Photo source: United Water Conservation District

Freeman Diversion
Hardened Ramp Design Hydraulic Plans

Prepared by:

Northwest Hydraulic Consultants Inc.
200 S. Los Robles Avenue, Suite 405
Pasadena, CA 91101
Tel: (626) 440-0080
www.nhcweb.com

NHC Project Contact:
Brady McDaniel, P.E.
Principal Engineer

NHC Reference 5007701
Final Report, Rev. R2
August 2023

Prepared for:

United Water Conservation District
1701n Lombard Street
Oxnard, CA 93030

Mr. Craig Morgan, P.E.
Engineering Manager
craigm@unitedwater.org

Document Tracking

Date	Revision No.	Reviewer	Issued for
February 9, 2023	R0	EWW	Client Review
February 22, 2023	R1	EWW	Final Report
August 12th 2023	R2	EWW	Revised Final

Report prepared by:

Barry Chilibeck, P Eng
Principal Engineer

Brady McDaniel, PE
Principal Engineer

02/22/2023
Jose Vasquez, PhD, P Eng
Principal Engineer

Robert Odell, PhD, PE
Senior Hydrotechnical Engineer

Report reviewed by:

Ed Wallace, PE
Senior Project Engineer

DISCLAIMER

This report has been prepared by Northwest Hydraulic Consultants Inc. for the benefit of United Water Conservation District for specific application to the Freeman Diversion . The information and data contained herein represent Northwest Hydraulic Consultants Inc. best professional judgment in light of the knowledge and information available to Northwest Hydraulic Consultants Inc. at the time of preparation and was prepared in accordance with generally accepted engineering and geoscience practices.

Except as required by law, this report and the information and data contained herein are to be treated as confidential and may be used and relied upon only by United Water Conservation District, its officers and employees. Northwest Hydraulic Consultants Inc. denies any liability whatsoever to other parties who may obtain access to this report for any injury, loss or damage suffered by such parties arising from their use of, or reliance upon, this report or any of its contents.

CREDITS AND ACKNOWLEDGEMENTS

The authors would like to thank for the support provided during the project, in particular:

- Craig Morgan, Engineering Manager, UWCD
- Murray McEachron, Principal Hydrologist, UWCD
- Brian Collins, COO, UWCD
- Larry Weber, University of Iowa.

The following NHC personnel participated in the study:

- Ed Wallace, Senior Project Engineer
- Hank Fehlman, Principal Engineer
- Brady McDaniel, Principal Engineer
- Jose Vasquez, Principal Engineer
- Rob Odell, Senior Engineer
- Daniel Maldoff, EIT, Hydrotechnical Engineer
- Pablo Rodriguez, P.Eng., Geomatics Engineer
- Cass Zhao, CAD Technician
- Travis Shinkle, Senior CAD Designer
- Barry Chilibeck, Principal Engineer.

EXECUTIVE SUMMARY

This design development report describes the hydraulic performance of the Hardened Ramp Fish Passage Alternative. United Water Conservation District (UWCD) developed the Hardened Ramp Fish Passage design for the Vern Freeman Diversion as part of a Multi-Species Habitat Conservation Plan (MSHCP). The design provides volitional fish passage through a design flow range of 45 cubic feet per second (ft^3/s) to 6,000 ft^3/s, as well as pass sediment and debris transported by river processes. The design has been developed in collaboration with National Marine Fisheries Service (NMFS) and California Department of Fish and Wildlife (CDFW) with the UWCD design team, using physical modeling as well as numerical analysis and engineering design. Design objectives for fish passage, sediment management and diversion yield were used to guide the project.

The key geometric and hydraulic features of the design are outlined in the document and preliminary hydraulic plans are appended to the report. This document builds on the engineering design reporting provided in the Hardened Ramp 30% Design Report (NHC and GEI, 2019), the 2020 Design Development Report (NHC, 2020), and Design Memorandum (NHC, September 2021). This report provides a full description of the components of the Hardened Ramp and Diversion. Design components that were provided in these earlier reports are summarized while components that were significantly advanced or altered are discussed in detail. Physical modeling completed by the USBR (2022,2023) was utilized in conjunction with hydraulic engineering to advance the designs, most notably for improvement of ramp hydraulics and sediment and debris management.

The ramp hydraulics were initially tested for baseline conditions, representing design conditions defined in previous reporting, by the USBR (2022) utilizing 1:12 and 1:24 scale physical models. The results showed the ramp hydraulics performed as expected from earlier designs with improvements made in the ramp low flow channel and approach hydraulics to the ramp. A revised low flow channel boulder arrangement, extension of rock placement, addition of training wall and bullnose extension were used to improve ramp hydraulics. Review of velocity and depth results collected on the model showed viable hydraulics and longitudinal pathways for volitional fish passage through a range of discharges.

A unique sediment management system was developed that consists of a flushing channel to remove sediment deposition outside of the intake and a desander to remove sediment that enters the intake. The flushing channel efficiency was improved through the addition of a training wall, sloped apron and lowered invert. The desander can isolate individual bays and remove sediment while still allowing for diversion, and is utilized most efficiently for tailwater elevations that allow for free surface flow, below 150 ft . Tests were run on the flushing channel and desander through a range of conditions to determine their efficiency (USBR 2023). These features are complementary, and both structures are required to effectively manage sediment. The physical model was used to assess the potential impacts of debris through a range of flows. The ramp design was shown to shed debris through the low flow channel with some potential for debris to rack on the baffles which would either be mobilized during high flows or removed after flood events.

The fish screen and bypass concept designs were developed to provide diversion through a range of flows up to the proposed 750 ft^3/s diversion design. The concept includes dual 170 ft long primary screens along with an integrated fish collector and bypass system. The concept was designed to meet NMFS criteria for minimum open area, sweeping and screen approach velocities.

The design includes operational flexibility through features such as the training wall, intake crest gate elevations and hardened ramp gates to meet potential diversion flow splits and to provide resilience for changing conditions. The project includes multiple sets of gates (intake, isolation, hardened ramp, etc.) that are utilized for operations. Initial design of these gates was completed in consultation with manufacturers and will advance through detailed design.

While this report provides a complete hydraulic design and associated plans for the system it is recognized that additional hydraulic analysis or refinements may be necessary as the overall design advances to 60%, 90% and Final construction documents. During the more detailed civil, structural, geotechnical and mechanical design, components items such as wall thickness and gate dimensions may need to be revised. Additional hydraulics would be needed to support these change. Additional analysis may also be necessary to address questions from regulatory agencies or if conditions have notably changed in the field.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

APPENDICES

ABBREVIATIONS

Acronym / Abbreviation	Definition
Agencies	CDFW and NMFS
CDFW	California Department of Fish and Wildlife
CFD	Computational Fluid Dynamics
DDR	Design Development Report
FCL	Flood Construction Level
HBOD	Hydraulic Basis of Design
MSHCP	Multi Species Habitat Conservation Plan
NHC	Northwest Hydraulic Consultants
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
PIV	Particle Image Velocimetry
RCC	Roller Compacted Concrete
SR	Sluicing Ratio
UWCD	Uniter Water Conservation District
USBR	United States Bureau of Reclamation
TW	Tail water
WSE	Water Surface Elevation

1 Background, Purpose and Overview

1.1 Project Background

United Water Conservation District (UWCD) is currently developing a Multiple Species Habitat Conservation Plan (MSHCP) to protect endangered Southern California Steelhead, Pacific Lamprey and other fish and wildlife species associated with operation of the Freeman Diversion on the Santa Clara River in Ventura County. The Hardened Ramp fish passage alternative has been developed by UWCD to provide safe volitional upstream and downstream fish passage at the Vern Freeman Diversion on the Santa Clara River, CA. The project includes the Hardened Ramp fish, new headworks and modifications that provide UWCD a diversion capacity of 750 ft^3/s. Hardened Ramp concepts developed by UWCD and their consultants prior to 2019 were modified in a Draft Basis of Hydraulic Design Report and 30 Percent Design plans in July 2019 (NHC and GEI, 2019)

The Hardened Ramp is a 480 ft long in-river feature that functions as a high gradient channel and replaces a section of the diversion structure to provide fish passage. A key feature in the modified concept included the use of an approximately 90 ft wide asymmetric ramp cross sectional shape with a rock-lined low flow channel and baffled shoulder. The design is intended to pass sediment and debris down the ramp while providing suitable hydraulics for fish passage at river discharges between 45 ft^3/s and 6,000 ft^3/s. The design allows simultaneous water diversion into the intake for UWCD's canal and fish passage in the ramp without flow control at the crest of the ramp (i.e., no flow control or reservoir impoundment), and the ramp was designed to carry at least the initial 1,200 ft^3/s of river flow before flow spills over the diversion dam crest. The diversion intake was relocated just upstream of the crest of the ramp in the modified design and utilizes overshot crest gates to control diversion flows.

The modified design concept was developed in collaboration between UWCD, National Marine Fisheries Service (NMFS) and California Department of Fish and Wildlife (CDFW). UWCD responded to NMFS and CDFW comments on the previous Hardened Ramp design and both agencies provided input during the hydraulic design process. The modified design concept was generally received favorably by NMFS and CDFW, and an extensive set of comments were provided (NMFS, 2019) suggesting potential design refinements and topics to be addressed in additional analysis or physical modeling. UWCD continued design development work on the Hardened Ramp in collaboration with NMFS and CDFW and produced a Design Development Report (DDR) that included a geomorphic assessment of river response to the proposed ramp; two-dimensional (2D) modeling of flow patterns at the diversion intake, ramp, diversion dam crest, and downstream channel; analysis of various ramp and baffle variations; and a preliminary assessment of diversion operations (NHC, 2020).

In the DDR, eight design modifications (MOD-1 through MOD-8) for the Hardened Ramp were tested in a three-dimensional (3D) numerical model, and one of them identified as MOD-6 was recommended. Extensive review and collaboration with NMFS and CDFW occurred during the DDR process, resulting in consensus on design approach. However, the DDR process also identified limitations in the numerical analysis and several important topics that would need to be addressed in physical modeling.

UWCD review of the DDR identified concerns regarding the diversion capability of the Hardened Ramp MOD-6 configuration relative to proposed operational flow splits (diversion flow vs. flow bypassed downstream) in the MSHCP. NHC confirmed the operational limitations of the MOD-6 design by comparing maximum feasible diversion flows against bypass flows for a range of typical fish flow conditions (NHC, 2021).

On review of the diversion capability results and with direction from UWCD, NHC considered several alternatives to the MOD-6 configuration to increase operational flexibility and diversion capability. These included potential physical changes to the ramp geometry and crest elevation, conveyance canal, and intake sill, and potential changes to operations using the ramp crest gates, a ramp headworks, and low-lift pumping. On review of the various options and with guidance from UWCD, the most effective approach for increasing operational flexibility and diversion capability was determined to be increasing the crest elevation of the ramp while maintaining the cross section and basic hydraulic characteristics developed in the DDR. This modification with a 2 ft crest raise was described as MOD-9, which became UWCD's preferred configuration. The process and supporting analysis for developing the MOD-9 design configuration was provided in a draft report and plan set NHC, 2021)(NHC, 2021).

The MOD-9 configuration retains a gated flushing channel adjacent to the ramp and downstream of the apron in front of the intake. The flushing channel was included in the DDR report as a feature to be included and not included in variations to be tested in physical modeling. The MOD-9 configuration increases diversion capability due to the change in relative elevation of the ramp rating curve compared to the canal, allowing a larger fraction of the total river flow to be diverted. The diversion can be regulated to reduce flows below the maximum diversion capability with a combination of the intake crest gates and the slide gates downstream of the fish screen bay. The crest gates would normally be operated at least partially up to maintain a vertical offset between the ramp crest and the diversion inlet, providing a physical barrier against bedload entrainment. The gates would be capable of being operated independently, thus allowing one or more section to be fully lowered when sediment transport rates are reduced in low flow conditions.

Management of the high sediment loads in the Santa Clara River is a key component to a successful design at this site. Preliminary sediment management alternatives for further investigation in the physical model were outlined (NHC, 2021), including use of the sluice or flushing channel; modifications of intake geometry to include a guide wall, submerged sill or bedload bypass conduits external to the intake; modification of the intake to include an interior sluice or vortex tube sediment bypass; apron modifications such as vanes or weirs to shed sediment towards the ramp; and operational schemes to promote sediment transport in the Hardened Ramp while maintaining diversions. Many of these options are constrained by the vertical space available between the ramp crest and the intake operating level. The flushing channel and internal sluice were identified as options for initial testing in the physical model.

The Hardened Ramp is a complex hydraulic structure and performance will be affected by river behavior, sediment and debris loads, and suitability of near-field hydraulics for fish passage. These topics required additional investigation to reduce uncertainty and refine the design. The next phase of work utilized physical modeling, which is considered the best tool for addressing sediment and debris performance as well as fine- and large-scale fish passage hydraulics.

The US Bureau of Reclamation (USBR, 2022) constructed and completed testing on two physical models of the hardened ramp alternative at scales of 1:12 and 1:24. The 1:24 scale model included a larger reach of the river and was used to observe hydraulic, sediment, and debris conditions, including flow distribution and patterns around the intake, flushing channel, and ramp entrance and crest for higher flows starting around 1,500 ft^3/s. A 1:12 scale model with mobile bed sections near the entrance and crest of the ramp focused on hydraulic conditions in the ramp and intake for flows between 250 ft^3/s and 6,000 ft^3/s.

Baseline condition testing was completed to compare the hydraulic, fish passage and sediment performance of the MOD-6 and MOD-9 configurations. This was followed by testing of variations to the design to improve performance through the design development process. NHC worked directly with the USBR and UWCD throughout this process providing designs to USBR for testing as part of design development. The design development work included multiple iterations of the low flow channel rock placement, de-sander internal sluicing system, flushing channel configurations and other river training works. The USBR documented the findings of the model testing (USBR, 2022) and subsequent stress and operation testing (USBR, 2023). The designs developed from the physical model are presented in this report.

1.2 Purpose

The intent of this document is to provide all information necessary to describe the hydraulic performance of the Hardened Ramp alternative. The design was developed through utilization of the physical models as well as numerical analysis and engineering design. The key geometric and hydraulic features of the design are outlined in the document and preliminary hydraulic plans are appended to the report. This document builds off of the engineering design reporting provided in the Hardened Ramp 30% Design report (NHC and GEI, 2019), the 2020 Design Development Report (NHC, 2020), and Design Memorandum (NHC, 2021)(NHC, 2021).

The primary focus of the reporting is on design features that have advanced through physical modeling or were not described in detail previously. Information or analysis that was previously provided in other design documents is summarized. The primary areas of advancement through this report include:

- Low Flow Channel Design Modifications
- Sediment Management including the refinement of the flushing channel, training wall and implementation of a de-sander concept
- Debris Management and River Training
- Fish Screen and Bypass Design
- Gate Design.

1.3 Hardened Ramp Overview

The Hardened Ramp concept is designed to be built into the existing Vern Freeman diversion structure, replacing an approximate 100 ft section of the structure crest adjacent to the diversion intake and sluice channel with the Hardened Ramp structure (Figure 1-1).

The Hardened Ramp concept is a considerable departure from traditional dam-headpond-fishway structures built for the diversion of water from rivers. The free-flowing, open channel configuration of the Hardened Ramp is designed to function as a steepened channel segment in the river, and not pond or dam water. The 5% sloped ramp is designed to continuously convey water and sediment and provide upstream fish passage for Southern California Steelhead and Pacific Lamprey for river flows between 45 ft^3/s and 6,000 ft^3/s.

The Hardened Ramp provides both flow conveyance, sediment transport and fish passage through its unique asymmetric structure which includes a 30 ft rock-lined low flow section and a 60 ft baffled high flow section (Figure 1-2). At the crest of the ramp, the low flow channel invert for the MOD-9 configuration is at an elevation of 156.5 ft and increases moving right into the baffled section.

The baffled right section has a 6% transverse slope directed towards the triangular low-flow channel on the left side. The wide, asymmetric cross-section provides multiple pathways for upstream passage over a range of acceptable water depths and velocities that allow fish to select differing swimming speeds, positions in the water column and energy expenditures during upstream movement.

To provide a more natural substrate, additional energy dissipation and lower velocities near the bed, the bottom surface of the low-flow channel is a roughened rock ramp bed with 1 to 2 ft diameter stone and larger rock 1.7 ft to 3.5 ft in diameter distributed placed to provide acceptable fish passage hydraulics in the low flow channel from 45 ft^3/s to approximately 500 ft^3/s. As flows expand laterally into the baffled section of the ramp, the staggered baffles dissipate energy and provide both low velocity refugia as well as fish passage continuity through a significant fraction of the wetted area. The steel-plated baffles are 5 ft wide and 3 ft high with a 2-½ ft gap or slot between them.

The Hardened Ramp structure, the right abutment adjacent to the ramp and foundation system are protected by a sloped rock apron that extends in front of the ramp and right abutment down to an elevation of 154 ft.

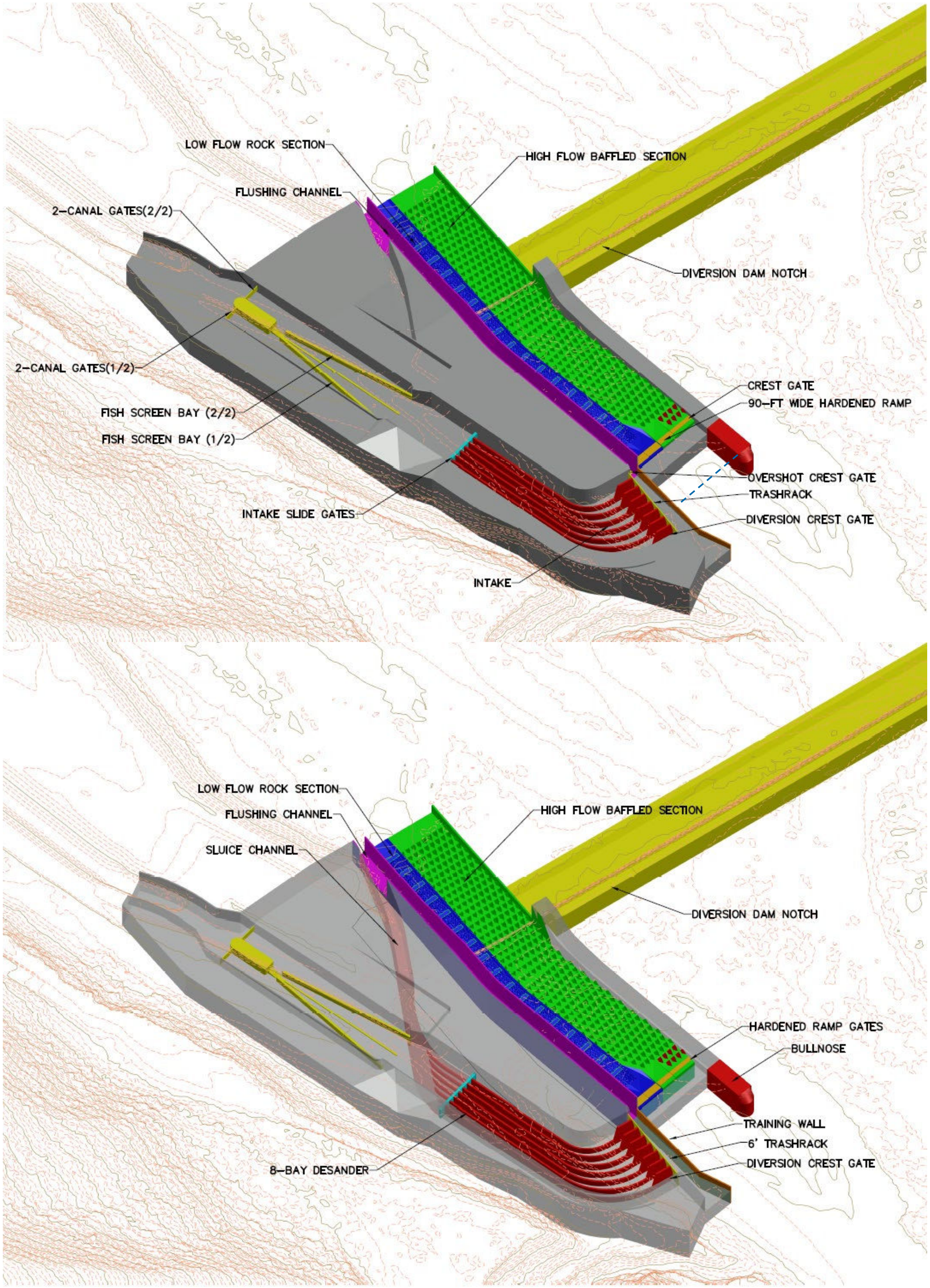


Figure 1-1 Inclined View of the Hardened Ramp System at Freeman Diversion. Translucid image on the bottom allows showing desander sluice channel under fish screen bays.

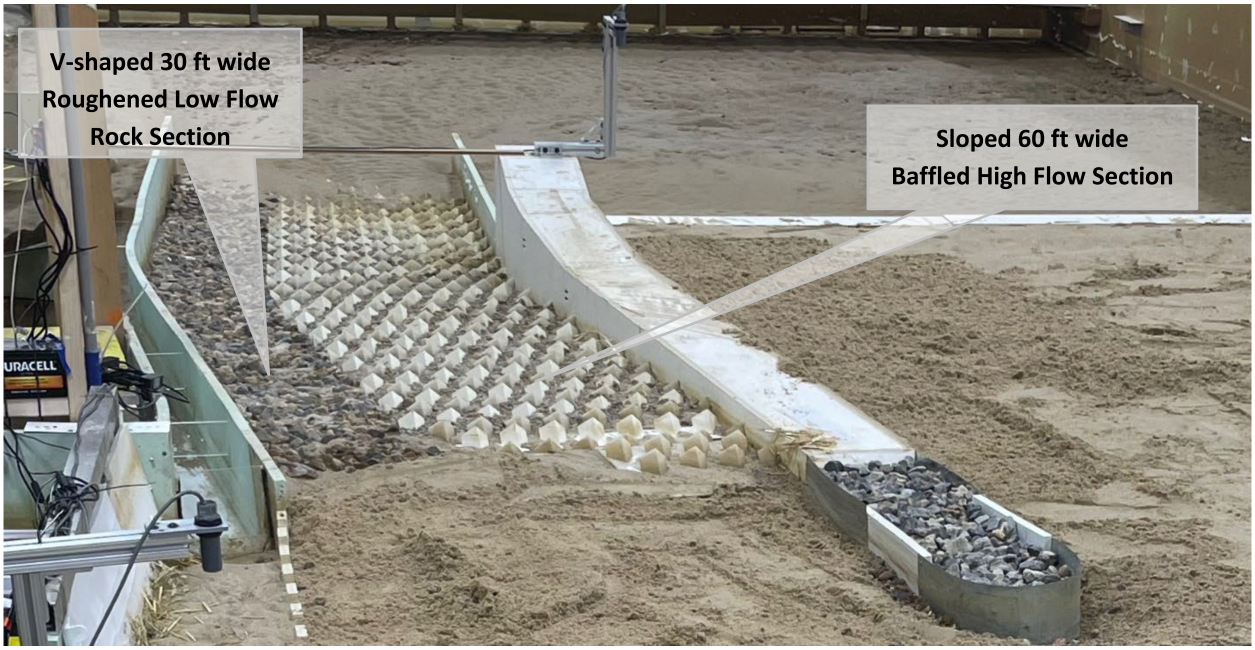


Figure 1-2 Asymmetric 90 ft wide Hardened Ramp Channel (photo of 1:24 scale physical model).

River flows are discharged downstream through the Hardened Ramp – the primary conveyance structure; or over the remaining diversion dam crest to the right of the ramp. The diversion intake structure controls the diversion and fish screening, and fish bypass operations and screened diversion flows are discharged down the canal system. A small amount of flow is returned to the river from the screen system to bypass fish from the screen bay back to the river. When needed, the flushing channel also provides downstream river discharge associated with sediment management. For a given upstream river discharge, the relative elevations and rated discharge capacities of the ramp, diversion dam crest, and diversion intake sill define the flow splits, or proportion of the total river flow conveyed by each structure.

During primary periods of upstream fish migration – currently defined as January 1st to May 31st annually – pneumatic overshot crest gates on the Hardened Ramp are fully open (down) across the width of the structure, providing conveyance of flows and sediment in the river with hydraulic connectivity for fish movement upstream though the low flow and baffled sections of the Hardened Ramp fishway. When the Hardened Ramp is operational, a portion of the total river flow less the ramp discharge is available for diversions – less fish bypass flows. As upstream flows increase the ramp continues to operate but a portion of the total river flow is spilled over the dam crest. Diversions may also be limited by UWCD water rights and permit conditions.

The diversion intake design utilizes eight 11 ft wide overshot crest gates on the intake sill to control the flows into the diversion intake. The four upstream gates are set at an invert elevation of 156.5 ft and the four downstream to an elevation of 155 ft. When operating, the overshot crest gates would increase the physical elevation offset to the ramp invert and help limit the mass and size of sediments entrained into the diversion intake. Downstream of the intake crest gates, both flows and water surface elevations are controlled by a system of undershot regulating slide gates downstream of the fish screens. An additional set of undershot intake slide gates, located downstream of the proposed diversion intake crest gates, would be utilized for the desander operation and to shut off the intake flows and isolate the structure.

A trash rack that extends to the design water elevation of 183 ft is in front of the intake gates with a 6-inch clear bar spacing to reduce the debris entrained into the intake. A 15 ft wide flushing channel is adjacent and parallel to the hardened ramp with an upstream invert elevation of 146 ft and a 2.5% slope. The flushing channel, desander sluicing channel and hardened ramp all terminate downstream at an elevation of 134 ft. The flushing channel is regulated by a 20 ft high overshot crest gate. The overshot crest gate was chosen for the ability to regulate ponded water levels upstream and to reduce the stress on fish. A sloping apron is in front of the intake crest gates that varies from an invert of 146 ft at the flushing gate entrance to 154 ft at the upstream extent of the training wall. This provides additional sediment storage and physical barrier to bedload for the diversion intake.

A 150 ft long training wall bound the sloping apron in front of the intake to concentrate flow and improve sediment transport when the flushing channel is opened. The training wall confines flows on the apron, improving the performance of the flushing channel and limiting sedimentation from entering the diversion intake. It creates a converging channel approximately 40 ft wide upstream, that narrows first to 25 ft at the beginning of the intake structure and finally to 15 ft at the entrance to the flushing channel. The top of the training wall has a “castle wall” type configuration with top elevation 162.0 ft and 5 ft wide by 2 ft deep notches, except for one deeper 5.5 ft notch located downstream to provide hydraulic connectivity with the hardened ramp.

A desander structure is located downstream of the intake crest gates to prevent sediment coarser than 0.5 mm from reaching and potentially settling on the fish screen bays. The desander traps bedload and suspended sediment that made it over the intake crest gates. Trapped sediment is periodically sluiced back to the downstream river using the hydraulic energy available, without the need for mechanical dredging or excavation. The desander is made of eight 5.75 ft wide parallel channels or bays that can operate independently. When one bay is full of sediment, it can be sluiced while diversion continues through the remaining 7 bays. Sediment sluiced from the desander bays passes through a culvert section underneath the diversion canal upstream of the fish screens. When passing through the culvert section, the 8 bays converge and discharge into a single 15 ft wide sluicing channel that merges with the downstream end of the flushing channel.

Coordinated control of both the regulating canal slide gates and the intake crest gates is required to control both the diversion flow, and water surface elevation and velocities in the screen bay. Flows diverted into the intake include both the canal diversion flows and the fish bypass flows. The dual bay fish screen system is sized to deliver 750 ft^3/s to the existing canal while meeting NMFS (2022a) and CDFW screen system design guidelines. Fish bypass flows are diverted to a fish monitoring station, and fish and bypass flows are released into the tailwater downstream of the Hardened Ramp.

2 Design Objectives

This section discusses the objectives for the design under the three primary areas of: Fish Passage, Diversion Yield and Sediment Management. These objectives were used when developing the project features and utilized for determining the success of the design.

2.1 Fish Passage Objectives

2.1.1 Amicus Brief Principles

The following principles or targets extracted from the NMFS Amicus Brief (2018) provide context and targets for development of the fish passage design. As discussed in the 20 February 2019 meeting with NMFS, some of the principles (such as maintenance activities and transit over or through partially open gates) may require further discussion or interpretation as the design is developed to follow the intent of the statements while addressing design challenges posed for the coordinated operation of the diversion and fish passage facilities.

> *"Improve steelhead passage opportunity both spatially and temporally for all flows between 45 and 6,000* ft^3/s *(by analyzing flow fields);*
>
> *The limited steelhead passage opportunities in an undisturbed system should not be further limited by facilities operations for sediment management or other periodic maintenance, for all steelhead passage flows (by facility design to sustain passage while conducting operations for sediment management and maintenance);*
>
> *Create upstream and downstream passage in the form of ramps (durable ramps at 5% or less that provide appropriate hydraulic conditions and designed in conjunction with headworks and screening facilities to provide upstream and downstream passage for life stages and prevent passage over the dam crest or under sluice gates for all steelhead passage flows);*
>
> *Preclude nuisance attraction flows (by conveying all discharges less than 1,200* ft^3/s *within the ramp);*
>
> *Steelhead shall not be challenged by or required transit partially open gates or weirs; and*
>
> *Install fish screens that protect all life stages of steelhead from impingement or entrainment (by installing screens that meet NMFS criteria and work in conjunction with headworks and ramps for all proposed diversion rates)."*

2.1.2 Design Basis Guidelines and Criteria

General design guidelines for roughened channels are included in CDFW (2009) and NOAA (2011), but specific hydraulic design criteria are limited. USBR (2007) also has published design guidelines for rock ramps, but the USBR guidance refers generally to other agency standards for use of swimming speeds as design parameters.

The proposed ramp length and drop height exceed guidelines for typical roughened channels from CDFW and NOAA. Other hydraulic design criteria like Energy Dissipation Factor (EDF) are difficult to apply due to lack of fish passage experience in similar structures. Although the Hardened Ramp alternative is favorably viewed by CDFW and NMFS, there is not a body of practical experience with such designs from which to draw empirical criteria or design parameters.

Allowable velocities (as a function of passage length) and depths have been published by CDFW (2002) and NOAA (2001, 2011, 2019) for road crossing design that provide some design basis and parameters. NMFS (2022a) provides the most recent guidance on criteria for fish passage, fish screening and bypass criteria, which form the primary guidance in the hydraulic design.

The proposed ramp length and drop height exceed guidelines for typical roughened channels from CDFW and NOAA. Other hydraulic design criteria like Energy Dissipation Factor (EDF) are difficult to apply due to lack of fish passage experience in similar structures. Although the Hardened Ramp alternative is favorably viewed by CDFW and NMFS, there is not a body of practical experience with such designs from which to draw empirical criteria or design parameters.

Table 2-1 summarizes design objectives and ecohydraulic criteria used in developing the design, recognizing that references to existing guidelines should be taken as targets or objectives rather than fixed criteria.

Table 2-1 Hardened Ramp Target Fish Passage Criteria.

Parameter	Objective/Target	Reference
Fish passage design flow range	45 to 6,000 ft^3/s	NFMS letter
Swimming Speeds	Steelhead trout: Cruising speed: 3.5 ft/s Sustained speed: 6 ft/s Darting speed: 10 ft/s Pacific lamprey: Cruising speed: < 1 ft/s Sustained speed: 2 to 3 ft/s Darting speed: 8 to 10 ft/s	Bell, 1991; Mesa, 2003; Keefer 2010
Ramp drop height	22.5 feet	Topographic mapping of the riverbed, dam crest
Operating water surface elevations (diversion)	158.5 to 164.0 (VFD datum, narrower range may be selected)	Existing dam crest and canal elevations

Parameter	Objective/Target	Reference
Fishway entrance conditions	Discernible as a higher discharge density from adjacent flow field Maximum head drop 1.5 feet	NOAA, 2011; Project team
Fishway exit conditions	Ambient velocity: < 4 ft/s Maximum head drop: 1 foot	NOAA, 2011
Minimum depth	Steelhead: 1 foot Lamprey: 1 inch	NOAA, 2011 AECOM, 2016
Maximum design passage velocity	Steelhead: 8 ft/s Lamprey: 2 ft/s (swimming, may be on margin, higher locally with suitable attachment)	NOAA, 2011; AECOM, 2016
Desirable velocity in passage pathways	Steelhead: 2 to 3 ft/s Lamprey: < 2 ft/s	CDFW, 2002; NOAA, 2011; project team
Energy dissipation factor (EDF)	No specific numeric criterion, but desirable maximum EDF of 7 to 8 ft-lb/ft^3/s are related to desirable velocities in passage pathways. EDF of 8 ft-lb/ft^3/s is approximately equivalent to a velocity of 2.6 ft/s on a 5% ramp	NOAA, 2011, CDFW, 2009

Due to the unique hydraulics of the ramp design and interaction of the hydraulic structures, a simple set of fixed criteria was not feasible to apply as in design of a more conventional structural fishway. As such, the design and analysis of the Hardened Ramp involves the modeling and review of complex hydraulic conditions over the applicable range of fish passage flows to determine whether the fundamental objective of providing hydraulic continuity; acceptable water velocities; adequate depths of flow; adequate stationing and resting areas; and multiple passage pathways are being met.

In addition to the primary fish passage objectives relating to anadromous species in Table 2-1, passage requirements for native fish species, are accommodated in the design through the criteria presented in Table 2-2.

Table 2-2 Hardened Ramp Native Fish Species Passage Criteria.

Parameter	Criteria	Comments
Native Fish Species	Arroyo chub Santa Ana sucker Partially-armored three-spine stickleback Rainbow trout: juvenile and adult	CDFW, 2020
Passage Season	June 1 through December 31	
Minimum Fish Passage Flow	4.0 ft^3/s	94% exceedance flow
Maximum Fish Passage Flow	270 ft^3/s	5% exceedance flow
Maximum Swimming Speed - Arroyo chub	2.6 ft/s	CDFW, 2021
Maximum Swimming Speed - Santa Ana sucker	3.0 ft/s	CDFW, 2021
Maximum Swimming Speed - Partially-armored threespine stickleback	1.5 ft/s	CDFW, 2021
Maximum Swimming Speed - Rainbow trout adult Rainbow trout Juvenile	 4.0 ft/s 2.0 ft/s	Estimated from values given for Sockeye and coho from 2 inches to 5 inches in length in Bell (1991)
Minimum Water Depth	2 x body depth	Powers and Orsborn, 1985, Webb, 1977
Resident Fish Passage Maximum Slope of Channel	5.0 percent	USBR, 2007

2.1.3 Volitional Fish Passage

2.1.3.1 Upstream Fish Passage

The Hardened Ramp is intended to provide multiple passage pathways for steelhead and Pacific lamprey at river flows up to 6,000 ft^3/s. The Hardened Ramp is intended to operate continuously during steelhead migration periods when steelhead are expected to be migrating, without shutdown of fish passage for operations such as sediment flushing. A portion of the high fish passage design flow will be conveyed in the ramp and a portion will be diverted to the canal system, spill at the dam crest or pass through other gates to assist with sediment management. A key design consideration for the Hardened Ramp is the ability to attract fish to enter the downstream end of the ramp. For the purposes of design development, NHC initially assumed that at least 1,200 ft^3/s should pass through the ramp at the high fish passage design flow and that higher flow rates would be desirable. The objective from the Amicus Brief of passing the first 1,200 ft^3/s in the ramp has been incorporated into the current work.

A design objective for the Hardened Ramp cross section is to provide multiple pathways for upstream passage and a range of potentially acceptable depth and velocity conditions that allow fish to select differing swimming speeds and energy expenditure during upstream movement. The Hardened Ramp uses an asymmetric sloped cross section, as recommended in the Hardened Ramp 30% Design Report (NHC and GEI, 2019). The design section provides multiple opportunities longitudinal and connectivity for passage for anadromous species at higher migratory flows where water depths are greater than 1 ft and less than 8 ft/s. Upstream passage for native fish is focused on lower flow regimes where discharge is wholly contained within the hardened ramp and occupies the low flow channel or shallow inundation of the baffled section. In general, these regions of possible passage are less than a 1 ft of depth and less than 1 to 2 ft/s.

Opportunities were developed to ensure fish passage was non-selective and varied. In the Hardened Ramp, the bed of the ramp is covered with a roughened rock surface which extends from the low flow channel through the baffled section. The relative roughness provides variation in hydraulics at shallow depths of flow to facilitate passage of smaller fish. The presence of larger regularly spaced boulders in and baffles provides hydraulic shadows of lower velocity for holding and staging. Fish can swim in a sustained mode through the structure or can move from structure to structure in a burst-and-rest mode.

2.1.3.2 Downstream Fish Passage

The unimpeded downstream movement of steelhead smolts, juvenile and adult fish is a key design objective. Because flows may be split between the diversion, Hardened Ramp, dam crest, and other gates, multiple pathways for downstream passage may occur. Differences in unit discharge, orientation of the structures, relative attractiveness of hydraulic conditions, and visual cues will influence the distribution and behavior of downstream migrants, but as the design operated as a flow-through structure with no head pond or storage, potential delay is minimized. The Hardened Ramp is the preferred route for downstream passage under low to moderate flow conditions as it preferentially discharges the initial 1,200 ft^3/s. As noted above, the relative concentration of fish to this route depends on several hydraulic and behavioral factors. The ramp is expected to provide suitable hydraulic conditions for downstream passage, although velocities may be quite high, and flow will be turbulent.

There is potential for juveniles to pass over the crest of the dam during periods when river flows exceed the combined flows in the diversion, Hardened Ramp, and any sluicing or auxiliary flow features. UWCD plans to resurface the downstream face of the dam and this will reduce the roughness of this surface, which presently has an irregular stepped profile associated with the roller compacted concrete (RCC) construction method and subsequent wear. Fish that pass over dam crest will drop into the tailwater below the dam. Injury associated with a fall of this height may be low but needs to be considered in the final design. Control of water surfaces upstream of the dam to limit the spill depth may provide behavioral avoidance of this route during operation of the ramp and diversion. During higher flows, spill depth of flow and tailwater elevation will be higher, reducing potential for abrasion and injury related to passage.

2.1.3.3 Fish Screening, Bypass and Release

The location and orientation of the intake can provide hydraulic and behavior cues that are likely to reduce entrainment when both the intake and Hardened Ramp are operational. Flow splits between the Hardened Ramp and intake are being developed as part of the Multi-Species Habitat Conservation Plan (MSHCP) process, and are not yet available. The focus in current design development efforts is therefore to provide operational flexibility to manage flow splits and, to the extent feasible, provide a direct and more open flow path towards the Hardened Ramp than the diversion intake. However, some proportion of downstream migrating fish will enter the diversion intake. As noted, increasing the capacity of the diversion to 750 ft^3/s implies that larger intake and fish screens are required. For fish that enter the diversion intake, new fish screens designed to meet NMFS (2022a) standards will be used to prevent entrainment in the canal, and screened fish will be bypassed and released back to the river. The location of the release will be into the tailwater of the facility which minimizes predation and complies with current guidance.

2.2 Diversion Yield Objectives

UWCD plans to request an increase in diversion capacity for the facility to 750 ft^3/s (existing capacity is 375 ft^3/s) and that the facility should be designed to operate using the existing gravity canal system downstream, considering planned system improvements (NHC, 2016c). The increased diversion capacity allows flow to be diverted more rapidly during high river discharges. The 750 ft^3/s capacity was used for the design basis and analysis of the structure. Future operations and flow splits are still being developed through the MSHCP. The design intent is for the project to not be the limiting factor in future flow splits.

2.3 Sediment Objectives

To guide the design process, NMFS/CDFW and UWCD agreed on the nine sediment management objectives detailed in Table 2-2. The UWCD team provided an initial list of sediment management objectives that were then expanded upon and revised by the agencies to provide input from a fish passage perspective. During the design process, the sediment management facilities were modified as much as possible to minimize effects perceived as potentially adverse to fish passage.

Table 2-3 Sediment objectives.

Issue	Sediment objective
1	Maintain the low-flow river channel near the left bank to enhance sediment transport through, and downstream of, the fish passage alternative.
2	Ensure the fish passage entrance(s) and exit(s) remain relatively free of sediment and debris such that fish have the ability to enter, ascend, and exit the hardened ramp. For the ramp, a migratory path of sufficient depth and velocity should be maintained throughout the range of fish passage flows.

Issue	Sediment objective
3	Design development should evaluate the sediment management capabilities of each design and estimate what is required to maintain fish passage for both alternatives. Utilizing information gathered from physical modeling, operations incorporated into and analyzed in the MSHCP should avoid or minimize sediment and/or debris removal from the fishway where surface flow is present unless sediment/debris is preventing fish passage. The physical models should be used to best estimate the frequency and extent of sediment/debris maintenance activities, and to inform the time required to implement sediment removal activities.
4	Minimize hydraulic recirculation zones that lead to sediment deposition upstream of the diversion intake and in the vicinity of the apron directly in front of the intake, which is also the fish passage exit area. Promote accelerating flow conditions or induced secondary currents that do not impede (agency preference) or interrupt (UWCD preference) fish passage while still enhancing sediment transport through and downstream of the dam. Design development should evaluate elements that enhance sediment transport continuity capabilities of fish passage designs.
5	Conduct sediment management of the diversion intake apron and/or the fish passage exit, in a manner that minimizes both delay and take of steelhead by not interrupting fish-passage and minimizing the risk of stranding and injury to fish during sediment maintenance operations. This includes but is not limited to the following: not causing unfavorable fishway entrance, transit route, or exit conditions (hydraulic and water quality); not causing nuisance attraction flows; and avoiding entrainment of adult or juvenile steelhead into non-fish passage flow routes (e.g., flushing channel). Utilizing information from physical modeling, operations incorporated into and analyzed in the MSHCP should avoid or minimize sediment management activities that impede (agency preference) or interrupt (UWCD preference) fish passage.
6	Within the diversion intake, develop a secondary settling area and non-mechanical system to flush sediment in lieu of mechanical cleaning. Some sediment accumulation within the intake channel is inevitable but to a degree that mechanical dredging may be efficient when performed outside the range of steelhead-passage design flows (45 to 6,000 ft^3/s).
7	The fish screens should be kept clean and functional. Capability to hydraulically sluice the fish screen channels in lieu of mechanical cleaning should be considered. Sluicing options would need to protect fish by meeting guidelines for fish bypasses.
8	Mechanical removal of sediment, when necessary, should be limited to periods outside the fish passage window and should minimize impacts to aquatic species.
9	If some manual sediment removal will be required via suction dredging or excavation from within the diversion intake, de-sander, near fish screens, and in the diversion conveyance canal during the fish passage season, then impacts to fish during these activities should be avoided or minimized.

3 Physical Modeling Design Development Overview

Physical model testing was a key design tool implemented to advance the Hardened Ramp concept during this phase of the design development. The US Bureau of Reclamation (USBR) built and tested the hardened ramp concept utilizing 1:12 and 1:24 scale physical models built at their facility in Denver, CO. The USBR completed documentation of the Baseline and Design Development Tests (USBR, 2022) as well as stress and operational testing (USBR, 2023). A full description of the methods and results can be found in these reports. The baseline testing took place from November 2021 to May 2022.

Design Development commenced afterwards through October 2022, followed by Operational and Stress Testing through the end of December 2022. The results from the physical modeling design development work will be summarized and referenced throughout this document as individual design considerations are discussed.

3.1 Baseline Testing

Baseline physical model testing was used to better understand the performance of the ramp and intake system and to compare variations of the initial MOD-6 and MOD-9 design geometries (with flushing channel open, closed and removed) through a range of flow conditions. The tests provided data that was used to identify areas for improvement and refinement in the subsequent design development phase. Below is a brief recap of the baseline testing and key findings for design development.

Table 3-1 USBR Baseline Testing.

Model	Test Plan	Key Findings
1:24 Baseline Testing	- Testing for a range of higher flows (3,000 ft^3/s, 6,000 ft^3/s, 12,000 ft^3/s and 30,000 ft^3/s) - MOD-6 and MOD-9 with flushing channel open, closed and removed	- A large sediment bedform consistently developed in front of the intake for all configurations leading to large amounts of sediment being ingested into the diversion - The sediment deposition could not be removed from the initial configurations of the hardened ramp - There was considerable deposition into the intake - The upstream approach flow hydraulics caused undesirable conditions across the face of the ramp
1:12 Baseline Testing	- Testing for a range of lower flows (270 ft^3/s, 1,500 ft^3/s, 3,000 ft^3/s and 6,000 ft^3/s - MOD-6 and MOD-9 with flushing channel open and closed	- Since 1:24 baseline testing showed a consistent bedform for all configurations, flushing channel removed scenario was not included to expedite the modeling schedule and to begin design development tests - The MOD-6 Geometry was not able to meet the desired flow splits/yield for 270 ft^3/s and 1,500 ft^3/s - The hardened ramp hydraulics generally looked good for fish passage, but the low flow channel hydraulics could be improved with more strategic placement of larger rock - The hydraulics at the upstream end of the ramp could be improved

3.2 Key Areas of Design Development

Design development was an iterative process using the laboratory models to test concepts and refine the design. Through multiple visits to USBR's lab in Denver, data and observations from the physical model were collected, and these findings were discussed with the design team to identify areas for improvement. NHC then performed engineering analysis to develop new concepts for physical model testing. Where practical, numerical models were also used to do preliminary testing of concepts and to supplement laboratory data.

The design development process undertook a systematic collaborative approach utilizing the expertise of the UWCD Team and the Agencies to improve the design. Design development addressed deficiencies from the baseline testing systematically with modifications developed through engineering analysis (e.g., calculations, numerical modeling, etc.) that were tested in the laboratory and demonstrated during witness test visits. In addition, some design aspects of the facility that had not been previously addressed were evaluated in more detail.

Primary issues identified in the baseline testing can be categorized as:

- Approach Flow Hydraulics
- Hardened Ramp Low Flow Channel Hydraulics
- Ingestion of Sediment into the Intake
- Sediment Bedform in front of the Intake.

Each issue is briefly discussed below and covered in more detail in subsequent sections of the report. A timeline of NHC's design development work on the major issues identified above is given in Appendix B.

3.2.1 Approach Flow and River Hydraulics

From the baseline testing it was observed that the alignment of the river thalweg (inset channel) upstream of the ramp had effects on hydraulics and sedimentation, as it impacted the flow distribution into the intake and the hydraulics just upstream of the ramp. Work in design development was focused on mitigating adverse hydraulic conditions at the ramp and diversion intake, as well as investigating measures to maintain or train the inset channel to the left bank.

Approach channel conditions looked at river right and central approach channel conditions. Physical modeling and testing found that extension of the bullnose wall from the right (looking downstream) upstream corner of the ramp tended to "capture" a river right alignment with the thalweg connecting to the Hardened Ramp inlet. Installing a training wall near the intake improved entrance conditions to the ramp and intake when the river channel was centrally located.

In addition to these changes, flooding characteristics were re-examined to set facility wall heights, tailwater conditions were investigated, and erosion and scour protection designs were developed. These subjects are described in Section 4.

3.2.2 Hardened Ramp Low Flow Channel Hydraulics

The baseline testing identified improvements that could be made in the low flow channel portion of the ramp and apron that is utilized for fish passage primarily at discharges under 500 ft^3/s. The arrangement and range of rock sizes in the ramp were updated which provided more favorable fish passage hydraulic conditions through a range of flows. Extension of roughness elements upstream of the ramp improved entrance/exit conditions for passage.

The changes to ramp low flow channel and inlet section hydraulics, as well as an overall description of ramp characteristics and physical modeling results is provided in Section 5.

3.2.3 Ingestion of Sediment into the Intake

For all model geometries tested a large amount of sediment was ingested into the intake. Design development of a solution to reduce or remove sedimentation was a primary focus of the physical modeling.

A desander concept was developed and was shown to work as an internal sluice to remove sediment that deposits within the intake. The effectiveness depends primarily on the entrance sediment conditions, the downstream tailwater level, and the discharge used. The concept works best in conjunction with an external system, such as the flushing channel that reduces the sediment load reaching the desander by removing sediment deposited on the apron in front of the intake. Without flushing the apron, attempting to sluice the desander by completely lowering the intake crest gate (removing a physical sediment barrier) results in very inefficient sluicing operations (i.e., very prolonged duration using high water discharges) because the desander becomes overwhelmed by sediment coming from the intake apron. The intake configuration, use of a training wall to improve efficiency, design of the intake channel to improve efficiency and accommodate the desander concept, and design of gates and trash rack are described in Section 6.

In addition, the fish screen and bypass system designs were advanced in this phase and are described in this section. Although not strictly related to the intake, this section also includes a description of the design for downstream passage during periods when flow spills over the diversion structure crest.

3.2.4 Sediment Bedform in Front of Intake

A persistent bedform (sand bar) was observed in front of the intake for all baseline conditions. Opening the flushing channel was not effective at removing the bedform. This prograding bedform regrades the channel upstream of the ramp and leads to sedimentation against and into the intake. Solutions to remove or manage this bedform were investigated including the addition of a training wall connected to the flushing channel, a concept which is currently used by UWCD to manage sediment deposition in front of their existing intake.

The flushing channel with a training wall was effective at clearing out the sediment bedform in front of the intake. Testing of the no flushing channel concept did not remove the sediment bedform. The combination of lowered flushing channel invert elevation, sloping apron and training wall allowed for shorter duration flushes with greater extent and volume of sediment removed. The castling of the training wall also provided benefits for river training and flow equalization.

Characterization of sediment conditions, and design and operation of sediment management facilities are described in Section 7.

4 Approach Flow and River Hydraulics

4.1 River Morphology

Figure 4-1 presents an aerial view of the Santa Clara River upstream of the Freeman Diversion. Upstream of the diversion dam, the Lower Santa Clara River is about 1000 ft across and is contained by high ground to the south (left bank) and a levee to the north (right bank). Within the river cross section, an active channel zone exists that is approximately 200 ft across with a meandering inset channel. This active channel zone is currently located along the left bank of the river cross section where it has been stable since the late 1950s or early 1960s. However, the alignment and stability of the active channel is affected by flooding and sediment pulses that can reset the morphology of the channel (Hydroscience & Engineering, 2021).

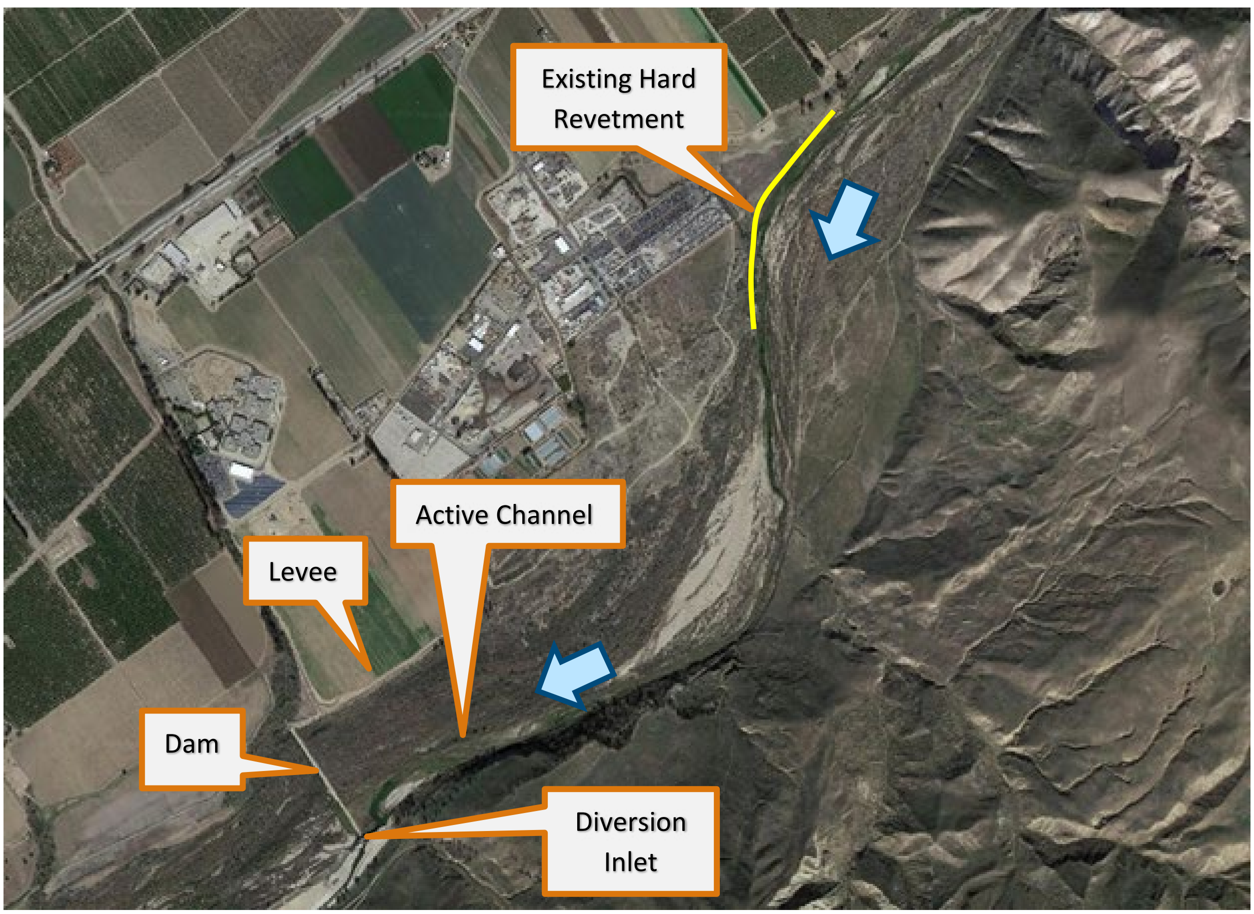


Figure 4-1 Aerial view of the Lower Santa Clara River reach upstream of Freeman Diversion.

4.1.1 Channel and Approach Flow Description

Figure 4-1 shows the locations of two 90-degree bends with radii of about 1,500 ft. Approximately 2,300 ft of hard revetment exists on the right bank of the first bend, although the blanket thickness and toe depth of the rock is unknown. This revetment works to guide the flow down to the second bend that follows the foot of the hills to the south. Here the river is held by eroded bedrock on left bank, where it has remained relatively stable. The reach then straightens as it approaches the dam. The current location of the inset channel pulls away from the left bank slightly just upstream of the diversion inlet. The channel bed in this location is influenced by sediment input from a steep tributary off South Mountain.

Figure 4-1 also shows a large point bar that has formed on the inside of the second bend and the heavy vegetation that has become established on the center and right bank of the straight reach near the dam.

4.1.2 Sediment, Bed Material and Riparian Vegetation

The Lower Santa Clara River has an estimated total sediment yield of 1,800,000 tons per year, including 620,000 tons per year of coarse bed load (NHC, 2017). Although it is recognized that most of the sediment load occurs during large floods, significant transport of sand bedload occurs during lower flows. The bed material size is variable, and consists primarily of sands, gravels, and cobbles on the bars and inactive sections of the channel, and sands in the active channel. Local vegetation is a mixture of riparian tree species (Fremont cottonwood, California black walnut), riparian scrub (Mulefat, Arroyo willow, Red willow, Sandbar willow), and stands of Arundo.

Figure 4-2 presents a view of the active river channel with vegetation just upstream of the Freeman Diversion intake.

Figure 4-2 View of Active Channel at Freeman Diversion Inlet (NHC 2021).

The alluvial bed of the Santa Clara River around Freeman Diversion is highly dynamic, responding to seasonal changes and those caused by droughts, floods, upstream landslides, vegetation growth and forest fires. Figure 4-3 shows example conditions of the river channel downstream of the dam. In 2008, the channel was mostly free of large vegetation, while by 2021 dense vegetation growth had encroached into the channel. Vegetation increases the channel roughness, increasing flow depth and reducing flow velocity, which in turn can promote sedimentation and bed aggradation. Large floods that remove vegetation from the channel could cause the opposite effect and lead to channel degradation.

Figure 4-3 Riverbed conditions around Freeman Diversion in 2008 and 2021 (from UWCD).

These dynamic changes in the riverbed translate into changes to tailwater levels downstream of the dam, which has been observed to vary greatly (Figure 9-1).

4.1.3 Geomorphic Context

Previous studies have documented both local and river-scale geomorphic conditions and processes in considerable detail (Stillwater Sciences, 2011, 2013; AECOM, 2014; NHC, 2016b, 2016a, 2017). Relevant geomorphic controls are summarized below:

- Bed material is highly varied, ranging from areas with nearly complete sand to areas with cobble-dominated bed material (mean D_{50} and D_{84} of 17±23 and 61±55 mm, respectively values ±1σ) and isolated boulders.
- The river has a characteristic slope of about 0.2%.

- Although the river channel and dam are approximately 1,000 ft wide, the entire channel width is active only during very large floods. Most flows are conveyed along a narrower (30 to 100 ft wide) inset “low flow channel”.
- Large scale channel morphology is set by uncommon (20-yr or higher recurrence interval) flood flows, and a smaller inset channel is adjusted to convey frequent (2- to 5-yr recurrence interval) floods.
- The river transports a considerable amount of fine (sand-small gravel) sized sediment during frequent flood flows.
- Deep-rooted riparian vegetation and local bedrock outcrops provide very high bank strength.
- The right bank revetment approximately 1.32 miles upstream of the dam and bedrock along the left bank just upstream of the dam act together to reinforce a position of the low flow channel against the left bank (south) valley wall approaching the dam. This situation is not expected to change until there is a substantial change in the upstream approach meander geometry.

Large scale river impacts were qualitatively assessed in the preliminary geomorphic assessment performed by NHC (2017); while riverbed changes near the ramp were assessed by physical modelling (USBR, 2022). The preliminary geomorphic assessment (NHC, 2017) suggested that:

- The hardened ramp design will modify geomorphic controls by lowering the elevation of the base level control at the dam and concentrating discharge at the location of the ramp.
- Upstream channel lowering increases flow concentration in ramp, generating very strong attraction flows. Under low-flood conditions, the bed should evolve in a way that reinforces the ramp being the dominant flow path. It appears the ramp should persist as a strong channel migration attractor unless the upstream channel configuration changes markedly.
- There were no indications that the ramp design will cause a major destabilization of the overall channel pattern and form. Vegetation will be able to follow the lower water surface, so bank strength will be maintained. The ramp is also located along the left bank bedrock that controls the low-flow channel position and will tend to reinforce this influence. Therefore, there is no reason to expect major disruption to the existing channel planform geometry.
- The low flow water surface upstream of the ramp will be lower with the ramp in place, and sandy soils likely create high hydraulic connectivity between groundwater that supports the right bank wetland upstream of the dam and the water surface in the channel. Therefore, some impact to the hydroperiod of the wetland may be expected.

4.1.4 Geotechnical Assessment

An assessment of geologic and geotechnical hazards was undertaken by NHC’s subconsultant, GEI Consultants Inc. The GEI (2020) assessment was undertaken specifically to better understand the conditions in and around the intake that would fundamentally affect the design, including the impacts of the unnamed drainage immediately upstream on the left bank. The finding of the report concluded that the proposed location footprint of the intake structure was feasible, but consideration should be made to several potential hazards as the design development continued. These potential hazards and issues included:

- The alignment of the structures crosses a major landslide complex that includes several active landslides that are likely to mobilize during the project service life,
- The fan terminus may be subject to future episodic debris flow events,
- Movement of the intake structure upstream would move it closer to a known alluvial fan hazard and increase the active sedimentation into the intake,
- Liquefaction risk of the unconsolidated sediments would require development of deep pile or RCC foundations to underlying bedrock, and
- Bedrock at the site is generally weak and relatively shallow, but suitable for foundation bearing.

4.2 Erosion Protection

Based on the recommendations of the geotechnical assessment (Section 4.1.4) erosion protection of the diversion and hardened ramp structures would be primarily achieved by placement of a thick RCC foundation directly on top of competent bedrock. However, some additional protection is required at the upstream and downstream bank transition zones and in areas of high velocity during floods, such as near the bullnose wall extension (Figure 4-14). Rock protection provides additional resistance to the erosive forces of the river since it is deformable and can absorb impact loads from high velocities and debris. It is assumed that the rock protection would be placed along with the RCC when the site is fully excavated and then covered with native material to the elevation of the natural channel bed.

At the upstream project transition, rock protection is shown on the plans provided in Appendix A along a 30-foot section of bank immediately upstream of the retaining wall. The top of the rock begins at elevation 178 feet and follows a 2:1 slope down to the approximate bedrock elevation of 130 feet (Figure 4-4). The upstream side of the rock protection would generally be buried below the elevation of the existing ground. On the downstream side, the elevation of the rock would stop at elevation 145 feet to match the inlet apron and guide the flow to the inlet when the riverbed is scoured. Channel velocities in the vicinity of the inlet apron are observed to be as high as 10 feet/sec during the 100-year event and would likely be highly turbulent.

Based on the U.S. Army Corps of Engineers (1994) approach for designing riprap, a rock size of D_{100} = 24 inches results using a safety factor of 1.5 and a C_v of 1.25. The rock size was checked using the approach recommended by the Federal Highway Administration (FWHA, 2009) in HEC-23 for flow near rounded piers, which resulted in a D_{100} = 30 inches. Selecting the more conservative value of D_{100} = 30 inches to account for turbulence, the design thickness of the rock protection would be 30 inches.

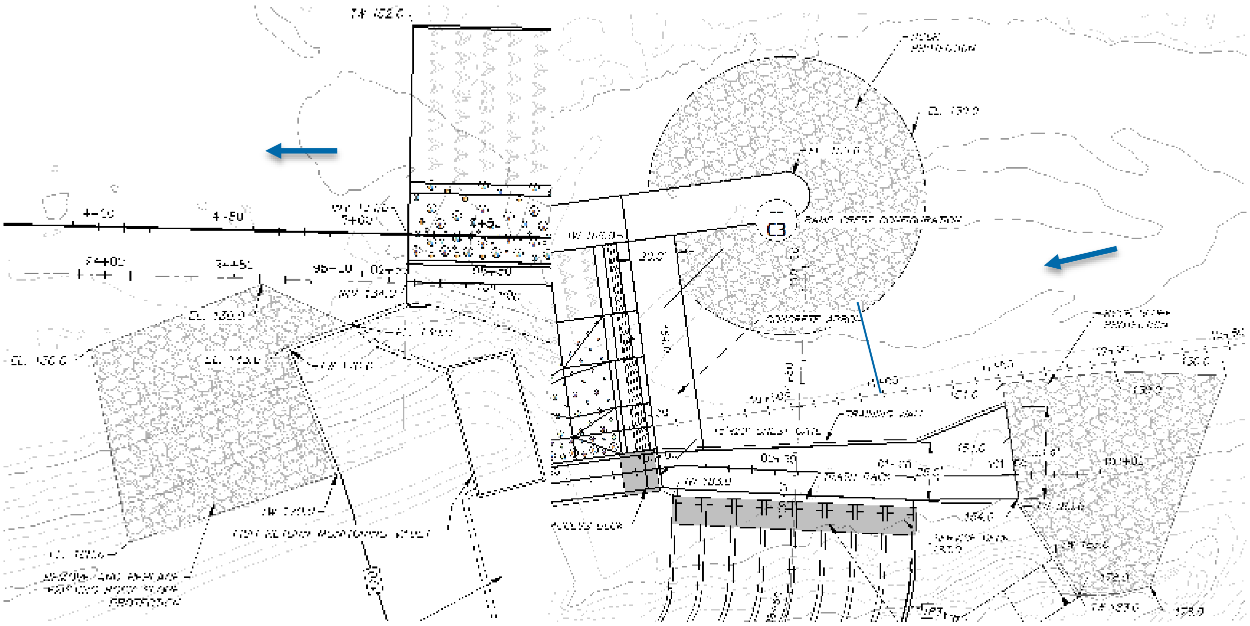


Figure 4-4 Downstream and upstream rock protection.

At the downstream project transition, rock is to be placed along the bank for 75 ft downstream of the retaining wall shown in the plans. The rock would begin at elevation 168 ft and extend down at a 2:1 slope to the approximate bedrock elevation of 130 ft (Figure 4-4). It is noted that the existing bank protection downstream of the flushing channel outlet has failed multiple times during flood events. Therefore, the retaining wall has been extended by 50 ft downstream of the proposed flushing channel outlet with the rock slope protection on the bank downstream.

Velocities just downstream of the flushing channel outlet can be as high as 15 ft/s based on HEC-RAS modeling. Using the U.S. Army Corps of Engineers (1994) rock sizing approach, this results in a D_{100} = 54 inches using a safety factor of 1.5 and C_v of 1.25. Checking with the FHWA (2009) approach for rock sizing near round piers results in D_{100} = 66 inches. Selecting a rock size of about D_{100} = 60 inches (approximately 4-ton rock) would result in a rock blanket thickness of 5 ft.

In addition to the upstream and downstream bank protection proposed, the plans include the placement of rock protection around the bullnose wall extension between the hardened ramp and the dam (Figure 4-4). It is noted that the physical model showed scour in this location during higher flow tests. Scour calculations suggest that the scour hole formed at the bullnose could extend down to the bedrock at around elevation 130 ft during the 100-year event. The formation of a large scour hole at this location would negatively affect the flow by creating additional turbulence, irregular flow patterns and energy losses.

For this reason, a cone of rock protection is to be placed around the bullnose with a top elevation of 155.0 ft, or one foot higher than the hardened ramp apron. The elevation of 155.0 ft at the crown was selected to be higher than the elevation of the neighboring apron but not so high as to create a large flow blockage when exposed by moderate scour. The rock would extend downward at a 2:1 slope to the elevation of local bedrock, or about 130.0 ft. Velocities upstream of the bullnose can reach 18 ft/s during the 100-year flood. Rock sizes require to resist hydraulic forces during such a flow would be too large to be reasonably place around the structure. Therefore, it is suggested that rock protection with D_{100} = 66 inches (approximately 5-ton rock) be installed to provide protection during moderate flood events, resulting in a rock blanket thickness of 5.5 ft.

The RCC that is to be placed as the foundation for the bullnose wall could be designed to include a sloping apron that extends into the channel to better protect the structure, and the rock protection could be placed over the sloping RCC. Although some rock in the protection may move during very infrequent flood events, most of the protection would remain and provide additional resistance to damage caused by high velocities and debris.

The inlet to the hardened ramp is another location that may require rock protection, though it is not yet indicated on the plans. Additional design of the RCC foundation is necessary to better understand whether a sloping extension of the concrete foundation upstream of the inlet might be sufficient to protect against scour and debris. If additional rock is used, the protection would match the elevation of the inlet at 154.0 ft and extend out to the bullnose on the north and along the training wall in the south.

4.3 Large Scale River Training Works

4.3.1 Active Channel Realignment

The realignment of the active channel away from the left bank would increase the distance between the inset channel and the diversion inlet, potentially complicating diversion access to low flows in the river. Within the period of record of aerial photos available (1927 to 2022), only four flood events have substantially shifted the active channel upstream of the Freeman Diversion Dam (NHC, 2017). However, since about 1960, the active channel zone has been held against the left bank, likely due to channel incision from changes in hydrology, gravel mining, and construction and maintenance of the first diversion inlet in the early 1990s (Hydroscience & Engineering, 2021).

4.3.1.1 Bendway Weirs

The use of rock training structures to discourage realignment of the active channel to the north was considered as a part of the study. A series of Bendway Weirs were assumed to be constructed in the current active channel to help create a deeper thalweg near the left bank and discourage migration. The Bendway Weir concept would require the construction of multiple structures that extend from the desired position of the channel thalweg across the right side of the active channel zone. The elevation of the weir crowns would be at about 161.5 ft NAVD88 at the furthest downstream weir and increase with the channel slope so that they are submerged during frequent floods.

The weirs would be constructed of large rock (FHWA Class 8 with D_{100} of about 5-ft) and pointed upstream at a 30 to 40-degree angle to encourage bending of the flow toward the left bank. The submerged weir structures would force the flow to slow down as it passes over the obstructions and encourage bed scour at the tips on near the left side of the active channel.

A conceptual layout of the Bendway Weir structures is presented in Figure 4-5. The figure shows an array of four weirs with lengths of about 150 ft in the low flow channel and a spacing to length ratio of about 2.5. In order to train the thalweg to remain along the left bank, the weirs would have to extend at least halfway into the active channel. It is noted that this is a not a standard application of Bendway Weirs, which are generally set on the outsides of bends with protrusion lengths between 15 to 30% of the bankfull channel width.

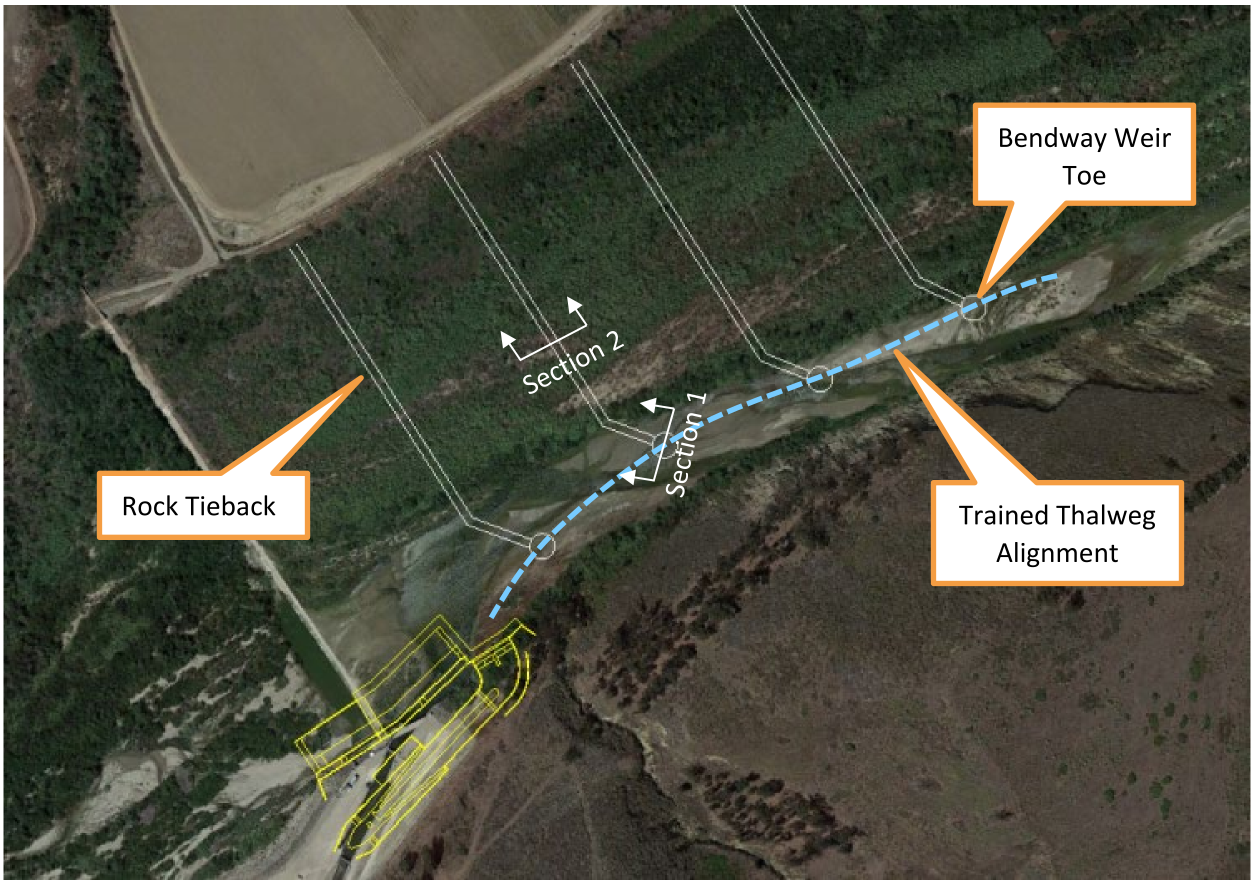


Figure 4-5 Conceptual layout of Bendway Weirs and rock tie-backs to hold channel along the left bank.

Figure 4-6 presents a cross section of the weir toe based on the concept design. The dimensions shown in the figure were developed using design guidance described by the USBR (2015). The weirs were also designed to contain sufficient volume of rock to launch down at a 2:1 slope to a scour depth of 10 ft below the ambient bed elevation (scour elevations for the 25-year flood were estimated to be 5 and 10 ft using the Lacey and Blench equations, respectively, for 1 mm sand).

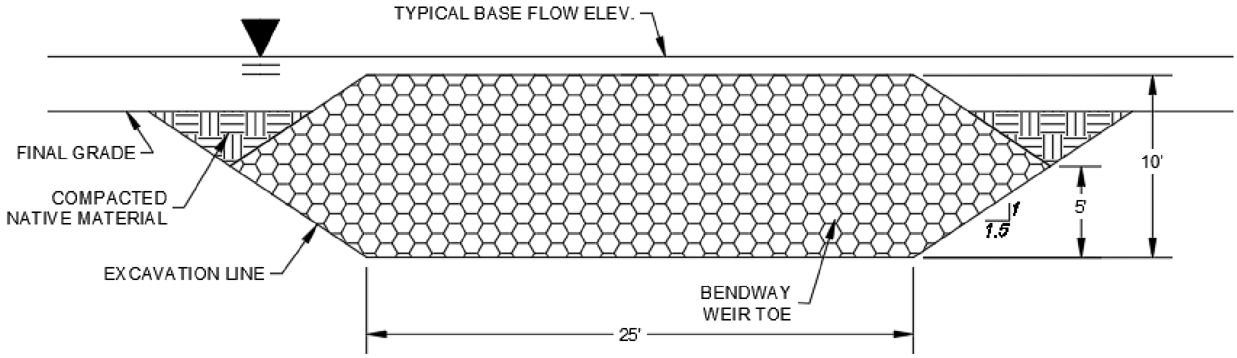


Figure 4-6 Cross section of Bendway Weir Toe Concept (Section 1 from Figure 7-14).

Guidance for the design of Bendway Weirs by the Bureau of Reclamation (2015) indicates that tie-backs and rock keys should be included to prevent flanking and erosion behind the structures. Of particular concern is that the erosion that could occur behind a structure and encourage the formation of a new active channel away from the left bank. To avoid this, buried rock tie-backs would be required in the design that extend from the weirs to the right bank levee, approximately 800 ft to the north. The total footprint area of trench excavation is estimated to be 4.9 acres, which can be used to estimate temporary impacts associated with the project. Permanent impacts can be estimated by the total area of the exposed rock crown, which is about 2.0 acres.

Figure 4-7 presents a cross section of a concept design of the tie-back structures. The tie-backs would be buried deep enough to resist anticipated scour (around 10 ft) and wide enough to remain stable during high flows. During the 25-year event, velocities could reach up to 13 ft/s in the channel. Therefore, the tie-backs would be constructed of large rock (FHWA Class 8 with D_{100} of about 5 ft) over the entire length of the structures.

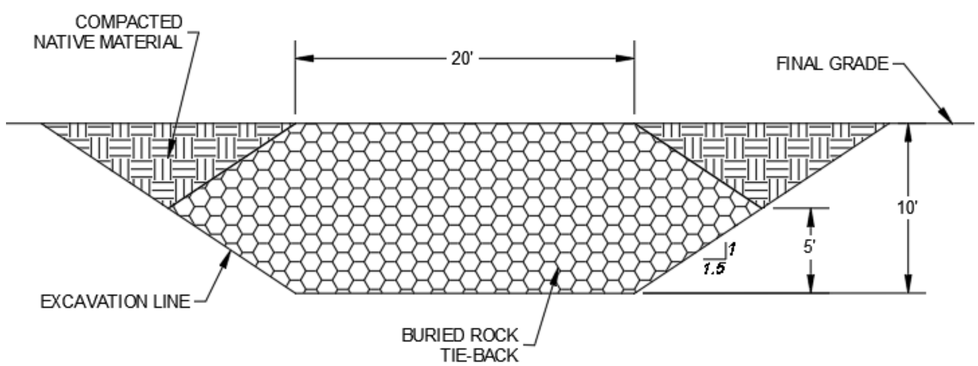


Figure 4-7 Typical Cross section of buried rock tie-back concept (Section 2 from Figure 7-14).

Due to the high velocities expected in the Santa Clara River during floods, non-cohesive bed material, and non-standard application of the structures, it is difficult to assess the performance of the Bendway Weirs with certainty. Confidence in the design is limited by the size and volume of rock required for stability during large flood events.

4.3.1.2 Operation and Maintenance Considerations

A second alternative for managing the effects of active channel realignment is through an Operation and Maintenance (O&M) program. The O&M program could be designed to anticipate the need for channel and infrastructure maintenance based on a variety of flooding scenarios and could include approved permits for access under specific conditions.

The cost and impacts associated with an O&M solution would be lower than for construction of the Bendway Weirs. Assuming that a new channel is to be excavated after a flood event that is 1,600 ft in length, 40 ft wide, and 5 ft deep results in an impact area of 1.3 acres, which is less than the impacts of the Bendway Weirs alternative. The project includes features such as the training wall, flushing channel and hardened ramp crest gates that can be utilized if there is sufficient flow to assist in creating hydraulic head to cut a localized channel. It is suggested that the O&M plan include guidance to utilize these features first and that cutting a channel would be a secondary option only utilized if required.

4.3.1.3 Recommendation

Based on the construction feasibility, unproven record for this application, environmental impacts and expected benefits of the structures, incorporation of the Bendway Weirs in the design is not recommended. It is recommended to pursue the development of a preliminary O&M program that can be used to mechanically maintain an open channel connection between the diversion inlet and active channel after floods.

4.4 Inset and Low Flow Channel Training

Within the active channel, an inset channel exists that transports typical flows. The inset channel is affected by common floods that can cause it to adjust or move away from the left bank. To help maintain the inset channel along the left bank, the use of local training structures was considered. The baseline condition physical modeling used a channel configuration found at the Freeman Diversion (USBR, 2022) that includes a separation of the inset channel from the left bank and a large bend back towards the bank just upstream of the proposed ramp entrance (Figure 4-8 and Figure 4-9).

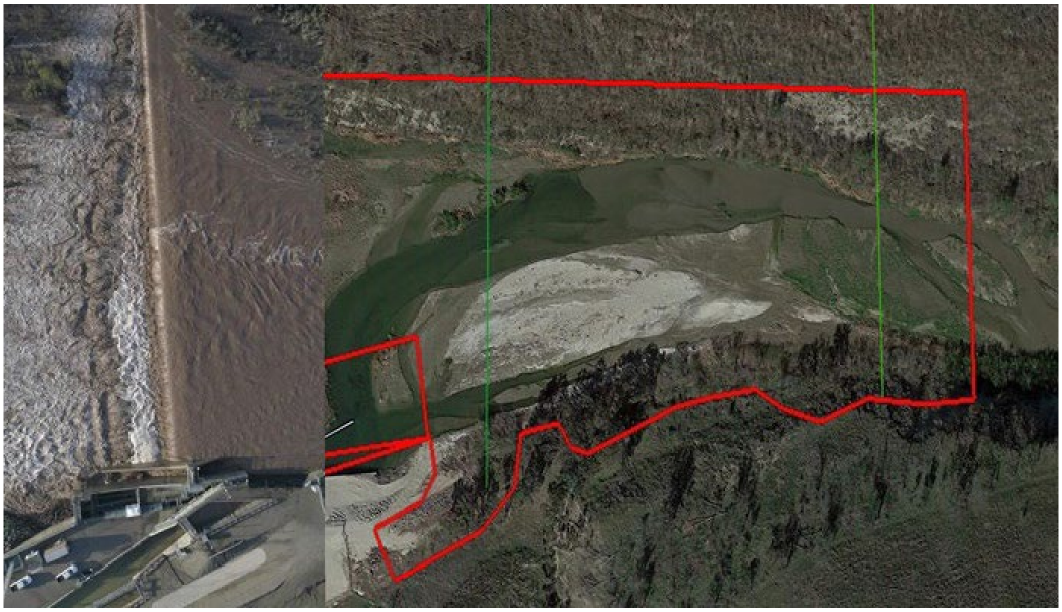


Figure 4-8 Aerial footage of 23,000 ft^3/s discharge over the dam crest, overlayed on a lower-flow aerial image and approximate upstream extents of the 1:24 model, outlined in red (from USBR, 2022).

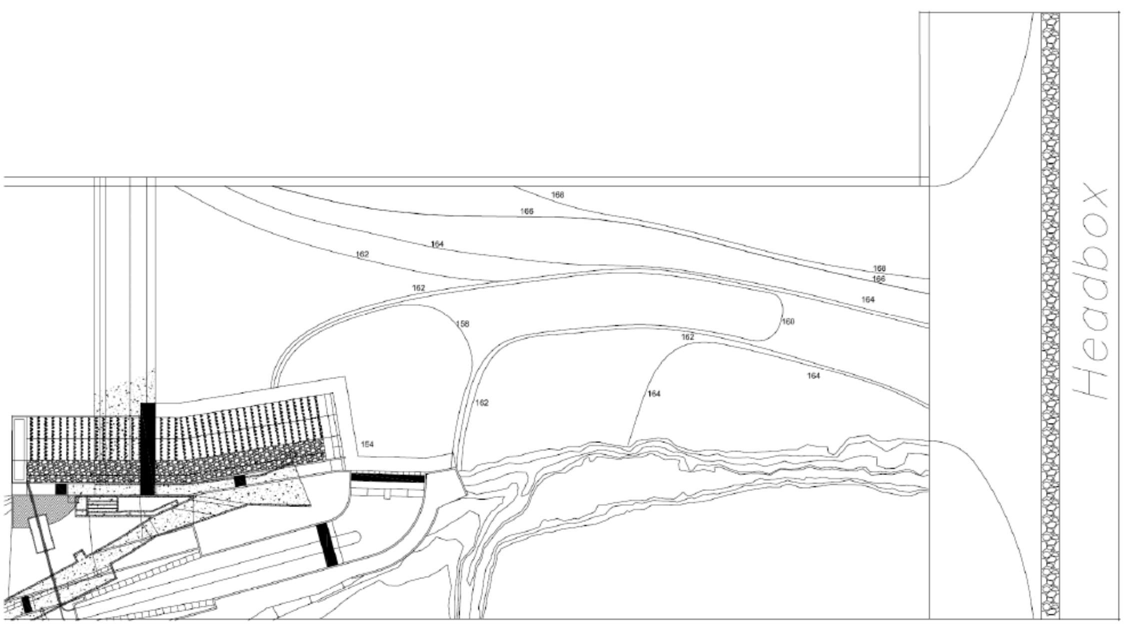


Figure 4-9 Layout of upstream topography adjustments used in 1:24 baseline tests (from USBR, 2022).

This planform resulted in adverse hydraulics at the lower range of flows. The flow distribution into the diversion was uneven and there was a high velocity transverse shear layer at the ramp entrance/exit that would negatively impact fish passage (Figure 4-10). These results suggest the need for river training works immediately upstream of the ramp and through the inset channel to align towards the left bank.

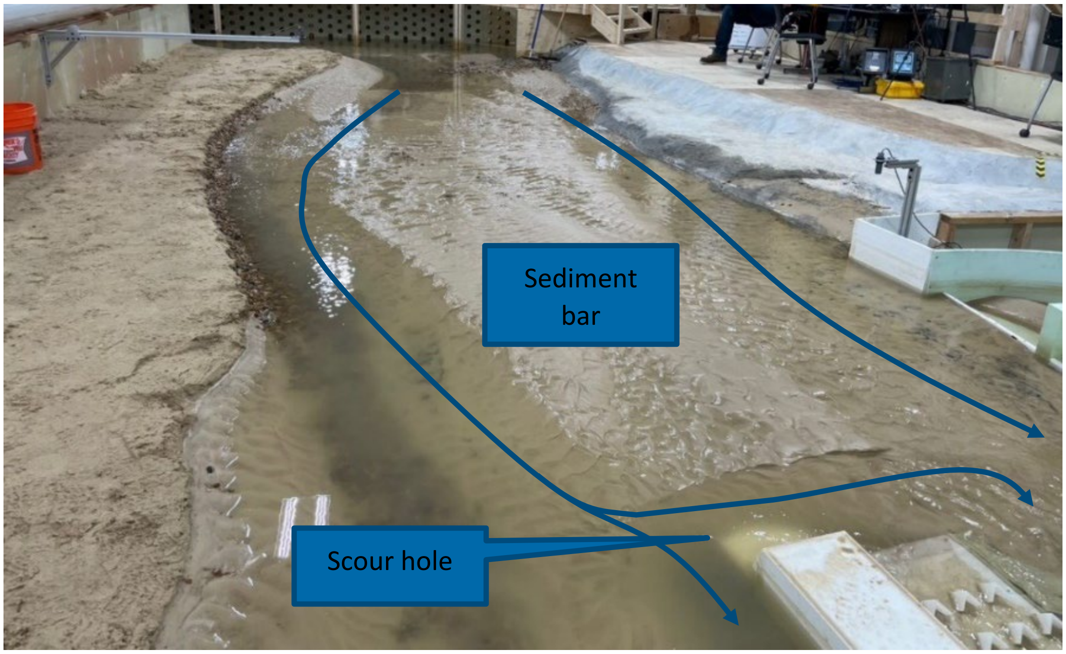


Figure 4-10 Upstream Channel with Baseline Configuration (initial design). Notice large sediment bar causing uneven flow towards ramp exit and intake, as well as deep scour hole on upstream end of bullnose.

To help maintain the inset channel along the left bank, the use of local training structures was considered. These structures included:

- an extension of the right abutment bullnose
- a flushing channel, and
- a training wall and apron
- short groins built out from the left bank.

USBR (2022) discusses other designs tested in the physical model. Using the physical model, training structure options were tested to see if they might be effective in encouraging the river to remain closer to the left channel bank where the diversion inlet is located. The following sections discuss only the physical features that were shown to have a benefit.

4.4.1 Right Abutment Bullnose

An extension was included on the right abutment bullnose to re-direct and orient the flows further upstream of the ramp. This has a benefit for fish passage by giving it time to become oriented normal to the ramp entrance and parallel to the banks. This eliminated the adverse conditions observed near the baffles () and helps to move any potential inset meander upstream.

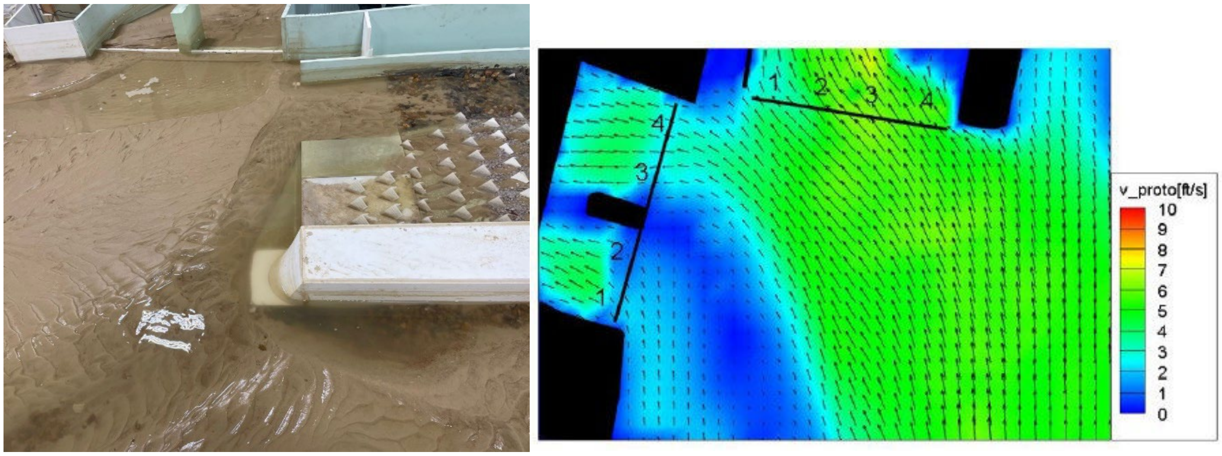


Figure 4-11 Post Bed Conditions showing scouring and preferred flow paths around the right wall (left); Prototype velocity color map of surface velocities in area upstream of diversion entrance for MOD-9 Design with Flushing Channel Closed (from USBR, 2022) (right).

This modification was first tested using numerical models to confirm change in velocity vector orientation and were then implemented into the physical model. The modifications to the physical model are shown in Figure 4-12 and in the revised plans.

Figure 4-12 Photographs (USBR, 2022) of the 1:12 model comparing the original design of the bullnose (left) to the extension that is used in the design (right).

Figure 4-13 shows the flow path and sedimentation after moving the thalweg of the inset channel near the left bank, extending the bullnose wall. Flow conditions and sedimentation improved, as described in the USBR (2022) report:

> *Flow conditions at the upstream end of the hardened ramp where fish exit the ramp into the upstream river were greatly improved by keeping the river thalweg to the left, adjacent to the riverbank. This provided a more uniform flow approaching the hardened ramp. Extending the right approach wall of the hardened ramp further upstream with a fully rounded bullnose geometry also reduced flow separation and improved hydraulic conditions for fish exiting the ramp, especially for river discharges greater than 1,500 ft^3/s.*

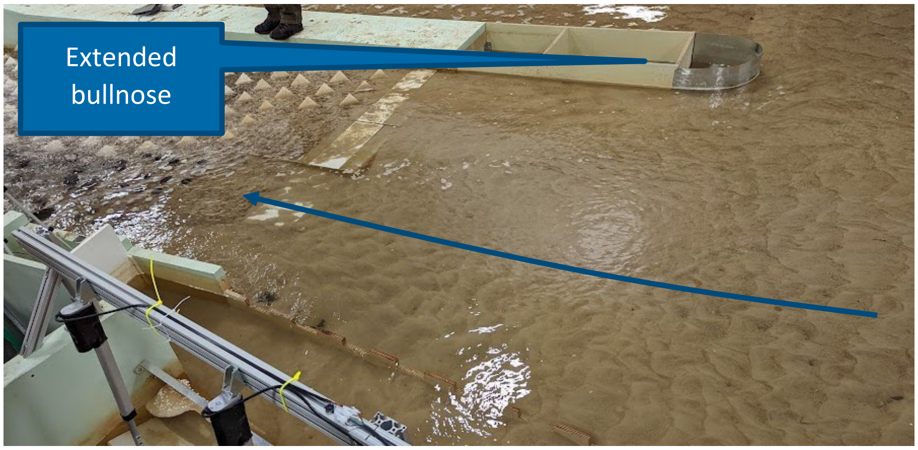


Figure 4-13 Flow path and sedimentation at the hardened ramp inlet with bullnose extension.

Figure 4-14, shows the potential for scour around the bullnose after a preliminary model run at high flows. Scour calculations were completed around the bullnose feature to size potential size and extent of necessary protection (discussed in Section 4.2).

Figure 4-14 Bullnose from the 1:24 physical model with sediment scoured around the toe post high flow model run, showing need for toe protection.

4.4.2 Flushing Channel

The general layout of the flushing channel, apron and training wall are shown in Figure 1-1, while a closer view including a longitudinal profile of the flushing channel and apron are shown in Figure 4-15. These features are utilized for sediment management, river training and diversion operations.

The 15 ft wide rectangular flushing channel is 483-ft long with a 2.54% longitudinal slope. Flow into the flushing channel is controlled by a 15 ft wide by 20 ft high overshot crest gate. The crest gate was chosen to replace the undershot gate to reduce potential effects on fish by spilling via a free surface rather than passing under pressurized high velocities through an undershot gate. The upstream invert of the channel is El. 146.0 ft. The downstream end of the flushing channel connects with the sluice channel coming from the desander providing a single common exit for both channels at invert El. 134.0 ft, see Figure 4-15.

During the design development phase in the physical model (USBR, 2022), the initial 30% design of the flushing channel (NHC, 2021) was substantially improved. The initial design considered a higher elevation channel with a flat upstream apron without a training wall and an undershot gate. Baseline testing of the initial design showed that sediment was only removed a short distance upstream from the gate, while a plunging flow developed when the flow discharged downstream, which could impact fish and generate a deep scour hole.

The entire invert of the flushing channel was lowered to match the downstream elevation of the hardened ramp, such that a deep tailwater pool was available, eliminating the plunging flow and local scour hole, similar to the hardened ramp gradual transition. Increasing the slope of the apron upstream of the flushing channel plus the addition of a training wall was necessary to concentrate flow and improve sediment transport, as discussed next.

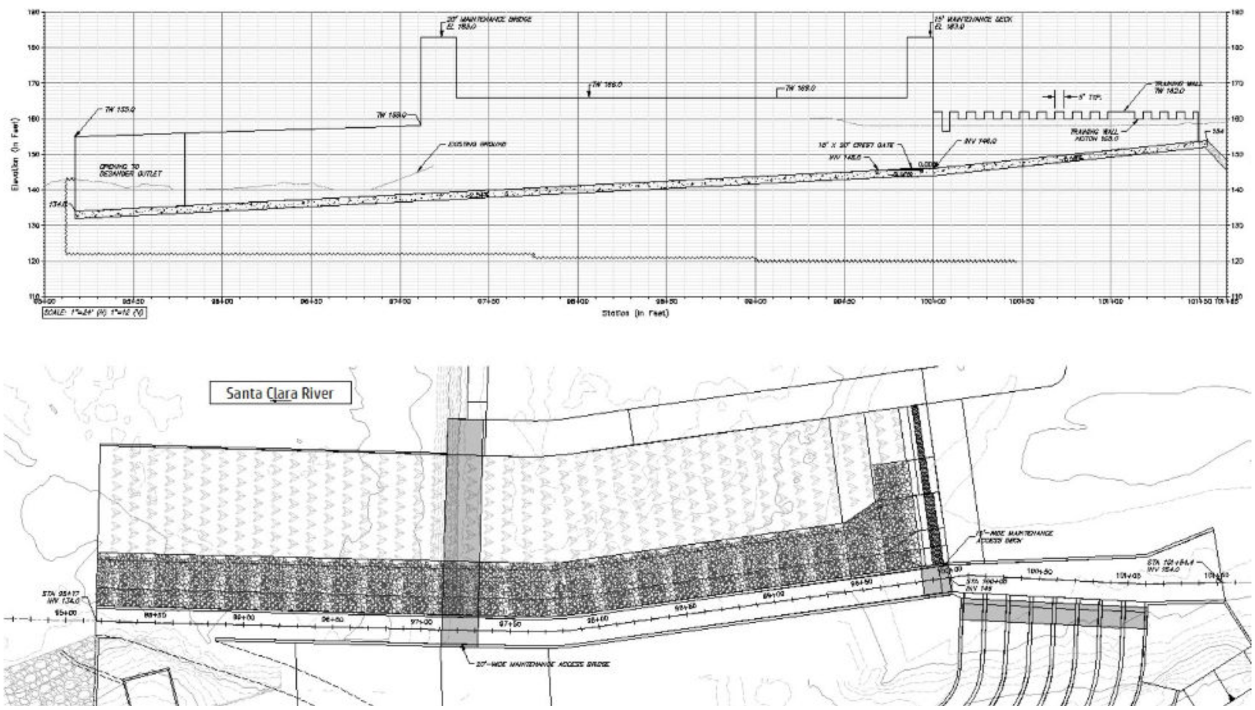


Figure 4-15 Flushing channel, apron, and training wall.

4.4.3 Training Wall and Apron

The training wall has the primary function of improving the efficiency of the flushing channel to clean the apron, while also partially blocking some of the bedload sediment coming toward the intake. A training wall was developed, located along the left bank adjacent to the diversion intake (Figure 4-15). The training wall serves multiple purposes, including for sediment management and flexibility of operations. The training wall and flushing channel work in tandem to help develop a channel along the left bank adjacent to the structure.

There are two important and interrelated design parameters for the training wall: its top elevation and its opening width at the upstream end of the intake. A tall training wall with narrow opening is better for flushing sediment but may reduce diversion flows. A short wall with wide opening is better for flow diversions but may prove ineffective to flush sediment. These effects were observed both in the CFD model (NHC, 2021) and physical model (USBR, 2022). In the physical model, openings of 15, 25 and 40 ft were tested, along with various top elevations of the training wall. The best results were found for an upstream opening of 25 ft (Figure 4-16) and a "castle wall" type configuration with top El. 162.0 ft (Figure 4-17).

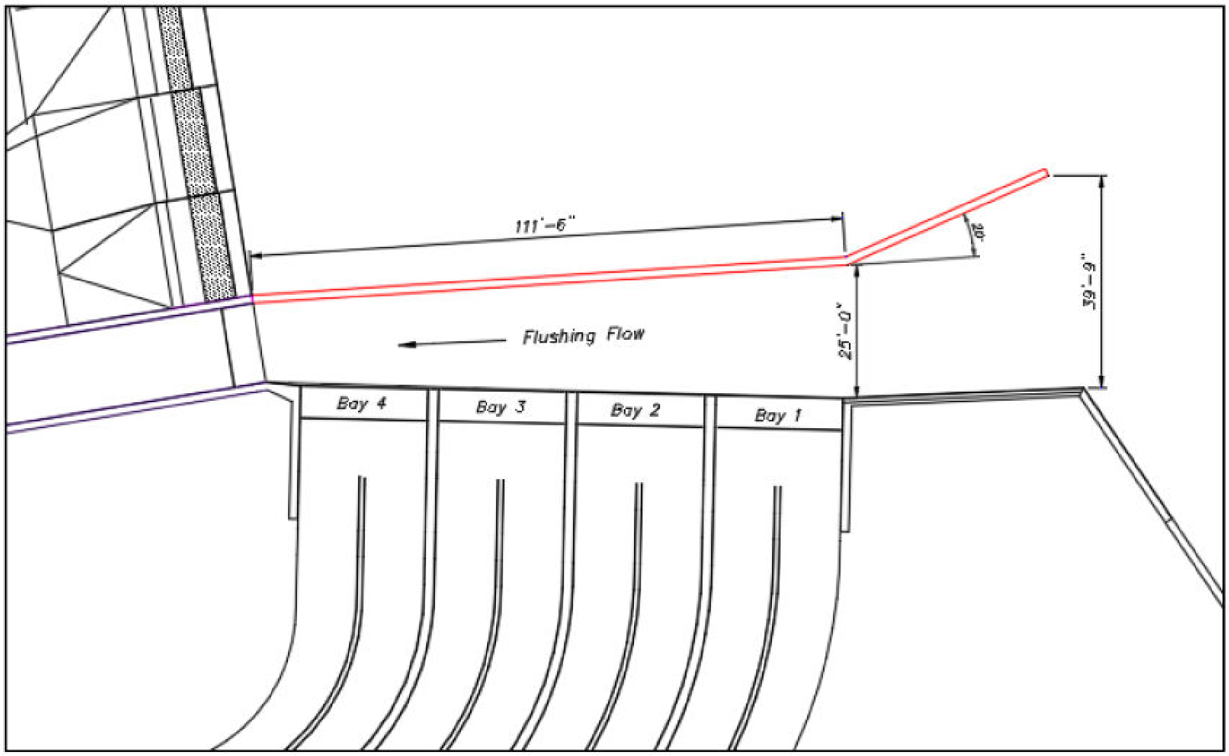


Figure 4-16 Training wall layout (USBR, 2022).

Figure 4-17 Image of training wall in 1:24 physical model, viewed from river right (USBR, 2022).

The gradual width reduction created by the training wall on the apron, from 25 ft to 15 ft, accelerates the flow when the flushing channel operates, ensuring an efficient and continuous sediment movement from the apron towards the flushing channel downstream. The castle training wall has 5 ft wide and 2 ft deep slot openings, which allow flow exchange across the entire wall length whenever water levels exceed El. 160 ft. Additionally, the first slot closest to hardened ramp has a lower El. 156.5 ft (same as low-flow channel invert) to provide hydraulic connectivity for fish and avoid creating an isolated pond.

The 145 ft long apron slopes 5.2% from its upstream end at El. 154.0 ft towards the entrance of the flushing channel at El. 146.0 ft. The apron is bounded laterally by the intake structure on the left side and the training wall on the right side (looking downstream). The initial 30% design of the apron (NHC, 2021) considered a flat structure at El. 154.0 ft, which proved infective to move sediment deposited in front of the intake[1]. During design development, the slope of the apron was increased to first to 2.3% and then to 5.2%. The 5.2% slope proved successful to efficiently mobilize sediment, while allowing some additional room for temporary storage of sediment between flushing operations.

Water moving toward the intake follows two possible paths. One water path is over the apron and through the opening at the upstream end of the training wall, and the other path is over the training wall. Bedload will freely enter through the upstream opening and move towards the apron. However, the training wall provides a physical barrier to allow cleaner surface flow above El. 160 ft to enter the diversion, while blocking and redirecting bedload-laden bottoms flows towards the ramp's low-flow channel. A small bottom-flow current forms following the right side of the training wall, carving a small channel toward the low flow channel (Figure 7-7), which could improve connectivity for fish under certain flow conditions.

Physical model tests showed the castle wall to help train the river to the left when compared to similar tests without the training wall in place (USBR, 2022). Inclusion of the flushing channel had more flow drawn to river left, resulting in more flow directed to the hardened ramp and less over the dam. Physical model testing showed local scour to occur along the right side of the training wall due to turbulent eddies from flow passing through castle wall notches however, this scour was not persistent along the entire length of the training wall and could not be relied on to provide hydraulic connectivity from the low flow inset channel to the Hardened Ramp for all flow conditions.

4.4.4 Low Flow Channel Training

The hardened ramp and flushing channel have the lowest invert elevations in the river cross section at the location of Freeman Diversion. If -as expected- the flushing channel operates during floods, it seems unlikely that the river thalweg would migrate away from the left bank. Nevertheless, the ability of the inset low flow channel or river thalweg to re-establish on the left bank after flood events that could modify the channel morphology was tested using numerical and physical models. This section describes tests that were conducted to evaluate reconnection of the inset channel against the left bank allowing for ramp and diversion operations.

---

[1] NHC (2021) recognized that apron might need to be modified: "It may also be possible to change the invert elevation of the flushing channel itself and the gate invert, along with shaping and sloping of the apron in front of the intake to promote sediment movement and entrainment."

4.4.4.1 Preliminary Numerical Modeling

Preliminary hydraulic modeling using a simplified erosion model was completed to test the potential for the formation of a channel that cuts back to the hardened ramp if the channel thalweg migrated after a flood event.

A HEC-RAS 2D hydraulic model of the Freeman Diversion and proposed hardened ramp design changes was developed for evaluating hydraulic conditions near the proposed river training structures. The model domain included the diversion dam, hardened ramp, diversion inlet and approximately 1,100 ft of river channel upstream. A simplified erosion model was developed that used the terrain, shear stress, and land cover rasters from the HEC-RAS 2D model as input files to calculate eroded surfaces over time.

To better understand how the system might respond to a change in the active channel, a hypothetical channel geometry was developed that considered a new alignment along the right side of the main river channel. The adjacent terrain was flat across the entire river with a constant gradient in the downstream direction. A 100 ft-wide pilot channel was constructed in the northern side of the river to simulate the formation of a new active channel. The erosion model does not simulate sediment deposition, which would tend to fill the blocked channel near the dam with sand during lower flows.

The eroded terrain from the model showed a small but distinct channel that was eroded out along the left bank. The general conclusion from the erosion modeling of the realigned channel suggests that the river would have sufficient energy and shear stress to cut a connection between a realigned active channel and the diversion inlet. The model assumed that the flushing channel was open during the receding limb of the flood hydrograph. Utilization of an O&M plan to cut an initial pilot channel would accelerate this process. This trend was explored further using the physical model, which is a more appropriate tool, but has limited right bank extents.

4.4.4.2 Physical Modeling

Two physical model tests were conducted to investigate implications of thalweg migration to river right (USBR, 2023). These tests were run at river flows of 250 ft^3/s (1:12 model) and 1,500 ft^3/s (1:24 model). Before the start of the tests, the channel was manually aligned to the right, away from the diversion intake to represent a large adverse morphological change.

The two tests showed similar findings: flow continued down the hardened ramp but could not access the upstream end of the diversion intake due to sediment blockage. Flow initially accessed the downstream end of the intake through the low notch in the training wall. Excavation of a pilot channel was required to reconnect the thalweg to the intake upstream of the training wall, and operation of the flushing channel could be used to clear sediment from the apron and head cut a formalized channel through the excavated pilot channel to the diversion intake, see Figure 4-18.

Figure 4-18 Results of a low flow reconnection test after thalweg migration.

4.4.4.3 Recommendations

The use of groins was tested in the physical model to re-establish the low flow channel and the results were inconclusive. Other possible options, such as raising the hardened ramp gates to increase water levels and redirect flow towards the intake, may also be beneficial, but were not tested. Mechanical excavation of a small pilot channel upstream of the training wall, in combination with operation of the flushing channel to assist in formation of a connecting channel, appears to work but requires permitted instream work.

Regardless, reconnection of the low flow channel with the hardened ramp occurred consistently through all model testing with the bullnose, training wall and lowered apron arrangement. In the case that the river channel's thalweg migrates to the right during a flood event (for example, because the flushing channel and intake remained closed during the flood), the diversion and flushing channel can be used to help to reconnect the channel to the diversion on the left bank once the flood has receded. Based on physical model testing, a river right thalweg effectively reconnects to the Hardened Ramp inlet simply due to its lowered elevation. Use of the notch gate in the training wall allows reconnection of low flows to the diversion intake.

NHC recommends these physical structures to assist in keeping the flow oriented onto the left bank. Installation of the bullnose extension will help to redirect and re-orient flows towards the ramp further upstream which will help to smooth the flow lines parallel to the ramp entrance. The training wall orients water upstream of the intake parallel to the banks. Used in conjunction with the flushing channel it lowers the bed elevations and creates a preferred path towards the intake and low flow channel to help to train the river towards the left bank. The flushing channel could be operated during floods to increase the left bank-oriented discharge and help prevent the river channel's thalweg from migrating away from the apron entrance and intake.

5 Hardened Ramp

Figure 5-1 provides a plan view of the primary components of the Hardened Ramp. This section describes the design objectives for the ramp, followed by the key design components, design development during physical modeling, and fish passage assessment.

Figure 5-1 Hardened Ramp Primary Components.

5.1 Hardened Ramp Objectives

The ramp was designed to meet the fish passage needs as outlined in the Amicus Brief (see Section 2.1) as well as the water diversion needs of UWCD while effectively managing sediment. The primary Hardened Ramp objectives are:

- provide primary upstream fish passage for steelhead and Pacific lamprey at river flows from 45 to 6,000 ft^3/s
    - At river flows from 45 to 500 ft^3/s, passage is primarily provided in the low flow section
    - At river flows from 500 to 6,000 ft^3/s, passage is primarily provided in the baffle section.

The Hardened Ramp can operate continuously during steelhead migration periods, without shutdown during sediment flushing or debris removal, which maximizes the possible fish passage window during larger flow events. The lowered ramp elevations provides a river thalweg – lowest point in the river channel – hence a tendency to keep a continuous low flow channel. A continuous graded channel provides improved conditions for fish movement. As the Hardened Ramp has no headpond or storage, there is continuous transport of sediments proportionate with river discharge and re-establishment of a more natural sediment flux.

5.2 Hardened Ramp Configuration

The initial ramp design and further modifications was numerically modeled extensively in the design development process and results reviewed with NMFS and CDFW. The Hardened Ramp shown in Figure 1-1 and in the attached plans (Appendix A) has the following characteristics:

- Ramp length – 480 feet

- Ramp slope - 5 percent
- Ramp width – 90 feet
- Cross section shape – 60 feet wide 30:1 cross slope in baffled section (2 feet cross slope); 30 feet wide 2 feet deep roughened channel low flow section
- Invert at crest – El. 156.5 to 159.0
- Invert at ramp entrance – El. 134.0 to 138.0
- Fish passage flow range – 45 ft^3/s to 6,000 ft^3/s river flow.

The right abutment is 20 ft wide at an elevation of 166 ft, located between the hardened ramp and dam. Initial simulations in the physical model showed adverse flow conditions for fish passage at the upstream exit to the ramp. To alleviate these conditions the right abutment wall and bullnose was extended upstream 80 ft (Figure 4-12). The bullnose terminates with a 10 ft radius on its upstream edge and is surrounded by erosion protection. The bullnose extension re-directed the flows which improved the approach flow conditions to the ramp. The development of the bullnose in the physical model was discussed previously in Section 4.4.1.

The Hardened Ramp section produced favorable fish passage hydraulics through a wide range of ramp flows in the physical model and was considered likely to meet the objective of steelhead passage at river flows from 45 ft^3/s to 6,000 ft^3/s (NHC, 2020). As per the Amicus Brief, the ramp has a slope of approximately 5% and conveys all discharges less than 1,200 ft^3/s to attract fish to enter the downstream end of the ramp and preclude nuisance attraction. The ability to convey 2,800 ft^3/s in the ramp when total river flows are 6,000 ft^3/s is a major factor in both the range of effective fish passage and relative attraction of the ramp entrance.

5.2.1 Baffled Channel Section

The baffled channel section is intended to provide fish passage at Hardened Ramp flows from approximately 500 to 2,800 ft^3/s (river flow of 6,000 ft^3/s). The baffles are a V-shaped sloped steel plate placed in a staggered pattern in transverse rows. The sloped baffle shape is expected to perform better than vertical walls for shedding debris in the ramp. A trade-off for fish passage exists between baffle width and their spacing (slot opening). Wider baffles provide ample refuge zones behind, while larger slot openings improve the passage of debris and sediment. However, if the slot opening becomes too large, velocity can increase and reach values that could hamper fish passage.

Baffle width and slot opening were modified to optimize opportunities for fish passage resulting from sheltering by baffles while maximizing open space to allow for debris passage. Through CFD model testing (NHC, 2020) it was found that the largest baffle width tested provided insufficient sheltering and overly high velocities, so a 5 ft baffle width with a 2.5 ft slot opening was adopted. The baffles extend to the slab of the ramp and the rock roughness extends around each baffle element.

The row-to-row spacing, lateral spacing, baffle elevation, shown in Figure 5-2, remained consistent with earlier designs. The physical model showed that flow conditions within the baffled section of the ramp were satisfactory after the addition of the bullnose (Section 4.4.1) and no changes were made to the baffles through design development. Design development for the baffle materials were discussed in the HBOD (NHC and GEI, 2019) and the testing for optimal spacing within the DDR (NHC, 2020).

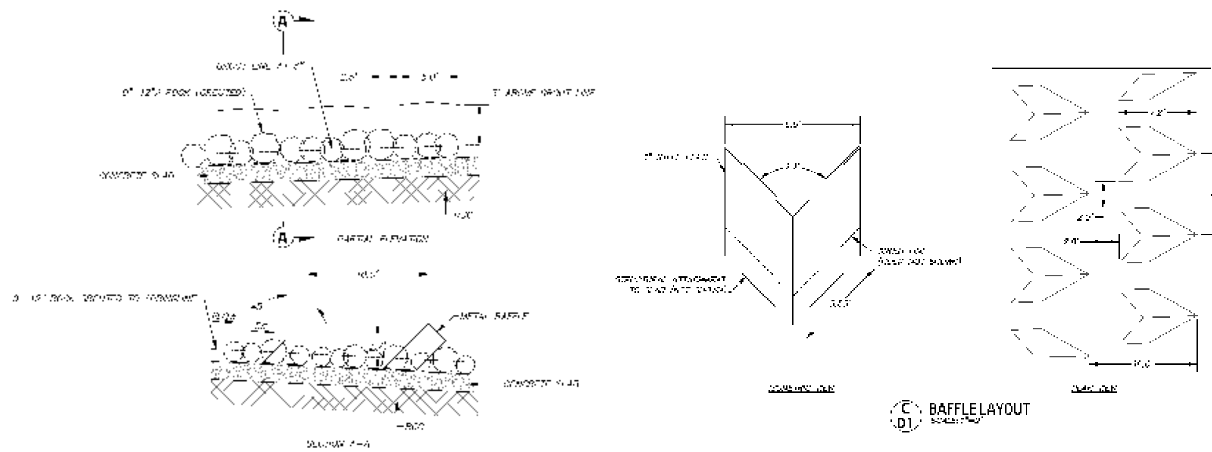


Figure 5-2 Hardened Ramp Baffle Detail.

5.2.2 Low Flow Channel Section

The low flow channel provides the primary fish passage for low design flows (45 ft^3/s to 500 ft^3/s). The low flow channel is constructed with a grouted base of 12 in to 18 in size rock to provide roughness, additional energy dissipation and lower velocities near the bed. Larger boulders from 20 in to 40 in diameter are placed in a repeating pattern across the channel to provide structured flow and acceptable fish passage hydraulics (Figure 5-3).

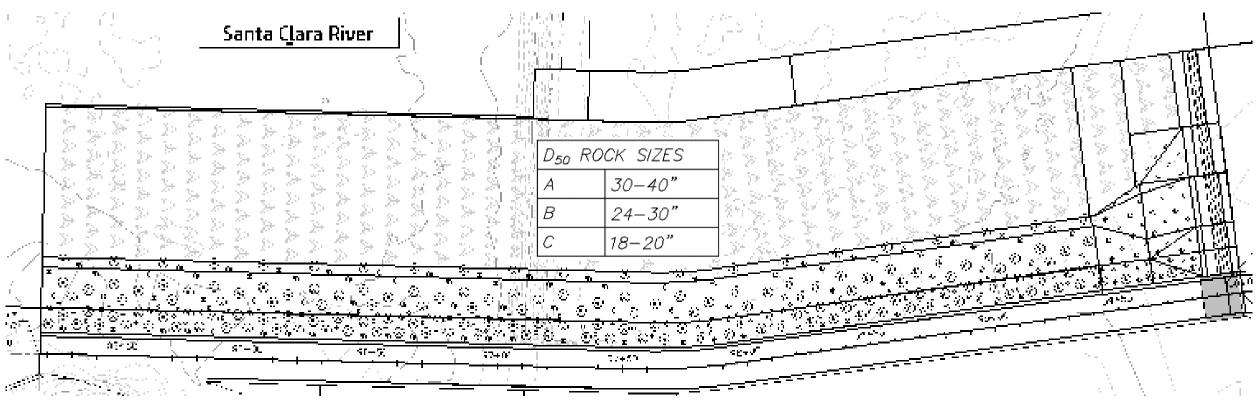


Figure 5-3 Hardened Ramp Rock Placement on Low Flow Channel.

The design development took place using physical modeling on the 1:12 model. The initial proposed design was presented to the agencies at a meeting and then tested by the USBR in the physical model where the design team and agencies were able to discuss and give feedback. The initial physical model low flow channel configuration included large roughness elements that crossed the channel perpendicular to the flow like weirs (left photos in Figure 5-4 and Figure 5-5). This trial #1 resulted in streaming flow with higher velocities and lower depths than optimal across the weirs. The formation also lacked holding area hydraulics along the transition slopes into the baffled section and diversity across the channel.

The second trial spread the large weir rock into varied patterns across the low flow channel section. Generally, the area of volitional fish passage started in the central portion of the low flow channel and shifted to the lateral edges as flows increased. However, initial physical testing found that the unstructured large boulder placement led to accelerations and adverse hydraulics between roughness elements. Observations from the 1:12 physical model baseline showed hydraulics in the low flow channel section could be improved with increased area of reduced velocities and increased diversity of fish passage routes (USBR, 2022).

Through discussion and testing in two physical model trials, options were explored to determine a design approach that would provide a better transition between the baffles and longitudinal pathways in and across the low flow channel that would be activated through a range of flows. A third trial was developed that included:

- Staggered side roughness slope between baffled section and low flow section to provide a transition.
- Development of an internal lower profile channel structure for low flow performance as "Staggered Z" in the thalweg.
- Redistribution of step weirs and gapping of roughness elements into bands rather than weir elements and providing central gap for sediment and low flow roughness.
- Redesign left bank roughness into gap graded, designed placement for secondary pathway of movement (mid flows).

From this information additional refinements were made for Trial #3. This included adding additional large roughness elements in the center (invert) of the low flow channel and along the south bank. Refinements to the low flow channel included primarily placement of additional 1 to 2 ft grouted rock roughness elements and boulder placement (right photos in Figure 5-4 and Figure 5-5).

Table 5-1 Rock Gradations Used in Low Flow Channel Section.

Rock Gradations
Size A (30-40")
Size B (24-30")
Size C (18-20")

Data collected on trial #3 by the USBR (Table 5-2) showed that the changes to the low flow channel results in lower velocities and higher depths near the centerline (where data was collected), both advantageous to fish passage (USBR, 2022). The additional rocks and staggering of roughness provide reduction in velocities on the margins of the channel as well. This design iteration was retained, and fish passage hydraulics were further evaluated (Section 5.3).

Table 5-2 Comparison of Velocities and Depths Between Trial #1 and Trial #3 of the Low Flow Section Improvements (From USBR, 2022).

	Version 1 (Centerline)		Version 2 (Centerline)	
River Flow	***Velocity***	***Depth***	***Velocity***	***Depth***
ft³/s	*ft/s*	*ft*	*ft/s*	*ft*
400	10.4	2.5	8.5	3.8
300	10.3	2.0	8.1	3.7
200	6.8	1.5	5.4	2.5

Figure 5-4 Trial #1 Low Flow Channel Weir Flow (Left) and Trial #3 Distributed Roughness Elements (Right).

Figure 5-5 Longitudinal View Looking Upstream of Ramp from Trial #1 (Left) and Trial #3 Configuration (Right) at 600 ft^3/s.

5.2.3 Ramp Exit Hydraulics

The upstream end of the ramp was modified in earlier modeling and design development to improve hydraulics and fish passage. The ramp exit shifts in the upstream direction from a 30'/60' low flow/baffled flow section to a 60'/30' split as the width of the low flow section transitions first from 30 ft to 40 ft over 20 longitudinal feet and then to a 60 ft wide channel over an additional 20 ft (Figure 5-6, Figure 5-7).

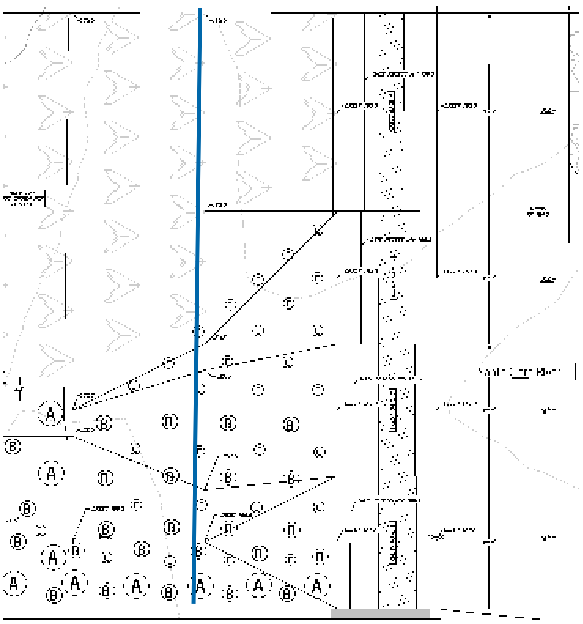


Figure 5-6 Plan View of Hardened Ramp Upstream Exit.

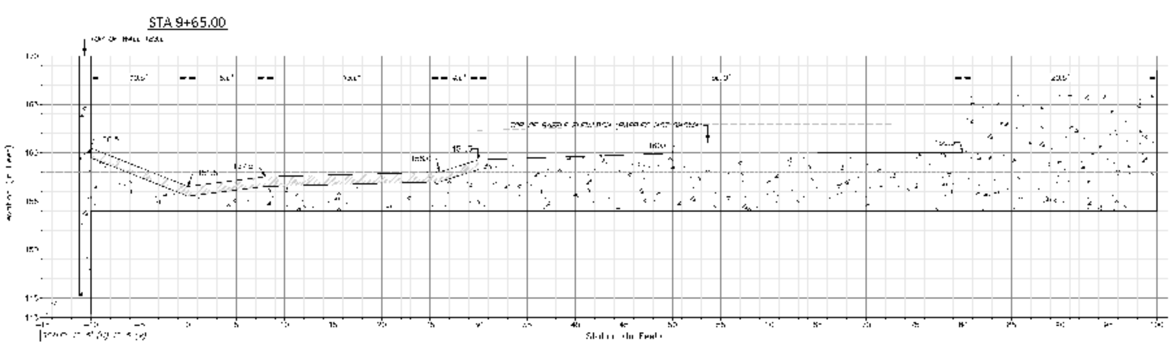


Figure 5-7 Cross-section view of ramp (blue line from figure above).

This longitudinal width transition both opens the section conveyance and adds nature-like roughness which smooths out the accelerating flows into the typical low flow channel down the remaining Hardened Ramp. Initial tests showed high velocities and low depths across the crest of the structure. To break this up roughness elements were added by extending the grouted 1 to 2 ft rocks up to the hardened ramp crest gates (Figure 5-8).

Figure 5-8 Roughness Elements at Crest of Structure during Baseline Conditions (Left) and Final Configuration (Right).

The addition of these grouted rocks was shown to reduce velocities and increase depths for data collected at river flow of 3,000 ft^3/s in the upstream section of the ramp (USBR, 2022), see Table 5-3 and Figure 5-9.

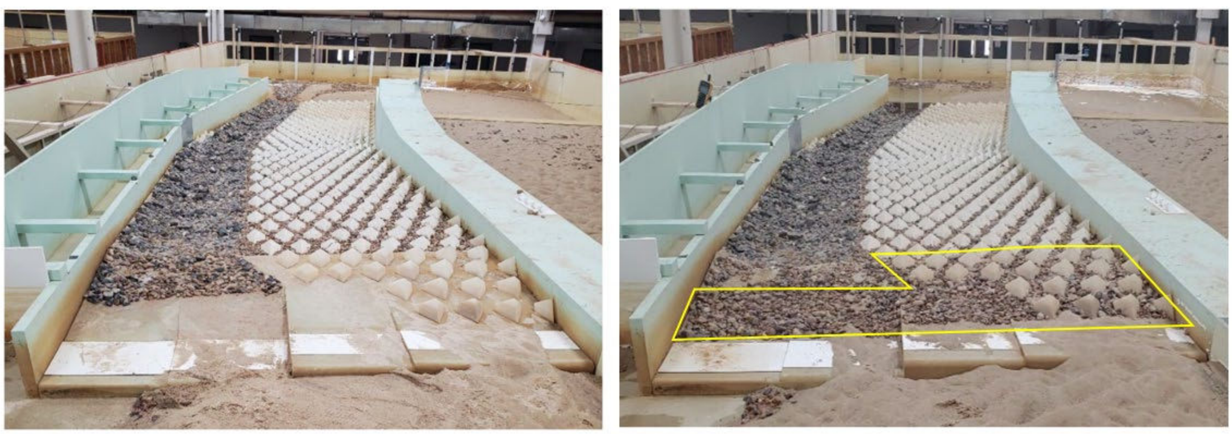


Figure 5-9: Upstream section of hardened ramp before (left) and after (right) additional rock placement. Additional rock placement extents shown in yellow.

Table 5-3 Hardened Ramp Roughness Testing at 3,000 ft^3/s (total river flow) from USBR 2022.

Location		Baseline		With Additional Rocks	
		Velocity	Depth	Velocity	Depth
-	-	*ft/s*	*ft*	*ft/s*	*ft*
Hardened Ramp	Gate 1	4.4	6	4.2	6
	Gate 2	4.8	5.1*	4.2	5.8
	Gate 3	3.9	2.2	3.7	3.6
	Gate 4	1.2	2.5	2	3
Low-Flow Section	Low-Flow Left	5.9	5.4	5.9	5
	Low-Flow Center	4.3	3.6	5.1	4.6
Baffles	Center of Baffle Section	1.1	**	0.7	**

* Measurement was taken in an area with sediment deposition that impacted depth.
** Depths were not measured in the baffled area

Boulders were subsequently added to the grouted rocks to provide additional flow diversity along the upstream end of the ramp, see Figure 5-8.

5.2.4 Hardened Ramp Gates

The hardened ramp entrance is 90 ft wide and has four stepped sills increasing in elevation from south to north. The three left gate sills are at a 20 ft width, and the rightmost gate at a 30 ft width. Gate sill invert elevations from left (south) to right (north) are set at 156.5 ft, 157.0 ft, 159.3 ft and 160.0 ft. The Obermeyer crest gates sit on the upstream 15 ft of the sill and lay flat when not deployed. Discussions with Obermeyer confirmed that these types of gates could work for this configuration overlapping to create a seal to minimize leakage.

The gates are raised to close off the Hardened Ramp to protect the ramp from bedload transport during large flow events. The gates will span across the ramp (90 ft) with stepped bottom elevations and a consistent top elevation of 164 ft. A plan view of the crest gate layout is given in Figure 5-10 and the hardened ramp upstream elevation cross sections are shown in Figure 5-11.

While the primary function of these gates is for ramp protection, they can also provide potential operational flexibility for diversions and maintenance activities by concentrating or blocking flows within sections of the ramp. The gates could be lowered during extreme flood events where impacts to water surface elevations are most critical. Further design of gate configuration, anchoring, sill, and operators will take place during additional stages of design.

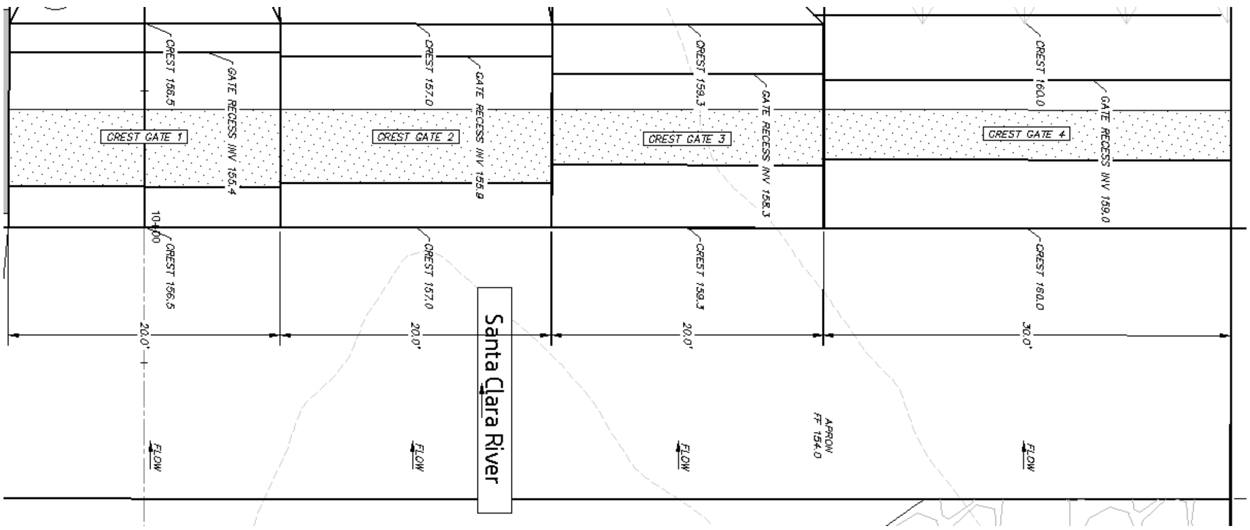


Figure 5-10 Hardened Ramp Crest Gate Plan View Schematic.

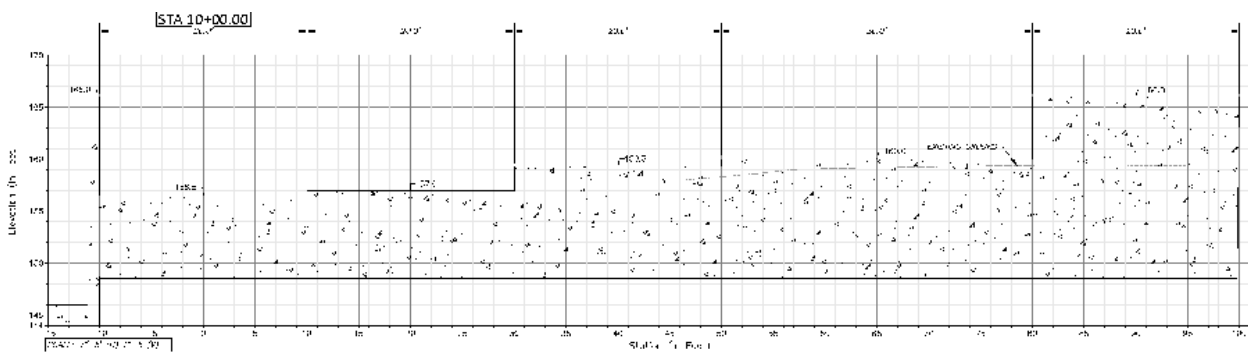


Figure 5-11 Hardened Ramp Crest Elevations Cross-Section.

5.2.5 Hardened Ramp Entrance Sedimentation

For configurations with a flushing channel in the 1:24 model, the baseline testing flushing channel had its downstream end discharging high above the riverbed and hardened ramp, causing a plunging flow when tailwater levels were below El. 145 ft. The plunging flow generated a deep local scour hole downstream of the flushing channel and adjacent to the hardened ramp (Figure 5-12). During design development the downstream end of the flushing channel was lowered to the same elevation of the hardened ramp (El. 134 ft) and combined with the exit of the sluicing channel coming from the desander, eliminating the plunging flow and scour hole with some transient sedimentation when sediment was being flushed (Figure 5-13).

Figure 5-12 Scour downstream of flushing channel for original design configuration, 1:24 model.


Figure 5-13 Flow conditions at exit of flushing channel and sluicing channel (left); and post-test conditions with transient sedimentation from sluice channel (right).

The conditions downstream of the ramp are described in the USBR (2022) report:

> *Flow streamlines downstream from the hardened ramp generally paralleled the topography of the left bank as flow exited the ramp and dispersed uniformly as it moved downstream. Sediment bedforms that developed downstream did not produce extreme localized scour or deposition features that would be detrimental to attracting fish into the ramp. Temporary deposition in this area due to flushing or sluicing was eventually moved and reformed by continual flow exiting the hardened ramp.*

The physical model showed a gradual bottom transition downstream between the hardened ramp structure and the natural alluvial stream, without the deep scour holes and bed armoring noticeable downstream of the adjacent dam structure (see Figure 5-14). Depending on downstream tailwater conditions, some sedimentation is observed on the downstream reach of hardened ramp which can potentially cover portions of the baffles and low flow channel in the portion of the ramp submerged in tailwater; but this would effectively reduce the length of the ramp, which is a benefit for fisheries.

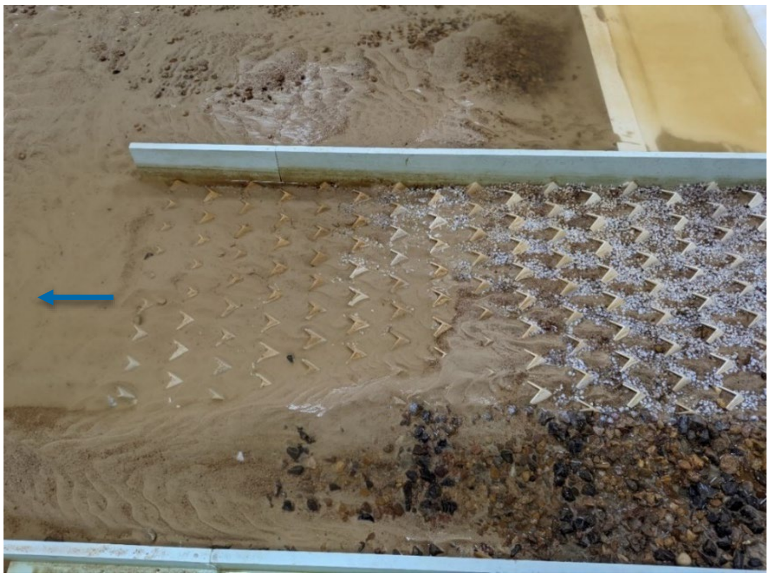


Figure 5-14 Gradual bed transition between hardened ramp and alluvial channel downstream. Compare with abrupt transition, armoring and scour hole downstream of dam (from the 1:24 physical model).

5.3 Hardened Ramp Fish Passage Assessment

A key feature of the Hardened Ramp is the ability to attract fish migrating upstream to enter the downstream end of the ramp and provide passage hydraulics over a wide range of discharges. The asymmetric section provides multiple flow pathways for upstream passage and a range of potentially acceptable depth and velocity conditions that allow fish to select differing positions in the water column, opportunities for lateral movement to resting areas, varied swimming speeds and energy expenditure during upstream movement. At lower flows, when the river flows entirely within the ramp, the broad conveyance capacity allows for fish passage. As flows increase, flow splits to the dam crest limit discharge increases in the ramp, retaining fish passage criteria up to the maximum fish passage design flow.

The fish passage metrics and criteria were developed in the NHC HBOD (2019) and are presented in Section 2.1. The primary metrics utilized for upstream fish passage were water depth and velocities, swimming speeds and swim distances by fish species and life stage. To assist in the interpretation of fish passage in the design development, NHC created a simple passage suitability model, and a preliminary suitability index was developed to incorporate suitable values of both velocity and depth. CFD modeling was used to develop hydraulic simulations that were evaluated. These evaluations resulted in design development of the baffled design and placement in the ramp and ramp inlet, and ramp inlet geometry modifications documented in the NHC DDR (2020).

Similar to the earlier HBOD analysis, evaluation of the following USBR physical model point and PIV data can be completed use simple metrics of depth and velocity for strong and weak swimmers – representing the larger anadromous steelhead and the smaller bodied native resident species – are developed from Table 2-1 and Table 2-2 and summarized in Table 5-4.

Table 5-4 Fish Passage Evaluation Metrics for USBR Model Data.

Fish Swimming Mode	Fish Passage Flow Range ft^3/s	Maximum Water Velocity ft/s	Minimum Water Depth ft
Strong Swimmers	45 to 6,000	8.0	1.0
Weak Swimmers	4 to 270	1.0	2 x body depths[1]

[1] a 12” FL fish at 5:1 FL/BD ratio would yield a minimum depth requirement of 5”.

5.3.1 Physical Model Performance

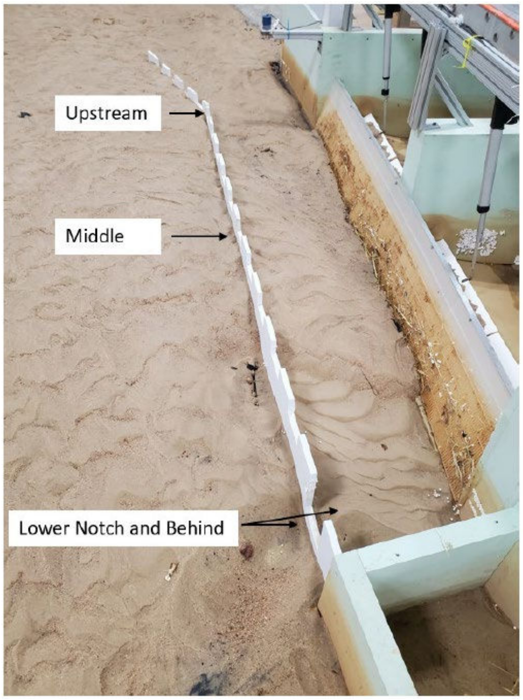
Data was collected and evaluated from the physical model to validate that the design meets fish passage criteria for depths and velocities (Table 5-4), and provides a range of fish passage opportunities through varied hydraulics of the ramp section. The physical model was considered the ideal tool to validate the design criteria, as high-resolution numerical modeling of the varied geometry of the channel and roughness would be computationally expensive and would be less representative of the roughness provided by the baffles, larger boulders and bed surface roughness installed throughout the Hardened Ramp.

5.3.1.1 Point Data Collection and Analysis

The USBR (2022) took direct measurements of velocity and depth on the model along the centerline of the low flow channel for velocities at 200 ft^3/s, 300 ft^3/s and 400 ft^3/s. The updated rock placement (Trial #3) reduced velocities to near or below the 8 ft/s criteria and raised depths above 1 ft for adult steelhead passage. Criteria for Pacific lamprey passage were not directly evaluated, but areas with acceptable velocity criteria and substrate for attachment were on the edges of most, if not all model runs, with sufficient depths and connectivity.

In response to concerns raised by NMFS and CFDW regarding connectivity of the low flow section upstream above the intake, and passage through the ramp inlet at high flows, additional model runs and data collection were undertaken. Measurements at the hardened ramp entrance were taken after the addition of rocks to increase roughness for a flow of 3,000 ft^3/s, which resulted in velocities below 8 ft/s and depths over 1 ft (Table 5-3).

USBR (2022) collected data along the training wall for a range of river flows between 100 ft^3/s and 1,500 ft^3/s, Figure 5-15. The velocities along the training wall were consistently at 3 ft/s or under, the depths are over 1 ft. Measurements were taken on both sides of the lowered notch for the lower flows (100 ft^3/s to 800 ft^3/s). The difference in water level was under 1 ft, with the differential only getting above one foot at 1,500 ft^3/s.



River Flow	Castle Training Wall Location	Velocity	Depth	Differential
ft^3/s	-	ft/s	ft	ft
1,500	*Upstream*	1.9	2.6	-
	Middle	2.8	3.8	-
	Lower Notch	2.8	5.8	-1.5
	Behind Lower Notch	0.4	7.2	
800	*Upstream*	2.3	2.0	-
	Middle	2.4	3.1	-
	Lower Notch	1.7	6.5	0.6
	Behind Lower Notch	0.6	5.9	
400	*Upstream*	2.8	0.7	-
	Middle	2.7	2.0	-
	Lower Notch	1.8	5.2	0.2
	Behind Lower Notch	0.4	5.0	
200	*Upstream**	N/A	N/A	-
	Middle	2.0	1.1	-
	Lower Notch	2.2	4.7	0.5
	Behind Lower Notch	0.6	4.2	
100	*Upstream**	N/A	N/A	-
	*Middle**	N/A	N/A	-
	Lower Notch	3.0	4.2	0.8
	Behind Lower Notch	1.1	3.4	

** No flow on the hardened ramp side of the castle training wall at this point.*

Figure 5-15 Data collected near the training wall from the 1:24 model (USBR).

5.3.1.2 PIV Data Collection and Analysis

The previous preliminary CFD modeling (NHC and GEI, 2019)(NHC and GEI, 2019)(NHC and GEI, 2019)(NHC and GEI, 2019) (NHC, 2020) showed potential fish passage pathways through the baffles. The baseline testing confirmed the hydraulics in the baffle sections were adequate and no changes were made to these sections in the subsequent physical modeling effort. In response to NMFS and CDFW, velocity mapping on the low flow and baffled section was completed to evaluate the possible pathways of volitional passage through the ramp over a range of discharges for strong and weak swimming fish using the criteria in Table 5-4. The USBR ran a series of tests for discharges of 100, 200, 300, 400, 500 and 600 ft^3/s where they collected PIV data which mapped surface velocity and collected point depth measurements which they provide in Figures 133 to 150 of their report (USBR, 2022).

The results provided by the USBR (2022) for 400 ft^3/s for the full ramp are reproduced below in Figure 5-16 to Figure 5-18 for reference.

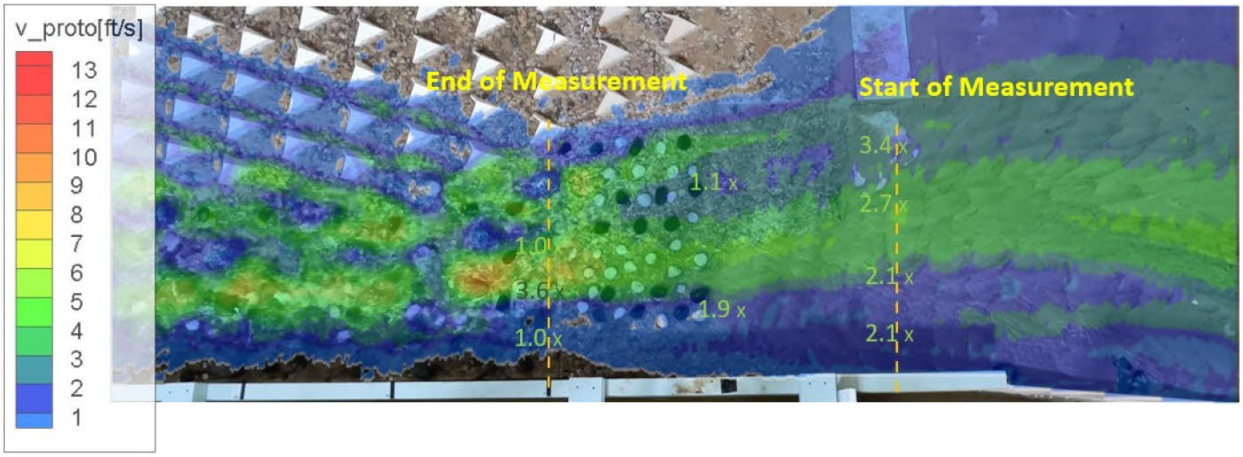


Figure 5-16 **Upstream hardened ramp configuration for fish passage at 400 ft³/s. Depths are shown in prototype feet with depths less than 1 ft denoted in red. Velocities less than 0.1 ft/s have been made transparent. Depths were shallow at the left-most gate of the hardened ramp due to separation of flow around the edge of the flushing channel (Image and Caption from USBR 2022)**

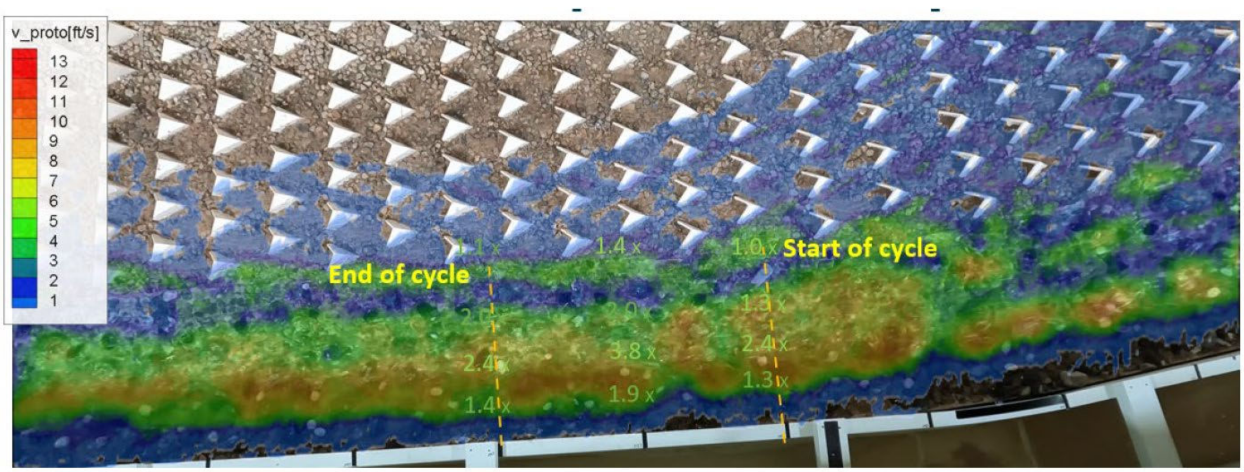


Figure 5-17 **Mid-ramp hardened ramp configuration for fish passage at 400 ft³/s. Depths are shown in prototype feet with depths less than 1 ft denoted in red. Velocities less than 0.1 ft/s have been made transparent.**

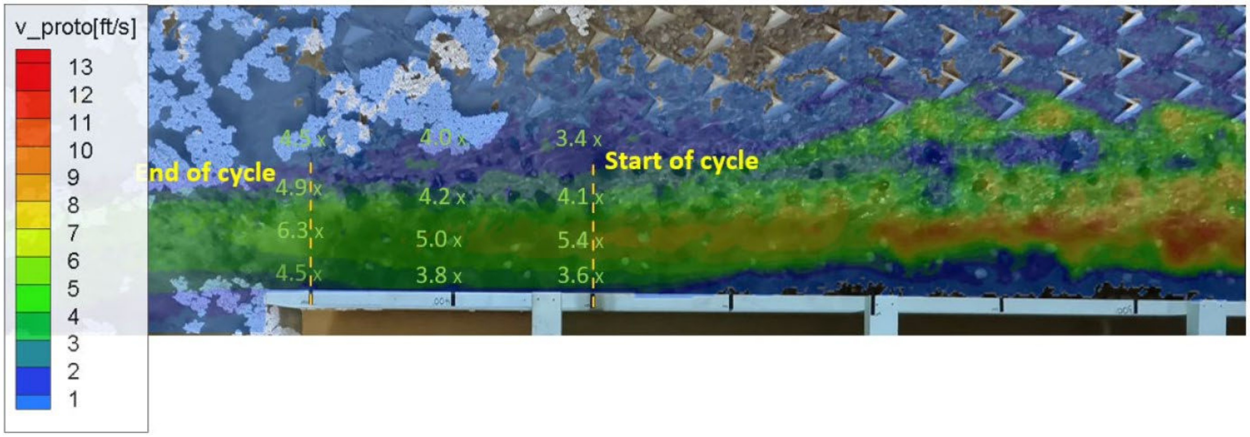


Figure 5-18 Downstream hardened ramp configuration for fish passage at 400 ft^3/s. Depths are shown in prototype feet with depths less than 1 ft denoted in red. Velocities less than 0.1 ft/s have been made transparent.

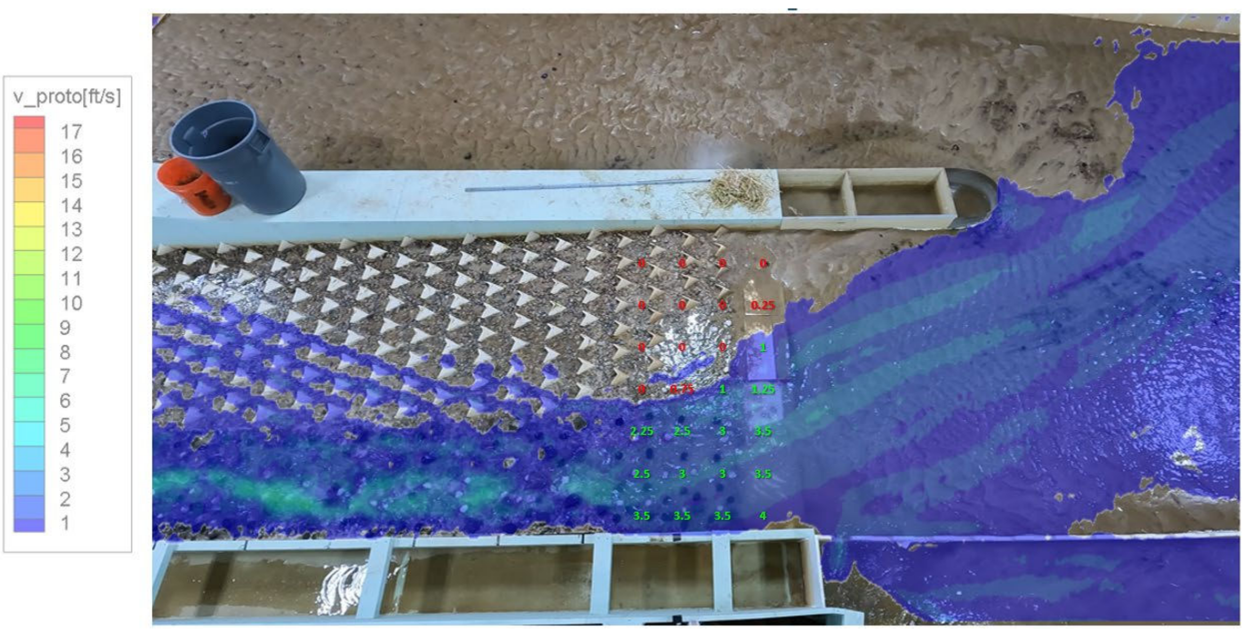


Figure 5-19 Upstream hardened ramp configuration with hardened ramp discharge of 320 ft^3/s. Depths are shown in prototype feet with depths less than 1 ft (including dry) denoted in red. Velocities less than 0.1 ft/s have been made transparent.

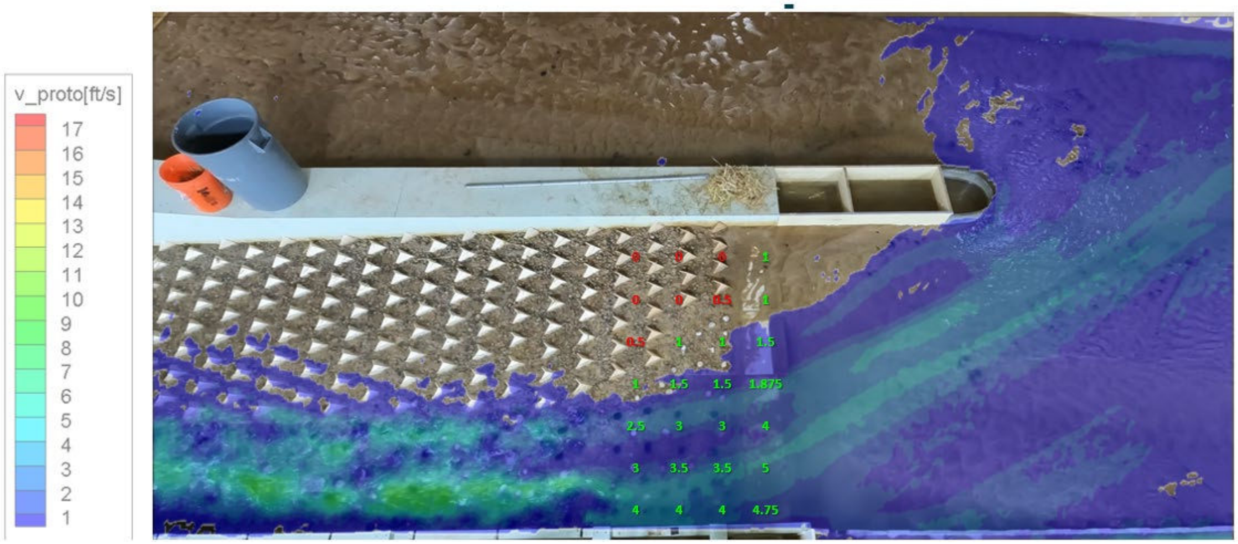


Figure 5-20 **Upstream hardened ramp configuration with hardened ramp discharge of 500 ft^3/s. Depths are shown in prototype feet with depths less than 1 ft (including dry) denoted in red. Velocities less than 0.1 ft/s have been made transparent.**

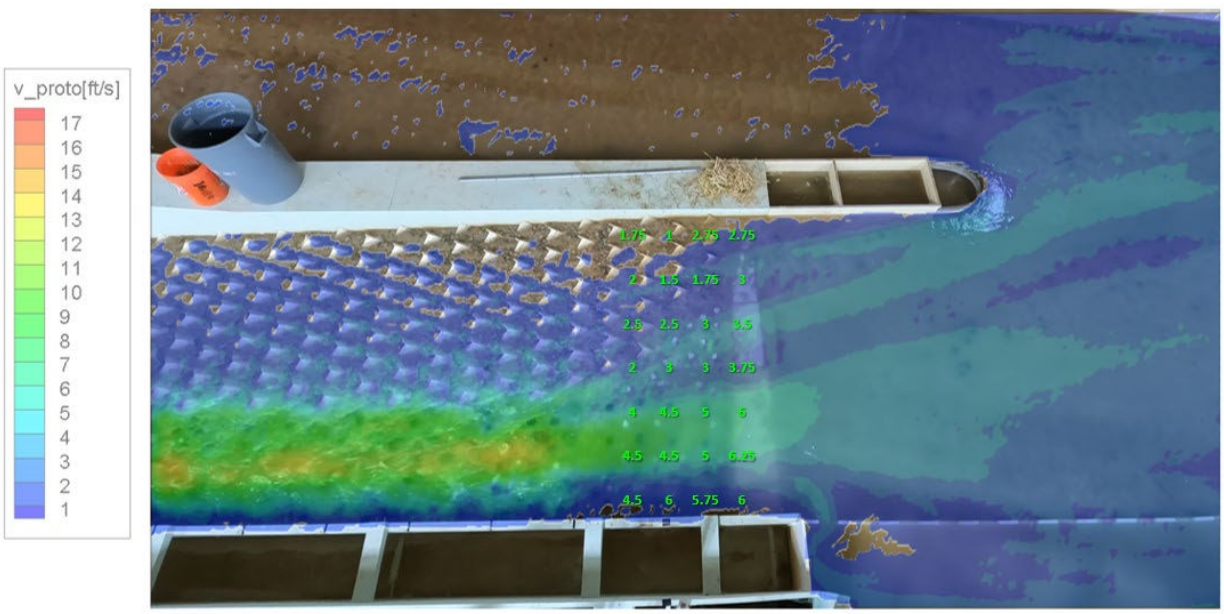


Figure 5-21 **Upstream hardened ramp configuration with hardened ramp discharge of 700 ft^3/s. Depths are shown in prototype feet with depths less than 1 ft (including dry) denoted in red. Velocities less than 0.1 ft/s have been made transparent.**

5.3.2 Summary

The results presented in Figure 5-16 through Figure 5-18, the addition of the structured rock surface successfully lowered velocities to under 8 ft/s in the low flow section of the hardened ramp during flow events less than 600 ft^3/s (USBR, 2022). The baffled portion of the hardened ramp started activation at 300 ft^3/s but did not meet the 1 ft minimum depth criteria for strong swimmers until approximately 400 ft^3/s. As the discharges increased the areas meeting criteria shifted from the center channel thalweg outwards into the baffles and left ramp wall.

During the stress and operational testing period (USBR, 2023), NHC worked with USBR staff to capture some additional data for varying upstream bed conditions and discharges on the model to observe additional potential ramp scenarios. The testing included hardened ramp discharges of 700 ft^3/s, 500 ft^3/s and 350 ft^3/s, with total river discharge and diversion discharge adjusted to achieve these hardened ramp flows. These tests were conducted following the upstream channel alignment variation testing, where the upstream thalweg was aligned to the right, and the thalweg alignment was not reset to the left prior to collecting the hardened ramp hydraulic data. A velocity field was measured using particle image velocimetry (PIV), and point depths at the crest of the hardened ramp were collected on a 12 ft x 12 ft prototype grid of sampling locations. The upstream channel alignment was found to have minimal impact on fish passage suitability, and the revised inlet rock placement and geometry mitigated adverse hydraulics observed in earlier model testing.

For the three tests, continuous pathways suitable for volitional fish passage for strong swimmers, per criteria (Table 5-4), were maintained. At 320 ft^3/s (Figure 5-19) and 500 ft^3/s (Figure 5-20) Hardened Ramp discharges, suitable depths and velocities are present along the left half of the hardened ramp, primarily in the low flow channel. For the 700 ft^3/s ramp discharge (Figure 5-21), depths exceeding 1 ft are present across the full width of the hardened ramp, with velocities at or below 8 ft/s through the baffled section of the ramp.

USBR (2022) provided PIV analysis for discharges of 100 to 600 ft^3/s, which were used to evaluate fish passage for strong and weak-swimming fish at the Hardened Ramp. Accordingly, the 100 and 200 ft^3/s results were used to evaluate weak swimming fish passage as the upper bound for passage is 270 ft^3/s. Hardened Ramp flows less than 100 ft^3/s were not modelled due to issues of model scale, but preliminary assessment of the low flow channel indicates that strong swimming fish criteria can be met at discharges down to 45 ft^3/s and weak swimming criteria can be met at further reduced discharges, with further detailed analysis of the low flow channel thalweg required for flows less than 10 ft^3/s.

In both sets of PIV results, Figures 145 through 150, continuous areas of the ramp meeting criteria (Table 5-4) are present along the right side of the wetted are at the ramp entrance, channel and exit. Continuity extends above the ramp into the river channel. NHC notes that this hydraulic continuity and connectivity maintained in the Hardened Ramp for weak swimming fish until flows of 500 to 600 ft^3/s, when the right channel edge contact the bullnose and surface velocities increased above 1 ft/s. Passage may be available at depth, but this was not evaluated in the PIV analysis.

The regions of passable hydraulics for strong swimmers remained consistent and unbroken, shifting laterally as flows engaged additional channel roughness as flows increase from 100 to 600 ft^3/s in USBR (2022) Figures 133 through 150. The right side of the channel extends across the baffled section with increasing depth and band of suitable velocity shifting laterally with increasing flow. The low flow/baffle section transition was evaluated and again the hydraulics were adequate and continuous for fish passage.

On the left bank, a small but significant area of suitable hydraulics for strong swimming fish is provided up to 700 ft^3/s (Figure 5-21), which is also the threshold where minimum depth criteria are fully met across the right side baffled section and velocity becomes the limiting factor for strong swimming fish. These model results are consistent with the intended design basis of the Hardened Ramp and support the previous CFD results provided in the NHC HBOD (2019). The results indicate that the Hardened Ramp will meet both strong and weak swimming fish passage criteria representing anadromous steelhead, lamprey and resident native fish through the range of defined fish passage flows.

6 Diversion Intake

6.1 Diversion Objectives

The diversion must operate over a range of river stages and flows and at water levels that provide fish passage on the Hardened Ramp. A range of diversion flows may be desired for a given river flow and stage. The diversion must be designed to minimize sediment intrusion and associated maintenance requirements, accommodate maintenance access when required, and provide safe downstream passage for fish that enter the intake.

Operation of the diversion through the entire target fish passage design flow range (45 to 6,000 ft^3/s) is desired, and it potentially could be operated at even higher flows. Because the hardened ramp has passive flow control at the crest of the ramp during fish migration periods to facilitate fish passage and sediment transport (i.e., no operation of the ramp crest gates is proposed to form a head pond) the ability to divert flow is linked to the hydraulic characteristics of the ramp.

UWCD has requested that the diversion be designed for a capacity of 750 ft^3/s (existing capacity is 375 ft^3/s), and that the facility be designed to operate using the existing gravity canal system downstream, considering planned system improvements (NHC, 2016c). The capacity of the improved canal system was previously analyzed using hydraulic modeling tools and the rating curve for the preferred alternative at the upstream end of the canal is used as the tailwater condition for assessing operation of the diversion. The increase in capacity will require addition of a parallel fish screen bay and new fish bypass and monitoring facilities. The eight crest gates, two downstream canal gates and two screen bays provide maximum operational flexibility to allow for water to be diverted and to meet acceptable fish screen criteria requirements.

6.2 Intake Configuration

The Diversion Intake shown in Appendix A have the following characteristics:

- Diversion sill length – 102 feet
- Diversion sill elevation – varies between 155 ft and 156.5 ft
- Flow control gates
  - Intake Crest Gates: 4 – 11 ft W x 8.5 ft H crest gates with a floor elevation of 156.5 ft at diversion intake
  - Intake Crest Gates: 4 – 11 ft W x 10 ft H crest gates with a floor elevation of 155 ft at diversion intake
  - Isolation Gates: 8 – 5.75 ft W x 9 ft H gates with a bottom elevation of 152.5 ft
  - Bottom Sluice Gates: 8 – 5.75 ft W x 5 ft H gates with a bottom elevation of 146 ft
  - Canal Gates: 4 – 9 ft W x 8 ft H regulating slide gates at screen bay outlet and head of canal
- Canal width – 22 feet (2 bays).

The diversion extracts water from the Santa Clara River through a 102 ft wide intake with eight 11 ft wide crest gates, located on the left bank perpendicular to the primary flow direction. Flows diverted into the intake include both the canal diversion flows and the screen bay fish bypass flows. The diversion entrance is kept clean by the Training Wall (Section 6.3) and Flushing Channel (Section 6.3). Details on sediment management are given in Section 7.

Water first enters through the trash rack (Section 6.4) which prohibits material above 6-inches from entering the diversion. Water surface elevations and in turn discharge are controlled through the eight upstream crest gates (Section 6.5). The water then travels through eight desander channel/bays (Section 6.6) where sediment that entered through the trash racks may deposit. The desander isolation and bottom sluice gates (Section 6.7) are located at the end of the desander channel. These can either be operated with the isolation gates open and the bottom sluice gate down, where water continues into the screen bay or for the isolation gates to be closed and the bottom sluice to be open during desander sluicing operations (Section 7.5). The downstream canal gates (Section 6.7) are utilized to set water surface elevations through the screen bays.

Coordinated control of both the regulating canal slide gates and the intake crest gates is required to control both the diversion flow and water surface elevation/velocity in the screen bay (Section 6.9). A portion of the total diversion flow passes the fish screens and is accelerated to the fish bypass piping and routed to a fish monitoring station. Fish and bypass flows are released downstream in the tailwater of the Hardened Ramp. The fish screen system is described in more detail in Section 6.9.

6.3 Flushing Channel, Training Wall and Apron

The flushing channel is designed to assist in removing sediment that builds up in front of the diversion intake. The training wall and upstream apron was designed to enhance the efficiency of the flushing channel to remove sediment. The dimensions and design development of these features were discussed in Section 1.1.1, with a plan and profile view on Figure 4-15.

To further ensure low flow connections between the Hardened Ramp and Diversion Intake, a "notch" was included at the downstream end of the training wall with an invert elevation of 156.5 ft. The notch was designed to have a removable bulkhead for operational flexibility, see Section 10.4. The impacts of these features on sedimentation are discussed in Section 7.

6.4 Trash Rack

The trash rack is located just outside of the intake crest gates and consists of a supported bar rack with a nominal 6-inch clear spacing. The current trash rack is "self-cleaning" and uses a chain system to hook and drag debris up the face of the rack, Figure 6-1. The rack extends 6 ft above the top of the diversion pad, Figure 6-2. Debris is deposited behind the rack, where it can then be removed using an excavator or other machinery. For the new Diversion Intake, a similar system is expected to be implemented, and will be explored further in detailed design. The rack will extend up to El. 189.0 ft, 6 ft above the floor elevation of 183 ft and extend up from a sill elevation of 155.0 ft at an angle of 10 degrees.

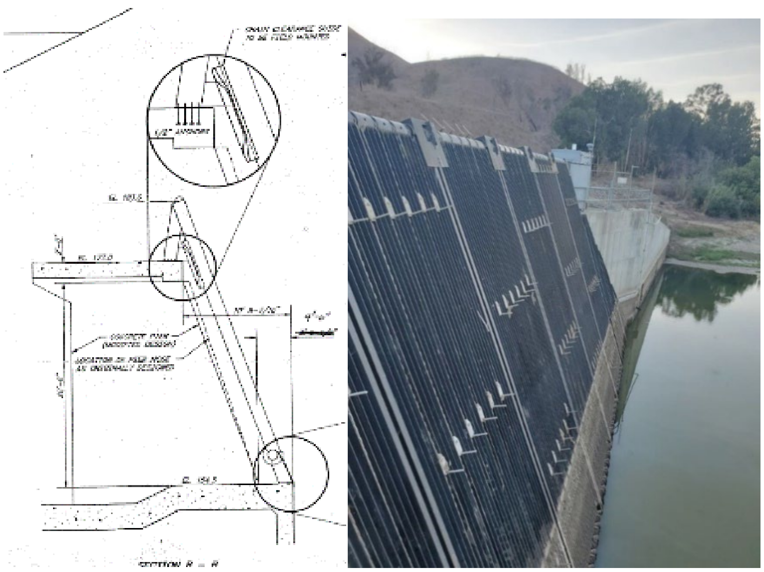


Figure 6-1 Existing Trash Rack Face Detail and Field Image.

Figure 6-2 Trash Rack, Maintenance Deck and Deposits.

6.5 Intake Crest Gates

The intake crest gates are designed to operate independently and maintain consistent diversion flows over a range of upstream river water surface elevations. Flow into each of the eight diversion bays will be controlled by an 11-ft wide crest gate. At fully open, the crest gates will lie flat with the inlet sill, corresponding to El. 156.5 ft for gates 1 to 4 (left side looking downstream) and El. 155.0 ft for gates 5 to 8 (right side looking downstream). At fully closed, the gates will raise up to elevation 165.0 ft, which would allow individual diversion bays to be closed during a river discharge of 6,000 ft^3/s. Based on these design elevations, the gate heights will be 8.5 ft and 10.0 ft high when fully raised in bays 1 to 4 and 5 to 8, respectively. Depending on upstream bed morphology and approach flow conditions, it is possible that diversion of the 750 ft^3/s design discharge could be achieved during higher river flows of up to about 10,000 ft^3/s. A conceptual drawing of the inlet crest gates and upstream trash rack are shown in Figure 6-3.

Spillway gates manufactured by Obermeyer Hydro, Inc. and Waterman are currently being considered for the inlet crest gates. Obermeyer spillway gates are hinged at the bottom and opened using inflatable air bladders that can be fully submerged during operation. When deflated, the bladder and gate collapse down into an approximately 6-inch deep notch in the channel sill. Machinery used to inflate the bladders and operate the gates will be located on the service deck over the intake that is set at an elevation of 183.0 ft, as shown in the plans (Appendix A). A design conversation with Obermeyer in 2022 indicated that the spillway gates should be capable of the fine adjustments necessary to automatically maintain target inflows into each of the diversion bays under typical hydraulic conditions.

Waterman tilting weir gates are also hinged at the upstream end and bolted to the channel floor. They are controlled by a cable drum that spans the inlet bay and is connected to each end of the gate by metal cables. The machinery used to operate the drums would also be placed on the overhead service deck at El. 183 feet. A design conversation with Waterman in 2022 indicated that the tilting weir gates could be automated to maintain constant design discharges as required.

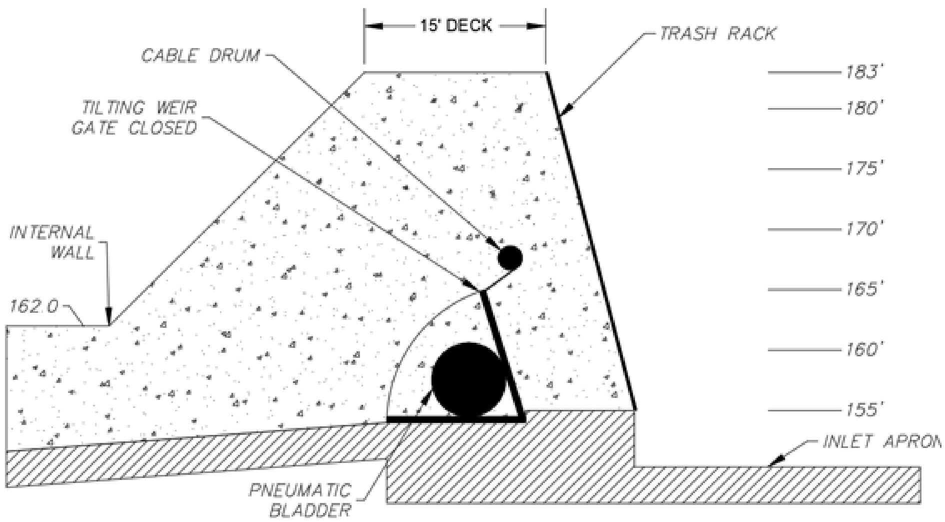


Figure 6-3 Conceptual drawing of inlet crest gates and trash rack.

6.6 Diversion Channel/Bays

Once the water passes the crest gates, it enters the diversion channel which consists of eight steep long channels that vary in slope from 3.25% to 3.82%. Due to the curvature of the structure, the bays vary in length, but average around 260-ft. The channel widths narrow from 11 ft to 5.75 ft moving downstream. The wall elevation at the entrance is set to the 100-year flood elevation of the facility, 183 ft. The walls stay at an elevation of 183 ft for the outside walls creating a physical barrier. For the interior walls the elevation quickly lowers to 162 ft where it stays until it transitions back up to 183 ft for the isolation gates at the desander exit. A plan view of the diversion channel is shown in Figure 6-4 with typical sections near the upstream, Figure 6-5, and downstream extent, Figure 6-6.

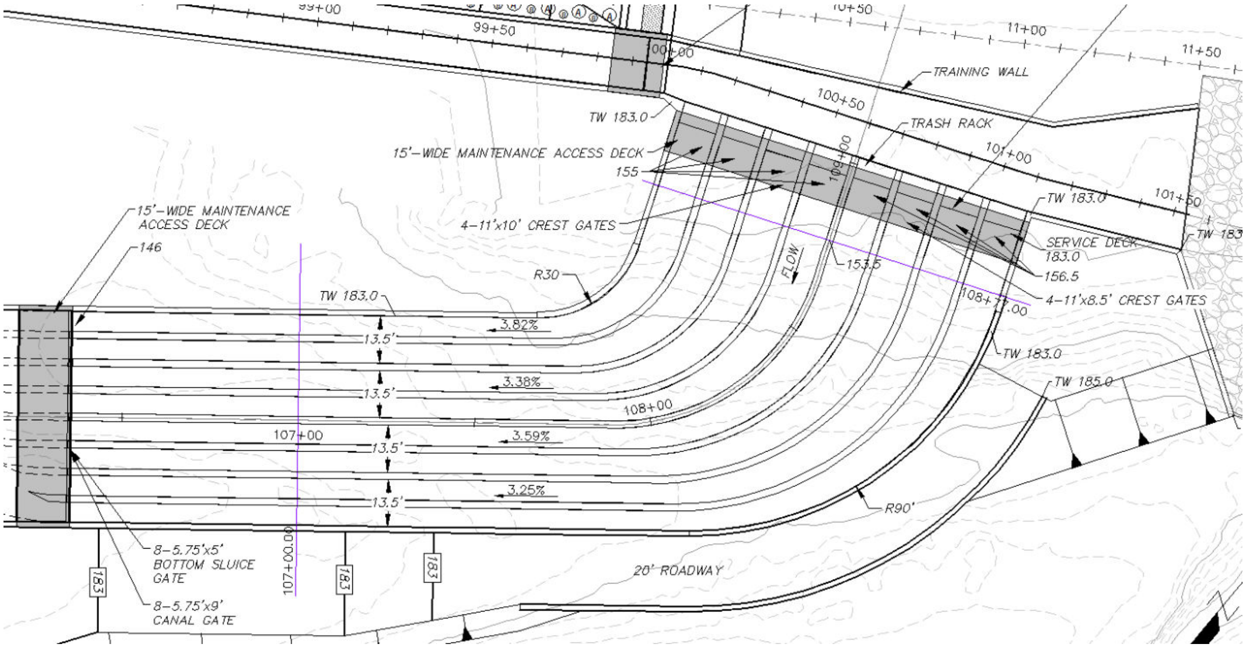


Figure 6-4 Plan view of diversion channel.

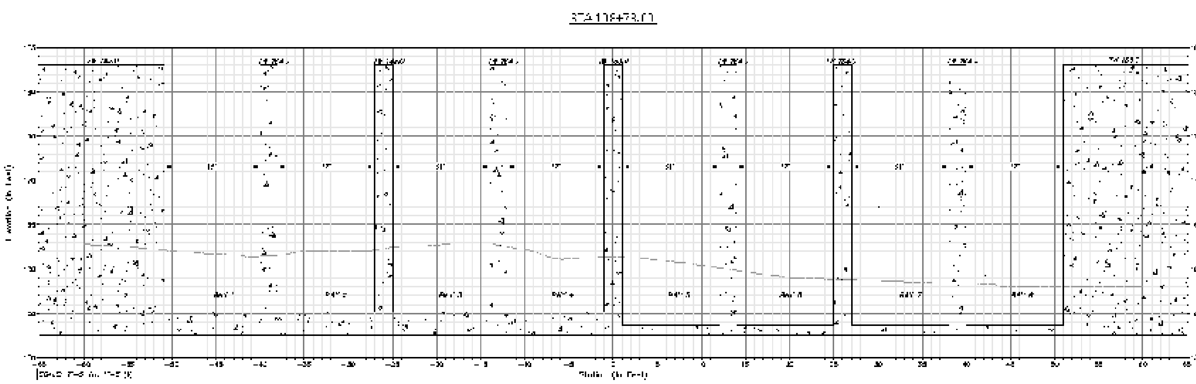


Figure 6-5 Upstream typical section of diversion channel.

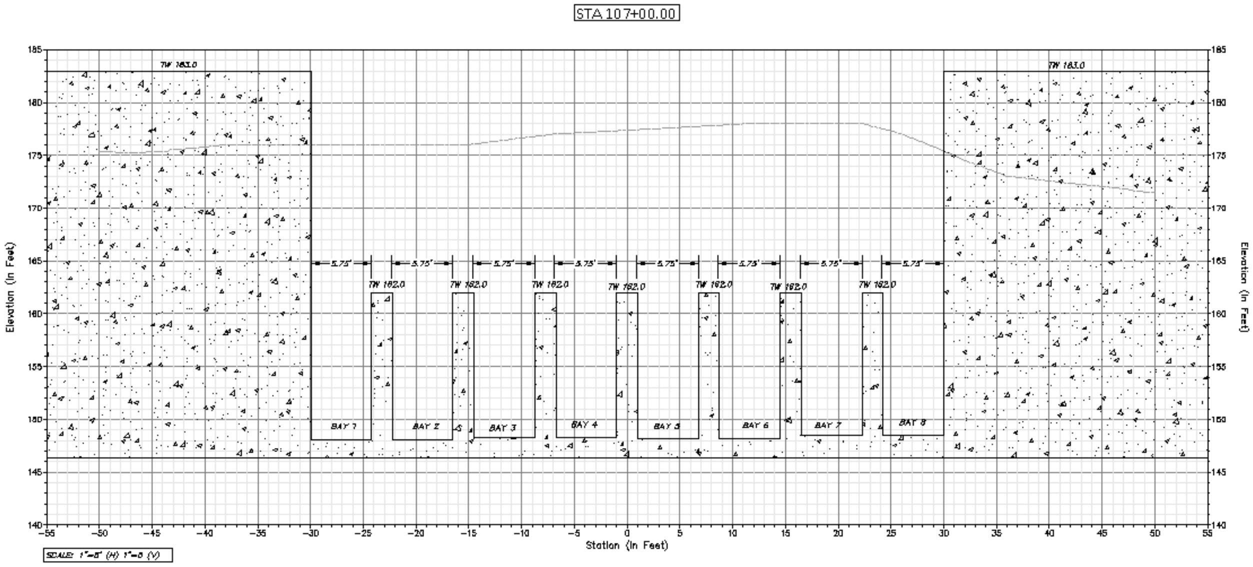


Figure 6-6 Downstream Typical Section of Diversion Channel.

6.7 Desander Isolation and Bottom Sluice Gates

The desander has two sets of eight gates that control the operation of the desander as well as provide hydraulic isolation of the intake during flood events. The eight desander isolation gates sit above eight bottom sluice gates The eight isolation gates and sluice gates are located at the downstream end of the desander bays at the entrance to the screen bay (Figure 6-4). The bottom sluice gates allow flow to enter the sluice channels of the desander system to remove sediment accumulations from the desander bays. The width of the desander gates match the 5.75 ft width of the diversion bays and are set at an invert El. 146 ft. The gate openings are 5 ft high with soffit elevations of 151 ft to match the size of the downstream sluice channel.

The bottom sluice gates are designed as slide gates that are either fully closed to block off the desander during normal diversion operation, or fully open when sluicing the desander. Each slide gate would be independently operated to sluice the diversion bays separately as required. The gates would be controlled by vertical rods attached to the top of the gates and extending up to the maintenance deck at El. 183 ft, where the machinery for operating the gates would be located. A conceptual drawing of the slide gates for the desander system are shown in Figure 6-7.

Slide gates manufactured by Waterman are being considered for use as the sluice gates. They require the installation of slide tracks in the concrete bay walls of about 4 inches deep per side. Since the eight gates are in a line across the bays, the 2 ft thick internal walls will have adjacent slide tracks cut into both sides of each wall. The Waterman slide gates will be designed to operate with loads from both hydrostatic pressure and sediment deposition on the upstream side.

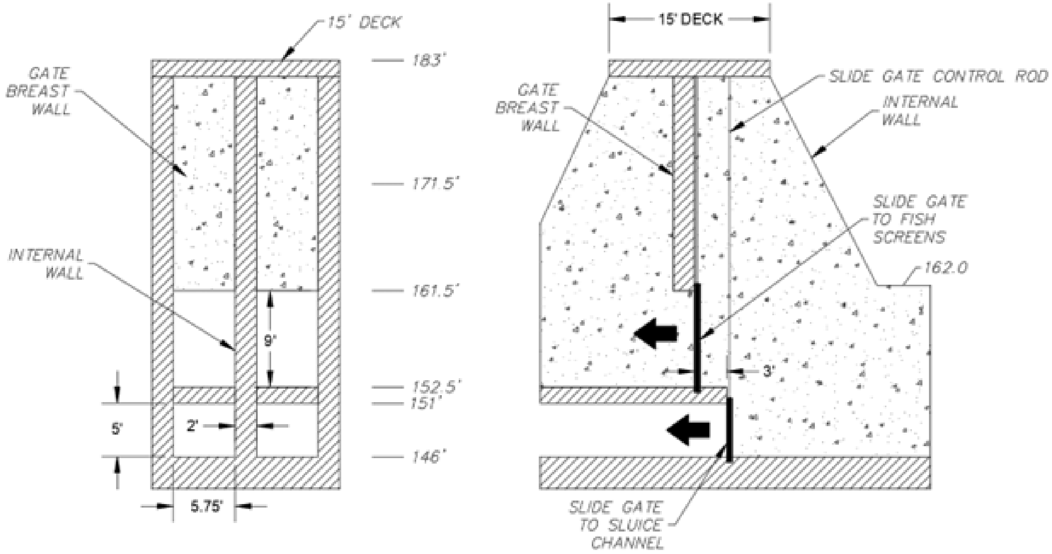


Figure 6-7 Conceptual drawing of desander isolation and bottom slide gates.

The desander isolation gates are designed to isolate the screen bay during sluicing of the upstream desander bay or during flood flows to protect the fish screens from sediment and debris. Each diversion bay would terminate at a 5.75 ft wide canal gate, which would remain open during normal operation of the diversion. The invert of the gates would match the invert of the screen bay entrance (El. 152.5 ft) and be 9 ft high with a soffit elevation of 161.5 ft. This top elevation would provide an additional 1.5 ft of clearance above the top elevation of the fish screens (El 160.0 ft). The isolation gates are designed with a gate breast wall that extends to the FCL of El. 183 ft (100-year design water surface elevation with 3 ft freeboard). The gates tie into walls on each side of the desander channel to provide the desired flood protection.

Similar to the sluice gates, the isolation gates would slide in vertical tracks cut into the interior walls of the diversion bays and be operated by control rods connected to machinery set on the maintenance deck. A breast wall would be constructed just downstream of the canal gates to prevent flow from overtopping the gates during flood flows. The isolation gates would be designed to remain closed during flood flows and reopen only once normal operating conditions return. The isolation gates, when closed, along with the outside walls provide a barrier at an elevation of 183 ft to provide protection from flood flows. Slide gates manufactured by Waterman are under consideration for use as the canal gates. As with the sluice gates, this would require 4-inch slide tracks in each side the diversion bay walls to guide the movement of the gates.

6.8 Diversion Dam

6.8.1 Dam Crest Modifications

Although the diversion dam is not a part of the intake it is described here as it relates to downstream fish passage like the screen bay fish bypass. The existing diversion dam is a static hydraulic structure that passes flows according to the hydraulic flow split between the diversion intake, hardened ramp and dam. The dam provides the flow conveyance to allow fish passage in the hardened ramp to river flows of 6,000 ft^3/s, as well as safe conveyance of larger extreme flood flows. In order to define the flow split between the hardened ramp and the dam, a notch was designed to control the elevation that initiates spills over the dam. The design includes a static 100 ft wide crest notch at El. 161.5 ft which is 0.5 ft below the existing dam crest. These dimensions were developed in an earlier NHC memorandum (NHC, June 2021).

The DDR (NHC, 2020) found that the most promising diversion dam crest modification was a wide fixed notch with shaping of a short ogee-type spillway to provide a controlled spill at the upper end of the fish passage design flow range and to improve safe downstream passage for outmigrants. This option has the advantage of not requiring additional operable gates in the river and not substantially constraining diversions outside of the fish migration season for the most frequent flow range.

An ogee section was developed to be shaped into the smoothed RCC surface optimized for a high capacity such that all design flows stay attached to the surface (see example diagram below, Figure 6-8).

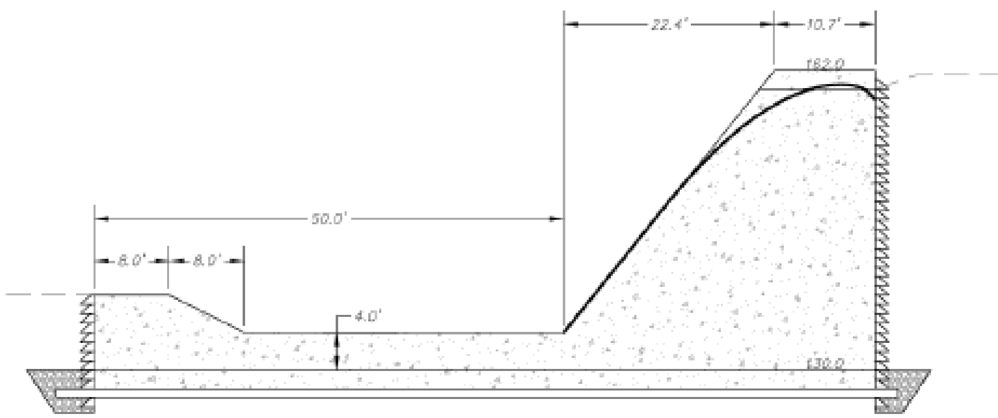


Figure 6-8 Potential Ogee section at the dam crest.

6.8.2 Juvenile Passage during Dam Spills

There is potential for juveniles to pass over the crest of the dam during periods when river flows exceed the combined flows in the Diversion, Hardened Ramp, and any sluicing or auxiliary flow features. NMFS and CDFW expressed concern over the possible passage of smolts over the thin weir flows on the wide dam crest and dam face. UWCD plans to resurface the downstream face of the dam, and this will reduce the roughness of this surface, which presently has an irregular stepped profile associated with the RCC construction method and subsequent wear (Figure 6-9).

Figure 6-9 Existing RCC Dam Face (September 2021).

In addition to the planned surfacing, the proposed notch will increase depth of flows over the dam during initial spills, reducing the likelihood of fish contact with the weir crest surface. The ogee shape and downstream face of the notch will be smooth concrete. Existing tailwater elevations reduce the water-to-water drop below current threshold-of-effects criteria and a relatively deep stilling basin provides a cushion pool to reduce the likelihood of impact and abrasion of smolts passing over the dam during spill flows. When water spills over the notch, tailwater levels are expected to be above El. 142 ft.

6.8.3 Nuisance Attraction Flows

The flow splits between the ramp and the dam crest, and how the spill is released over the crest influences attraction of upstream adult migrants to the ramp. The spill flow could potentially distract and lead fish towards the dam and away from the Hardened Ramp, causing delay in passage or reducing opportunity to migrate upstream. Both the relative magnitude of the discharges (flow split) and the hydraulic characteristics, influenced by the notch depth and width, are factors in attraction and distraction flow characteristics. With the design invert elevations, the hardened ramp is preferentially the first structure to water up and passes the initial 1,200 ft^3/s of discharge as required by the Amicus Brief (NMFS, 2018).

The tailrace hydraulics were first examined using a HEC-RAS 2D Model (NHC, 2021), and results showed that at lower flows, the discharge intensity provided by the Hardened Ramp is significantly greater than the spill flows and nuisance distraction to the dam is limited. For downstream river flows below 2,175 ft^3/s (upstream river flow up to nearly 3,000 ft^3/s depending on diversion rate), distraction flows do not appear to be significant compared to ramp flows. As crest discharges increase with increasing river flows, distraction flow intensity increases but the Hardened Ramp discharge intensity remains higher.

The physical model supported the 2D model results, with the ramp discharges visibly much greater in intensity than the dam flows. Relative to the hardened ramp, the discharge intensities of the flows from the dam crest are much less due to the large cross sectional area of the river relative to the hardened ramp. The stilling basin of the dam created a classic "roller" hydraulic in the tailwater which tended to reduce the downstream-directed flow momentum and energy (Figure 6-10). The hydraulic signal in terms of water velocity, turbulence and noise is stronger at the Hardened Ramp, which is intended to cue volitional upstream passage of fish in the ramp.

Figure 6-10 Diversion Dam tailrace hydraulic on 1:24 physical model.

6.9 Fish Screen and Bypass

The single fish screen currently installed in the Freeman diversion services operations for diversion flows up to 375 ft^3/s. The screen operates with tailwater control, using the current downstream canal gate to control water levels in the screen bay. However, diversion flows to the screen rely on the existing upstream regulating gates to generate large head losses at high river water levels to control both flows and water levels into the screen bay. The turbulence resulting from diversion flows passing below the undershot gate create poor approach hydraulics on the screen itself with surging and jetting. The installed fish screen does not meet current NMFS standards and an increase in diversion rate is not possible without increasing the screen area.

6.9.1 Fish Screen Design Development

Fish screen systems considered in the NHC design to date have included the re-use of the existing screen bay, a V-screen configuration, and a parallel screen bay configuration. The parallel screen bay option was selected by NHC for use in preliminary designs. The earlier NHC 30% design and DDR proposed a dual screen bay design (Figure 6-11), geometrically similar to the current fish screen.

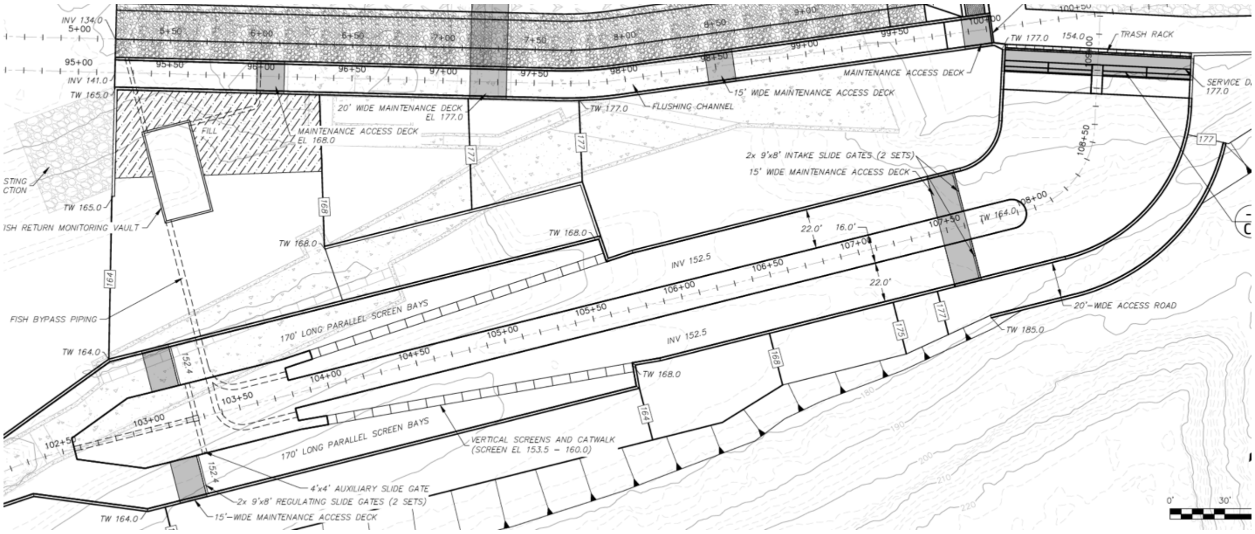


Figure 6-11 NHC 30% design dual bay fish screen (2021).

The screens were 170 ft long and would divert 375 ft^3/s per screen. The parallel screen bays would improve operational flexibility. One bay could be closed for maintenance while still diverting 375 ft^3/s through the second bay. The design provided sufficient screen area for full design diversion of 750 ft^3/s, and used separate headworks and regulating gates that allowed isolation and use of one screen to optimize hydraulics at minimum expected operational diversion flows and for mechanical sediment removal. Porosity plates were assumed to be used behind the screens to provide uniform distribution of flows and a small head loss through the screens.

Following design development of the desander, sluice channel and gate arrangements, the screen approach channel design had changed significantly. The upstream channel leading from the intake crest gates was now occupied by the desander and separating wall between the two screen bays had been shortened to the entrance to the fish screen bay (Figure 6-12). The individual screen bays were retained until further design development work and modelling.

In both designs, crest gates at the intakes control inflows to the screen to meet diversion and fish bypass requirements while the downstream regulating gates control screen bay water levels. At river water levels greater than the top of screen (El. 160 ft), the diversion crest gates control head losses and inflow to prevent overtopping of the screen bay. At river elevations less than El. 160 ft, the downstream regulating gates can control both inflow and screen bay water elevations. Description of the coordinated diversion operations are detailed in Section 10.

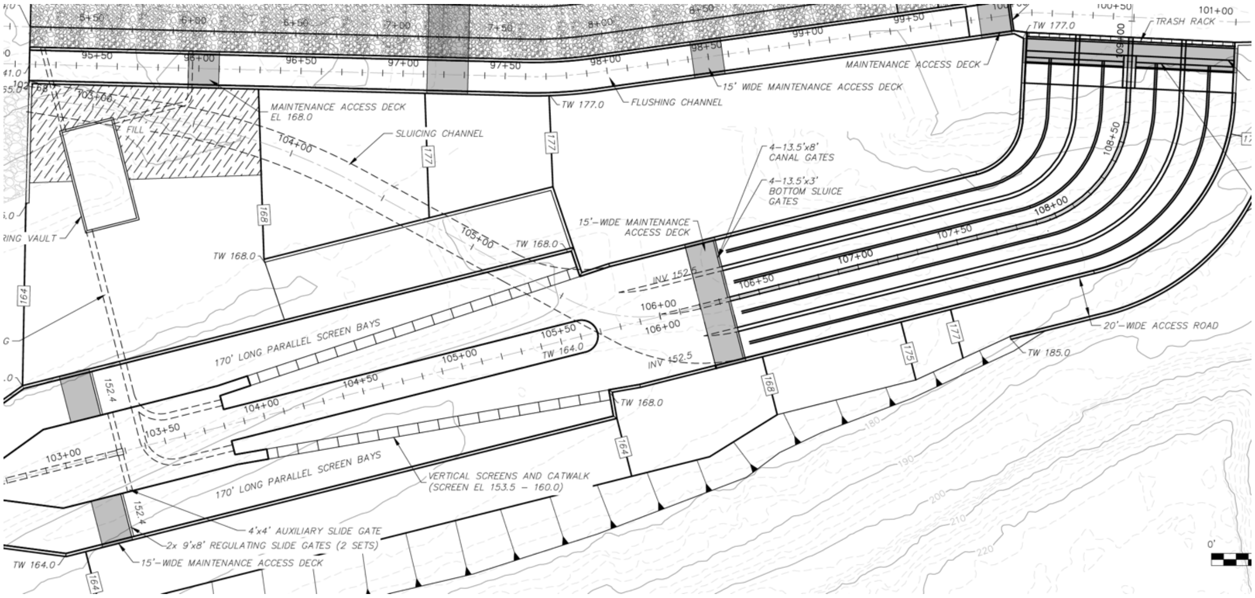


Figure 6-12 Interim NHC desander/design dual bay fish screen (2022).

The 2022 fish screen design was subject to initial CFD modeling during development of the desander to determine the minimum length of connecting channel required for a asymmetric desander outflow (only 75% width of desander diverting) to expand and become uniform entering the screen bays (Figure 6-13). This distance (red arrow) was used in the current design.

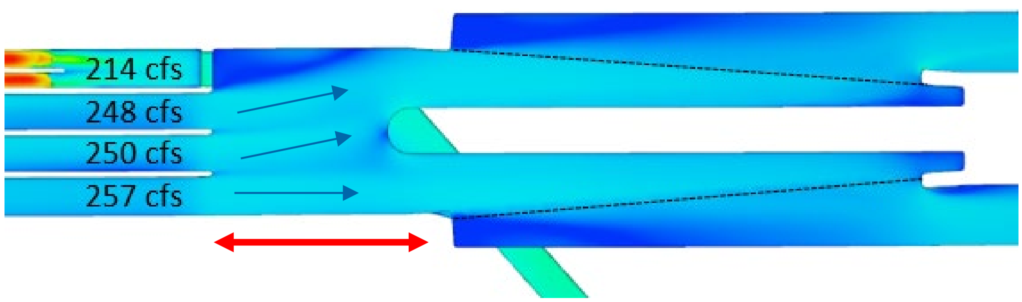


Figure 6-13 CFD modeling of asymmetric desander discharge to screen bays.

Hydraulic calculations and 1D spreadsheet were used to estimate required screen area, resulting approach and sweeping velocities, bypass flows and water levels for a range of diversion flows. Further hydraulic modeling, using the depth-averaged 2D model IBER (Bladé, 2005), was used to assess the 2022 design in terms of screen bay hydraulics where the screen and porosity plate system was simulated as a series of vertical openings or slats along the screen alignment to represent the nominal head loss. Both the diversion discharge and bypass flow were added to the model as boundary conditions. (Figure 6-14).

Hydraulics were non-ideal with significant flow separations and eddies formed off the bullnose feature separating the screens. The skew of the screens relative to the approach channel led to high hydraulic loading on the initial screen sections, and several recirculations were noted in abrupt transitions and dead corners. Geometries were too large or too small, which resulted in non-ideal hydraulics and velocity structure across the screen bay and along the fish screen. Examples of these are provided in Figure 6-14.

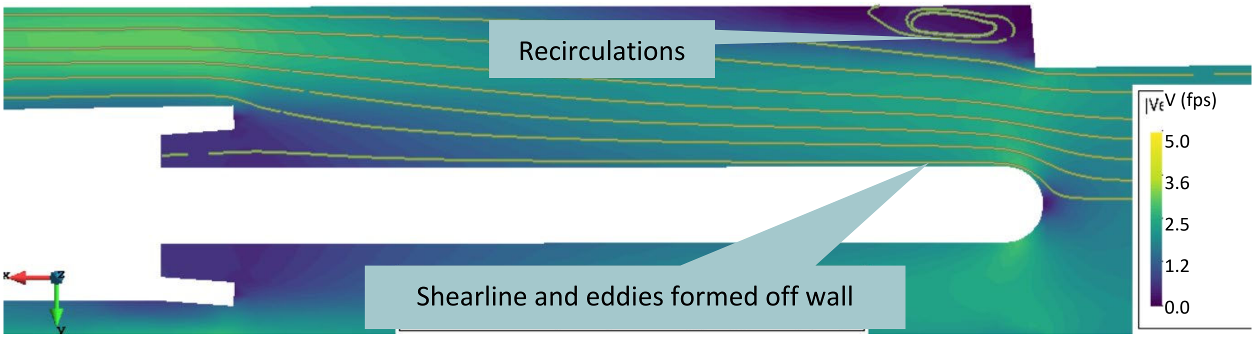


Figure 6-14 Depth-averaged 2D model of the 2022 design screen bay.

The fish screen and screen bay were modified to address several of the larger hydraulic issues – which is the current 2023 design provided in the drawings. These are a significant modification of the 30% and 2021 designs. The proposed design uses dual vertical screens in a single bay separated by a 2 ft concrete single wall. The layout has primary fish screens that are 170 ft long at a 7 degree angle to flow with a 15 ft long secondary dewatering screen in the fish collector (Figure 6-15).

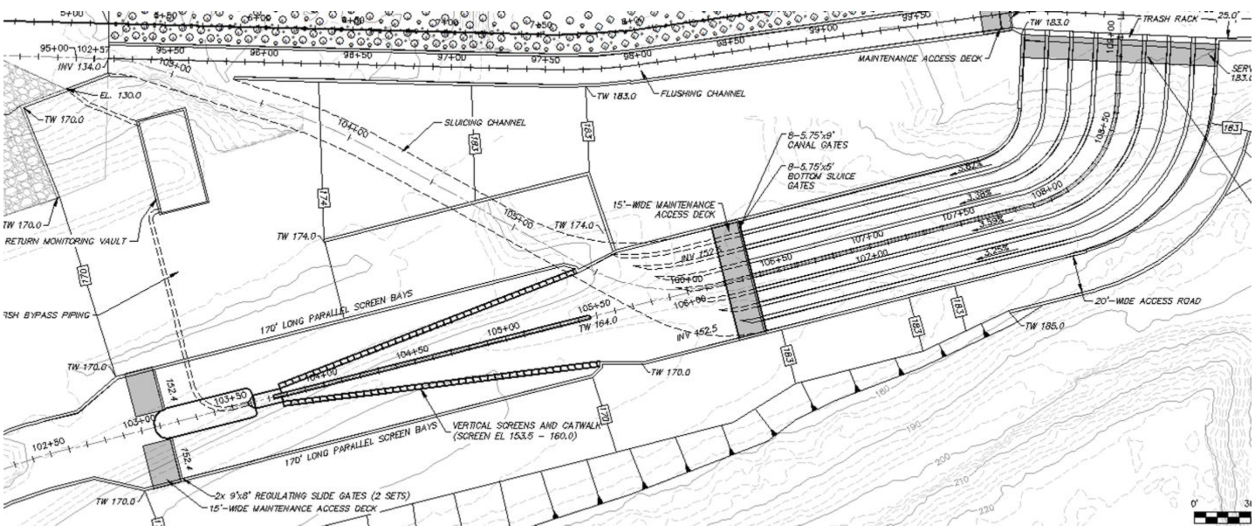


Figure 6-15 2023 Freeman Dam Hardened Ramp fish screen and bypass system.

The screen bay invert is at El. 152.5 ft with a 1 ft high sill at the bottom of the screens to provide space for a small accumulation of sediment without interference with screen cleaning equipment. The screen bay is approximately 60 ft wide, matching the approach channel from the desander which provides uniform approach hydraulics into the bay. Individual isolation and dewatering of screen bays is not expected as sedimentation into the design screen bay will be greatly reduced with use of the desander during diversion operations. This allowed for removal of the large bulkhead access pad between the screens and improvement in the approach hydraulics.

This configuration provides approximately 2,130 ft^2 of screen area. With the screen bays operating at a water surface El. 159.5 ft, resulting approach velocities will be less than 0.4 ft/s at a design maximum diversion of 750 ft^3/s. The effective screen area is managed by control of the screen porosity plates located behind the fish screen panels and the canal gates downstream of each screen. The secondary screen contains a floor ramp and outlet – a capture weir – which controls the total bypass flow ratio with changing diversion flow rates. The floor ramp decreases the cross-sectional area to provide flow acceleration and the capture weir provides a critical flow hydraulic that ensure fish transport downstream.

Like the earlier designs, water elevations in the screen bay are controlled to ± El. 159.0 ft through use of the canal regulating gates. Closure of the one set of canal gates behind a screen renders that bay hydraulically neutral, allowing smaller diversion flows to be processed with a single, reduced area screen.

6.9.2 Fish Screen and Bypass Objectives and Criteria

The operational objective of the fish screen and bypass system is to safely remove fish from the diversion flows and collect and transport them to the tailrace of the Hardened Ramp without injury or delay. The fish screening and bypass system follows guidance provided by NMFS (2022a) with the following features:

- Screen bay configuration – 60 ft wide x 200 ft long screen bay with dual angled 7° offset 170 ft long primary screens
- Screen sill at El. 153.5 ft and top of screen at El. 160.0 ft
- Nearly constant screen bay operating water level at ± El. 159.0 ft
- Primary effective fish screen area estimated at 2,130 square feet at maximum operating water level of El. 159.5 ft
- Screen approach and sweeping velocities to meet NMFS criteria over a range of diversion flow rates
- Parallel to the flow wedge wire fish screens meeting NMFS criteria of minimum open area (27%) and clear openings (0.069 in), design proposes 50% open area wedge wire.
- Modular integrated porosity plate/fish screen units with mechanical brush screen cleaner
- Integrated fish collector/capture weir with secondary screens and water-jet screen cleaner.

The following sections detail the fish screen, collector and bypass hydraulic design development and details.

6.9.3 Fish Screen Hydraulics

Following on the redesign of the fish screen, additional 2D hydraulic modeling was performed to refine the primary screen hydraulics and secondary screen/collector hydraulics. Changes were made to the modelled porosity of the fish screen/ porosity plate system to adjust the through-screen discharge and unit discharge along the fish screen to have:

- uniform flow through the effective screen area
- screen approach velocities less than 0.4 ft/s
- screen sweeping velocities at least 2 times approach velocities
- sufficient sweeping velocities such that the residence time of a fish adjacent to the primary screen is less than 60 seconds
- Ensure sweeping velocities are increasing in a downstream direction along the screen face with no decelerations or dead spots
- Ensure the hydraulic transition to the fish collector is accelerating not greater than 0.2 ft/s per foot of collector length
- An ideal capture velocity of 7 ft/s should be attained at the fish collector weir section and collector hydraulics should preclude burst swimming evasion and escape.

Additional porosity modeling on the primary and secondary screens and modification to the collector weir elevation resulted in a design that met the design criteria. The porosity was kept uniform over the primary screen surface in the model, which combines both the screen and the porosity plate head loss and was set at 33% open area. Porosity in the secondary screen along the fish collector was reduced to 16%.

Results of the hydraulic simulations of the fish screen and collector are provided in Figure 6-16 and Figure 6-17. Note all the preliminary 2D modeling results are with a uniform porosity and no tuning of the primary and secondary porosity boundaries.

The results in Figure 6-16 indicate a uniform approach condition and relatively little hydraulic interference from the central wall separating the screens or the small “bump-out” to accommodate the screen on the sidewalls of the bay. A velocity profile was extracted from the center flowline, with a relatively constant but slightly increasing velocity greater than 2.0 ft/s entering the screen bay from the upstream channel and Desander. From STA+1190 to STA+20[2], mean flow in the screen bay increase in velocity from 2.7 to 3.3 ft/s. As flow passed from the primary screen bay to the collector is accelerates to the weir opening where it exceeds the expected capture velocity of 7 ft/s.

[2] Flow direction is from right to left; primary screen bay STA 200 to STA 020; collector/secondary screen from STA 020 to STA 005.

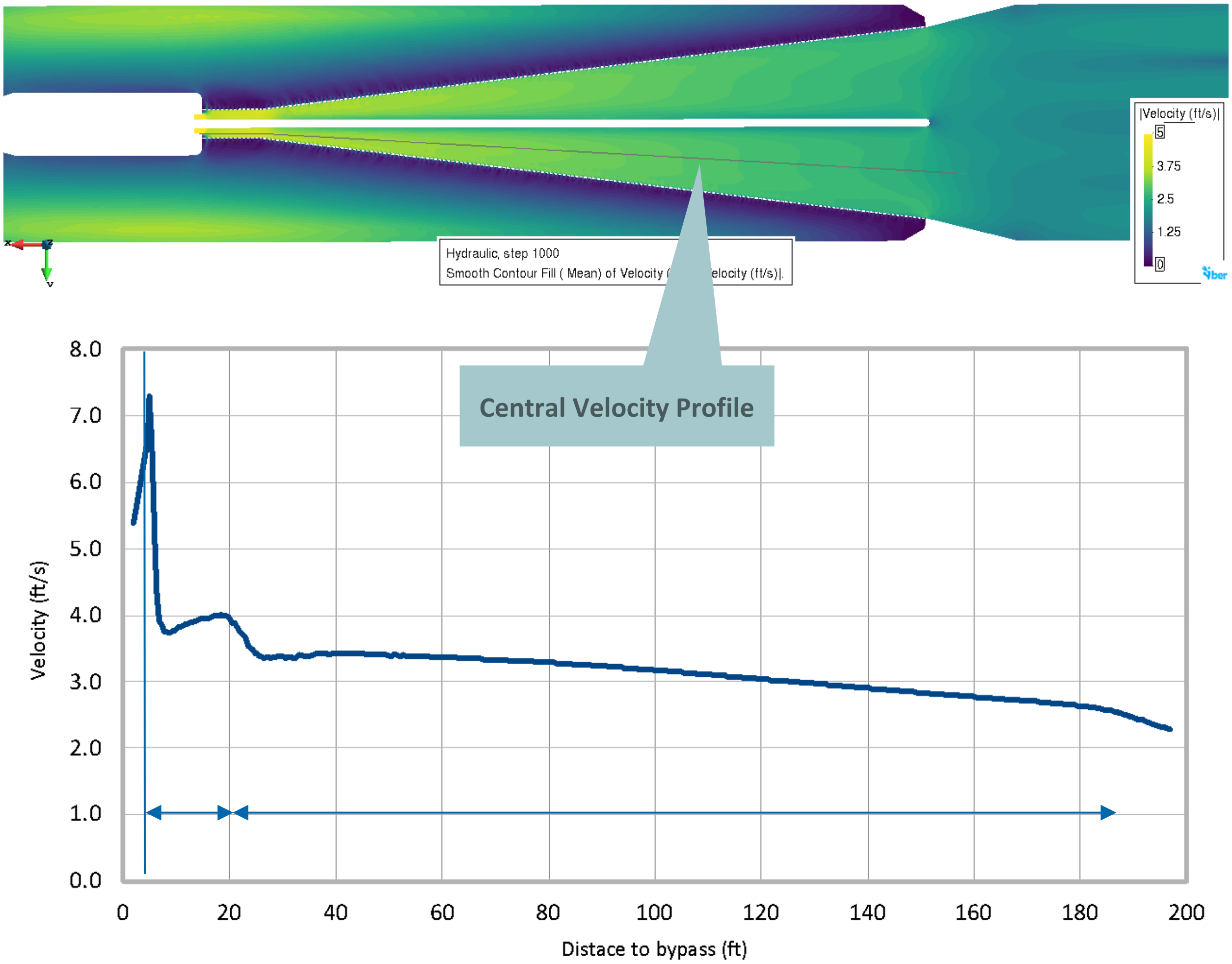


Figure 6-16 Depth-averaged hydraulic model results from 2023 design - Central velocity profile – at 750 ft^3/s diversion.

In Figure 6-17, the profile was shifted approximately 12 in off the screen surface and both approach velocities (normal to the screen face) and sweeping velocities (parallel to the screen face) were extracted from the depth averaged data. Figure 6-17, moving along the screen face from STA 190, sweep velocities increase gradually from greater than 2.5 ft/s to 3.5 ft/s at STA 020 entering the collector. In the same, profile, approach velocities increase from slight greater than 0.2 ft/s to slightly greater than 0.4 ft/s. Viewed right to left, the approach velocities that would be sensed by a fish passing down the screen are generally uniform and increasing gradually in the downstream direction. The average sweeping velocity is 3.38 ft/s, which results in a neutral particle travel time of 50 seconds, less than the 60 second criteria.

When flows enter the collector, approach velocities reduce to near zero and sweeping velocities increase rapidly towards the capture weir outlet. The velocity profile through the collector in Figure 6-17 is more representative of the mean flows and velocities passing through the weir.

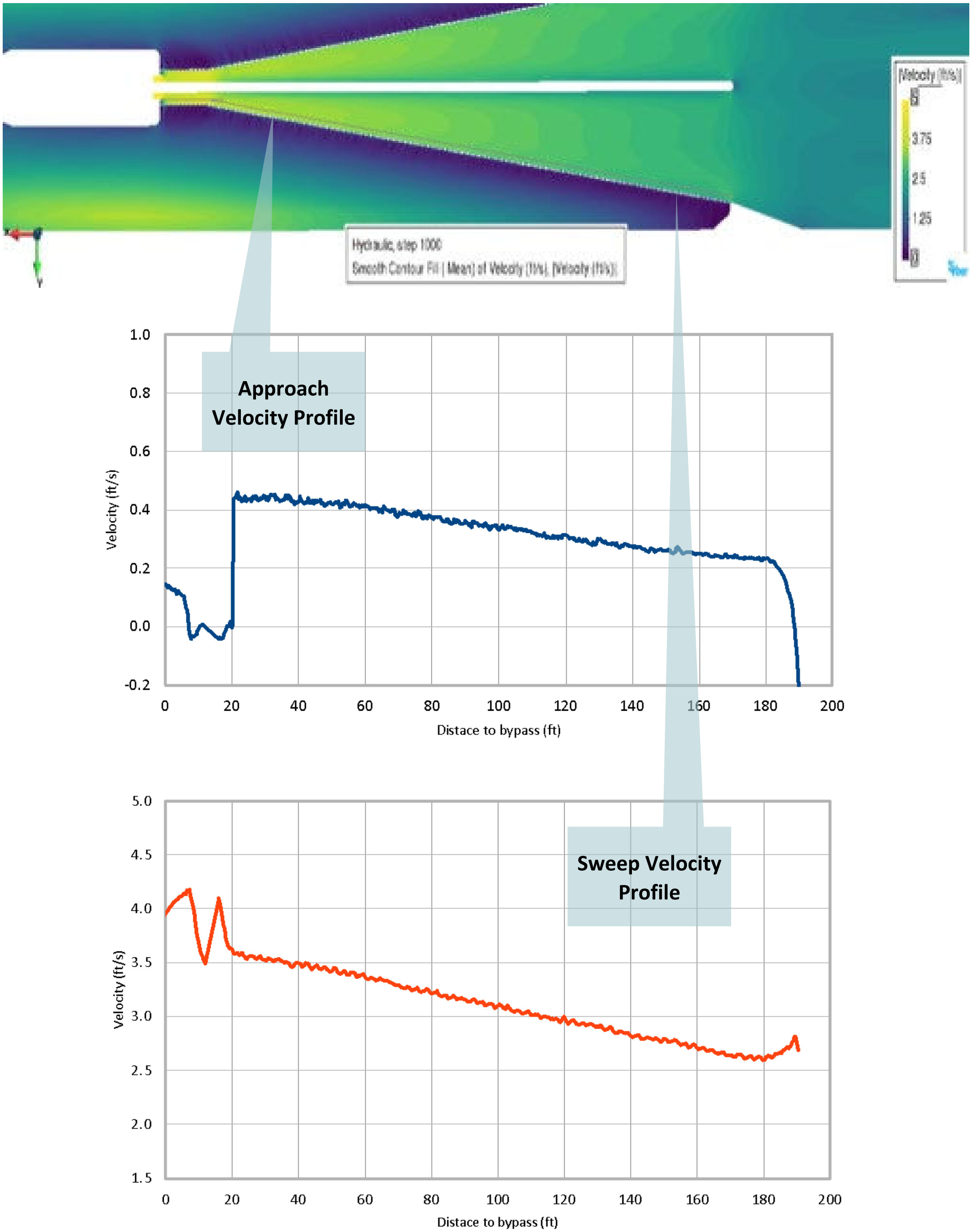


Figure 6-17 Depth-averaged hydraulic model results from 2023 design - 12" off screen velocity profile with approach and sweeping components – at 750 ft^3/s diversion.

6.9.4 Fish Collector Hydraulics

The fish collector is a 15-ft long by 2.5-ft wide flume section at the end of the primary fish screen. It includes a set of secondary screens along a section of the collector sideway; a bottom ramp that slopes upwards from the screen bay floor reducing the conveyance section; and a variable height, 1-1/2 ft wide ogee-type capture weir. The ramp section rests on the variable ogee crest and is hinged on the leading edge. It may be sealed and water-tight against the sidewall of the collector or be back-flooded, as per Figure 6-18.

Adopting a bypass flow at least 5% of diversion flow, weir height and required invert elevations were estimated using hydraulic calculations and a modified operations spreadsheet used earlier to calculate hydraulic capacities of the design and flow splits, which includes rating curves for the Hardened Ramp (Table 6-1). These hydraulic calculations are estimates that include the influence of the secondary screens and estimates of head losses due to accelerating flow. Note that diversion flows are rounded up to the next integer

Table 6-1 Diversion and Bypass Flow Hydraulics.

River cfs	No. of Screens	Diversion flow cfs	Bypass flow cfs	Screen Bay WSE ft	Flow depth ft	Weir Invert ft	Weir Flow cfs	Weir Velocity ft/s	Bypass Ratio	Min. Collector Length ft
50	1	25	2.0	158.20	0.60	157.60	2.2	2.44	3.2	8.4
100	1	50	3.0	158.70	0.75	157.95	3.1	2.75	2.3	7.8
200	1	100	5.0	159.00	1.10	157.90	5.5	3.35	2.1	8.9
400	1	200	10.0	159.00	1.65	157.35	10.1	4.09	1.3	4.8
800	1	325	17.0	159.00	2.35	156.65	17.2	4.87	1.9	11.6
1,600	2	750	19.0	159.29	2.51	156.78	18.9	5.03	1.8	11.2
3,200	2	750	19.0	159.29	2.51	156.78	18.9	5.03	1.8	11.2

The hydraulic analysis of the fish collector indicates the range of operation of the invert of the capture weir at the exit of the screen bay is from El. 156.65 ft to El. 157.95, approximately 1.3 ft. Depths of flow over the weir are estimated to range from 0.6 ft to 2.51 ft. Minimum depth of flow at low diversions could be managed with a removable weir plate to reduce the section width; weir flow conditions meet criteria at flow greater than 4 ft^3/s.

Screen bypass ratios – the ratio of the mean capture weir velocity divided by the mean velocity entering the collector – are slightly greater than criteria at full diversion indicating that the velocities entering the collector are relatively low in proportion to the capture weir flow velocity. Using the estimated channel velocity at the entrance of the collector and the estimated weir velocity, the minimum collector length was estimated using the acceleration criteria of 0.2 ft/s per ft. The design collector length is slightly larger at 15 ft to ensure no criteria issues. The last 2 ft of the collector is the solid transition through to the downwell to the bypass pipe with the weir section and ogee outfall.

6.9.5 Fish Bypass System

Downstream of the collector weir, the fish bypass system must be designed to safely convey fish and bypass flows to the tailrace without delay and injury in an open surface flow system following guidance provided in NMFS (2022a). The design drawings include in Appendix A describe the general arrangement of the fish collector, downwell and bypass pipe, and a detail is provided in Figure 6-18.

The capture weir discharges into the bypass downwell which transitions into the bypass pipe leading from screen bay. The downwell volume is sized to provide an EDF of 10 ft-lb/ft^3/s at full diversion, and the sloped floor directs flows through the right canal wall through the pipe along the bottom of the channel to minimize hydraulic disturbance. The floor slopes and geometry will be designed to ensure the transition minimizes adverse hydraulics and sloped to be free draining with no regions where either fish, sediment or fine debris will be retained during operation.

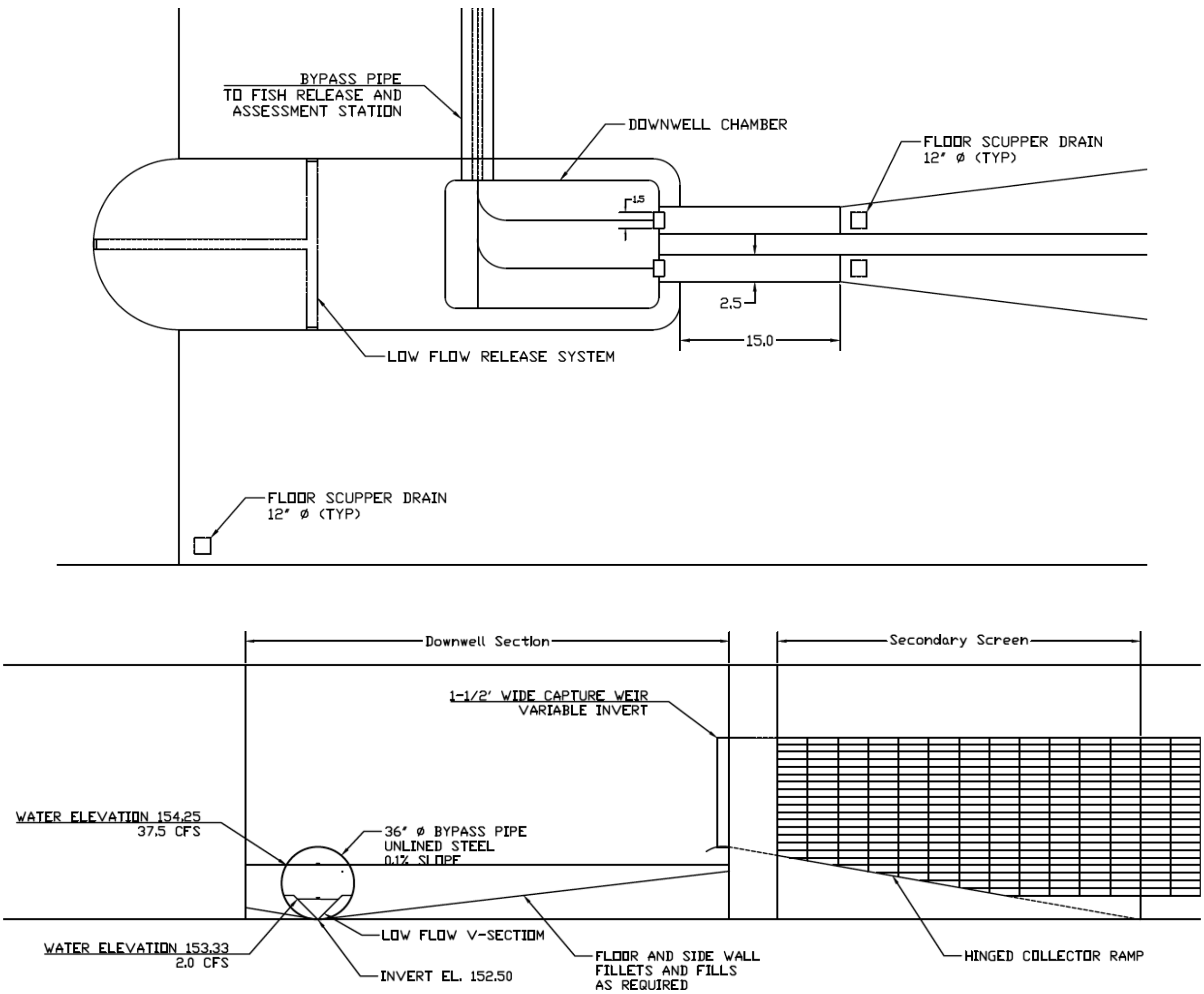


Figure 6-18 Fish collector, downwell and bypass pipe – plan and section.

The bypass consists of a 36-inch steel pipe, set at a 0.1% slope with a custom V-section sized to provide minimum required depth of flow at minimum bypass discharge. To ensure uniform flow, the pipe slope matches the expected HGL and the bypass pipe daylights in the Fish Assessment Facility at El. 152.38 ft and a full diversion water surface elevation of 154.13 ft. To ensure the bypass pipe retains minimum flow depths during low diversions and bypass flows, a small 1 ft by 1 ft 90° steel V-section will be welded into the pipe invert forming a continuous channel. Hydraulic calculations show that 10 in of flow depth (WSE 153.33 ft) are retained at flows of 2 ft^3/s using the design slope of 0.1% (Figure 6-19).

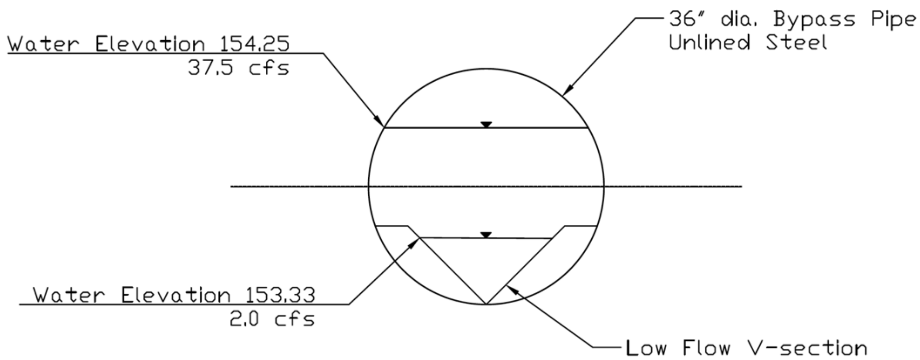


Figure 6-19 Fish bypass Pipe section detail at downwell chamber at 2.0 and 37.5 ft^3/s.

6.9.6 Fish Assessment Facility

The area outlined on the drawings is the proposed location of the Fish Assessment facility which contains:

- Fish bypass wyes and valves to shunt bypass flows to dewatering and fish separation screens from fish return system
- Horizontal dewatering screens or floor screens (2.5 ft^3/s per sq. ft) to reduce sampling flows to 1to 2 ft^3/s
- Horizontal V-channel fan screen to separate fish from flow to holding tanks for enumeration, sampling and tagging, as required by biological programs
- Small pumps, aerators, wet lab and holding tanks
- Open channel flow (e.g., return trough) to fish release channel and bypass outfall exterior of the Fish Assessment Facility.

6.9.7 Bypass Outfall / Fish Release Channel

The current bypass outfall is a pipe outlet in the sidewall adjacent to the existing fishway entrance. The 36" bypass pipe falls directly into the fishway tailrace/entrance. A suspended outfall pipe above high tailwater levels would alleviate the backwatering but result in a high freefall drop at low water conditions into a relatively small residual pool.

NHC considered the use of a nature-like outfall channel or Denil fishway constructed in the side slope and rock protection on the left bank of the river channel downstream at the Hardened Ramp and Flushing Channel outlet.

An open channel would be difficult to construct in the rough rock slope, would need lining to remain water-tight, and would be difficult to zig-zag down the slope to retain a mild slope, moderate turbulence and energy losses and acceptable depths of flow. While a Denil fishway section may be energetically appropriate, concerns were expressed regarding fish attraction at the tailrace.

The design development suggests use of a similar system to what is currently used, consisting of a similar 36" pipe which can be articulated and (e.g., raised and lowered) according to tailwater conditions (see drawings). This system would provide direct diversion of juvenile fish to the tailwater and ensure safe hydraulics during discharge.

6.9.8 Fish Screen Operation and Maintenance

Operability and reliability are important features of any fish screen in order to provide diversion flows and the safe removal and conveyance of fish from the diverted water. As fish screens are largely static structures that operate by open surface gravity flow, their performance is related to both design and construction. Openings, sharp edges in high velocity flow and hot spots are all issues related to screens that lead to loss, impingement or injury to fish. For example, very tight tolerances and craftsmanship are required in the fabrication, assembly and installation of the fish screen components. "Fry tight" is a term used to describe the tolerances of finish required in terms of gaps and openings in fish screen systems, which related to the NMFS (2022a) criteria of gaps or openings no greater than 0.069 in (1.75 mm) in the fish side of the system.

The proposed fish screen system at Freeman Dam should utilize a uniform modular screen panel and porosity plate head loss control system. A modular system allows for systematic fabrication of larger uniform components with fewer gaps and edges. For the primary screen, NHC suggests 17 – 10 ft long by 6-1/2 ft high screen panels and a similar number of porosity control panels located 2 ft behind them. Stainless steel wedge wire profile screen material should be used in a parallel orientation with an open area between 35% to 40% matching the maximum porosity of the system.

Variable, behind-screen head loss control is critical for the operation of the screen system. By transferring the potential head loss from the screen surface to the porosity plate system, through-screen velocities are reduced as well as the potential for fish impingement on the screen surface. The porosity control on the primary screens should utilize the industry standard UHMW polyethylene panel / stainless steel plate orifice or slot system which can provide open areas from 40% to near zero. The secondary screens in the fish collector typically used a fixed porosity once designed and tested.

The project will utilize a 6 in clear spacing on the trash rack so debris loading during flood events will still be extremely high, and mechanical screen cleaning during diversion will be constant to ensure even flow distribution over the effective screen area. NHC has reviewed some mechanical and hydraulic systems and recommends a mechanical brush or wiper system be used on the outside surface of primary screens and a water jet system on the secondary screens, where access to the surface of screen panels is less ideal for passing fish at higher velocities. Secondary debris removal may be required in the Fish Assessment Facility when dewatering and secondary screening is underway as a matter of the debris loading and screen cleaning operations.

Sedimentation of the screen bay and lower portion of the screen surface is an issue that hampering screen operations and cleaning on the current Freeman dam screen system. The Desander is expected to significantly reduce sedimentation in the screen bay. In addition, a 1 ft high sill is designed under the proposed screen edge to lift it up and out of any sediments that may settle in the bay during diversion operations. The floors of the screen bay upstream and downstream of the fish screen should be canted and sloped towards floor drains installed for dewatering and cleaning during non-diversion periods.

Operations of the Freeman dam sometimes entails relatively small diversion flows. In lieu of operating the downstream regulating canal gates, NHC suggest design and installation of a gated diversion pipe system from behind the screens to the downstream side of the canal gates. A 24 to 36 in diameter pipe system would allow the regulated diversion of flows of 50 ft^3/s and less.

7 Sediment Management

At 2,700 $t/km^2/year$ (1.4 mm/year denudation rate), sediment production in the Santa Clara River watershed is extremely high, amongst the highest in the world (Stillwater Sciences, 2011) and comparable to sediment yields observed in the Andes and Himalayas Mountain ranges (Figure 7-1). High sediment production is in part because the mountains in this area experience the fastest long-term tectonic uplift rates in the continental United States of up to 9 mm/year (Stillwater Sciences, 2011). Sediment concentrations are also extremely high, easily exceeding 10,000 mg/L (Figure 7-2) and translating into very high sediment loads of multiple grain sizes reaching Freeman Diversion.

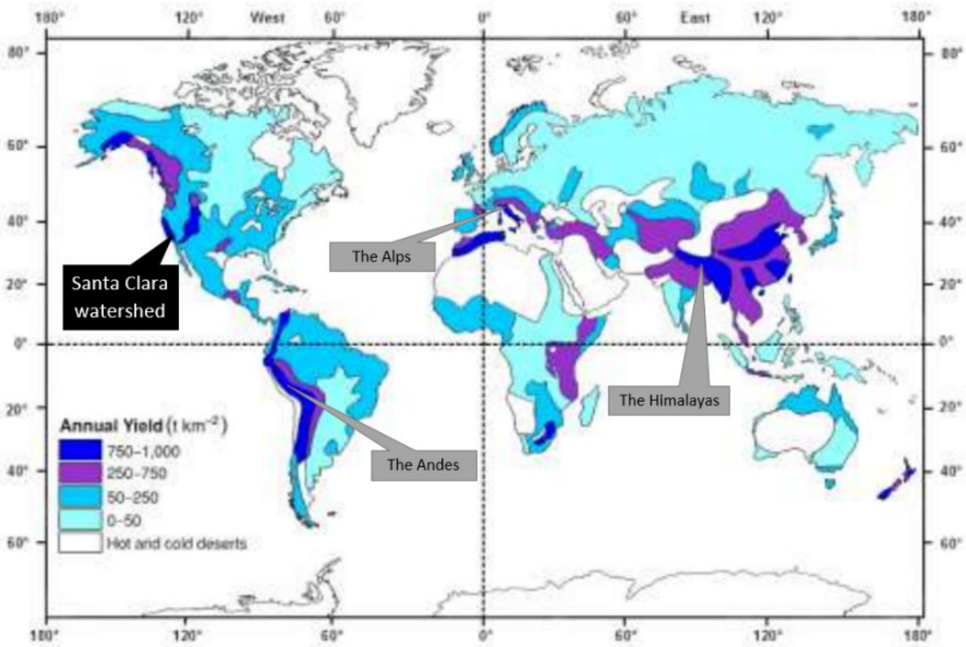


Figure 7-1 **World annual sediment yield (Flemming, 2011).**

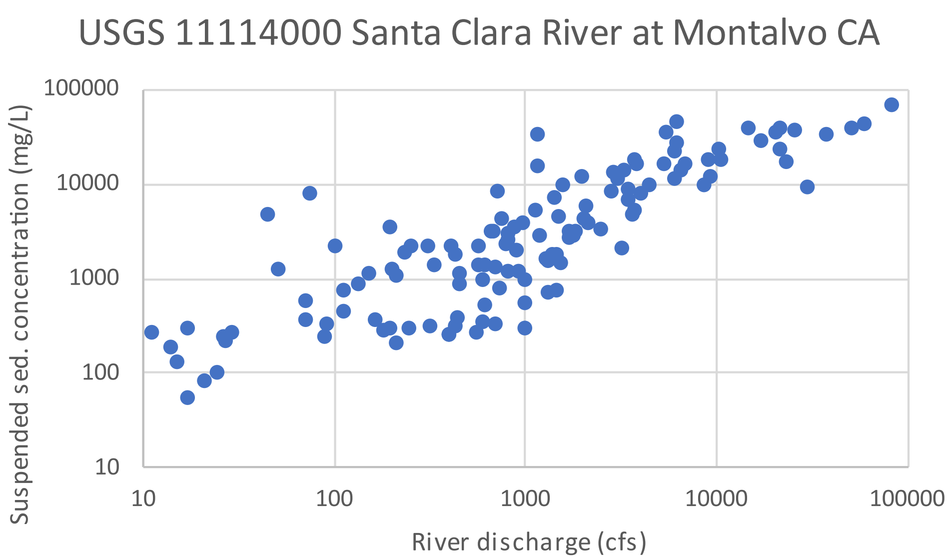


Figure 7-2 **Suspended sediment concentration Santa Clara River at Montalvo (USGS Station 11114000, 1970-1995).**

7.1 Bedload and Suspended Load

Sediment in the Santa Clara River spans a wide range of grain sizes, from silt to boulders. The largest sediment sizes, such as coarse sand, gravel and boulders typically move rolling over the riverbed as bedload, while smaller sediment sizes in the sand and silt range are transported suspended in the water column as suspended load. This distinction between bedload and suspended loas is important because it influences how sediment behaves, how sediment loads split at the diversion and hence how it needs to be managed.

Since suspended load comes into the intake with diverted water, there is nothing that can be done outside the intake to manage suspended load, which must be dealt with inside the intake structure by means of the desander facility. For bedload, the strategy is to prevent it from entering the intake by using the training wall and crest gates and then remove it by operating the flushing channel.

When the Santa Clara River approaches Freeman Diversion and flow velocity reduces, coarse bedload sediment coming from upstream will be mostly directed towards the intake with the remaining bedload continuing downstream towards the hardened ramp. Without the presence of a training wall, bedload forms a large bar or bedform upstream of the ramp and on the apron in front of the intake trashrack, affecting flow hydraulics (Figure 7-3).

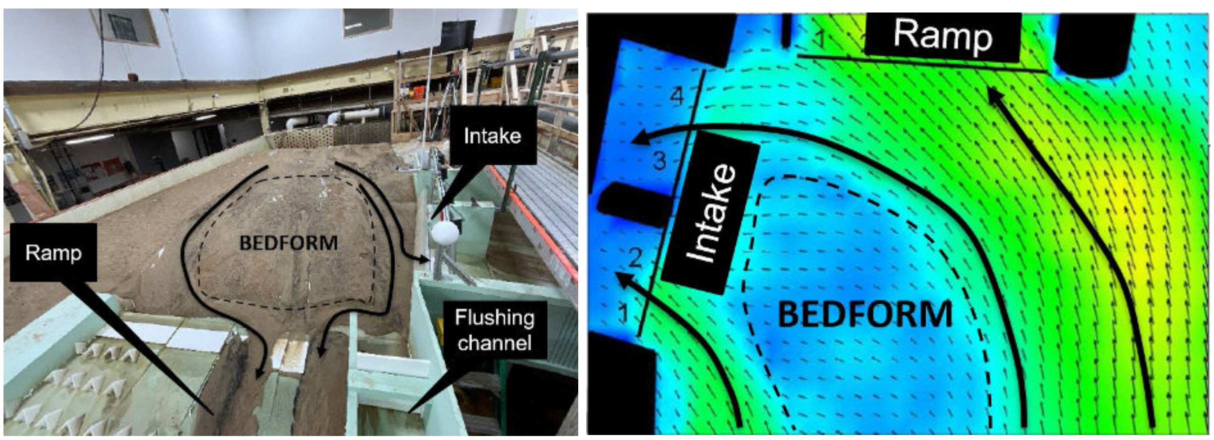


Figure 7-3 Example of sediment bedform observed in physical model and how it affects the approach velocity field (adapted from baseline testing figures C-56 and B-13 in USBR 2022).

Suspended sand would tend to fall from suspension into bedload, possibly settling on the apron, over the intake gates or within the diversion area located between the fish screens and intake trashrack. The intake crest gates can temporarily stop bedload from entering the intake, but if bedload deposits are not removed, they will eventually overcome the gates and make its way into the diversion (Figure 7-4).

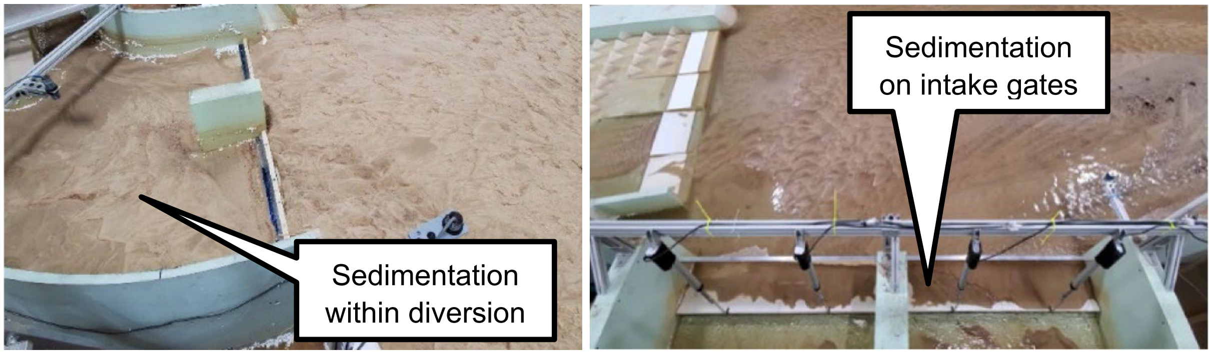


Figure 7-4 Example of sediment deposition within the diversion and on top of intake crest gates observed in physical models (adapted from baseline testing figures B-5 and C-62 in USBR 2022).

Excessive sediment deposition is undesirable because it would modify the approach flow conditions to the ramp exit (Figure 7-3), potentially reducing its hydraulic performance and reducing the ability of fish to exit the hardened ramp. Sedimentation could potentially block the intake (Figure 7-4) reducing diversion yield. Sedimentation could also partially bury the fish screens or block the fish bypass pipes. Therefore, there is a need for a sediment management system (Figure 7-5) to control and manage sediment in order to prevent or remove excessive sedimentation from the facility.

7.2 Sediment Management Objectives

To guide the design process from a fish passage perspective, NMFS and CDFW and UWCD agreed on the nine sediment management objectives detailed in Table 7-1, which also contains a brief description of how each objective influenced a particular aspect of the design. During the design process, the sediment management facilities (Figure 7-5) were modified as much as possible to minimize effects perceived as potentially adverse to fish passage.

For example, the invert of the flushing channel was lowered to the same elevation of the ramp to eliminate plunging flow at its downstream end, while the flushing gate was changed from a sluice gate to a crest gate to minimize the risk of injury to fish that could potentially be entrained. The training wall and enlarged bullnose wall were introduced to straighten the approach flow and minimize sedimentation around the ramp exit. A low notched opening was introduced in the training wall to provide hydraulic connectivity between the apron and hardened ramp, to minimize the risk of stranding fish.

The sediment management system is intended to transport sediment using the available energy of the flow driven by the difference between the upstream headwater levels and downstream tailwater levels. In this way, it is not necessary to rely on mechanical devices to dredge or excavate sediment deposits under normal conditions. Mechanical cleaning may be necessarily in certain cases, but during dry conditions outside the fish passage window.

Table 7-1 Sediment objectives and how they influenced the design.

Issue	Sediment objective	Design
1	Maintain the low-flow river channel near the left bank to enhance sediment transport through, and downstream of, the fish passage alternative.	Operation of flushing channel is expected to keep river channel near the left bank
2	Ensure the fish passage entrance(s) and exit(s) remain relatively free of sediment and debris such that fish have the ability to enter, ascend, and exit the hardened ramp. For the ramp, a migratory path of sufficient depth and velocity should be maintained throughout the range of fish passage flows.	Bullnose extension and training wall help straightening approach flow to hardened ramp reducing sediment accumulation at fish exit
3	Design development should evaluate the sediment management capabilities of each design and estimate what is required to maintain fish passage for both alternatives. Utilizing information gathered from physical modeling, operations incorporated into and analyzed in the MSHCP should avoid or minimize sediment and/or debris removal from the fishway where surface flow is present unless sediment/debris is preventing fish passage. The physical models should be used to best estimate the frequency and extent of sediment/debris maintenance activities, and to inform the time required to implement sediment removal activities.	The hardened ramp is expected to self-clean of sediment and debris for most flow conditions; especially when river flows are increasing. Some floating debris could get stuck on top of the steel baffles when river flows are receding, but this could be removed on the dry after the flood passes, with no effect on fish passage.
4	Minimize hydraulic recirculation zones that lead to sediment deposition upstream of the diversion intake and in the vicinity of the apron directly in front of the intake, which is also the fish passage exit area. Promote accelerating flow conditions or induced secondary currents that do not impede (agency preference) or interrupt (UWCD preference) fish passage while still enhancing sediment transport through and downstream of the dam. Design development should evaluate elements that enhance sediment transport continuity capabilities of fish passage designs.	Training wall straightens flow on the apron in front of the intake, minimizing recirculation zones. The training wall creates a converging channel (from 40 ft to 15 ft wide) that accelerates flow to promote sediment movement when flushing and does not impede fish passage during normal operation.

Issue	Sediment objective	Design
5	Conduct sediment management of the diversion intake apron and/or the fish passage exit, in a manner that minimizes both delay and take of steelhead by not interrupting fish-passage and minimizing the risk of stranding and injury to fish during sediment maintenance operations. This includes but is not limited to the following: not causing unfavorable fishway entrance, transit route, or exit conditions (hydraulic and water quality); not causing nuisance attraction flows; and avoiding entrainment of adult or juvenile steelhead into non-fish passage flow routes (e.g., flushing channel). Utilizing information from physical modeling, operations incorporated into and analyzed in the MSHCP should avoid or minimize sediment management activities that impede (agency preference) or interrupt (UWCD preference) fish passage.	Nuisance flows will be very limited due to sporadic operation of flushing channel and low sluicing discharge of desander. Both structures discharge next to the ramp's low flow channel at the same invert elevation. The low invert profile and 2.5% slope of the narrow 15 ft flushing channel (compared to 5%-sloped 90 ft wide ramp) combined with crest gate should minimize risk of injury to fish that could potentially be entrained.
6	Within the diversion intake, develop a secondary settling area and non-mechanical system to flush sediment in lieu of mechanical cleaning. Some sediment accumulation within the intake channel is inevitable but to a degree that mechanical dredging may be efficient when performed outside the range of steelhead-passage design flows (45 to 6,000 ft^3/s).	The desander provides a secondary non-mechanical settling area to trap and sluice sediment deposited within the diversion area.
7	The fish screens should be kept clean and functional. Capability to hydraulically sluice the fish screen channels in lieu of mechanical cleaning should be considered. Sluicing options would need to protect fish by meeting guidelines for fish bypasses.	The desander will reduce the amount of sediment passing downstream into the fish screen bays, helping to keep the screens clean and functional.
8	Mechanical removal of sediment, when necessary, should be limited to periods outside the fish passage window and should minimize impacts to aquatic species.	Mechanical removal of sediment during the fish passage window is not part of current sediment management plan.
9	If some manual sediment removal will be required via suction dredging or excavation from within the diversion intake, de-sander, near fish screens, and in the diversion conveyance canal during the fish passage season, then impacts to fish during these activities should be avoided or minimized.	Manual removal of sediment by dredging or excavation during the fish passage window is not part of current sediment management plan.

7.3 Sediment Management System

The hardened ramp sediment management system relies on multiple sediment facilities to manage different types of sediment and provide resilience for a range of potential flow conditions, as shown in Figure 7-5. The hardened ramp provides continuous and uninterrupted sediment passage during most flow conditions (except extreme floods when the ramp is closed for protection). The desander and flushing channel provide intermittent transport for sediment accumulated either within the diversion (desander) or the apron in front of the intake (flushing channel).

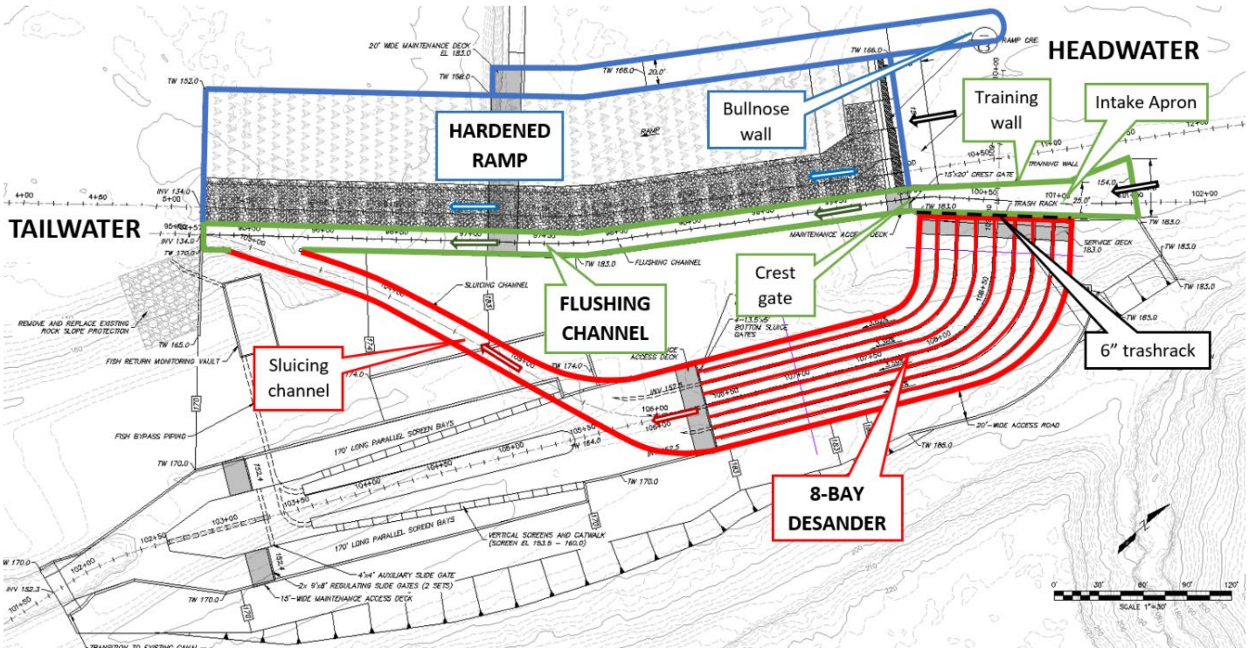


Figure 7-5 Schematic of hardened ramp sediment management system (arrows indicate direction of sediment movement).

The flushing channel/apron and desander are split upstream at the 6-inch opening intake trash rack but converge at their downstream tailwater ends (Figure 7-5). The flushing channel is intended to remove coarse bedload sediment deposited on the apron in front of the diversion intake. If these coarse sediment deposits on the apron were allowed to build up above the intake crest gate elevations, coarse bedload could roll over the gates and enter the diversion (Figure 7-4). Because the flushing channel and training wall provide a straight path between the apron and tailwater channel downstream, it can clean the apron quite effectively in a short period of time.

Diverted water will transport suspended sediment (silt and sand) into the intake facility. Some of the suspended sand will settle within the diversion, in addition to any bedload sediment that rolled over the intake gates. Sedimentation within the diversion could reduce diversion yield due to reduction of hydraulic area and increased head loss, while affecting the operation of the fish screens. The desander shown in Figure 7-5 is an internal sluicing facility intended to trap and remove sediment that deposits within the diversion, before it could reach the fish screens downstream. A brief description of the sediment management system components is provided below.

7.3.1 Hardened Ramp

- Due to its large size (90 ft wide) and invert elevation (between El. 156.5 ft and El. 160.0 ft), the hardened ramp conveys between 45% and 68% of the total Santa Clara River inflow approaching Freeman Diversion (see Figure 10-7).
- The hardened ramp is the main pathway for water and sediment to concentrate and move downstream, providing continuous and uninterrupted sediment passage during most flow conditions.
- Due to the high flow velocity and turbulence on the ramp, most bedload sediment entering the ramp is resuspended and quickly transported downstream without depositing, i.e., the hardened ramp self cleans of sediment along most of its length.
- Some deposition on the lower end of the ramp occurs due to tailwater effects, providing a gradual transition from the hardened ramp to the alluvial riverbed downstream.
- As shown in Figure 7-3, a large bedload bedform persisted in front of the intake structure for all modeled conditions (MOD-6, MOD-9, with and without flushing channels). This bedform regrades the channel and builds up deposits in front of the intake gates.

7.3.2 Flushing Channel

- Because the hardened ramp cannot by itself remove the bedload sediment directed towards the diversion and deposited in front of the intake, the flushing channel is required to perform that task.
- The flushing channel provides a straight and direct path between the apron in front of the intake and tailwater channel downstream (Figure 7-5) to clean the apron efficiently and reduce the volume of material that would otherwise make it into the intake facility.
- Its operation requires turning out the diversion.
- The flushing channel is planned to be used sporadically at high flows above 3,000 ft^3/s.
- Moves sediment downstream providing some continuity for sediment.
- Can be used effectively at high downstream water levels (Figure 7-6).
- Creates a large footprint of sediment removal upstream to re-grade the channel (Figure 7-7).
- Used to train the river to the left bank by increasing concentrated flow.
- By conveying part of the incoming river flow, it reduces velocities and depths at the ramp increasing the ability of the ramp to provide adequate fish passage conditions at high flows.

7.3.3 Desander

- Allows for uninterrupted fish passage when sluicing.
- Removes both suspended and bedload that enters and settles within the intake.
- One or more bays can be sluicing sediment while diversion continuous through remaining bays, though with lower diversion discharge.

- Because it requires a longer path to move sediment (Figure 7-5), it is less efficient than the flushing channel at removing sediment.
- It is most efficient at lower tailwaters, becoming inefficient at high tailwaters (requires more water to remove same volume of sediment, Figure 7-13).

Because the hardened ramp was already described in Section 5, only a detailed description of the flushing channel and desander is presented below.

7.4 Flushing Channel

The general layout of the flushing channel, apron and training wall are given in Section 4.4. The flushing channel is needed to remove coarse sediment deposited on the apron in front of the intake. The alignment of the flushing channel provides a direct connection between the apron upstream and downstream river channel, allowing efficient movement of sediment. Without the flushing channel coarse sediment deposition over the apron could block the intake, while finer sediment could enter the diversion overwhelming the desander. The flushing channel is expected to operate during elevated river flows above 3,000 ft^3/s, which is possible because it is rather insensitive to high tailwater levels.

During the design development phase in the physical model (USBR, 2022), the initial 30% design of the flushing channel (NHC, 2021) was substantially improved. The initial design considered a higher elevation channel with a flat upstream apron without a training wall and with an undershot gate. Baseline testing of the initial design showed that sediment was only removed a short distance upstream from the gate, while a plunging flow developed when the flow discharged downstream, which could impact fish and generate a deep scour hole. The entire invert of the flushing channel was lowered to match the downstream elevation of the hardened ramp, such that a deep tailwater pool was available, eliminating the plunging flow and local scour hole, similar to the hardened ramp's gradual transition to the downstream channel.

7.4.1 Intake Apron

The 145 ft long apron slopes 5.2% from its upstream end at El. 154.0 ft towards the entrance of the flushing channel at El. 146.0 ft. NMFS and CDFW have expressed some concerns that the space over the apron could become a deep isolated permanent pool where fish could congregate. However, this space is expected to quickly fill up with sediment, so a shallow pool condition would likely prevail most of the time. Also, the training wall's deep slot near its downstream end provides hydraulic connectivity to the hardened ramp to prevent fish from becoming isolated.

7.4.2 Training Wall

The purpose of the training wall is to concentrate flow and increase velocity and sediment transport upstream of the flushing channel. The training wall has the primary function of improving the efficiency of the flushing channel to clean the apron, while also partially blocking some of the bedload sediment coming toward the intake, as discussed below.

Without the training wall, the flushing channel could not effectively remove sediment from the entire apron. It is well known that a submerged flushing gate located in a wide reservoir will only remove sediment a short distance upstream (Morris and Fan, 1998). This was also identified by idealized CFD simulations conducted by NHC (2021), where it was shown that when the flushing channel operated, near-bed velocity was high only immediately upstream of the flushing gate (see Figure 4-14 in NHC 2021)[3]. To increase velocity and sediment transport rates upstream of the flushing gate, NHC (2021) investigated the possibility of adding a tall “guide wall” and a short “submerged sill”, which can be seen as variations of the training wall, but results from the idealized CFD model were inconclusive.

7.4.3 Flushing Channel Efficiency

Tests conducted in the 1:24 physical model showed that the flushing channel with the sloping apron and castle training wall was effective in removing sediment deposited on the apron in front of the intake. All tests were conducted with the diversion turned out and some initial sedimentation on the apron. After lowering the flushing gate completely, bed levels on the apron were observed to be lowered (scoured) at rates roughly between 5 and 8 ft/hr in prototype[4].

Figure 7-6 shows bed levels measured in the 1:24 physical model along the apron in front of the intake before (orange line) and after (blue line) a flushing operation was conducted. Four tests were conducted for river discharges between 1,500 and 6,000 ft^3/s. Although the flushing channel is not expected to operate at 1,500 ft^3/s that test provided an additional data point to better understand its operation and confirm that the hardened ramp was not dewatered. The plan view in each plot shows the location of the five points where sediment depths were measured before and after each test. Besides initial and final bed profiles, each plot shows the river discharge, tailwater (TW) level, duration of the test in prototype time[4], and the average bed lowering (erosion) at the 5 measuring points. The profiles also give an approximate indication of the invert elevations for the intake gates (El. 155.0 ft and El. 156.5 ft).

It was observed that bed levels on the apron would tend to build up to around El. 160 ft (Figure 7-6), approximately coinciding with the invert of the castle wall notches, at least during the short duration of the physical model tests. The initial bed levels were in most cases above the intake invert elevations. This means that if the intake gates were to be fully opened, for example to divert water during low flow conditions or sluice the desander, large amounts of sediment could be ingested by the intake. Fortunately, all flushing tests were successful in bringing down final bed levels below the intake invert elevations, to values close to the invert profile of the apron.

---

[3] This is also true for the hardened ramp. Although the ramp moves all sediment entering the structure, bed velocity is only high immediately upstream of ramp exit and helps explain why the ramp is not capable of removing the bedload bedform upstream.

[4] The time scale for sediment transport rates between the 1:24 physical model and the protype remains unknown; but it was assumed to be the same as Froude’s time scale for flow: $(24)^{0.5} = 4.9$. This is probably conservative, meaning that durations in the real prototype would be likely shorter than reported here.

Figure 7-7 shows an example map of the bed changes caused by the operation of the flushing channel for a river flow of 3,000 ft^3/s, as measured in the 1:24 physical model (USBR 2023). Red shades indicate erosion. In agreement with the profile in Figure 7-6, the map in Figure 7-7 shows intense bed scour along the apron, mostly in excess of 7 ft in front of the intake. The scouring effect of the flushing channel is felt some distance upstream, approximately 100 ft in Figure 7-7, but this value might have been constrained by the short duration of the test. Figure 7-7 also shows erosion along the right (north) side of the training wall, creating a small erosion channel directed towards the ramp's low flow channel. This small erosion channel has also been observed in low flow test and it is believed to be caused by the training wall interserting and redirecting flow.

To test the influence of tailwater levels, for one of the tests with a river flow at 2,800 ft^3/s, tailwater levels were raised from El. 144 ft to El. 152 ft, submerging the entire flushing channel. Despite the higher tailwater levels, the erosion rate on the apron was not observed to decrease, proving the flushing channel design is robust and able to work effectively with relatively high tailwater levels, which is not the case for the desander, as will be discussed later.

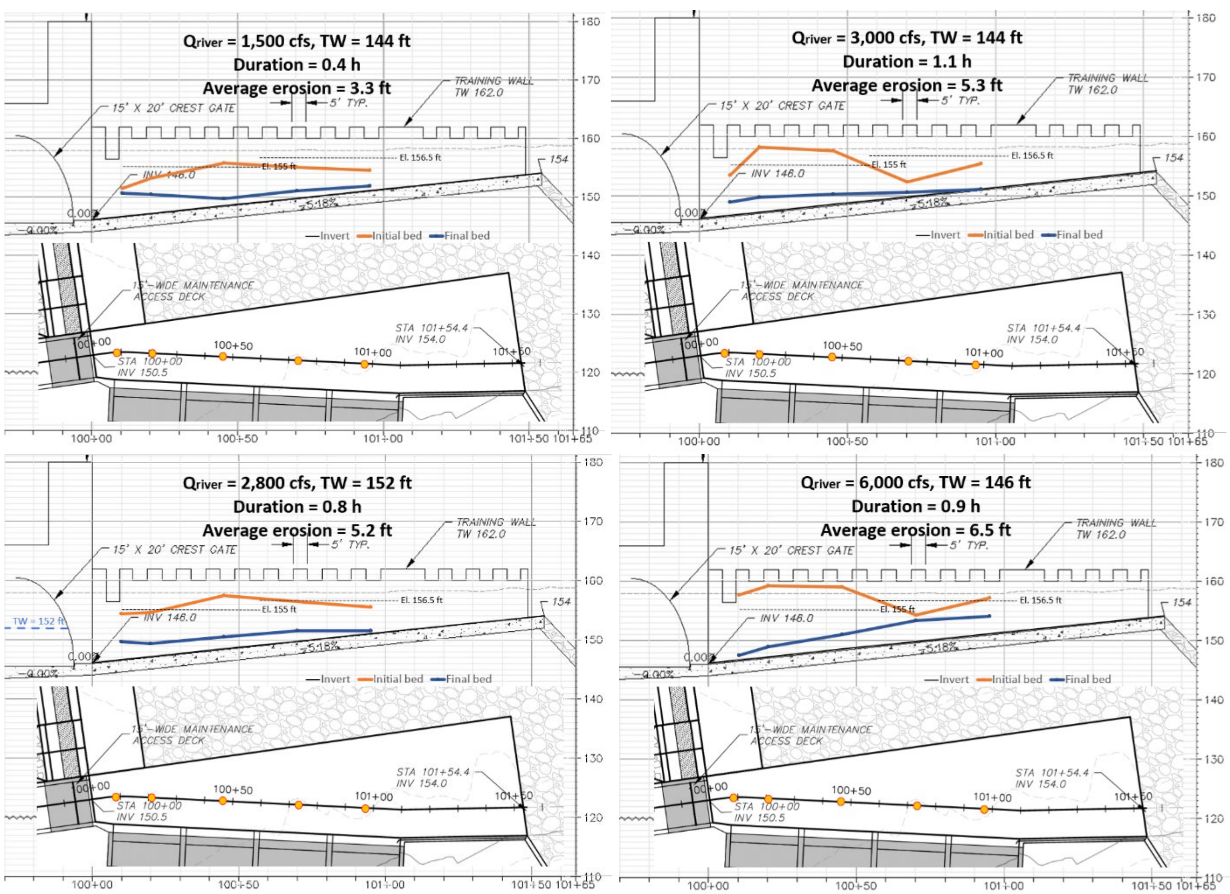


Figure 7-6 **Bed level profiles along intake apron when flushing channel operates under various flow conditions (protype durations scaled up 4.9 times from model data).**

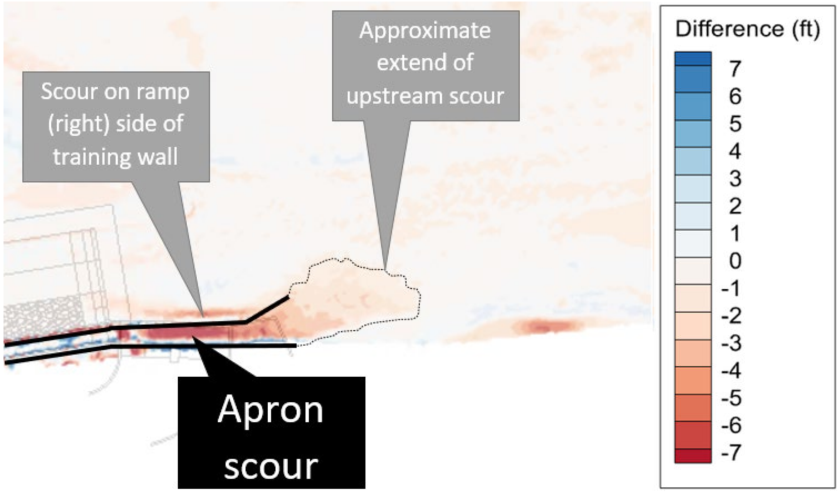


Figure 7-7 Bed level changes caused by flushing channel operation at river flow of 3,000 ft^3/s. Negative values indicate erosion (USBR 2023).

Although the flushing channel is quite effective at removing sediment, it does so without generating very high velocities on the apron and flushing channel itself. Velocities measurements made in the 1:24 physical model using particle image velocimetry (PIV) show that velocity magnitudes on the apron and flushing channel are generally below 10 ft/s (USBR, 2023) and comparable to those found on the hardened ramp (Figure 7-8). This is probably because although having the same downstream invert elevation (134 ft), the slope of the flushing channel (2.5%) is only half of that of the hardened ramp (5%), causing a deeper flow along the flushing channel.

One concerned expressed by NMFS and CDFW was the possibility that the deeper apron could create an isolated deep pool of water where fish could congregate and potentially be injured when the flushing gate was opened. As Figure 7-6 shows, sedimentation on the apron quickly restores bed levels to values found upstream of the hardened ramp (above El. 156.5 ft). The deep notch in the training wall located closest to the ramp remained free of sediment, due to intense flow going through it, ensuring hydraulic connectivity for fish between the apron and the hardened ramp. Also, since flow velocities during flushing are not very high (Figure 7-8) the potential for abrasion injury to fish is probably not as great as initially believed.

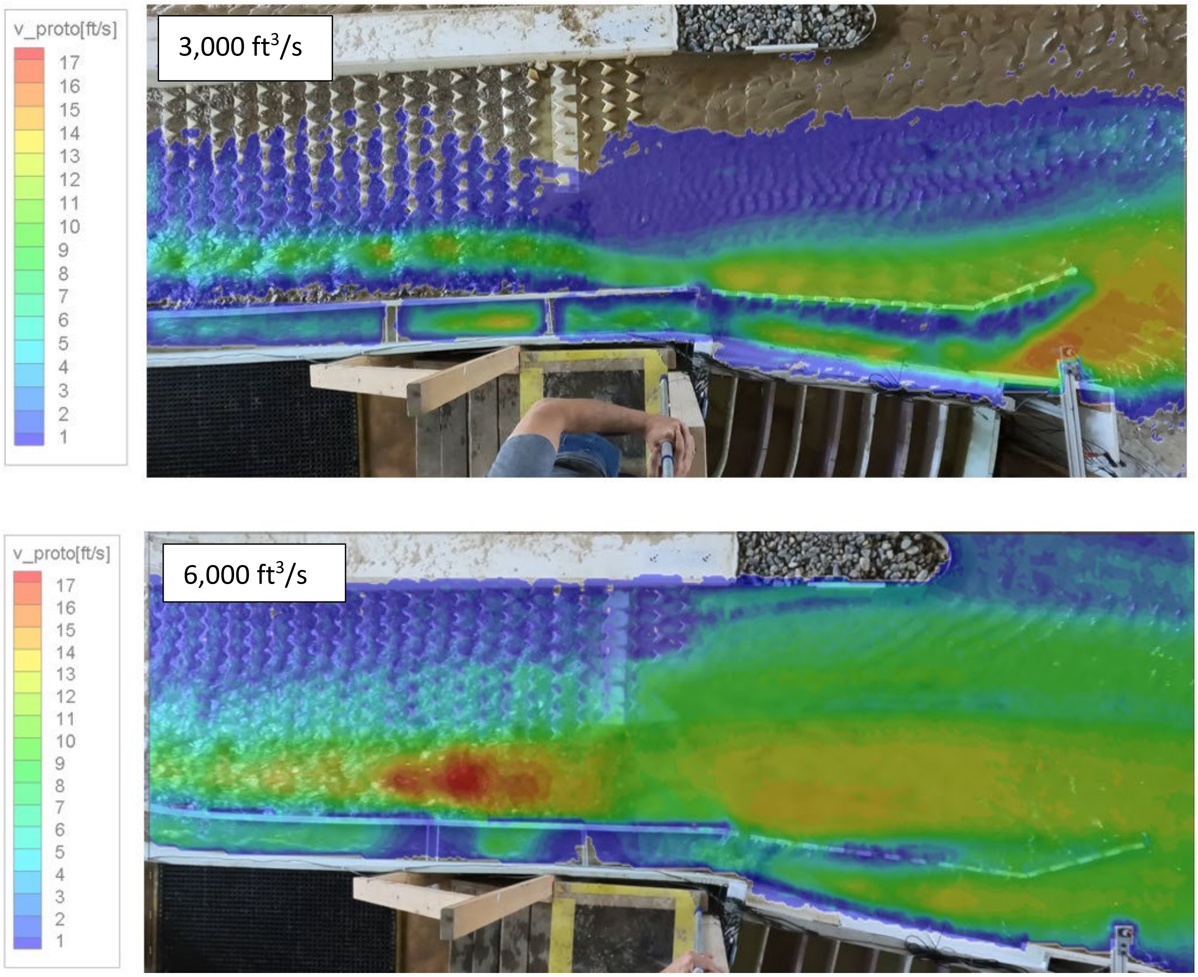


Figure 7-8 Surface velocity measured in 1:24 physical model during flushing operation at river discharges of 3,000 ft^3/s and 6,000 ft^3/s (USBR, 2023).

7.5 Desander Channel

A desander is a hydraulic structure designed to trap and remove a fraction of the sediment entering a river diversion intake. Although a desander can trap all bedload (coarse sand and gravel) they are mainly intended to trap sand transported in suspension that cannot be excluded, hence their name. Finer suspended sediment (silts and clay) will generally not be trapped in the desander and would pass through the fish screens and canal to be removed in a desilting basin downstream. Current operations use this system effectively to minimize fine sediment transport to UWCD's recharge ponds.

Desanders typically have large footprints and work as settling basins decreasing flow velocity and turbulence, promoting the settling of sediment. Using the hydraulic head difference available between the desander and the river reach downstream of the dam, the sand deposited on the bottom of the desander can be periodically sluiced back to the river downstream without the need for mechanical excavation or dredging (Figure 7-10). Desanders are very common in intakes located in rivers with high sediment loads (Figure 7-1), like those found in the Andes (Vasquez, 2007), the Himalayas (NHC, 2016a) and the Alps (Bouvard, 1992), with some examples in the US (Garde and Ranga Raju, 1977).

The main objective of the desander is to decrease both the concentration and maximum grain size of suspended sediment in the diversion canal downstream from the desander, to minimize sedimentation downstream, especially on the fish screen bays.

7.5.1 Freeman Diversion Desander

The general layout of the desander is shown in Figure 7-5, a plan view and longitudinal profile of the desander bays and sluice channel is shown in Figure 7-9. Figure 6-7 shows a conceptual design of the gates at the downstream end of each bay.

The desander has eight parallel bays with width varying from 11 ft upstream to 5.75 ft downstream. The longitudinal slope of each bay varies between 3.3% and 3.8%. The sluicing channel downstream is 15 ft wide with a 3% slope and merges with the flushing channel downstream. The culvert underneath the diversion canal functions as a smooth transition, with curved walls that help turn the flow to the right and connect the 8 desander bays into a single sluice channel.

The downstream exit of the sluice channel has its invert at El. 134.0 ft. The downstream invert of all bays is at El. 146.0 ft. The upstream invert of bays 1 to 4 on the left half of the intake (looking downstream) is at El. 156.5 ft, while bays 5 to 8 on the right half is at El. 155.0 ft. Each bay is controlled by 3 gates, two downstream and one upstream. At the downstream end there is one 5.75 ft wide by 9 ft high canal gate and one 5.75 ft wide by 5 ft high bottom sluice gate, while upstream there is one 11 ft wide intake crest gate. Intake gate heights are either 8.5 ft (invert at El. 156.5 ft) or 10.0 ft (invert at El. 155.0 ft) depending on their location.

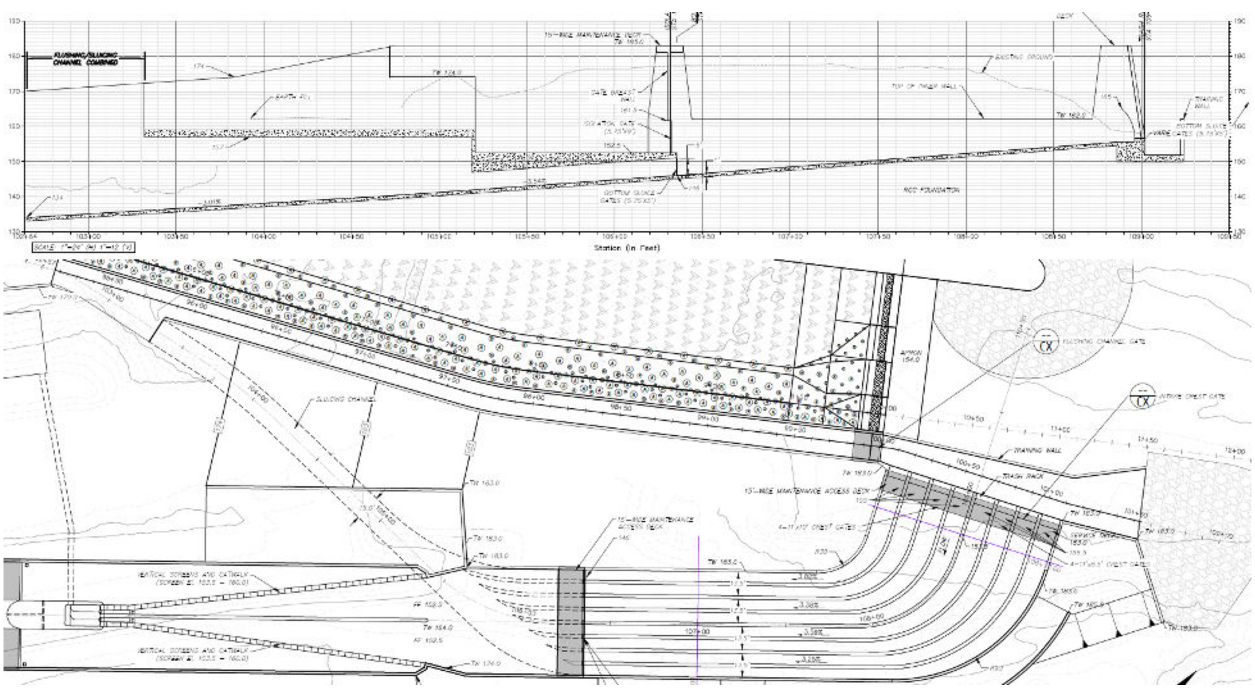


Figure 7-9 Plan view and profile of desander and sluice channel.

7.5.2 Normal diversion operation and sediment sluicing

A desander bay has two main modes of operation: normal diversion operation and sediment sluicing (Figure 7-10). During normal operation, the bottom sluice gate remains closed, while the canal gate is fully open. Flow through a bay continues downstream towards the canal connecting to the fish screens, while sediment continuously settle within the bay. Once a bay is filled with sediment and it is time to sluice, gate opening is reversed. Flow towards the fish screens is interrupted by closing the canal gate while the sluice gate is open. Flow is redirected through a bottom culvert underneath the canal and towards the sluicing channel and downstream river. The red arrows in Figure 7-5 indicate the path of sediment during sluicing. Inflow to the bays is controlled by the intake gates during both normal operation and sluicing.

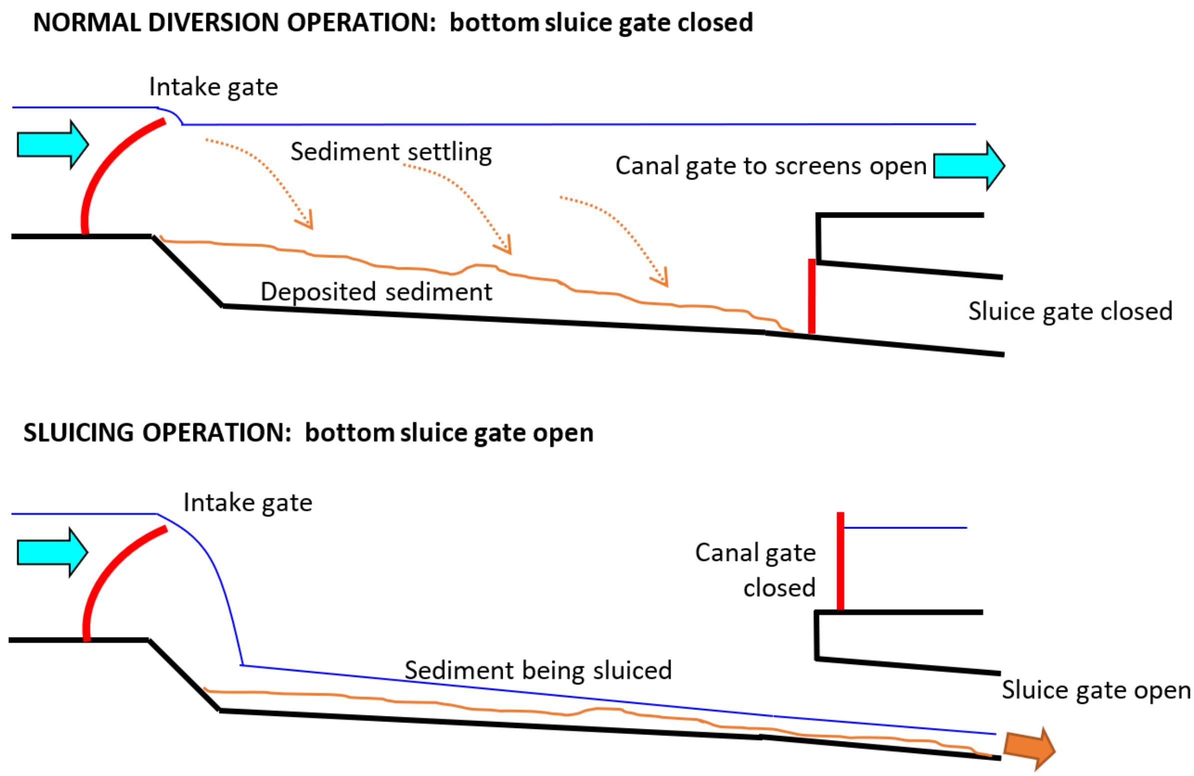


Figure 7-10 Typical operation of desander bay.

Having 8 bays provides increased flexibility in the operation of the desander, which can adjust to changes in diversion discharge, sediment loads in the river and the rate of sediment infilling in the desander. Typically, one bay will be sluicing while the remaining seven bays continue diverting water downstream. Once the bay being sluicing is cleaned, it will return to normal operation and the next bay will enter sluicing mode, continuing cycling through all bays.

7.5.3 Trap efficiency and sluice ratio

Important design parameters for a desander are the sediment trap efficiency during normal operation and sluice ratio during sluicing operation. The sediment trap efficiency of the desander during normal diversion operation is defined as the percent of incoming sediment that is trapped within the desander. The trap efficiency varies with grain size and discharge, being higher when sediment is coarser and discharge lower. Figure 7-11 shows the estimated sand trap efficiency curves for various sediment sizes and diversion discharges. The solid curves were estimated using analytical methods (Garde and Ranga Raju, 1977) and the red dots using a CFD model. According to these desander trap efficiency estimates, all sediment coarser than 0.5 mm will be trapped and at least 65% of sediment coarser than 0.25 mm.

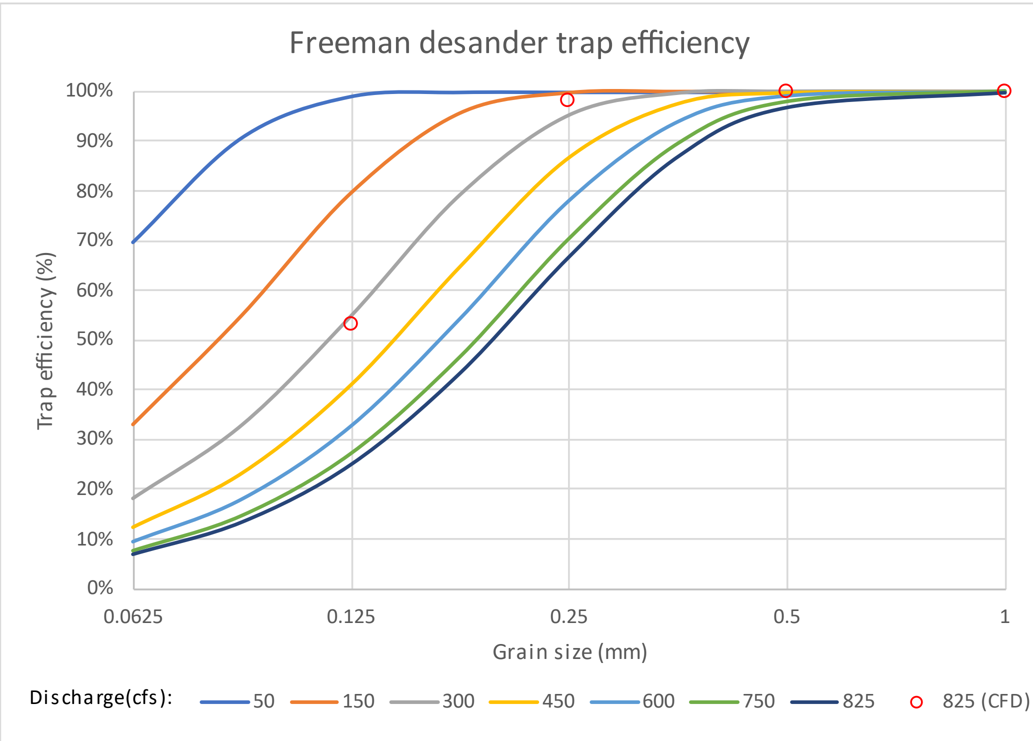


Figure 7-11 Estimated desander trap efficiency by grain size and diversion discharge.

The volumetric ratio between sluiced sediment and water, or sluice ratio, can be used as a measure of sluice efficiency:

$$Sluice\ ratio\ (SR) = \frac{Sluiced\ sediment\ volume}{Volume\ of\ water} = \frac{Sluiced\ sediment\ volume}{(sluice\ discharge)(sluice\ time)}$$

$$SR = \frac{Vol.}{Q_s T_s}$$

Where:	SR	sluice ratio
	Vol. (ft^3)	bulk volume of sediment sluiced from desander bay
	Qs (ft^3/s)	sluice discharge
	Ts (s)	sluice time.

A higher sluice ratio means that more sediment can be sluiced using the same amount of water, indicating a more efficient sluice operation. Table 7-2 shows sluice ratios observed in scale physical models of three desanders built in Peru that were designed to divert flows between 640 and 3,200 ft^3/s (IHHS, 1989, 1990, 1992). The sluice ratios in the models ranged from 0.2% to 1.8% (due to scale effects the real sluice ratio in the prototype is expected to be higher). It was observed that sluice ratio was not very sensitive to the sluice discharge, but it was sensitive to the invert slope. Increasing the slope of the desander bays and the slope of the sluicing channel led to higher flow velocities and hence higher values of sluice ratio. However, the maximum slope that can be achieved in practice is typically constrained by the available hydraulic head difference between headwater and tailwater levels.

Table 7-2 Sluice ratios observed in physical models of 3 Peruvian desanders (IHHS 1989, 1990, 1992).

Desander	Physical model scale	Diversion discharge (ft^3/s)	Desander bay slope	Sluicing channel slope	Sluice ratio range
Chavimochic	1:15	3,200	2.2%	>3%	1.25%
Yanango	1:30	640	3%	3%	0.9-1.4%
San Gaban	1:45	700	3%	1.0%	0.2-0.4%
				1.5%	0.4-1.1%
				2.5%	0.9-1.8%

The main limitation for the successful operation of a desander is the hydraulic head available for sluicing sediment back to the river downstream. The head is mainly controlled by downstream tailwater levels. As tailwater levels increase, the amount of water flow (sluice discharge) needed to sluice a given volume of sediment typically increases, i.e., the sluice ratio decreases.

7.5.4 Design Development

The design and performance of the Freeman Diversion desander was greatly improved during design development. Several versions of the desander system were tested in the 1:24 physical model (see Table 20 in USBR, 2022). The initial versions considered four desander bays with inlet elevation at 156.5 ft and slopes varying between 2.3% and 3.3%, a 3 ft high bottom sluice opening and sluicing channel with 1.6% slope.

In agreement with observations in other desanders, the low invert slopes of the desander and sluicing channel was causing a low sluice efficiency, requiring long sluice times to clean the bays (low sluice ratio). Additionally, the 3 ft high sluice opening was considered too small. For the final version, the number of bays was doubled to eight with steeper slopes between 3.3% and 3.8%, the sluicing channel slope increased to 3.0% and the height of the sluice opening increased to 5 ft. Also, the inlet invert of bays 4 to 8 was lowered by 1.5 ft to El. 155 ft to improve flow diversion capacity during low flows.

7.5.5 Physical Model Testing

USBR (2023) used the 1:24 model of the desander to stress test the sluice operation of the desander under various flow conditions. Results are shown in Table 7-3 and Figure 7-12. Most sluice tests were conducted in the longest bays 1 and 2, but there were a couple of tests in bays 3 and 4 (test 2.9a). Figure 7-12 shows longitudinal profiles of the desander invert, initial bed profiles measured at 5 points, values of tailwater levels (TW), volume of sediment sluiced (Vol.), sluice discharge (Qs), sluice time (Ts) needed to completely clean the bays, and computed sluice ratio (SR). Most tests were conducted for a river discharge around 3,000 ft^3/s (Table 7-2), with some tests conducted at 1,500 ft^3/s and 6,000 ft^3/s.

In agreement with observations in other desanders (Table 7-2), the sluice ratio in the desander seems to be rather insensitive to river discharge or sluice discharge, but quite sensitive to tailwater levels. Excluding the last data point from Table 7-2 (Test 2.9a), Figure 7-13 shows how the sluice ratio for bay 1 decreases with increasing tailwater levels. For each foot that tailwater levels increase above El. 144 ft, the sluice ratio drops roughly by 0.1%. Based on the linear trend shown in Figure 7-13, the sluice ratio appears to vanish when tailwater levels exceed El. 154 ft, which is reasonable because that elevation is close to the upstream invert elevation of bay 1Figure 7-12, meaning there is not enough head available to drive the flow and overcome the head losses in the system.

Except for Test 2.9a (Table 7-3) sluice ratios vary between 0.2% and 1.5%, which is within the range observed in other desanders (Table 7-2). Notice that although tests 2.9 and 2.9a have the same river discharge and tailwater level, the sluice ratio for test 2.9a is almost one order of magnitude lower, despite having much higher sluice discharge. The reason is that in order to increase sluice discharge for test 2.9a, the intake crest gate was fully lowered, causing sediment deposited on the intake apron to be ingested into the desander. The additional sediment ingested by the desander required far longer time to be fully cleaned, increasing the sluice ratio. The other tests were run after the intake apron had been flushed. It is recommended to flush sediment from the apron using the flushing channel to prevent the desander from being overwhelmed and its sluice ratio to drop.

Table 7-3 Sluice ratios observed in 1:24 physical model of Freeman desander (USBR, 2023).

Test	River discharge (ft^3/s)	Tailwater level (ft)	Sluice discharge (ft^3/s)	Sluiced volume (ft^3)	Sluice ratio Bay 1	Sluice ratio Bay 2
2.1	2,800	144.5	181	12,900	0.87%	0.83%
2.2	2,930	145.1	232	11,400	0.84%	0.78%
2.3	2,700	144.6	338	14,300	0.80%	0.80%
2.5	3,025	151.3	256	12,000	0.23%	0.24%
2.6	2,860	152.0	320	14,700	0.23%	0.22%
2.3a	3,100	144.3	494	15,300	1.05%	-
2.7	1,580	143.4	316	1400	0.77%	1.10%
2.9	6,100	145.0	189	3000	0.90%	1.56%
2.9a*	6,100	145.0	940	1400	0.10%	0.13%

* Intake gate was fully lowered to increase sluice discharge, allowing bed deposits from the apron to be ingested by intake. Results shown are for bays 3 and 4, not 1 and 2.

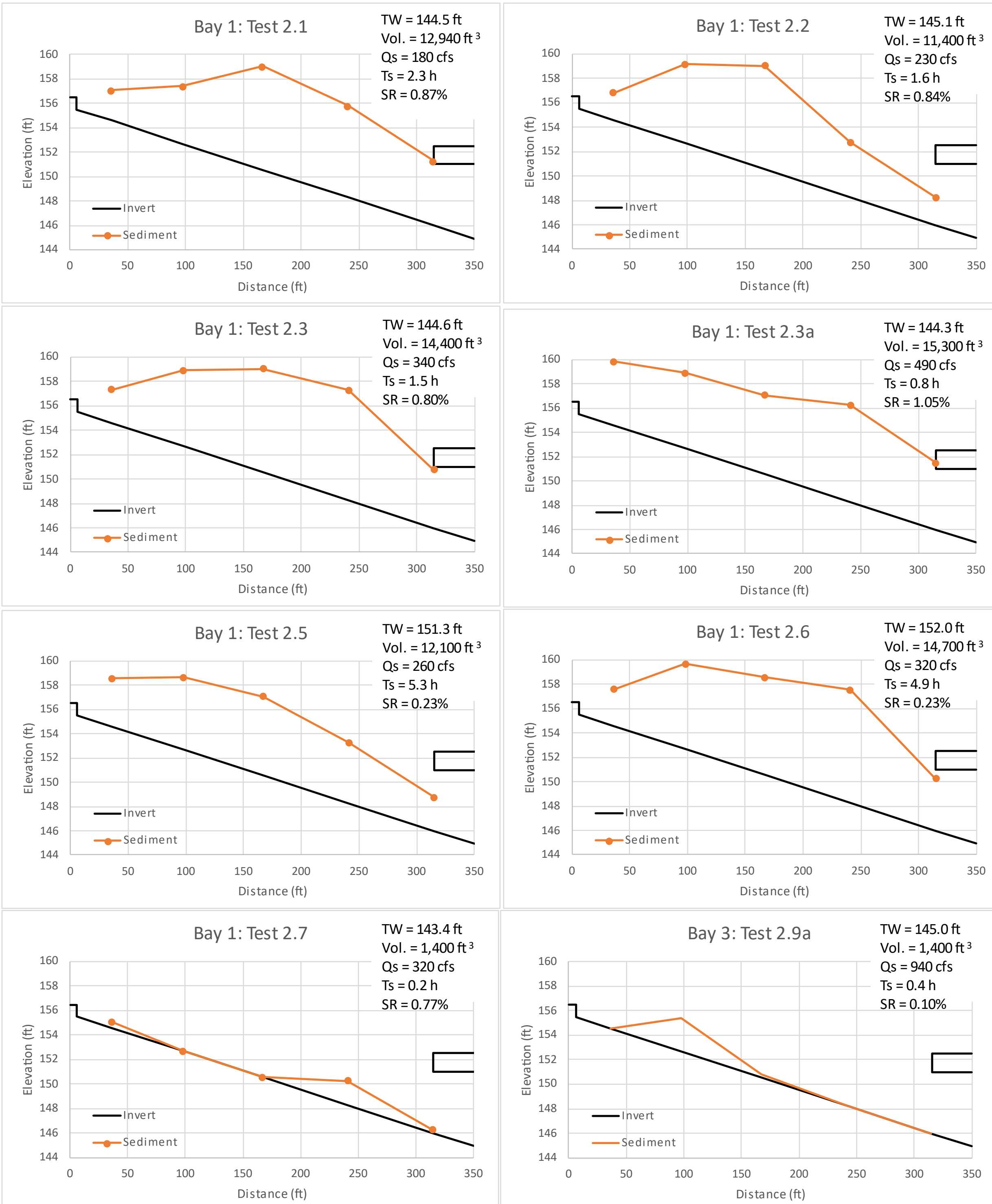


Figure 7-12 Flow conditions and initial bed profiles along desander bay measured in 1:24 physical model. TW = tailwater level, Vol. = volume of sediment sluiced, Qs = sluice discharge, Ts = sluice time, SR = sluice ratio.

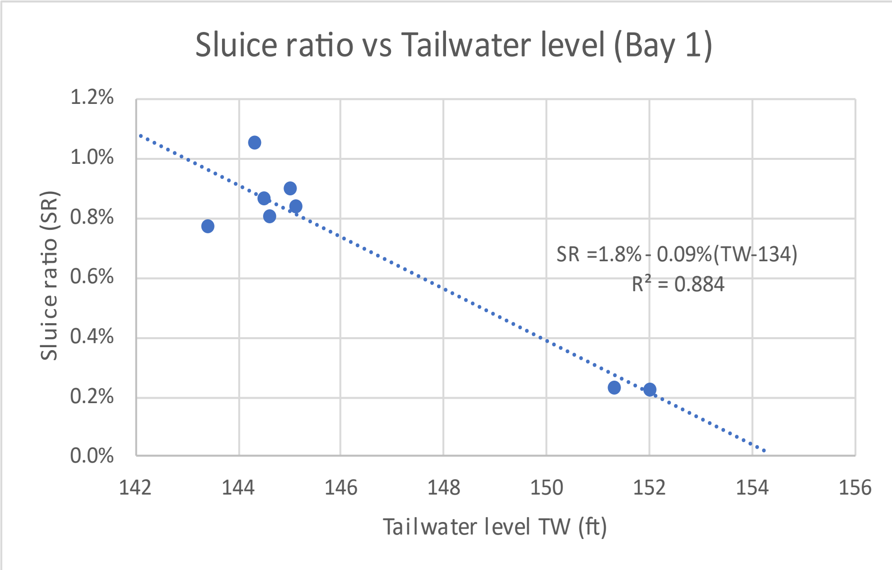


Figure 7-13 Decrease in sluice ratio caused by high tailwater levels.

Because the flushing and sluicing channel inverts are set at the same elevation as the hardened ramp, these systems have a similar flexibility allowing for them to fill in during times of higher tailwater and then flushing clean during high discharges, see Figure 7-14. The reduction in the desander sluice ratio observed at high tailwater levels (Figure 7-13) is due to sedimentation along the sluicing channel (Figure 7-14), which reduces the effective slope of the sluicing channel. As shown in Table 7-2 (San Gaban desander), reducing the slope of the sluicing channel causes a reduction in the sluice ratio.

Figure 7-14 Flushing and Sluicing Channels in the 1:24 scale model.

7.6 Sediment Management Operations

The frequency of sediment removal operations, such as opening the flushing channel and sluicing the desander, will be highly variable as it will depend on the future hydrology conditions and sediment loads in the Santa Clara River (Figure 7-2). Sediment removal operations will be more frequent when sediment loads in the river are higher, which typically occurs during high river flows.

7.6.1 Flushing Channel

Although, due to scale effects in the 1:24 physical model, it is not possible to determine the times needed to flush sediment, they can be roughly estimated. The total volume of sediment flushed during the physical model tests is unknown, but from the profiles in Figure 7-6, the volume (prototype) of sediment flushed from the apron in front of the intake was in the order of 10,000 ft^3. Considering the additional sediment removed upstream (Figure 7-7), the total volume flushed was probably at least double that number. Figure 7-6 shows that in the 1:24 physical model it is possible to clean the apron in a short duration, which likely corresponds to less than one hour in the prototype. Hence the duration of the flushing channel operation is expected to be short, being able to remove probably at least 20,000 ft^3 of bedload per hour of operation, guaranteed for tailwater levels up to 152 ft (and probably higher).

Because the flushing channel provides a straight and shorter path for sediment (Figure 7-5) and can pass higher discharges, it can remove sediment much more effectively than the desander, i.e., higher sediment volumes in shorter operational time. This is good not only for sediment management but also to limit the impacts on fish and diversion (the diversion must turn out during sediment flushing). The latter is not a concern for the desander, which can sluice sediment while diversion continues.

The flushing channel can be used at high flows to remove sediment and prevent it from entering the system and piling up during larger flows with higher tailwater conditions. The flushing channel can remove a larger gradation of material than the desander which is behind the trash rack which limits sediment that enters to 6" or less. The flushing channel reduces the volume of bedload (typically sand) from piling up onto the gates and being suspended into the intake; which keeps the intake cleaner and reduces the frequency of use of the desander.

7.6.2 Desander

For the desander, assuming a tailwater level at El. 145 ft and sluice ratio around 0.8% with a sluice discharge around 200 ft^3/s, the desander could sluice about 6,000 ft^3 of sand from a bay in about one hour. However, if tailwater levels rise to El. 152 ft and sluice ratio drops to 0.2% (Figure 7-12), the volume sluiced will reduce proportionally to only 1,500 ft^3 per hour. According to the tailwater rating curve used in the physical model (Figure 9-1), a tailwater level of El. 152 ft typically corresponds to a river discharge around 18,000 ft^3/s, although one data point from 2022 showed a tailwater level near El. 151.5 ft for a discharge of 3,000 ft^3/s, so there is a very strong natural variability (see Section 9.4).

During extreme sediment load events, it may be possible that one or two desanders bays are being sluiced at any given time, sequentially cycling through all bays. However, the operation of the desander may be limited in practice during high flow events that generate high tailwater levels that excessively reduce the sluice ratio (Figure 7-13).

The desander is most efficient at lower tailwaters. Ideally the desander is used to clean the system at lower flows where the tailwater allows for free surface flows (below El. 150 ft). The desander can cycle through bays to continue diversion, which is an advantage over the flushing channel that typically requires the diversion to turn out. The operation of the desander has no hydraulic impact on the hardened ramp.

7.6.3 Flushing channel and desander

NMFS and CDFWs have expressed a preference for sediment management without the flushing channel. However, sediment removal operations of the flushing channel and desander complement each other (Table 7-4).

Table 7-4 Main features of the flushing channel and desander.

Item	Flushing channel	Desander
Main purpose	Remove bedload to prevent it from entering intake	Remove sediment that entered intake. Prevent sedimentation of screen bays
Sediment size	Bedload, coarse material	Suspended sand > 0.5 mm bedload < 150 mm (6”)
River flow range	High > 3,000 ft^3/s	Low, when TW < 150 ft
Diversion	Interrupted	Decreased, but possible by cycling through bays
Effect on ramp	When it operates, it decreases flow velocity and depth on ramp	None

Not having a flushing channel would be highly problematic for several reasons. The desander collects both suspended sediment and bedload small enough to pass through the 6” opening trash racks. Any larger material (above 6”) will collect on the apron indefinitely in front of the trash rack, unless removed mechanically after floods recede (the ramp will be ineffective to move such coarse sediment).

The desander will fill faster without the flushing channel to remove the volume of material in front of the gates during high flows, and it is inefficient to ineffective at high tailwaters. Requiring a large amount of water to provide less efficient sluicing. This is an issue for Freeman Diversion where the downstream water levels vary depending on factors not controlled by the diversion design (Figure 4-3). Without the flushing channel, the desander would likely become overwhelmed by sediment losing its ability to maintain the fish screens and fish bypass pipes clean and functional.

Similarly, having only a flushing channel without a desander would be problematic. The flushing channel would not handle sediment that makes it into the intake, which was shown to be plentiful from baseline (Figure 7-4) and other testing. The flushing channel would need to be utilized for a larger range of flows to keep the apron clean, since the intake would only be cleaned mechanically. In summary, the combination of the flushing channel and desander provide the most flexibility in operation and sediment management and both should be implemented.

8 Debris Management

The potential for woody vegetation entrained in the Santa Clara River to negatively impact the operation of the new diversion and hardened ramp structure was analyzed. Tests were completed in the physical model to determine the potential impact of debris (USBR, 2023). This section reviews the potential types of floating debris at the ramp, options for managing debris, results from the model tests and recommendations.

8.1 Debris Characterization

UWCD has documented floating debris types and loading rates in the vicinity of the Freeman Diversion for a variety of discharge scenarios. Open-File Report 2021-01 (United Water Conservation District, 2021) indicates that the primary sources of floating vegetative debris in the Santa Clara River by the diversion are from watercress, cattails, mulefat, cottonwoods, willows, and Arundo. Debris type in the river generally depends on the storm size and subsequent river discharge.

Table 8-1 presents the debris characteristics and loading rates as a function of discharge as presented in the Open-File Report 2021-01. The table indicates that three main mixes of debris should be considered in the physical model: (1) a combination of Arundo and riparian vegetation only, (2) Arundo and vegetation with some trees, and (3) large trees with some Arundo. It is noted that trees simulated in the model should include a rootwad on one end. Examples of these three debris mixes are shown in Figures Table 8-1 to Figure 8-3.

Table 8-1 Debris characteristics as a function of discharge recommended for the physical model (source: United Water Conservation District, 2021).

	Test Flow Cubic Feet Per Second (cfs)			Type and Size of Debris						
Flow Category	Total River	Ladder/ Ramp/ Crest	Diversion	Floating or Neutral	Tree Size	Arundo	Smaller Mix Vegetation	Ratio Tree/Arundo/ Mix veg.	Frequency/ Volume of Debris Expected	Recommended Application
Low Flow Storm	800	425	375	Floating	no	yes	watercress/ cattails	0/40/60	30 sec.	Mass 4' by 20'
Low Flow Storm	1,500	1,000	500	Floating	no	yes	watercress/ cattails	0/40/60	30 sec.	Mass 4' by 20'
Medium Flow Storm	6,000	5,250	750	Floating	15'	yes	4” chips	30/40/30	30 sec.	Small mass of debris
Medium Flow Storm	30,000	30,000	0	Floating	15'-20'	yes	4” chips	60/30/10	singular Added 1 per min.	Large mass of debris followed by singular
High Flow Storm	70,000	70,000	0	Neutral	15'-30'	yes	4” chips	60/30/10	3 large pieces of debris/ sec	Singular application

Figure 8-1 Mix of Arundo and riparian vegetation.

Figure 8-2 Arundo and vegetation with some small trees.

Figure 8-3 Mostly large trees and some Arundo.

8.2 Mitigation Options

NHC reviewed potential debris management and mitigation options prior to the physical modeling. Options considered are given below.

8.2.1 Operations of Gates

Smaller debris caught near the diversion inlet and on the hardened ramp baffles could be dislodged by deliberate operation of gates at the inlets of the flushing channel and hardened ramp. It is assumed that operation of the gates could potentially remove Arundo and smaller trees but may not be appropriate for larger trees.

8.2.2 Diversion Inlet Floating Boom

Figure 8-4 presents the conceptual layout of a floating shear boom that would deflect debris past the diversion inlet and towards the low flow channel of the hardened ramp. The approximately 100-foot boom would consist of 5 to 10 buoyant 10 to 20-foot segments with diameters of 24 inches that are connected to form a catenary tensioned structure that is anchored at each end. The location of the boom end points is critical. In general, shear booms will only be effective at an angle of 30 degrees or less to the flowlines. Shear booms are not designed to retain debris but to shunt it away or direct it to an area designed for collection and removal. The shearing action minimizes debris loading, forces in the tension elements and anchor requirements. The Odin Boom product recommended by Tuffboom for the project is shown below in Figure 8-5.

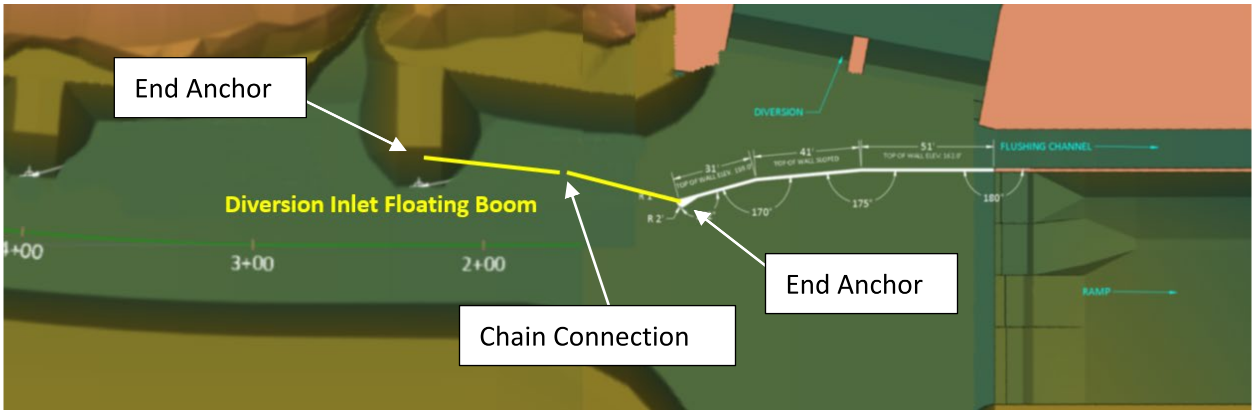


Figure 8-4 Plan view of Diversion Inlet Floating Boom Concept.

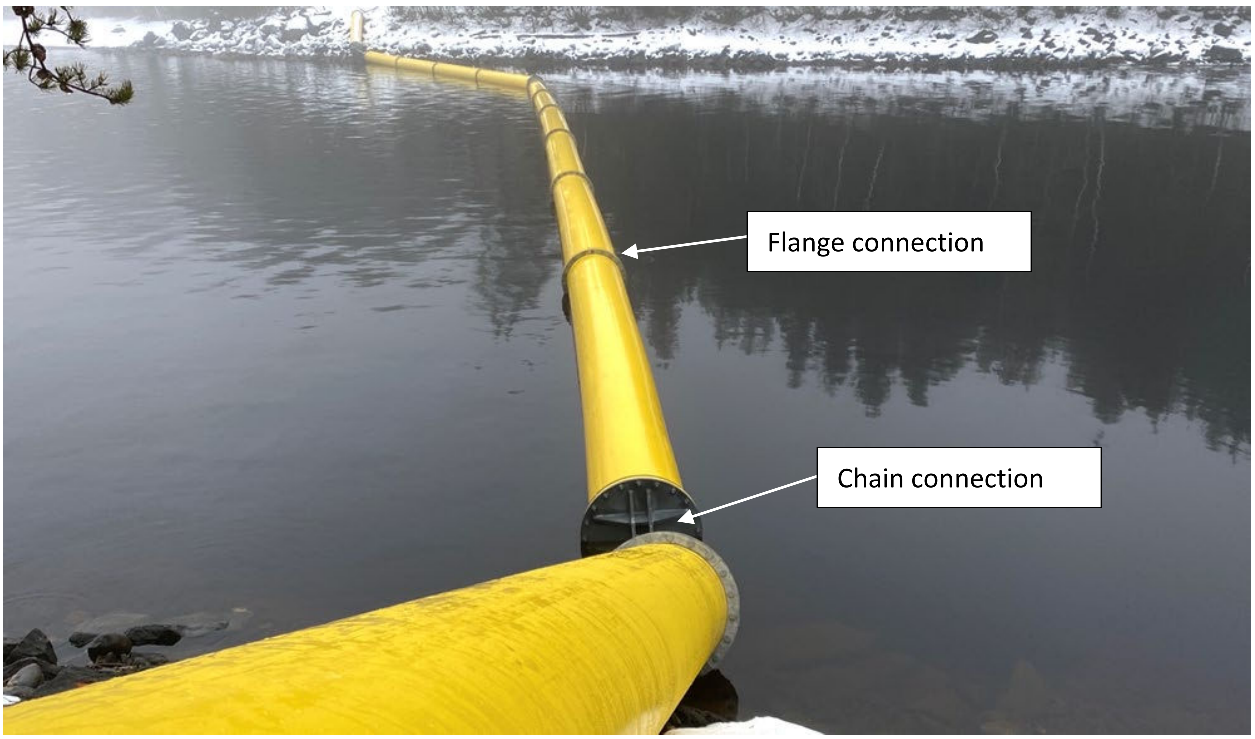


Figure 8-5 Odin Boom recommended by Tuffboom for use as a floating shear boom for the project

8.2.3 Diversion Inlet Fixed Boom

A second mitigation option for protecting the diversion inlet only could include application of a fixed shear boom instead of a floating boom. The fixed boom would also span from the farthest downstream groin to the end of the training wall and be permanently attached to columns spaced about 20 to 30 feet apart. The boom would be ridged, 3 feet deep, and float between elevations 156 feet and 166 feet. Figure 8-6 presents a photo example of a fixed boom structure.

8.2.4 Inlet and Ramp Floating Boom

A second floating boom alternative could be tested that spans both the diversion inlet and hardened ramp, see Figure 8-7. The approximately 450-ft boom would be constructed using a product like the Odin Boom that is connected to anchors at each end. The end anchors would be located on the second to farthest downstream groin and on the hardened ramp wall adjacent to the dam crest. An additional mid-section anchor would be connected to the boom and anchored in the bed to help maintain the shape of the boom and reduce loading on the end anchors. The position of the mid-anchor in the structure should be selected such that the average angle of each boom section is within the 30-degree tolerance for shearing floating debris.

Figure 8-6 Example of constructed fixed boom structure.

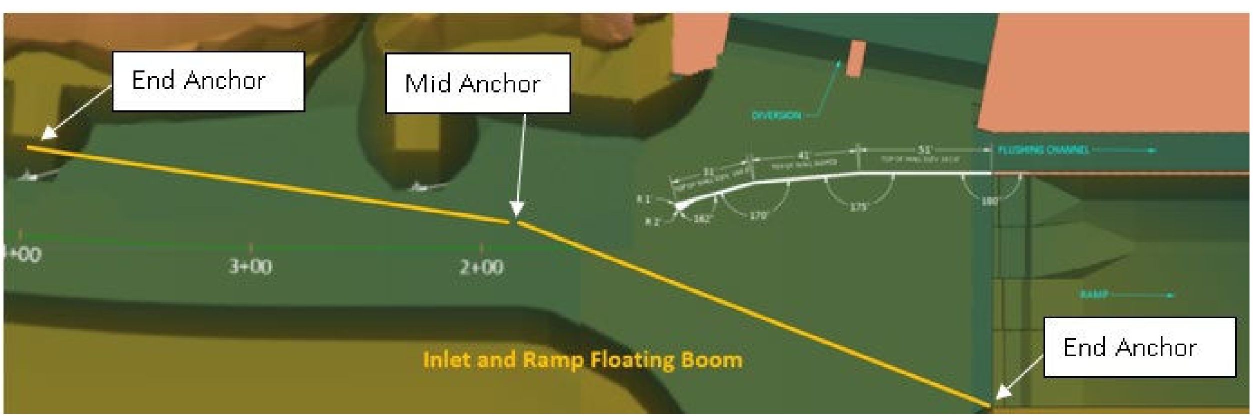


Figure 8-7 Plan view of the Inlet and Ramp Floating Boom Concept.

8.3 Physical Model Testing

Testing on the physical model was completed with no debris mitigation structures other than the diversion inlet trash rack. The tests for these conditions were designed to better understand the behavior of debris passing through the system and to identify locations where debris may collect or impact operations of design components. Testing included in a range of flows, with debris both fed from upstream to verify potential for accumulation on the hardened ramp baffles, and manually placed on the baffles to assess hydraulic impacts of debris on hardened ramp flow. The model debris included both Arundo, simulated using hay, and large woody debris, simulated using wooden dowels and sticks of a range of sizes.

A full write-up of the debris testing for both the 1:12 and 1:24 models can be found in the USBR report (USBR, 2023). The key findings from that report are summarized below.

From the 1:24 Scale model:

- When debris was first introduced at 6,000 ft^3/s, the only point of accumulation was on the right side of the hardened ramp where the baffles were exposed.
- Debris that was inserted by hand directly onto the baffles was only retained on the right side of the ramp.
- When flow was ramped from 6,000 ft^3/s to 12,000 ft^3/s, all debris fed from upstream was passed, and debris that had accumulated on the hardened ramp was dislodged and passed downstream.
- When flow dropped back to 6,000 ft^3/s, newly added debris plugged the intake structure trash rack, severely reducing diversion capacity.

From the 1:12 Scale Model:

- Debris remained on the right portion of the baffled section of the hardened ramp, as was seen in the 1:24 physical model; debris along the left side of the hardened ramp did not accumulate due to increased velocity as the ramp transitions to the low flow section, see Figure 8-8 .
- Point velocity and depth measurements collected upstream, adjacent to and downstream of debris mats show minor increases in depth and velocity but these were localized around the debris accumulations and did not affect the overall fish passage hydraulics of the ramp.
- The range of flows where the hardened ramp is more susceptible to debris accumulation appears to be relatively small, 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 increase.
- Debris accumulation on the intake trash rack significantly reduced diversion capacity.

Figure 8-8 Physical Model Debris Test Results with Arundo (Hay) stuck on the Right Side Baffles.

8.4 Debris Management Operations

The physical model showed that most woody debris and Arundo was cleared downstream and did not accumulate on the hardened ramp. The right portion of the hardened ramp was susceptible to accumulation of Arundo clusters on baffle tips at lower flows. This debris accumulation had minor, localized impacts on flow depth and velocity. Woody debris accumulation was rare and would often clear from the baffled area during changes in discharge. The main area of concern identified in the physical model testing was debris accumulation on the trash rack.

Physical model testing indicated that it would be appropriate to clear any debris accumulated on hardened ramp baffles following the completion of the high flow event, and that measures to divert of trap debris, such as a debris boom or other screening system, are not warranted and will be challenging to implement. The minor, localized impact of debris caught on baffles supports that debris can be left in place for the duration of a high flow event without impeding the general function of the hardened ramp. A debris structure placed in the river is likely to retain debris that would have otherwise been passed downstream by flows.

The trash rack will have a cleaning system that was not included in the physical model, which will help to keep the structure clean. Additionally, mechanical removal of debris from the trash rack can take place from the facility using large equipment such as a long reach excavator, if required.

The inclusion of a structure such as a floating debris boom that would collect debris that currently passes downstream is not recommended at this time. A structure intended to divert debris to the low flow channel is likely to become fouled with Arundo. Accumulation of debris on debris retention structure may create an issue at higher flows, when the transport capacity for the Santa Clara River is great and most large material is transported; the collection of debris can create additional maintenance raise water surface elevations, and increasing flooding potential. It is likely more beneficial to UWCD to work to pass debris instead of capturing it.

We suggest that UWCD start with the existing design and have equipment and staffing available for physical removal for post storm clean-up. Over time, debris passage performance can be monitored and, if necessary, secondary options such as a floating debris boom could be implemented in the future.

Maintenance of the ramp is anticipated following flood events. Maintenance would include inspection of the ramp and baffles and removal of debris. Due to the width of the ramp, some sediment and debris accumulation may be tolerable, and debris removal might be required only on an annual basis or after very large events, depending on the success of the ramp design in shedding debris. Routes for equipment to access the ramp in the dry, will be defined further as the design advances. Maintenance paths for access across the ramp during flood events were explored but not deemed operationally feasible at this time.

9 Design Resilience and Stability

9.1 Future and Current Flood Design Elevations

The facility was designed for the 100-year Flood Elevation plus 3-ft of freeboard. This water level was developed using a 2D model for a larger river reach (see Appendix C). The 2D model has been utilized in previous phases and was updated to reflect the most recent project features. The 100-year Ventura County estimate of 226,000 ft^3/s was used, which is greater than the existing FEMA 100-year mapped recurrence. The results with the crest gates up show a minor raise in water surface elevation (compared to existing condition) immediately upstream. For the 100-year design event the gates should be kept down as to not negatively impact the upstream water surface elevation.

A 100-year water level of 180 ft was utilized to represent the conditions at and upstream of the dam and an elevation of 166 ft for downstream of the dam. With freeboard considerations that translated into an elevation of 183 ft that was used for the exterior walls, walls around the desander bays that are used to isolate the facility from the events and for the isolation gates. The walls at the downstream end of the facility would be set to 170 ft, similar to the existing design height.

The model is based on existing river topography. Updated LiDAR was recently collected by the County which should be available in the near future. The results are very dependent on the surrounding topographic conditions. It is recommended that the model geometry be updated to reflect the most recent topography and roughness values/assumptions are updated to reflect any changes in conditions as part of the subsequent design phases. These results can then be used to confirm the appropriateness of the water levels computed and corresponding wall heights. Prior to construction/completion FEMA should be engaged and a no-rise analysis should be completed to meet regulatory flood mapping requirements.

9.2 Climate Change

Beginning in late 2022 and continuing through the first several weeks of January 2023, the Santa Clara River watershed was subjected to a series of atmospheric rivers. The Santa Clara River and the Vern Freeman Diversion was subjected to significant high flow conditions. United and its consultants are continuing to analyze the impacts of these recent high flow conditions on the current design of the Hardened Ramp, including whether additional modeling and/or design work are necessary.

NOAA fisheries has recently released guidance on how to improve the resilience of fish passage facilities to climate change (NMFS, 2022b). The document gives guidance on how to incorporate resilience to climate change into designs to reduce risk to anadromous fish species and ensure a facility will function successfully over the design life of the facility. Updated estimates of the design flood event will likely raise the height of some physical feature of the diversion. The review will include downscaled hydrological data to assess a range of potential future conditions and their implications. The areas of impact will be more focused on peak flows in the system as well as other potential changes to the geomorphology and sediment supply from increased precipitation, fire and drought.

Consideration of drought duration may influence vegetation dynamics affecting channel movement and sediment dynamics, particularly in the tailrace. It is recommended that this updated modeling be done in conjunction with new LiDAR topography and roughness estimates of the system. The hardened ramp design has incorporated features that provide flexibility to changes in conditions and flow splits, this resilience should help with adjusting to future climate conditions. Additionally, the hardened ramp design is resilient to tailrace aggradation.

These topics will be discussed further with the regulatory agencies prior to incorporation of any design changes.

9.3 Effects of Bedload Transport

The CFD model and 2D flood model for proposed conditions were used to evaluate velocities on the Hardened Ramp for the purpose of assessing stability of the ramp materials. During both fish passage and flood flows, high velocities are generated on the ramp surface. Velocities over 20 ft/s were simulated, and these flows will likely be accompanied by sediment and debris.

Very large rock sizes would be required to provide stability against modeled velocities if placed as an engineered streambed material in the low flow roughened channel portion of the ramp. These sizes would be difficult to obtain and place in a configuration that provides reliable hydraulics for fish passage. For these reasons, the entire ramp was designed as a grouted rock surface and baffles are envisioned to be constructed of heavy plate steel embedded to concrete sills, leading edges and surface exposed to high velocities near the bed. Wear plate design will be undertaken as part of further design.

The durability of the rock within the low flow channel is a critical element in the design. UWCD has provided information on the quantities, quality and costs of rock proposed in the Hardened Ramp design. Pending detailed design, the evaluation of large rock as roughness elements in the Low Flow section will be evaluated as more information is available, and if required, alternative solutions developed. This would include material selection, anchoring and constructability, followed by modeling and evaluation.

The steel baffles are expected to deform under some conditions when absorbing impact energy from debris and resisting bedload erosion in the baffled section of the Hardened Ramp. Deformation of the steel baffles is preferred over use of a more rigid material such as concrete that is likely to spall or fracture under similar impacts. Small deformations are unlikely to affect performance of the baffles, and corrosion of the baffles can be reduced by selection of an appropriate alloy. Corrosion is expected to be a rate that provides service life of at least 30 years. This also avoids the concern that spalling or cracking of concrete elements could lead to water intrusion and corrosion of reinforcing steel. The steel baffles at the ramp crest can be designed to fail at the connection to the sills if subjected to a very large impact force, such that they could be replaced or repaired, if needed, without structural modification of the ramp.

9.4 Project Tailwater Conditions

Tailwater conditions at the ramp were defined in the physical model based on available historic information collected at the site by UWCD, see Figure 9-1. The plot was provided based on best available data to USBR and the fit curve (red dots) was used for the physical modeling completed, see USBR (2022), USBR(2023).

Long-term changes in the downstream tailwater is a dynamic variable that can change with the river geomorphic conditions. The tailwater may raise if there is increased vegetation downstream or deposition of sediment from flooding or sediment routing. The design should be reviewed if these conditions arise. The tailwater may decrease if the more concentrated flow results in additional channelization and local incision. The hardened ramp design provides flexibility for reasonable variations after events and from long-term processes. If there are channel altering events (aggradation or degradation) site specific O&M adjustments may be required.

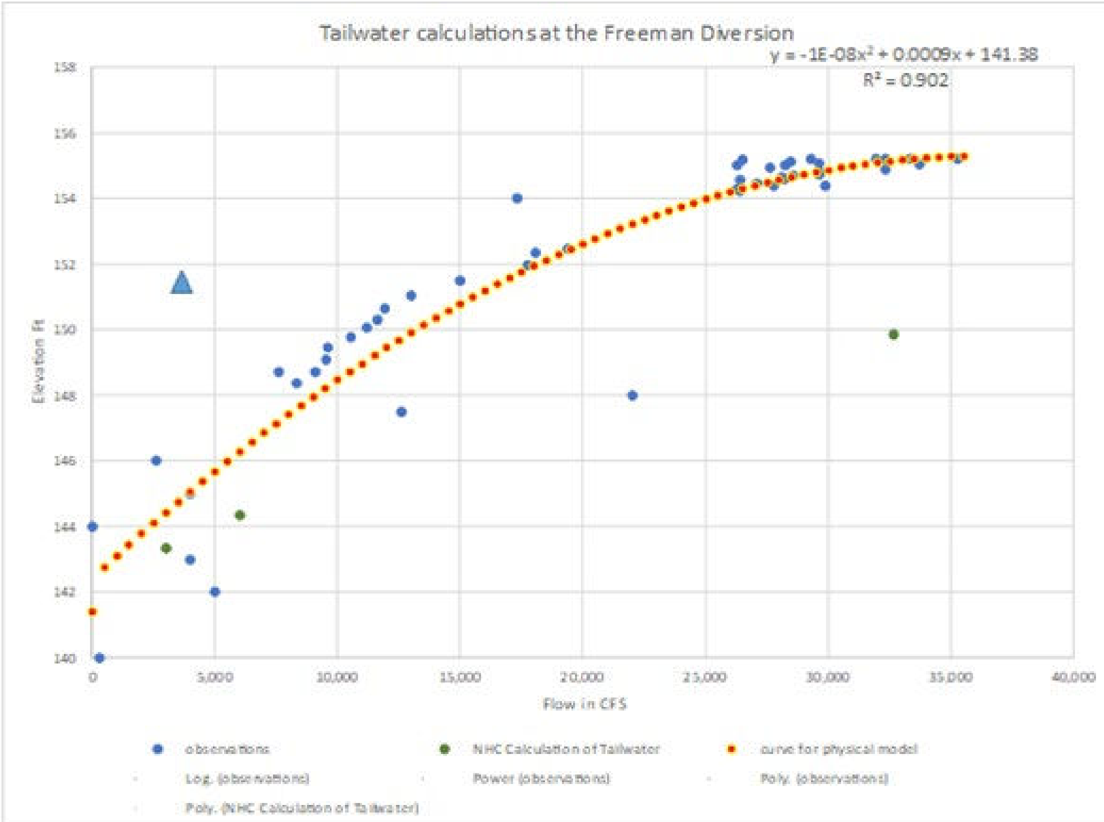


Figure 9-1 Tailwater Curve from collected data at freeman Diversion (from UWCD).

9.5 Diversion Flexibility

The diversion was designed to provide flexibility in operations. Features such as: eight crest gates (at varying elevations), training wall bulkhead, downstream canal gates and two screen bays provide maximum operational flexibility to divert a range of flows while meeting acceptable fish screen criteria requirements. The flexibility in sediment management (Section 7) is critical to keep these functions operational. The operational flexibility is important to provide options so that the facility can meet the defined diversion and fish passage needs for a range of future scenarios.

10 Operations

UWCD has requested an increase in diversion capacity for the facility to 750 ft^3/s (existing capacity is 375 ft^3/s) and that the facility should be designed to operate using the existing gravity canal system downstream, considering planned system improvements (NHC, 2016c). The increased diversion capacity allows flow to be diverted more rapidly during high river discharges. The 750 ft^3/s capacity was used for the design and analysis of the structure.

Future operations and flow splits are still being developed through the Multi-Species Habitat Conservation Plan (MSHCP). In the absence of established operations and flow splits the design focused on operational flexibility to get up to the full diversion flow of 750 ft^3/s for a range of conditions. The intent is for the project design to not be the limiting factor in future flow splits.

10.1 Diversion Operations

Diversion operations are controlled by a series of gates at the intake, entrance to the screen bay channel and the canal. A description of these gates is given in Section 6. The following primary controls are used to set operations.

- Primary control for diversions – Intake Crest Gates
- Primary water level control in screen bay –Canal Gates
- Isolation gates for flood protection – Desander Canal and Bottom Sluice Gates.

The design provides operational flexibility by having the invert of the intake crest gate at two elevations of 156.5 ft and 155 ft. This increases the maximum diversion capability, which in practice can be reduced to desired operational flows through regulation with a combination of the overshot intake crest gates and the regulating canal gates downstream of the fish screens. To further describe possible diversion operations, diagrams were developed to indicate operational water levels at example river flows. The general operational features are given below in Figure 10-1.

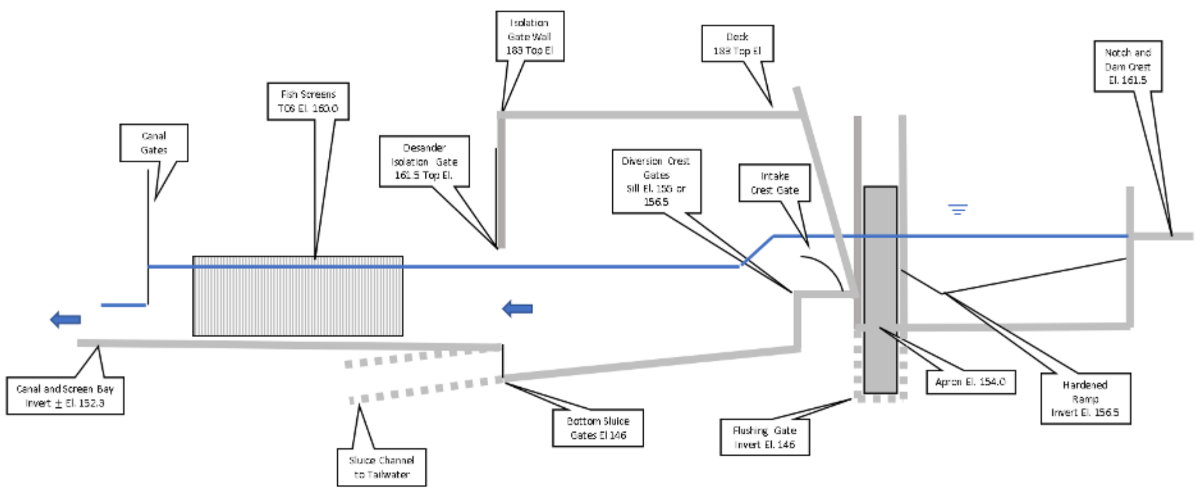


Figure 10-1 Gate Operation Features.

10.1.1 Extreme Flood Flows or Turn-Out Operations

When the diversion is closed due to extreme flood flows or turn-outs (see Figure 10-2), the desander isolation gates and bottom sluice gates would be closed isolating the diversion. The project conveys extreme floods over the diversion dam crest. For floods with water surface elevations above an elevation of 165 ft, water would overtop the intake crest gates and fill the diversion channel bays. The physical barrier of the intake crest elevation should help to reduce entrainment of bedload into the facility. It is expected that suspended sediment and other debris would settle into the diversion, to be removed either through use of the desander after the flood events.

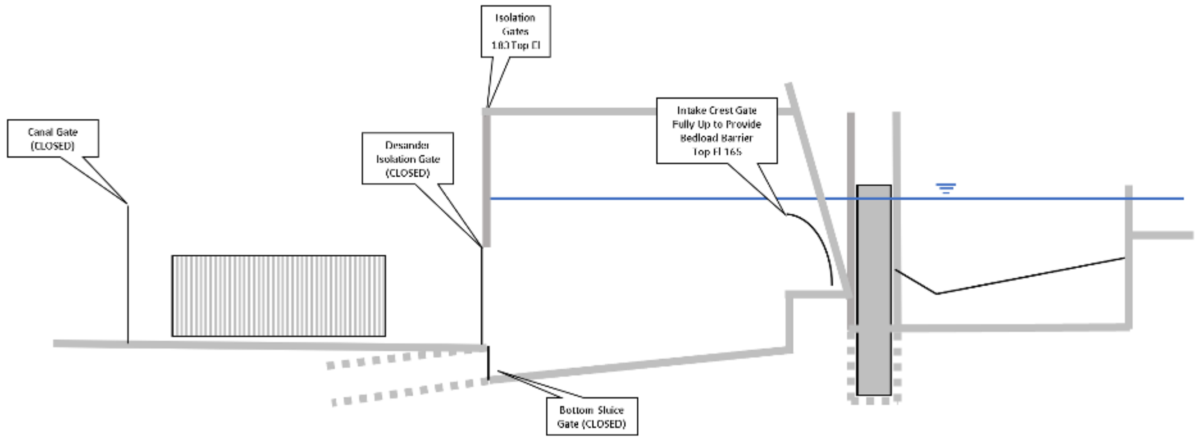


Figure 10-2 Flood Operations and Diversion Closure.

10.1.2 High River Flow Operations

Between upstream river El. 166 to El. 162 ft (see Figure 10-3), ramps flows and discharges over the dam crest would range from 6,000 to 3,000 ft^3/s. Diversions could be made, but because the water elevations exceed the height of the fish screens (El. 160 ft), the intake crest gates would have to be raised to create head loss and allow the screens to operate. The canal gate would be partially opened to regulate the canal discharge and regulate screen bay water surface elevations.

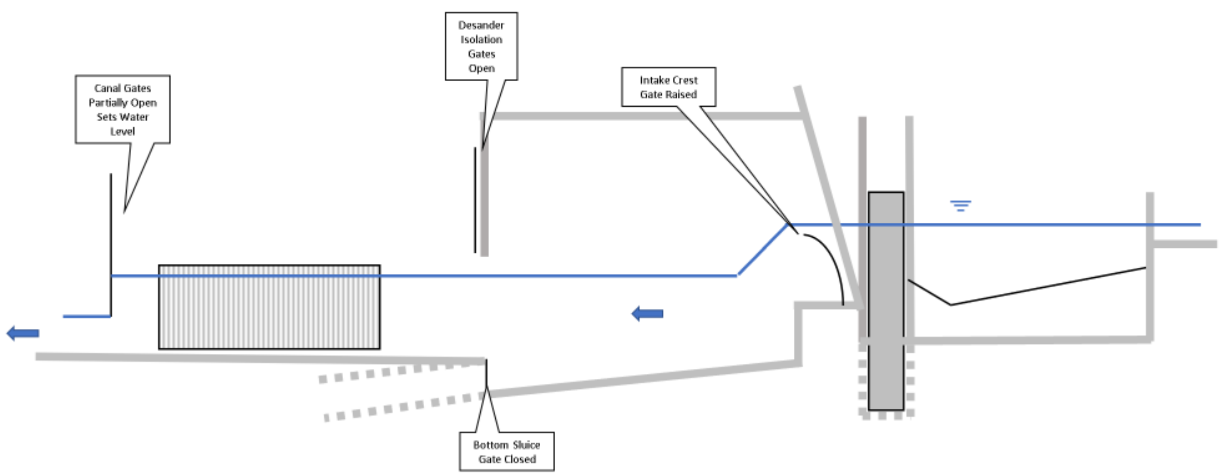


Figure 10-3 High River Flow Diversion Operation.

10.1.3 Moderate River Flow Operations

At upstream river flows of 1,200 to 1,000 ft^3/s, spill over the dam crest will cease and all flows will be discharged by the Hardened Ramp through the baffled and low flow sections (see Figure 10-4). The intake crest gates would be lowered but only to the extent required to convey diversion flows (i.e., the flow split) and maintain the vertical offset between the intake gate crest and the ramp invert to prevent sedimentation. The canal gate would also be regulated to ensure water depths in the screen bay for approach and sweeping velocity criteria on the screen, as well as ensure hydraulic control at the crest gates. The crest gate would be lowered as river flows dropped and the canal gate adjusted accordingly.

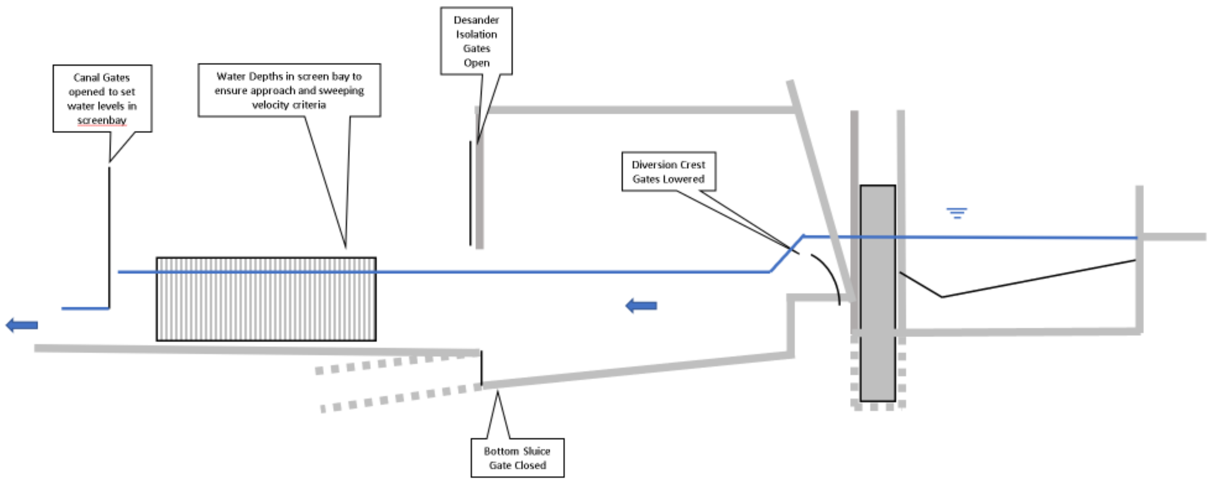


Figure 10-4 Moderate River Flow Diversion Operation

10.1.4 Low River Flow Operations

At low river flows of less than 400 ft^3/s, all Hardened Ramp flows would be largely contained in the roughened low flow section at river El. 160 ft (see Figure 10-5). This would provide at least 3 ft of head over the diversion sill with the crest gates lowered into a free flow condition. The canal gate would be used exclusively to control diversions and screen hydraulics for maximum diversions. For low flows where maximum diversion is not needed it will be a combination of canal and intake crest gates. As flows decrease, the physical offset decreases and the potential ingestion of infilled sediment on the apron and in front of the intake increases. At low flows, this is expected to be primarily sand. The desander should reduce sediment entrained into the screen bays and enable low flow diversions.

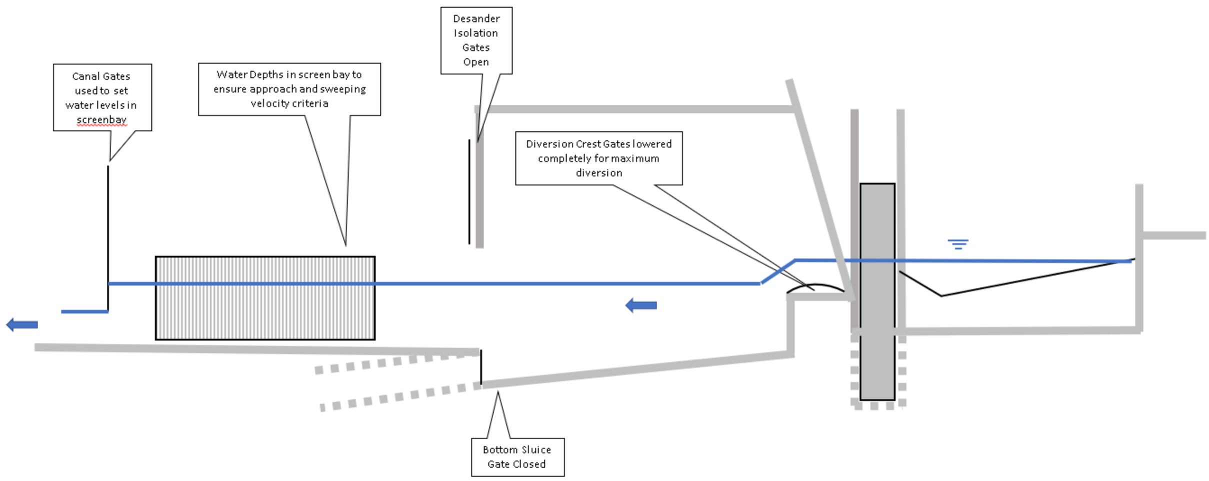


Figure 10-5 Low River Flow Diversion Operation.

10.1.5 Diversions During Desander Operations

The desander allows for diversion operations to continue during sluicing. This is performed by closing the isolation gate in the bay/channel that will be sluiced while opening the bottom sluice gate. The remaining bays will function normally to divert water similar to the figures given above. This operation was shown previously in Figure 7-10.

10.2 Dam and Hardened Ramp Flow Splits

Hardened Ramp flow splits were developed using a modification of the HEC-RAS 2D model developed for the DDR and are presented below in Table 10-1and Figure 10-6. Although there is a small amount of spill at lower flows, this preliminary geometry sufficiently meets the objective for 1,200 ft^3/s in the ramp before there is a significant depth of spill at the diversion dam crest. The 2D model included diversion flow of 750 ft^3/s and 75 ft^3/s of fish bypass flows for all the simulations.

Table 10-1 MOD-9 Design with 100' Dam Crest Notch at El 161.5'.

Upstream River Flow (ft^3/s)	Hardened Ramp Flow (ft^3/s)	Diversion Dam Flow (ft^3/s)	Upstream Water Surface Elevation (ft)
1,175	349	0	160.3
1,675	837	13	161.7
2,125	1,192	157	162.4
2,866	1,604	437	163.1
3,573	1,985	763	163.7
5,570	2,777	1,968	164.7
6,803	3,182	2,796	165.2

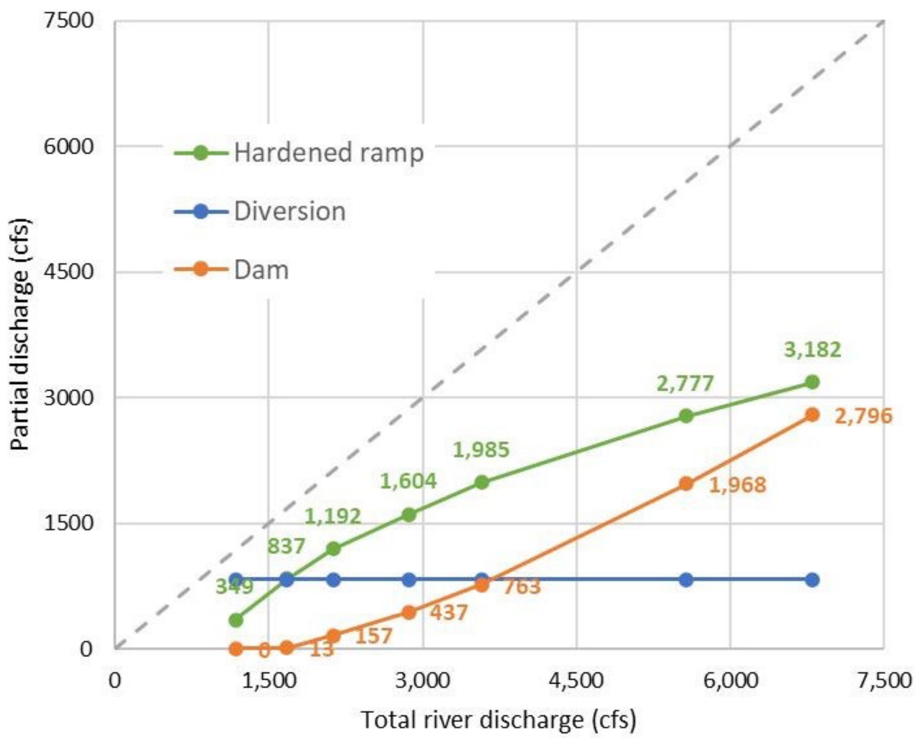


Figure 10-6 Hardened Ramp and Diversion Flow Splits from 2D Model.

Figure 10-7 shows the flow splits through the hardened ramp, diversion and dam measured during baseline testing in the 1:24 physical model (USBR, 2022) when river discharges were between 1,500 ft^3/s (lower limit of the 1:24 model) and 12,000 ft^3/s with the flushing channel closed. The physical model results maintain the requirement for the first 1,200 ft^3/s to go down the ramp and follow the same general distribution as those calculate in the 2D model (Figure 10-6).

The physical model results show that due to its large size (90 ft wide) and invert elevation (between El. 156.5 ft and El. 160.0 ft), the hardened ramp is the main flow path, conveying between 45% and 68% of the total Santa Clara River flow reaching Freeman Diversion. At a river flow of 6,000 ft^3/s, which is the upper limit of the fish passage window, the ramp passes around 2,800 ft^3/s.

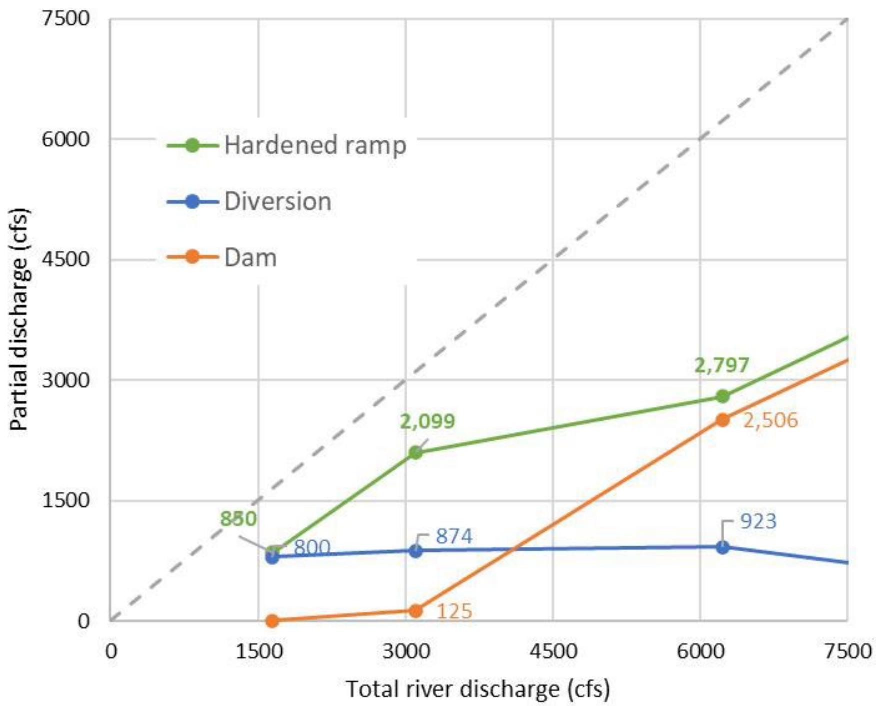


Figure 10-7 Flow splits measured in the 1:24 physical model during baseline testing with flushing channel closed (adapted from Tables 15 and B-4, USBR 2022).

10.3 Screen Operations

The proposed fish screen at Freeman dam has been designed for full diversion of 750 ft^3/s and meeting the NMFS (2022a) hydraulic criteria. To provide full functionality of the screen and meet criteria over a range of possible diversion flows, the operation of the fish screen can be modified. Hydraulics of the screen can be modified through:

- Operation of the diversion intake crest gates:
  crest gates can be raised to use up excess head at high river flows and water levels (Section 10.1)
- Operation of the canal gates:
  maintain water levels in the screen bay and regulating diversion discharges downstream

- Operation of the number and open area of the screen porosity panels:
  maintains screen flow hydraulics and reduce screen area to match diversion flows to NMFS (2022a) criteria with safe removal and bypass of downstream migrant fish.
- Operation of the capture weir elevation:
  maintain required bypass flow relative to diversion flow diverted.

To assess the possible screen operations and effects on screen hydraulics, NHC's previous operational hydraulic model was modified to include the desander and screen bay – including the screen size and orientation, fish collector and capture weir section. A range of possible river discharges and diversions were examined to see how the intake and fish screen could be operated to optimize both diversion capacity and fish screening objectives.

A range of river discharges from 100 to 3,200 ft^3/s were used with assumed diversions of 50% of the flow up to 750 ft^3/s maximum diversion. The hydraulic calculations and analysis indicate that:

- River discharges that result in elevations greater than ± El. 159.5 ft at the diversion intake, around 200 ft^3/s, the diversion intake crest gates are required to control inflows and moderate head losses to ensure water elevations in the screen bay are maintained at ± El. 159.3 ft. This screen bay elevation is required to meet minimum screen area criteria at 750 ft3/s maximum diversion.
- At river elevations less than ± El. 159.5 ft at the diversion intake, screen bay water surface elevations decrease. The downstream canal gates can be used to control diversion rates at these water surface elevations. This can be used as the screen bay water elevations are above minimum elevations required for diversion flow releases down the canal based on the canal rating curve.
- To maintain, fish screening and passage criteria on the fish screen, the effective area of fish screen can be reduced using the screen porosity control plates. The screen area can be reduced to 30% of one screen and total closure of the other screen bay to ensure NMFS (2022) criteria are met at diversion flows of 50 ft^3/s. At diversion flows of less than 50 ft^3/s, the collector weir may be reduced to ensure minimum depth criteria are maintained in the bypass flow over the collector weir.
- The ability to sequentially close porosity panels along the primary screen is an effective tool to optimize screen area and maintain optimum fish screening and downstream passage hydraulics with the proposed screen design.
- The capture weir controlling the bypass flow at the downstream end of the fish collectors can be controlled with a variable weir plate. The design range of elevation change for the capture weir is from El. 156.5 ft. to El. 158.0 ft.
- Diversion, isolation, sluice and canal gates; screen porosity panels and the fish screen cleaning systems; and capture weir controls can be operated automatically via programming through the project SCADA systems.

10.4 Yield Potential Calculations from Physical Model

Physical model testing with all design development improvements incorporated measured maximum diversion capacity under rivers flows ranging from 250 ft^3/s and 2,000 ft^3/s (USBR, 2023). Tests examined the impact on diversion yield of sedimentation on the apron upstream of the diversion intake, and of opening and closing of a removable bulkhead at the downstream training wall notch.

Under all operating scenarios, it was found that the maximum diversion capacity was approximately 700 ft^3/s to 750 ft^3/s, and that the entire river flow could be diverted at flows at or below that maximum diversion capacity. This yield potential is an increase over previous designs that were shown to have limitations for flow splits at lower discharges. Physical model testing also showed that maximum diversion capacity was insensitive to sedimentation on the apron upstream of the intake, with maximum measured capacity similar for a given river discharge with the apron filled with and cleared of sediment. However, these were cases where the sediment had not infilled above the elevation of the raised crest gates. If sedimentation continued or flows reduced, sediment would form obstructions to diversion into the intake.

The training wall bulkhead was shown to mainly impact diversion capacities at river flows around 750 ft^3/s to 1,000 ft^3/s, with capacity differences ranging from 50 ft^3/s to 120 ft^3/s. With the apron cleared of sediment, removal of the bulkhead (lowering the invert of the notch) resulted in an increase in diversion capacity, as flow was drawn from the right side of the training wall to the intake. The opposite occurred if the apron was aggraded with sediment; removal of the bulkhead reduced maximum diversion capacity, as flow spilled from the apron to the right toward the hardened ramp.

At both higher and lower river flows, the bulkhead had negligible impact on maximum diversion capacity. This test suggests that adding or removing the bulkhead can be used to either increase diversion rate or preferentially divert flow to the hardened ramp for fish passage at intermediate river discharges, depending on the state of apron sedimentation.

11 Design Development Review and Next Design Steps

This document has provided a complete summary of the hydraulics for the Hardened Ramp concept for the Freeman Diversion Facility. This work has a foundation in previous NHC design documents including:

- The Hydraulic Basis of Design Report or HBOD (NHC and GEI, 2019) which provided preliminary plans; hydraulic, geotechnical, and structural analysis; and a preliminary cost estimate for the proposed alternative.
- The 2020 Design Development Report followed (NHC, 2020) to address detailed comments that were provided on the draft Hydraulic Basis of Design Report (HBOD) by National Marine Fisheries Service (NMFS) and California Department of Fish and Wildlife (CDFW).
- The 2021 Design Modification Report provided information on the process that led to the MOD-9 alternative design modification was documented by (NHC, 2021).

This report summarizes the design from the reports above and advanced items through the use of physical models, numerical modeling and engineering design. The design development within this report was supported by findings from the physical model testing ((USBR, 2022) (USBR, 2023)). Notable design development was completed in the following areas:

- Low Flow Channel Design Modifications (Section 5.2): Used the 1:12 physical model to refine the rock placement and confirmed that the changes improved fish passage to meet the defined criteria.
- Sediment Management (Section 7): Sediment management systems were developed to manage the sediment bedform in front of the ramp (flushing channel and training wall) and to remove sediment ingested into the intake (desander). Both systems are required to manage the extremely high sediment load of the Santa Clara River.
- Debris Management (Section 8): Completed testing on the physical model which determined that debris build-up effects and provided recommendations for O&M of the ramp.
- River Training Works (Section 4): A right abutment bullnose extension and training wall were included to train the upstream ramp conditions. Development of an O&M program was preferred to large scale training works (bendway weirs) that would have a large footprint and are not tested for the application.
- Fish Screen and Bypass Design (Section 6.9): Completed the preliminary design of the dual bay fish screen that meets defined fisheries criteria. Designed the associated fish collector, bypass and assessment facilities.
- Gate Design (Section 6): Developed proposed design configurations in consultation with manufacturers for the intake crest gates, hardened ramp crest gates, desander isolation slide gates and canal gates.

Objectives for sediment, diversions and fish passage were identified (Section 2). These objectives were addressed through design development by providing sediment management options, adjusting the design to be flexible for operations and flow splits, and refining fish passage features.

This report provides full hydraulic design plans for the system. However, it is recognized that additional hydraulic analysis or refinements may be necessary as the overall design advances to 60%, 90% and Final Design plans. During the more detailed design on items such as the civil works, structural, geotechnical and mechanical components items such as wall thickness and gate dimensions may need to be revised. Additional hydraulics would be needed to support these changes. Additional analysis may also be necessary to address questions from regulatory agencies (NMFS, CDFW, FEMA) or if conditions have notably changed in the field.

The next phase of design work will be up to update the civil, structural, geotechnical plans and other items from the previous 30 percent design (NHC and GEI, 2019) to reflect the updated hydraulics in preparation for future design phases. This work is underway and will include an updated cost estimate.

12 References

AECOM (2014). Sediment Transport Analysis Santa Clara River at Freeman Diversion. Report prepared by AECOM for United Water Conservation District.

AECOM, et al, (2016). Freeman Diversion Fish Passage, Hardened Ramp Design, DRAFT 60% Hydraulic Design Report. Prepared for United Water Conservation District. June 2016

Bell, (1991). Fisheries Handbook of Engineering Requirements and Biological Criteria. US Army Corps of Engineers, North Pacific Division. Fish Passage Development and Evaluation Program. Portland, OR.

Bladé, E., (2005). "Modelación del flujo en lámina libre sobre cauces naturales. Análisis integrado con esquemas en volúmenes finitos en una y dos dimensiones". PhD dissertation, Universitat Politècnica de Catalunya.

Bouvard, M. (1992). Mobile Barrages and Intakes on Sediment Transporting Rivers. IAHR Monograph. Balkema, Rotterdam.

Bulle, H. (1926). Untersuchungen uber die Geschiebeableitung bei der Spaltung von Wasserlaufen (Research on sediment discharge in channel diversions). VDI Verlag, Berlin.

CDFW, (2002). Culvert Criteria for Fish Passage, California Department of Fish and Game, May 2002.

CDFW, (2009). Part XII California Salmonid Stream Restoration Manual, Fish Passage Design and Implementation, California Department of Fish and Game, April 2009.

Dutta, S. (2017). Bulle-Effect and its Implications for Morphodynamics of River Diversions. Doctoral Dissertation in Civil Engineering in the Graduate College of the University of Illinois at Urbana-Champaign.

Federal Highway Administration (FHWA) (2009). Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Third Edition. Volume 2. HEC No. 23. September 2009.

Flemming, B.W. (2011). Geology, morphology and sedimentology of estuaries and coasts. In: Flemming, B.W., Hansom, J.D. (Eds), Treatise on Estuaries and Coasts, Volume 3, Estuarine and Coastal Geology and Morphology. Elsevier, Amsterdam, pp. 7–38.

Garde, R. J. and Ranga Raju, K. G. (1977). Mechanics of sediment transportation and alluvial stream problems. Wiley Eastern Ltd., New Delhi.

GEI Consultants (2020). Vern Freeman Diversion: Geologic and Geotechnical Evaluation of Proposed Intake Structure. Submitted to Northwest Hydraulic Consultants. Submitted on March 26th, 2020.

Hydroscience & Engineering (2021). "Review and analysis of meander geometry along the Santa Clara River near the Vern Freeman Diversion Dam." Technical Memorandum. Prepared for: United Water Conservation District. Prepared by: Benjamin Swanson, PhD and Larry Weber, PE, PhD. December 15, 2021.

IHHS (1989). Investigación Sobre Modelo Hidráulico del Desarenador Chavimochic (Physical Model Study of Chavimochic Desander). Hydraulics Institute, University of Piura, Peru. July 1989.

IHHS (1990). Investigación sobre el Modelo Hidráulico de las Obra de Captación de la Central Hidroeléctrica Yanango (Physical Model Study of Yanango 119iversión). Hydraulics Institute, University of Piura, Peru. December 1990.

IHHS (1992). Investigación en el Modelo Hidráulico de la Obra de Derivación de la Central Hidroeléctrica San Gaban II (Physical Model Study of San Gaban 119iversión). Hydraulics Institute, University of Piura, Peru. December 1992.

Keefer, (2010). Testing adult Pacific lamprey performance at structural challenges in fishways. North American Journal of Fisheries Management. 30: 376-385.

Mesa, (2003). Swimming performance and physiological responses to exhaustive exercise in radio-tagged and untagged Pacific lampreys. Transactions of the American Fisheries Society 132: 483-492.

Morris, G.L. and Fan, J. (1998). Reservoir Sedimentation Handbook. McGraw-Hill.

NEA (2016). Kali Gandaki 'A' Hydropower Plant Rehabilitation Project. Hydraulic Testing of Proposed Headwork Modifications. Technical Memo prepared for Nepal Electricity Authority, Dec. 2016.

NHC (2016a). Evaluation of Vern Freeman Diversion Dam Modification Alternatives (NHC Reference 6001627). Technical Report Prepared by Northwest Hydraulic Consultants for United Water Conservation District.

NHC (2016b). Addendum 1: 400-ft Wide Notch with Pneumatically-Actuated Gates 1 Evaluation of Vern Freeman Diversion Dam Modification Alternatives Detailed Feasibility Examination (nhc reference 6001627). Report prepared by Northwest Hydraulic Consultants for United Water Conservation District.

NHC (2016C). Sediment Transport and Deposition Assessment of Freeman Diversion Conveyance System, Phase 2 Evaluate Alternatives. Report prepared by Northwest Hydraulic Consultants for United Water Conservation District.

NHC (2017). Vern Freeman Refined Notch Analysis Geomorphic and Hydraulic Assessment, Appendix A Geomorphic Assessment. NHC Ref. No. 6002702.

NHC and GEI. (2019). Vern Freeman Diversion Hardened Ramp Fish Passage Improvements, Draft Hydraulic Basis of Design Report and 30 Percent Design. Prepared for United Water Conservation District.

NHC (2020). Vern Freeman Diversion Hardened Ramp Fish Passage Improvements, Design Development Draft Report. Prepared by Northwest Hydraulic Consultants for United Water Conservation District. Pasadena, CA. September 2020.

NHC (2021). Vern Freeman Diversion, Hardened Ramp Design Modifications, Draft Report. Prepared by Northwest Hydraulic Consultants for United Water Conservation District. Pasadena, CA. September 2021.

NMFS, (2018). Pages from Anthony Spina Amicus Brief, Document 179-1 #25444; Case 2:16-cv-03869-DOC-PLA. January 19, 2018.

NMFS, (2019). Review of Draft Hydraulic Basis of Design Report for Hardened Ramp Fish Passage Improvements dated July 21, 2019. Letter to Mauricio Guardado, United Water Conservation District. National Marine Fisheries Service, signed by Alecia van Atta.

NMFS, (2022a). NOAA Fisheries West Coast Region Anadromous Salmonid Passage Design Manual, NOAA Fisheries West Coast Regional Office, 1201 Northeast Lloyd, Portland, Oregon 97232.

NMFS, (2022b). NOAA Fisheries West Coast Region Guidance to Improve the Resilience of Fish Passage Facilities to Climate Change – 2022. NOAA Fisheries West Coast Regional Office, 1201 Northeast Lloyd, Portland, Oregon 97232.

NOAA, (2001). Guidelines for Salmonid Passage at Stream Crossings. National Oceanic and Atmospheric Administration, Southwest Region, September 2001.

NOAA, (2011). Anadromous Salmonid Passage Facility Design. National Marine Fisheries Service, July 2011.

Ordoñez, J.I. (2013). Sediment Control at Large Canal Diversions. Proceedings of the 35th IAHR World Congress. Chengdu, China.

Powers, P.D. and J.F. Orsborn. (1985). Analysis of barriers to upstream fish migration. An investigation of the physical and biological conditions affecting fish passage success at culverts and waterfalls. Final Project Report, part 4 of 4, submitted to Bonneville Power Administration.

Stillwater Sciences (2011). Geomorphic Assessment of the Santa Clara River Watershed, Synthesis of the Lower and Upper Watershed Studies. Ventura and Los Angeles Counties, California.

Stillwater Sciences (2013). Freeman Diversion Dam Removal Sediment Transport Modeling, Santa Clara River, CA. Technical Memorandum prepared by Stillwater Sciences for California Trout .

United Water (United Water Conservation District) (2021). Recommended Debris Loading for the Freeman Diversion Fish Passage Physical Model. Open-File Report 2021-01. January 2021.

U.S. Army Corps of Engineers (USACE) (1994). Hydraulic Design of Flood Control Channels. 1110-2-1601. Washington, D.C.

USBR (2011). Robles Diversion Dam Left Bank Fishway, Hydraulic Laboratory Report HL 2011-04. US Bureau of Reclamation. Denver Colorado. August 2011.

USBR (2015). Bank Stabilization Design Guidelines. Report No. SRH-2015-25. Albuquerque Area Office. Science and Technology. Policy and Administration (Manuals and Standards). Yuma Area Office. Denver. June 2015.

USBR (2022). Physical Hydraulic Modeling of the Freeman Diversion Hardened Ramp Fish Passage Alternative. Hydraulic Laboratory Report HL-2022-03. Bureau of Reclamation. October 2022.

USBR (2023). Physical Hydraulic Modeling of Operational and Stress Testing of the Freeman Diversion Hardened Ramp Fish Passage Alternative. Hydraulic Laboratory Report HL-2023-01. Bureau of Reclamation. January 2023.

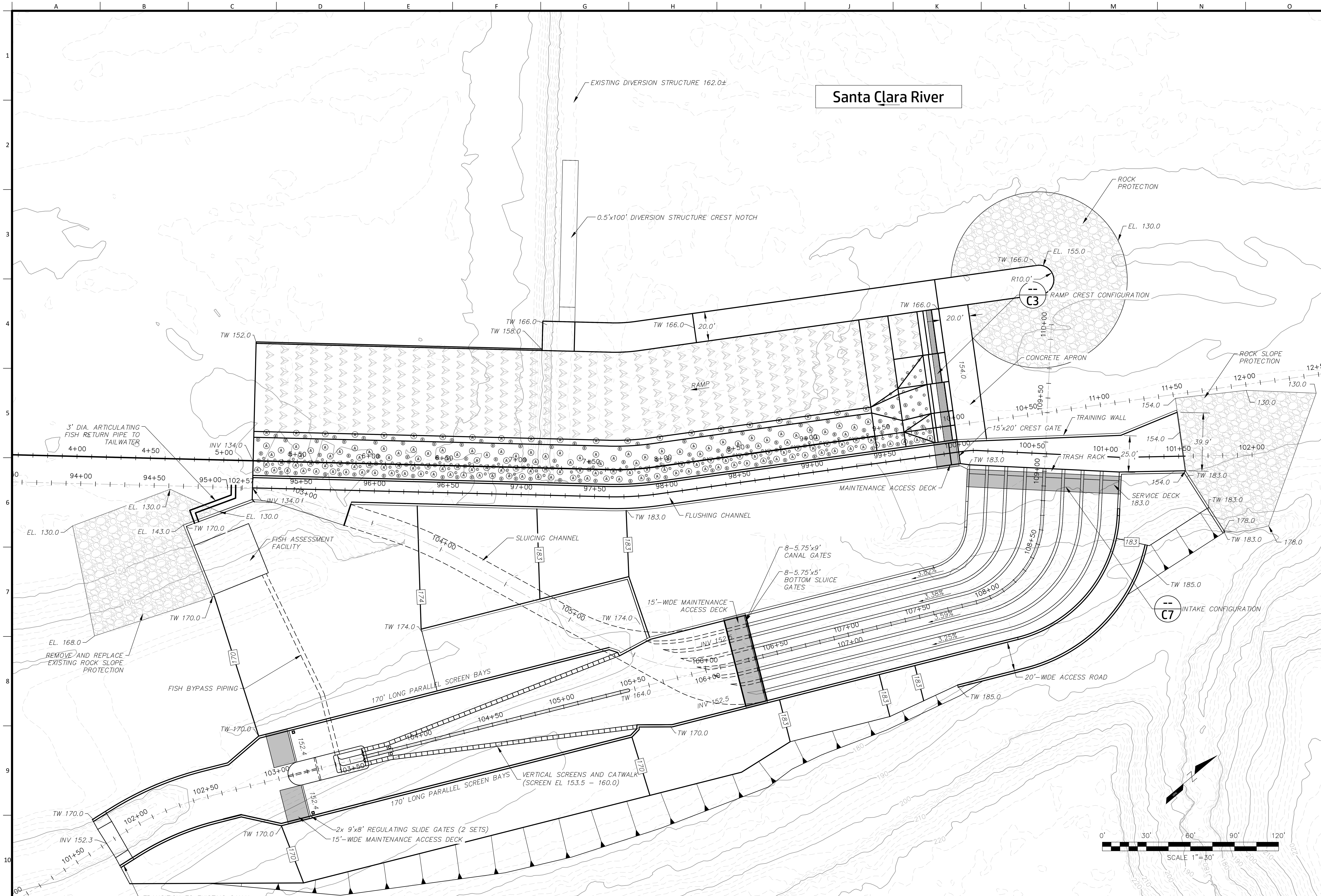
Vasquez, J.A. (2005). Two-Dimensional Numerical Simulations of Flow Diversions. 17th Canadian Hydrotechnical Conference. Edmonton, Alberta, August 17–19, 2005.

Vasquez, J.A. (2007). Asymmetric Flow in Symmetric Desanders. 18th Canadian Hydrotechnical Conference. Winnipeg, Manitoba, August 22 – 24, 2007.

Webb, P.W. (1977). Effects of size on performance and energetics of fish. In: T.J. Pedley (ed.) Scale Effects in Animal Locomotion. Academic Press, New York. 315-330.

APPENDIX A

DRAFT HYDRAULIC PLAN SET



LAST SAVED BY: czhao LAST DATE SAVED: 08/14/2023 09:42
FILE LOCATION: \\MAINFILE-VAN\PROJECTS\ACTIVE\6004761 VERN FREEMAN DAM\96 CAD\02_PRODUCTION\0100_CURRENT\VFD CIVIL 30PCT.DWG

Revisions		
Description	Date	No.

Drawing Information

Date	14 August 2023 (09:43)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD CIVIL 30PCT
Plotted Scale	

PRELIMINARY
NOT FOR CONSTRUCTION

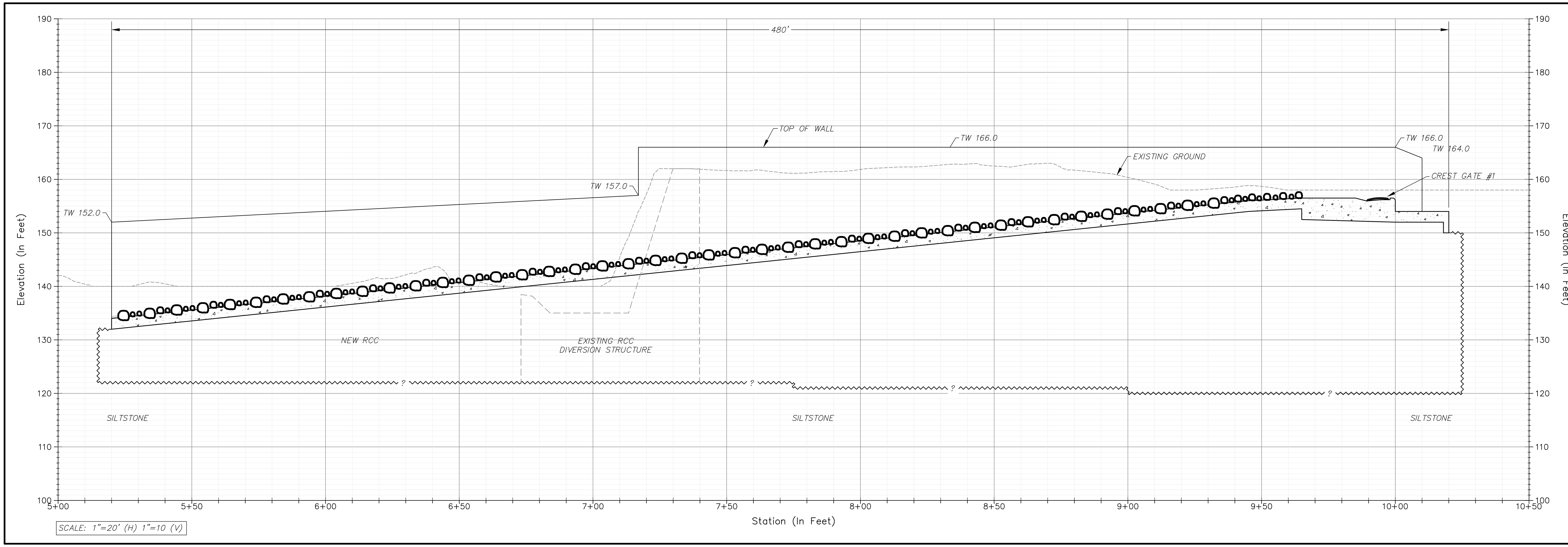
Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

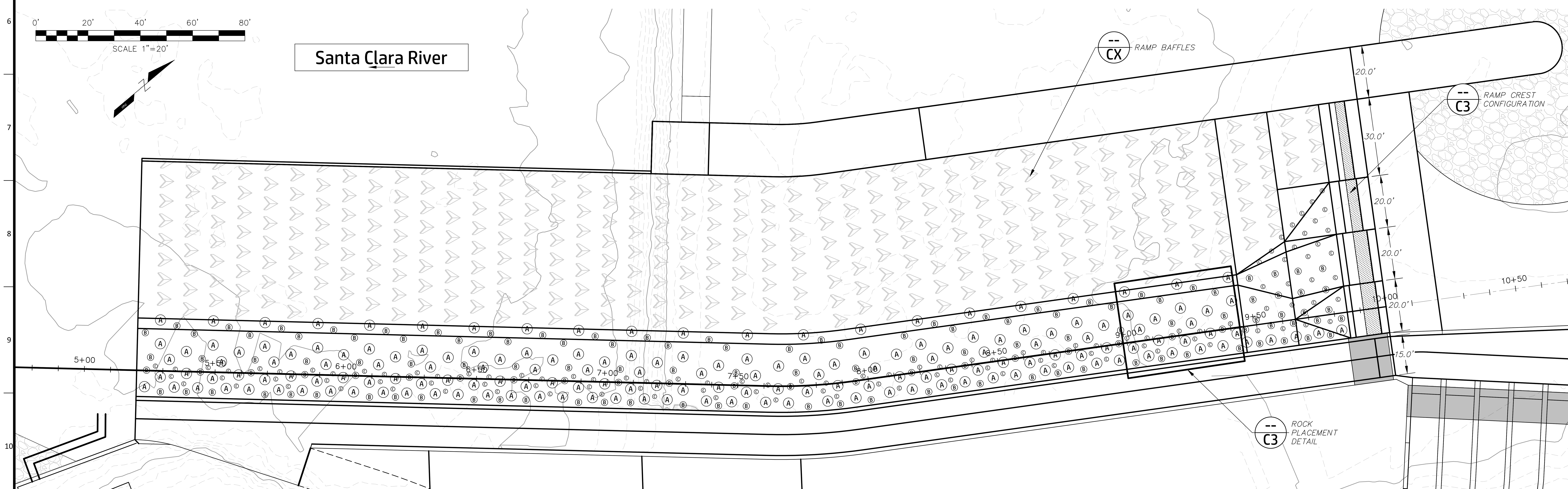
Site Plan

nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

Job Number 6004761
Sheet Number C1
Sheet 3 of 11





Revisions

No.	Date	Description

Drawing Information

Date	14 August 2023 (09:44)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD CIVIL 30PCT
Plotted Scale	

PRELIMINARY
NOT FOR CONSTRUCTION

Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Hardened Ramp Plan & Profile

nhc
northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

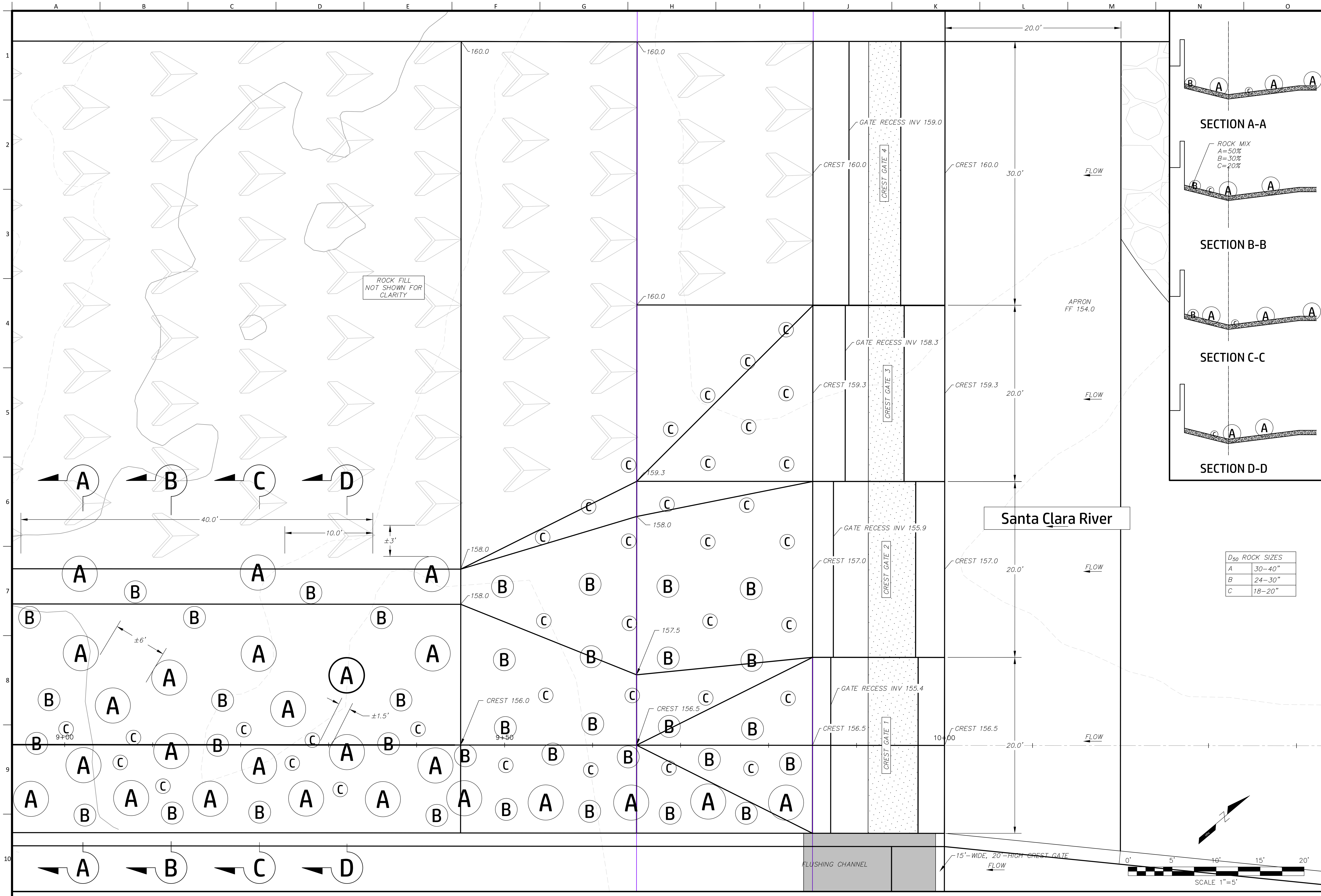
United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

Job Number
6004761

Sheet Number
C2

Sheet 4 of 11

LAST SAVED BY:czhao LAST DATE SAVED:08/14/2023 09:42
FILE LOCATION: \\MAINFILE-VAN\PROJECTS\ACTIVE\6004761 VERN FREEMAN DAM\96 CAD\02_PRODUCTION\0100_CURRENT\VFD CIVIL 30PCT.DWG



Revisions
No. | Date | Description

Drawing Information
Date 14 August 2023 (09:44)
Status 30% Submittal
Designer
Drafter
Checked
File Name VFD CIVIL 30PCT
Plotted Scale

PRELIMINARY
NOT FOR CONSTRUCTION

Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Hardened Ramp Entry Plan

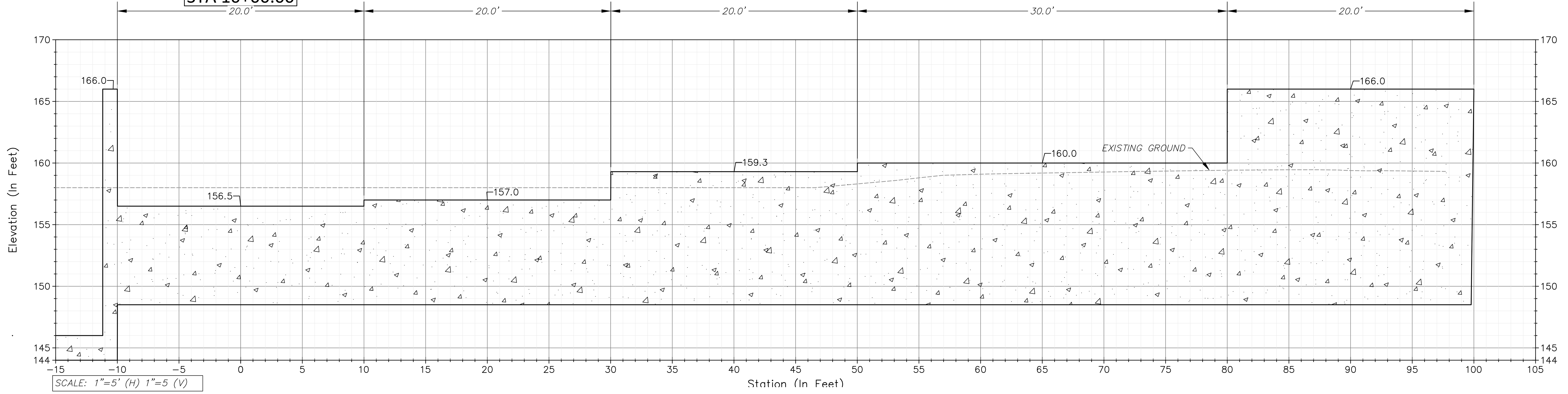
nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

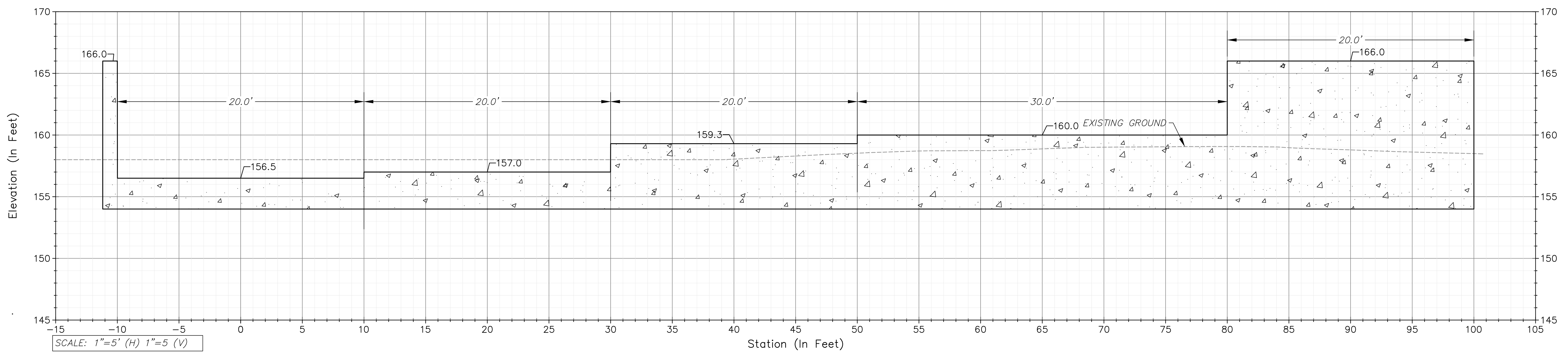
Job Number 6004761
Sheet Number C3
Sheet 5 of 11

LAST SAVED BY: czhao LAST DATE SAVED: 08/14/2023 09:42
FILE LOCATION: \\MAINFILE-VAN\PROJECTS\ACTIVE\6004761 VERN FREEMAN DAM\96 CAD\02_PRODUCTION\0100_CURRENT\VFD CIVIL 30PCT.DWG

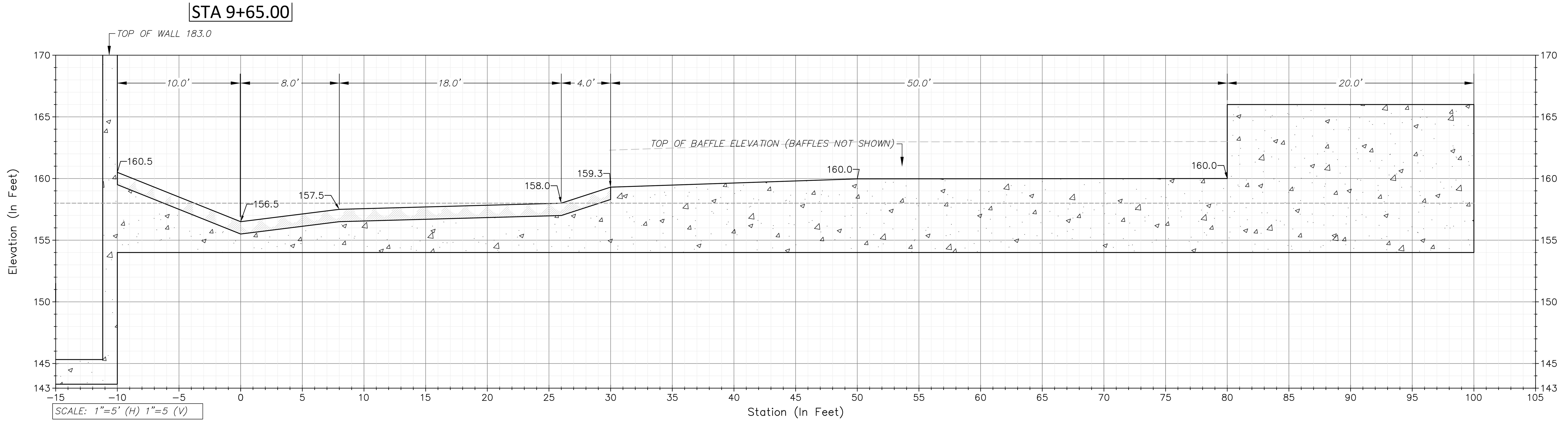
STA 10+00.00



STA 9+85.00



STA 9+65.00



Revisions		
No.	Date	Description

Drawing Information

Date	14 August 2023 (09:44)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD CIVIL 30PCT
Plotted Scale	

PRELIMINARY
NOT FOR CONSTRUCTION

Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Roughened Channel Entry Sections

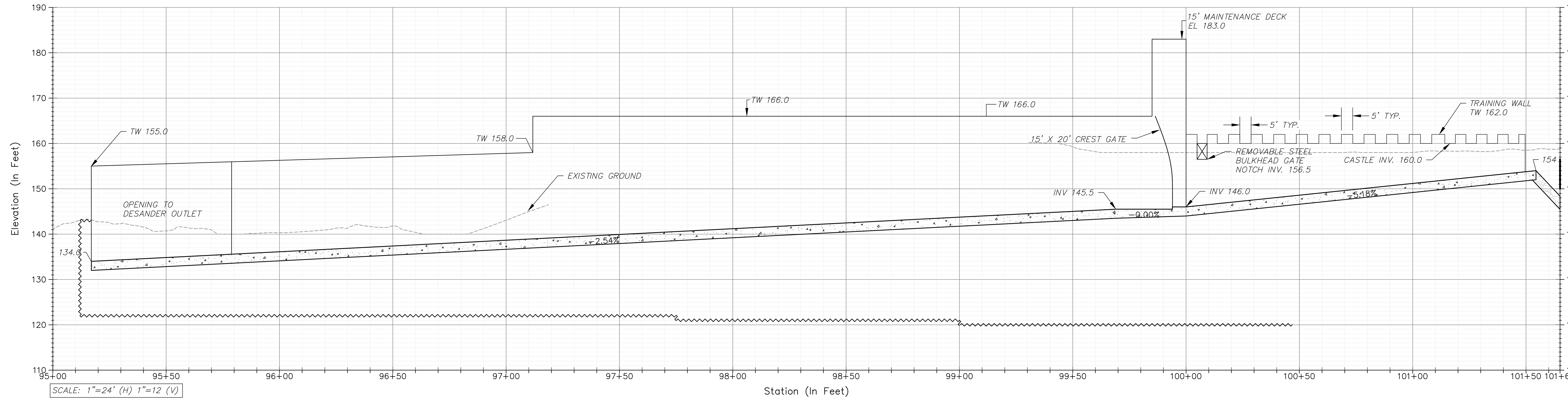
nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

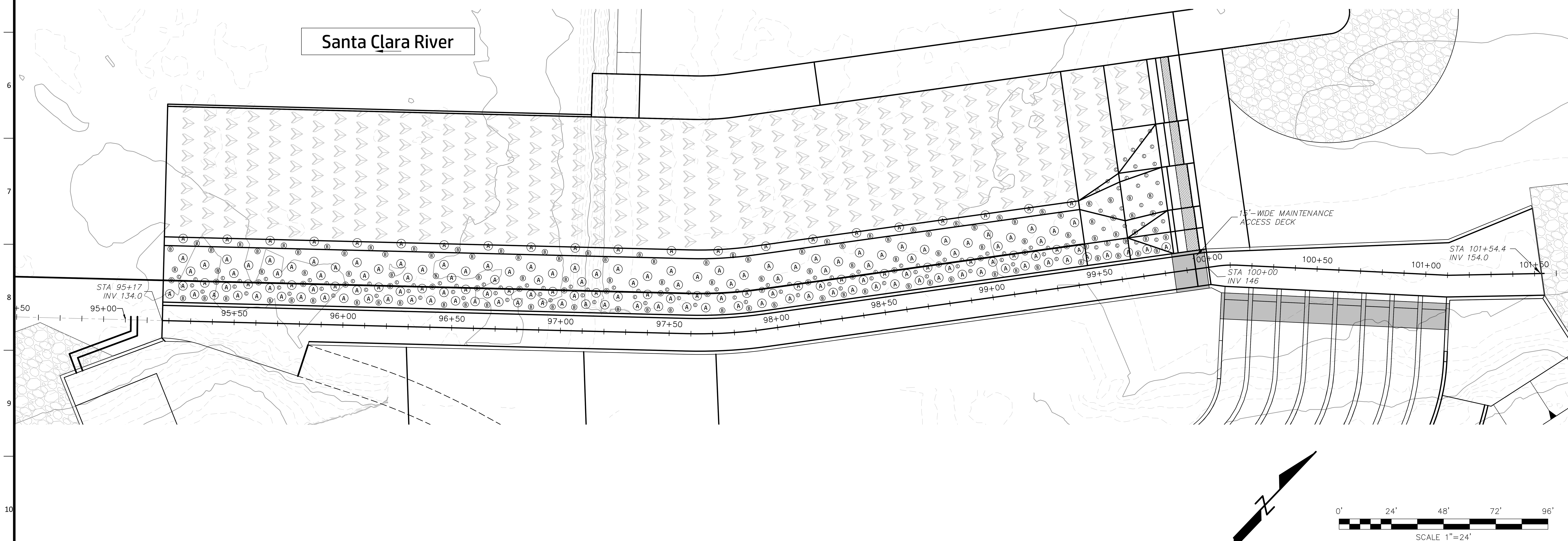
United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

Job Number 6004761

Sheet Number C4

Sheet 6 of 11





PRELIMINARY
NOT FOR CONSTRUCTION

Revisions

No.	Date	Description

Drawing Information

Date	14 August 2023 (09:44)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD CIVIL 30PCT
Plotted Scale	

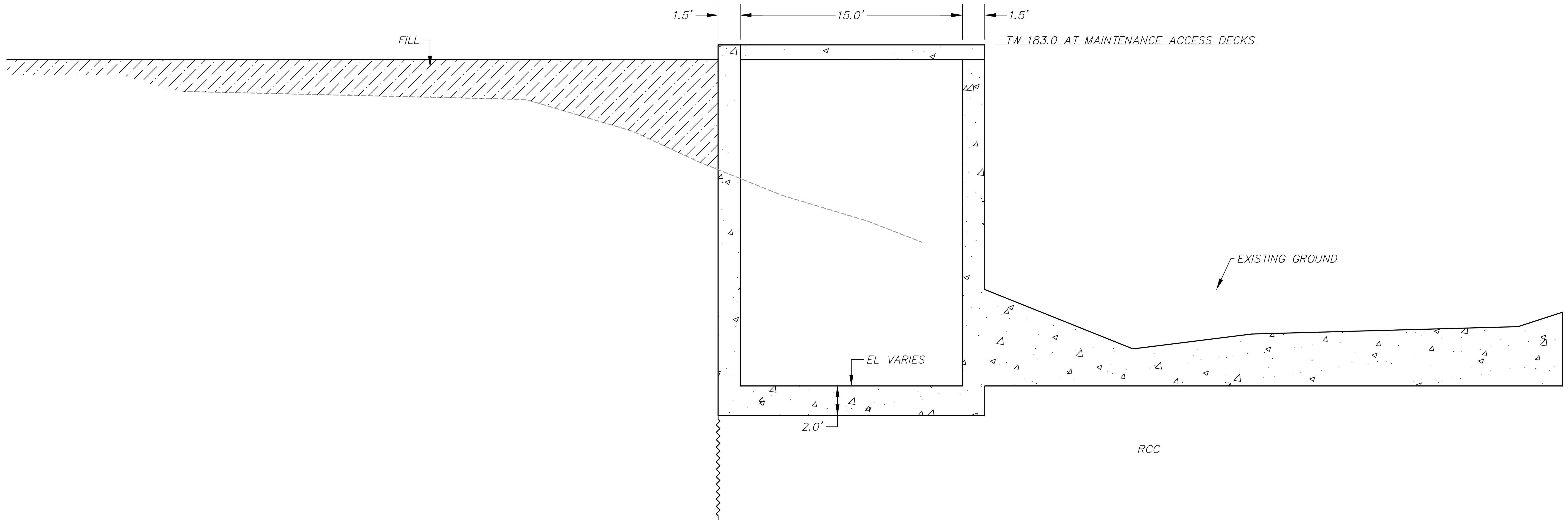
Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Flushing Channel Plan & Profile

nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

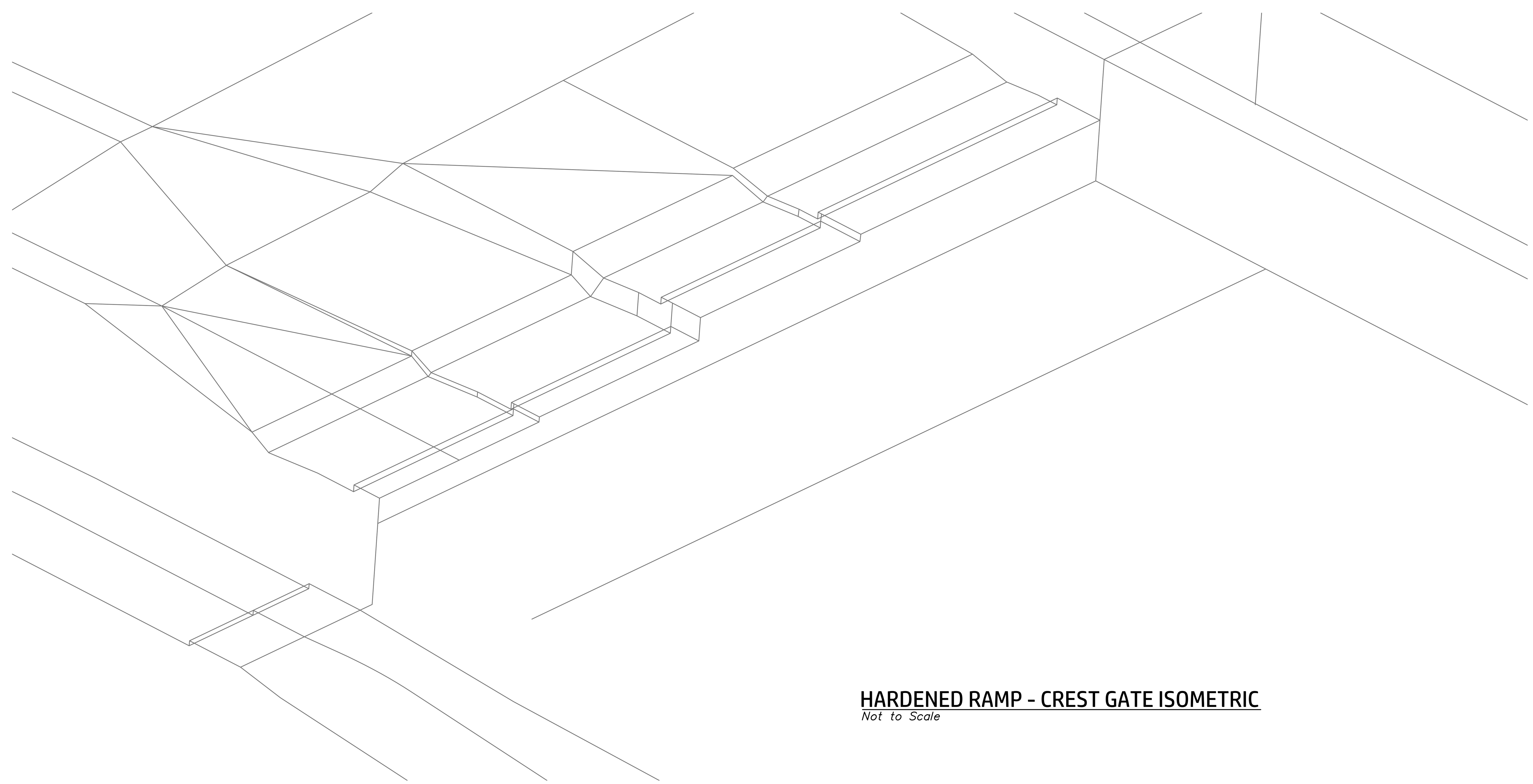
United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

Job Number 6004761
Sheet Number C5



FLUSHING CHANNEL - TYPICAL SECTION

SCALE: 1"=5'



HARDENED RAMP - CREST GATE ISOMETRIC

Not to Scale

Revisions

No.	Date	Description

Drawing Information

Date	14 August 2023 (09:44)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD CIVIL 30PCT
Plotted Scale	

PRELIMINARY
NOT FOR CONSTRUCTION

Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Flushing Channel Typical Section

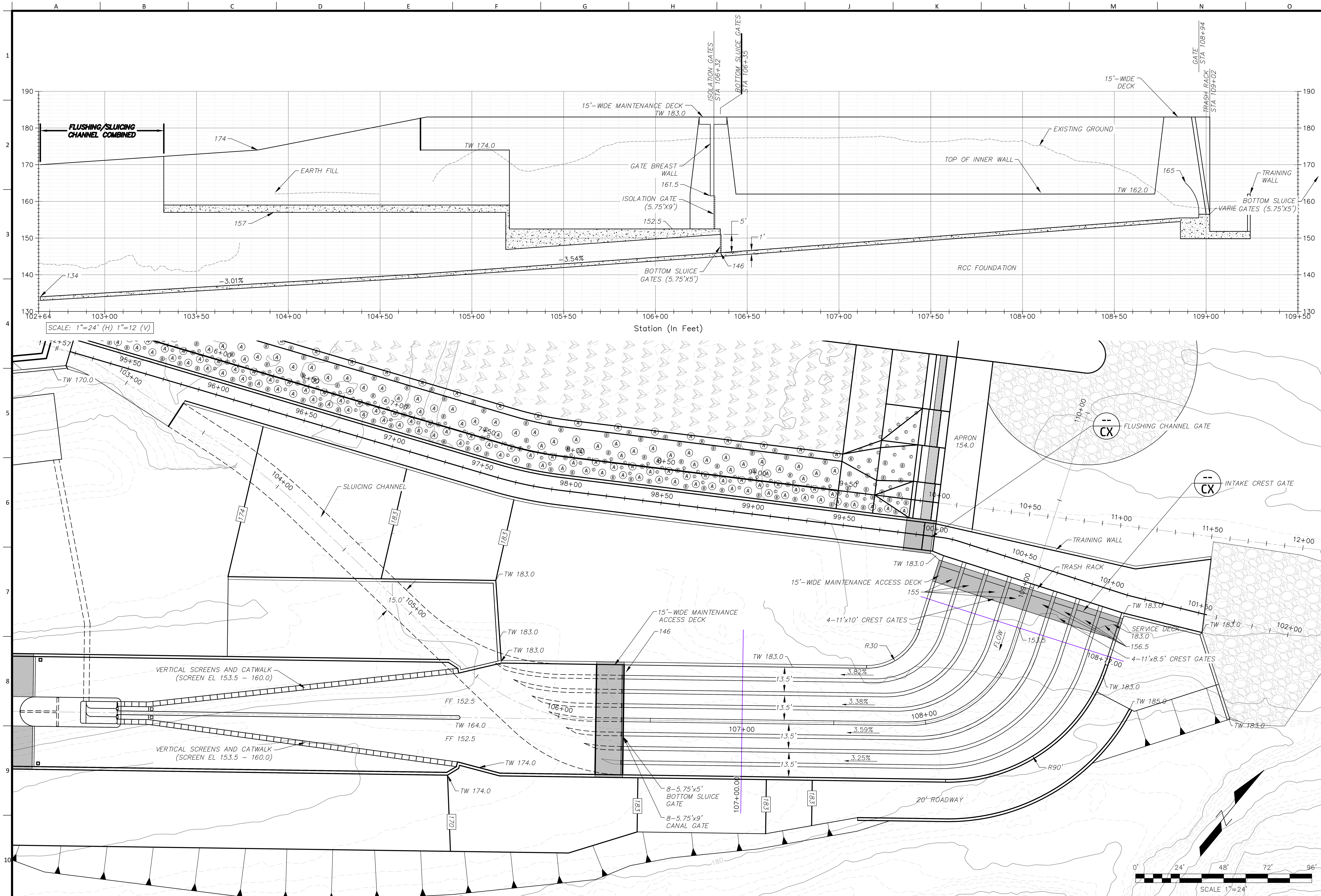
nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

Job Number 6004761

Sheet Number C6

Sheet 8 of 11



Revisions

No.	Date	Description

Drawing Information

Date	14 August 2023 (09:44)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD CIVIL 30PCT
Plotted Scale	

PRELIMINARY
NOT FOR CONSTRUCTION

Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Diversion & Sluicing Channel Plan & Profile

nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

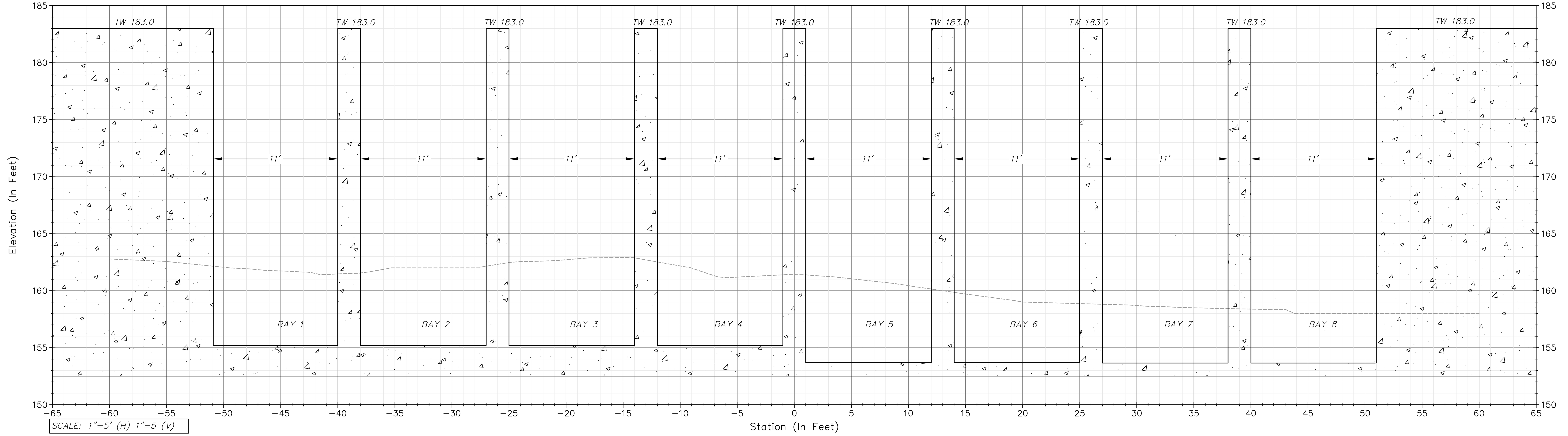
United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

Job Number 6004761

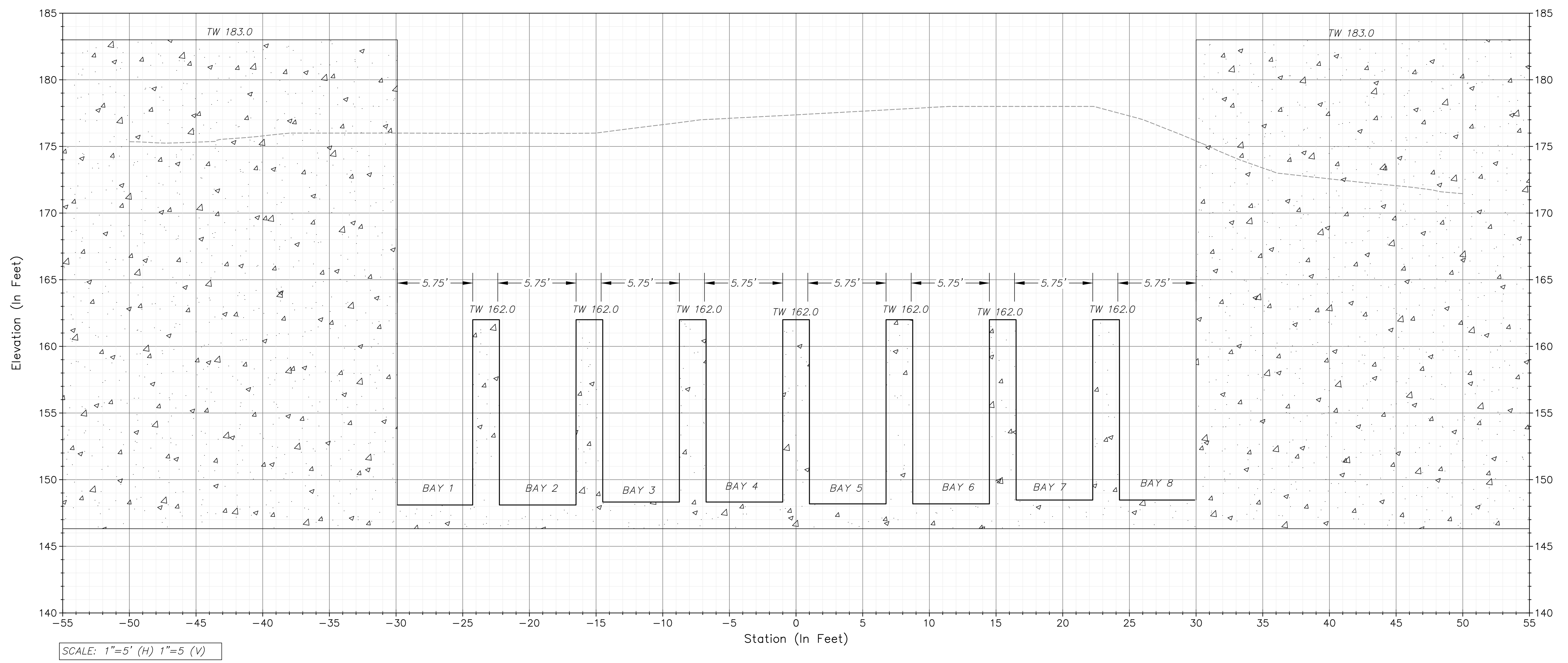
Sheet Number C7

LAST SAVED BY: czhao LAST DATE SAVED: 08/14/2023 09:42
FILE LOCATION: \\MAINFILE-VAN\PROJECTS\ACTIVE\6004761 VERN FREEMAN DAM\96 CAD\02_PRODUCTION\0100_CURRENT\VFD CIVIL 30PCT.DWG

STA 108+73.00



STA 107+00.00



Revisions

No.	Date	Description

Drawing Information

Date	14 August 2023 (09:44)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD CIVIL 30PCT
Plotted Scale	

PRELIMINARY
NOT FOR CONSTRUCTION

Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Diversion Channel Sections

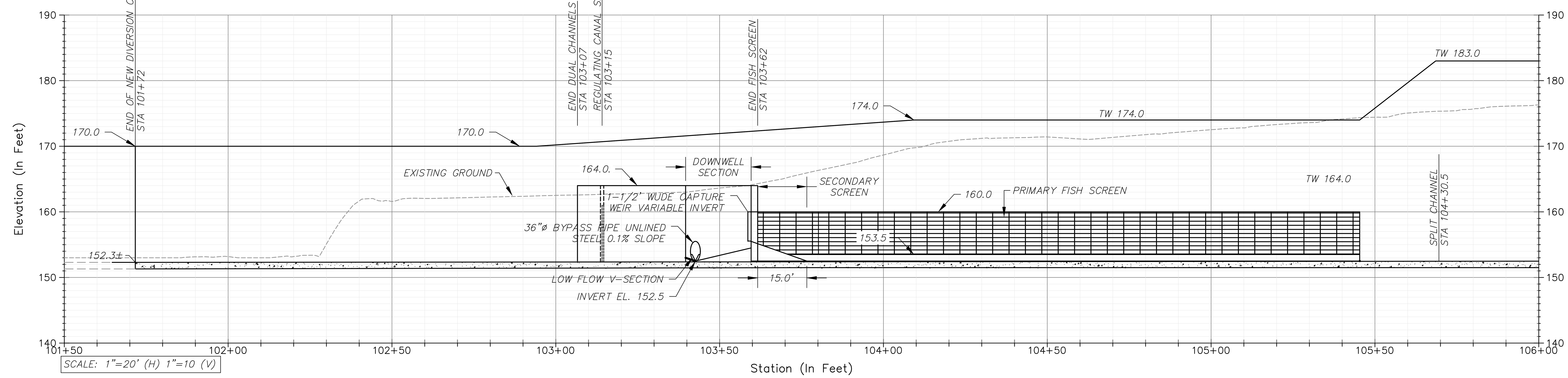
nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

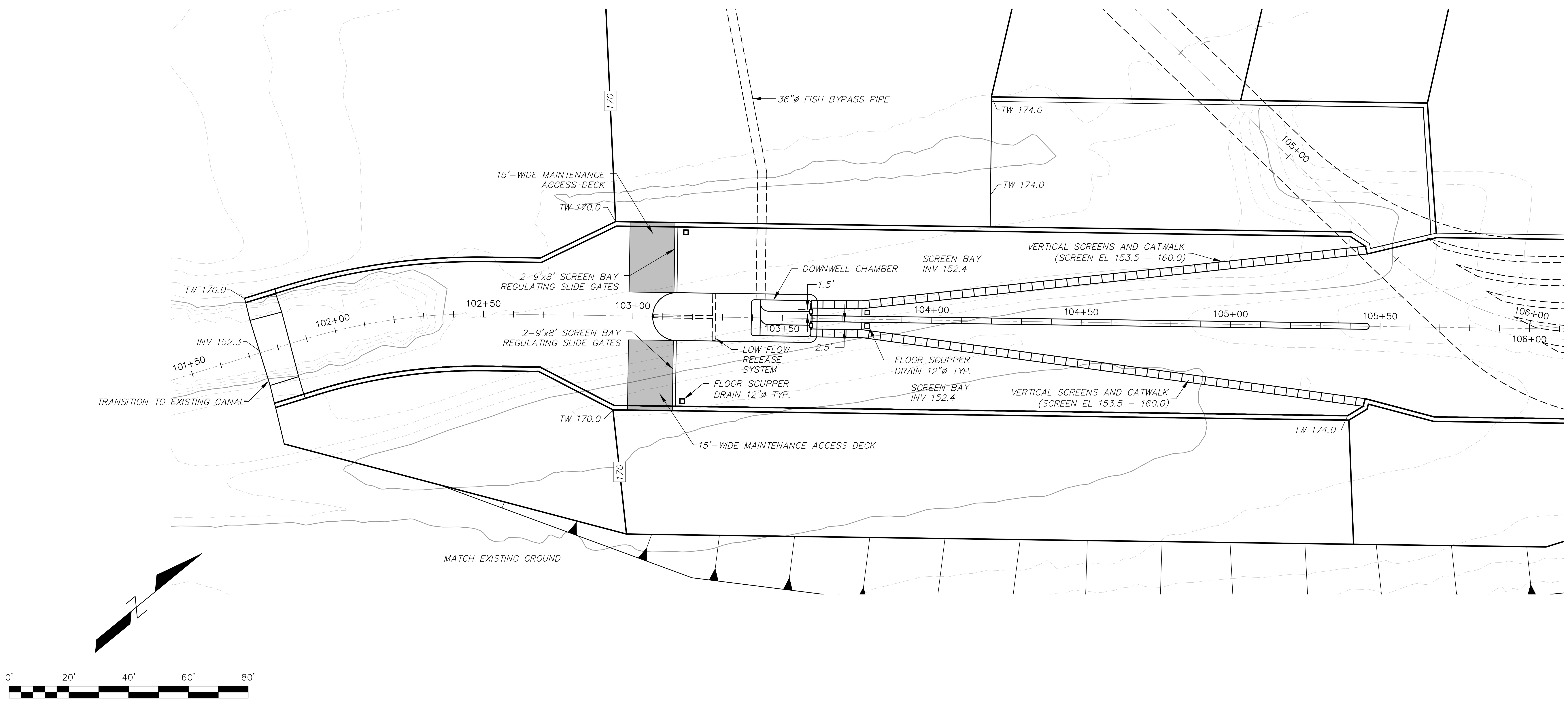
United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

Job Number 6004761

Sheet Number C8

LAST SAVED BY: czhao LAST DATE SAVED: 08/14/2023 09:42
FILE LOCATION: \\MAINFILE-VAN\PROJECTS\ACTIVE\6004761 VERN FREEMAN DAM\96 CAD\02_PRODUCTION\0100_CURRENT\VFD CIVIL 30PCT.DWG





Revisions

No.	Date	Description

Drawing Information

Date	14 August 2023 (09:44)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD CIVIL 30PCT
Plotted Scale	

PRELIMINARY
NOT FOR CONSTRUCTION

Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Diversion Channel Plan & Profile

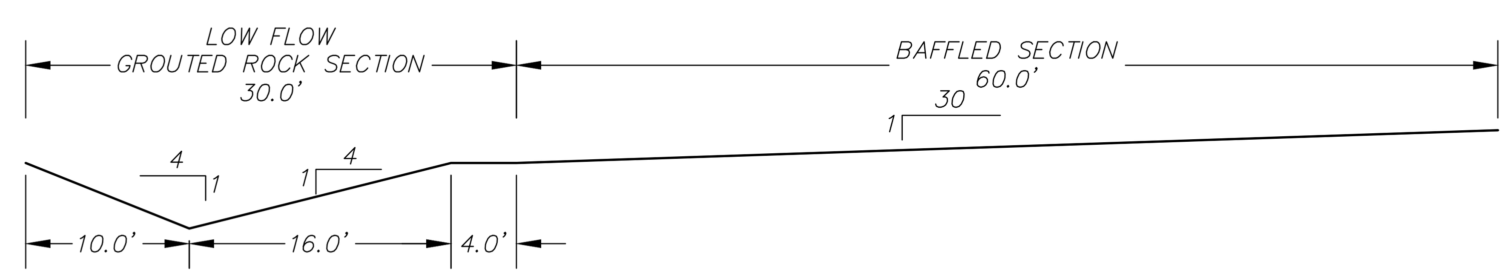
nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101-2479
phone: (626) 440-0080
www.nhcweb.com

United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

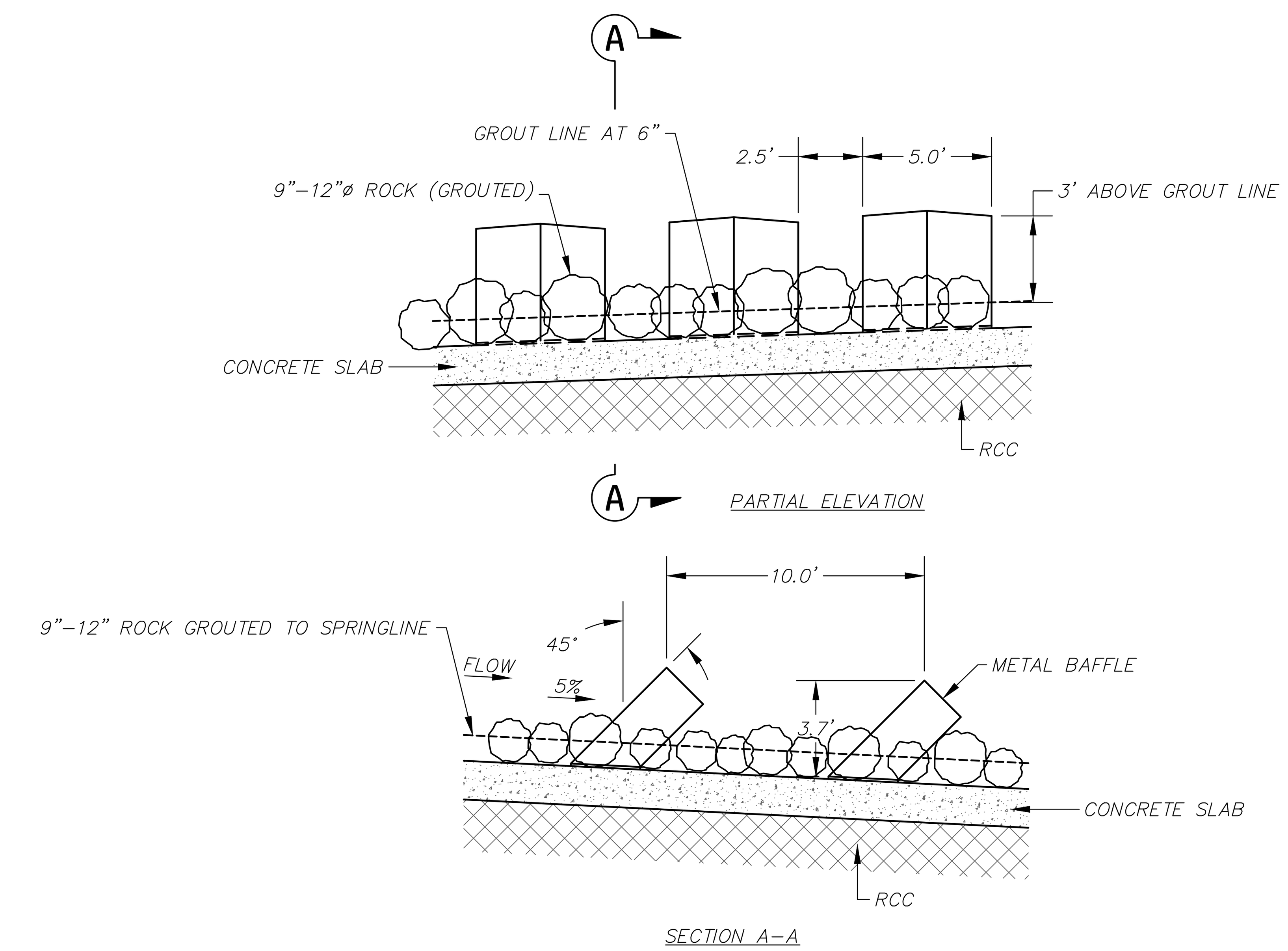
Job Number 6004761

Sheet Number C9

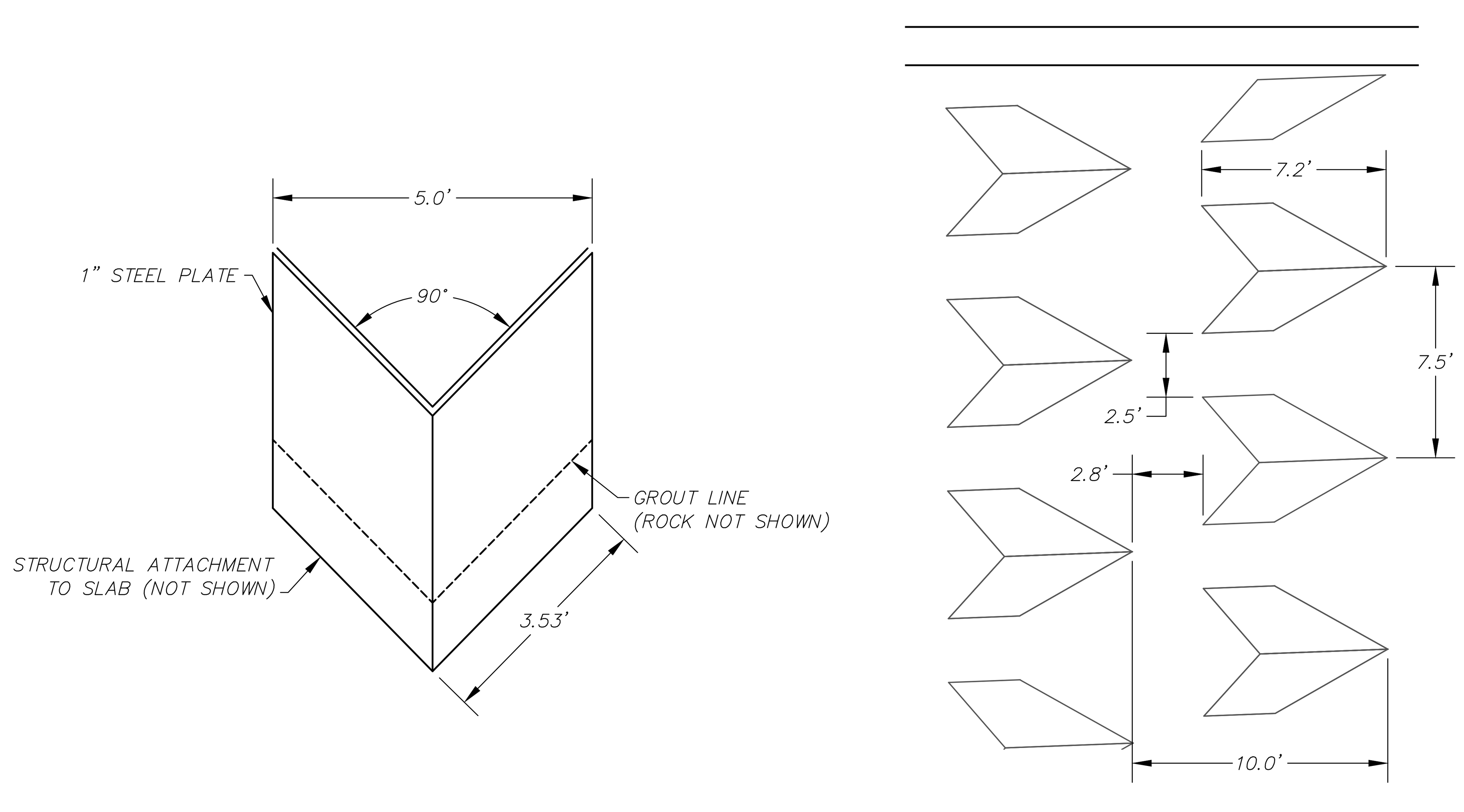
LAST SAVED BY: czhao LAST DATE SAVED: 08/14/2023 09:42
FILE LOCATION: \\MAINFILE-VAN\PROJECTS\ACTIVE\6004761 VERN FREEMAN DAM\96 CAD\02_PRODUCTION\0100_CURRENT\VFD CIVIL 30PCT.DWG



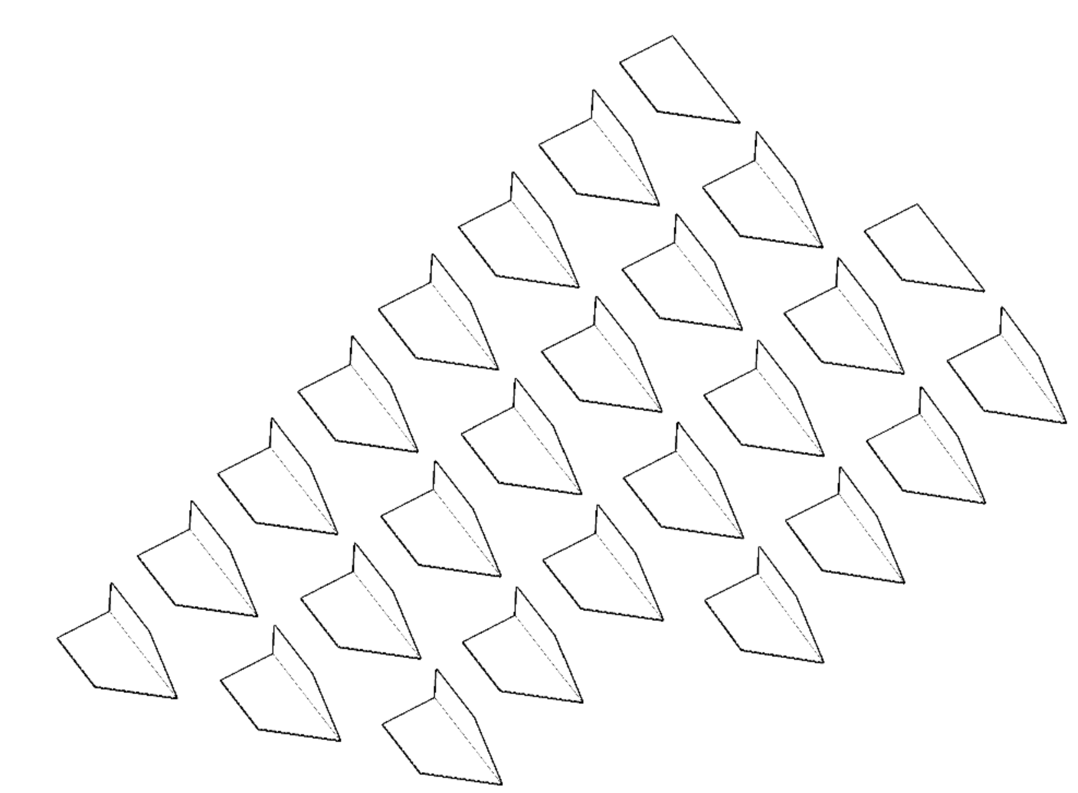
A/D1 RAMP BASE GEOMETRY
Not to Scale



B/D1 TYPICAL BAFFLE
SCALE: 1"=5'



C/D1 BAFFLE LAYOUT
SCALE: 1"=2'



D/D1 BAFFLE GEOMETRY/3D VIEW
SCALE: 1"=10'

Revisions		
Description	Date	No.

Drawing Information

Date	14 August 2023 (09:44)
Status	30% Submittal
Designer	
Drafter	
Checked	
File Name	VFD DETAILS
Plotted Scale	

PRELIMINARY
NOT FOR CONSTRUCTION

Vern Freeman Diversion Hardened Ramp Fish Passage Improvements

Details

nhc northwest hydraulic consultants
200 south los robles avenue, suite 405
pasadena, california 91101–2479
phone: (626) 440–0080
www.nhcweb.com

United Water Conservation District
1701 Lombard Street, Suite 200
Oxnard, California 93030
(805) 525-4431
www.unitedwater.org

Job Number 6004761
Sheet Number D1
Sheet 11 of 11

LAST SAVED BY: czhao LAST DATE SAVED: 05/23/2023 09:55
FILE LOCATION: \\MAINFILE-VAN\PROJECTS\ACTIVE\6004761 VERN FREEMAN DAM\96 CAD\02_PRODUCTION\0100_CURRENT\VFD DETAILS.DWG

APPENDIX B

DESIGN DEVELOPMENT TIMELINE

FREEMAN DIVERSION
HARDENED RAMP

PHYSICAL MODEL DESIGN DEVELOPMENT TIMELINE

Prepared for:

United Water Conservation District
Santa Paula, CA

Prepared by:

Northwest Hydraulic Consultants Inc.
Pasadena, CA

December 2022

NHC Ref. No.5007701

1 DESIGN DEVELOPMENT THROUGH ENGINEERING ANALYSIS AND PHYSICAL MODEL TESTING

Design development of the hardened ramp fish passage alternative used a combination of engineering analysis and physical model testing. Prior to commencing physical model testing, engineering analysis showed that the Mod-6 design of the hardened ramp would not meet the water diversion needs of United Water Conservation District (United) and the Mod-9 design was developed to provide operational flexibility to meet United's water diversion needs.

Baseline physical model testing was used to compare variations of the initial MOD-6 and MOD-9 design geometries (with flushing channel open, closed and removed) through a range of flow conditions. It also provided data that was used to identify areas for improvement and refinement in the subsequent design development phase.

Design development was an iterative process using the laboratory models to test concepts and refine the design. Through multiple visits to USBR's lab in Denver, data and observations from the physical model were collected, these findings were discussed with the design team to identify areas for improvement. NHC then performed engineering analysis to develop new concepts for physical model testing. Where practical, numerical models were also used to do preliminary testing of concepts and to supplement laboratory data.

2 BASELINE TESTING

Below is a brief recap of the baseline testing and key findings for design development. The baseline testing took place from November 2021 to May 2022. Design Development commenced afterwards through October 31st, 2022.

Model	Test Plan	Key Findings
1:24 Baseline Testing	• Testing for a range of higher flows (3000cfs, 6000cfs, 12000cfs and 30000cfs) • MOD-6 and MOD-9 with flushing channel open, closed and removed	• A large sediment bedform consistently developed in front of the intake for all configurations leading to large amounts of sediment being ingested into the diversion • The sediment deposition could not be removed from the initial configurations of the hardened ramp • Considerable deposition into the intake • The upstream approach flow hydraulics caused undesirable conditions across the face of the ramp
1:12 Baseline Testing	• Testing for a range of lower flows (270cfs, 1500cfs, 3000cfs and 6000cfs) • MOD-6 and MOD-9 with flushing channel open and closed	• Since 1:24 baseline testing showed a consistent bedform for all configurations, flushing channel removed scenario was not included to expedite the modeling schedule and to begin design development tests • The MOD-6 Geometry was not able to meet the desired flow splits/yield for 270cfs and 1500cfs • The hardened ramp hydraulics generally looked good, but the low flow channel hydraulics could be improved with more strategic placement of larger rock • The hydraulics at the upstream end of the ramp could be improved.

3 DESIGN DEVELOPMENT PROGRESSION/TIMELINE

Intent of design development is to develop a robust design solution that addresses issues identified in baseline testing. The MOD-9 invert elevation with flushing channel geometry was advanced for development as a result of baseline findings.

Primary issues identified in the baseline testing could be categorized as:

- Approach Flow Hydraulics
- Hardened Ramp Low Flow Channel Hydraulics
- Ingestion of Sediment into the Intake
- Sediment Bedform in front of the Intake

Each issue was addressed systematically with modifications developed through engineering analysis (calculations, numerical modeling, etc.) that were tested in the laboratory and demonstrated during witness test visits.

3.1 Approach Flow Hydraulics

From the baseline testing it was observed that the alignment of the river thalweg (river low-flow channel) upstream of the ramp had a pronounced impact on hydraulics and sedimentation. Notably it impacted the flow distribution into the intake and the hydraulics just upstream of the ramp. Work in design development was focused on removing adverse hydraulic conditions at the ramp and diversion intake, as well as looking into measures to keep or reattach the river low-flow channel (RLFC) to the left bank.

1. Initial formation of ideas to address approach channel hydraulics started during 1:24 witness test, so they could be implemented after baseline testing. (January 2022)
2. CFD Modeling of baseline conditions to study flow distributions. (April 2022)
3. Preliminary Groyne Design and proposed new upstream river low flow channel (RLFC) developed. Initial numerical modeling to support design for physical model. (April 2022)
4. Testing of upstream RLFC realignment closer to the left bank in physical model, showed improvement in hydraulic conditions. (May 2022)
5. Testing of bullnose extension in the 1:24 model, showed improvement in hydraulics into the ramp's baffled section by realigning and removing the adverse approach flow angle. (May 2022)
6. Testing of proposed groynes in physical model show local effect of groynes, however full analysis of the impact of groynes is outside of the boundaries of the physical model will have to be assessed through engineering and river geomorphic analysis. (June 2022).
7. Testing of the ability of the system to re-establish a left bank RLFC using the 1:12 physical model at low flows as part of stress testing. (December 2022)

Key findings: The extension of the bullnose and training the river low flow channel left improved entrance conditions to the ramp and intake. Initial tests with the final training wall to re-establish the low flow channel to the left bank were encouraging.

3.2 Hardened Ramp Low Flow Channel Hydraulics

The baseline testing identified improvements that could be made in the low flow channel portion of the ramp that is utilized for fish passage primarily at discharges under 500 cfs. Iterative process with lots of feedback on the model with the agencies.

1. Observed during the 1:12 tests prior to modeling team laboratory visit, did some preliminary modifications in the model during the agency visit to better understand the system. (May 2022)
2. Developed concepts collaboratively with agencies to test in the physical model which USBR implemented in test sections. (June 2022)
3. Observation of test sections during agency visit, areas for improvement were identified and more modifications were looked at on the model, a general pattern was identified for implementation. Rock was extended up towards the entrance. (August 2022)
4. Updated low flow concept based on feedback from August laboratory visit, updated large rock placement in the low flow channel, concepts were implemented by USBR. (August 2022)
5. Particle image velocimetry (PIV) tests on updated concept from August laboratory visit (September 2022)
6. Demonstration of the updated concept during Agency laboratory visit (October 2022)

Key findings: Updated arrangement and range of rock sizes in the ramp provided more favorable fish passage hydraulic conditions through a range of flows. Extension of roughness elements upstream of the ramp improved entrance/exit conditions for passage.

3.3 Sediment Bedform in Front of the Intake

A persistent bedform (sand bar) was observed in front of the intake for all baseline conditions. Opening the flushing channel was not effective at removing the bedform. This prograding bedform regrades the channel upstream of the ramp and leads to sedimentation against and into the intake. Solutions to remove or manage this bedform were investigated including the addition of a training wall connected to the flushing channel, a concept which is currently used by United to manage sediment deposition in front of their existing intake.

1. Review of potential sediment management options. (April 2022)
2. Initial training wall design, since baseline flushing without one was ineffective. (April 2022)
3. Initial training wall tests in the physical model using initial design of flushing channel and apron, looked at width of the approach channel opening. (May 2022)
4. Updated training wall concept, tested with 2D model for ramp hydraulic impacts. Flushing channel invert lowered from El. 154 to El. 151.5 ft, sluice gate replaced by crest gate, tested in the physical model. (August 2022)
5. Testing of the no flushing channel concept from CDFW to see if it would help to remove bedform. Tests were run for: (1) no flushing channel and re-aligned intake; (2) no flushing channel, realigned intake and bed disturbance features. Some local changes but no major change to overall sediment bedforms. Was not pursued further as a way to remove sediment bedform. (September 2022)

6. Updated training wall and flushing channel concept to increase slope of apron by lowering inlet of Flushing Channel to El. 146ft, while also providing additional temporary storage for sediment. Downstream exit of flushing channel was lowered to 134ft to improve hydraulics into downstream channel. (October 2022)
7. Training wall updated to include “castling” to improve connectivity between the hardened ramp and intake for flow balancing and fisheries. Wall was shown to help with diversity of conditions. (October 2022)
8. Ran a series of stress tests on the final flushing channel and training wall concept to see volume of sediment removed and ramp dynamics. (November 2022)

Key findings: The flushing channel with a training wall was effective at clearing out the sediment bedform in front of the intake. Testing of the no flushing channel concept did not remove the sediment bedform. The combination of lowered flushing channel invert elevation, sloping apron and training wall allowed for shorter duration flushes with greater extent and volume of sediment removed. The castling of the training wall also provided benefits for river training and flow equalization.

3.4 Ingestion of Sediment into the Intake

For all model geometries tested (MOD6, MOD9, with and without flushing channel) a large amount of sediment was ingested into the intake. Design development on a solution to reduce or remove sedimentation was the focus.

1. Review of potential sediment management options including internal sluicing. (April 2022)
2. Development of Desander concept as a way to trap and sluice sediment deposited behind the intake gates. Developed as a concept to be used in conjunction with external features outside of the trash racks. (May 2022)
3. Initial desander concept developed and tested, the concept did not include downstream sluicing channel and was fairly inefficient. Decided to increase the slope of the bays and connect it downstream to the flushing channel exit. (May 2022)
4. Updated desander concept with steeper slopes was developed and tested with preliminary numerical modeling. (July 2022)
5. USBR tests the Desander Concept #2, it was shown to work better but had some issues with internal hydraulics to improve. Discussed increasing the opening height to enhance efficiency. (August 2022)
6. The No Flushing Channel Alternative was tested with turbulence-generating bed disturbance features in front of the intake and a rotated intake. These were shown to produce local changes, but sediment ingestion was qualitatively observed to increase due to higher turbulence causing sediment resuspension (September 2022).
7. Updated the Desander Concept to increase efficiency. This included having a 5ft opening (was 3ft) and lowering the downstream invert. Two of the intake gates were lowered from 156.5 to 155ft. The downstream elevation of the sluicing channel was lowered and the slopes of all channels increased. (September 2022)
8. Tested the updated concept, which was shown to work for a wide range of flows but loses efficacy at higher tailwater conditions, typically associated with higher flows. (October 2022)

9. Increased the number of gates from 4 to 8 at the intake to help with efficacy of the desander, did stress testing over a wide range of conditions. Found that for a fully gate down scenario with unflushed upstream channel more bedload entered the intake and the efficiency plummeted. (November 2022)

Key findings: The desander concept works as an internal sluice to remove sediment that deposits within the intake. The effectiveness depends primarily on the entrance sediment conditions, the downstream water level, and the discharge used. Concept works best in conjunction with an external system, such as a flushing channel. A test with completely lowered gate (no sediment barrier) without flushing the apron resulted in very inefficient operations.

4 SUMMARY

The design development process undertook a systematic collaborative approach utilizing the expertise of the group to improve the design. The process used a combination of engineering analysis, physical modeling, numerical modeling, and design to develop a viable hardened ramp design that meets the needs of United’s water diversion and fish passage for adult Southern California Steelhead.

APPENDIX C

FLOOD HYDRAULICS APPENDIX

VERN FREEMAN DIVERSION
HARDENED RAMP

APPENDIX C

FLOOD HYDRAULICS APPENDIX

Prepared for:

United Water Conservation District
Santa Paula, CA

Prepared by:

Northwest Hydraulic Consultants Inc.
Pasadena, CA

January 2023

NHC Ref. No.5007701

1 2D MODEL DESCRIPTION

Hydraulic analysis of the Santa Clara River in the vicinity of the Vern Freeman Diversion Dam was conducted using the 2D computational routines available in the HEC-RAS hydraulics computation package. Models were developed to represent both existing and with-project conditions. Project components incorporated in the with-project model included the proposed hardened ramp, consisting of a 30-ft wide rock-lined low flow channel and 60-ft wide sloping overbank with local baffle features. The model domain extends approximately 10,000 ft upstream and 11,000 ft downstream of the existing diversion. A general model mesh element size of 50 feet was used throughout the model, with more refinement in the vicinity of the proposed diversion dam and along the hardened ramp. The Courant condition was used to dynamically adjust the computational time step, and the full momentum equations were employed. Model roughness (Manning's n) values ranged from 0.035 to 0.090 along the study reach, with a constant n value of 0.060 applied to the hardened ramp. The model domain draped over the existing condition terrain is shown in Figure 1. The terrain with the hardened ramp incorporated is illustrated in Figure 2.

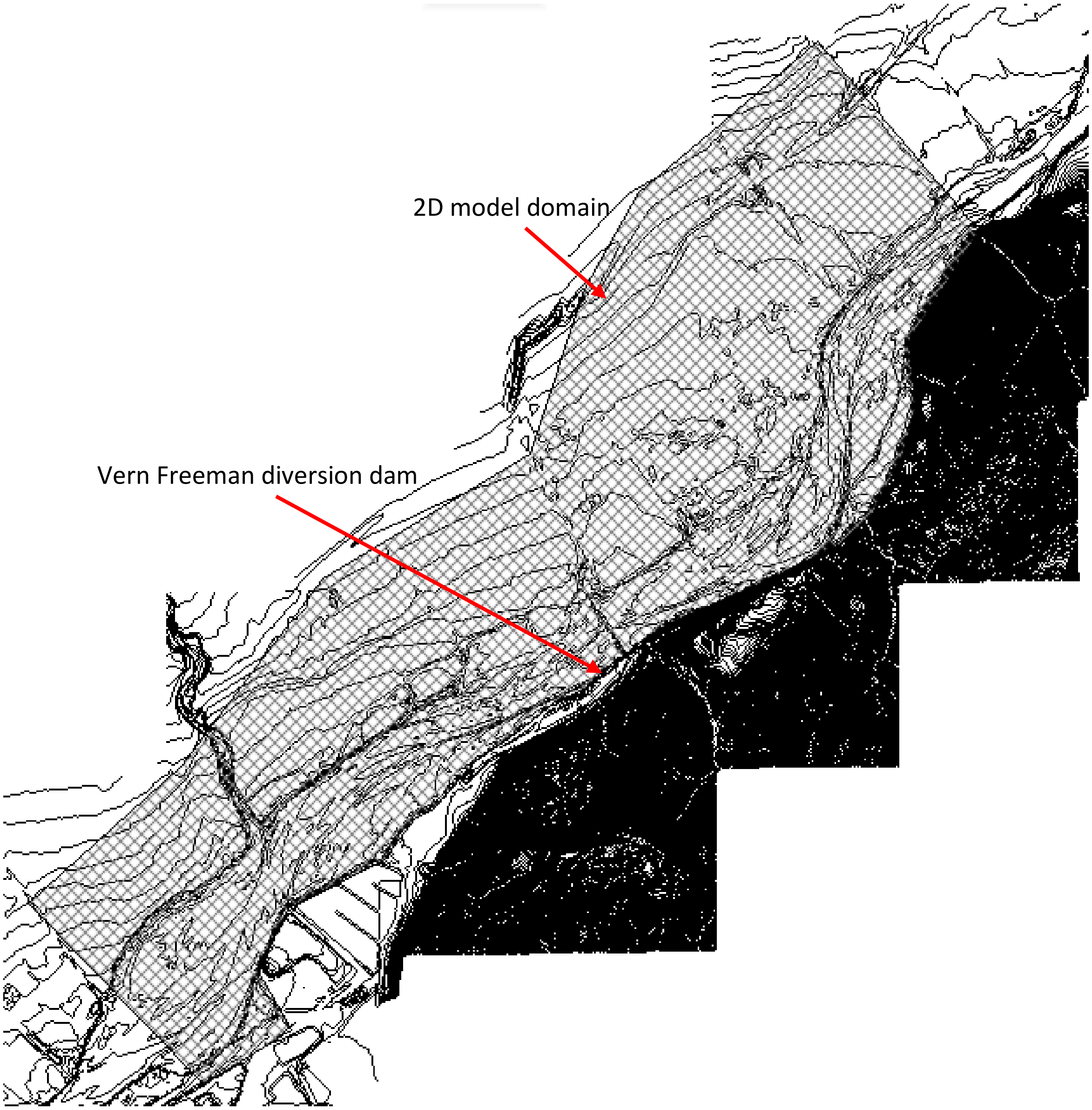


Figure 1 Model domain and existing conditions terrain

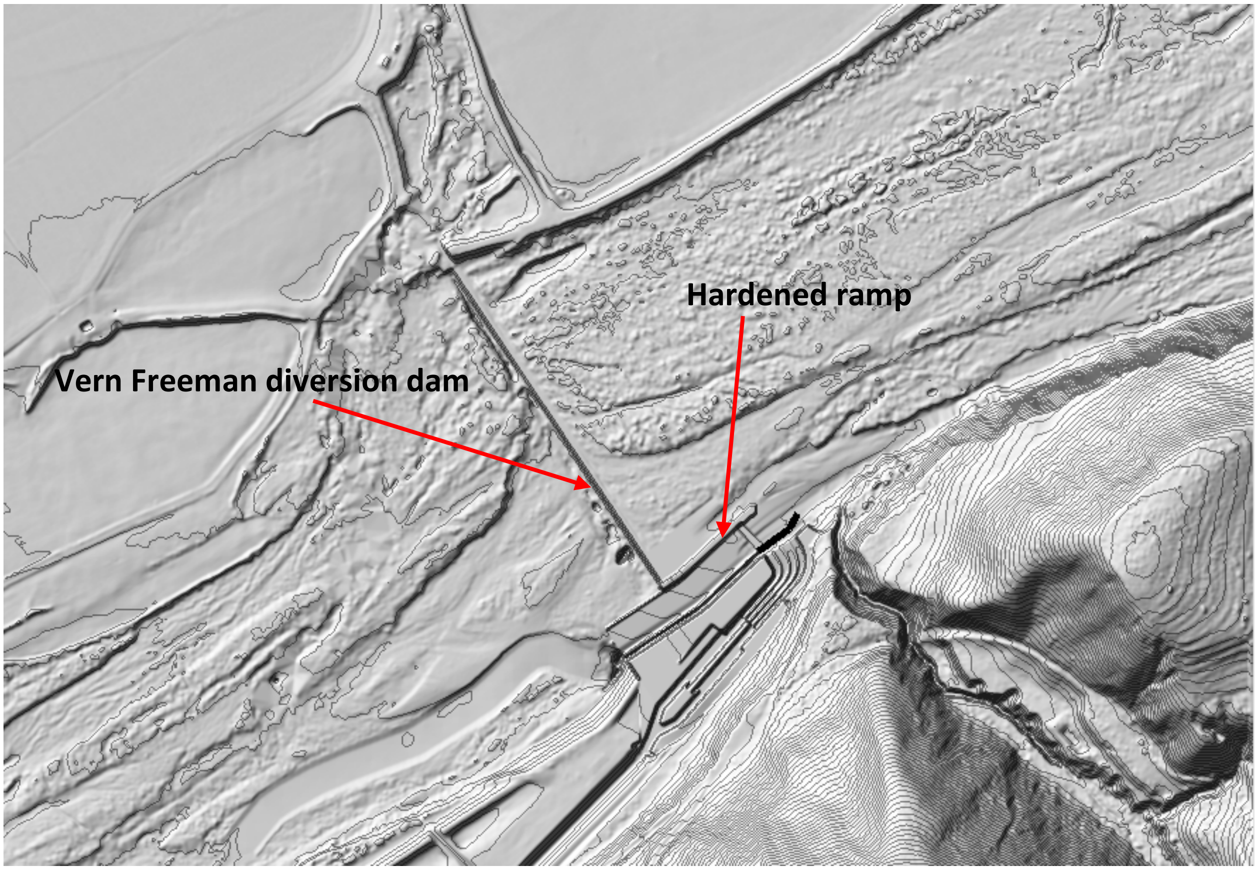


Figure 2 Project conditions terrain

A dynamic simulation was computed for each model, with flows increased from 0 to 226,000 cfs over a 32-hour simulation period. The range of flows applied extend to the peak flow rate associated with the current estimate of the 100-year flood in the lower reach of the Santa Clara River. Peak flow rates for 2-year through 100-year events at the project site are presented in Table 1 (provided by Ventura County, used in AECOM's 2016 sediment transport study to support the hardened ramp hydraulic design). Flows passing over the existing diversion dam over the simulation period are illustrated in Figure 3.

RETURN PERIOD (YEARS)	DISCHARGE (CFS)
2	9784
5	32544
10	59212
25	109384
50	160686
100	226000

Table 1 Flow frequency relationship for project site (from AECOM, 2016)

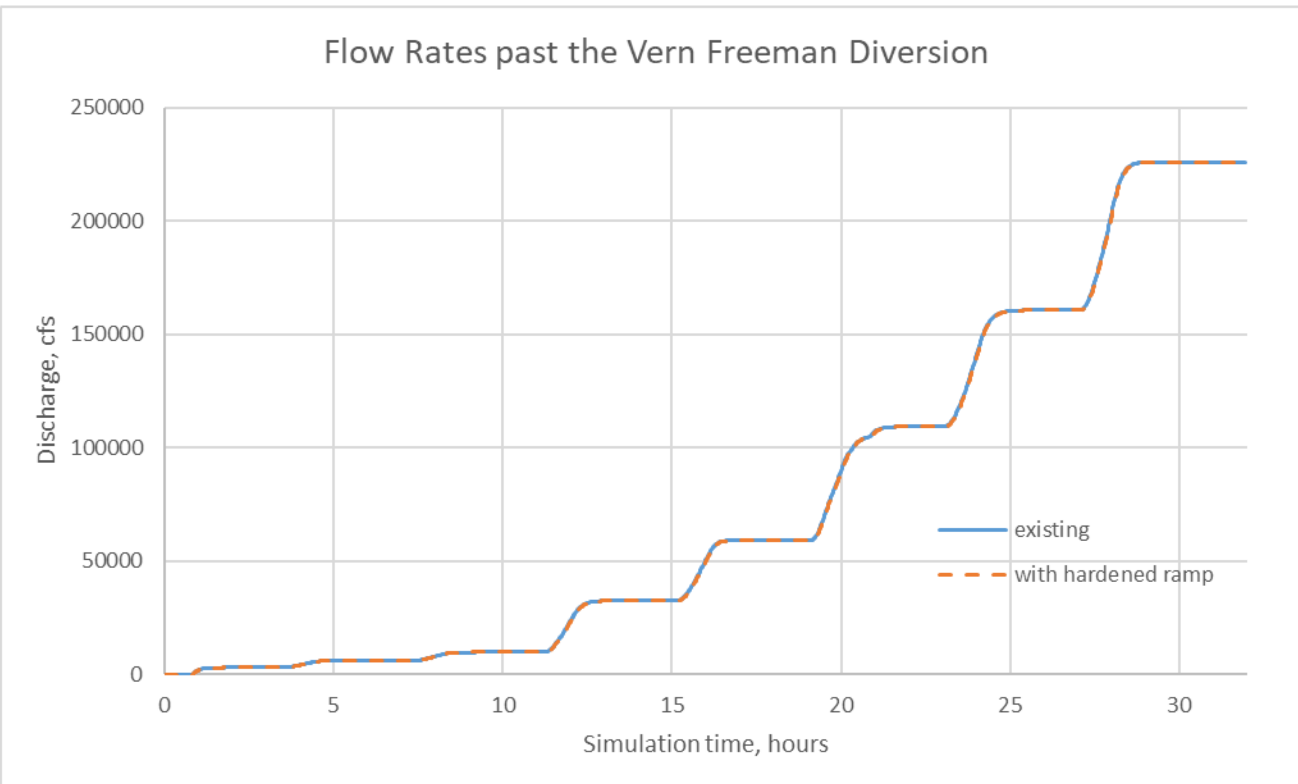


Figure 3 Flows passing the Vern Freeman Diversion Dam during simulations

It is noted that the discharges presented in Table 1 are larger than those used in development of the current FEMA flood maps of the local vicinity. FEMA's peak 100-year discharge is 161,000 in the project reach.

2 2D MODEL RESULTS FOR 100-YEAR PEAK FLOOD FLOWS

Model results at flow rates associated with the 100-year flood peak are contrasted for existing and with-project conditions in Figures 4 and 5. The simulations indicate that the proposed project would drop local flood levels in the project vicinity by as much as 6 feet under 100-year flood conditions. The computed water surface elevation drop is exhibited in an area very local to the proposed ramp, and computed floodplain extents are little changed for the with-project condition. Local flow velocities are significantly increased in the vicinity of the proposed ramp under100-year flood conditions. On the ramp itself maximum velocities of about 26 ft/sec were computed at peak 100-year flow rates. The computed variation in with-project 100-year water surface elevations in the project vicinity is shown in Figure 6.

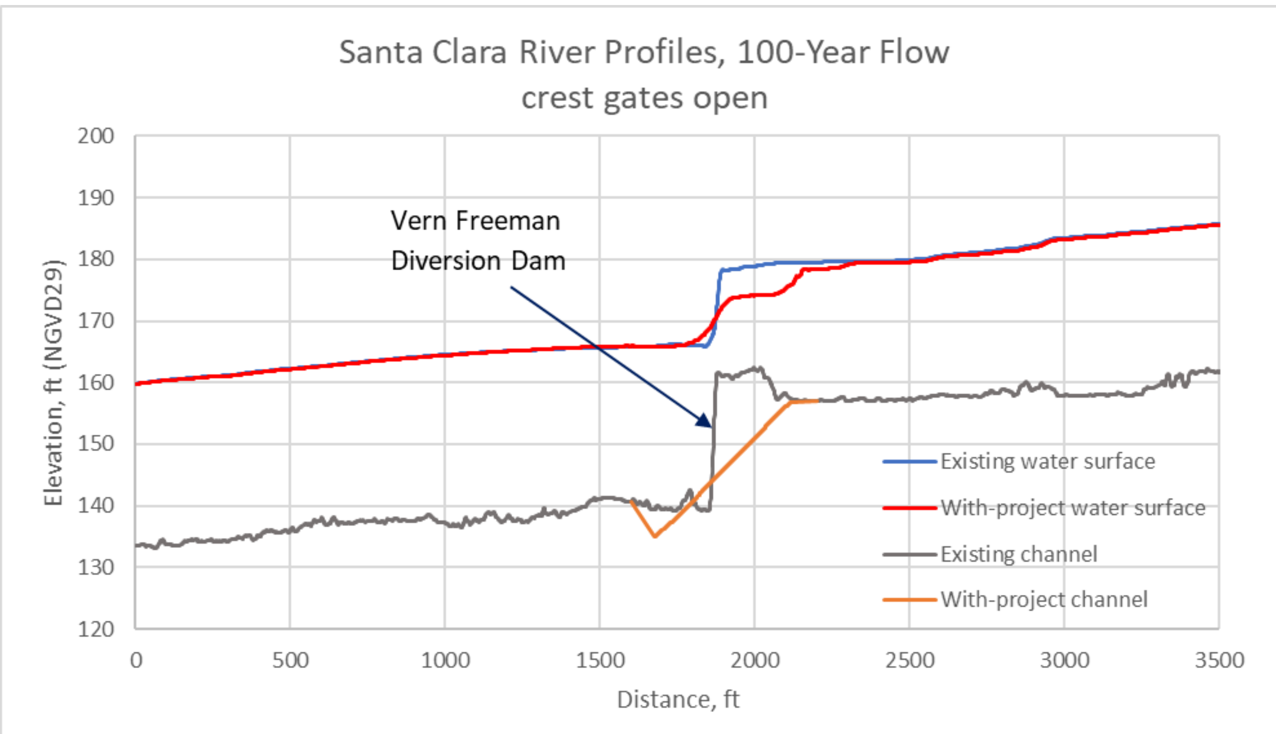


Figure 4 Existing and with-project water surface profiles for peak 100-yr flow conditions

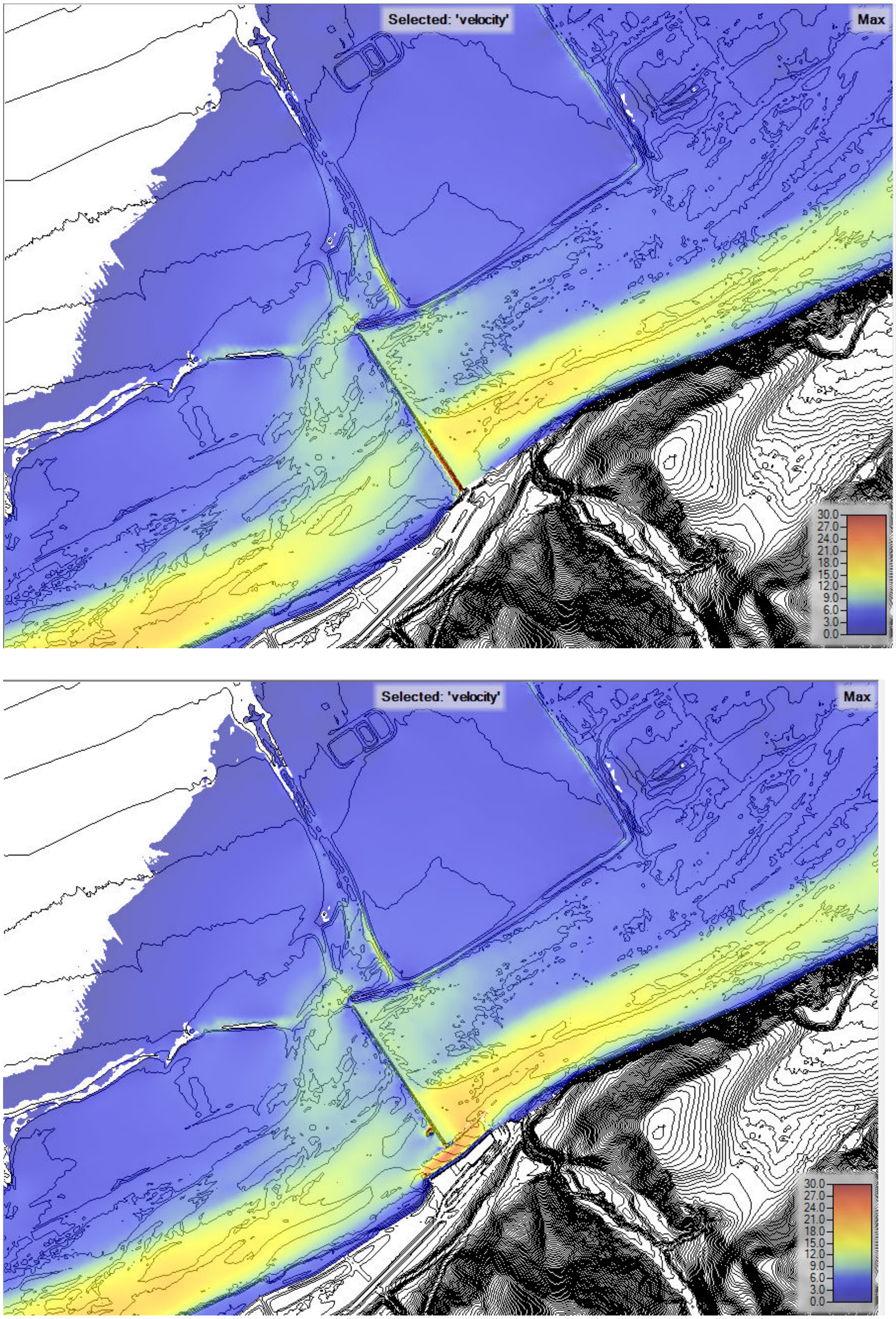


Figure 5 Modeled velocities (ft/s) and flood extents in existing (top) and project (bottom) conditions for peak 100-yr flow rates

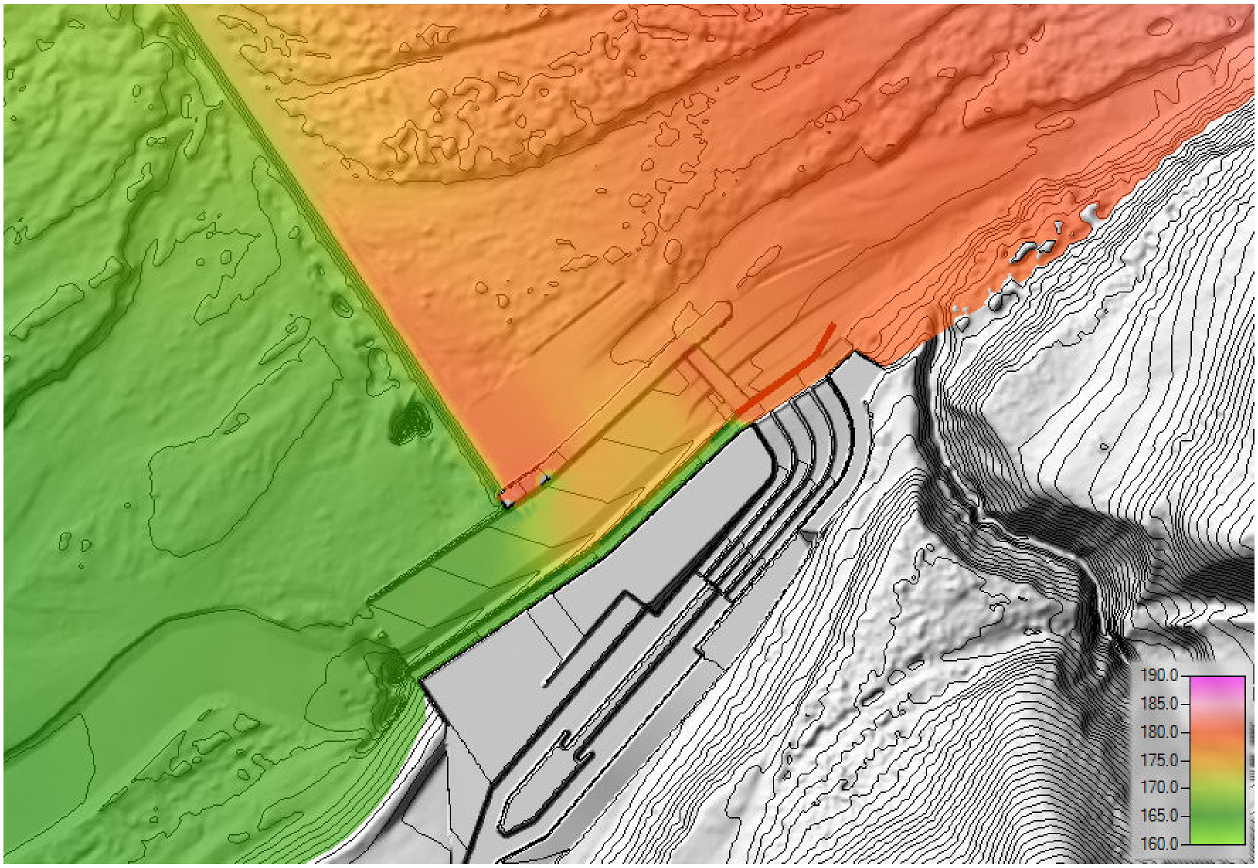


Figure 6 **100-year water surface elevations (ft NGVD29) in the vicinity of the proposed hardened ramp**

With-project simulation results presented in Figures 4-6 represent hard ramp conditions with the crest gates fully open. Profile result for with-project conditions with the crest gates raised to an elevation of 164 ft are presented in Figure 7. Water surface profiles are locally higher under 100-year flood conditions with the crest gates at elevation 164 ft, compared to the fully open condition, but are not significantly different than those computed for the existing condition at locations upstream of the project limits.

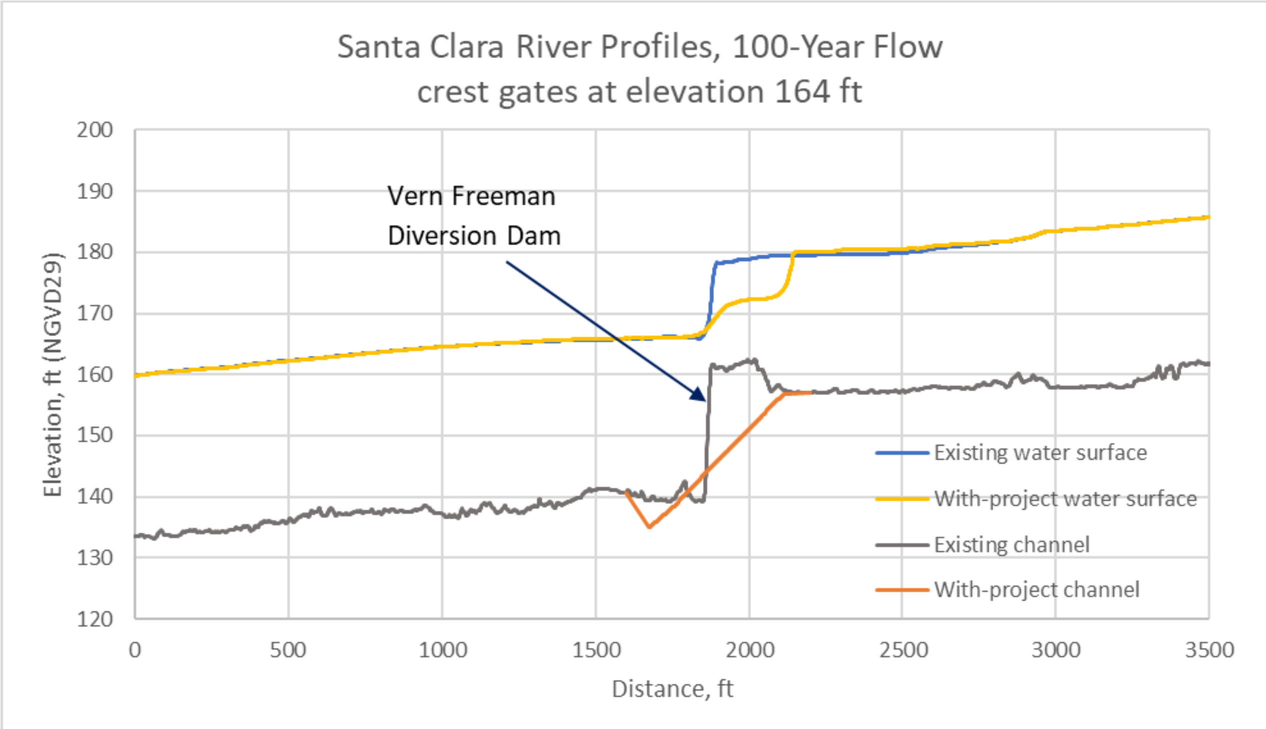


Figure 7 **Channel and flood profiles, 100-year flow, with-project crest gates at elevation 164 ft**