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St. Mary Diversion Dam, Rock Ramp & Fish Screen Model Study

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Hydraulic Investigations and Laboratory Services Group
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14. ABSTRACT
A 1:12-scale physical hydraulic model of St. Mary Diversion Dam Facility was constructed at Reclamation’s Hydraulics Laboratory in Denver, Colorado to examine the hydraulic performance of the proposed replacement of the existing facility. Replacement structures include a broad crested weir dam, rock ramp with the low flow channel for energy dissipation and fish passage, a sedimentation sluice channel with two sluice gates, a trash rack and diversion headworks with a fish screen, and a fish bypass in the diversion canal. The model study examined hydraulics and operational performance of the facility. Approach and sweeping velocities near the flat plate fish screens were measured and documented at the varying river and diversion flow rates. A brief summary of the testing of several initial designs is given, with more detailed information on the tests of the final rock ramp design.

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St. Mary Diversion Dam, Rock Ramp & Fish Screen Model Study

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

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Cover Photo: St. Mary Diversion Dam viewed from upstream in 2012 (Photo by: Bryan Heiner)

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A 1:12-scale physical hydraulic model of the St. Mary Diversion Dam Facility was constructed at Reclamation's Hydraulics Laboratory in Denver, Colorado to examine the hydraulic performance of the proposed replacement facility. Replacement structures include a broad crested weir dam, rock ramp with low flow channel (for energy dissipation and fish passage), a sedimentation sluice channel with two sluice gates, a trash rack and diversion headworks with a fish screen, and a fish bypass in the diversion canal. The model study examined the hydraulic and operational performance of the facility. Approach and sweeping velocities were measured near the flat plate fish screens at diversion flow rates from 600 ft^3/sec to 850 ft^3/sec with varying river flow rates.

The model study was performed in close cooperation with the design engineers, with the primary objectives of verifying the hydraulic performance of the project features and assisting the designers with the layout of the dam, rock ramp, low flow channel, fish screen and other appurtenant structures.

Rating curves were developed for the dam structure with no diversion flow and with canal diversion flow rates of 600 and 850 ft^3/sec. Hydraulic conditions along the fish screen were evaluated to ensure the adequacy of the fish screen for the protection of juvenile bull trout, a threatened species. Approach velocities were set less than or equal to 0.8 ft/sec (NMFS 1997) using baffles located 0.75-ft behind the screen face set at 25% open area for screens 1-10, 20% open area for screens 11-20 and 17.5% open area for screens 21-30. Fish are bypassed back to the river through a rectangular bypass channel with a flow of approximately 40 ft^3/sec.

During non-irrigation season all flow is passed through the off-season sluice channel on river left. Up to 400 ft^3/sec can pass through the sluice channel without overtopping the entrance sill of the trash rack. Stop log slots are also available to provide sluicing of up to 750 ft^3/sec when the stop logs are installed. Velocities on the left side of the sluice channel (nearest the trashrack) are higher than those on the right due to the curvature of the river upstream of the sluice, making multiple passage routes for upstream migrating fish.

During diversions, approach velocities upstream of the trashrack remain below the 2 ft/sec design velocity. Gate operations were evaluated for diversion flow rates of 600 ft^3/sec and 850 ft^3/sec (640 ft^3/sec and 890 ft^3/sec through the headworks when the bypass is considered). It is recommended that at least 4 gates be operated at a time for the 600 ft^3/sec diversion and at least 6 gates open for the 850 ft^3/sec. This produces good approach velocities along the fish screen. In emergency situations a minimum of 2 gates can be used for a 600 ft^3/sec canal diversion and a minimum of 3 gates can be used for a 850 ft^3/sec canal diversion, approach velocities along the fish screen with the minimum gates open will not be uniform.

Velocities and flow patterns down the rock ramp are in a range that should allow most species of fish to find a passage route over the diversion dam. The low flow channel of the rock ramp will be completely grouted and provide passage routes for all species up to about 125 ft^3/sec with a maximum velocity of 4.4 ft/sec. The rock ramp should also allow successful passage up through 10,000 ft^3/sec where velocities can be as high as 19.2 ft/sec but vary across the entire rock ramp due to the roughness of the rock and slight irregularities in the ramp surface.

Introduction

The St. Mary Diversion Dam is part of the Bureau of Reclamation's (Reclamation) Milk River Irrigation Project in north-central Montana. The dam is located just east of Glacier National Park near Babb, Montana, 0.75 miles downstream of Lower St. Mary Lake as shown in Figure 1. It consists of a 198-foot-long and 6-foot-high concrete weir and sluiceway that diverts water from the St. Mary River into the 29-mile-long St. Mary Canal. The canal passes through two steel-plated siphons and five concrete drop structures before discharging into the North Fork of the Milk River. Construction was completed for both the diversion dam and canal in 1915.

In 1999, the U.S. Fish and Wildlife Service (FWS) listed bull trout (*Savelinus confluentus*), a species native to the St. Mary River drainage, as a threatened species under the Endangered Species Act (ESA). This listing requires that Reclamation provide improved passage and protection in any modification or replacement of the dam facilities. When listed, FWS concluded that the St. Mary Diversion Dam negatively affects native bull trout by creating a seasonal barrier to upstream migration (Mogen and Kaeding 2005a, 2005b and 2008) and by causing entrainment of fish into the St. Mary Canal during the irrigation season (Wagner and Fitzgerald 1995, Mogen and Kaeding 2000 and 2002).

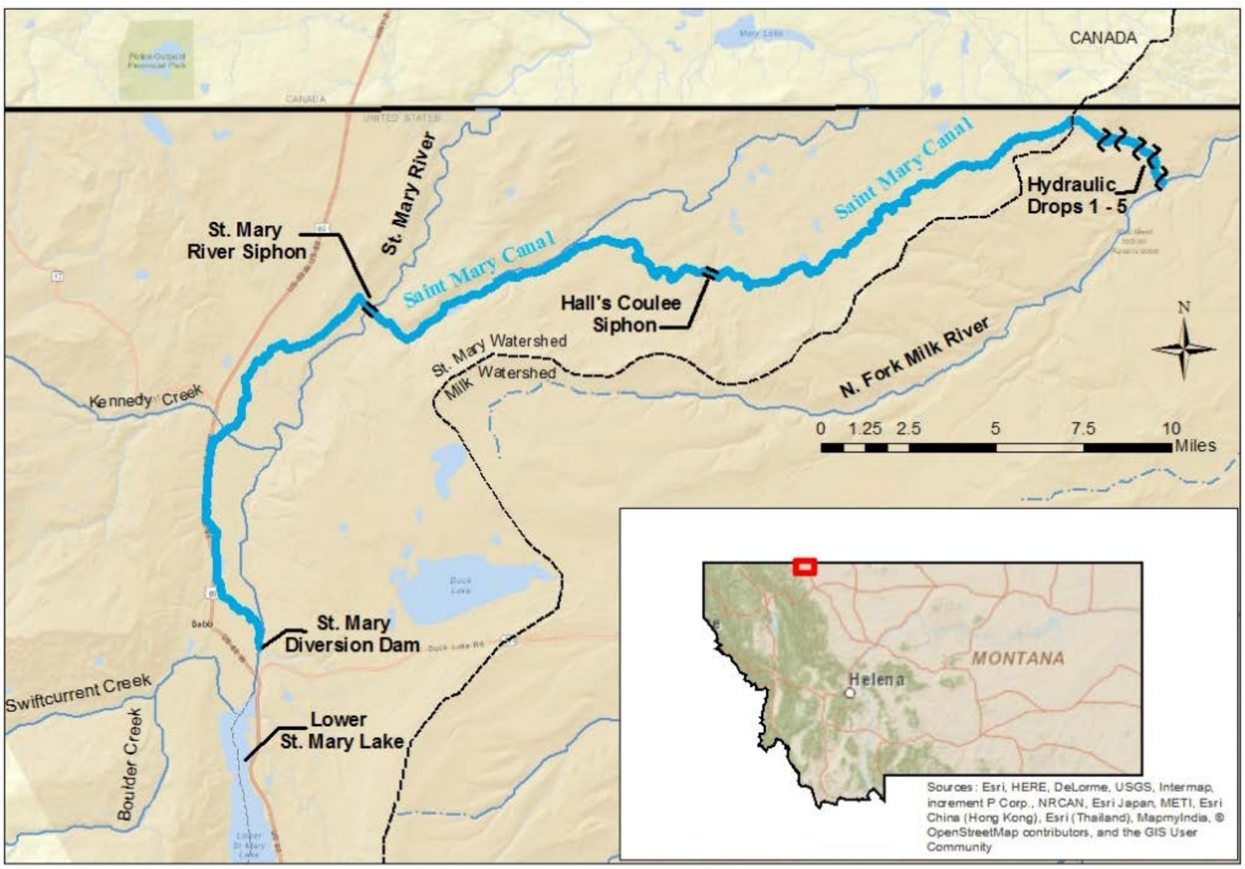

Figure 1. An overview map of the St. Mary Diversion Dam and related features.

The St. Mary Diversion Dam is a concrete ogee-shaped, uncontrolled weir with an overhead abandoned 3-span truss bridge (one span has been removed Figure 2), and downstream horizontal slab. The spillway is ungated and has no mechanical features but is subdivided into two equal sections by a concrete bridge pier supporting two abandoned 97.5-foot Pratt trusses that span the crest of the spillway. Each section of the spillway weir is 94 feet 10 inches wide. The weir crest elevation is 4471.0 ft, and was raised to elevation 4472.0 ft by mounting 1 ft high weir-boards on top of the entire spillway crest. Figure 3 provides an overview of the spillway from the upstream channel. The sluiceway is located to the left of the spillway and consists of six openings with inverts at elevation 4466.0 ft controlled by 4- by 4-inch stoplog planks (Figure 4). The St. Mary Canal headworks is located on the left abutment and is controlled by eight 5x5.5-foot headgates with gate sills at elevation 4466.0 ft (Figure 6). The headworks is a concrete structure 59 feet wide, 22 feet long, with an upstream weir in front of the gates with a crest at elevation 4467.0 ft.

The 29-mile St. Mary Canal was constructed between 1907 and 1915. The unlined canal was designed to convey 850 ft^3/sec at a flow depth of 9 feet. The canal was excavated to a bottom width of 26 feet with 2:1 side slopes at a channel invert slope of 0.000095 (Interior 1981). The diversion dam is used to divert water into the canal from March through September. During the non-diversion period, the sluiceways are opened and the canal headgates are closed. Although the canal was designed to convey 850 ft^3/sec; the condition of the canal limits diversion to a maximum of 650 ft^3/sec with current operations typically around 600 ft^3/sec. During March and early April, all river flow more than about 100 ft^3/sec is typically diverted. From June to August, diversions often reach 75 percent of total river flow. Diversion decreases sharply in late August and September (Mefford 2003).

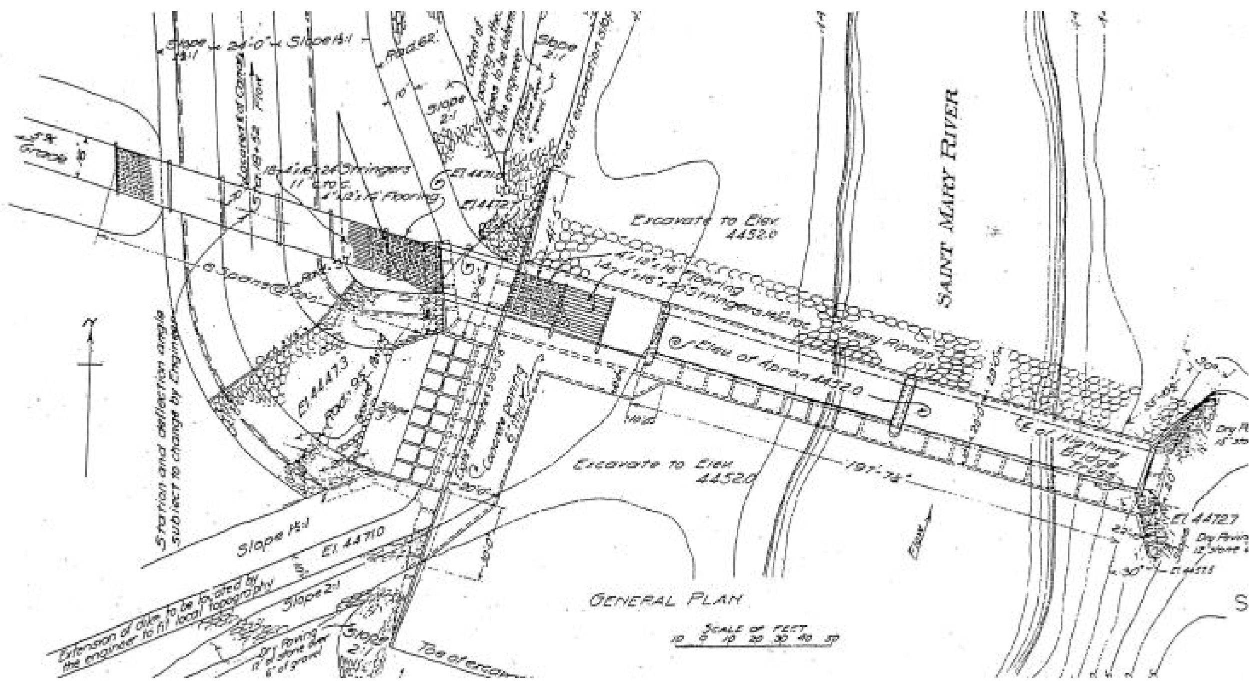

Figure 2. Original plan view drawing of the existing St. Mary Diversion Dam and Headworks (January 1914. Note that elevations are based on project datum that is approximately 4.27 meter (14 feet) lower than NAVD88)

Figure 3. Existing St. Mary Dam Diversion weir, sluiceway and abandoned bridge during low flow fall season (the view is from the upstream left bank).

Figure 4. St. Mary Diversion sluiceway gates (the view is from the upstream left bank, standing on top of the headworks structure).

Figure 5. St. Mary Diversion headworks structure (the view is from downstream of the headworks, with flow towards the reader).

Recent examinations of the 100-year old diversion dam, headworks, and canal revealed substantial freeze-thaw damage to exposed concrete surfaces. Concrete core samples taken from the piers on the dam and sluiceway indicated the condition of the concrete is very poor where exposure to ice and frequent freeze-thaw cycles have degraded the strength (Mefford 2003). Based on available inspection data and visits to the structure, the weir (Figure 6), abutments (Figure 7), piers (Figure 8), sluiceways (Figure 9) and diversion headworks will likely all require demolition and replacement in the near future. Each year, continued modifications and repairs to the facility are made which greatly reduce the risk of catastrophic failure of the structures until replacement is possible.

Figure 6. St. Mary Diversion weir concrete failure and exposed rebar.

Figure 7. St. Mary Diversion right abutment damage and exposed rebar.

Figure 8. Concrete degradation on the downstream end of piers surrounding sluice bays.

Figure 9. Concrete degradation on the upstream sluice bay pier noses.

Project Background & History

Over the past 15 years Reclamation's Technical Service Center (TSC), Reclamation's Montana Area Office (MTAO), U.S. Fish and Wildlife Service (FWS), The Blackfeet Nation, Montana Department of Natural Resources and Conservation (DNRC), Milk River Irrigators, St. Mary Rehabilitation Working Group, National Marine Fisheries Service (NMFS) and many other stakeholders have joined in a collaborative effort to help protect the threatened bull trout at the St. Mary Diversion.

Early Design Concepts

In 2003, Reclamation conducted a conceptual design study and outlined two potential concepts (Mefford et al. 2003). Concept 1 recommended rehabilitating the diversion weir and replacing the headworks and sluiceway (Figure 10). Concept 2 recommended replacing all of the existing structures (Figure 11). Both concepts included fish screens to prevent entrainment in the canal and a rock channel fishway to allow passage over the dam. Both concepts proposed using a flat plate fish screen in the canal with a bypass at the end of the screen to return fish the river. Design goals were to maintain approach velocities of 0.4 ft/sec, provide a 1-ft sill on the bottom of the screen and limit fish exposure to the screen to 60 seconds or less. The 1-ft sill was to provide better protection of bull trout, recognizing the bottom-oriented behavior of bull trout (Beyers et al. 2002). The rock channel fishway for both options would provide passage on river right by extending a rock fishway at a 3.5 percent slope approximately 150 feet downstream of the weir. The difference between the two options was the extent to which the existing weir structure was rehabilitated or replaced. Each design was sized to allow flows of 650 (current capacity), 850 (original design capacity), and 1,000 ft^3/sec (increased capacity) into the canal. The 2003 construction costs for concept 1 and concept 2 were estimated to be \$6.9M and \$10M, respectively.

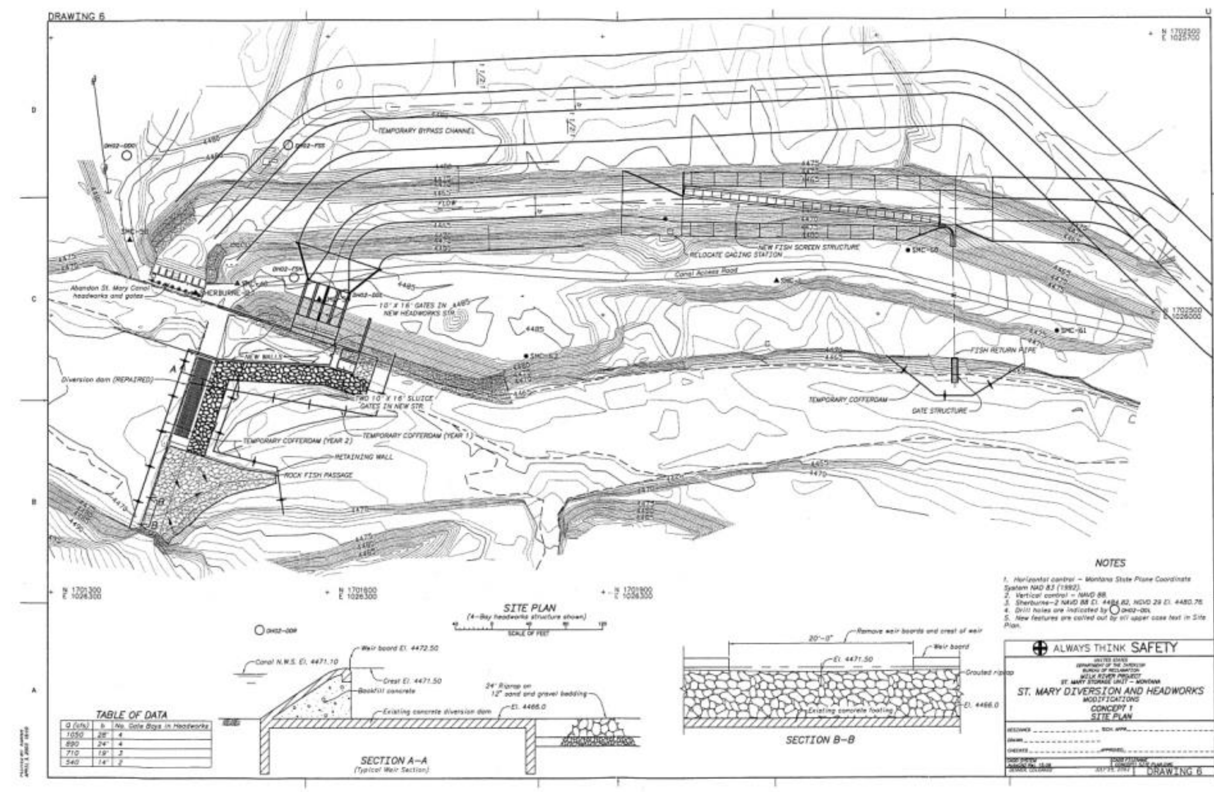

Figure 10. St. Mary Diversion rehabilitation concept 1 as presented in Mefford (2003).

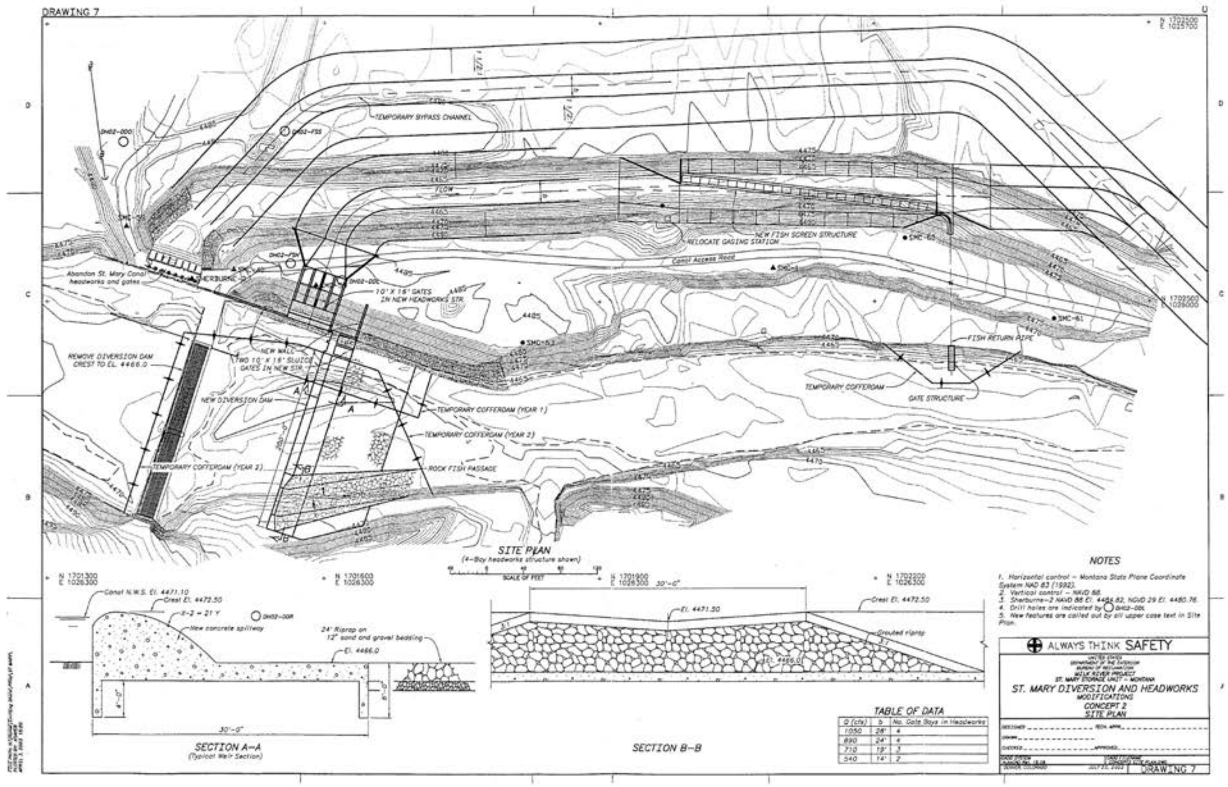

Figure 11. St. Mary Diversion rehabilitation concept 2 as presented in Mefford (2003).

Since the 2003 concepts were put forward, many other configurations have been proposed and investigated by a wide range of individuals and groups. TD&H Engineering Consultants (TD&H 2006) provided cost estimates and recommendations for numerous alternatives to repair or replace the existing diversion and canal.

More recently, the MTAO funded Reclamation's Technical Service Center (TSC) to develop a replacement of the existing diversion facility. The TSC has worked in conjunction with stakeholders to develop a design that met as many expectations and requirements as possible. Team discussions and design reviews suggested more than 6 different viable diversion and headworks options. Notable designs included a 3-foot dam raise, a rock ramp to provide passage and many different configurations of concrete fish ladders and fish screen locations. The complete extent of these options is not provided in detail in this report as many were not developed beyond the feasibility level.

2014 Kinked Ogee Crest 60% Design

In January of 2014, MTAO and other stakeholders met to complete a 60 percent design review for the St. Mary Diversion Dam and Headworks. The designs were developed by the TSC with a multidisciplinary team led by Jason Wagner, P.E. of the TSC Civil Structures Group. The 60 percent design was to be robust, simple and easy to operate. The fish ladder and fish screen were designed appropriately for "fingerling" (60 mm) juvenile bull trout with a maximum screen approach velocity not to exceed 0.80 ft/sec at 1,080 ft^3/sec flow in the canal. Typical design criteria for salmonids would usually only allow 0.4 ft/sec approach velocity but the multi-agency biological review team obtained an exception (Reclamation 2012) due to the species and life stages present at the facility. Field investigations at the unscreened St. Mary diversion determined that essentially all (98%) of salmonids and 100% of bull trout sampled from 2002-2006 were greater than 60 mm in length (Mogen et al. 2011).

Figure 12 provides a plan view drawing of the proposed 2014 60% design. Included in the design are the following features: A) New diversion dam located downstream of the existing structure with a kinked platform and an ogee shaped profile and an offset weir height to align flow and allow more flow on the right bank of the river. B) New sluice bays and overshot bays on both the right and left side of the dam adjacent to the abutments to allow sediment and floating debris removal and aid in ladder attraction. C) New headworks that consists of a trashrack with a maximum 2 ft/sec approach velocity and nine 5x5.5-ft steel slide gates. D) New fish screen 180-ft long by 7.5-ft tall and fish bypass to protect fish from entrainment. E) All species fish ladder located on the right bank of the river. F) New lowered sluice channel on river left to allow non-irrigation flows to pass without inundating the headworks.

Due to the complexity of the project, TSC constructed a 1:12 physical model (Heiner & Shupe 2016) of the complete structure and several numerical models of individual components of the proposed 60% design to ensure that all the hydraulic and structural components would function as intended and that the project would succeed in both protecting bull trout and providing the necessary diversion to the St. Mary Canal. Figure 13 provides an overhead view of the 2014 physical hydraulic model with annotations. Testing the physical model helped the design team identify several areas of concern with the proposed design. The following items were noted from the physical and numerical models: A) The kinked ogee crest dam caused severe scour downstream of the apron which resulted in an unstable river bed. B) The entrance to the fish ladder would be difficult for species to locate due to extreme turbulence and eddies. C) The headworks were too large, resulting in small gate openings to operate the canal at the desired flowrates. D) The overshot bays intended for flushing floating debris over the dam were not in ideal locations and would require excessive operational adjustments and maintenance. E) Operation of the facility on two sides of the river would be difficult due to the access limitations on the right bank. F) Having the fish ladder on the opposite side of the river from the headworks required maintaining two thalwegs which created additional complexity and increased operation and maintenance costs.

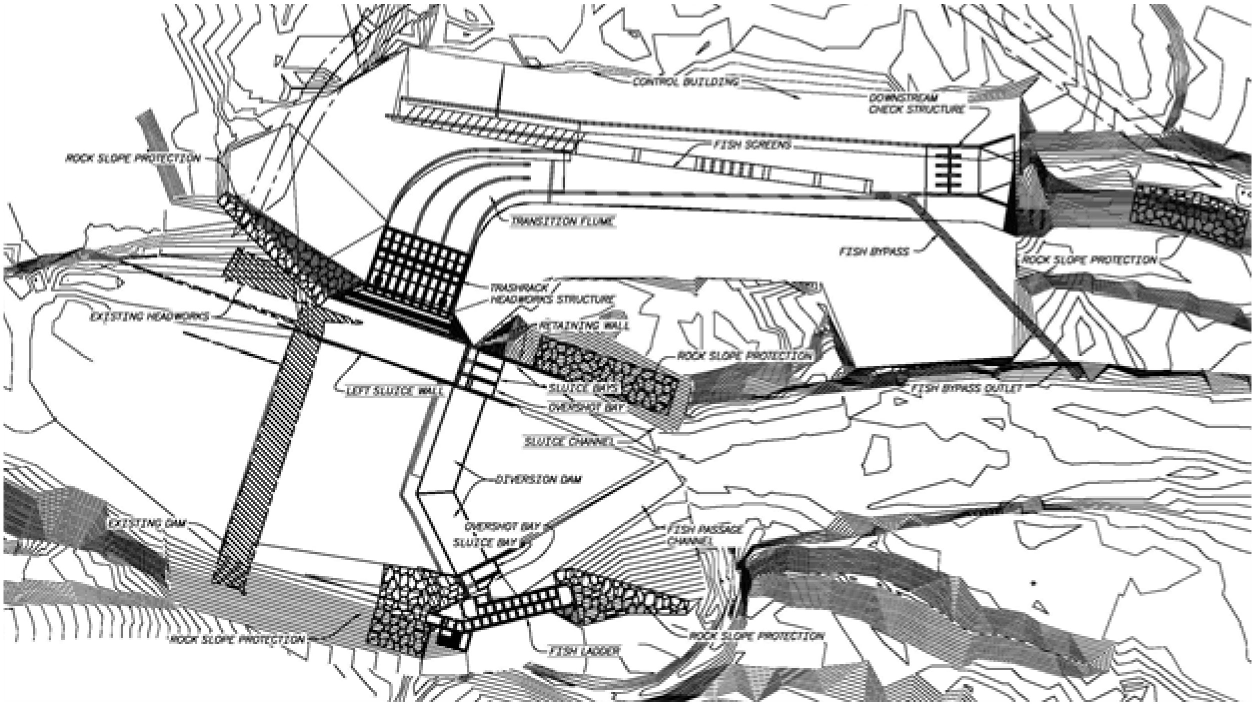

Figure 12. Plan drawing of the 2014 60% design of the St. Mary Diversion Dam and Headworks.

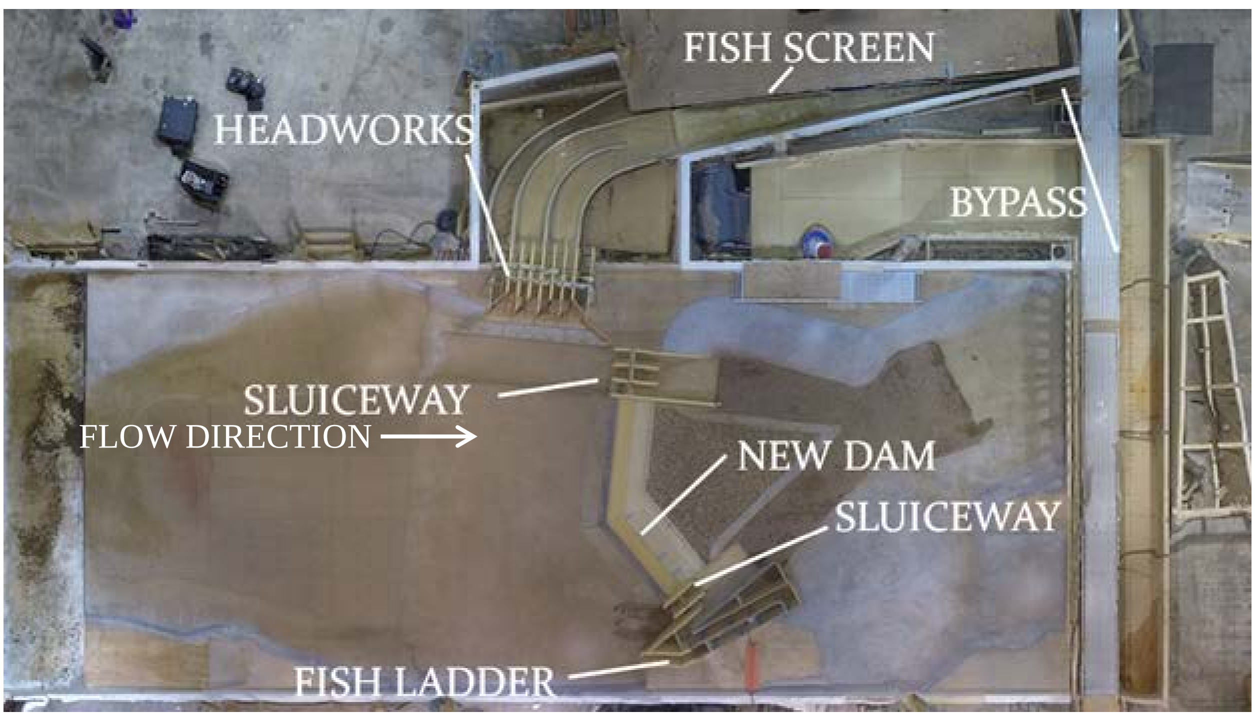

Figure 13. Aerial photograph of the 2014 60% design physical model with annotations.

2015 Linear Broad Crested Weir 60% Design

Following the first modeling efforts project members re-worked the 60% design in 2015 and incorporated the changes shown in Figure 14 (Heiner & Shupe 2016). Modifications included in the 2015 60% re-design are: A) A broad crested weir with reduced length (now 183-ft) and suitable energy dissipation basin downstream to prevent scour. B) The fish ladder located on river left to allow for easier operation, provide better attraction and limit the access needed on the right bank. C) The headworks was reduced from nine to six 5x5.5-ft gates. D) The overshot bays were removed on both sides of the diversion dam and the right bank sluice gate was removed to reduce O&M requirements. Figure 15 provides an aerial view of the modifications to the physical hydraulic model with annotations. Additional information obtained during the model study enabled design team members to achieve the following: A) Set baffle configurations to allow uniform approach velocities for the fish screen at diversion flow rates of 650, 850 and 1080 ft^3/s. Velocity uniformity was un-achievable without baffling due to the screens proximity to upstream gates and channel curvature. B) Generate head discharge relationships for the dam with and without the fish ladder and headworks operational to verify that upstream water surface elevations were not too high and to provide a reasonable discharge curve for each structure. C) Verify the amount of headloss produced by the headworks and fish screens to ensure canal discharges could be met with adequate canal depths. D) Confirm that all flow during the non-irrigation season can pass through the sluice gates without inundating the headworks with water. E) Ensure that fish exiting the ladder would not be immediately swept into the headworks and back to the river via the fish screen and bypass.

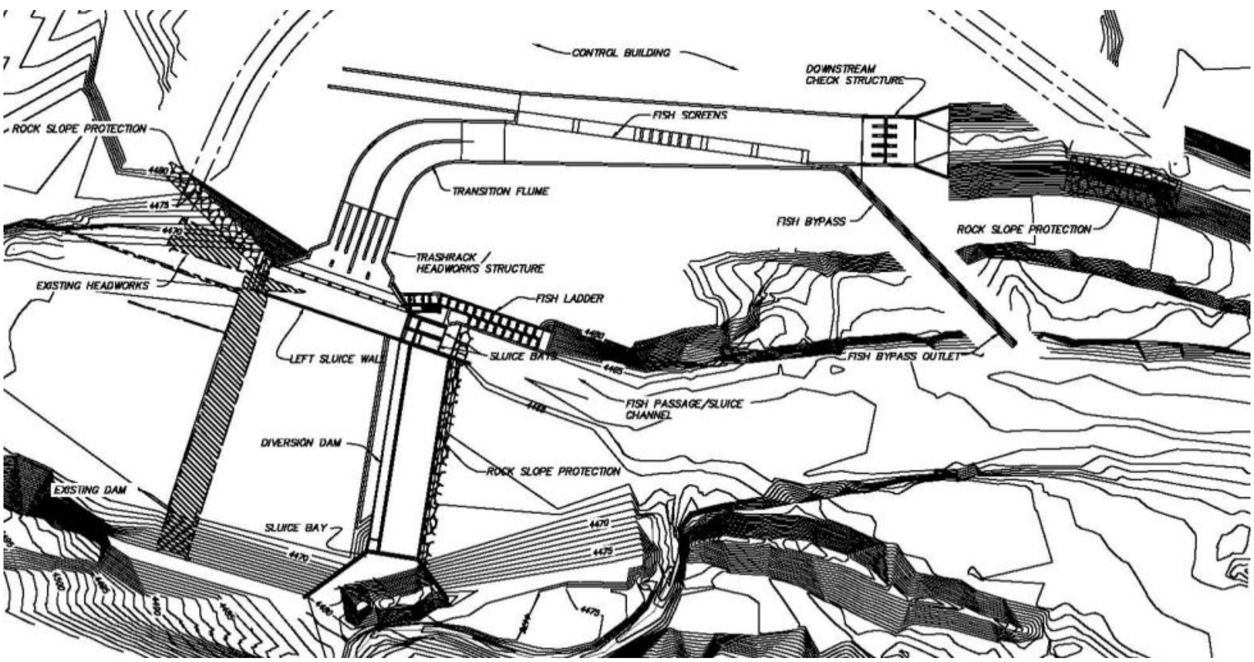

Figure 14. 2015 proposed 60% design of the St. Mary Diversion Dam and Headworks.

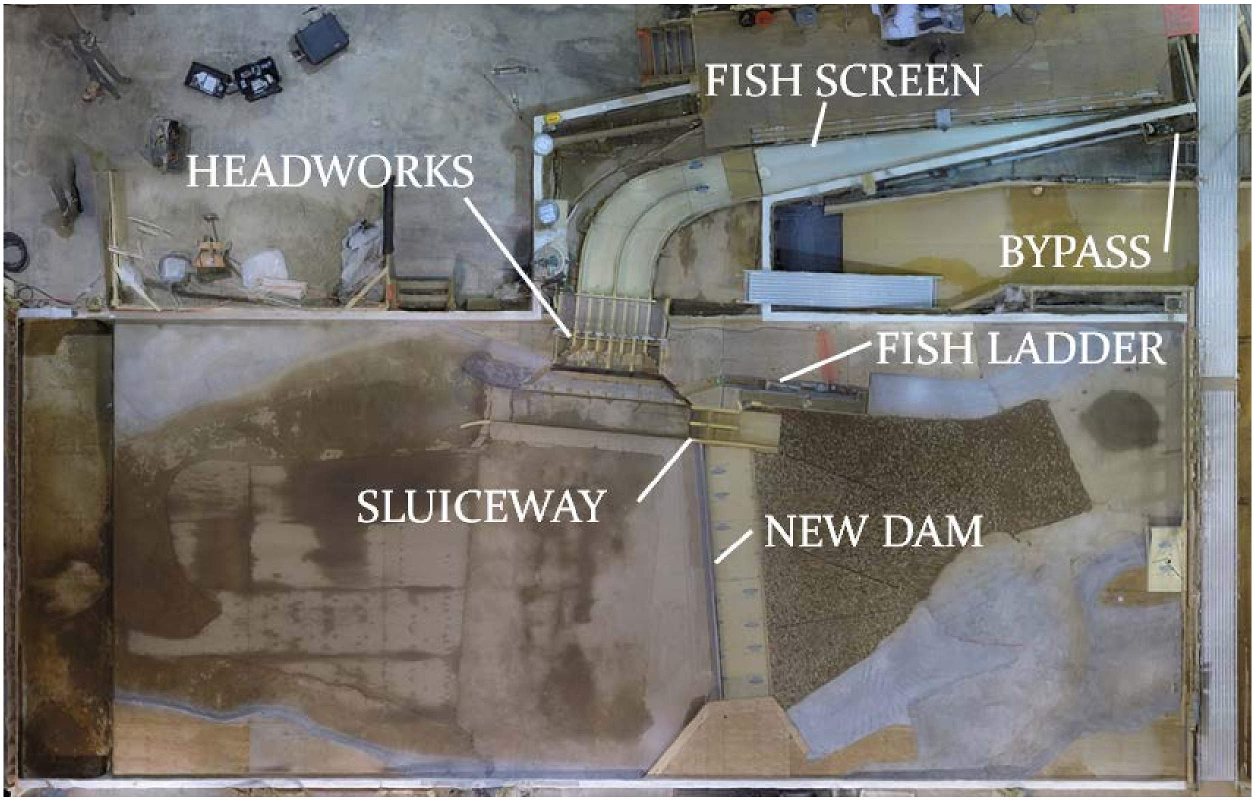

Figure 15. Aerial photograph of physical model for the modified 2015 60% design of the St. Mary Diversion Dam with annotations.

2016 Human Life Safety Design

Following the 2015 60% redesign modeling efforts, the MTAO expressed some concerns regarding the public safety of the design that had been selected. At some flow rates, the structure developed a strong hydraulic jump, which causes localized reversal of surface flow and has the potential to trap floating objects for extended periods of time. Although the number of people expected to pass through the site is low, there is some risk involved in designing a hydraulic structure that exhibits potentially hazardous conditions. To reduce the risk, MTAO requested that Reclamation's Hydraulics Lab investigate ways to prevent the dangerous flow conditions from occurring.

To analyze human life safety concerns the Hydraulics Laboratory developed 3D printed 1:12 scale human dummies modeled after an average human 5-ft 8-in tall and 160-lb. Float test dummies were manufactured with material that provided a realistic density for a human body and were equipped with appropriately buoyant life jackets (small, medium and large). The dummies were introduced into the model upstream from the diversion dam (Figure 16). Video documentation was used to time how long each dummy was retained in the reverse-flow zone created by the diversion dam and surrounding infrastructure. A total of 17 configurations were tested, but due to limited financial resources, a comprehensive report was not developed discussing each option. Two options were recommended by the Hydraulics Lab that significantly reduced the risk of becoming trapped in the hydraulic jump for extended periods. Kent and Heiner (2017) present Option 1A (Figure 17) as a 5-ft high diversion dam with a 15-ft long concrete apron and no end sill, and Option 1B (Figure 18) as a rock ramp with 10 percent grade. Both options were outlined with advantages and disadvantages presented.

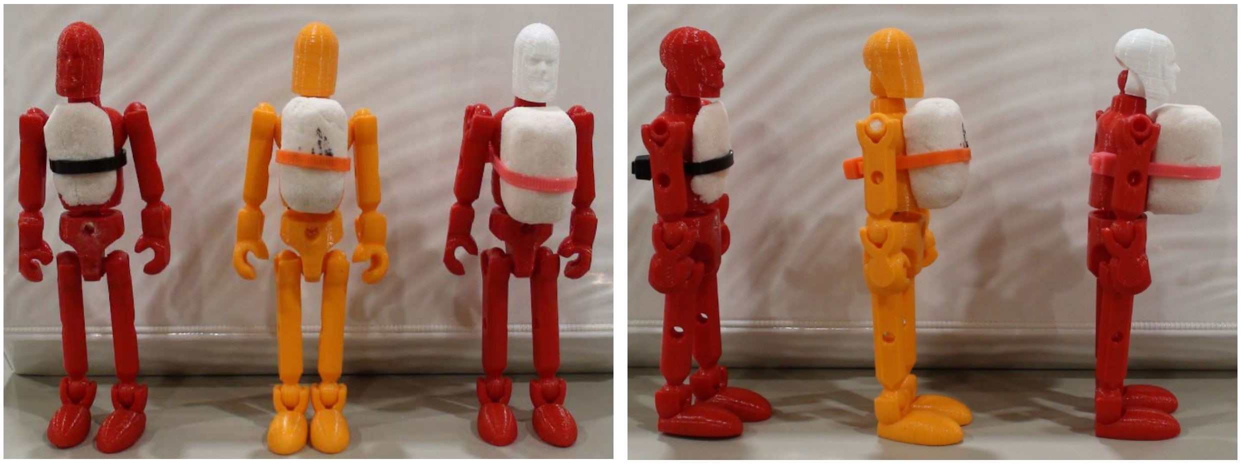

Figure 16. 3D Printed 1:12 scale 5-ft 8-in human dummies with small, medium, and large life jackets attached.

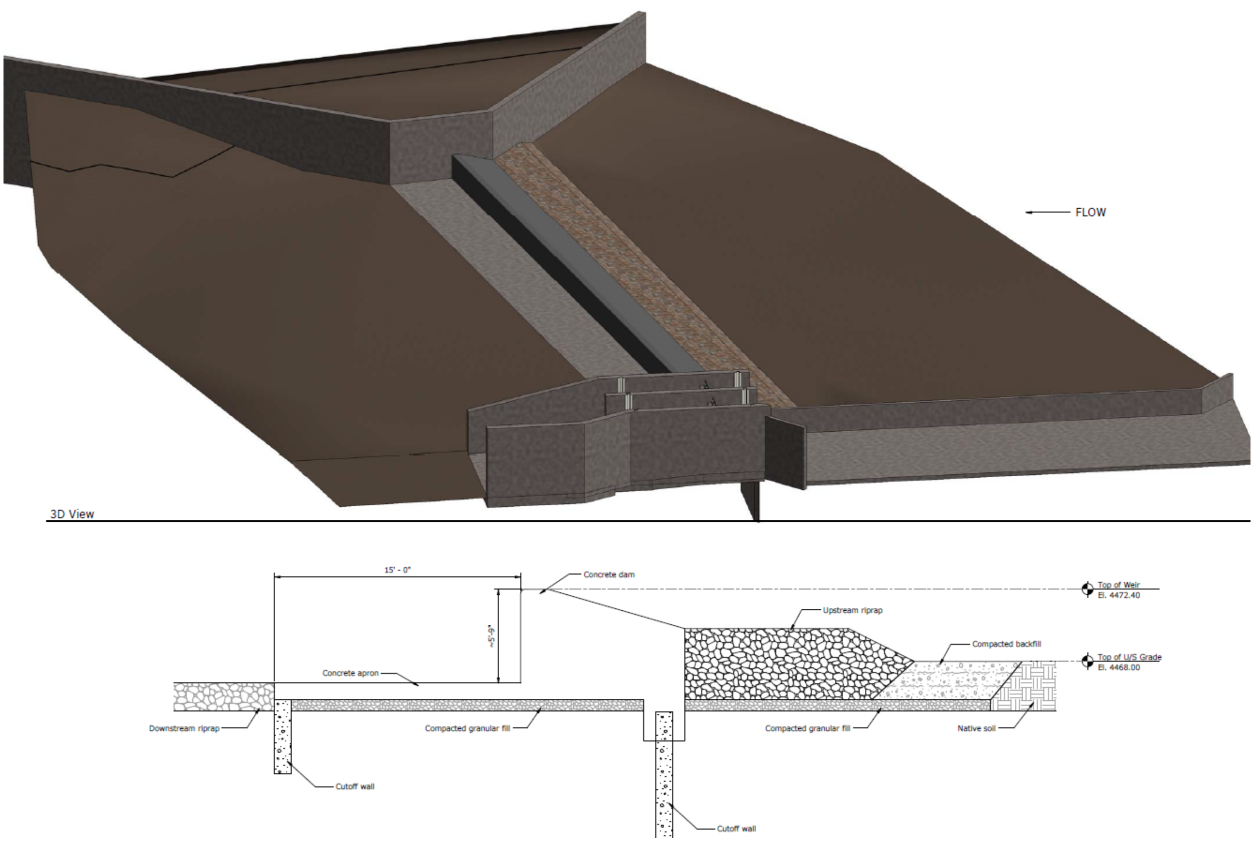

Figure 17. 3D render and cross-sectional view of Option 1A, a broad crested weir with concrete apron sized to mitigate dangerous flow conditions associated with a hydraulic jump (flow is from right to left in both the 3d image and cross section).

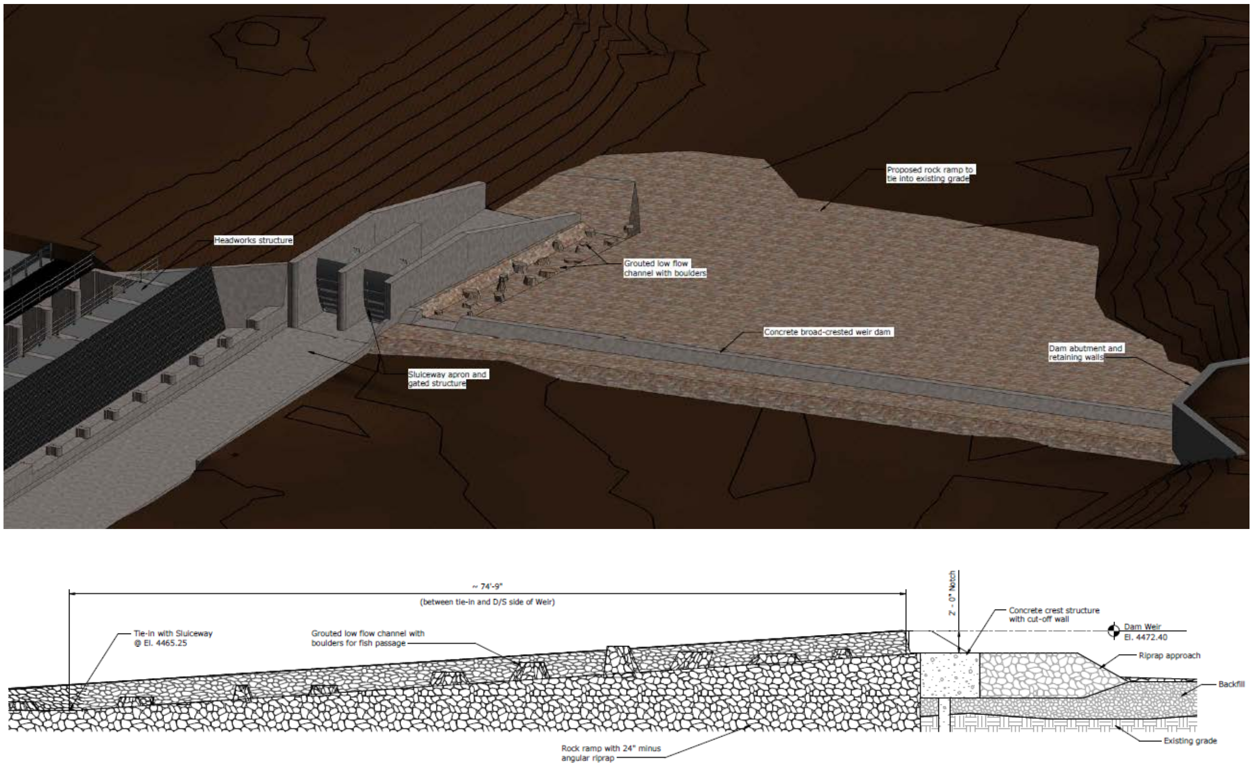

Figure 18. 3D rendering and cross-sectional view of Option 1B, a rock ramp sized to limit the dangerous flow conditions associated with a hydraulic jump (flow is from bottom to top in the 3d image and from right to left in the cross section.

2017 Rock Ramp Design

The Montana Area Office (MTAO 2017) and Biological Review Team (Fish and Wildlife 2017) both recommended that the Rock Ramp option (Option 1B) be physically modeled and designed by the TSC. Drawings for the 2017 Rock Ramp Design are provided in Appendix A. The list below provides a summary of general concepts, assumptions, criteria, objectives, and basic dimensions of the design:

- 850 ft^3/sec maximum design flow-rate for the canal, determined based on a white paper put together by the MTAO (Colloton 2018).
- Rock ramp with 2-ft minus rip-rap on a slope varying from 5-7 percent.
- Rock ramp will be grouted downstream of the crest (tapered from 30-ft of grout on river left to 20-ft on river right).
- Low flow channel that provides 1-ft depth of fish passage at 100 ft^3/sec flow over the rock ramp.
- 2-ft flat area on the crest of the diversion dam to allow fish monitoring equipment to be installed.
- Place structure elevations as high as possible to reduce dewatering requirements. Pump-out tests showed extreme dewatering would be needed if structures extend into the groundwater level (Sullivan and Earle 2017).

- Less than 2 ft/sec of velocity approaching the trashrack structure (trashrack bar size and spacing not finalized).
- 185-ft long concrete diversion dam with 2-ft wide broad crested weir cap and a cutoff wall extending about 10-ft underground.
- Headworks approximately 141-ft wide by 13.5-ft high with a sloped trash rack and six 7.75-ft wide by 6-ft high slide gates separated by pier walls with stop log slots for isolation and maintenance.
- 329.75-ft fish screen with 30 individual screen panels that are 9.5-ft wide by 4-ft high (making total screened area 1140 ft^2) with blanking panels above and below the 4-ft high screens. Access panels, screen cleaning docking stations and access gates are located intermittently along the 329.75-ft. The fish screen sill will remain at a constant elevation of 4167 feet with a varying height along the length of the screen due to the slab sloping to towards the bypass.
- Fish screens will be some type of V-shaped wire, welded wire or profile bar with 0.25-in slots and an approximate open area between 57-78% depending on what type of screen bars are selected (design is targeting 65% open area).
- A rectangular fish bypass channel returning fish to the river downstream. The current design uses a 2-ft width, but this may change depending on bypass flows and downstream water surface elevations.
- Trapezoidal low flow channel (14-ft top width, 8.5-ft bottom width 2-ft depth) in the rock ramp with an approximate slope of 6 percent, a bottom elevation of 4470.4 feet and large boulders (3- to 4-ft diameter) to create a riffle-pool channel and passage routes.
- Off-season sluiceway 25-ft wide by 230-ft long with two 10-ft radial gates with individual bays to control the flow.

Model Description

Model Objectives

The St. Mary Diversion model study focused on the following objectives:

- Evaluate the effectiveness of the dam structure for passing the maximum design flow rate of 10,000 ft^3/sec, while maintaining a stable riverbed downstream of the dam and not exceeding historical maximum upstream reservoir water surface elevations.
- Evaluate the effectiveness of the sluice bays and gates for passing sediment and floating debris downstream.
- Determine the maximum river flow rate that can pass through the sluice bay without inundating the headworks.
- Achieve uniform approach velocity distributions along the fish screens, with approach velocities, perpendicular to the screen face, that are less than or equal to 0.80 ft/sec (NMFS, 1997). Uniformity of approach velocity is defined as being achieved when no individual approach velocity measurement exceeds 110% of the criteria (NMFS 2011).

- Evaluate baffling techniques and/or other structures or modifications needed to achieve acceptable velocity distributions across the fish screen.
- Evaluate the screening bay for eddies or recirculation zones where fish might potentially hold.
- Evaluate rock ramp hydraulics to ensure that fish will be able to pass over the diversion dam.
- Evaluate the low flow channel for fish passage and attraction.
- Evaluate the potential for fish to be entrained into the canal headworks as they exit the low flow channel.

Model Scale

Similitude between the model and the prototype is achieved when the ratios of the major forces controlling the physical processes are equal in the model and prototype. Since gravitational and inertial forces typically dominate open channel flow, Froude-scale similitude was used to establish a kinematic relationship between the model and the prototype. The Froude number is defined as

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

where v = velocity, g = gravitational acceleration, and d = flow depth. When Froude-scale similitude is used for a 1:12 scale, the following relationships exist between the model and prototype where the r subscript refers to the ratio of model to prototype:

Length ratio:	$L_r = L_{model}/L_{prototype} = 1:12$
Pressure ratio:	$P_r = 1:12$
Velocity ratio:	$V_r = L_r^{1/2} = (12)^{1/2} = 1:3.46$
Time ratio:	$T_r = L_r^{1/2} = (12)^{1/2} = 1:3.46$
Discharge ratio:	$Q_r = L_r^{5/2} = (12)^{5/2} = 1:498.83$

Model Features

A 1:12 scale physical hydraulic model was tested at Reclamation's Hydraulics Laboratory in Denver, Colorado to ensure that all of the hydraulic and structural components would function as intended. The physical model included all the features of the 2017 rock ramp design (listed above) except for the complete 2-ft wide bypass channel. The entrance to the 2-ft wide bypass channel was modeled as designed but flows were then passed through a pipe which returned to the laboratory sump instead of the river portion of the model. Figure 19 provides an aerial photograph of the complete physical model with annotations.

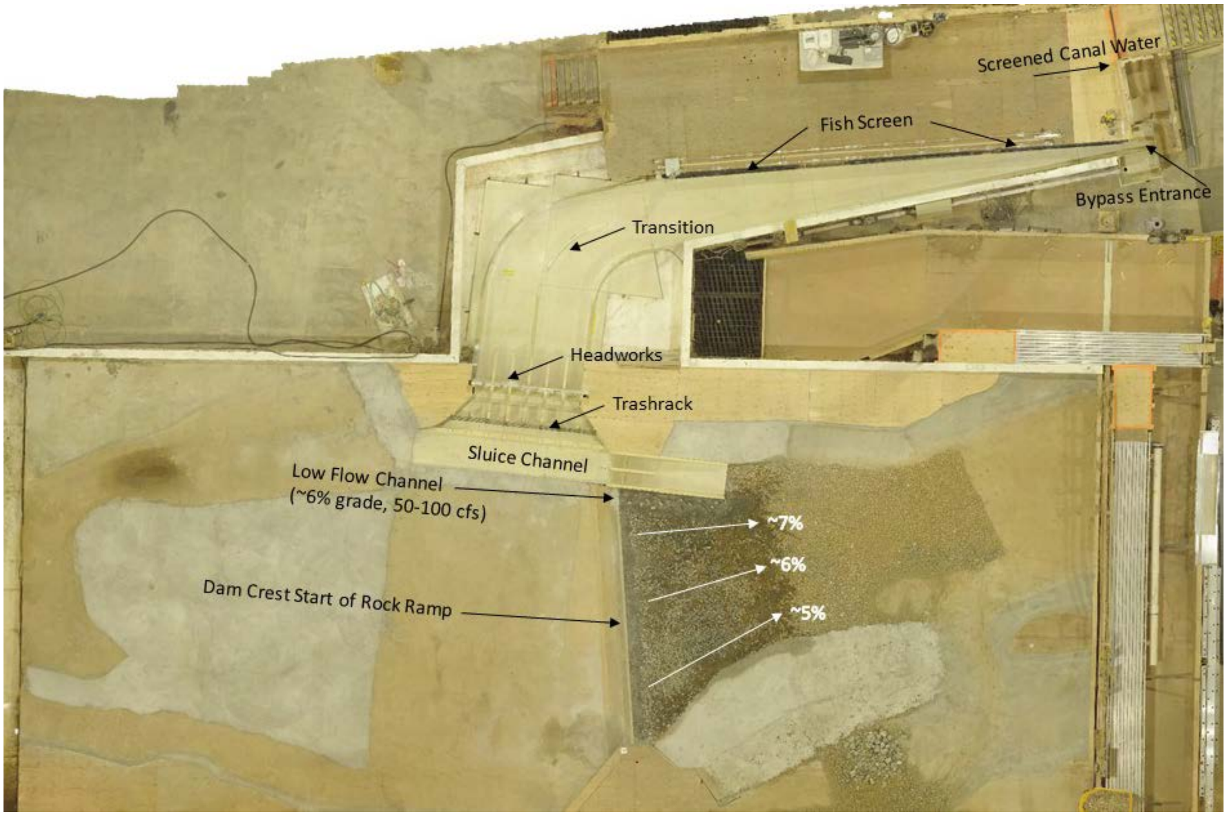

Figure 19. Aerial photograph of the 2017 rock ramp design as constructed in hydraulics laboratory with annotations.

The model was constructed with a combination of rigid polyurethane foam, epoxy coated plywood, acrylic, PVC, aluminum and various perforated plates. Figure 20 is a view from the upstream reservoir looking towards the trashrack with key features annotated. The trashrack was not a true scale model of the trashrack that will be constructed, but used a grating material that approximated the expected open area of the prototype bar rack. The rock ramp and low flow channel were constructed using 1.5-in minus crushed and rounded rock with pea gravel and sand mixed in to provide armoring. The diversion dam was constructed of epoxy coated plywood and PVC and structural dimensions within 1/32-in of prototype design in elevation, location and size. Figure 21 provides a view of the model from the downstream left bank looking at the rock ramp, low flow channel, and diversion dam. The low flow channel was completely grouted using a mix of Quikrete mortar and DirtGlue. Large rock (3- to 4-in diameter) were used to generate the nature-like boulder riffles (Figure 22). The fish screens were modeled using PVC support structures and perforated aluminum plate screen panels with 63 percent open area (5/32-in diameter holes on a 3/16-in staggered pattern). These screen panels closely match the open-area ratio of several of the profile bar, V-shape and welded wire screens that are being considered for the prototype structure. Baffles were constructed using several different custom-punched perforated plates with open area ratios ranging from 7.5% to 63%. Baffle plates were installed 0.75-in downstream of the fish screen (Figure 23) when required, and only one perforated baffle plate was used to control flow through each screen bay. Each 9.5-in wide by 4-in tall (9.5-ft x 4-ft prototype) screened section had a blanking panel above and a sediment exclusion sill below. The total gross wetted screen area was 1140-in^2 (1140-ft^2 prototype).

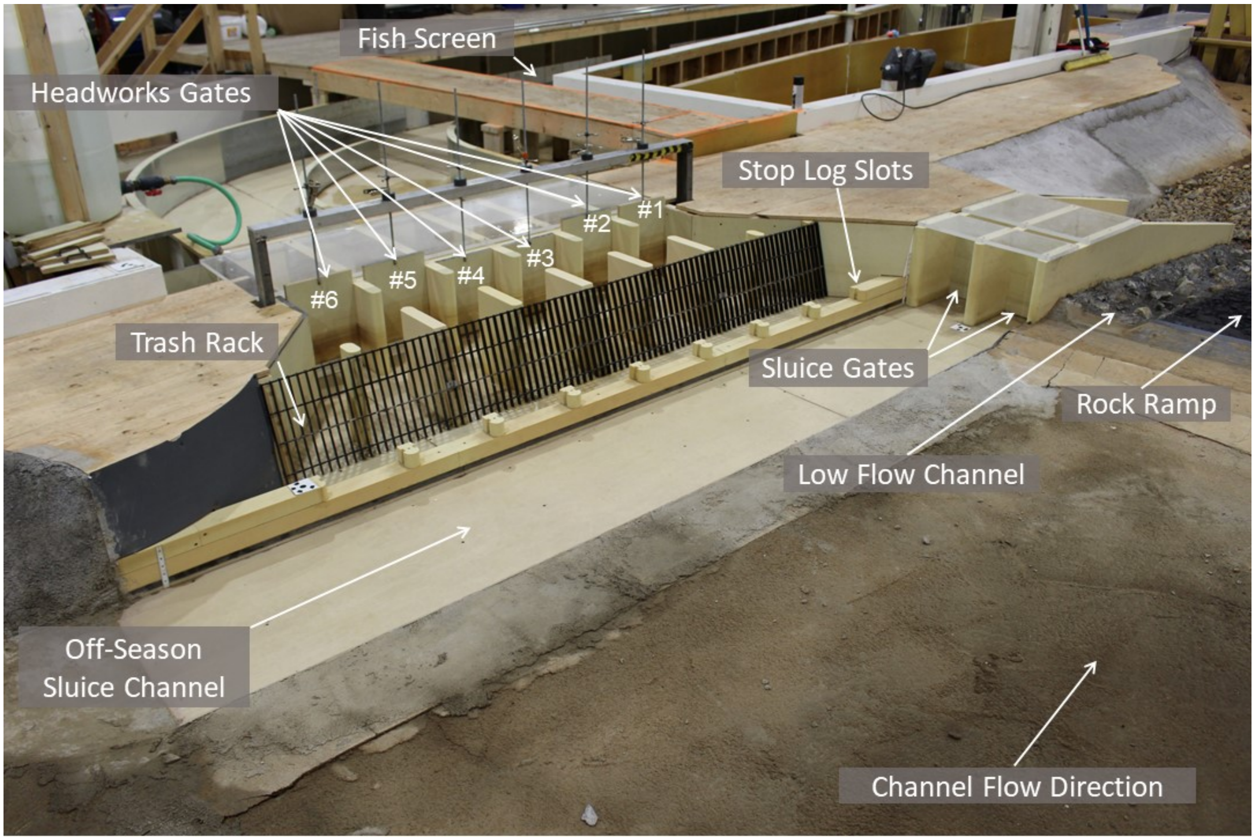

Figure 20. Off-season sluice channel, headworks gates, sluice gates, stoplog slots and trashrack (the view is from the upstream middle of the river channel).

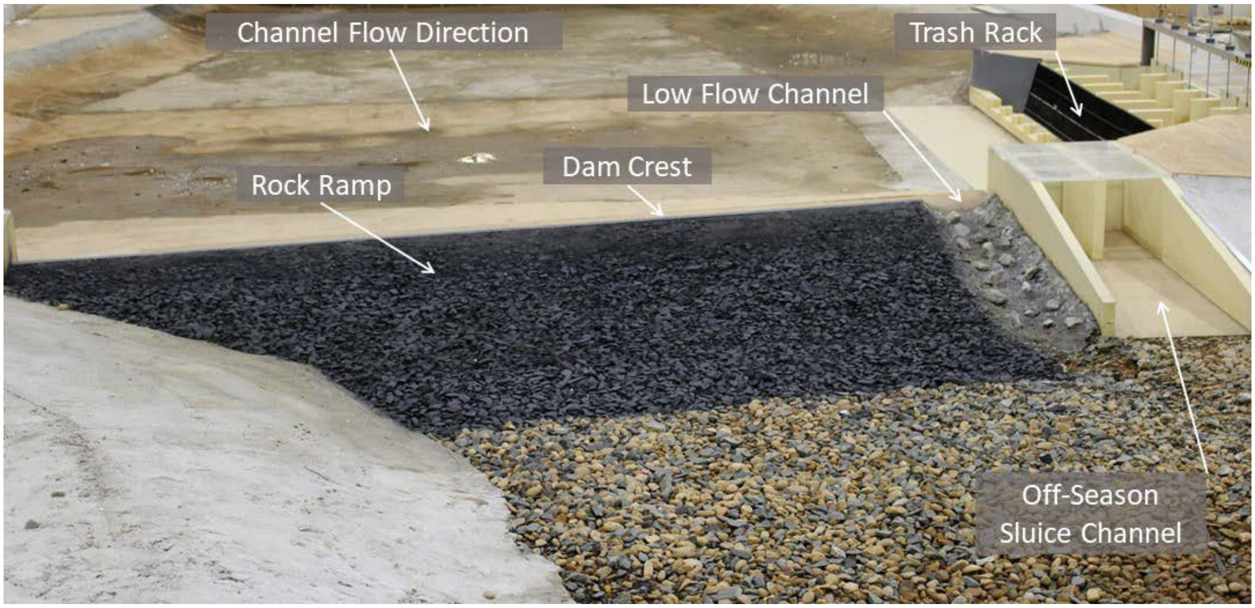

Figure 21. Rock ramp, low flow channel, off-season sluice, dam crest and trashrack (the view is from the downstream middle of river channel).

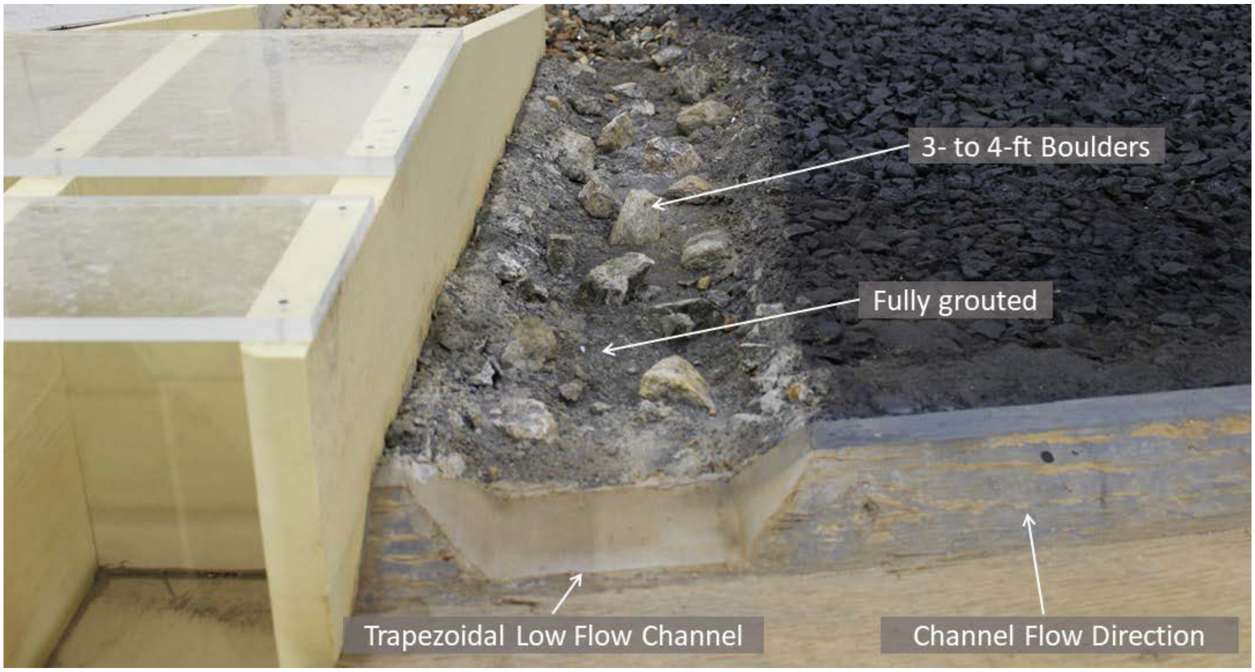

Figure 22. Low flow channel as viewed from the upstream side of the diversion dam.

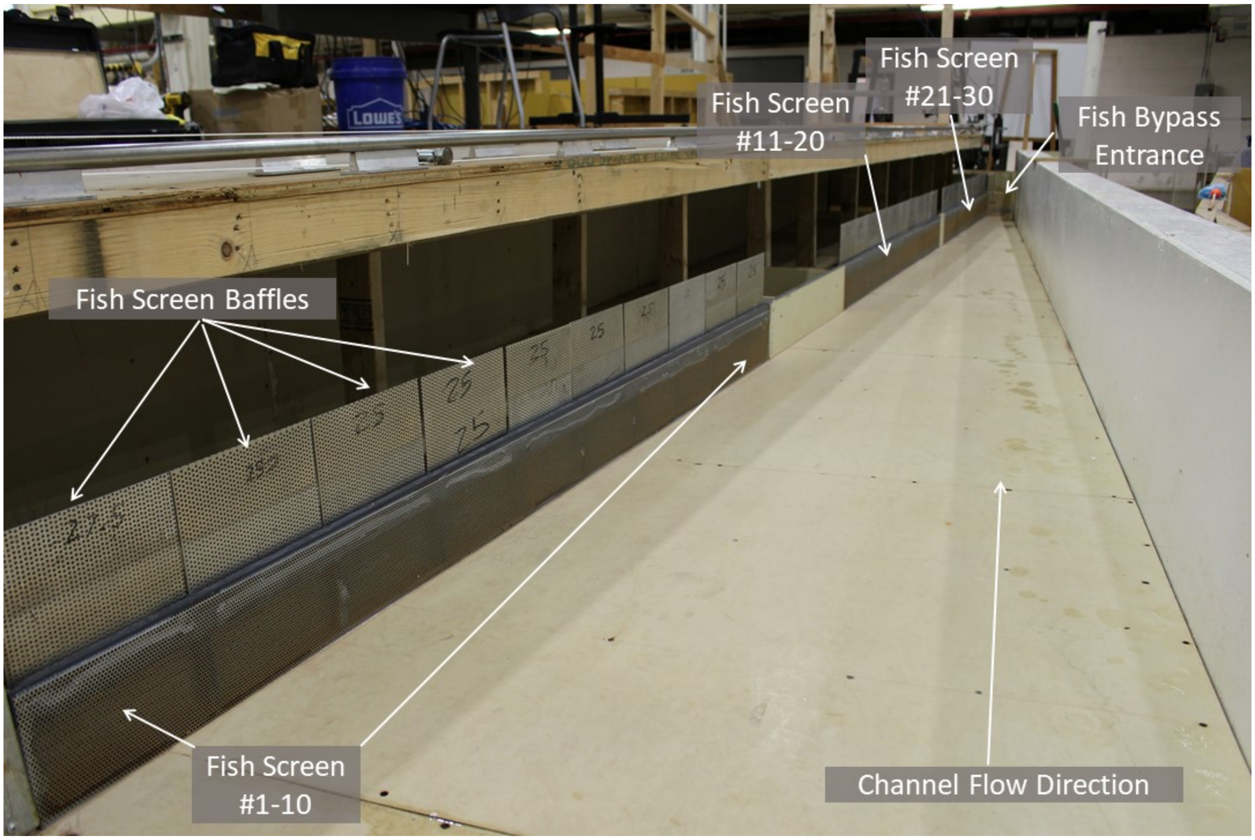

Figure 23. Fish screens, baffle plates, and fish bypass in diversion channel (the view is from upstream looking toward the fish bypass).

Instrumentation

A 240,000-gallon storage reservoir under the laboratory floor supplied water for the hydraulic model through an automated flow delivery and measurement system. Inflow to the model was measured with venturi meters. A 44,000 pound weigh-tank facility was used to calibrate the laboratory venturi meters at regular intervals to an accuracy of $\pm 0.25\%$.

Piezometer taps and stilling wells were equipped with ultrasonic level sensors (Massa Pulstar Plus 150 kHz), with an accuracy of ± 0.1% of the target range (24 inches) and a measurement resolution of 0.01 inches. Point gauges accurate to 0.001-ft were also attached to each stilling well providing redundancy and a method to calibrate the electronic sensors if necessary. Figure 24 shows the 6 locations where water surface data were collected in the model: A) upstream river channel, B) downstream river channel, C) screen #1 upstream side (where fish will be), D) screen #1 downstream of the fish screen (non-fish side), E) screen #30 upstream side, just upstream of the entrance to the bypass channel (where fish will be), and F) screen #30 downstream side, just upstream of the canal check control structure (non-fish side). The target water surface elevation in the diversion channel upstream of the fish screen (Location C) was controlled by adjusting a slide gate located downstream of the screen in the same location as the check gates in the prototype. It was also necessary to ensure that the water surface elevation at location F remained above 4471.0 which is the normal water surface elevation of the canal.

Flow through the fish bypass was measured with a 3-inch full port magnetic flow meters (Siemens MAG6000), accurate to ± 0.25%. The diversion flow rate was measured with a ramp flume accurate to ± 2.83% at the maximum flow rate. The head on the ramp flume was measured with a piezometer tap and stilling well equipped with an ultrasonic level sensor (Massa Pulstar Plus) and point gauge with accuracies as previously mentioned.

Figure 24. Location of head measurements taken in the St. Mary Diversion Dam model.

Three-dimensional velocity data were collected at the fish screen using a Nortek Vectrino+ side looking 3-D velocimeter (Figure 25) with an accuracy of ± 0.5% of measured value. The approach (perpendicular to the screen) and sweeping velocities (parallel to the screen) were measured 1.375 inches in front of the screen face in the model (1.375-ft prototype), which was as close to the screen as the instrumentation would allow. Screen measurements during field verification tests of fish screens are recommended to occur at a prototype distance of 3-in (0.25-in. model) (Mefford 2009), but this was not possible due to model scale and instrument limitations relating to interference with boundaries and acoustic reflections off the screen and baffles. Velocity patterns and distributions in the model will be similar in direction and location to those in the prototype, but will not provide the scaled velocity magnitudes that would be obtained when measuring 0.25-in (3-in prototype) from the screen face. For this reason, all velocity measurements presented in this report are used solely to identify uniformity of approach velocity. All velocity measurements in this report are referred to in prototype units unless noted otherwise.

Figure 25. Nortek Vectrino+ 3D velocimeter mounted in the model at 0.50 depth centered at the end of screen number 30.

Model Results

Rating Curves

The model study was conducted with a range of river flow rates from 125 ft^3/sec up to 10,000 ft^3/sec, the 100-year flood based on an analysis of mean daily flow rates (Cheng 2011), and with diversion canal flow rates of 0, 600 and 850 ft^3/sec. Rating curves were developed for the dam structure with no diversion flow and with diversion canal flow rates of 600 and 850 ft^3/sec. When the diversion canal was operating, the fish bypass was set to the expected normal bypass flow rate of around 40 ft^3/sec. Figure 26 provides a plot of four rating curves for the St. Mary Diversion Dam including no diversion, 600 ft^3/sec and 850 ft^3/sec passing down the diversion. Appendix B provides both tabulated and plotted rating curves for each tested condition.

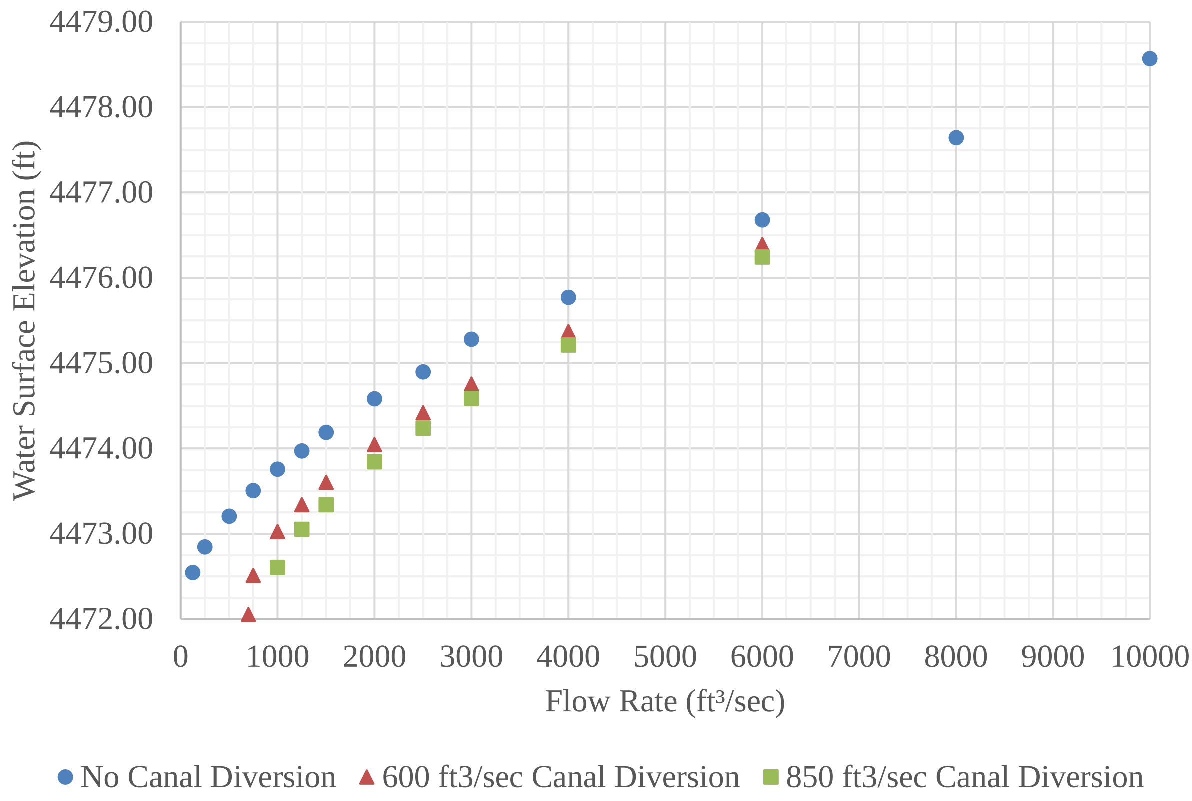

Figure 26. Rating Curves for the St. Mary Diversion Dam with No Diversion, 650 ft³/sec and 850 ft³/sec passing down the canal diversion (additional details in Appendix B).

Fish Screen Hydraulics

Approach and sweeping velocities were measured at 0.5 times the water depth from the water surface at 3 horizontal locations on each screen and directly in front of each pier, making in five total measurements on each screen at locations of 0, 0.25, 0.50, 0.75 and 1 times the width of the screens (0 is the upstream pier and 1 is the downstream pier). The depth of 0.5 times the water depth was chosen after a comparison of the velocity profiles in front of each screen at multiple depths (0.25, 0.5, and 0.75 times the depth) showed that little variation existed between measurements at different depths at the same horizontal screen location (Heiner and Shupe 2014). Continuous traversing velocity measurements were also taken across the entire length of the screens as a redundant velocity check. Both methods yielded similar results. Velocities samples were obtained using a Nortek Vectrino Plus side looking ADV, sampling at 25 Hz. Data were first collected with the un-baffled configuration (baffles 100% open). Baffles were then adjusted to best achieve uniform velocities across the screen (NMFS, 1997 and 2011). A detailed table of all baffle configurations tested is not presented in this report. Relevant fish screen velocity criteria for this study as agreed upon by the Biological Review Team (BRT) are as follows:

1.) The approach velocity must not exceed 0.80 ft/sec for active screens, which is the criteria for fry-sized fish.

2.) The screen design must provide for nearly uniform flow distribution over the screen surface. Uniformity of approach velocity is defined as being achieved when no individual approach velocity measurement exceeds 110% of the criteria.

3.) Screens must have a sweeping velocity greater than the approach velocity.

Head losses across the fish screen and baffles were measured and injected dye was used to examine flow conditions throughout the model. There were both eddying and recirculation along the upstream face of the fish screens because of their location immediately downstream of a sharp short radius bend. These eddies and recirculation could be improved if the distance between the channel bend and start of the screen was increased. Although eddies and recirculation zones existed, no one area presented concerns with stagnant zones. Eddies and recirculation change depending on gate operations. When all gates were open, eddies and recirculation were lowest.

Figure 27 and Figure 28 provide the measured sweeping and approach velocities for 600 ft^3/sec diversion with and without baffles, respectively. Figure 29 and Figure 30 provide the measured sweeping and approach velocities for 850 ft^3/sec diversion with and without baffles, respectively. The x-axis on each plot represents the screen location with screen 1 (closest to the headworks) on the left side and screen 30 (closest to the fish bypass) on the right side, the numerical numbering of screens increases in the direction of flow (left to right). Vertical black lines representing the support piers between each screen are shown but not labeled. Blanking panels are included between gates 20 and 21 (grayed area), where a movable cleaning system will rest when not in operation. The blanking panel between screens 10 and 11 (grayed area) includes a gate structure, which will allow equipment to pass between the upstream and downstream side of the screen.

Baffling downstream of the fish screen at each flow condition was required to uniformly distribute approach velocities across the screen. Numerous screen baffle configurations were tested in the laboratory to best meet approach velocity uniformity requirements (±10% of average measured approach velocity). Baffles set at 25% open area for screens 1-10, 20% open area for screens 11-20 and 17.5% open area for screens 21-30 were found to maintain approach velocities that were within ±10% of the average measured approached velocity for both the 600 ft^3/sec and 850 ft^3/sec diversion rates. Velocity uniformity requirements were not met for a few screens near the blanking panels and gate sections. The locations where screens did not meet criteria were minimal and were always present regardless of the baffle configuration. This only becomes a concern when diverting 850 ft^3/sec, because this is the only condition where velocities might exceed the 0.8 ft/sec design requirement.

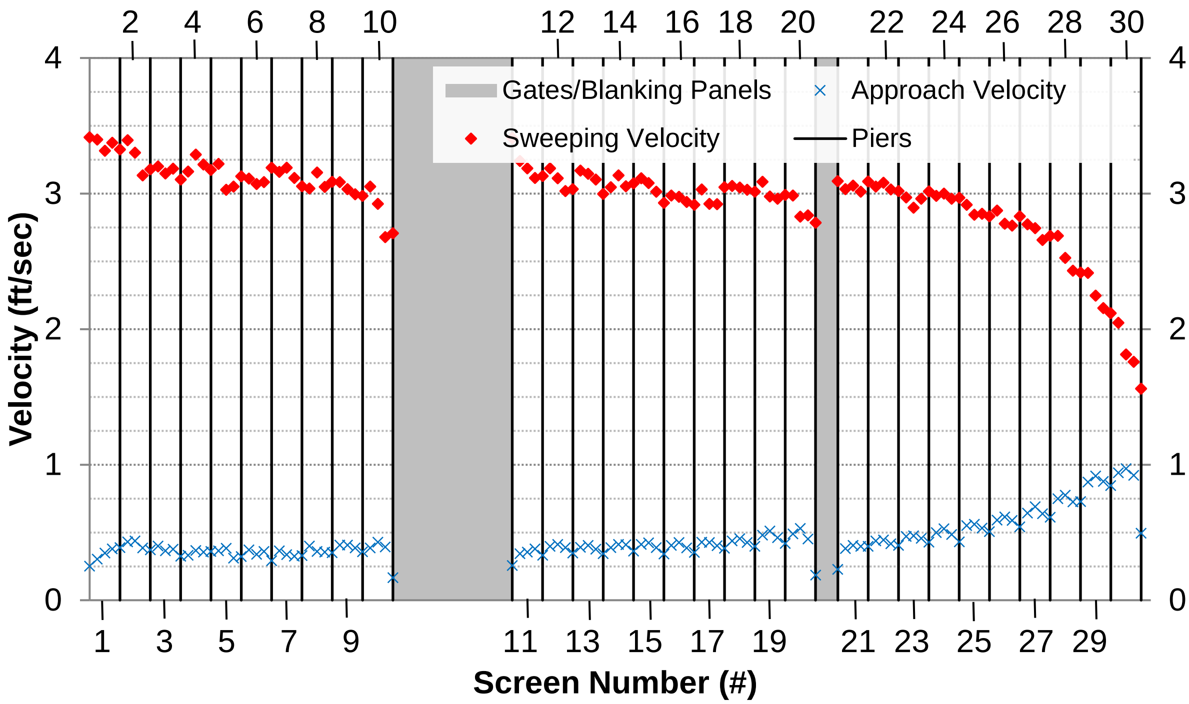

Figure 27. Approach and sweeping velocities at 600 ft³/sec diversion with baffle 100% open.

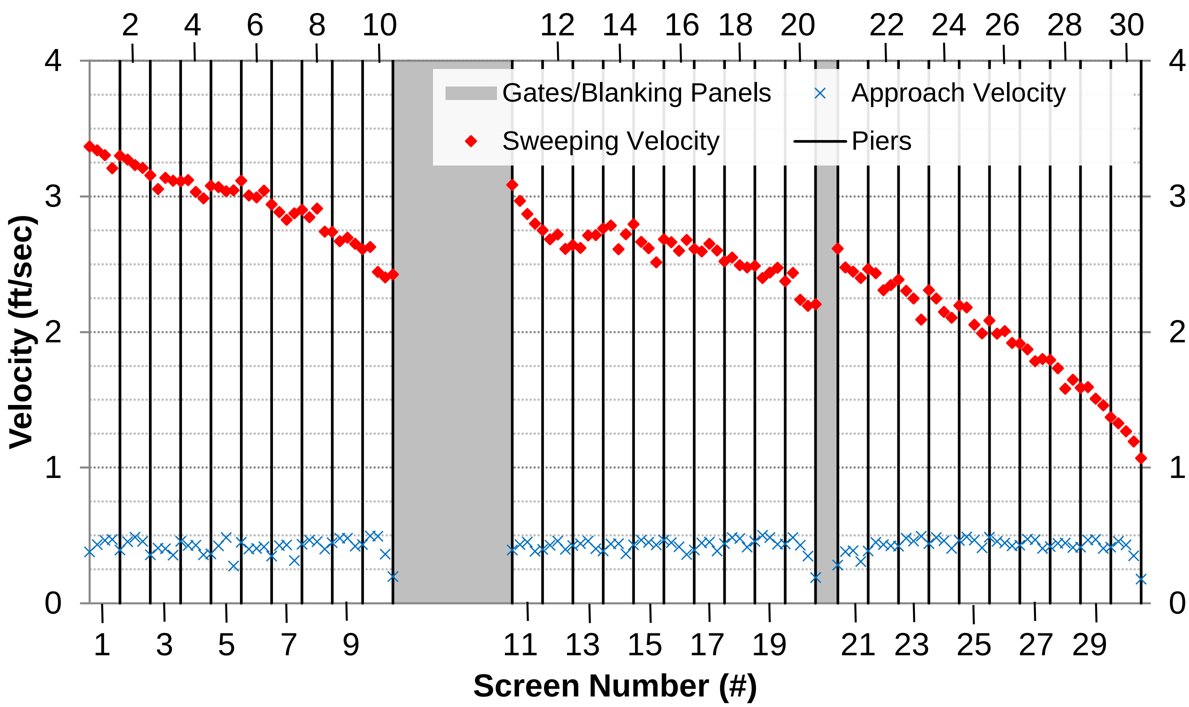

Figure 28. Approach and sweeping velocities at 600 ft³/sec diversion with baffles installed (25% screens 1-10, 20% screens 11-20, and 17.5% screens 21-30).

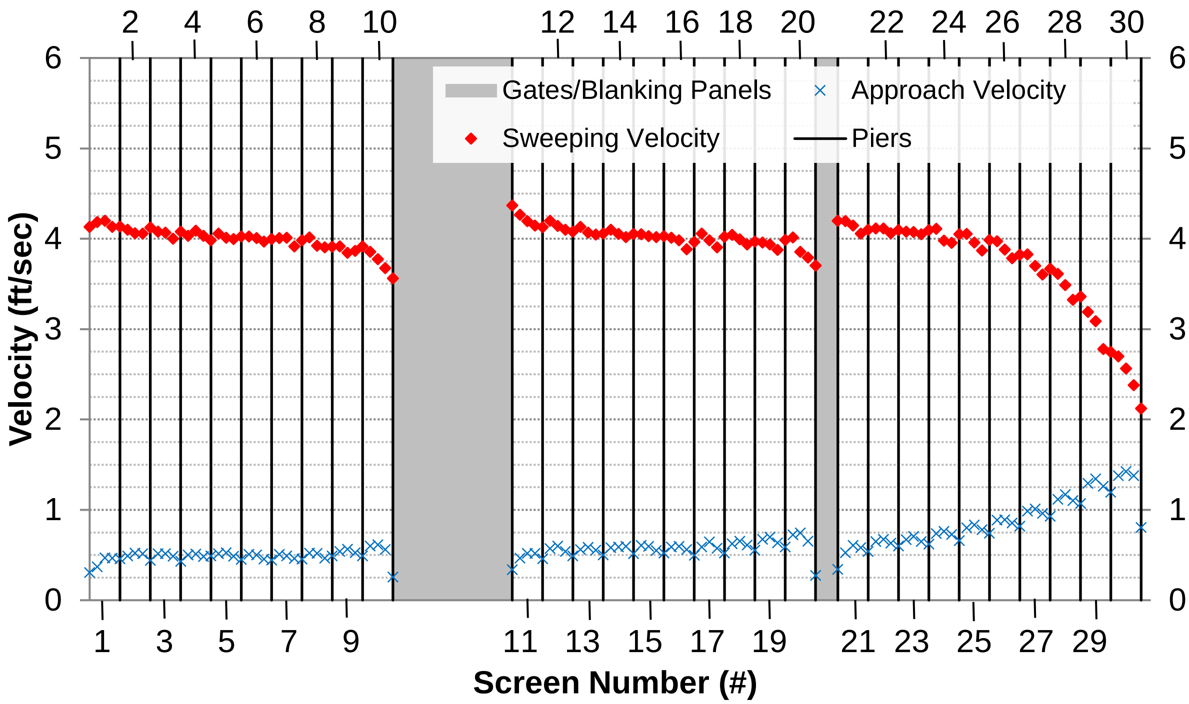

Figure 29. Approach and sweeping velocities at 850 ft³/sec diversion with baffle 100% open.

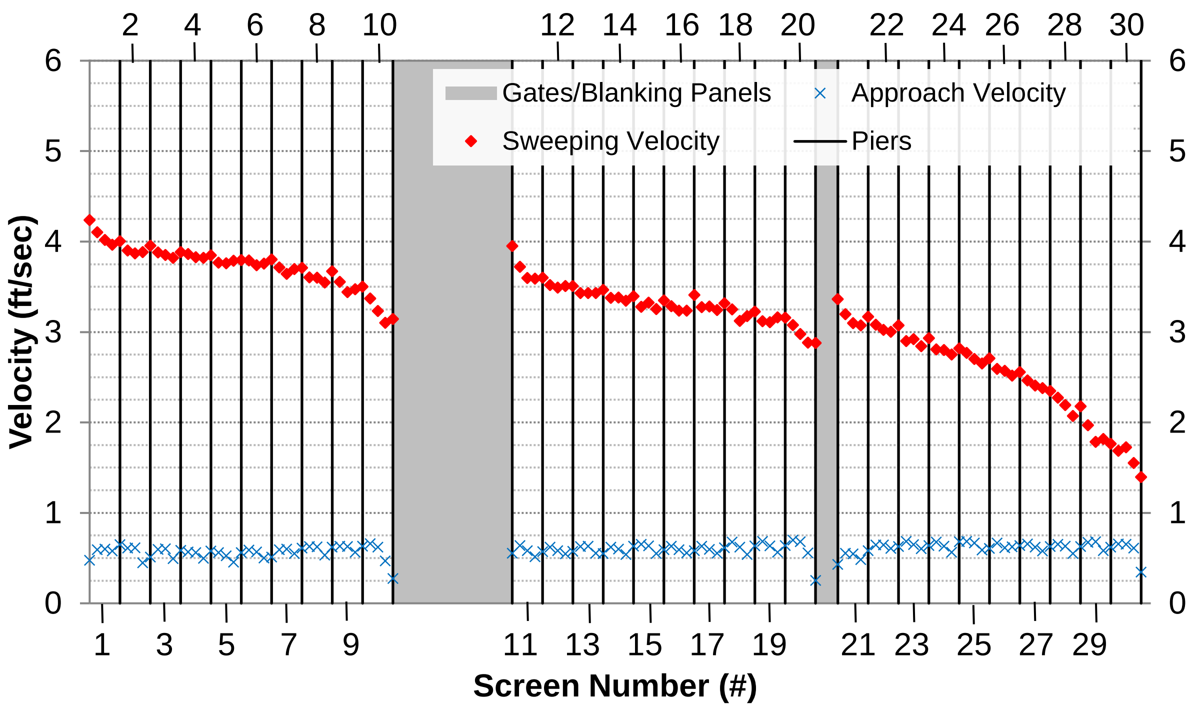

Figure 30. Approach and sweeping velocities at 850 ft³/sec diversion with baffles installed (25% screens 1-10, 20% screens 11-20, and 17.5% screens 21-30).

Considering an open screen area of 1140 ft^2 at 600 ft^3/sec and 850 ft^3/sec, approach velocities should be around 0.53 ft/sec and 0.75 ft/sec respectively. It is difficult to make measurements of fish screen approach velocities in the model that perfectly represent actual screen approach velocities in the prototype. To begin, it is impractical to make measurements exactly at the screen face in either situation. In prototype installations, measurements are typically made at 3-in from the screen face (Mefford 2009), but model measurements could be made no closer than 1.375-in from the screen (16.5-in prototype). As a result, the measurement plane was separated from the screen face. Considering a rectangular prismatic control volume between the screen face and the measurement plane, the flow through the measurement plane is the sum of the flow through the screen face and the net flows through the upstream, downstream, top and bottom ends of the prism. Figure 27 through Figure 30 show that sweeping velocities were high at the upstream ends of the screens and low at the downstream ends, so there was a net positive flow into the control volume through the end sections. Thus, the flow through the measurement plane should be expected to be less than the flow through the screen face, and the difference should increase as the distance from the screen face is increased. No inference can be made relative to the velocities on the top and bottom plane. In addition, approach velocities are difficult to accurately measure in both models and prototypes when the sweeping velocity is large relative to the approach velocity. Even small misalignments of the velocity probe with the axis of the screen structure can cause the sweeping velocity to significantly affect the measured approach velocity. Care was taken to align the probe to the screens as accurately as possible, but measurement uncertainty due to this effect cannot be eliminated.

To overcome the difficulties in measuring the magnitude of actual approach velocities, approach velocity magnitude at the fish screen was calculated based on a mass balance between the screen open area and the flow rate down the diversion. Table 1 provides the measured and mass balance calculated approach velocities for each of the diversion flow rates investigated. The measured average approach velocity is between 80 and 85% of the mass balance calculated approach velocity, which is similar to results that have been seen in other field and lab screen evaluations (Svoboda, et al. 2017). Table 2 provides the maximum and minimum measured approach velocities. A separate column is included that adjusts the maximum measured velocity by the ratio of the average mass balance approach and the measured average approach velocity. Adjusting the maximum approach velocity in this manner provides a means of checking if the measured approach velocity from the model remains less than or equal to 110% of the average allowed approach velocity. Considering the allowable design approach velocity is 0.8 ft/s, no velocities can exceed 110% of that, so if the adjusted maximum remains less than or equal to 0.88 ft/s the screen should remain in criteria. At 850 ft^3/sec when baffles are installed the adjusted maximum measured approach velocity is 0.89 ft/s, which is slightly above the 0.88 allowed to meet criteria. If 850 ft^3/sec is ever sent down the canal a post construction evaluation should be conducted, and baffles adjusted slightly if necessary to ensure no velocities exceed the 110% criteria. Total head loss across the screen is also provided in Table 1. As baffling is installed to improve approach velocities head loss is increased. Head loss is calculated based on the measured water surface difference from the fish side of screen #1 to the non-fish side of screen #30. From the model results it is recommended that the baffles be set as follows:

- Screens #1-10 – 25% open area baffles
- Screens #11-20 – 20% open area baffles
- Screens #21-30 – 17.5% open area baffles

Post construction baffling in this manner should provide uniform approach velocities for diversion flow rates from 600 to 850 ft^3/sec.

Table 1. Measured and mass balance average approach velocities and screen head loss for 600 and 850 ft^3/sec diversion flow rates.

Diversion Flow Rate (ft^3/s)	Baffle % Open (#1-10, #11-20, #21-30)	Measured Average Approach Velocity (ft/s ±Standard Deviation)	Mass Balance Average Approach Velocity (ft/s)	Total Head Loss Across Screen (ft)
600	100-100-100	0.45 ±0.15	0.53	0.21
600	25-20-17.5	0.42 ±0.06	0.53	0.39
850	100-100-100	0.64 ±0.24	0.75	0.30
850	25-20-17.5	0.59 ±0.07	0.75	0.58

Table 2. Measured maximum and minimum approach velocities with the maximum measured approach velocity corrected by the mass balance difference.

Diversion Flow Rate (ft^3/s)	Baffle % Open (#1-10, #11-20, #21-30)	Measured Maximum Approach Velocity (ft/s)	Approach Velocity Mass Balance Adjustment ($AVG_{mass\ balance}/AVG_{measured}$)	Maximum Approach Velocity Adjusted to Mass Balance (ft/s)
600	100-100-100	0.97	1.18	1.14
600	25-20-17.5	0.50	1.26	0.63
850	100-100-100	1.43	1.17	1.67
850	25-20-17.5	0.70	1.27	0.89

Sluice Channel Operation

During non-irrigation season all flow is passed through the off-season sluice channel on river left. A discharge of up to 400 ft^3/sec can pass through the sluice channel without overtopping the entrance sill of the trash rack. When 2-ft tall stoplogs are installed across the front of the intake (see Figure 20), the flow that can pass through the sluice channel is increased to 750 ft^3/sec without any flow entering the trashrack and headworks gate structure. When river flows are at or above 650 ft^3/sec and stoplogs are installed, flow passes down the low-flow channel of the rock ramp, allowing fish passage at higher velocities. Velocities on the left side of the channel (nearest the trashrack) are higher than those on the right due to the curvature of the river upstream of the sluice. The higher velocities on the left side should mobilize sediment and flush it downstream while providing a lower velocity fish passage route on the right side of the sluice as shown in Figure 31. Depths and average velocities for the off-season sluice channel are provided in Table 2.

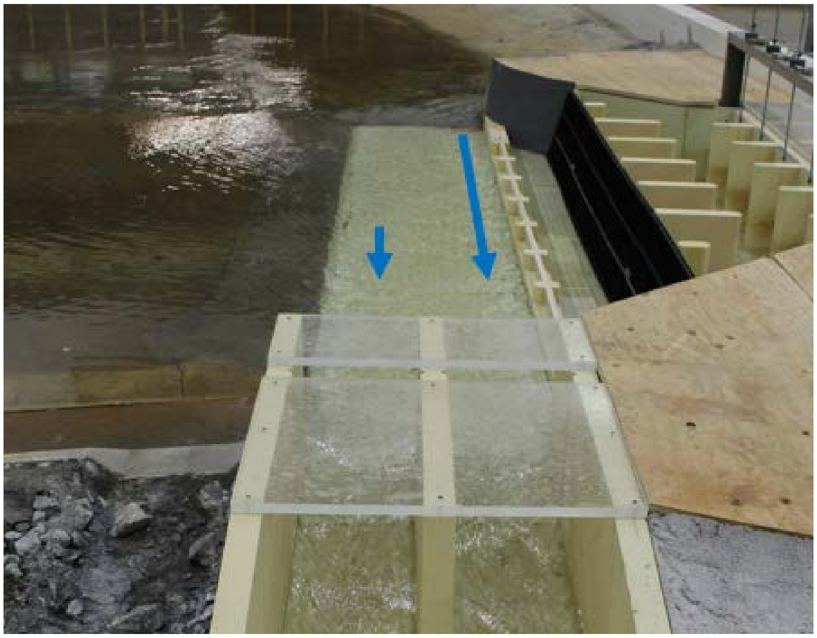

Figure 31. Low flow sluice on river left directly in front of trash rack (flow is from top to bottom).

Table 3. Off-Season sluice channel flow rates, depth and average velocities (flows above 400 ft^3/sec require the use of 2-ft tall stoplogs).

Flow Rate (ft^3/sec)	Upstream Depth (ft)	Downstream Depth (ft)	Average Velocities at Upstream End (ft/sec)	Average Velocities at Downstream End (ft/sec)
100	1.25	1.50	3.2	2.7
150	1.38	1.63	4.4	3.7
200	1.75	1.75	4.6	4.6
249	2.50	1.81	4.0	5.5
299	2.88	1.88	4.2	6.4
349	3.13	1.88	4.5	7.4
399	3.50	2.00	4.6	8.0
449*	3.75	2.06	4.8	8.7
499*	4.13	2.06	4.8	9.7
549*	4.63	2.13	4.7	10.3
599*	4.75	2.19	5.0	10.9
629*	5.00	2.19	5.0	11.5
663*$^+$	5.19	2.25	5.1	11.8
698*$^+$	5.31	2.38	5.3	11.8
748*$^+$	5.50	2.50	5.4	12.0

*2-ft tall stoplogs are required to prevent water from entering the trashrack and headworks gates.

$^+$flow passes down the low-flow channel of the rock ramp in addition to through the off-season sluice gates.

Trashrack Operation

Design criteria require that velocities upstream of the trashrack should not exceed 2.0 ft/sec. Velocity measurements taken upstream of the trashrack with a handheld Sontek ADV confirmed that no velocities exceeded the criterial. Velocity measurements are not provided in this report because spot checks were evaluated at many operating scenarios over the duration of testing and were not recorded because the 2.0 ft/sec threshold was never exceeded.

Headworks Gate Operation

Gate operations for both 600 and 850 ft^3/sec diversion rates were investigated. To gain the best velocity uniformity over the fish screen it is recommended that symmetrical gate operation be used with at least 4 gates open for the 600 ft^3/sec diversion and all 6 gates open for the 850 ft^3/sec. Table 3 and Figure 32 provide the gate openings and headworks loss information for a canal diversion of 600 ft^3/sec (600 ft^3/sec + 40 ft^3/sec bypass total flow through headworks). Table 4 and Figure 33 provide the gate openings and headworks loss information for a canal diversion of 850 ft^3/sec (850 ft^3/sec + 40 ft^3/sec bypass total flow through headworks). As the flows in the St. Mary River increase, gate openings decrease which increases the total head loss across the headworks for both canal diversion rates. Total head loss across the headworks was measured in the model by taking the difference between the upstream river water surface elevation (Location A in Figure 24) and the water surface elevation in the fish screen section on the fish side of screen number 1 (Location C in Figure 24).

During design team model visits, concerns were expressed about the recommendation to operate all 6 gates. Tests were performed to determine the minimum number of gates needed to divert the necessary flow. Gates were adjusted (closing one and adjusting the others) so that the water surface elevation entering the canal was at least 4471.00 ft. A 600 ft^3/sec diversion flow down the canal can be obtained with a minimum of 2 gates in operation, and an 850 ft^3/sec canal diversion can be maintained with a minimum of 3 gates in operation. These operational scenarios are not recommended as they can cause violation of fish screen approach velocity criteria, but they can be used to obtain the necessary canal diversion if required.

Table 4. Gate openings and headworks loss for a diversion flow of 600 ft³/sec and upstream river flow from 700-6000 ft³/sec.

Upstream River Flow (ft^3/sec)	Gates Open (#)	Gate Opening (ft)	Headworks Loss (ft)
700	1,2,3,4,5,6	6.00	0.32
750	1,2,5,6	3.44	0.76
1000	1,2,5,6	2.69	1.28
1250	1,2,5,6	2.44	1.59
1500	1,2,5,6	2.25	1.85
2000	1,2,5,6	2.06	2.30
2500	1,2,5,6	1.94	2.67
3000	1,2,5,6	1.81	3.01
4000	1,2,5,6	1.66	3.62
6000	1,2,5,6	1.47	4.58

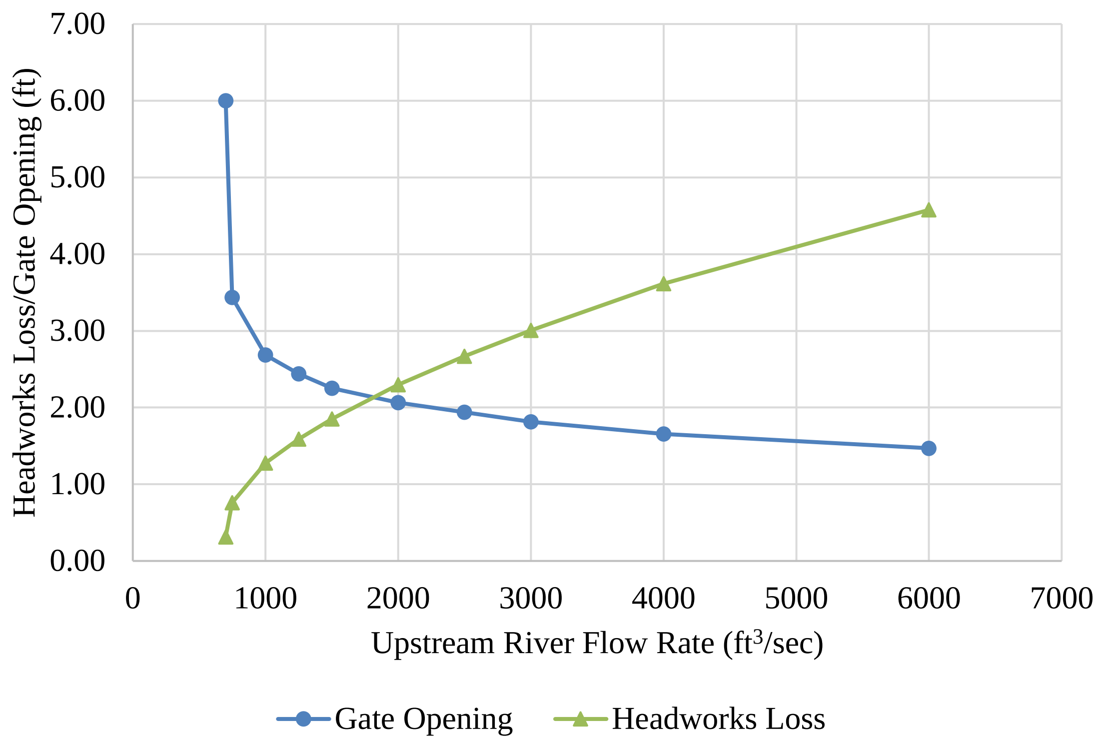

Figure 32. Gate Opening and Headworks Loss for 600 ft³/sec diversion.

Table 5. Gate openings and headworks loss for a diversion flow of 850 ft^3/sec and upstream river flow from 1000-6000 ft^3/sec.

Upstream River Flow (ft^3/sec)	Gates Open (#)	Gate Opening (ft)	Headworks Loss (ft)
1000	1,2,3,4,5,6	3.44	0.83
1250	1,2,3,4,5,6	2.63	1.30
1500	1,2,3,4,5,6	2.38	1.56
2000	1,2,3,4,5,6	2.03	2.07
2500	1,2,3,4,5,6	1.88	2.47
3000	1,2,3,4,5,6	1.75	2.84
4000	1,2,3,4,5,6	1.59	3.46
6000	1,2,3,4,5,6	1.41	4.49

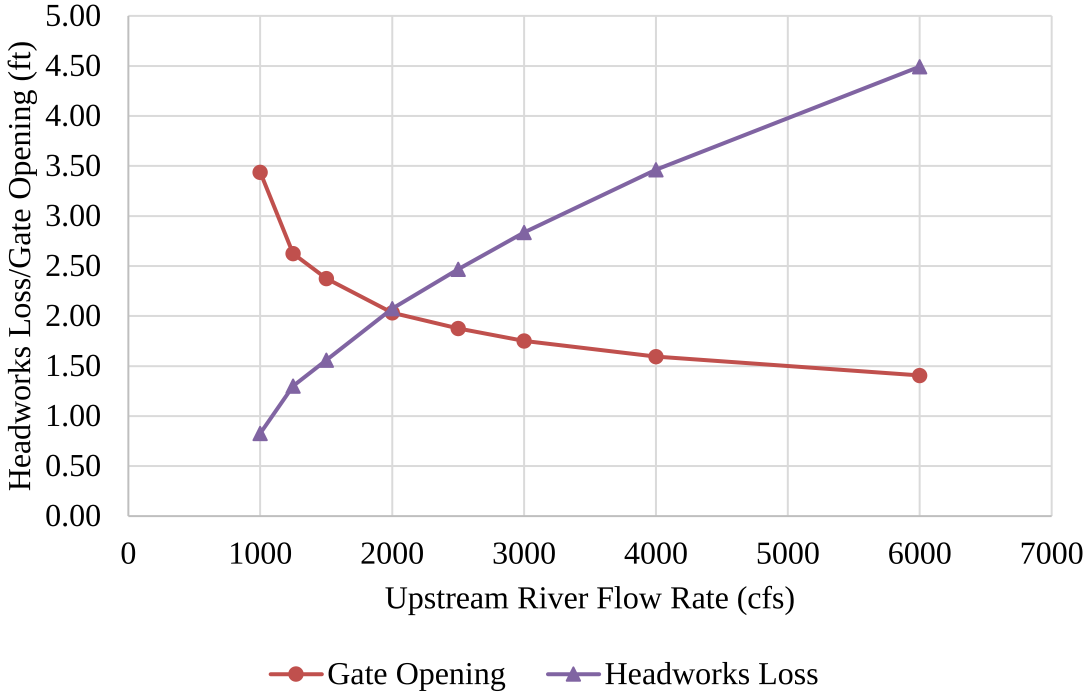

Figure 33. Gate opening and headworks loss for 850 ft^3/sec diversion.

Rock Ramp and Low-Flow Channel Hydraulics

Depth and velocities in both the low-flow channel and rock ramp were measured to ensure that passage requirements were satisfied. Table 5 provides a summary of the average depth and calculated cross-sectional velocity in the low-flow channel for discharges of 25, 50, 75, 100 and 125 ft^3/sec down the low-flow channel. Depths were measured using a graduated staff with 1/32-in resolution. The cross-section average velocity was calculated from the area of the low-flow channel at the corresponding depth in the channel. Figure 34 provides contours of the surface velocities down the low-flow channel of the rock ramp for each tested flow rate. Surface velocities were measured using a technique called Large-Scale Particle Image Velocimetry (LSPIV). Short video clips of particles moving down the rock ramp were recorded and post-processed. Particles were tracked from one frame to another and the relative distance moved over the frame rate is converted into surface velocity. Individual frame by frame results are averaged over a specified duration of time (10 seconds of video) to provide an average of the surface velocities. Surface velocities are typically larger than the average cross-sectional velocity. Due to minimal depth (less than 2.5-in in the model) and high turbulence down the rock ramp, no other methods were successful at measuring the velocities in the low-flow channel.

Table 6. Depth and velocity in the low-flow channel for varying flow rates.

Flow Rate (ft^3/sec)	Minimum Depth (ft)	Maximum Depth (ft)	Average Depth (ft)	Cross-Section Average Velocity (ft/sec)	Maximum Velocity from LSPIV (ft/sec)
25	0.5	1	0.75	2.27	2.58
50	0.75	1.25	1	3.64	3.80
75	1	1.75	1.375	3.90	4.27
100	1.25	2.25	1.75	4.04	4.07
125	1.5	2.5	2	4.55	4.43

Table 7 provides rock ramp depth and velocity measurements for flow rates ranging from 500-6000 ft^3/sec passing over the rock ramp into the downstream river. Cross-sectional average velocity is calculated for each flow rate based on the average depth and a rock ramp width of 180 ft. Depths and velocities over the rock ramp vary greatly, allowing areas for fish to pass over the entire ramp. LSPIV measurements were also used to measure the surface velocities across the rock ramp. Figure 35 through Figure 42 provide velocity contour plots of the surface velocities for flow rates ranging from 360-5360 ft^3/sec over the rock ramp (canal operations were set at 600 ft^3/sec with 40 ft^3/sec bypass for each test scenario), with flow from bottom of the figure to the top. Table 8 provides the maximum velocity from each of the contour plots. These plots are provided to show the variation of velocities over the rock ramp. A few locations where locally high velocities exist need to be addressed in the final design to prevent scour downstream of the rock ramp. The main area of concern is the river right bank at the end of the rock ramp where high velocities are noticed as the flow turns around the transition of the rock ramp into the existing bank. This modification can be easily accomplished by re-grading the right back to improve its alignment with the downstream channel over a longer distance.

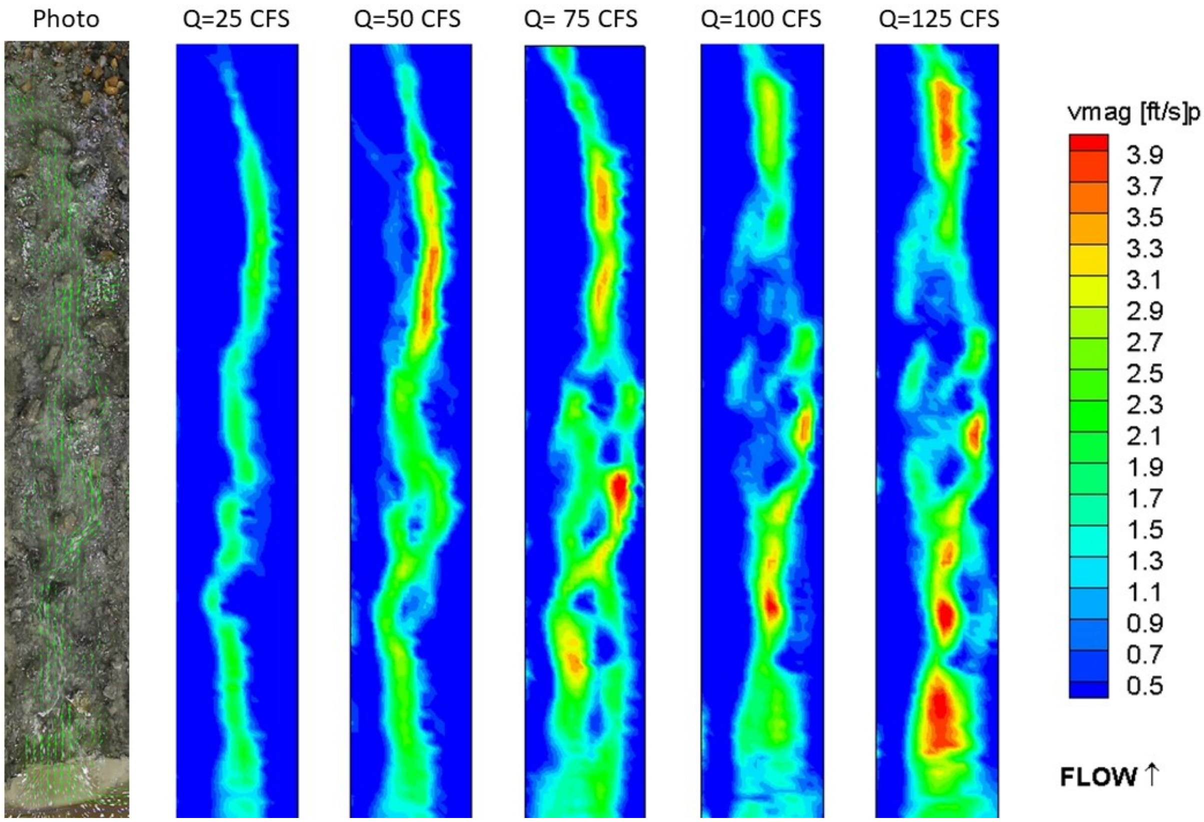

Figure 34. Surface velocities for the low-flow channel at discharges ranging from 25-125 ft^3/sec (CFS) (flow from bottom to top).

Table 7. Rock ramp depth and velocity measurements for flow rates from 500-6000 ft^3/sec passing over the rock ramp into the downstream river.

Flow Rate (ft^3/sec)	Minimum Depth (ft)	Maximum Depth (ft)	Average Depth (ft)	Cross-Section Average Velocity (ft/sec)
500	0.25	1.75	1.00	2.78
750	0.50	2.00	1.25	3.33
1000	0.63	2.38	1.50	3.70
1250	0.75	2.75	1.75	3.97
1500	0.88	3.13	2.00	4.17
2000	1.25	3.75	2.50	4.44
2500	1.50	4.50	3.00	4.63
3000	1.75	5.25	3.50	4.76
4000	2.25	6.25	4.25	5.23
6000	3.50	7.00	5.25	6.35

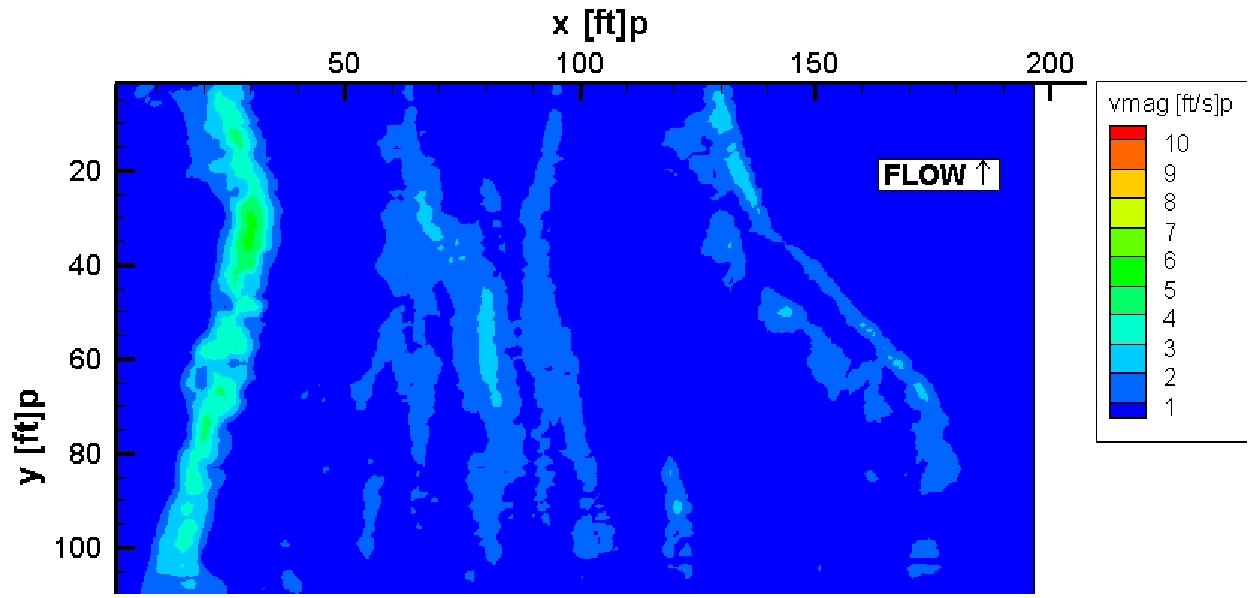

Figure 35. Velocity contours for a flow of 360 ft³/sec over the rock ramp with 640 ft³/sec being diverted through the headworks.

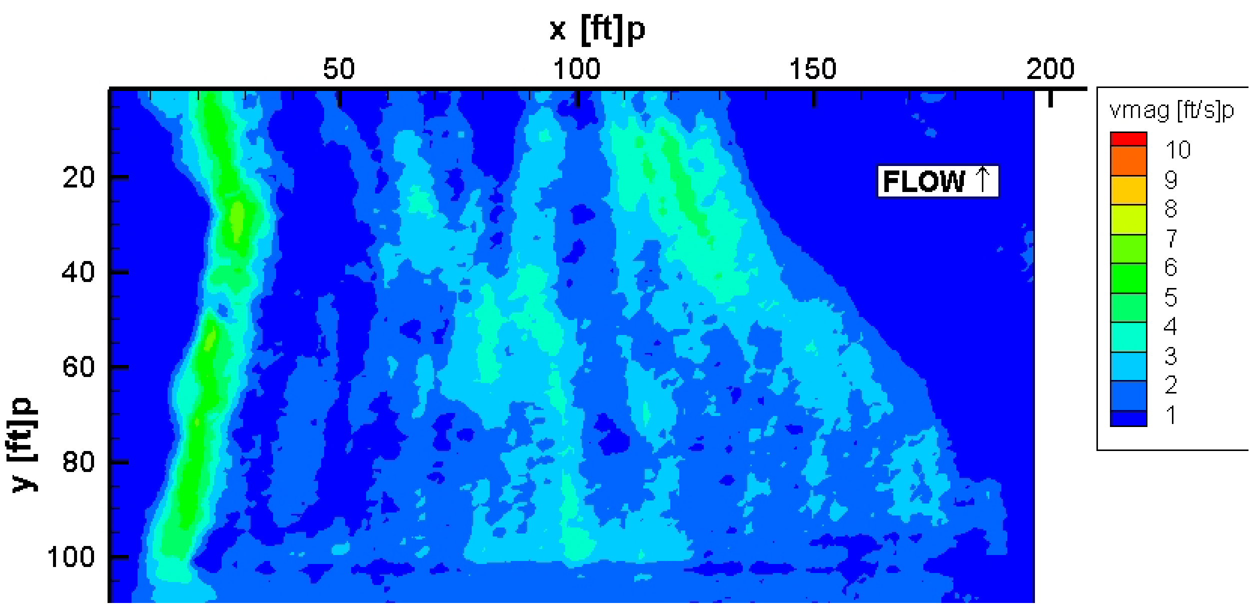

Figure 36. Velocity contours for a flow of 610 ft³/sec over the rock ramp with 640 ft³/sec being diverted through the headworks.

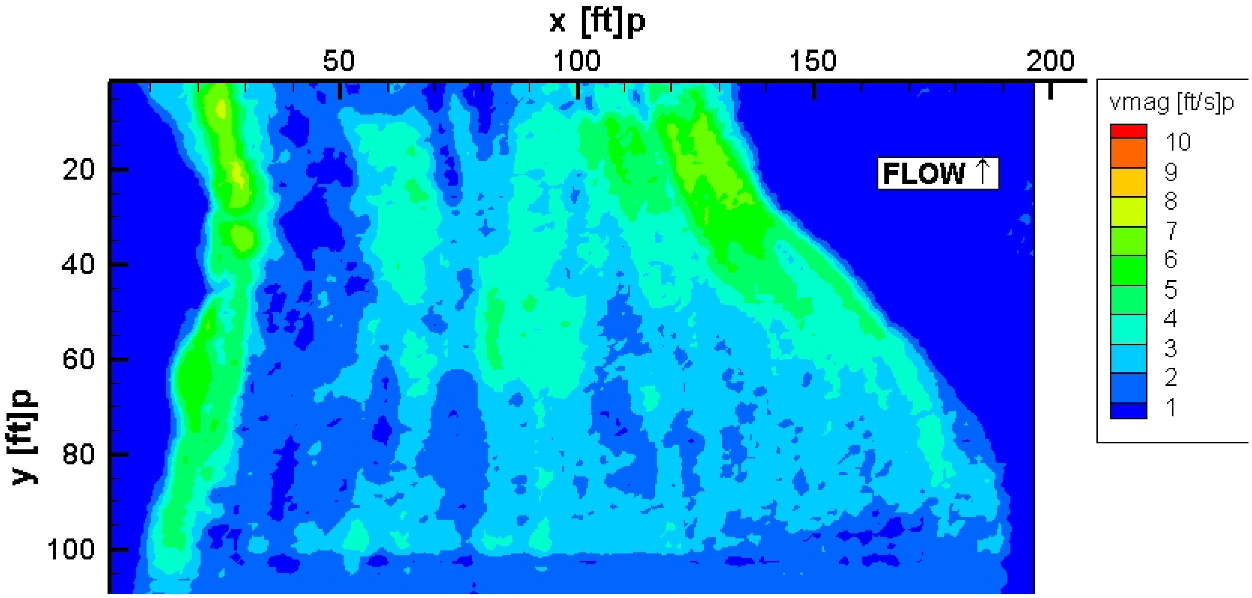

Figure 37. Velocity contours for a flow of 860 ft^3/sec over the rock ramp with 640 ft^3/sec being diverted through the headworks.

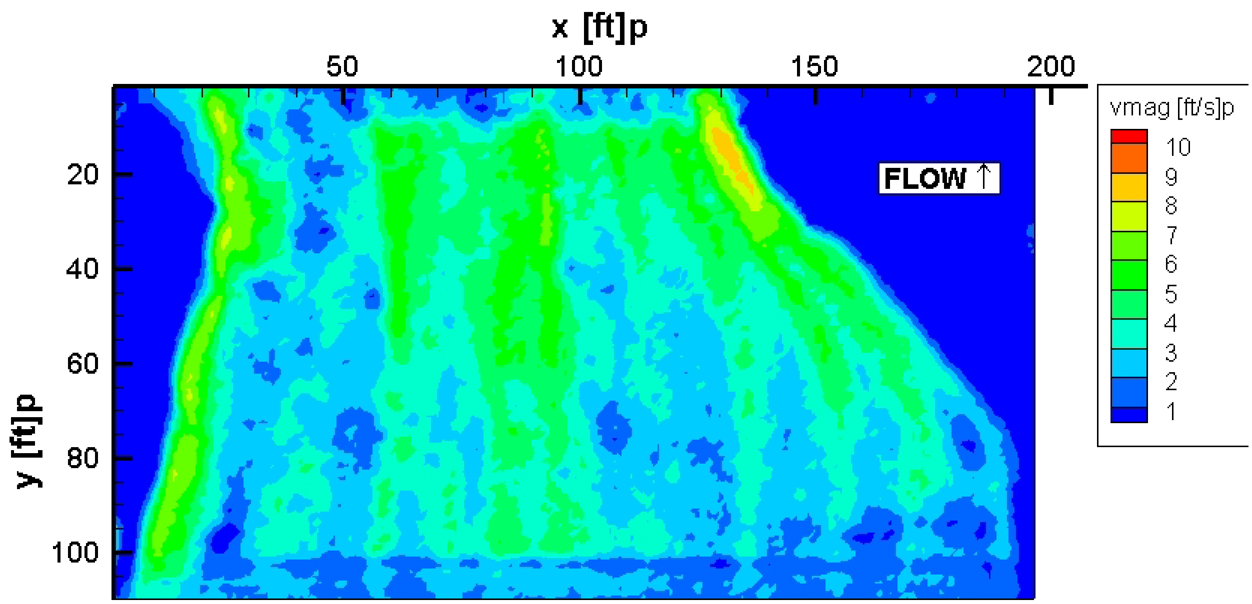

Figure 38. Velocity contours for a flow of 1360 ft^3/sec over the rock ramp with 640 ft^3/sec being diverted through the headworks.

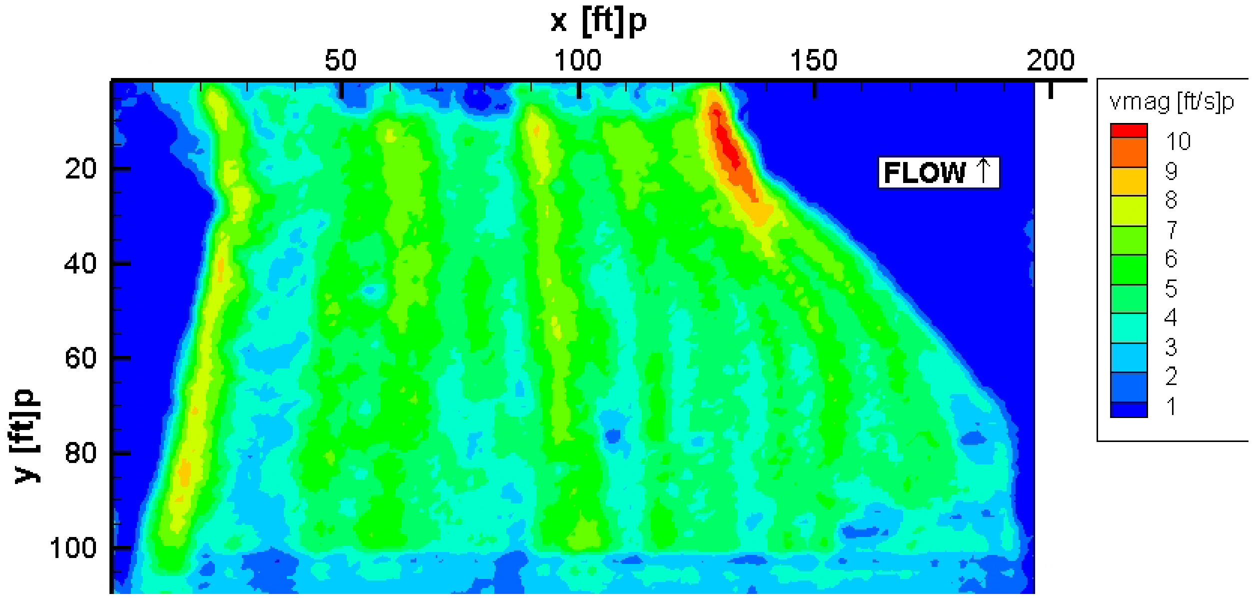

Figure 39. Velocity contours for a flow of 1860 ft³/sec over the rock ramp with 640 ft³/sec being diverted through the headworks.

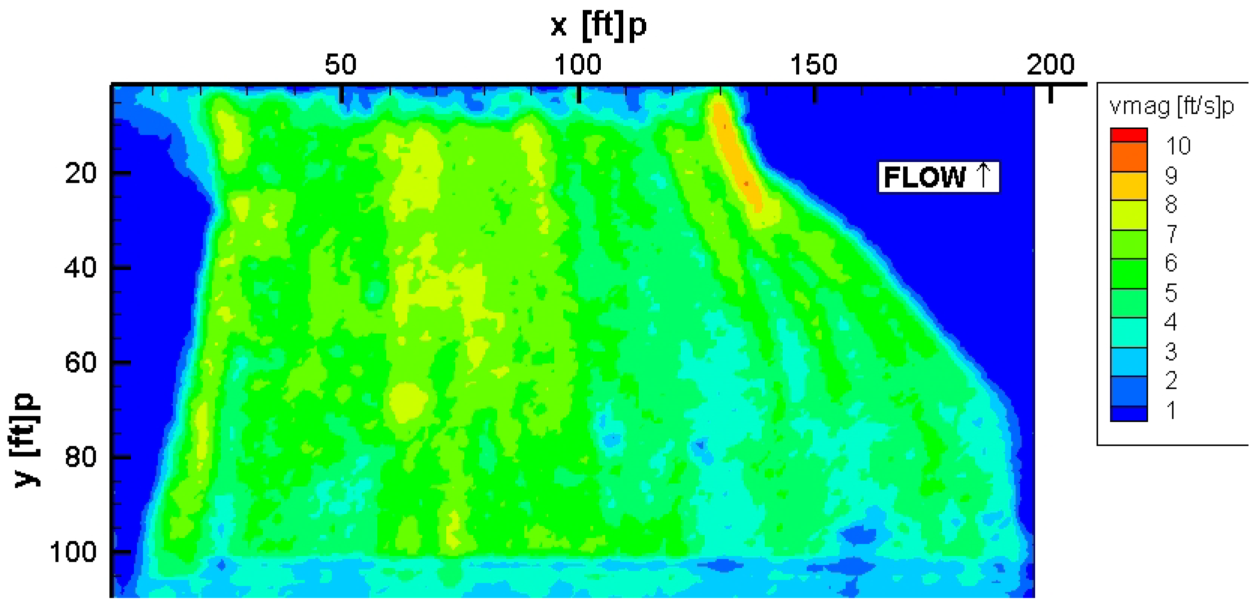

Figure 40. Velocity contours for a flow of 2360 ft³/sec over the rock ramp with 640 ft³/sec being diverted through the headworks.

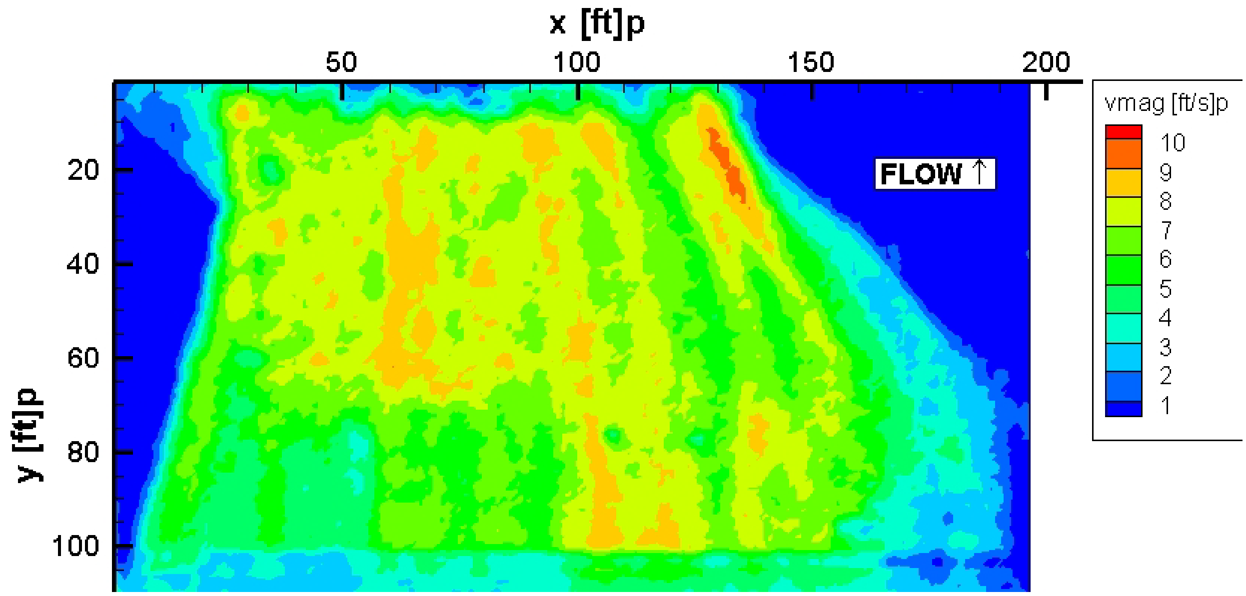

Figure 41. Velocity contours for a flow of 3360 ft³/sec over the rock ramp with 640 ft³/sec being diverted through the headworks.

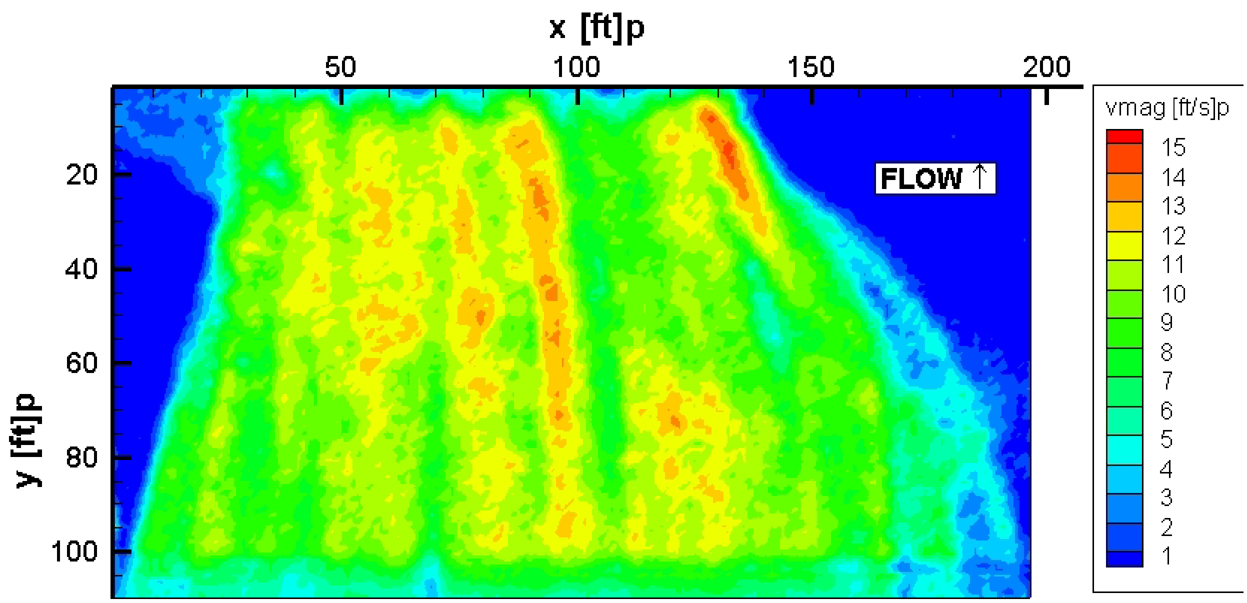

Figure 42. Velocity contours for a flow of 5360 ft³/sec over the rock ramp with 640 ft³/sec being diverted through the headworks.

Table 8. Maximum velocities measured on the rock ramp at flow rates from 360-5360 ft^3/sec passing over the rock ramp obtained using LSPIV technique.

Flow Rate (ft^3/sec)	Maximum Velocity from LSPIV (ft/sec)
360	5.40
610	6.53
860	7.55
1360	9.01
1860	10.78
2360	9.15
3360	9.52
5360	14.93

At the maximum flow passing over the rock ramp (10,000 ft^3/sec), velocity measurements and depths were obtained using a 2D FlowTracker Acoustic Doppler Velocimeter. Figure 43 provides the locations and Table 8 provides the velocity magnitude and depth for those locations. These measurements should be used in the final sizing of riprap and for determining the extent and depth of grouting for grouted riprap.

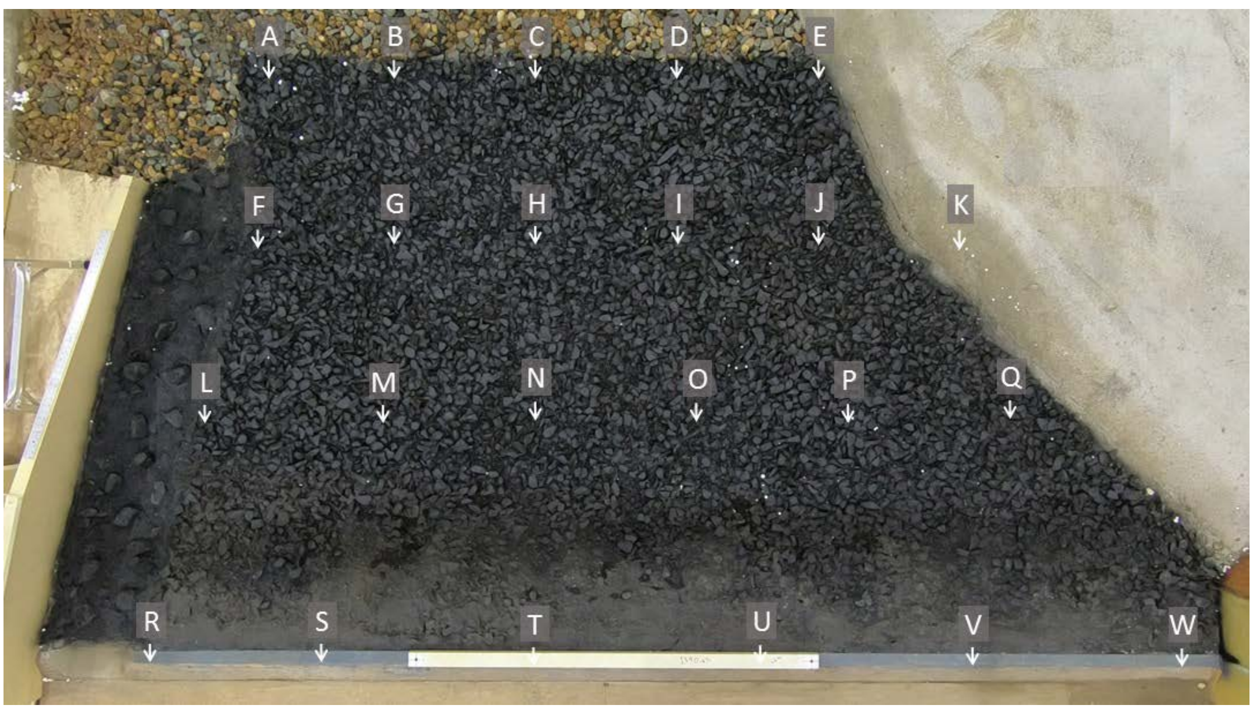

Figure 43. Locations where point velocity measurements were obtained at 10,000 ft^3/sec passing over the rock ramp.

Table 9. Velocity magnitudes and depths measured over the rock ramp with 10,000 ft^3/sec.

Location (see Figure 43)	Velocity Magnitude (ft/sec)	Depth (ft)
A	8.3	7.5
B	10.2	7.5
C	12.1	7.5
D	16.2	7.5
E	17.7	8
F	16.8	5.5
G	17.5	5.5
H	10.1	5.5
I	12.9	5.5
J	12.5	5.5
K	17.9	7
L	19.1	4.5
M	19.3	4.5
N	18.4	4.5
O	13.4	4.5
P	11.5	4.5
Q	10.0	7
R	10.7	5.5
S	10.9	5.5
T	11.1	5.5
U	10.6	5.5
V	9.8	5.5
W	9.8	5.5

Energy Dissipation & Downstream Channel

Energy dissipation over the rock was evaluated by analyzing locations where scour would occur. Model tests were performed using a representative 18-in minus (prototype) material. Slight scour was present at the maximum discharge of 10,000 ft^3/sec. When tested for 5 hours in the model (17.3 hrs prototype) a scour hole developed on the left side of the rock ramp that was 2-in-deep, 12-in-long and 6-in-wide, located about 20-in downstream of the crest. The scour developed gradually and appeared to be stable after the 5 hours. Due to this observed scour, it is recommended that the rock ramp be constructed with a well graded 24-in minus riprap or river rock and that the grouted section be extended to 30-ft downstream of the crest on the left side of the rock ramp.

High velocities also caused scour to occur on the river right bank where the rock ramp meets the existing grade. It is recommended that the transition to the downstream river channel be made more gradually to reduce these high velocities and the potential for scouring. The slope of the rock ramp is approximately 5 percent on the right bank and 7 percent of the left bank which allows flows to slope gradually toward the left bank. This should concentrate flows on river left and allow fish to find the low-flow channel in the event that low depths are occurring on the rock ramp.

Conclusions

Since 2014, modeling has been used to investigate many different design considerations for the St. Mary diversion dam, headworks, fish screen and canal modifications. The latest model of the rock ramp provides design information needed to ensure successful passage of bull trout and other species for a wide range of flows passing over the diversion dam. A new headworks, canal transition, and fish screen are designed to successfully prevent bull trout and other species from being drawn into the canal system.

TSC design engineers successfully modeled the new diversion dam with rock ramp. The proposed diversion dam is 185 ft long and around 6 ft tall with a 5-7 percent partially grouted rock ramp transitioning the 2-ft-wide crest into the natural channel. A new headworks approximately 141 ft wide by 13.5 ft high with a sloped trash rack and six 7.75-ft-wide by 6-ft-high slide gates will allow diversion of up to 890 ft^3/sec (850 ft^3/sec in the canal plus 40 ft^3/sec bypasses). Fish will be protected from entering the canal by a 329.75-ft-long fish screen with 30 individual 60-63% open area fish screen panels that are 9.5-ft wide by 4-ft high with blanking panels above and below the screen. Uniform approach velocity distributions are achieved when baffles are provided about 0.75 ft behind the screen at 25% open area for screens 1-10, 20% open area for screens 11-20 and 17.5% open area for screens 21-30. Fish will be returned to the river by a rectangular bypass channel discharging into the river downstream of the rock ramp on river left. Fish passage over the rock ramp will be available at all flows above 50 ft^3/sec by means of the rock ramp itself or through a fully grouted trapezoidal shaped low flow channel with large boulders. During the off-season, flows will be passed downstream using a 25-ft-wide by 230-ft-long sluiceway with two 10-ft radial gates.

Eddies and recirculation were both present along the upstream face of the fish screens as a result of being immediately downstream of a sharp, short radius bend. These eddies and recirculation could be improved if the distance between the channel bend and start of the screen was increased. Baffling of the fish screens was sufficient to provide nearly uniform approach velocities. Average approach velocities will remain below screen criteria (0.80 ft/sec). The velocity uniformity limit (110% of criteria) was exceeded at a few locations where the screens were in close proximity to the blanking panels, but the extent of these locations was insignificant and the issue was present regardless of the baffle configuration.; this only becomes a concern at diversions of 850 ft^3/sec when the approach velocities may exceed 0.80 ft/sec.

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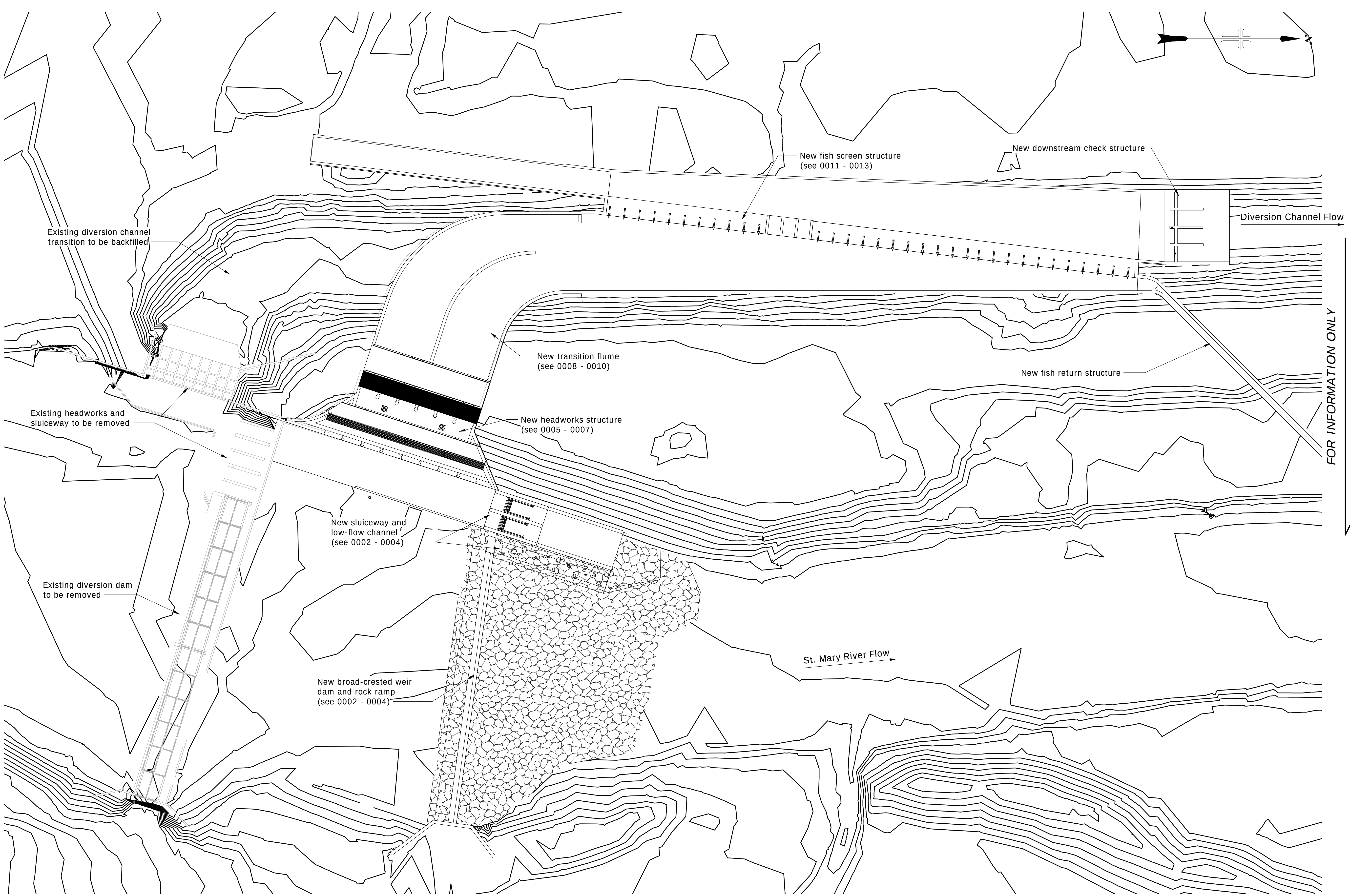
Appendix A – Drawings

The included drawings are for internal use only. These drawings have not been peer-reviewed. The model construction represented the data in these drawings as close as possible.



Overall Site Plan

0 30' 60'

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OVERALL SITE PLAN

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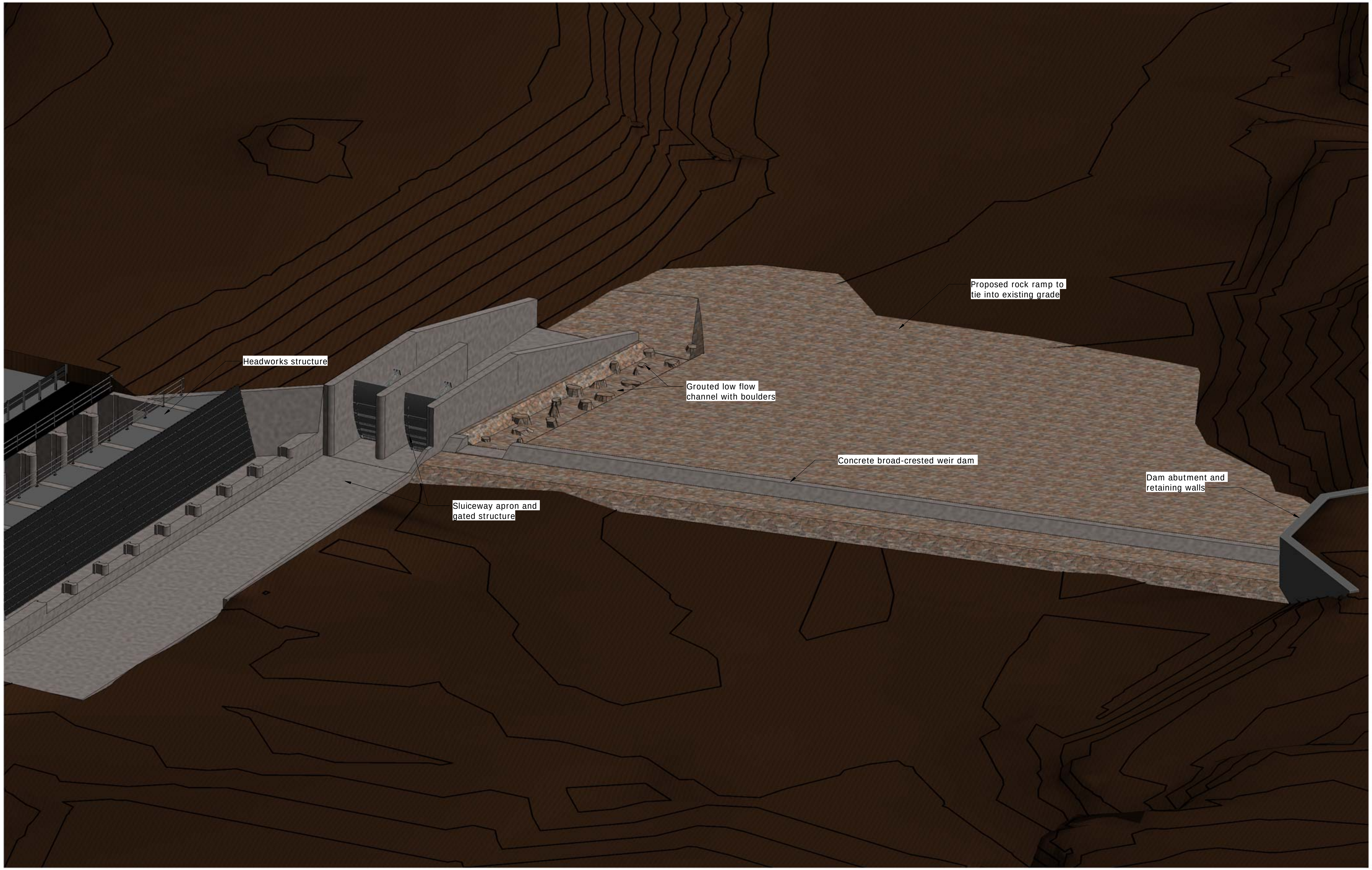

3D View

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DIVERSION DAM AND SLUICEWAY

3D VIEW

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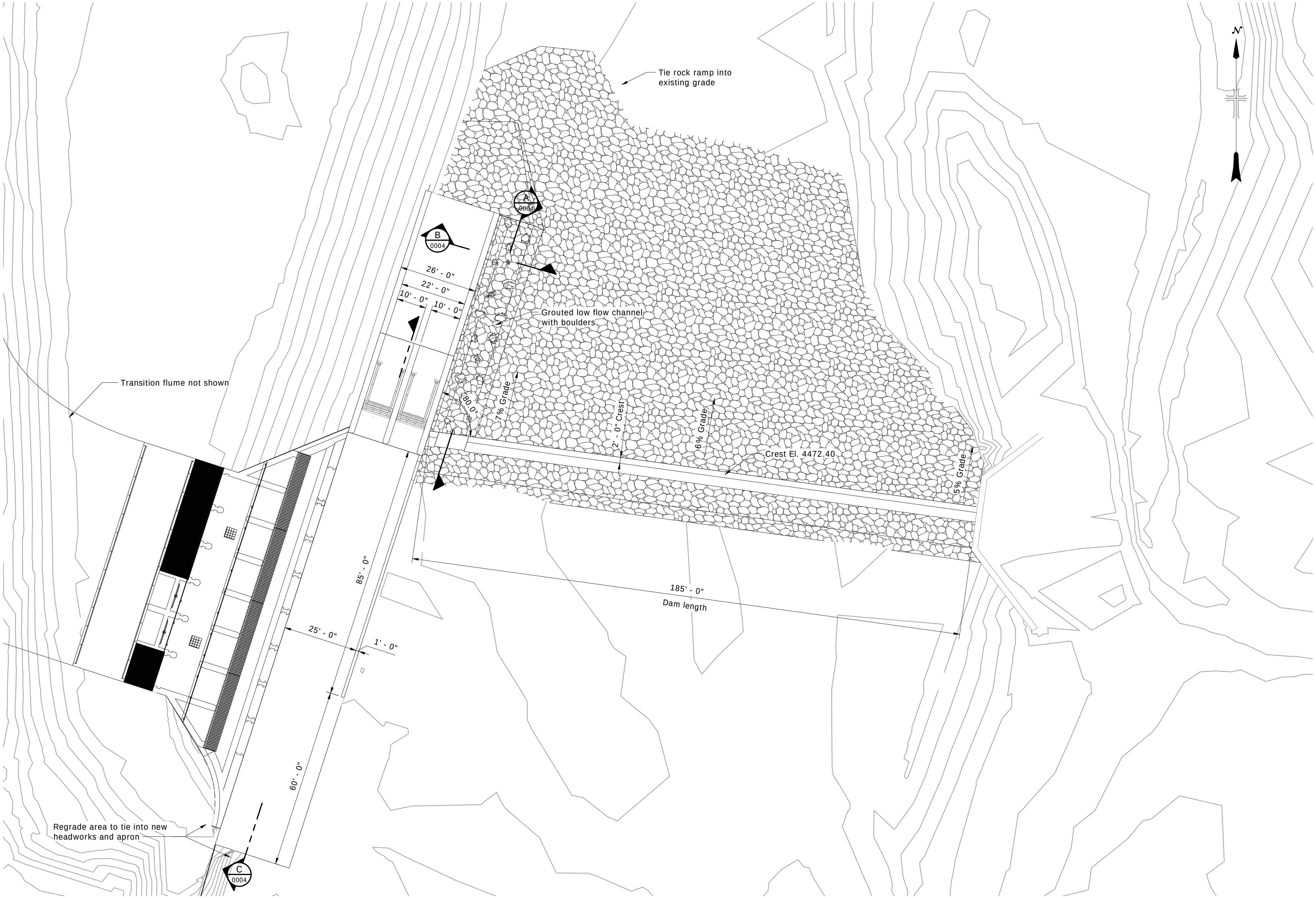

Site Plan

0 8' 16' 32'

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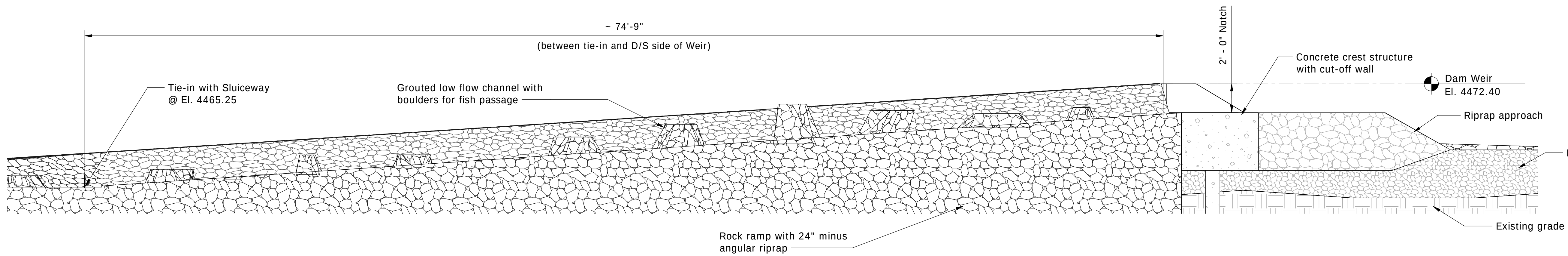
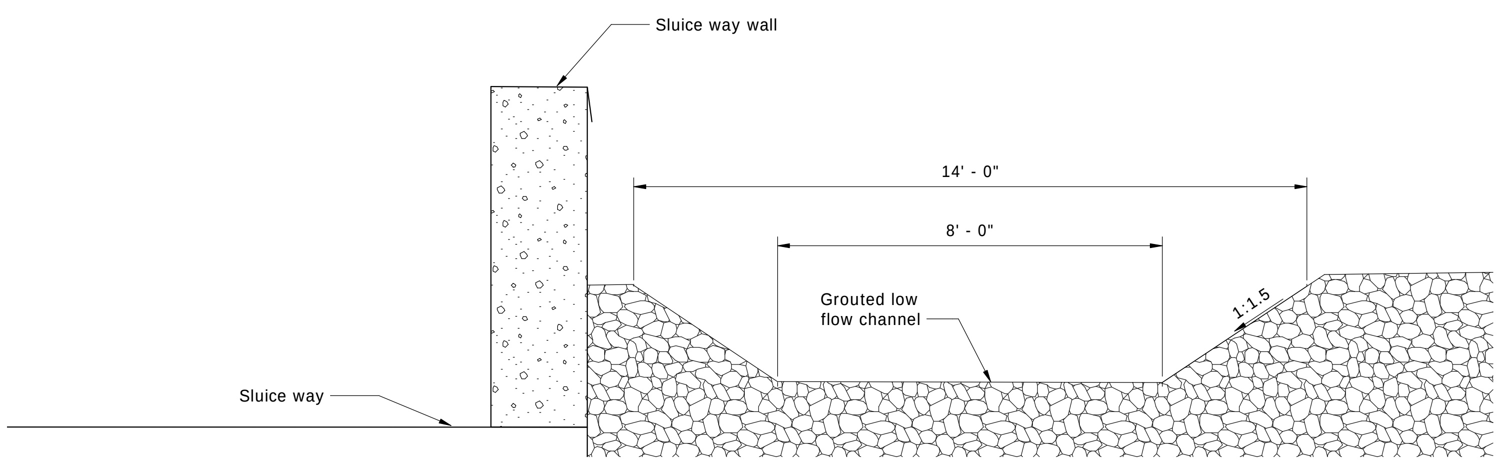


A 0003 Low Flow Channel Profile



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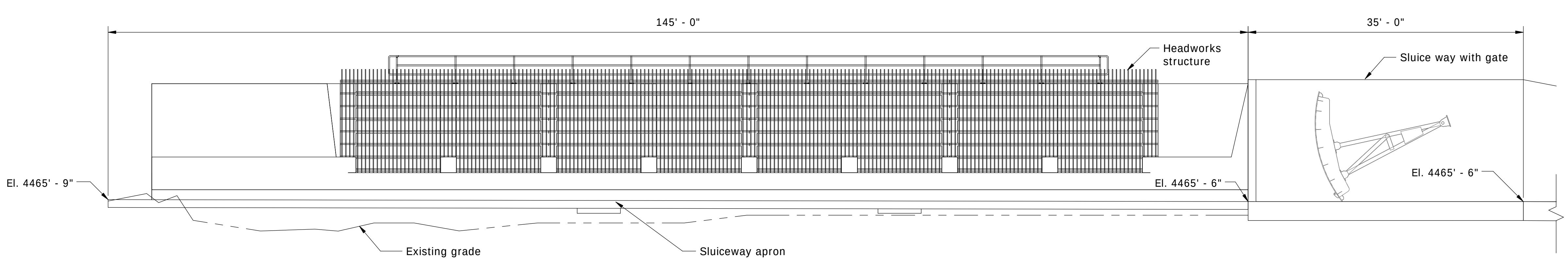

C 0003 Sluiceway Profile

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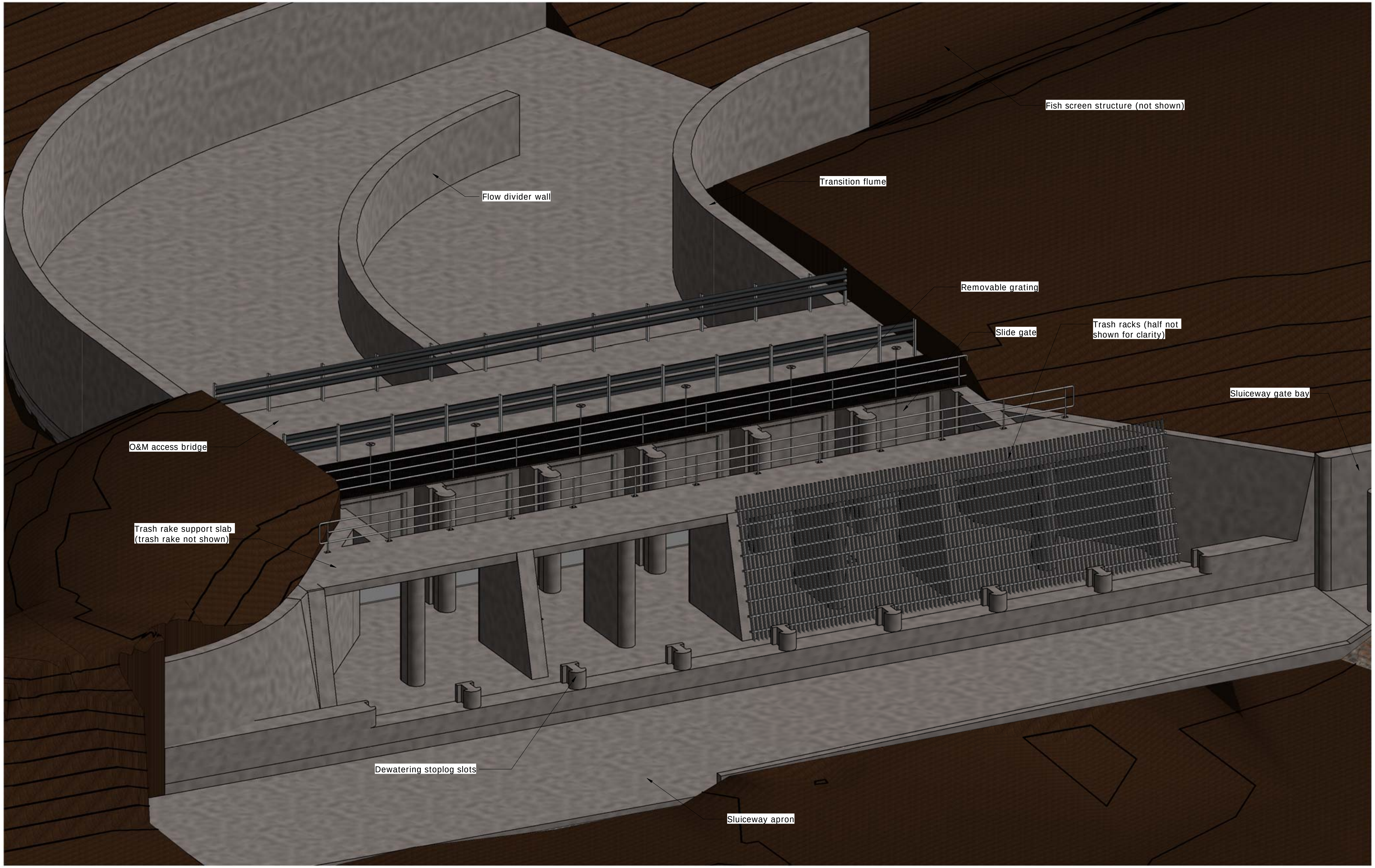

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HEADWORKS STRUCTURE

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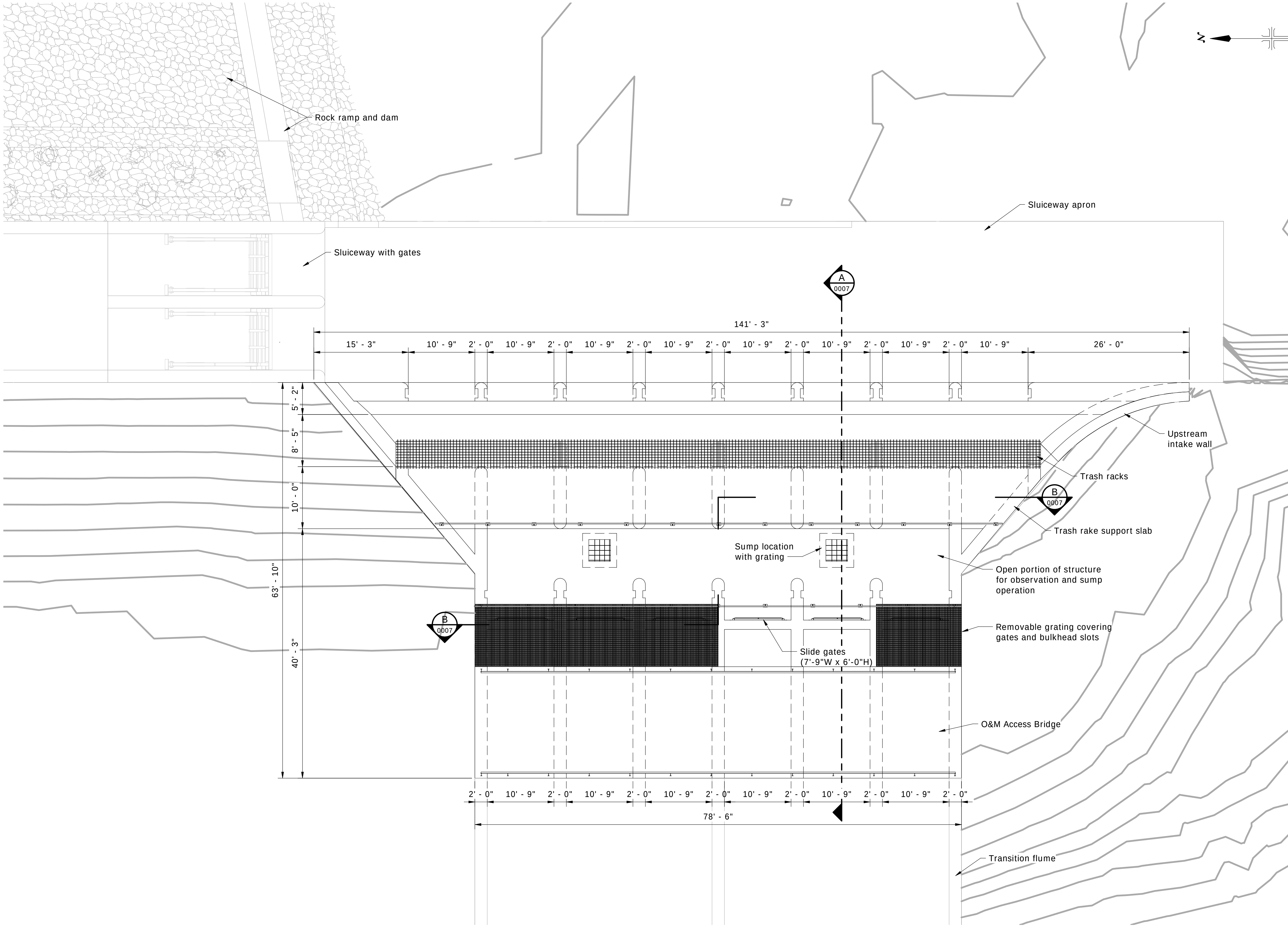

Site Plan

0 4' 8' 16'

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HEADWORKS STRUCTURE

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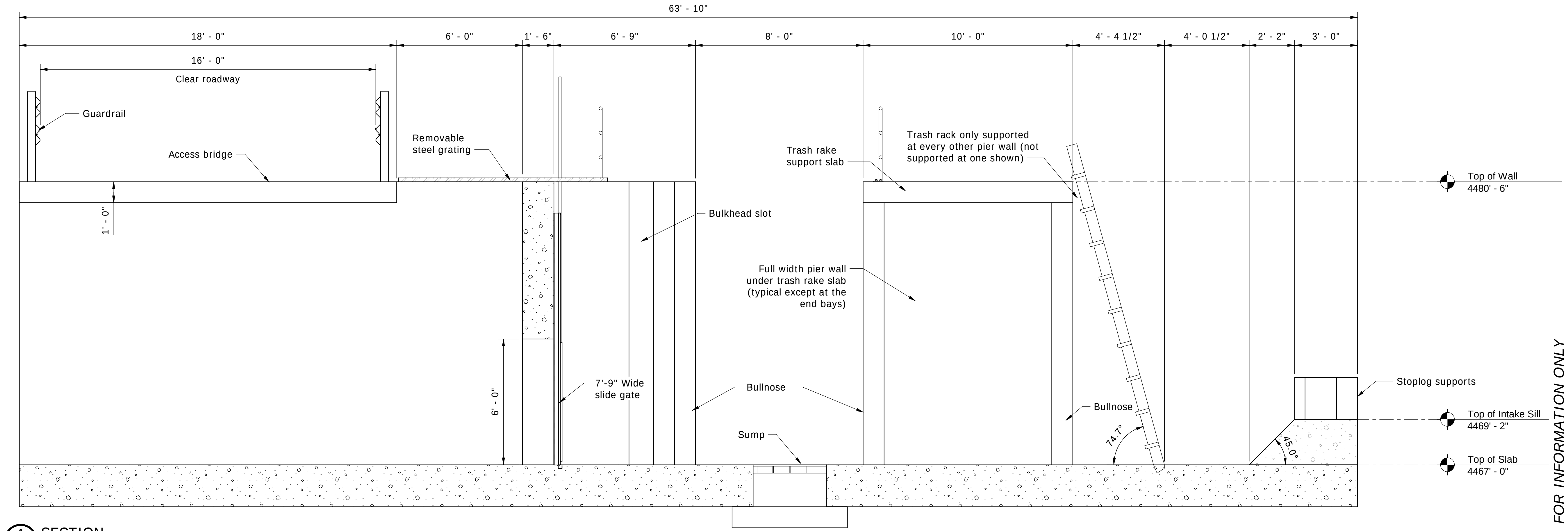

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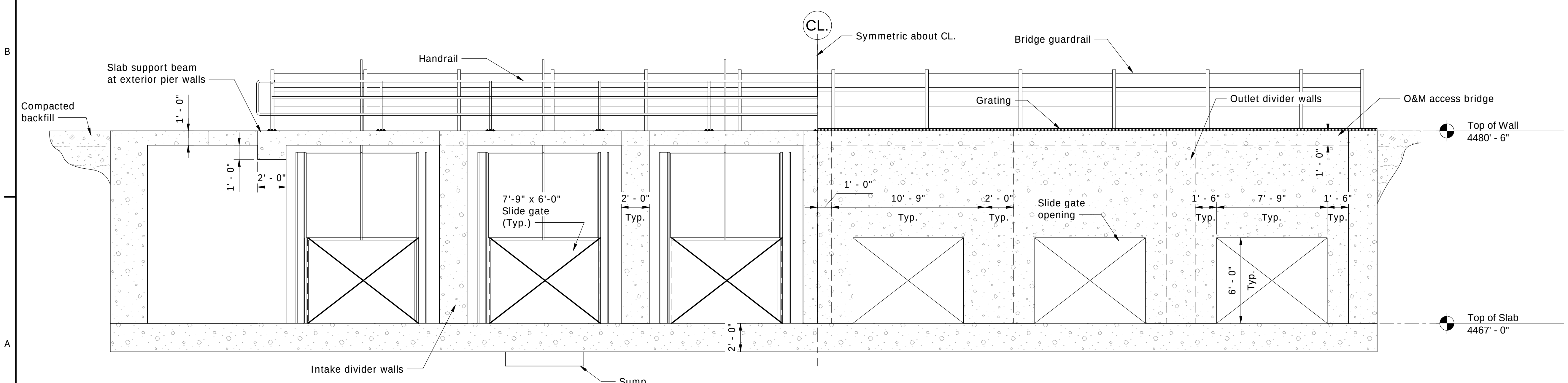

B SECTION 0006

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HEADWORKS STRUCTURE

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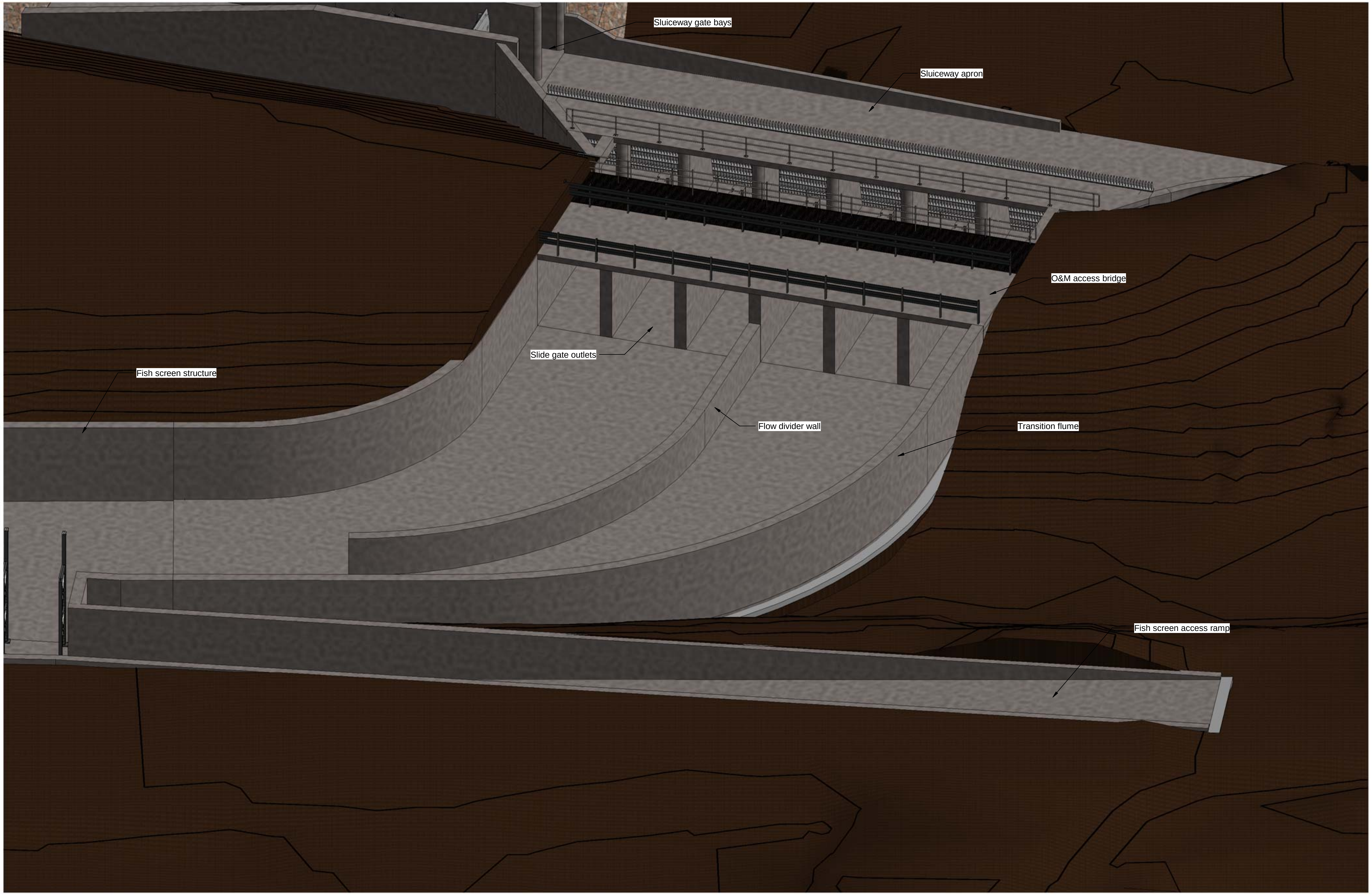

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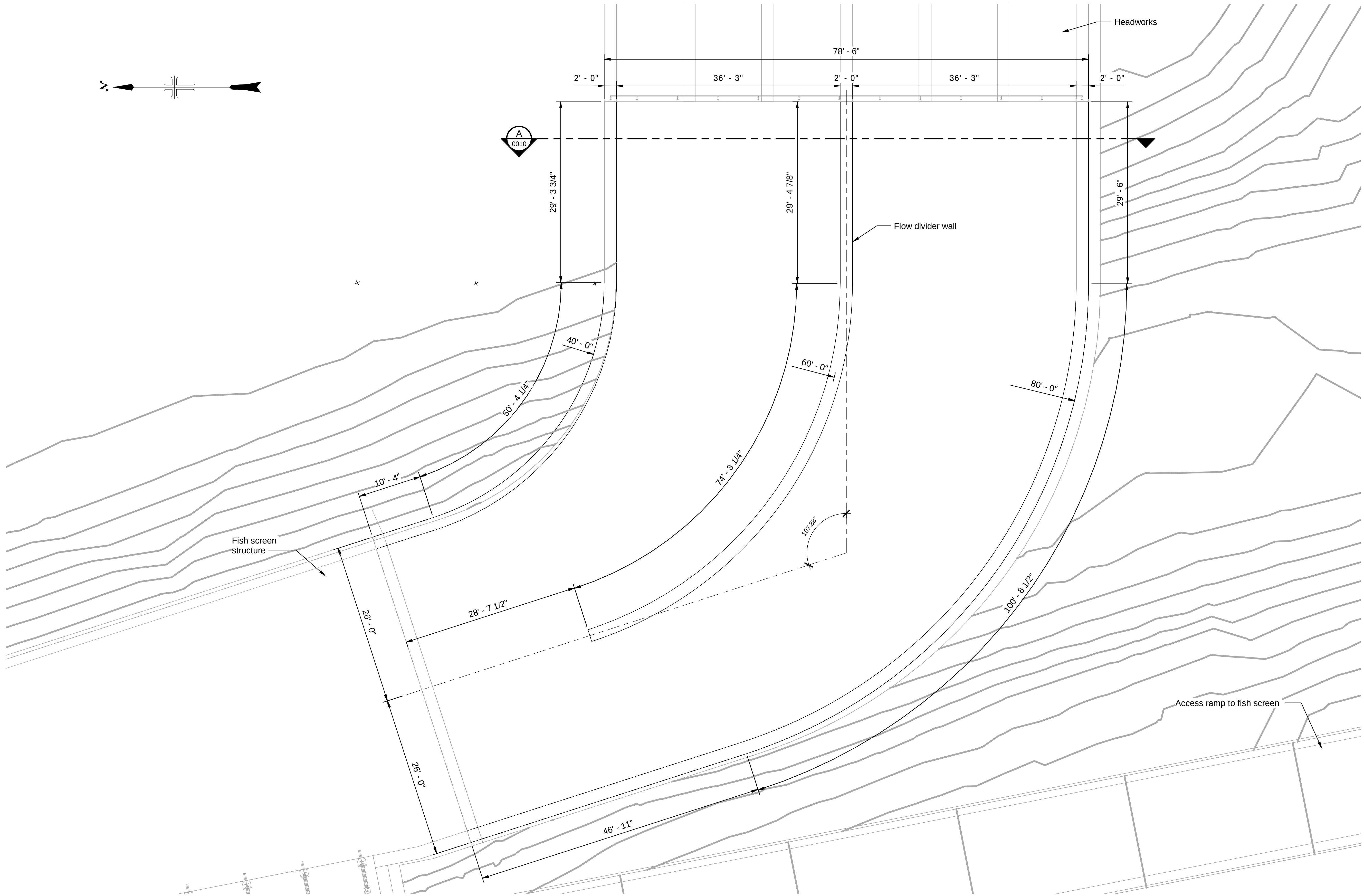

Site Plan

0 4' 8' 16'

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TRANSITION FLUME

SITE PLAN

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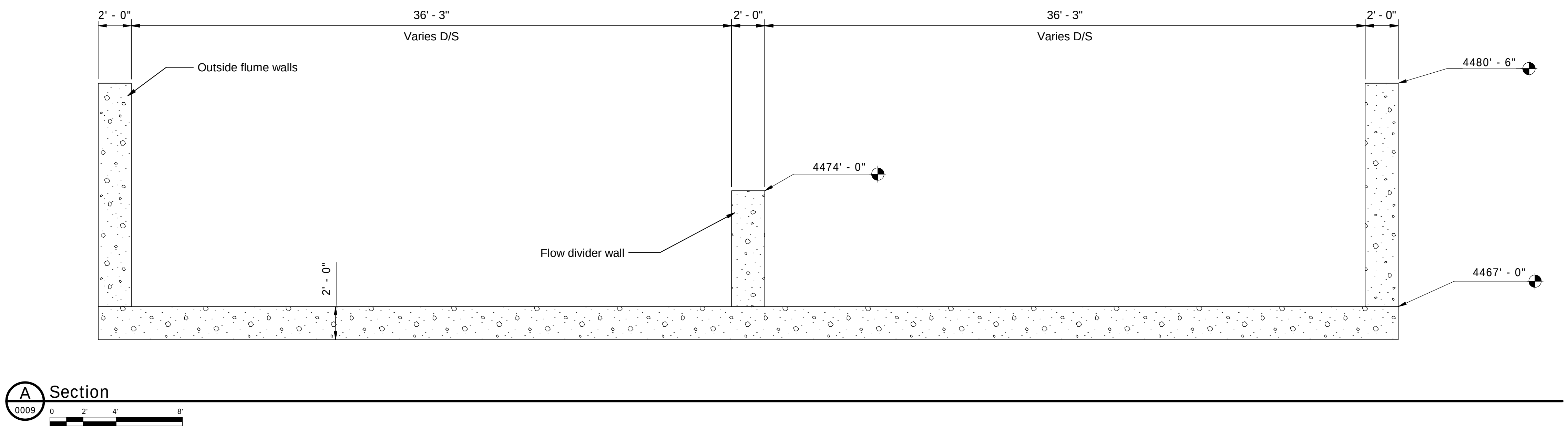

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TRANSITION FLUME

SECTIONS

SM-HLR-0010

DESIGN PHASE:
Project Status

DATE AND TIME PLOTTED:
3/22/2018 10:41:25 AM

CAD SYSTEM:
Revit 2017.2

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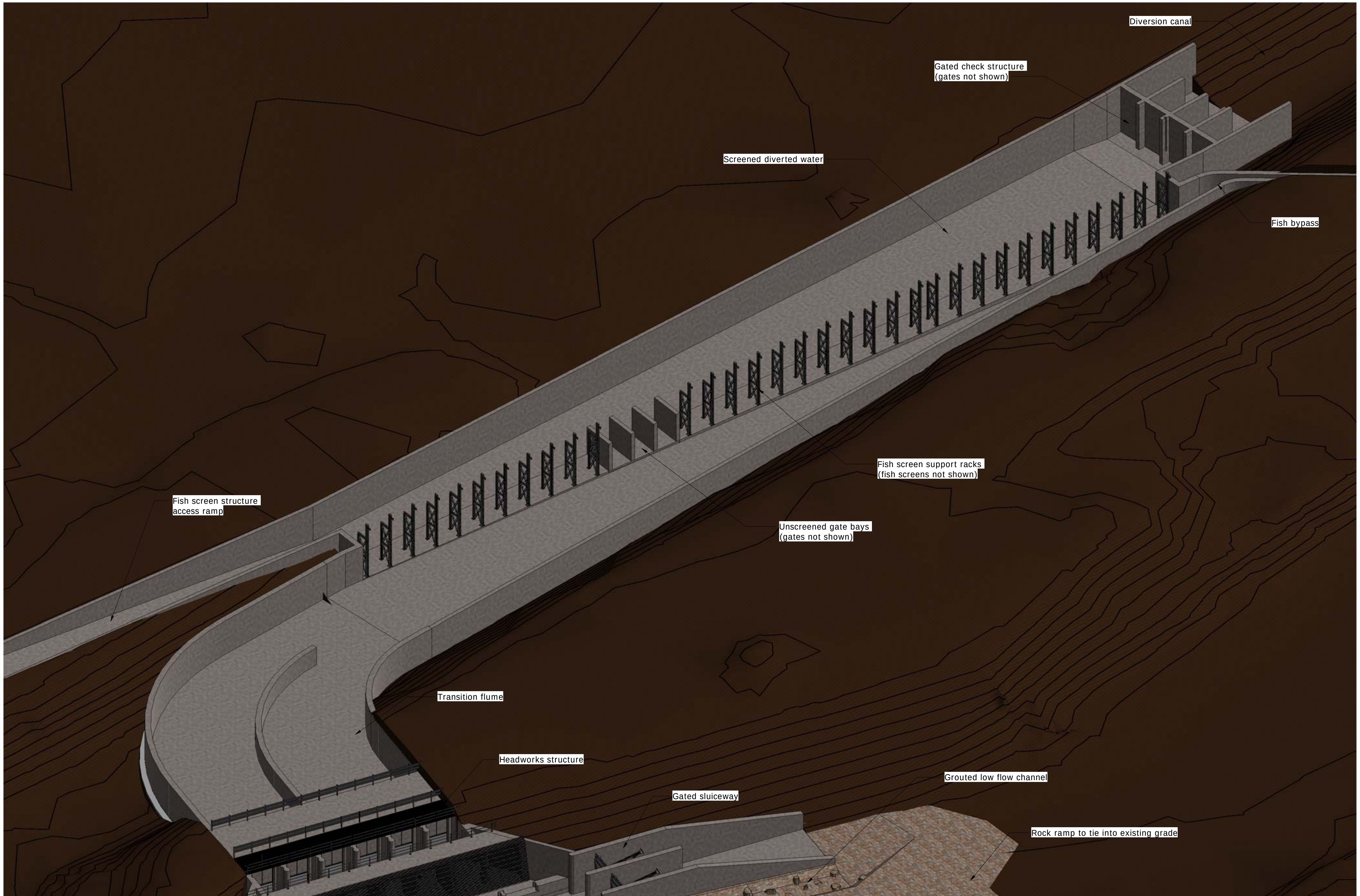

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Managing Water in the West

ALWAYS THINK SAFETY

U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MILK RIVER PROJECT - MONTANA
MILK RIVER STORAGE DIVERSION
ST. MARY DIVERSION DAM REPLACEMENT
HYDRAULIC LAB REPORT

REV NO	DATE	NAME PROF. ABBR.	ST A	DESCRIPTION

Designer
DESIGNED
Author
DRAWN
Checker
CHECKED
TECH. APPR.
APPROVED Approver

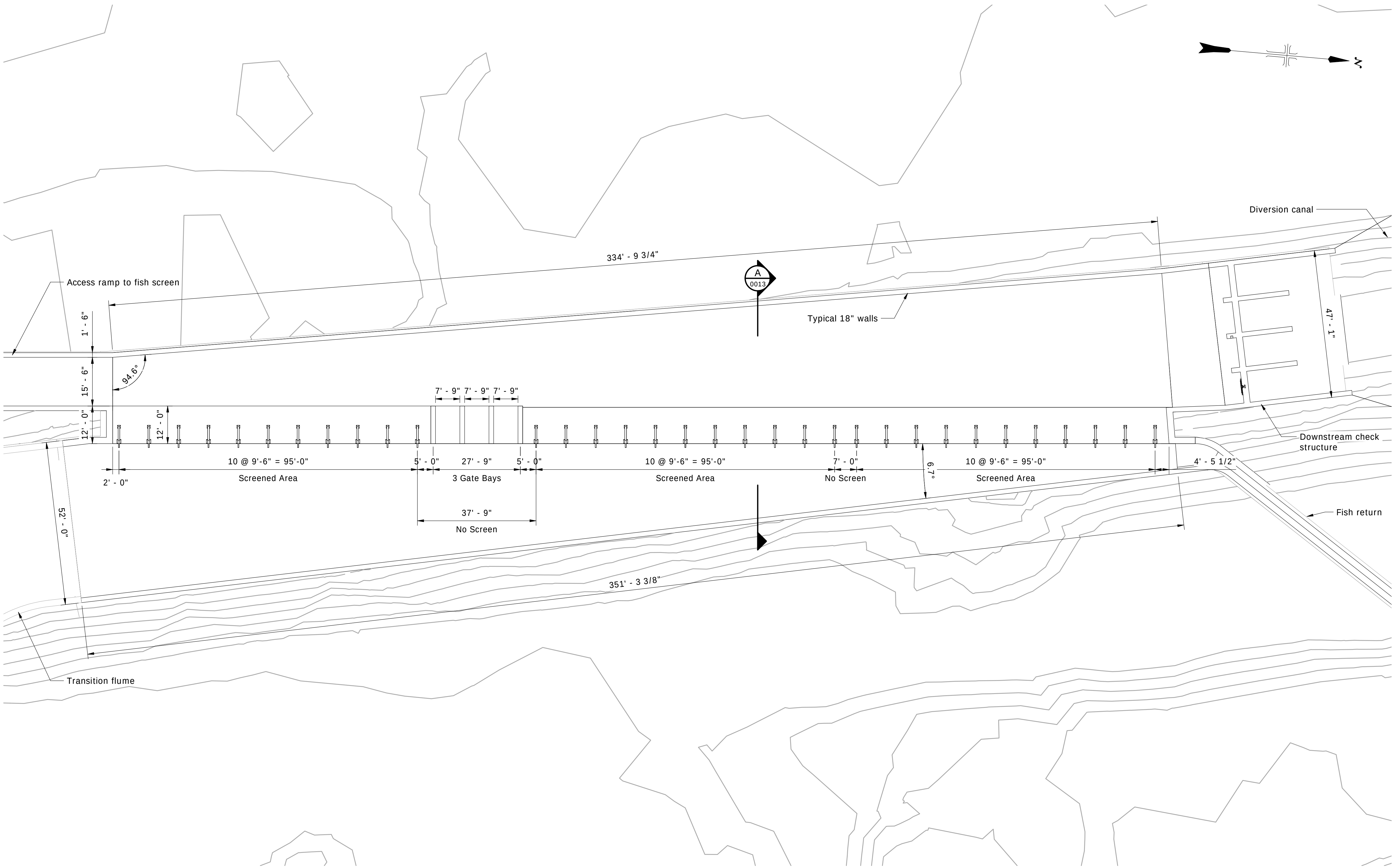
03/22/18

FISH SCREEN STRUCTURE

3D VIEW

SM-HLR-0011

RECLAMATION
Managing Water in the West



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Site Plan

0 8' 16' 32'

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ST. MARY DIVERSION DAM REPLACEMENT

HYDRAULIC LAB REPORT

REV NO	DATE	NAME PROF. ABBR.	STA	DESCRIPTION

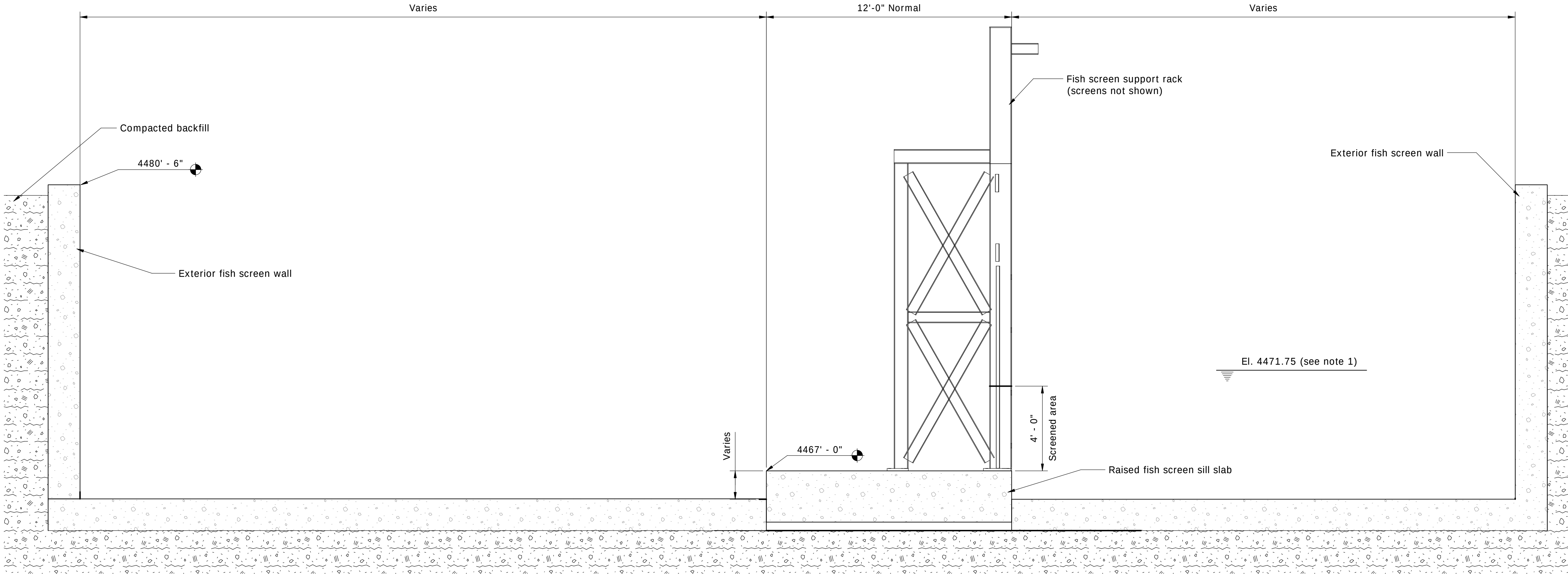
DESIGNED RYAN KENT, P.E.
DRAWN
CHECKED
TECH. APPR.
APPROVED Approver

03/20/18

FISH SCREEN STRUCTURE

SITE PLAN

SM-HLR-0012



A Section

0 2' 4' 6'

Notes:

1) This is the elevation of the normal checked water surface. However, a barrier plate is being planned to ensure ice forming at the surface does not damage the screen. This plate will stick down into the water 9" making the top of the screened area El. 4471.00.

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FISH SCREEN STRUCTURE

SECTIONS

SM-HLR-0013

Appendix B – Rating Curves

Rating curves were developed for the dam structure with no diversion flow and with diversion canal flow rates of 600 and 850 ft^3/sec. Diversion flow rate refers to the amount of water being delivered to the downstream canal. For each rating curve where the diversion canal was operating, the fish bypass was operated at 40 ft^3/sec. Total diversion flows from the river (through the trashrack and headworks gates) for each case would equal the diversion flow rate plus the bypass flow. This appendix includes tabulated and plotted rating curves for no diversion (Table 9 & Figure 44), 600 ft^3/sec (Table 10 & Figure 45) and 850 ft^3/sec (Table 11 & Figure 46).

No Diversion Rating

Table 10. Upstream river, diversion, fish ladder, and fish bypass flow rates with corresponding upstream river water surface elevations when the diversion is not in operation.

Upstream River Flow Rate (ft^3/sec)	Upstream River Water Surface Elevation (ft)	Rock Ramp Flow Rate (ft^3/sec)	Diversion Flow Rate (ft^3/sec)	Fish Bypass Flow Rate (ft^3/sec)
125	4472.54	125	0	0
250	4472.84	250	0	0
500	4473.20	500	0	0
750	4473.50	750	0	0
1000	4473.76	1000	0	0
1250	4473.97	1250	0	0
1500	4474.19	1500	0	0
2000	4474.58	2000	0	0
2500	4474.90	2500	0	0
3000	4475.28	3000	0	0
4000	4475.77	4000	0	0
6000	4476.68	6000	0	0
8000	4477.64	8000	0	0
10000	4478.57	10000	0	0

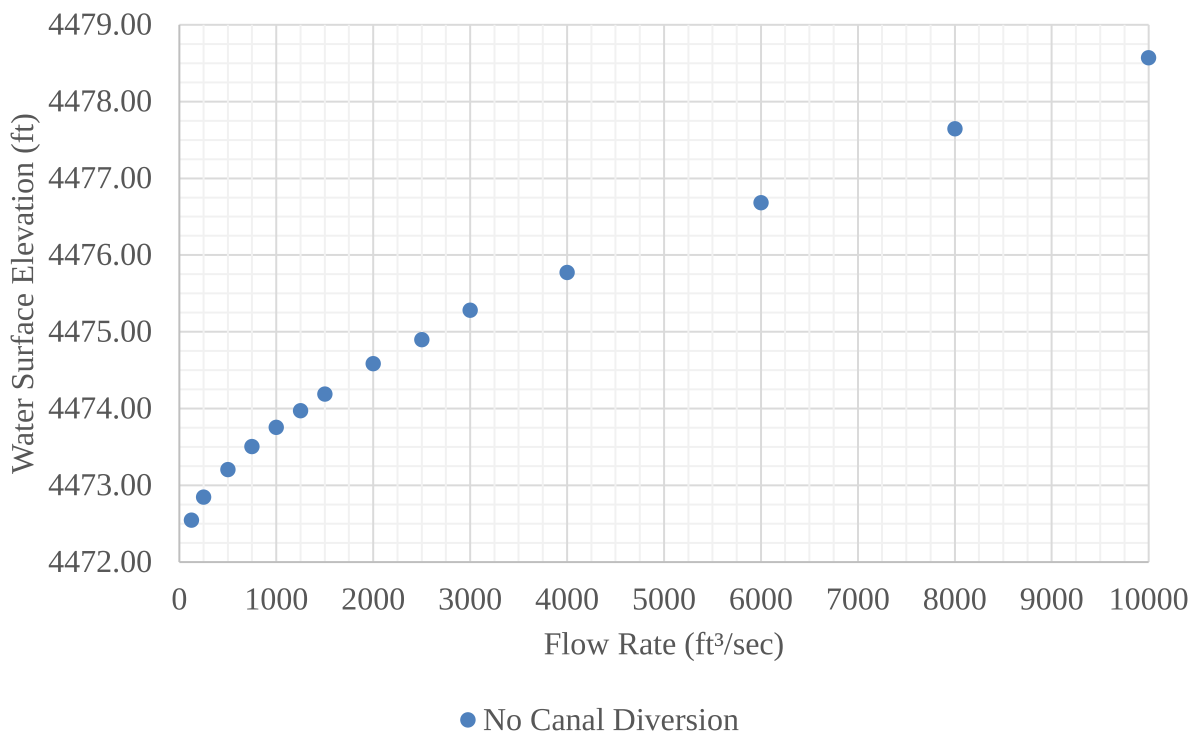

Figure 44. Dam rating curve with no diversion flow.

650 ft^3/sec Diversion Rating

Table 11. Upstream river, diversion, fish ladder, and fish bypass flow rates with corresponding upstream river water surface elevations with a diversion of 600 ft^3/sec.

Upstream River Flow Rate (ft^3/sec)	Upstream River Water Surface Elevation (ft)	Rock Ramp Flow Rate (ft^3/sec)	Diversion Flow Rate (ft^3/sec)	Fish Bypass Flow Rate (ft^3/sec)
700	4472.06	60	600	40
750	4472.51	110	600	40
1000	4473.03	360	600	40
1250	4473.34	610	600	40
1500	4473.60	860	600	40
2000	4474.05	1360	600	40
2500	4474.42	1860	600	40
3000	4474.76	2360	600	40
4000	4475.37	3360	600	40
6000	4476.39	5360	600	40

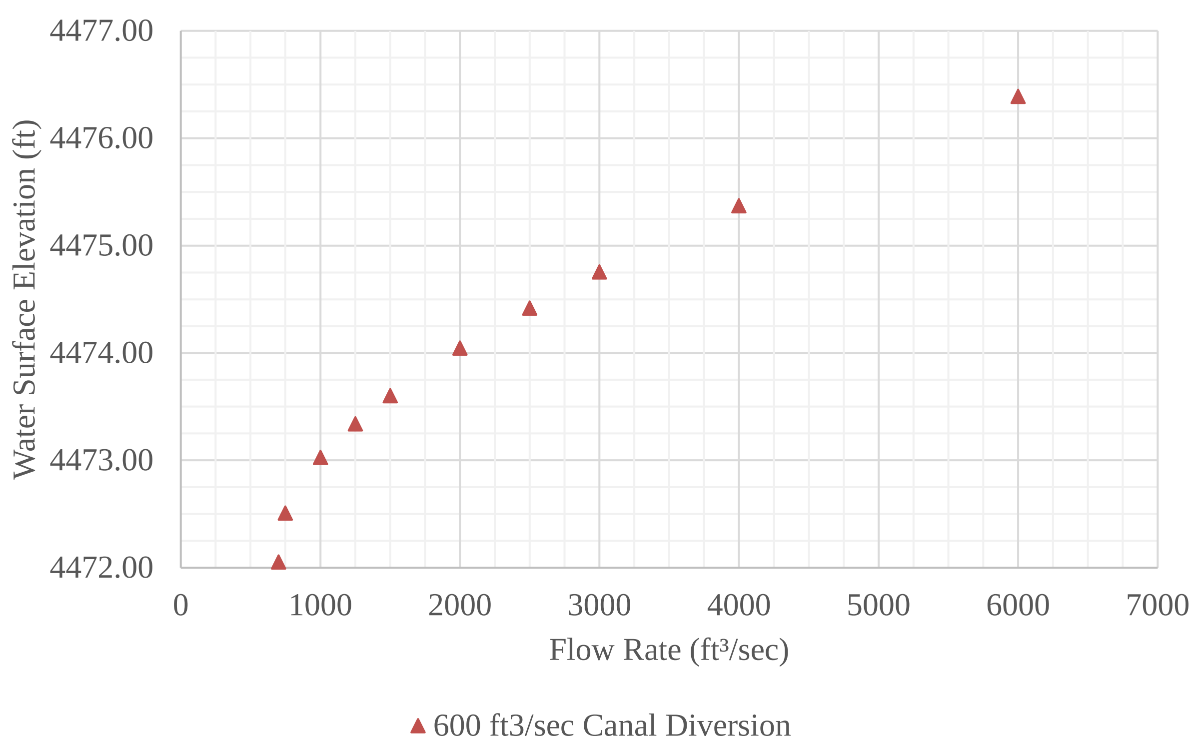

Figure 45. Rating curve for diversion flow rate of 650 ft^3/sec.

850 ft^3/sec Diversion Rating

Table 12. Upstream river, diversion, fish ladder, and fish bypass flow rates with corresponding upstream river water surface elevations with a diversion of 850 ft^3/sec.

Upstream River Flow Rate (ft^3/sec)	Upstream River Water Surface Elevation (ft)	Rock Ramp Flow Rate (ft^3/sec)	Diversion Flow Rate (ft^3/sec)	Fish Bypass Flow Rate (ft^3/sec)
1000	4472.61	110	850	40
1250	4473.05	360	850	40
1500	4473.34	610	850	40
2000	4473.84	1110	850	40
2500	4474.24	1610	850	40
3000	4474.59	2110	850	40
4000	4475.21	3110	850	40
6000	4476.24	5110	850	40

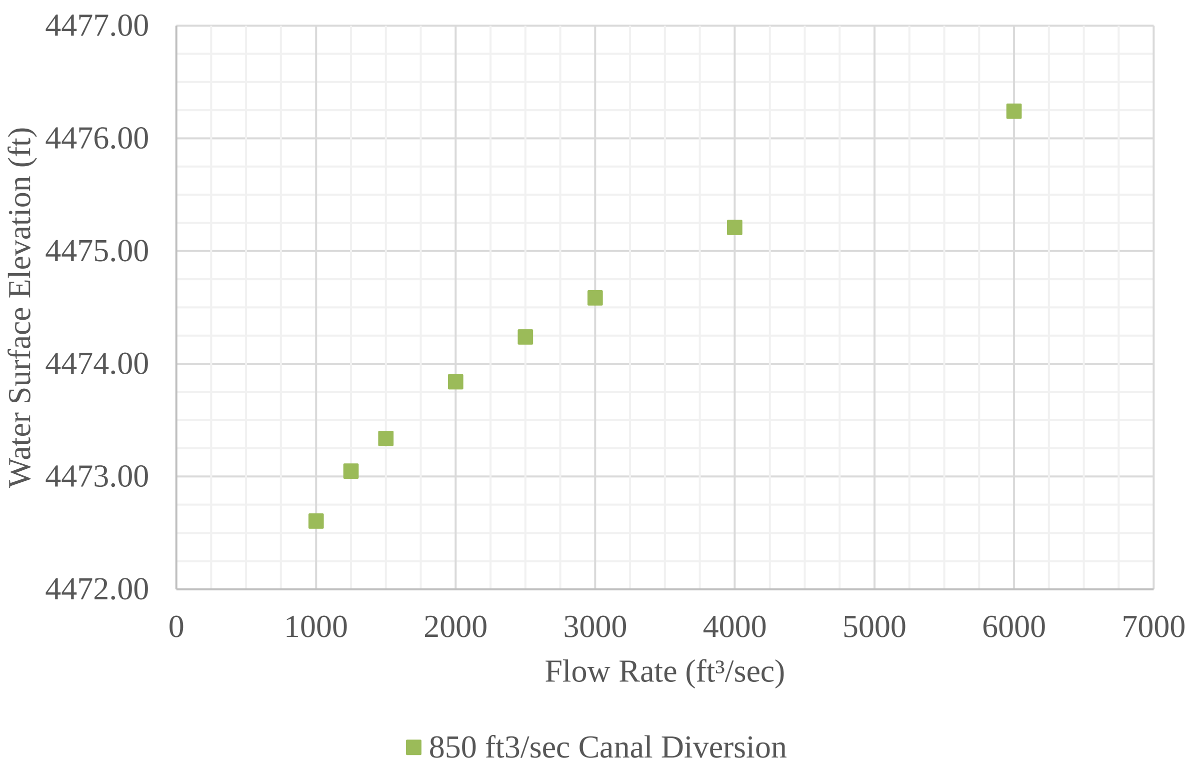

Figure 46. Dam rating curve for diversion flow rate of 850 ft^3/sec.