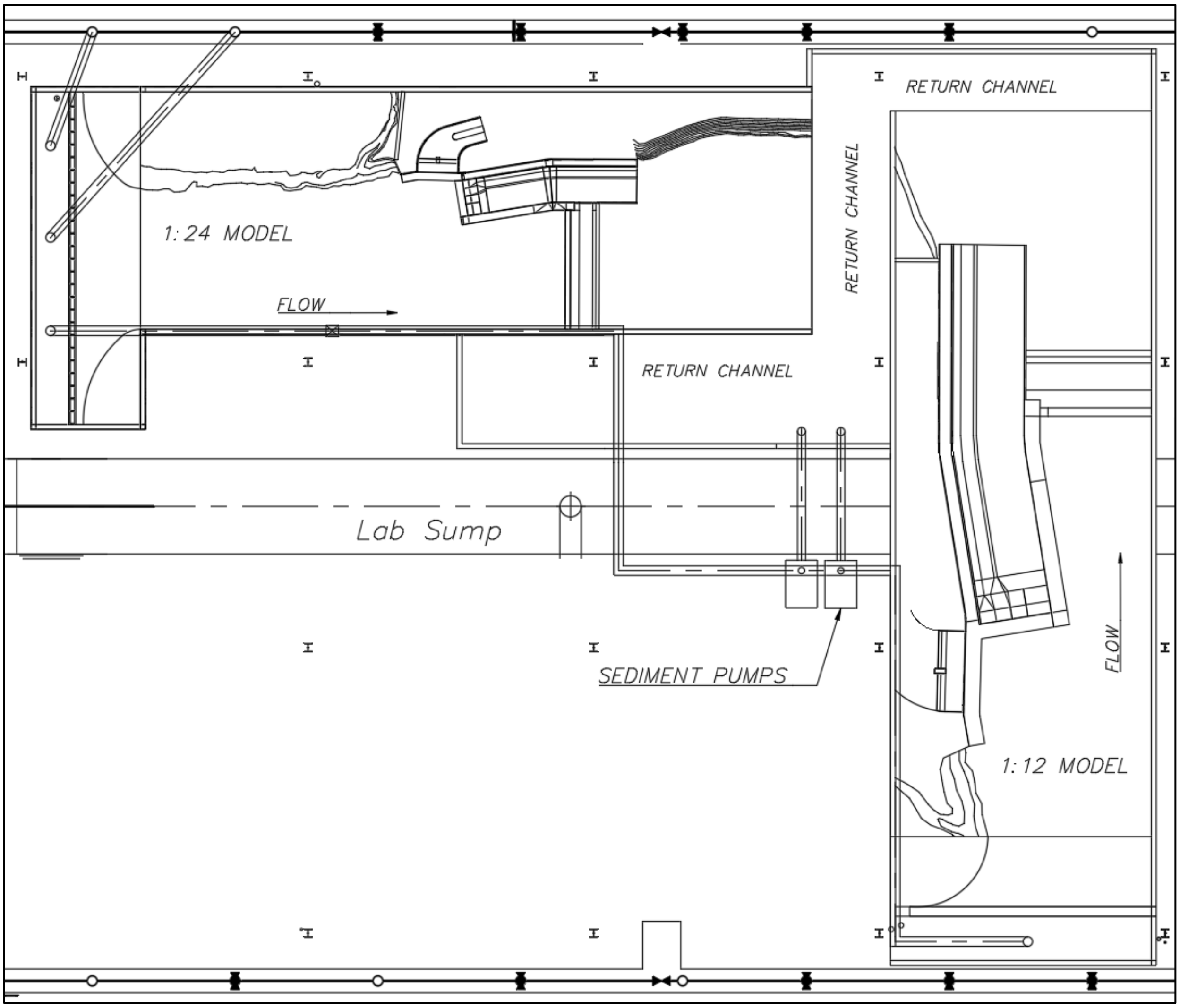


Figure 2. Layout of 1:24 and 1:12 physical models in the hydraulics laboratory with common sediment pumps and return channel.

Both models were operated by first slowly filling the model box (typically from the main lab system) to not disturb the mobile bed and then setting the desired discharge with the control valve and pump variable frequency drive once the box had filled and water reached the downstream model extent. Model boundary conditions included river discharge from the pumps on the upstream end of the model, tailwater elevations on the downstream end of the model box, and canal diversion discharge and water level controlled by canal intake gates and a downstream control gate for flow exiting the diversion.

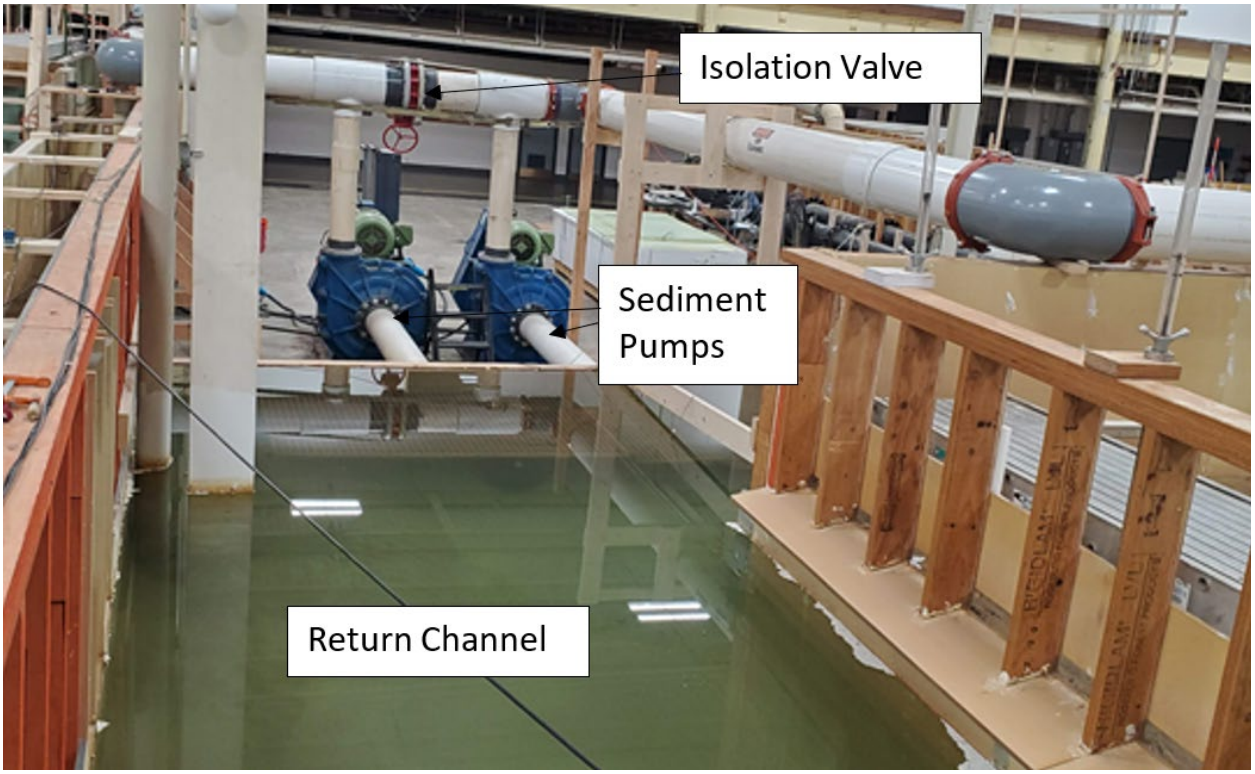


Figure 3. Photograph of common return channel and sediment pumps to recirculate sediment laden flow into both models.

Instrumentation and Data Acquisition

A variety of instrumentation was used to measure and control hydraulic conditions on both models and to record sedimentation trends and patterns (Table 1). Both models included a data acquisition system to record and display live measurements and control settings. Systems for both models included a 12-volt power supply, analog to digital converter and laptop computer. Raw measurements were scaled, processed, recorded, and displayed live using the data acquisition software DASYLab.

Table 1. Instrumentation used for hydraulic and sediment measurements on both 1:24 and 1:12 models.

Measurement		Sensor / Measurement Technique
Discharge	River (1:24)	Siemens Electromagnetic Flowmeter
	River (1:12)	Siemens Ultrasonic Flowmeter
	River (1:12, < 300 ft^3/s)	Venturi Flow Meter
	Canal Intake Diversion	MassaSonic Downlooker (V-Notch Rating)
	Dam Crest	MassaSonic Downlooker (Rating)
Water Level	Elevation	MassaSonic Downlooker
	Depth - Point Locations	Steel Ruler
Velocity	Flow Patterns	LSPIV (Large Scale Particle Image Velocimetry) from GoPro video
	Point Measurement (depth > 2 inch)	Flow Tracker ADV
	Point Measurement (depth < 2 inch)	Nixon Meter
Sedimentation	Topography	Photogrammetry
	Topography	FARO Focus 5 Laser Scanner
	Depth - Point Locations	Steel Ruler
Intake Gate Position	Elevation (1:24)	Scale on side walls of intake
	Elevation (1:12)	Potentiometer

1:24 Data Collection

The 1:24 data acquisition system recorded water levels and discharge for each model run. Water surface elevations were measured with MassaSonic acoustic downlooking sensors at several locations shown in Table 2 and Figure 4. Total river discharges were measured with a Siemens electromagnetic flow meter, canal discharge was measured with a V-notch weir and flows over the dam crest were estimated with water surface elevation rating curves developed during shakedown testing.

Point measurements of both flow depth and velocity were made primarily on the upstream end of the hardened ramp at locations of the four ramp crest gates. Velocities at these locations were measured with a Nixon rotary meter due to shallow depths. Large Scale Particle Image Velocimetry (LSPIV, or simply PIV) was also used for the area near the canal intake and hardened ramp exit extending upstream (see LSPIV section).

Bathymetric maps showing bed changes and extents of sediment deposition and erosion were created using photogrammetry for the baseline testing completed in late 2021. In early 2022 the Hydraulics Lab acquired a FARO Focus 5 Laser Scanner for bed mapping from that time on.

Table 2. Water level sensor locations on 1:24 model. Sensors 1 and 4 were removed for this portion of testing.

Water Level Sensor Number	Location	Distance Upstream from Dam (ft)	Description
1	Upstream Channel	660	Center of channel about 150 ft from right model extent (this sensor was removed)
2	Upstream Intake	396	Immediately upstream of the canal intake near left wall
3	Downstream Intake	300	Center of canal intake about 60 ft downstream from intake gates
4	Dam Crest - Right	12	Near the river right model extent (this sensor was removed)
5	Dam Crest - Left	12	Near the ramp wall, river left side of dam
6	Tailwater	-256	Immediately downstream of the hardened ramp entrance

Figure 4. View of water level sensor locations on 1:24 model (looking upstream). Sensors 1 and 4 were removed for this portion of testing.

1:12 Data Collection

The 1:12 data acquisition system recorded water levels, canal intake gate elevations, and discharge for the canal diversion. Water surface elevations were measured with MassaSonic acoustic downlooking sensors at several locations shown in Table 3 and Figure 5. Total river discharge was measured with a Siemens acoustic flow meter but was not recorded electronically during testing. Flows shown on the meter's display screen were recorded manually. Canal discharge was measured with a V-notch weir and flows over the dam crest were estimated with water surface elevation rating curves developed during shakedown testing similar to the 1:24 model.

Point measurements of both flow depth and velocity were made primarily on the upstream end of the hardened ramp at locations of the four ramp crest gates (Figure 6). Velocities at these locations were measured with a FlowTracker Acoustic Doppler Velocimeter (ADV) for model flow depths greater than about 2 inches and a Nixon rotary meter at shallower depths. Point velocity and depth measurements were also made on the hardened ramp structure during design development of fish passage features. PIV was also used for the area near the canal intake and hardened ramp fishway exit (upstream end of ramp) extending upstream as well as at key locations on the hardened ramp (see LSPIV section). Bathymetric maps showing bed changes and extents of sediment deposition and erosion were created using a FARO Focus 5 Laser Scanner and TecPlot software.

Table 3. Water level sensor locations on the 1:12 model.

Water Level Sensor Number	Location	Distance Upstream from Dam (ft)	Description
1	Upstream Intake	396	Immediately upstream of the canal intake near left wall
2	Downstream Intake	300	Center of canal intake about 30 ft downstream from intake gates
3	Dam Crest - Left	12	Near the ramp wall, river left side of dam
4	Tailwater	-256	Immediately downstream of the hardened ramp entrance

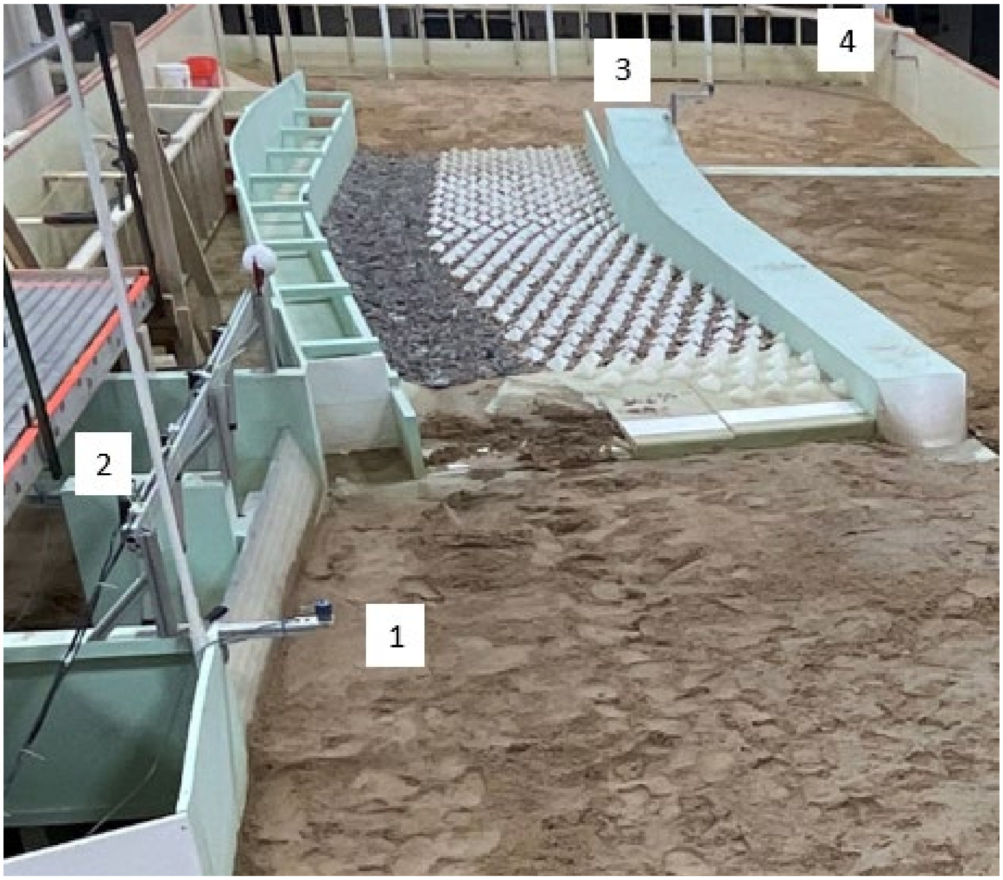


Figure 5. View of water level sensor locations on 1:12 model (looking downstream).

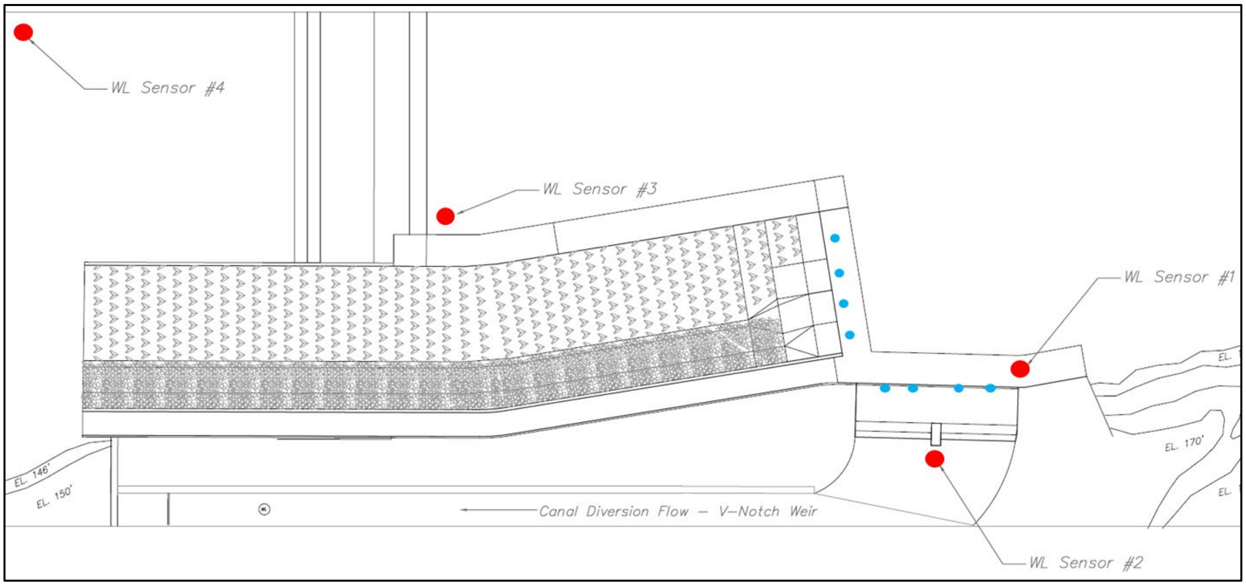


Figure 6. Layout of water level sensor locations in the 1:12 model (red) and point velocity and depth measurement locations (blue).

LSPIV

Large-scale particle image velocimetry (LSPIV) was used to capture surface velocities for areas of interest on both models. To capture these data, several GoPro Hero10 cameras recorded videos at a rate of 30 frames per second. Seeding material was evenly dispersed into the model until a minimum of 10 seconds of full coverage was obtained. Frames were separated into individual images using RIVeR 2.2 (Patalano, 2017). These frames were then processed using PIVLab software (Thielicke and Stamhuis, 2014). PIVLab outputs velocity vector maps of the water surface, which were saved in ASCII comma separated format and brought into TecPlot360 to generate velocity color contour plots.

On both the 1:24 and 1:12 models, LSPIV was used to produce velocity plots of the upstream region of key features. This included the hardened ramp fish exit, bullnose, and canal intake. Additionally, on the 1:12 model, LSPIV was used to develop velocity fields around the low flow channel rock configurations and downstream of the hardened ramp.

LiDAR Scanning

Point clouds of the movable bed were obtained with a FARO FocusS70 terrestrial LiDAR scanner. The 3D scans were analyzed in Scene, FARO's 3D scan processing and registration tool (FARO, 2022), which produced a point cloud with 1-3 mm model resolution. Collected point cloud data were brought into AutoCAD and TecPlot Focus where color contour maps were generated.

Design Development Configurations

Previous Design Development Configurations

Physical hydraulic model testing to observe sediment management and fish passage hydraulics of a 30% hardened ramp fish passage alternative provided by Northwest Hydraulic Consultants (2020) was completed in 2022. Results and observations from baseline testing were used to modify various components of the 30% design to improve performance (Mortensen and Shinbein, 2022). A chronological log of all modeling activities from the start of the study until the end of the original design development (Table 4) is presented to help show the process of baseline testing up to design development and the process for testing changes during design development. MOD6 and MOD9 refer to design alternatives evaluated during testing. This report expands upon the original MOD9 design development to include operational and stress testing and is discussed in further detail below.

Table 4. Modeling construction, testing, general observations, and site visit activities in chronological order from baseline testing through design development.

DATE	ACTIVITY	NOTES
1:24 Construction and Shakedown		
June - September 2021	Construction of 1:24 model	
June 2021	Sediment transport shakedown tests in 3 ft flume	Results suggested sediment mix is transporting appropriately
September - October 2021	1:24 model shakedown testing	Added more sands to the mixture and adjusted upstream topography
Sept 21-23, 2021	1:24 model shakedown testing - United Water laboratory visit #1	General model operation and compared model flows to unmanned aerial system video at Freeman Dam
October 13, 2021	Dr. Larry Weber - independent site visit	Overview of 1:24 shakedown testing ahead of first agency visit
October 26-27, 2021	Agency site visit #1	Overview of 1:24 model and initial findings, near end of shakedown testing
1:24 Baseline		
November 2021	Further 1:24 bathymetry adjustments	Adjusted upstream bathymetry based on recommendations from United Water and Dr. Larry Weber
	1:24 baseline testing - MOD6	
December 2021	1:24 baseline testing - MOD9	Flow and sediment patterns very similar for both MOD 6 and MOD9, sediment stabilization takes longer at MOD9
January 2022	1:24 baseline data processing and draft summary document submitted	
January 5-7, 2022	United Water laboratory Visit #2	Witness test of 1:24 baseline test results, ran longer duration MOD9 test to see if bedform is same
Feb 6 and 9, 2022	Meeting to present 1:24 baseline test results	
1:12 Construction and Shakedown		
October 2021 - February 2022	Construction of 1:12 model	
March 2022	Shakedown testing of 1:12 model	Compared hydraulic data and general sediment patterns to 1:24 model to confirm model performance
1:12 Baseline		
April 2022	1:12 Baseline Testing - MOD6	Changed test procedure to test from 6,000 ft^3/s to 270 ft^3/s, more representative of falling limb of the hydrograph

DATE	ACTIVITY	NOTES
	1:12 Baseline Testing - MOD9	Flow and sediment patterns very similar for both MOD 6 and MOD9
May 3-4, 2022	United Water laboratory visit #3	1:12 baseline testing demonstration and further development of flushing and training wall configurations
May 24-25, 2022	Agency site visit #2	Focused on 1:12 baseline testing demonstration and initial desander concept
Design Development		
May 2022	Desander version 1 design and construction	Included only the desander portion of the intake without the sluicing culvert or channel
June 2022	Desander version 1 testing	Tested desander version 1 with sediment sluicing rates
	Extended hardened ramp wall bullnose testing in 1:24 and 1:12 models	Reduced separation off the hardened ramp wall, allowing more streamlined flow for fish exiting the ramp
July 2022	Hardened ramp upstream roughness testing in 1:12 model	Compared surface roughness covering the upstream end of the hardened ramp, showed no significant difference
	Upstream river training works testing (groynes)	No significant difference in hydraulic and sediment results with groynes along left bank
	Desander version 2 design and construction	Included updated desander with single guide walls and sluicing culvert and channel downstream
August 2022	Desander version 2 testing	Desander upgrades effective, but more streamlined flow needed in culvert for improved downstream fish passage and sediment transport
August 15-17, 2022	Agency site visit #3	Focused on version 2 of desander and sluicing system in 1:24 model and observed fish passage feature concepts in 1:12 model
August 2022	1:12 low flow channel testing	PIV and point measurements of full fish features on 1:12 model hardened ramp
	No flushing channel tests with sediment features	Intake in line with left wall of hardened ramp, tested with and without sediment features, some differences but little change to general bed load deposition
September 2022	Desander version 3 design and construction	
	Desander version 3 testing	More streamlined flow conditions within the culvert, intake gate modulation needed to keep sluice cleared for tailwater elevations above 145 ft
October 2022	Flushing channel with castle training wall testing	Effective flushing and improved passageway for fish along training wall at low flow conditions
October 3-5, 2022	Agency site visit #4	Demonstration of version 3 of the desander, low flow section of hardened ramp, and castle training wall

Training Wall Stress Testing (1:24 Scale Model)

A training wall was included as part of the flushing channel design in the primary modeling report. The latest version included a training wall with slots along the top ("castle wall") and an apron floor sloping from elevation 154 ft to elevation 146 ft at the inlet to the flushing channel (5.2% slope) (Mortensen and Shinbein, 2022). The downstream slot closest to the flushing channel was designed to have a removable bulkhead in the prototype. For the model, a removable piece was inserted into the downstream notch, as needed, to raise the invert to an elevation equal to that of the rest of the castle wall slots (Figure 7). When the bulkhead is open, the notch was removed or "unblocked"; when the bulkhead is closed, the notch was inserted or "blocked".

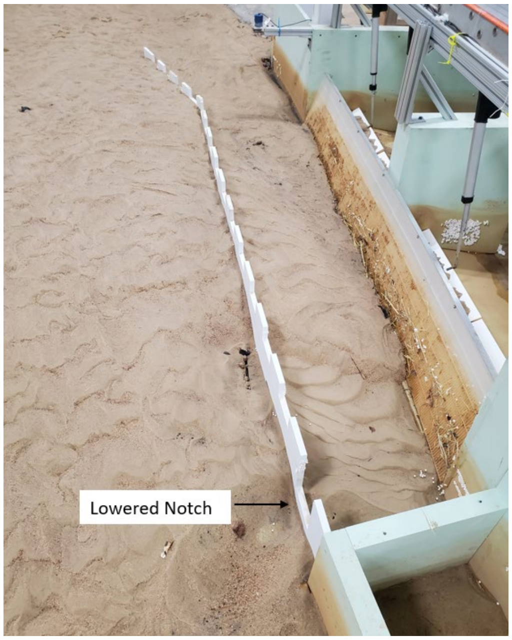


Figure 7. Lowered notch of the castle training wall configuration shown in the 1:12 model.

In the 1:24 model, castle wall refinement was tested at three river flows (Table 5). The flushing channel was operated until sediment in front of the apron was cleared and the flushing operation was drawing sediment from the upstream riverbed, without changing the deposition amounts along the apron. Pre- and post-flushing channel operation, the diversion flow was set to 800 ft^3/s. During flushing operations, the diversion was closed.

Table 5. Test matrix for training wall stress tests of the hardened ramp design in the 1:24-scale physical model. The diversion was closed prior to operating the flushing channel.

River Flow (ft^3/s)	Flushing Channel Operation
1,500	Open
3,000	Open
6,000	Open

Desander Stress Testing (1:24 Scale Model)

To help manage sediment at the canal intake, a desander sluicing system was developed by Northwest Hydraulic Consultants and constructed in the 1:24 model. Initial testing of this system was reported in Mortensen and Shinbein (2022), however refinement of this system continued in this round of testing. The final system from the primary modeling report included four inner bay walls to isolate flow from each canal intake gate within independent channels upstream from the fish screens. Each bay contained additional inner guide walls to direct flow around the bend toward the fish screens and a lower-level outlet known as the sluicing culvert located beneath the canal and fish screens. During sluicing operations, a bottom gate leading to the sluicing culvert was opened to release sediment-laden flows back to the river via a sloped sluiceway while a top gate immediately above the bottom gate was closed to prevent back flow of diversion water from the other bays into the sluicing bay. The geometry of the desander was not changed from the last round of tests in the primary modeling report (Table 6). The desander was not tested in the 1:12 model due to space limitations in the laboratory.

For this round of tests, the four original intake gates were replaced by eight gates that were separated by the extended inner guide walls (Figure 8). This created 8 separate channels upstream that were controlled by four sluice gates. For example, channels 1 and 2 formed sluice bay 1, channels 3 and 4 formed sluice bay 2, and so on. To determine approximate desanding rates for set volumes of sediment in the system, the modified desander was tested at river flows of 1,500 to 6,000 ft^3/s for a variety of different sluicing flow rates and gate configurations (Table 7).

Table 6. Modeled desander geometry (Mortensen and Shinbein, 2022).

Desander Geometry						
Bay	**Sill Elevation**	**Invert Upstream Elevation**	**Invert Downstream Elevation**	**Culvert Height**	**Desander Slope**	**Sluiceway Slope**
1	156.5	155.5	146.0	5.0	3.25%	3.0%
2	156.5	155.5			3.59%	
3	155.0	154.0			3.38%	
4	155.0	154.0			3.82%	

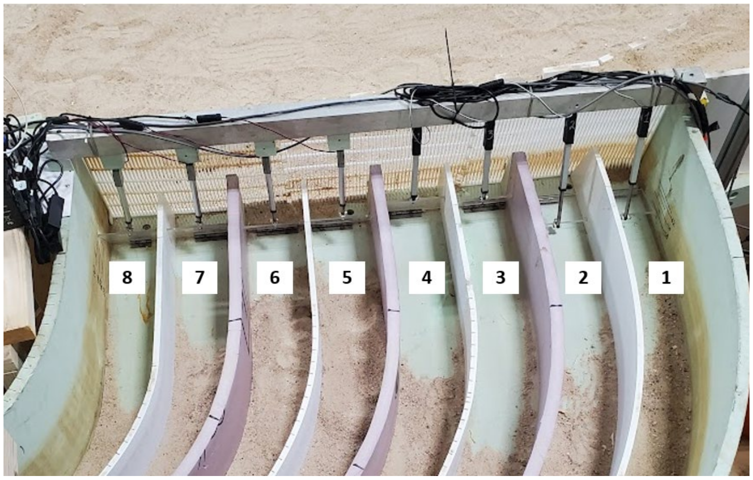


Figure 8. Added canal intake gates (looking upstream toward intake) with individual actuators installed to adjust gate settings remotely.

Table 7. Test matrix for desander testing of the hardened ramp design in the 1:24 scale physical model. Target diversion flow was approximately 800 ft^3/s for all tests.

River Flow (ft^3/s)	Target Total Sluiceway Flow (ft^3/s)	Tailwater Elevation (ft)	Bays Operated
3,000	300	144	1
3,000	450	144	1
3,000	600	144	1
3,000	300	150	1
3,000	450	150	1
3,000	600	150	1
3,000	500	144	1
1,500	Range of Flows	144	1
3,000	Range of Flows	144	1
6,000	Range of Flows	146	1

Debris Passage (1:24 and 1:12 Scale Model)

Debris testing was conducted in both the 1:12 and 1:24 scale models for different flows, flushing operations, and debris mixes to observe potential debris accumulation in the baffled portion of the hardened ramp (Table 8). Debris was classified as tree, Arundo, or mixed vegetation. Trees were modeled as woody debris using dowels. Arundo, a grass-like, invasive species, was modeled as hay. Mixed vegetation was modeled as assorted pieces of debris. In the 1:24 scale model, flows were increased and decreased incrementally as debris was added to simulate a basic hydrograph. In the 1:12 scale model, velocities were measured around debris mats to observe the impacts of the debris mats on the flow field. Flows under 1,500 ft^3/s were not tested because there was no flow in the baffled portion of the hardened ramp.

Table 8. Test matrix for debris testing of the hardened ramp design in both physical models. Since the intake is not utilized at flows above 6,000 ft^3/s, the diversion was closed. Flows under 1,500 ft^3/s were not tested because there was no flow in the baffled portion of the hardened ramp.

River Flow (ft^3/s)	Target Diversion Flow (ft^3/s)	Tree/Arundo/Mix	Flushing Channel Operation	Physical Model Used for Testing
1,500	500	0/40/60	Closed	1:12
6,000	750	30/40/30	Closed	1:12
12,000	0	60/30/10	Open	1:24

Variations in Upstream Approach Channel Orientation (1:24 and 1:12 Scale Model)

Variations in the upstream approach channel were made in both the 1:24 and the 1:12 scale models (Table 9). The upstream river approach was set such that the approach channel thalweg skewed to river right with no connection to the diversion. Operations, such as flushing channel operations, were then used to reestablish connectivity with the diversion intake. Assessment of the reconnection was entirely visual in both models. Additionally, a different configuration of rocks was added to the upstream portion of the hardened ramp (Figure 9). For more information on the configuration of rocks used in the rest of the low flow channel of the hardened ramp, please see Mortensen and Shinbein, 2022.

Table 9. Test matrix for upstream approach channel orientation testing of the hardened ramp design in the 1:12 physical model.

River Flow (ft³/s)	Target Diversion Flow (ft³/s)	Flushing Channel Operation	Physical Model Used for Testing
250	0	Open/Closed as needed	1:12
1,500	0	Open/Closed as needed	1:24

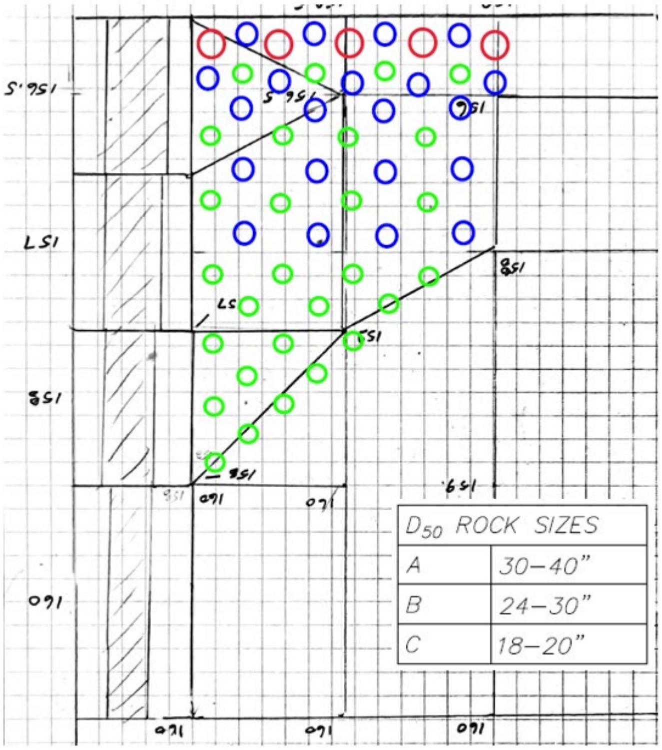


Figure 9. Rock configuration for upstream portion of the low flow channel in the 1:12 physical model. Rock size A is denoted by a large red circle, B is denoted by a medium blue circle, and C is denoted by a small green circle. Flow is from left to right.

Low-Flow Diversion Capacity (1:12 Scale Model)

Low-flow diversion tests aimed to document the possible range of diversion flows at various river discharges for the current design (Table 10). These tests started with the diversion gates fully closed and then the gates were opened, incrementally, until fully open at a set river flow rate. Tests were performed under various sediment loads and with the downstream notch of the castle wall blocked and unblocked (Figure 7). For some tests, sediment depths were measured along the apron to document general deposition patterns.

Table 10. Low-flow diversion test matrix. The diversion gates were operated from fully closed to fully open to document the range of flow available for diversion at a given river flow rate. A castle training wall was installed for all tests.

River Flow (ft^3/s)	Test Series	Training Wall Notch
2,000	Normal sediment conditions with intake gates 1-8 operated.	Blocked
1,500	Normal sediment conditions with intake gates 1-8 operated.	Blocked
1,250	Normal sediment conditions with intake gates 1-8 operated.	Blocked
1,000	Normal sediment conditions with intake gates 1-8 operated.	Blocked
750	Normal sediment conditions with intake gates 1-8 operated.	Blocked
500	Normal sediment conditions with intake gates 1-8 operated.	Blocked
250	Normal sediment conditions with intake gates 1-8 operated.	Blocked and Unblocked
2,000	Apron full of sediment with intake gates 1-8 operated.	Blocked
1,500	Apron full of sediment with intake gates 1-8 operated.	Blocked
1,250	Apron full of sediment with intake gates 1-8 operated.	Blocked
1,000	Apron full of sediment with intake gates 1-8 operated.	Blocked
750	Apron full of sediment with intake gates 1-8 operated.	Blocked and Unblocked
500	Apron full of sediment with intake gates 1-8 operated.	Blocked and Unblocked
250	Apron full of sediment with intake gates 1-8 operated.	Blocked
1,000	No sediment on intake apron with only gates 5-8 operated.	Unblocked
750	No sediment on intake apron with only gates 5-8 operated.	Unblocked
500	No sediment on intake apron with only gates 5-8 operated.	Unblocked
250	No sediment on intake apron with only gates 5-8 operated.	Unblocked
1,250	No sediment on intake apron with gates 1-8 operated.	Unblocked
1,000	No sediment on intake apron with gates 1-8 operated.	Unblocked
750	No sediment on intake apron with gates 1-8 operated.	Unblocked
500	No sediment on intake apron with gates 1-8 operated.	Unblocked
250	No sediment on intake apron with gates 1-8 operated.	Blocked

Results

Training Wall Stress Testing

The training wall ("castle wall") design developed during the previous modeling effort was only tested without flushing operations (Mortensen and Shinbein, 2022). During stress testing, river flows ranging from 1,500 to 6,000 ft^3/s were tested with the flushing channel open for comparison with the flushing channel closed configuration. Many of these tests were run concurrently with desanding operations. The flushing channel was operated until sediment in front of the apron was cleared and the flushing operation was drawing sediment from the upstream riverbed without deposition along the apron. There were some discrepancies in the length of time the flushing channel was operated, but overall trends could still be observed. The flushing channel effectively removed large amounts of sediment over the course of approximately ten minutes in model scale (Table 11). As sediment mobilized at higher flows, more sediment moved down the flushing channel.

Table 11. Average change in depth of sediment after the flushing channel was operated at a particular river flow. When the flushing channel was operated, the diversion was closed. After flushing operations were complete, the target diversion was 800 ft^3/s.

River Flow (ft^3/s)	Tailwater (ft)	Average Time of Flushing Channel Operation (model minutes)	Average Change in Sediment Depth After Flushing (prototype ft)
1,500	144	5	-3.25
3,000	144	13	-5.25
6,000	146	11	-6.5

The flushing channel effectiveness is also reflected in the LSPIV velocity measurements and bed elevation difference maps, especially when compared to the flushing channel closed configuration that was previously tested in Mortensen and Shinbein, 2022 (Figure 10 through Figure 18). With the flushing channel open, velocities were higher along the apron leading into the flushing channel. However, most velocities remained under 10 ft/s, even at a river flow rate of 6,000 ft^3/s. A comparison of the bed elevation difference maps for all flows shows less sediment on the apron after flushing operations.

When the flushing channel was open at a river flow of 1,500 ft^3/s, the baffled area of the hardened ramp was completely dry with some water remaining in the low flow section. In contrast, when the flushing channel was closed, the baffles were at least partially active at the same flow rate. At a river flow of 6,000 ft^3/s, the baffles remained active whether the flushing channel was open or closed.

Dye tests were performed to observe the interaction between the downstream flushing channel exit and the sluice channel exiting the desander (Figure 19 and Figure 20). At the downstream end of the hardened ramp, there was little recirculation from the outflow of the flushing channel near the sluice channel, even at higher flow rates. Additionally, while some flow passed between the upstream notches of the castle wall, little water flowed over the downstream notch of the castle wall into the hardened ramp.

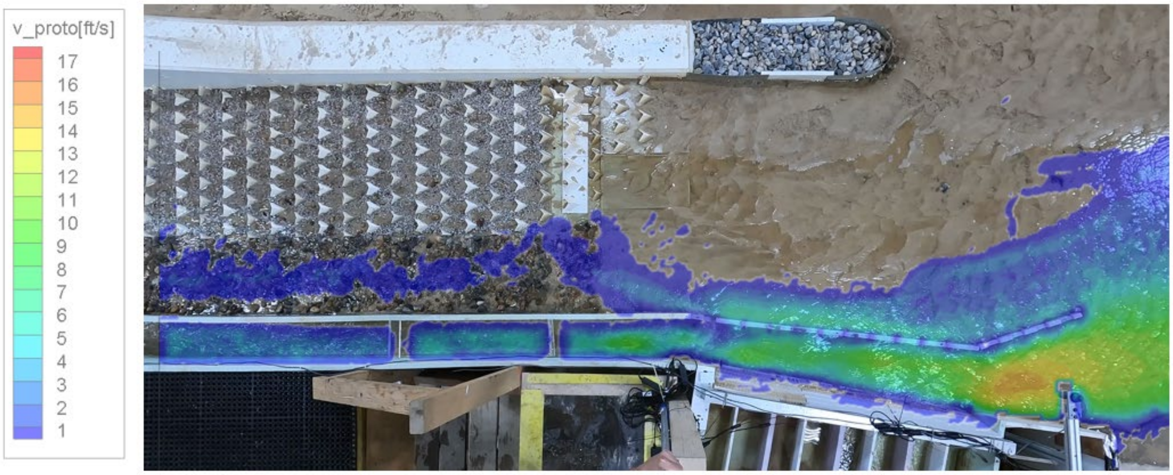


Figure 10. LSPIV data with flushing channel open at a river flow of 1,500 ft^3/s. Velocities less than 0.1 ft/s have been made transparent.

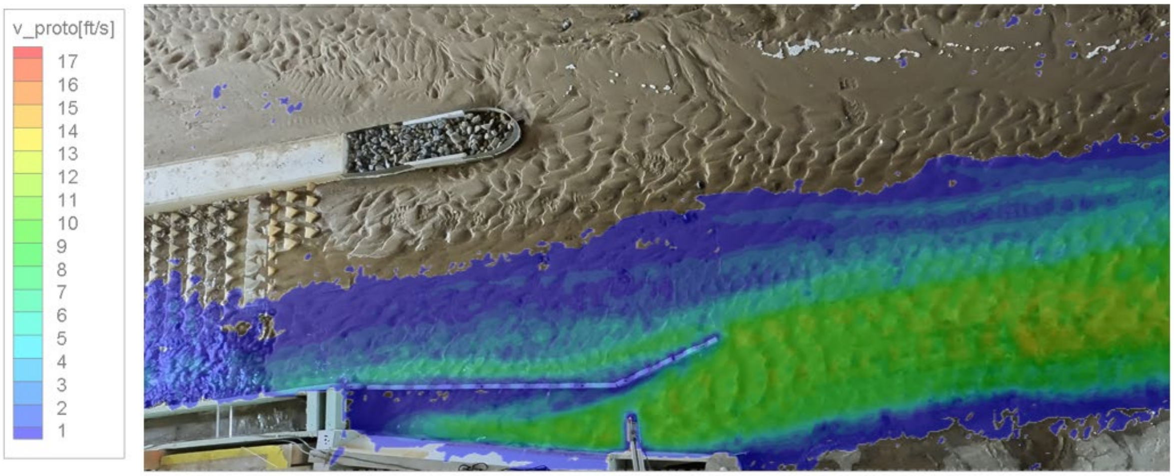


Figure 11. Previously collected LSPIV data for flushing channel closed at river flow of 1,500 ft^3/s. Velocities less than 0.1 ft/s have been made transparent (Mortensen and Shinbein, 2022).

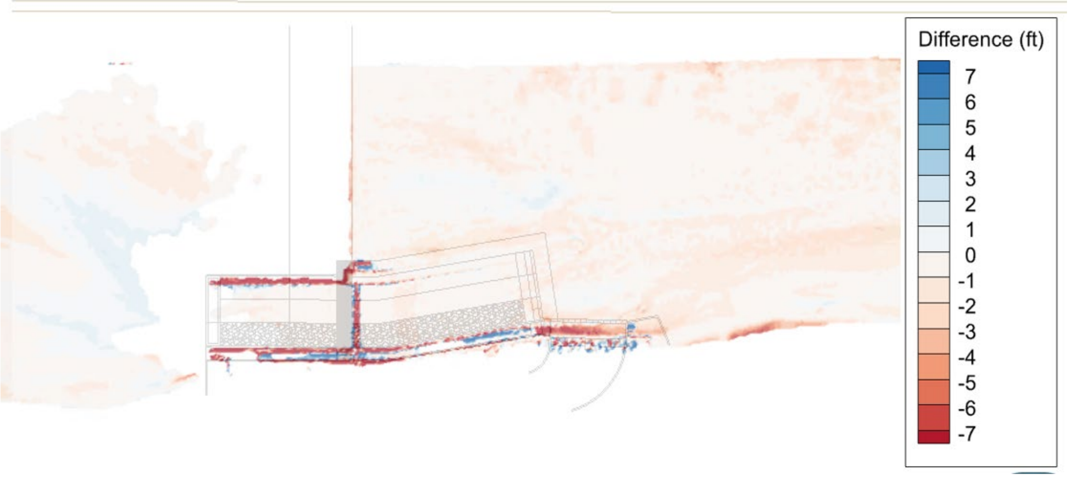


Figure 12. Bed elevation difference map between the post-run bathymetry with the flushing channel open minus the post-run bathymetry with the flushing channel closed for a river flow of 1,500 ft^3/s.

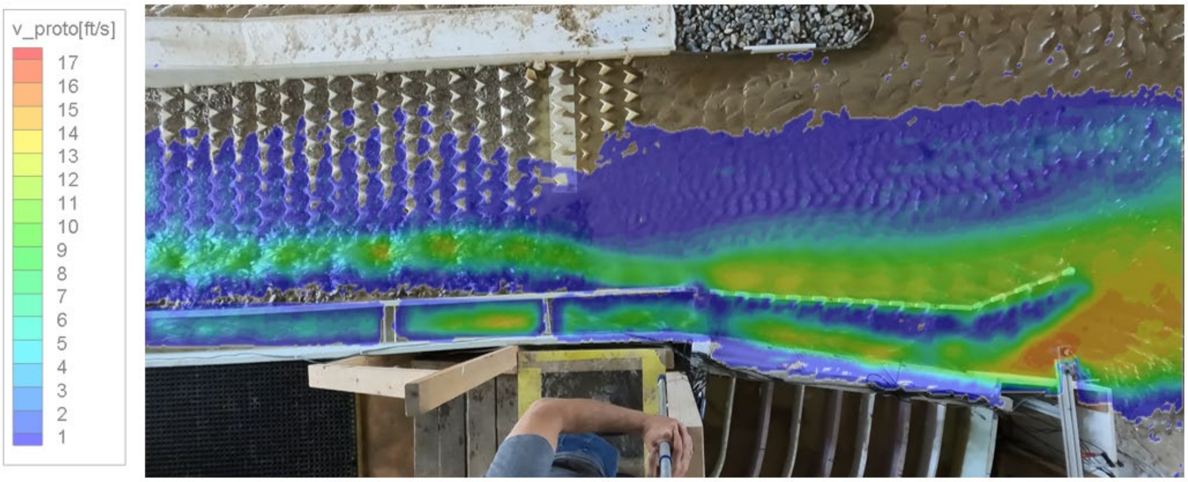


Figure 13. LSPIV data with flushing channel open at a river flow of 3,000 ft^3/s. Velocities less than 0.1 ft/s have been made transparent.

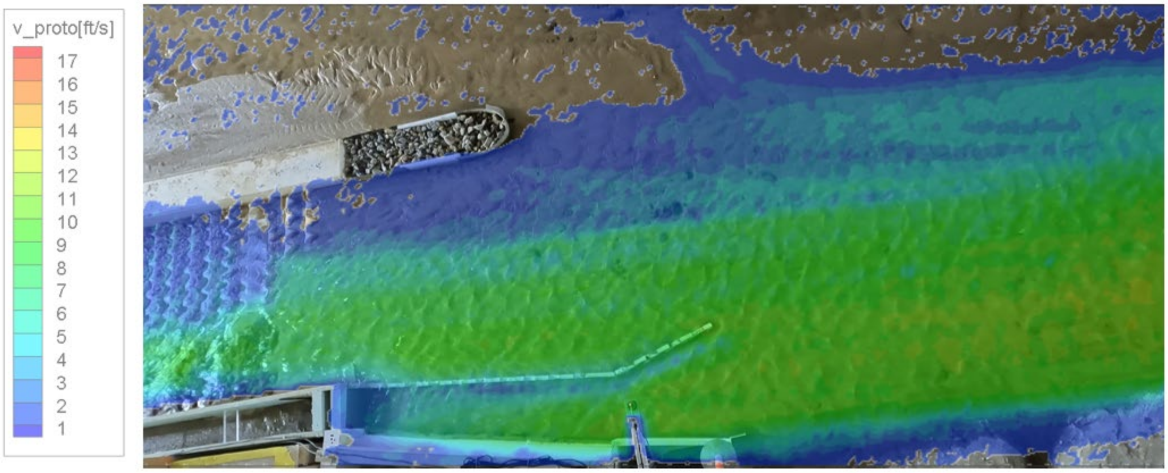


Figure 14. Previously collected LSPIV data for flushing channel closed at river flow of 3,000 ft^3/s. Velocities less than 0.1 ft/s have been made transparent (Mortensen and Shinbein, 2022).

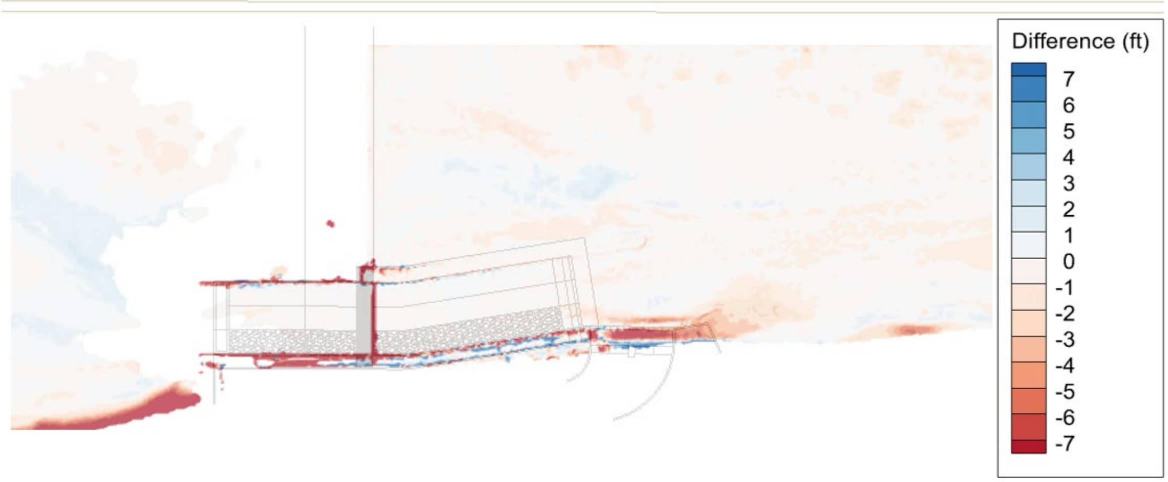


Figure 15. Bed elevation difference map between the post-run bathymetry with the flushing channel open minus the post-run bathymetry with the flushing channel closed for a river flow of 3,000 ft^3/s.

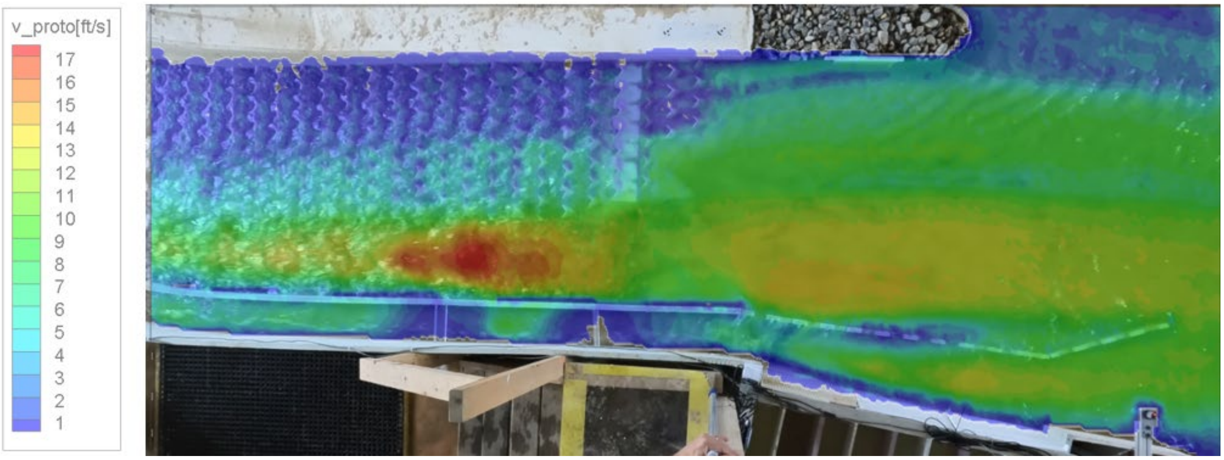


Figure 16. LSPIV data with flushing channel open at a river flow of 6,000 ft^3/s. Velocities less than 0.1 ft/s have been made transparent.

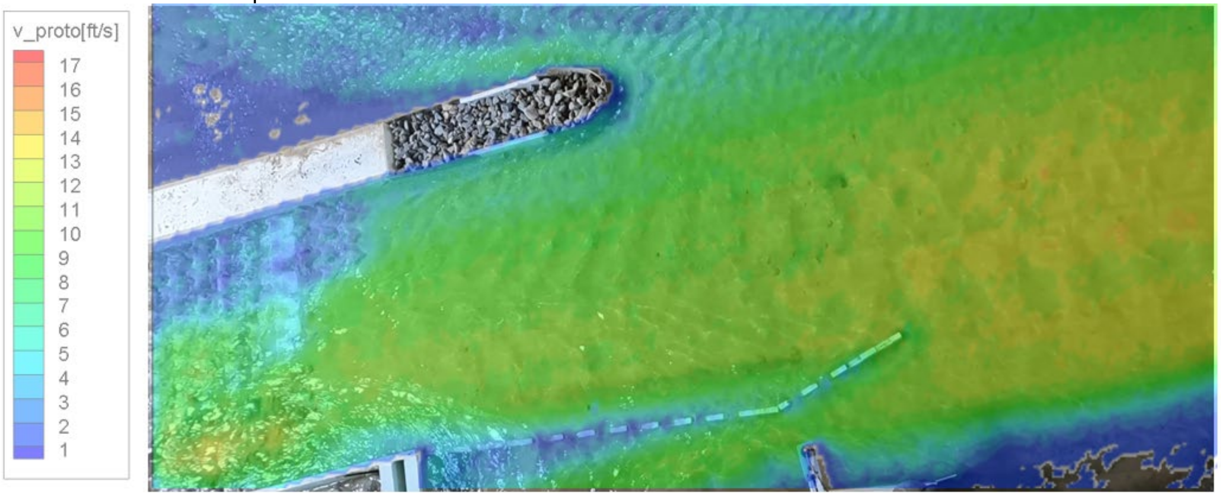


Figure 17. Previously collected LSPIV data for flushing channel closed at river flow of 6,000 ft^3/s. Velocities less than 0.1 ft/s have been made transparent (Mortensen and Shinbein, 2022).

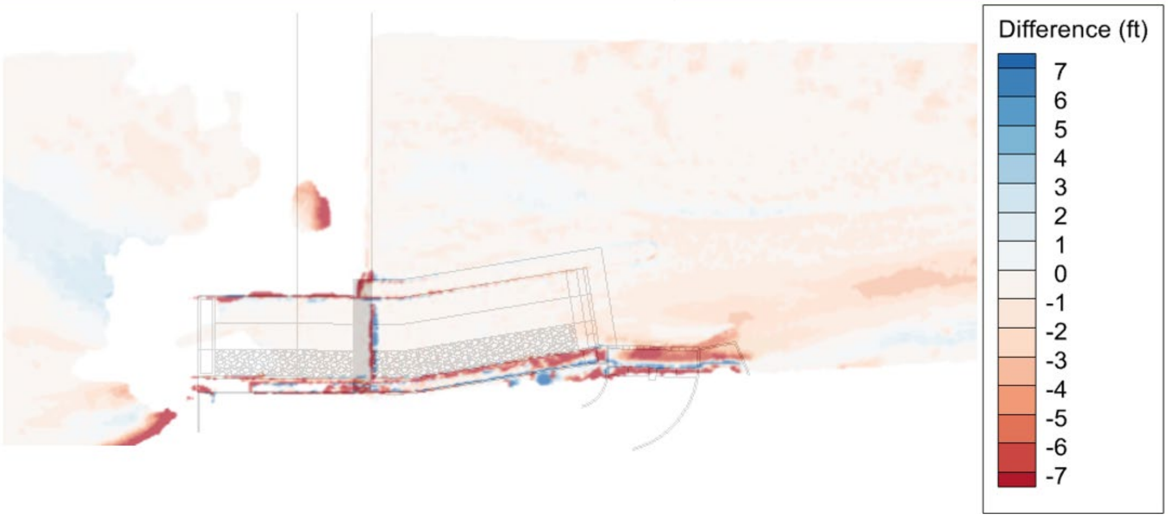


Figure 18. Bed elevation difference map between the post-run bathymetry with the flushing channel open minus the post-run bathymetry with the flushing channel closed for a river flow of 6,000 ft^3/s.

Figure 19. Flushing channel and lowered notch of castle wall open at a river flow of 6,000 ft^3/s. There is little interaction of the dye over the notches, with the majority passing through the flushing channel.

Figure 20. Downstream end of flushing channel open at river flow of 6,000 ft^3/s. The desander sluice is closed and there is little recirculation in the vicinity of the desander exit.

Desander Stress Testing

Based on the final geometry from the previous modeling effort, the desander system was stress tested under a series of different operating conditions. To best determine operating conditions of interest, Northwest Hydraulic Consultants observed the desander operating from November 14-16, 2022. During this model site visit, several preliminary tests were run including:

1) Flushing channel impacts on the desander system
2) Desander operation with one channel at a time and two channels at a time
3) Desander operation at elevated tailwater conditions

During the site visit, Northwest Hydraulic Consultants concluded that the flushing channel was functioning as intended and was effective in reducing sediment entrainment into the desander by drawing sediment from the apron prior to desander operation. The flushing channel was closed for the remaining desander tests to keep more sediment on the apron of the desander, which created a worst-case operating condition. More discussion on the efficacy of the castle training wall system and flushing operations can be found in the previous Training Wall Stress Testing section. Initial tests showed that flushing times were similar for each desander bay, and that further tests could be conducted using only one desander bay as representative of the system.

For all tests, sediment depths were measured at five locations in each channel of the desander (Figure 21) and in three locations within the sluice channel to estimate sluiced sediment volumes. Sluiceway flow rates and time needed to clear sediment from each channel were also recorded. These measurements were used to calculate the volume of sediment moved over the time required to sluice, referred to as a sluice ratio, SR (Equation 1). The volume of sediment mobilized was calculated separately for each channel since the geometry through the desander varies by channel. The spreadsheet utilized for this calculation was provided by Northwest Hydraulic Consultants. A key factor that was not considered in the sluicing ratio are differences in the saturation of the sediment in the channel. Although sediment was added into the channels while water was present, some tests were conducted using sediment that accumulated through diversion flow, as opposed to manual loading. Differences in sediment saturation and compaction in the channel changed the amount of time needed to sluice while not impacting the volume of sediment estimated.

$$SR = \frac{Sed\ Volume_{Before\ Sluice} - Sed\ Volume_{AfterSluice}}{Sluice\ Discharge\ (cfs) * Time\ to\ Sluice(s)} \quad (1)$$

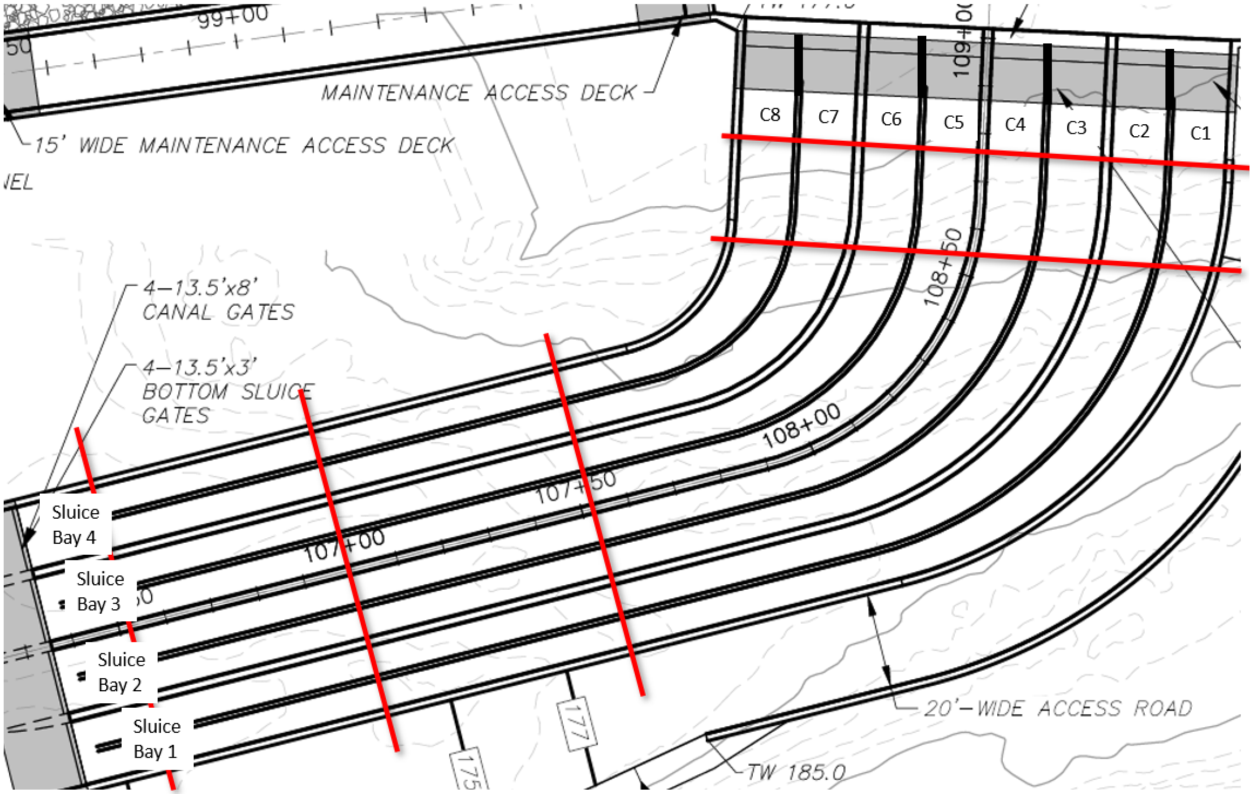


Figure 21. Desander system shown with eight gates, eight corresponding channels (denoted by C and a number), and four sluicing gates and bays. Red lines denote sections at which sediment depth was measured for each test.

Desander Operation with One or Two Channels

The first set of tests was performed at a river flow of 3,000 ft^3/s for three sluicing discharges. Once the diversion was set to 800 ft^3/s, the bottom sluice gate of bay 1, which connects channels 1 and 2, was opened. This procedure involved closing the canal gate at the downstream end of channels 1 and 2 near where the fish screen was located so all flow could be directed into the sluiceway. Tests were performed without changing the intake gate openings, which were partially open. As both intake gates were set to the same elevation, this condition represented a condition where two channels were sluiced at the same time. This intake gate setting provided the lowest sluice flow rate while still maintaining an acceptable diversion. The first test, which had the lowest sluice flow rate, took the longest time to clear the sediment out of channels 1 and 2, approximately 28 minutes in model scale (Table 12). However, as this scenario had a very high sediment volume sluiced, it had some of the highest sluice ratios at this flow rate. After the channels were cleared of sediment, the sluice gate was closed, the sediment was manually reloaded into the channels, and then the test was repeated at a higher sluice flow rate. For the following two tests at 3,000 ft^3/s river flow, even though the time to sluice decreased, the sluicing ratio decreased, meaning it was less efficient at clearing sediment out of the desander. This may also be in part because the sluicing channel was gradually filling with sediment as the testing progressed. Overall, however, there was little variation in the sluicing ratio with ranges from 0.78-0.87%.

An additional test was run at 6,000 ft^3/s with channels 1 and 2 open concurrently. Even though the sluice flow remained relatively low, the time to sluice was much shorter than the tests at 3,000 ft^3/s.

This is partially due to the low volume of sediment in the desander and sluicing channels prior to sluicing which produced an elevated sluice ratio, especially in channel 2. Other factors, such as tailwater elevation and sluice flow rate, had a more significant impact on sluice ratio and are discussed in greater detail below.

For tests where one channel was operated at a time, once the diversion was set to 800 ft^3/s, one intake gate was completely closed, while the other was lowered to the lowest setting (fully open). This was done while closing the canal gate at the downstream end of channels 1 and 2, where the fish screen is located, so all flow could be directed into the sluicing channel near the sluiceway. The bottom sluice gate of bay 1, which connects channels 1 and 2, was then opened. These one-channel tests had higher sluice flow than the two-channel tests because the intake gate was fully open for these tests. This produced a much shorter sluice time (Table 12), and a higher sluicing ratio at 3,000 ft^3/s when compared to sluicing both channels at the same time. The sluicing ratio increased from approximately 0.8% using two channels to 1% using one channel. For 6,000 ft^3/s, a significantly lower sluice ratio was observed than for the tests with operation of 2 channel gates. This is partially caused by the high sluiceway flow compared to the volume of sediment sluiced. The low sluice ratio is also caused by ingestion of sediment from the apron in front of the intake when the intake gate was fully lowered, preventing the channels from clearing as quickly as more sediment was constantly introduced. A higher sluice ratio could be obtained by not lowering one gate as far, thus reducing sediment ingestion.

Table 12. Sediment sluicing comparisons. Times are represented in model scale and volumes are represented in prototype scale.

Operation	River Flow (ft^3/s)	Tailwater (ft)	Time to Sluice (model minutes)	Sluiceway Flow (ft^3/s)	Volume of Sediment Sluiced (prototype ft^3)	Channel 1 Sluice Ratio	Channel 2 Sluice Ratio
Two Channels at a Time	3,000	144.5	28	181	12,900	0.87%	0.83%
	3,000	145.1	20	232	11,400	0.84%	0.78%
	3,000	144.6	18	338	14,300	0.80%	0.80%
	6,000	145.0	6, 3*	189	3,000	0.90%	1.56%
One Channel at a Time	3,000	144.3	10, N/A	494	15,300	1.05%	N/A
	1,500	143.4	2, 4	316	1,400	0.77%	1.10%
	6,000	145.0	5, 5	940	1,800	0.10%**	0.13%**

* The channels took significantly different amounts of time to sluice, even though they were operated concurrently. Thus, both times are noted.

**Desander channels 3 and 4 were operated for this test. The low sluice ratio is caused by the high sluice flow rate and the low volume of sediment in the channel at the start of the test.

Desander Operation with Elevated Tailwater Conditions

Additional desander stress testing was conducted at elevated tailwater conditions. As tailwater increased, more water was backed up into the sluiceway, reducing the effectiveness of the sluicing operations. These tests aimed to assess the impact of elevated tailwater on sluicing times (Table 13). All tests were run with a river flow of 3,000 ft^3/s. When comparing sluice ratio to tailwater, there is a clear negative trend. The higher the tailwater, the less effective the sluicing operations became. At a sluice flow of approximately 300 ft^3/s, the time to clear the channel of sediment was approximately 1 hour in model scale at the elevated tailwater. Similarly, the sluicing ratio drops from close to 1% to approximately 0.25%. The sluice ratio seems to drop approximately 0.1% for each additional foot of tailwater level.

Table 13. Sediment sluicing comparisons for elevated tailwater configurations. Times are represented in model scale and volumes are represented in prototype scale.

River Flow (ft^3/s)	Tailwater (ft)	Time to Sluice (model minutes)	Sluiceway Flow (ft^3/s)	Volume of Sediment Sluiced (prototype ft^3)	Channel 1 Sluice Ratio	Channel 2 Sluice Ratio
3,000	151.3	65	256	12,000	0.23%	0.24%
3,000	152.0	60	320	14,700	0.23%	0.22%

Debris Passage

Debris testing was performed using different techniques in the two physical models. In the 1:24 model, hay-like debris representing large clumps of Arundo was introduced at 6,000 ft^3/s, flow was increased to 12,000 ft^3/s while still introducing debris into the model, and finally flow was reduced back to 6,000 ft^3/s. When debris was first introduced, the only point of accumulation was on the right side of the hardened ramp where the top of the baffles were not fully submerged. Debris was inserted by hand into the hardened ramp to see if additional debris would accumulate elsewhere in the ramp, but only the top of the baffles on the right side of the ramp retained debris (Figure 22). All debris passed at 12,000 ft^3/s as the baffles were submerged. When flow was reduced to 6,000 ft^3/s, newly added debris clogged the intake trashrack, severely reducing diversion capacity but it did not accumulate in the hardened ramp (Table 14).

Figure 22. Debris mats accumulated in the 1:24 model at a river flow of 6,000 ft^3/s. All debris mats were flushed downstream at 12,000 ft^3/s.

Table 14. Debris testing in the 1:24 model.

River Flow (ft^3/s)	**Diversion Flow (ft^3/s)**	**Water Surface Elevation at Intake Entrance (ft)**	**Water Surface Elevation at Fish Screen (ft)**	**Tailwater (ft)**
6,000	820	164.5	160.2	145.7
12,000	1,200	166.8	165.6	147.2
6,000	565	164.8	156.4	145.6

In the 1:12 model, debris was introduced into the model in a similar manner to the 1:24 model. Once debris mats accumulated within the hardened ramp, point depth and velocity measurements were recorded in a grid surrounding the mats. After the test concluded, the debris mats were removed, and measurements were retaken in the same grid for comparison. Two types of debris mats were introduced. One was entirely comprised of simulated Arundo using hay. The other contained woody debris (dowels) in addition to hay (Figure 23). Woody debris is not expected to mobilize in the river at low flows and was thus only added at 6,000 ft^3/s.

During testing, debris accumulated on the right portion of the baffled section of the hardened ramp, as was seen in the 1:24 physical model. This was largely due to increased velocity as the ramp transitions to the low flow section. As a result, there were not clear trends for the impacts of debris mats on depth or velocity (Table 15 and Table 16). However, observations of the debris mats indicated that depth and velocity depended on where the debris mat was located within the baffled portion of the hardened ramp and the proximity to other debris mats.

The range of flows susceptible to debris accumulation appears to be relatively narrow at approximately 1,500 to 6,000 ft^3/s. Even within that flow range, woody debris rarely accumulated and would often clear from the baffled area when flow rates in the hardened ramp increased or decreased. Debris accumulation on the intake trashrack significantly reduced diversion capacity. This study did not include testing of debris removal methods.

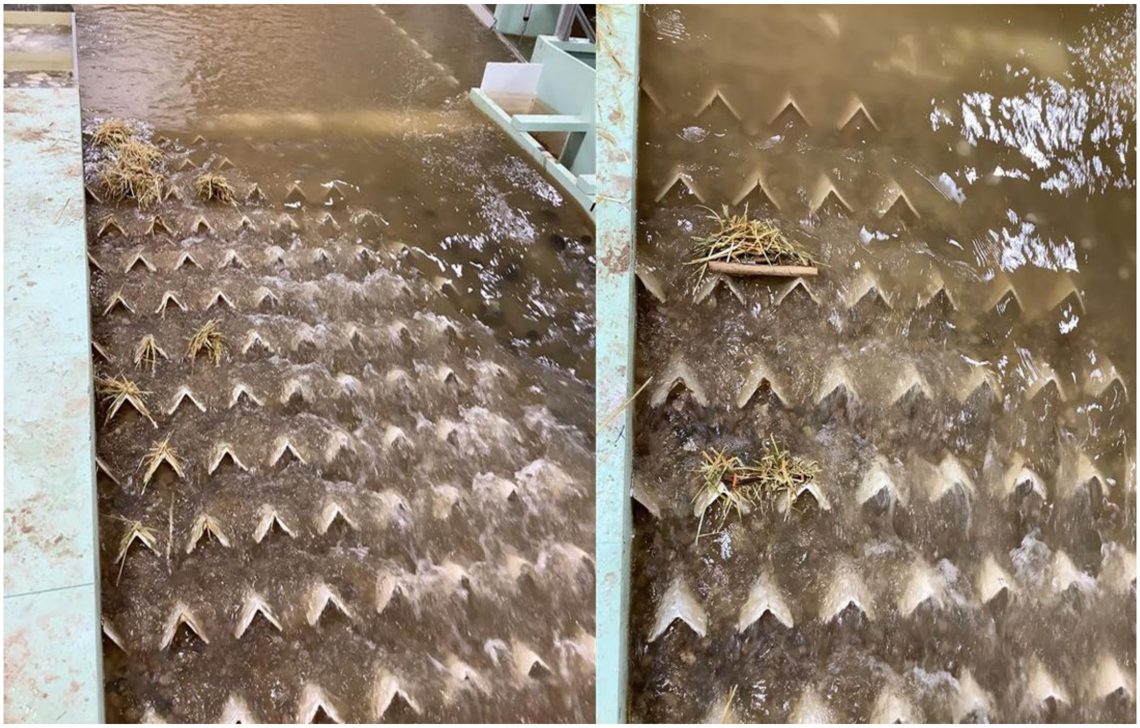


Figure 23. Debris in the 1:12 model showing debris mats (left) and debris mats with woody debris incorporated (right) at a river flow of 6,000 ft^3/s. Point depths and velocities were measured around the debris mats and between adjacent baffles.

Table 15. Comparison of depths and point velocities with and without debris mats at 1,500 ft^3/s in the 1:12 model. Woody debris was not utilized as there was not enough flow to mobilize the debris.

	Without Debris		**With Debris**			
Measurement Location with Respect to Accumulated Debris	**Depth (ft)**	**Velocity (ft/s)**	**Depth (ft)**	**Velocity (ft/s)**	**Change in Depth (ft)**	**Change in Velocity (ft/s)**
Upstream Left	2.0	11.8	2.3	4.4	-0.3	7.5
Upstream Right	1.6	1.4	1.9	1.0	-0.3	0.5
Middle Left	2.5	12.7	2.3	9.0	0.3	3.6
Middle Right	1.8	4.0	1.8	3.2	0.0	0.8
Downstream Left	2.4	12.2	2.0	11.7	0.4	0.5
Downstream Right	1.8	11.5	1.8	6.8	0.0	4.7

Table 16. Comparison of depths and point velocities with and without mats of debris and woody debris at 6,000 ft^3/s in the 1:12 model.

	Without Debris		With Debris				With Woody Debris			
Measurement Location with Respect to Accumulated Debris	**Depth (ft)**	**Velocity (ft/s)**	**Depth (ft)**	**Velocity (ft/s)**	**Depth Change**	**Velocity Change**	**Depth (ft)**	**Velocity (ft/s)**	**Depth Change (ft)**	**Velocity Change (ft/s)**
Upstream Left	4.0	4.8	4.9	4.9	-0.9	-0.1	3.0	7.6	1.0	-2.8
Upstream Right	3.5	4.4	3.4	3.2	0.1	1.2	3.0	8.5	0.5	-4.1
Middle Left	3.9	6.8	4.4	6.5	-0.5	0.4	3.3	8.2	0.6	-1.4
Middle Right	3.5	6.8	4.6	4.2	-1.1	2.6	3.5	5.2	0.0	1.6
Downstream Left	3.3	8.5	4.0	8.7	-0.8	-0.2	2.3	13.7	1.0	-5.3
Downstream Right	3.3	7.2	3.8	6.2	-0.5	0.9	1.8	12.1	1.5	-4.9

Figure 24. Accumulation of debris on trashrack post-test.

Variation in Upstream Approach Channel Orientation

To evaluate the resilience of the diversion to changing river conditions, a test was performed with the alignment of the upstream river channel thalweg manually moved toward the right bank, thus disconnected from the intake structure (Figure 25). After this approach condition was set in the 1:24 model, one test was run at 1,500 ft^3/s. Higher flows were not tested as the river reaches a bank-full condition starting at about 6,000 ft^3/s, which would make it easier to reconnect the diversion and the river.

Initially, flow from the hardened ramp passed through the downstream notch in the training wall to pool at the downstream end of the apron. The flushing channel was then opened, which resulted in headcutting of a flow path between the downstream notch in the training wall and the flushing channel; there was no formation of a continuous flow path to the thalweg upstream of the intake, and diversion flow remained limited. To establish upstream connectivity, excavation of a pilot channel was required upstream of the training wall. After cutting the pilot channel, operation of the flushing channel assisted in clearing sediment from the apron to re-establish operation of the diversion intake.

Two pilot channel alignments were tested, and each was similarly effective in reconnecting flow between the thalweg and intake on the left bank. After approximately 15 minutes, model time, the thalweg had shifted back to the diversion intake and the test was ended (Table 17).

Figure 25. Redirected upstream approach channel in the 1:24 model at 1,500 ft^3/s.

Table 17. Actions utilized to reconnect river to diversion intake in the 1:24 model at 1,500 ft^3/s.

Time (Model minutes)	Action	Result
0	Initial condition	Approach channel realigned toward river right; no flow connection with diversion inlet
1	Flushing channel opened	Headcutting of sediment along the apron to castle wall notch; no diversion flow
5	Pilot channel cut from active flow upstream of castle wall to upstream end of apron	Headcutting of sediment along the apron continued upstream; flow connection to diversion established
14	Pilot channel cut from active flow upstream of castle wall to upstream end of apron (steeper pilot channel)	Headcutting of sediment along the apron continued upstream; flow connection to diversion established

In the 1:12 model, the riverbed was similarly modified so that the upstream approach channel thalweg was disconnected from the diversion intake (Figure 26). For only this test with the modified upstream approach channel, the upstream flushing channel invert was still configured in a previous design configuration with the invert at elevation 154 ft instead of elevation 146 ft. The use of the modeled flushing channel to reestablish a connection between the upstream channel and the intake was expected to be less effective than if the invert was set to the lowered elevation, but nonetheless provided some indication of its potential utility.

River flows were set to 250 ft^3/s to see if connection could be reestablished at a significantly lower flow. Reconnection of the channel to the intake followed a similar sequence to what was observed in the 1:24 model at 1,500 ft^3/s, with excavation of a pilot channel between the thalweg and the apron upstream of the training wall required to reconnect flow. Operation of the flushing channel, even with the higher invert elevation as modeled, assisted with formation of the flow path to the intake.

Figure 26. Redirected channel at 250 ft^3/s river flow when the bed was first set.

Low-Flow Diversion Capacity Testing

Low-flow diversion capacity testing was completed to document the intake diversion flows that could be achieved under different sediment conditions in the 1:12 model. For this set of tests, the flushing channel invert elevation was set to the most recent design at elevation 146 ft. Maximum diversion rates for river flows between 250 and 2,000 ft^3/s were tested. In some situations, both blocked and unblocked configurations of the most downstream notch in the castle training wall were evaluated. (Table 18). A test series for a set river flow began with the intake gates fully closed and continued until the intake gates were fully open. This elevation is based on desirable operating conditions in the fish bypass at the end of the desander as it is currently designed. The target water surface elevation in the fish screen bay was between elevations 159.5 and 160.5 ft for maximum diversions, where possible. At lower flow rates, the river water surface elevation was lower than these desired elevations and, as such, the target elevation could not be reached within the intake. However, it should be noted, since this model did not have a desander installed in the intake, water surface elevations may be higher due to the presence of channel walls and differing geometries.

For the first set of tests, a typical amount of sediment was accumulated on the apron in front of the intake (as defined in Mortensen and Shinbein, 2022). For this condition, 80-100% of the river flow could be diverted into the intake for tested flow rates between 250 and 1,500 ft^3/s. At 250 ft^3/s river flow rate, more flow could be diverted with an unblocked downstream notch than with a blocked downstream notch.

Table 18. Low flow split tests with training wall installed and sediment set to typical accumulation conditions.

River Flow (ft^3/s)	Maximum Diversion (ft^3/s)	Training Wall Notch
2,000	1064*	Blocked
1,500	1300	Blocked
1,250	1000	Blocked
1,000	850	Blocked
750	650	Blocked
500	480	Blocked
250	200	Blocked
	250	Unblocked

* V-notch weir used to measure diversion flow rate was overtopped. Data value is estimated.

For the second set of tests, the apron of the diversion was manually loaded with sediment to the elevation of the castle wall notch, an approximate elevation of 160 ft, to determine the impacts of deposited sediment on diversion capacity (Figure 27, Table 19). For river flow rates of 250 and 500 ft3/s, less water was diverted with an apron full of sediment than with a typical amount of accumulated sediment.

Table 19. Low flow split tests with apron manually filled with sediment.

River Flow (ft^3/s)	Maximum Diversion (ft^3/s)	Training Wall Notch
2000	1450	Blocked
1500	1175	Blocked
1250	1075	Blocked
1000	890	Blocked
750	660	Blocked
	660	Unblocked
500	415	Blocked
	415	Unblocked
250	180	Blocked

Figure 27. Diversion intake filled with sediment during testing at a river flow of 250 ft^3/s.

For the next set of tests, sediment was removed from the apron to the extent possible (Figure 28). Only gates 5 through 8 of the diversion intake were operated (Table 20). Despite this, 88-100% of the river flow could be diverted. Without the sediment on the lower portion of the apron, water levels were too low to pass over the training wall notches into the hardened ramp, even when the downstream castle training wall notch was unblocked. However, when the intake gates were closed, nearly all flow could pass into the hardened ramp with the notch unblocked. Tests were also conducted with all 8 intake gates operating. Results showed slightly lower maximum diversion capacity for river flows of 750 ft^3/s and less in comparison to only gates 5-8 operating.

Flow split data between the diversion, hardened ramp, and dam crest are documented in Appendix A for incremental diversion rates. Sediment accumulation depths on the intake apron measured about halfway between castle wall and intake gate are documented for relevant tests in Appendix A.

Figure 28. Diversion intake cleared of sediment during testing with a river flow rate of 250 ft^3/s.

Table 20. Low flow split tests with no sediment on apron. Only intake gates 5 through 8 were operated.

River Flow (ft^3/s)	Maximum Diversion (ft^3/s)	Training Wall Notch
1,000	880	Unblocked
750	670	Unblocked
500	480	Unblocked
250	250	Unblocked

Table 21. Low flow split tests with no sediment on apron. All gates 1-8 were operated.

River Flow (ft^3/s)	Maximum Diversion (ft^3/s)	Training Wall Notch
1,250	1086	Unblocked
1,000	830	Unblocked
750	630	Unblocked
500	370	Unblocked
250	180	Blocked

Conclusions

Physical hydraulic modeling for operational and stress testing of the Freeman Diversion hardened ramp fish passage alternative was conducted to support design development efforts. Movable bed models were constructed and tested at a 1:24 model scale and a 1:12 model scale.

Key findings from both physical models are summarized below.

- Training Wall Stress Testing - When the flushing channel was open, high velocities were measured at the upstream end of the apron adjacent to the river, but the velocity decreased along the length of the apron to the intake entrance of the desander. Although most of the apron area contained velocities under 10 ft/s, sediment was quickly and efficiently removed, as demonstrated by a comparison of the bed elevation difference maps that show less sediment on the apron after flushing operations. When the flushing channel was open at a river flow of 1,500 ft^3/s, the baffles were completely dry, but some flow remained in the low flow section of the hardened ramp. In contrast, when the flushing channel was closed, the baffles in the hardened ramp were at least partially active at the same flow rate. At 6,000 ft^3/s river flow, the baffles were active regardless of whether the flushing channel was open or closed. While there was some flow that passed between the upstream notches of the castle wall, little water flowed over the lowered notch of the castle wall into the hardened ramp. Overall, the flushing channel and castle training wall configuration was effective at removing large amounts of sediment, especially as flow increased. Additionally, downstream of the hardened ramp, there was little flow interaction between the desander sluiceway and flushing channel outlets.

- Desander Stress Testing - The desander can be operated to sluice one bay at a time or multiple bays, concurrently, by partially opening the intake gates to control the sluice flow. Sluicing can be operated with total sluice flows as low as 200 ft^3/s at tailwater levels of 145 ft or lower. The sluicing ratio (ratio of volume of sediment removed to volume of water used) for these tests tended to be around 0.8% to 1% for tailwater levels between 144 and 145 ft but dropped to approximately 0.2% when tailwater levels rise above 151 ft. If an intake gate was fully lowered, the sluice ratio dropped to 0.1% even with low tailwater levels due to sediment ingestion from the apron. These impacts to efficacy of the desander should be monitored as the design development continues.

- Debris Testing - In the model, debris accumulation was observed from approximately 1,500 to 6,000 ft^3/s. Debris mats were retained on the right portion of the baffled section of the hardened ramp. The impact of debris mat retention on depth and velocity of flow in the baffled section was unclear. General trends showed the impact of the debris mat on depth and velocity heavily depended on where the debris mat was located within the baffled portion of the hardened ramp and the proximity to other mats. Woody debris rarely accumulated within the hardened ramp and would often clear from the baffled area when the flow in the hardened ramp increased or decreased. Debris accumulated on the trashrack on the intake structure. Once the trashrack was clogged with debris, diversion capacity was severely reduced.

- Variations in Upstream Approach Channel Orientation - Tests were performed to evaluate the resilience of the diversion to changes in approach channel position and alignment. With the upstream approach channel shifted so that the river thalweg was positioned near the right bank, the flushing channel could be operated to successfully shift the thalweg back to the left after a pilot channel was excavated, reconnecting it with the diversion intake.

- Low-Flow Diversion Capacity - Maximum diversion rates for a given discharge depended on the sediment depths on the apron in front of the intake and the presence of an operational bulkhead in the form of a lowered notch at the downstream end of the castle training wall. For all tested sediment conditions, 72-100% of the river flow could be diverted into the intake. With a 250 ft^3/s river flow rate and normal sediment accumulation on the intake apron, more flow could be diverted with an unblocked downstream notch than with a blocked downstream notch. When the apron was full of sediment at river flow rates of 250 and 500 ft^3/s, less water was diverted than under a normal sediment condition. Results with sediment removed from the apron showed slightly lower maximum diversion capacity for river flows from 250 to 750 ft^3/s with all 8 gates operating in comparison to only gates 5-8 operating. With no sediment present on the apron, flows could not overtop the lowered notch of the training wall to activate the hardened ramp, even when the bulkhead was open.

References

Mortensen, J., and M. Shinbein. 2022. Physical Hydraulic Modeling of the Freeman Diversion Hardened Ramp Fish Passage Alternative. Hydraulic Laboratory Report HL-2022-03. Bureau of Reclamation, Denver, Colorado.

FARO. 2022. *Scene 4.0.086*. Retrieved from https://www.faro.com/.

Northwest Hydraulic Consultants. 2020. Vern Freeman Diversion Hardened Ramp Fish Passage Improvements – Design Development. Report to United Water Conservation District.

Patalano, A. 2017. Rectification of Image Velocity Results (RIVeR): A Simple and User-Friendly Toolbox for Large Scale Water Surface Particle Image Velocimetry (PIV) and Particle Tracking Velocimetry (PTV). *Computers & Geosciences*, Vol. 109, 323-330.

Thielicke, W. and E. Stamhuis. 2014. PIVLab – Towards User-friendly, Affordable and Accurate Digital Particle Image Velocimetry in MATLAB. *Journal of Open Research Software 2(1):e30*.

Appendix A

Flow Split and Sediment Accumulation Model Data

Test	**4.1**
River Flow (cfs)	**2,000**
Description	**Normal sediment conditions with intake gates 1-8 operated. Blocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed 100	
Dam Elevation (ft)	163.1
Dam Flow (cfs)	1120
Ramp (cfs)	780
Diversion (cfs)	100
WL_Fish Screen	160.1

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 350	
Dam Elevation (ft)	162.8
Dam Flow (cfs)	800
Ramp (cfs)	850
Diversion (cfs)	350
WL_Fish Screen	161.8

	Average Elevation (ft)
Gate 1-4	162.3
Gate 5-8	161.8

Qdiv (cfs) = 500	
Dam Elevation (ft)	162.7
Dam Flow (cfs)	700
Ramp (cfs)	800
Diversion (cfs)	500
WL_Fish Screen	161.9

	Average Elevation (ft)
Gate 1-4	161.7
Gate 5-8	161.3

Qdiv (cfs) = 650	
Dam Elevation (ft)	162.6
Dam Flow (cfs)	600
Ramp (cfs)	750
Diversion (cfs)	650
WL_Fish Screen	161.4

	Average Elevation (ft)
Gate 1-4	161.4
Gate 5-8	161

Qdiv (cfs) = 800	
Dam Elevation (ft)	162.5
Dam Flow (cfs)	515
Ramp (cfs)	685
Diversion (cfs)	800
WL_Fish Screen	161.1

	Average Elevation (ft)
Gate 1-4	160.9
Gate 5-8	160.5

Qdiv (cfs) = 1064*	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	935.87
Diversion (cfs)	1064.14
WL_Fish Screen	159.3

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3
3	3
5	2
7	2.5
Notch	3.2

* V-notch weir used to measure diversion flow rate was overtopped. Value was estimated.

Test	**4.2**
River Flow	**1500**
Description	**Normal sediment conditions with intake gates 1-8 operated. Blocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed 90	
Dam Elevation (ft)	162.7
Dam Flow (cfs)	700
Ramp (cfs)	710
Diversion (cfs)	90
WL_Fish Screen	160.1
	157.8

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 350	
Dam Elevation (ft)	162.4
Dam Flow (cfs)	430
Ramp (cfs)	720
Diversion (cfs)	350
WL_Fish Screen	161.5

	Average Elevation (ft)
Gate 1-4	162
Gate 5-8	161.1

Qdiv (cfs) = 500	
Dam Elevation (ft)	162.3
Dam Flow (cfs)	350
Ramp (cfs)	650
Diversion (cfs)	500
WL_Fish Screen	161.1

	Average Elevation (ft)
Gate 1	161.6
Gate 2	160.8

Qdiv (cfs) = 650	
Dam Elevation (ft)	162.1
Dam Flow (cfs)	205
Ramp (cfs)	645
Diversion (cfs)	650
WL_Fish Screen	161.1

	Average Elevation (ft)
Gate 1	161
Gate 2	160.2

Qdiv (cfs) = 800	
Dam Elevation (ft)	161.9
Dam Flow (cfs)	79
Ramp (cfs)	621
Diversion (cfs)	800
WL_Fish Screen	160.6

	Average Elevation (ft)
Gate 1	159.8
Gate 2	160.3

Qdiv (cfs) = 1300	
Dam Elevation (ft)	160.7
Dam Flow (cfs)	0
Ramp (cfs)	200
Diversion (cfs)	1300
WL_Fish Screen	159.4

	Average Elevation (ft)
Gate 1	Fully Open
Gate 2	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.3
3	3.5
5	2
7	3
Notch	3.2

Test	**4.3**
River Flow	**1250**
Description	**Normal sediment conditions with intake gates 1-8 operated. Blocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed 70	
Dam Elevation (ft)	162.4
Dam Flow (cfs)	430
Ramp (cfs)	750
Diversion (cfs)	70
WL_Fish Screen	157.4

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 350	
Dam Elevation (ft)	162.1
Dam Flow (cfs)	205
Ramp (cfs)	695
Diversion (cfs)	350
WL_Fish Screen	161

	Average Elevation (ft)
Gate 1-4	161.8
Gate 5-8	161

Qdiv (cfs) = 500	
Dam Elevation (ft)	161.9
Dam Flow (cfs)	80
Ramp (cfs)	670
Diversion (cfs)	500
WL_Fish Screen	160.3

	Average Elevation (ft)
Gate 1-4	161.3
Gate 5-8	160.4

Qdiv (cfs) = 650	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	600
Diversion (cfs)	650
WL_Fish Screen	160.1

	Average Elevation (ft)
Gate 1-4	160
Gate 5-8	160.2

Qdiv (cfs) = 1000	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	250
Diversion (cfs)	1000
WL_Fish Screen	158.8

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.6
3	3.4
5	2
7	2.7
Notch	3.1

Test	**4.4**
River Flow	**1000**
Description	**Normal sediment conditions with intake gates 1-8 operated. Blocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed 75	
Dam Elevation (ft)	162
Dam Flow (cfs)	140
Ramp (cfs)	785
Diversion (cfs)	75
WL_Fish Screen	157.3

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 400	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	600
Diversion (cfs)	400
WL_Fish Screen	159.9

	Average Elevation (ft)
Gate 1-4	160.7
Gate 5-8	160.4

Qdiv (cfs) = 530	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	470
Diversion (cfs)	530
WL_Fish Screen	160

	Average Elevation (ft)
Gate 1-4	160
Gate 5-8	159.9

Qdiv (cfs) = 650	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	350
Diversion (cfs)	650
WL_Fish Screen	159.7

	Average Elevation (ft)
Gate 1-4	159.4
Gate 5-8	159.3

Qdiv (cfs) = 850	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	150
Diversion (cfs)	850
WL_Fish Screen	159.1

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.7
3	3.5
5	2.2
7	2.8
Notch	3.2

Test 4.5

River Flow 750

Description **Normal sediment conditions with intake gates 1-8 operated.**
Blocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed 85	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	665
Diversion (cfs)	85
WL_Fish Screen	155.3

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 215	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	535
Diversion (cfs)	215
WL_Fish Screen	160.2

	Average Elevation (ft)
Gate 1-4	161.1
Gate 5-8	160.6

Qdiv (cfs) = 320	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	430
Diversion (cfs)	320
WL_Fish Screen	159.8

	Average Elevation (ft)
Gate 1-4	160.2
Gate 5-8	160.2

Qdiv (cfs) = 490	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	260
Diversion (cfs)	490
WL_Fish Screen	159.1

	Average Elevation (ft)
Gate 1-4	159.3
Gate 5-8	159.4

Qdiv (cfs) = 650	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	100
Diversion (cfs)	650
WL_Fish Screen	159.6

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.4
3	3.6
5	2.2
7	2.8
Notch	3.1

Test	**4.6**
River Flow	**500**
Description	**Normal sediment conditions with intake gates 1-8 operated. Blocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed 100	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	400
Diversion (cfs)	100
WL_Fish Screen	156.5

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 220	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	280
Diversion (cfs)	220
WL_Fish Screen	159.3

	Average Elevation (ft)
Gate 1-4	160.1
Gate 5-8	160.1

Qdiv (cfs) = 350	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	150
Diversion (cfs)	350
WL_Fish Screen	159.3

	Average Elevation (ft)
Gate 1-4	159.3
Gate 5-8	159.4

Qdiv (cfs) = 480	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	20
Diversion (cfs)	480
WL_Fish Screen	158.8

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.7
3	3.7
5	2.2
7	2.3
Notch	3.2

Test 4.7

River Flow 250

Description Normal sediment conditions with intake gates 1-8 operated.
Blocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed 80	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	170
Diversion (cfs)	80
WL_Fish Screen	155.3

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 165	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	85
Diversion (cfs)	165
WL_Fish Screen	158.9

	Average Elevation (ft)
Gate 1-4	158.6
Gate 5-8	158.7

Qdiv (cfs) = 200	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	50
Diversion (cfs)	200
WL_Fish Screen	159.5

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.7
3	3.7
5	2.2
7	2.5
Notch	3.1

Test 4.8

River Flow 250

Description Normal sediment conditions with intake gates 1-8 operated.

Unblocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed 60	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	190
Diversion (cfs)	60
WL_Fish Screen	157.1

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 165	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	85
Diversion (cfs)	165
WL_Fish Screen	159.1

	Average Elevation (ft)
Gate 1-4	159.1
Gate 5-8	158.4

Qdiv (cfs) = 250	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	0
Diversion (cfs)	250
WL_Fish Screen	157.8

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Test 4.9

River Flow 2000

Description Apron full of sediment with intake gates 1-8 operated.
Blocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed 120	
Dam Elevation (ft)	163.1
Dam Flow (cfs)	1120
Ramp (cfs)	760
Diversion (cfs)	120
WL_Fish Screen	155.6

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 350	
Dam Elevation (ft)	162.9
Dam Flow (cfs)	900
Ramp (cfs)	750
Diversion (cfs)	350
WL_Fish Screen	160

	Average Elevation (ft)
Gate 1-4	162.6
Gate 5-8	161.8

Qdiv (cfs) = 500	
Dam Elevation (ft)	162.8
Dam Flow (cfs)	800
Ramp (cfs)	700
Diversion (cfs)	500
WL_Fish Screen	161.7

	Average Elevation (ft)
Gate 1-4	161.9
Gate 5-8	161.2

Qdiv (cfs) = 650	
Dam Elevation (ft)	162.7
Dam Flow (cfs)	700
Ramp (cfs)	650
Diversion (cfs)	650
WL_Fish Screen	162

	Average Elevation (ft)
Gate 1-4	161
Gate 5-8	160.4

Qdiv (cfs) = 800	
Dam Elevation (ft)	162.6
Dam Flow (cfs)	600
Ramp (cfs)	600
Diversion (cfs)	800
WL_Fish Screen	161.5

	Average Elevation (ft)
Gate 1-4	160.7
Gate 5-8	160

Qdiv (cfs) = 1450	
Dam Elevation (ft)	162
Dam Flow (cfs)	140
Ramp (cfs)	560
Diversion (cfs)	1300
WL_Fish Screen	160.5

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.3
2	5
3	7.6
4	8.9
Notch	9.6

Test 4.10

River Flow 1500

Description Apron full of sediment with intake gates 1-8 operated.
Blocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed 85	
Dam Elevation (ft)	162.7
Dam Flow (cfs)	700
Ramp (cfs)	715
Diversion (cfs)	85
WL_Fish Screen	157.9

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 350	
Dam Elevation (ft)	162.4
Dam Flow (cfs)	430
Ramp (cfs)	720
Diversion (cfs)	350
WL_Fish Screen	162

	Average Elevation (ft)
Gate 1-4	161.3
Gate 5-8	161.2

Qdiv (cfs) = 500	
Dam Elevation (ft)	162.3
Dam Flow (cfs)	350
Ramp (cfs)	650
Diversion (cfs)	500
WL_Fish Screen	161.4

	Average Elevation (ft)
Gate 1-4	160.9
Gate 5-8	160.9

Qdiv (cfs) = 650	
Dam Elevation (ft)	162.1
Dam Flow (cfs)	205
Ramp (cfs)	645
Diversion (cfs)	650
WL_Fish Screen	161.3

	Average Elevation (ft)
Gate 1-4	160.2
Gate 5-8	160.2

Qdiv (cfs) = 1175	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	325
Diversion (cfs)	1175
WL_Fish Screen	159.8

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	2.7
2	4.5
3	7.2
4	8.1
Notch	9.7

Test 4.11

River Flow 1250

Description Apron full of sediment with intake gates 1-8 operated.
Blocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed 110	
Dam Elevation (ft)	162.3
Dam Flow (cfs)	350
Ramp (cfs)	790
Diversion (cfs)	110
WL_Fish Screen	155.6

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 350	
Dam Elevation (ft)	162.1
Dam Flow (cfs)	205
Ramp (cfs)	695
Diversion (cfs)	350
WL_Fish Screen	161.5

	Average Elevation (ft)
Gate 1-4	161.7
Gate 5-8	160.6

Qdiv (cfs) = 500	
Dam Elevation (ft)	162
Dam Flow (cfs)	140
Ramp (cfs)	610
Diversion (cfs)	500
WL_Fish Screen	161.3

	Average Elevation (ft)
Gate 1-4	161
Gate 5-8	159.9

Qdiv (cfs) = 650	
Dam Elevation (ft)	161.6
Dam Flow (cfs)	0
Ramp (cfs)	600
Diversion (cfs)	650
WL_Fish Screen	160.2

	Average Elevation (ft)
Gate 1-4	160.5
Gate 5-8	159.4

Qdiv (cfs) = 1075	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	175
Diversion (cfs)	1075
WL_Fish Screen	159.3

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	2.1
2	4.1
3	6.4
4	9.8
Notch	9.3

Test 4.12

River Flow 1000

Description Apron full of sediment with intake gates 1-8 operated.

Blocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed 100	
Dam Elevation (ft)	161.8
Dam Flow (cfs)	22
Ramp (cfs)	878
Diversion (cfs)	100
WL_Fish Screen	156.2

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 400*	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	600
Diversion (cfs)	400
WL_Fish Screen	160.7

	Average Elevation (ft)
Gate 1-4	160.8
Gate 5-8	160

Qdiv (cfs) = 530	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	470
Diversion (cfs)	530
WL_Fish Screen	160.1

	Average Elevation (ft)
Gate 1-4	160.3
Gate 5-8	159.5

Qdiv (cfs) = 650	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	350
Diversion (cfs)	650
WL_Fish Screen	159.9

	Average Elevation (ft)
Gate 1-4	159.5
Gate 5-8	158.9

Qdiv (cfs) = 890	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	110
Diversion (cfs)	890
WL_Fish Screen	158.9

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.2
2	5.2
3	5.8
4	9.2
Notch	9.8

Test 4.13

River Flow 750

Description Apron full of sediment with intake gates 1-8 operated.
Blocked and unblocked downstream training wall notch.

Unblocked/Notch Open

Flows Splits

Qdiv (cfs) = 660	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	90
Diversion (cfs)	660
WL_Fish Screen	158.9

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Blocked/Notch Closed

Flows Splits

Qdiv (cfs) = Closed 80	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	670
Diversion (cfs)	80
WL_Fish Screen	158.5

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 215	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	535
Diversion (cfs)	215
WL_Fish Screen	160.3

	Average Elevation (ft)
Gate 1-4	161.4
Gate 5-8	160.4

Qdiv (cfs) = 330	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	420
Diversion (cfs)	330
WL_Fish Screen	160.1

	Average Elevation (ft)
Gate 1-4	160.6
Gate 5-8	159.6

Qdiv (cfs) = 480	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	270
Diversion (cfs)	480
WL_Fish Screen	159.4

	Average Elevation (ft)
Gate 1-4	159.5
Gate 5-8	158.9

Qdiv (cfs) = 640	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	110
Diversion (cfs)	640
WL_Fish Screen	159

	Average Elevation (ft)
Gate 1-4	157.8
Gate 5-8	157.2

Qdiv (cfs) = 660	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	90
Diversion (cfs)	660
WL_Fish Screen	158.9

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	3.4
2	5.8
3	7.3
4	8.4
Notch	10.5

Test	**4.14**
River Flow	**500**
Description	**Apron full of sediment with intake gates 1-8 operated.** **Blocked and unblocked downstream training wall notch.**

Unblocked/Notch Open

Flows Splits

Qdiv (cfs) = 415	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	85
Diversion (cfs)	415
WL_Fish Screen	157.9

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Blocked/Notch Closed

Flows Splits

Qdiv (cfs) = Closed 95	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	405
Diversion (cfs)	95
WL_Fish Screen	155.7

Corresponding Gate Settings

(Closed)	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 220	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	280
Diversion (cfs)	220
WL_Fish Screen	159

	Average Elevation (ft)
Gate 1-4	159.6
Gate 5-8	159.1

Qdiv (cfs) = 350	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	150
Diversion (cfs)	350
WL_Fish Screen	158.9

	Average Elevation (ft)
Gate 1-4	158.1
Gate 5-8	157.7

Qdiv (cfs) = 415	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	85
Diversion (cfs)	415
WL_Fish Screen	157.9

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	4.5
2	5.6
3	7.2
4	8.2
Notch	7.7

Test	**4.15**
River Flow	**250**
Description	**Apron full of sediment with intake gates 1-8 operated. Blocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed 60	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	190
Diversion (cfs)	60
WL_Fish Screen	155.9

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 160	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	90
Diversion (cfs)	160
WL_Fish Screen	159.3

	Average Elevation (ft)
Gate 1-4	159
Gate 5-8	158.4

Qdiv (cfs) = 180	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	70
Diversion (cfs)	180
WL_Fish Screen	159.2

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Apron Sediment Depths

Gate	Prototype Depth (ft)
1	4.5
2	5.7
3	7.6
4	8.4
Notch	8.7

Test	**4.16**
River Flow	**1000**
Description	**No sediment on intake apron with only gates 5-8 operated. Unblocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed	
Dam Elevation (ft)	162.1
Dam Flow (cfs)	200
Ramp (cfs)	700
Diversion (cfs)	100
WL_Fish Screen	159.3

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 200	
Dam Elevation (ft)	162
Dam Flow (cfs)	140
Ramp (cfs)	660
Diversion (cfs)	200
WL_Fish Screen	161.1

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	161.3

Qdiv (cfs) = 400	
Dam Elevation (ft)	161.7
Dam Flow (cfs)	0
Ramp (cfs)	600
Diversion (cfs)	400
WL_Fish Screen	160.8

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	160

Qdiv (cfs) = 880 (max)	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	120
Diversion (cfs)	880
WL_Fish Screen	159.8

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	Fully Open

No significant sediment accumulation during test

Test	**4.17**
River Flow	**750**
Description	**No sediment on intake apron with only gates 5-8 operated. Unblocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	635
Diversion (cfs)	115
WL_Fish Screen	156.6

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 300	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	450
Diversion (cfs)	300
WL_Fish Screen	160.5

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	159.6

Qdiv (cfs) = 450	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	300
Diversion (cfs) *	450
WL_Fish Screen	159.4

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	158.9

Qdiv (cfs) =670	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	80
Diversion (cfs)	670
WL_Fish Screen	159.6

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	Fully Open

Test 4.18

River Flow 500

Description No sediment on intake apron with only gates 5-8 operated.
Unblocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	420
Diversion (cfs)	80
WL_Fish Screen	155.8

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 200 (v-notch)	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	300
Diversion (cfs)	200
WL_Fish Screen	160.5

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	160.1

Qdiv (cfs) = 480	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	20
Diversion (cfs)	480
WL_Fish Screen	158.5

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	Fully Open

No significant sediment accumulation during test

Test	**4.19**
River Flow	**250**
Description	**No sediment on intake apron with only gates 5-8 operated. Unblocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed (75 cfs)	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	175
Diversion (cfs)	75
WL_Fish Screen	155.6

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 120	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	130
Diversion (cfs)	120
WL_Fish Screen	159.2

	Average Elevation (ft)
Gate 1-4	Closed (gates 1-2), 161 (gates 3-4)
Gate 5-8	156.2

Qdiv (cfs) = 250	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	0
Diversion (cfs)	250
WL_Fish Screen	157.8

	Average Elevation (ft)
Gate 1-4	Closed
Gate 5-8	Fully Open

Sediment at end of test on Apron

Gate	Prototype Depth (ft)
1	1
2	0.7
3	0.7
4	1
Notch	0.6

Test	**4.20**
River Flow	**1250**
Description	**No sediment on intake apron with gates 1-8 operated. Unblocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed	
Dam Elevation (ft)	162.5
Dam Flow (cfs)	515
Ramp (cfs)	655
Diversion (cfs)	80
WL_Fish Screen	160.1
WL U/S Intake	162.5

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 250	
Dam Elevation (ft)	162.4
Dam Flow (cfs)	430
Ramp (cfs)	570
Diversion (cfs)	250
WL_Fish Screen	161.2
WL U/S Intake	162.3

	Average Elevation (ft)
Gate 1-4	162.4
Gate 5-8	161.4

Qdiv (cfs) = 500	
Dam Elevation (ft)	162.1
Dam Flow (cfs)	200
Ramp (cfs)	550
Diversion (cfs)	500
WL_Fish Screen	160.8
WL U/S Intake	162

	Average Elevation (ft)
Gate 1-4	161.4
Gate 5-8	160.5

Qdiv (cfs) = 1086	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	164
Diversion (cfs)	1086
WL_Fish Screen	158.9
WL U/S Intake	159.2

	Average Elevation (ft)
Gate 1-4	Fully Lowered
Gate 5-8	Fully Lowered

Test	**4.21**
River Flow	**1000**
Description	**No sediment on intake apron with gates 1-8 operated. Unblocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed (130 cfs)	
Dam Elevation (ft)	162
Dam Flow (cfs)	140
Ramp (cfs)	730
Diversion (cfs)	130
WL_Fish Screen	155.9
WL U/S Intake	162.1

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1	Fully Closed
Gate 2	Fully Closed

Qdiv (cfs) = 350	
Dam Elevation (ft)	161.6
Dam Flow (cfs)	0
Ramp (cfs)	650
Diversion (cfs)	350
WL_Fish Screen	160.5
WL U/S Intake	161.6

	Average Elevation (ft)
Gate 1	161.2
Gate 2	160.5

Qdiv (cfs) = 500	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	500
Diversion (cfs)	500
WL_Fish Screen	160.4
WL U/S Intake	161.2

	Average Elevation (ft)
Gate 1	160.3
Gate 2	160

Qdiv (cfs) = 830	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	170
Diversion (cfs)	830
WL_Fish Screen	159.1
WL U/S Intake	159.8

	Average Elevation (ft)
Gate 1	Fully Open
Gate 2	Fully Lowered

Test	**4.22**
River Flow	**750**
Description	**No sediment on intake apron with gates 1-8 operated. Unblocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed (110 cfs)	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	640
Diversion (cfs)	110
WL_Fish Screen	156.4
WL U/S Intake	161.5

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 400	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	350
Diversion (cfs)	400
WL_Fish Screen	159.2
WL U/S Intake	160.7

	Average Elevation (ft)
Gate 1-4	160.2
Gate 5-8	159.5

Qdiv (cfs) = 630	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	120
Diversion (cfs)	630
WL_Fish Screen	159
WL U/S Intake	159.3

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Lowered

Test	**4.23**
River Flow	**500**
Description	**No sediment on intake apron with gates 1-8 operated. Unblocked downstream training wall notch.**

Flows Splits

Qdiv (cfs) = Closed (90 cfs)	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	410
Diversion (cfs)	90
WL_Fish Screen	157.4
WL U/S Intake	160.7

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 200	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	300
Diversion (cfs)	200
WL_Fish Screen	159.8
WL U/S Intake	160.3

	Average Elevation (ft)
Gate 1-4	160.4
Gate 5-8	161.8

Qdiv (cfs) = 370	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	130
Diversion (cfs)	370
WL_Fish Screen	159.4
WL U/S Intake	159.4

	Average Elevation (ft)
Gate 1-4	Fully Open
Gate 5-8	Fully Open

Test 4.24

River Flow 250

Description No sediment on intake apron with gates 1-8 operated.
Blocked downstream training wall notch.

Flows Splits

Qdiv (cfs) = Closed (~75cfs)	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	175
Diversion (cfs)	75
WL_Fish Screen	157.7
WL U/S Intake	159.9

Corresponding Gate Settings

	Average Elevation (ft)
Gate 1-4	Fully Closed
Gate 5-8	Fully Closed

Qdiv (cfs) = 120	
Dam Elevation (ft)	dry
Dam Flow (cfs)	0
Ramp (cfs)	130
Diversion (cfs)	120
WL_Fish Screen	158.5
WL U/S Intake	159.5

	Average Elevation (ft)
Gate 1-4	159.3
Gate 5-8	159.3

Qdiv (cfs) = 180	
Dam Elevation (ft)	Dry
Dam Flow (cfs)	0
Ramp (cfs)	70
Diversion (cfs)	180
WL_Fish Screen	159
WL U/S Intake	159.1

	Average Elevation (ft)
Gate 1-4	Fully Lowered
Gate 5-8	Fully Lowered