# ALTERNATIVES ANALYSIS

July 28, 2020

## Al Helenberg Memorial Boat Launch Safety Improvements

*Castle Rock, Washington*

Prepared for

**City of Castle Rock**
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# SIGNATURE PAGE

The information and data in this report were compiled and prepared under the supervision and direction of the undersigned.

Lynn Simpson

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## NEED AND PURPOSE

### PROJECT NEEDS

Boaters using the Castle Rock Al Helenberg Memorial Boat Launch ramp and floating docks on the Cowlitz River during high wintertime river flows have stated that using the upstream lane of the ramp is difficult or dangerous.

#### Location

The Al Helenberg Memorial Boat Launch was constructed in 2010 approximately 1,300 feet upstream of the State Route 411 Bridge to provide access to the Cowlitz River (see figures in Appendix A).

#### Background

Hydrology in the project area was discussed in the technical memorandum by WEST Consultants, Inc. to the City of Castle Rock (City) dated November 1, 2016 (see Appendix A). Excerpts from this memorandum are included below:

> USGS Gage *142430000 Cowlitz River at Castle Rock, Washington* is located approximately 1,400 feet downstream of the ramp at the Hwy 411 Bridge (A Street) and has a period of record of 90 years (1926 to present). Mean daily flow records are available for the prior 10 years (2006 – 2016) and mean stage are available for the prior years. Mean daily flow data for the 2006 – 2016 period were plotted and reviewed, and a flow duration curve developed. Based on review of the flow data, three flows were chosen to be simulated in the hydraulic models:
>
> - 30,000 cfs – represents the approximate upper limit of usability of the ramp
> - 9,000 cfs – represents a typical winter flow rate
> - 5,000 cfs – represents a typical summer flow rate
>
> (cfs = cubic feet per second)

### BASIC PROJECT PURPOSE

The basic project purpose is to reduce the streamflow velocity at the boat ramp and floats during high river flows to improve safety and river access for boaters, as well as to provide improved access during emergencies on the river. At the same time, the City would like sedimentation conditions at the ramp to not be made worse by the project.

Streamflow needs to be reduced in the river to achieve the basic project need and purpose. Therefore, this project is water dependent.

### OVERALL PROJECT PURPOSE AND GEOGRAPHIC AREA

Install an instream structure upstream of the boat ramp and floats that will reduce stream velocity during high river flows of up to 30,000 cfs to a safe velocity for boaters to use the boat ramp and floats, and it should not significantly increase sedimentation at the ramp or floats.

## PROJECT CRITERIA

The following criteria are required to meet the overall project purpose:

### STREAM VELOCITY

The selected alternative must reduce stream velocity at the boat ramp and floats to conditions that will be safe for boating activities at flows of up to 30,000 cfs (approximately 42 feet in elevation NAVD88). Stream velocity should not be increased in surrounding areas to cause increased bank erosion on adjacent properties.

### SEDIMENTATION

The selected alternative will not increase sedimentation rates in the boat ramp and float areas.

## PRACTICABILITY EVALUATION FOR INSTREAM STRUCTURE

### IN-STREAM STRUCTURE ALTERNATIVES

The no-action alternative does not meet the need or overall project purpose, so it is not practicable and will not be addressed further in this document.

**Structures Using Natural Materials**

Engineering techniques for the instream structure that were considered for this project included those that use more natural materials to divert river flow away from the boat ramp and float areas. These alternatives included wood and/or rock structures that would need to have a top elevation at the 43-foot elevation (one foot above the 30,000 cfs flow) to be effective. This would require the natural structures to have a large footprint to remain stable during high flows. There have been structures installed in other southwest Washington streams (Grays River and North Fork Toutle River) to reduce flow velocities. These structures failed during the first high flows after they were constructed. In addition, these structures do not meet the sedimentation criterion because they create eddies, resulting in downstream sediment build-up. Because the risk of failure is significant, and they do not reduce sedimentation, these structures are not practicable.

**Engineered Structures**

An engineered, velocity-reduction structure that has been successful in the Columbia River in Portland, Rainier, and Longview is a series of panel walls with gaps between them (see attached photographs). This design reduces stream velocity while still allowing flow through and around the structures to avoid an eddy effect. The attached technical memorandum reviewed 16 alternatives that included different wall placement locations and angles. Stream velocities, sheer stress, and sedimentation were assessed to determine the preferred alternative.

***Preferred Alternative***

The attached technical memorandum that addresses velocity-reduction alternatives was for the original consideration of a three-panel wall. Since that document was completed, the final design of a two-panel wall was selected because engineers saw that the panel next to the bank was not affecting velocity or sedimentation rates. Therefore, the results of the analysis also applies to the two-wall design.

The technical memorandum states that 16 alternatives were considered, and it provides details of the preferred alternative results. This alternative was selected because it will reduce velocities near the end of the boarding floats from 5 feet per second (ft/s) to 3 ft/s for the 30,000 cfs flow. Velocities at the end of the boarding floats will be reduced from 2.3 ft/s to 1.3 ft/s for the 9,000 cfs flow, and velocities will remain approximately the same for the 5,000 cfs flow. This velocity reduction is favorable from a hazard perspective and would provide a safer ingress/egress zone for boaters, particularly during higher flow conditions. The model shows that sedimentation conditions along the upper portion of the ramp are not expected to change significantly as a result of the project.

Between the wall and the bank, stream velocity is expected to increase compared to existing conditions. The existing bank is not sufficiently protected against erosion, so bank protection will be required between the wall and the boat ramp.

About 100 feet upstream of the structure, reductions in shear stress are expected along the right (west) bank. Additional sediment deposition may occur in this area; however, significant changes to the channel morphology or bank-erosion potential are not expected to occur.

Immediately east of the structure, increases in velocity and shear stress are expected as more of the flow is directed toward the center of the channel. In this location, additional bed scour is likely to occur.

A second technical memorandum was written for the project by WEST Consultants dated June 5, 2020 (*Hydraulic Analysis of Castle Rock Boat Launch Safety Improvements Project for Conditional Letter of Map Revision*; see Appendix B). It concluded that the slightly higher water surface elevations that would result from the proposed project will remain below the published effective elevations and recommends that a CLOMR not be developed for the project.

**Practicability Conclusion**

Alternatives evaluated that are not practicable include the no-action alternative and those that use natural materials. Of the 16 engineering alternatives reviewed, the preferred alternative is recommended by the engineers because it best balances the need to reduce flow velocities at the boat ramp and floats without significantly increasing sedimentation. Therefore, the preferred alternative of a two-panel wall at the proposed location is the practicable solution that best meets project criteria.

## BANK PROTECTION

The best practicable alternative that meets project criteria for the instream structure is the two-panel wall. This will require bank protection because the panel wall is expected to increase shear stress between the wall and the riverbank. Because of the increased shear stresses, additional bank protection will be necessary to protect the existing infrastructure west of the project that includes buildings, parking lot, boat ramp, and the proposed maintenance road that could be damaged if the bank failed.

One of the City's goals for this project is to provide an area for bank fishing. The existing heavily vegetated banks near the boat ramp do not allow for fishing on the bank because it is difficult to cast in heavy vegetation. The City of Castle Rock Shoreline Master Program states that recreation is an important shoreline use as stated in the City's Shoreline Master Program, so the City proposes to provide fishing access in this area that already has sufficient parking and restroom facilities.

# APPENDIX

# Technical Memo

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DRAFT

Expires 12/5/16

**To:** Tom Gower, P.E., City Engineer

**Company:** Gibbs & Olson, Inc.

**Date:** November 1, 2016

**Cc:** David Vorse, Public Works Director
City of Castle Rock, WA

**From:** Hans R. Hadley, P.E., CFM
Senior Hydraulic Engineer

**Subject:** Al Helenberg Boat Launch Velocity Reduction Structure Alternatives Analysis

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**Introduction**

The Al Helenberg Memorial Boat Launch was constructed in 2010 approximately 1,300 feet upstream of the State Route 411 Bridge to provide access to the Cowlitz River. A project location map is shown in **Figure 1** (all figures are provided in **Appendix A**). Boaters indicate that during larger wintertime river discharges, high streamflow velocities at the boat launch make use of the upstream lane of the ramp difficult or even dangerous. In an effort to address these concerns, the City of Castle Rock would like to implement a project that will reduce streamflow velocities at the ramp to improve both safety and access for boaters. At the same time, the City would like the sedimentation conditions at the ramp to not be made worse by the project (if possible). In support of this effort, WEST Consultants, Inc. (WEST) was contracted by the City Engineer, Gibbs & Olson, to perform hydraulic analyses of multiple alternative velocity reduction structures. The purpose of the evaluation is to understand the relative ability of each alternative to reduce streamflow velocities at the launch site and its potential effect on sedimentation conditions. The final selected alternative consists of three reinforced 30-ft long concrete panels supported by a combination of vertical and battered steel H-piles. The locations of the recommended panels are shown in **Figure 2**.

**Site Reconnaissance**

A site reconnaissance was conducted on August 30, 2016. Observations of the channel and floodplain area were made and documented with color photographs (**Appendix B**). The

Manning's 'n' roughness value for the channel is estimated to be 0.03. The Manning's n roughness value for the left (south) overbank is estimated to be 0.12. The Manning's n roughness values for the right (north) overbank are estimated to range between 0.04 and 0.12. Overbank roughness values were estimated based on the investigator's judgment and experience. It is recognized that the Cowlitz River transports a significantly large amount of easily transportable sand size material during high flow events. It is also recognized that changes in bedform morphology can occur with variations in flow. As flows increase the bedforms change from dunes to plane bed, resulting in lower 'n' values during significantly larger flows. Therefore, the channel roughness value was assumed to be slightly larger than the 0.025 value used by the Corps of Engineers' in their high flow HEC-RAS model.

Riprap with a median diameter of about 18-inches was observed along the toe of the banks upstream and downstream of the ramp. The extents of the riprap is unknown as there is significant sediment and vegetative cover. Riprap was also observed along the upstream face of the ramp. It is understood that this material was placed in this location to replace the material that had eroded during the first winter following the completion of the boat launch. Riprap was also observed along the left bank of the channel. It is understood that this was placed to provide erosion protection for the levee.

Sediment deposits were observed beneath the boarding floats. The median bed material size was observed to be coarse sand ($D_{50}$ = 1 mm). Sediment deposits were also observed along the banks adjacent to the ramp. However, the majority of this material appeared to have been recently side-cast as part of the ramp cleanup effort following the December 9, 2015 high water. Photos from both during and after the December 9, 2015 flood were provided by the City and are shown in **Appendix B** (Photos 13-16).

**Survey**

Bathymetric survey of the channel was conducted in August and September of 2016 by Gibbs & Olson. High density survey capable of supporting the development of a 2-dimensional model was collected from approximately 2,150 feet upstream to 900 feet downstream of the ramp. Four channel cross sections were also surveyed in the 900-foot reach immediately downstream of the high-density survey in order to provide additional data needed for the development of the 1-dimensional hydraulic model. The horizontal coordinate system for the survey is NAD 83 Washington State Plane South Zone, US Foot. The vertical datum for the survey is the North American Vertical Datum of 1988 (NAVD 88).

**Hydrology**

USGS Gage *142430000 Cowlitz River at Castle Rock, WA* is located approximately 1,400 feet downstream of the ramp at the Hwy 411 Bridge (A Street) and has a period of record of 90 years (1926 to present). Mean daily flow records are available for the prior 10 years (2006 – 2016) and mean stage are available for the prior years. Mean daily flow data for the 2006 – 2016 period were plotted and reviewed, and a flow duration curve developed (**Figure 3**). Based on review of the flow data, three flows were chosen to be simulated in the hydraulic models:

- 30,000 cfs – represents the approximate upper limit of usability of the ramp
- 9,000 cfs – represents a typical winter flow rate
- 5,000 cfs – represents a typical summer flow rate

**Sediment Transport**

Sediment transport conditions in the Cowlitz River are highly influenced by the delivery of sediment from the Toutle River, which flows into the Cowlitz River about 2.4 miles upstream of the project site. The Toutle River continues to deliver significant quantities of silt- and sand-sized sediment as a result of continued erosion of the debris avalanche created by the eruption of Mount Saint Helens in May 1980. The U.S. Army Corps of Engineers has continued to manage sediment in the Toutle River, most recently raising the spillway elevation at the Sediment Retention Structure to improve the sediment trapping efficiency. They have also dredged the lower portion of the Cowlitz River near the confluence with the Columbia River. However, significant quantities of sand-sized material will continue to be transported through the project reach as both bed load and suspended load.

Much of the fine sediment in the Cowlitz River is transported as suspended load during high flow events. As a result, sediment deposition occurs in areas of low velocity and low shear stress such as the areas along the banks, the inside of channel bends, and near obstructions to flow. During the December 2015 high water event (Q = 83,700 cfs), significant sedimentation occurred at the boat launch. The surface of the ramp was buried in as much as 2.5 feet of sand-sized material. Post-flood photographs indicate that some amount of the deposited sediment located near the base of the ramp was eroded as the river's discharge decreased. However, a significant amount of sediment remained in the immediate vicinity of the transverse floats, causing them to be partially grounded during low water conditions.

A significant portion of the sediment that was deposited at the ramp was likely conveyed by the river as suspended load. Therefore, significant changes in velocity and shear stress at and near the ramp location should be expected to affect the sedimentation conditions. The primary objective of this project is to increase boater safety by reducing velocities during high water conditions. Alternatives that result in significant velocity reductions and/or create an eddy would be expected to increase the rate of sediment deposition.

**1-Dimensional Hydraulic Modeling**

The primary purpose of the 1-dimensional model is to provide a starting downstream boundary condition for the 2-dimensional hydraulic model. This removes the expense of collecting additional high density survey data needed to extend the 2-dimensional model downstream to the Hwy 411 bridge. The 1-dimensional model can also be used in the future for developing a FEMA Conditional Letter of Map Revision (CLOMR) and Letter of Map Revision (LOMR) that would be required should the proposed project move forward to final design and construction.

HEC-RAS version 5.0.1 software (USACE, 2016) was used to develop an existing conditions steady state hydraulic model for the Cowlitz River in the vicinity of the project site. The upstream boundary of the model is located approximately 2,150 feet upstream of the ramp. The downstream boundary of the model is located approximately 1,350 feet downstream of the ramp and is coincident with USGS gage 142430000 at the downstream face of the highway 411 Bridge. As seen in Figure 1, a total of 15 cross sections are used in HEC-RAS to represent the geometry of the channel and floodplains. The cross-section geometry is based on the Gibbs & Olson survey within the channel and LiDAR data from the U.S Army Corps of Engineers for the overbanks. The downstream boundaries for the three simulated flows are specified as known water surface elevations based on rating curve data from the USGS gage.

**Two-Dimensional Modeling**

Since the purpose of the project is to modify flow velocities, evaluation and development of the various conceptual designs requires detailed information about the effects of the structures on local flow dynamics. A 2-dimensional hydraulic model was developed to evaluate potential changes in the magnitude and direction of flows and magnitude of shear stresses in the vicinity of the ramp.

The two-dimensional hydrodynamic software modeling program Sedimentation and River Hydraulics – Two-Dimensional (SRH-2D) Version 3.1.1 (dated July 2016), developed by the US Bureau of Reclamation (USBR), was used to simulate the hydraulic conditions of the Cowlitz River near the project site.

The model mesh was developed using the Surface-water Modeling System (SMS) Version 11.2.9 (SMS) developed by Aquaveo (2015). Model development involved the following steps:

1. Development of a conceptual model using arcs (polylines) to parse the modeled area into multiple zones defined by unique characteristics such as land use, Manning's n hydraulic roughness value, and specific project sites.
2. Assignment of mesh node spacing for each zone. The mesh node spacing varies significantly within the computational domain depending on the resolution required, with larger spacing in the floodplain and significantly smaller spacing in the channel where more detailed model output is required. Spacing ranges from 5 feet near the project site to 50 feet along the periphery of the floodplain.
3. Interpolation of topographic data points to the mesh. Topographic data in the SRH-2D model are based on the DTM developed for the project area.
4. Assignment of a downstream boundary condition. A water surface elevation boundary condition was assigned in SRH-2D that was equal to the water surface elevation at Cross Section 479 in the one-dimensional HEC-RAS model for each evaluated flow.
5. Pre-processing of model input data (mesh, inflow and outflow parameters, monitor lines, simulation times, output intervals) using the SRH-2D pre-processor to create the input files for the model.

**Table 1** shows the Manning's n values for each land use type specified in the SRH-2D model.

**Table 1 –Manning's n Values**

| Land Use Type | Manning's n Value |
|---|---|
| River | 0.03 |
| Field/Open | 0.04 |
| Pavement | 0.015 |
| Dense Residential/Commercial | 0.12 |
| Forest | 0.1 |
| Rural Residential | 0.08 |

An existing conditions model was initially developed to provide a basis for comparison of the potential alternatives. It should be noted that the existing piles and floating logs used to deflect debris away from the ramp are not represented in the model. Based on the simulated flow depths and velocities, computational limitations of the model prohibit representation of very small features such as individual piles, without introducing model instabilities. As the piles currently represent a very small blockage to flow relative to the entire channel cross section it is assumed that they have very limited impact on flow characteristics and that the existing conditions model is a good representation of existing flow patterns. The existing conditions model was then modified to represent 16 conceptual variations of potential velocity reduction structures. The design variations are all based on the general premise of steel H-piles driven into the stream bed which would be used to support precast reinforced concrete panels between the H-piles. The panels would be keyed into the channel bed and would have top elevations of 43.0 ft, which is approximately 1 foot above the 30,000 cfs flow elevation. The design variations ranged in location from immediately upstream of the ramp to a point approximately 300 feet upstream of the ramp (approximately 200 feet upstream of the existing debris deflector). The designs also considered various alignments, solid vs. discontinuous panels, and various panel and total structure lengths. The use of a single structure vs. multiple structures in tandem was also evaluated.

Based on review of the model output from the 16 modeled alternatives, Concept 7d is considered the preferred alternative. Concept 7d, located approximately 300 feet upstream of the ramp, is approximately 150 feet long, angled approximately 45 degrees to the channel bank in a downstream direction, and consists of three 30-foot long panels with 30-foot spacing between the panels (**Figure 2**).

**Results**

Simulated velocities for the 30,000 cfs, 9,000 cfs, and 5,000 cfs flows under existing conditions are provided in **Figure 4**, **Figure 5**, and **Figure 6**, respectively. Simulated shear stresses for the 30,000 cfs flow under existing conditions are shown in **Figure 7**. Simulated velocities for the 30,000 cfs, 9,000 cfs, and 5,000 cfs flows for the preferred alternative are provided in **Figure 8**, **Figure 9**, and **Figure 10**, respectively. Simulated shear stresses for the 30,000 cfs flow for the preferred alternative are shown in **Figure 11**. The 2-D model results indicate that the preferred alternative would reduce velocities near the end of the boarding floats from 5 ft/s to 3 ft/s for the 30,000 cfs flow. Velocities at the end of the boarding floats would be reduced from 2.3 ft/s to 1.3 ft/s for the 9,000 cfs flow and velocities would remain approximately the same for the 5,000 cfs flow. The reduction in velocities is favorable from a hazard perspective and would provide a safer ingress/egress zone for boaters, particularly during higher flow conditions.

Model results indicate that the proposed structure will not create an eddy which, as previously mentioned, would likely exacerbate existing sedimentation issues. However, it is noted that shear stresses would be reduced downstream of the proposed structure which may result in an increase in sedimentation during high flow events when the Cowlitz River is transporting a significant suspended sediment load. Incipient motion calculations using Shield's equation suggests that the coarse sand (1 mm) is mobilized for shear stress values greater than 0.02 lb/ft$^2$. The existing conditions model results indicates that shear stress values near the base of the ramp and the transverse floats are about 0.27 lb/ft$^2$ and 0.32 lb/ft$^2$, respectively for a discharge of 30,000 cfs. The preferred alternative model results indicate that the shear stress values near the base of the ramp and the transverse floats are about 0.08 lb/ft$^2$ and 0.18 lb/ft$^2$, respectively for a discharge of 30,000 cfs. For both existing conditions and the preferred alternative, shear stress values at the upper portion of the ramp are less than 0.01 lb/ft$^2$. Sedimentation conditions along the upper portion of the ramp are not expected to change significantly as a result of the project. Although the shear stress values will be greater than required to transport coarse sand-sized material, sediment deposition rates for the area near the base of the ramp and the transverse floats are likely to increase for the proposed alternative compared to existing conditions due to the excessive supply of sediment delivered by the Toutle River. Periodic sediment removal will likely be required.

The proposed project is expected to change the flow directions and velocities in the immediate vicinity of the structure. As seen in **Figure 8**, the velocity along the bank is expected to increase compared to the existing conditions. This area (approximately 100 ft long) may require bank protection if the existing bank is not sufficiently protected. Bank protection would likely be in the form of a riprap revetment or combination of riprap toe and vegetation. Additional reconnaissance for this location is recommended to determine the adequacy of the bank material to resist erosion.

The portion of the proposed structure located furthest from the bank will create a zone of lower velocity immediately downstream. This area is likely to accumulate sediment over time. However, the structure was location 300 ft upstream of the ramp to lessen the chances that the sediment deposition in this low velocity zone would extend to the ramp location.

Minor reductions in shear stress occur along the right (west) bank for a distance of about 100 ft upstream of the structure. Additional sediment deposition may occur in this area. Shear stress values for areas further upstream were not significantly changed by the proposed project. Significant changes to the channel morphology or bank erosion potential are not expected to occur for this area.

Increases in velocity and shear stress are expected to occur immediately east of the structure as more of the flow is directed toward the center of the channel. In this location, additional scour of the channel bed is likely to occur.

The proposed project is located within a regulatory FEMA floodplain and floodway. According to FEMA regulations, the project should not cause a rise in the regulatory floodplain and floodway elevations. A complete no-rise hydraulic analysis based on the FEMA Base Flood was not conducted as part of the alternatives analysis detailed in this memo; however, the modeling conducted for the alternatives analysis indicates that that the proposed alternative is likely to increase water surface elevations for the Base Flood. A no-rise analysis using FEMA methodology will need to be conducted for the chosen alternative. If a rise is shown to occur, the project will require a Conditional Letter of Map Amendment (CLOMR) to be submitted by the City to FEMA prior to implementation of the project. Following completion of the project, a Letter of Map Revision (LOMR) will need to be submitted by the City to FEMA.

If you have any questions, please do not hesitate to contact me at 503-485-5490.

# APPENDIX A
# FIGURES

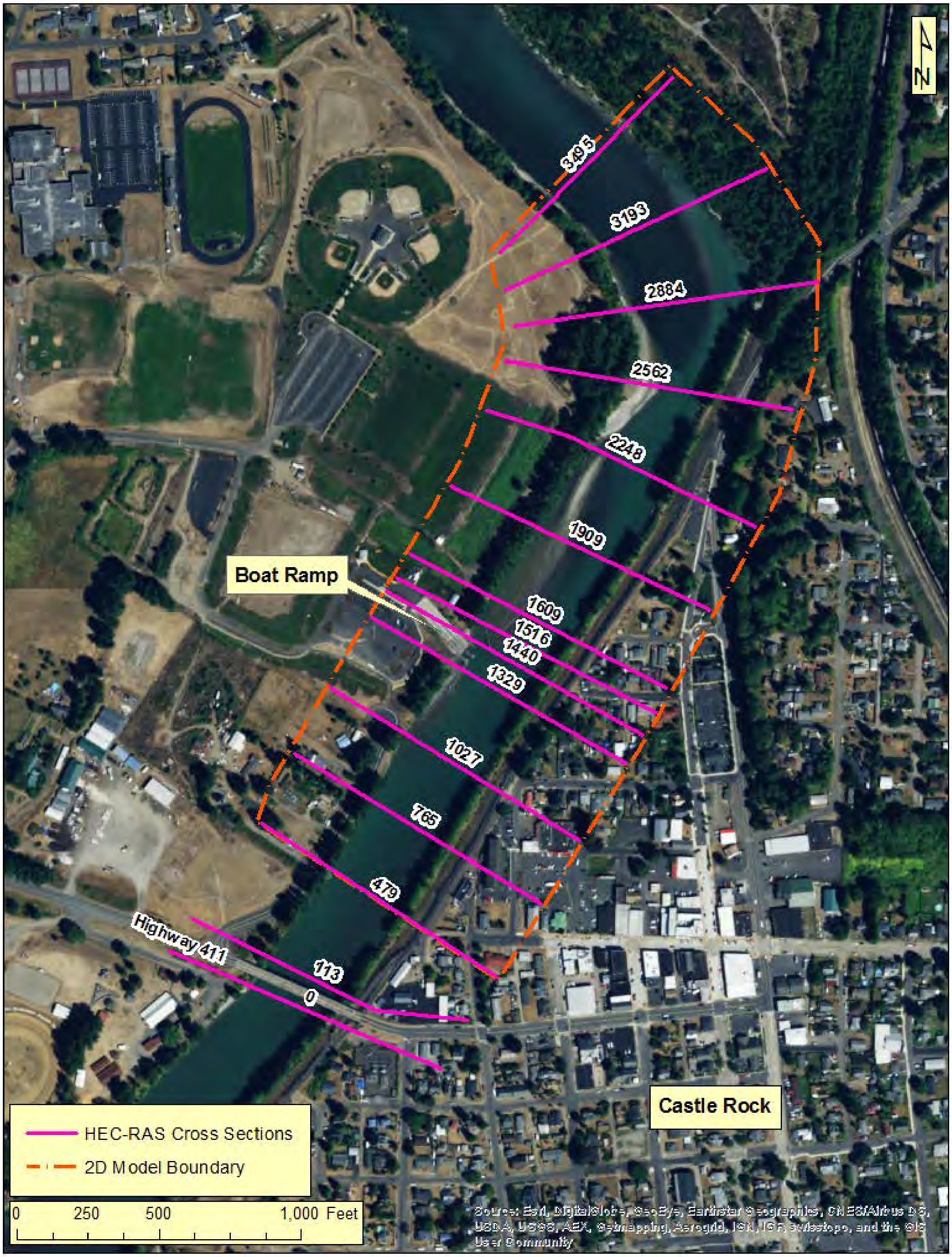
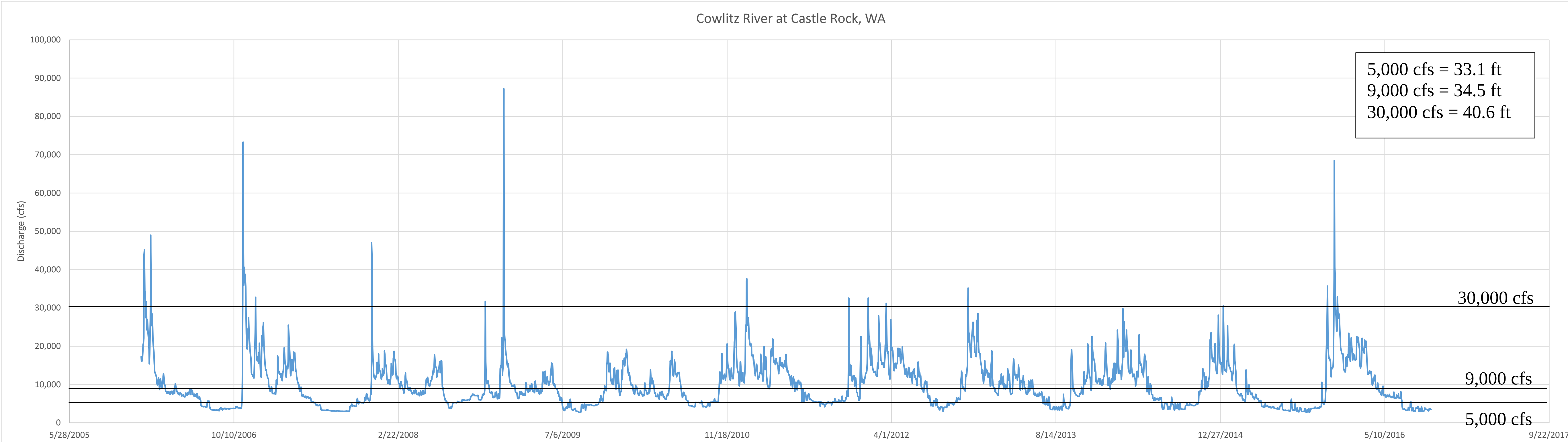



**Figure 1. Location Map**


Figure 2. Preferred Alternative (7d)



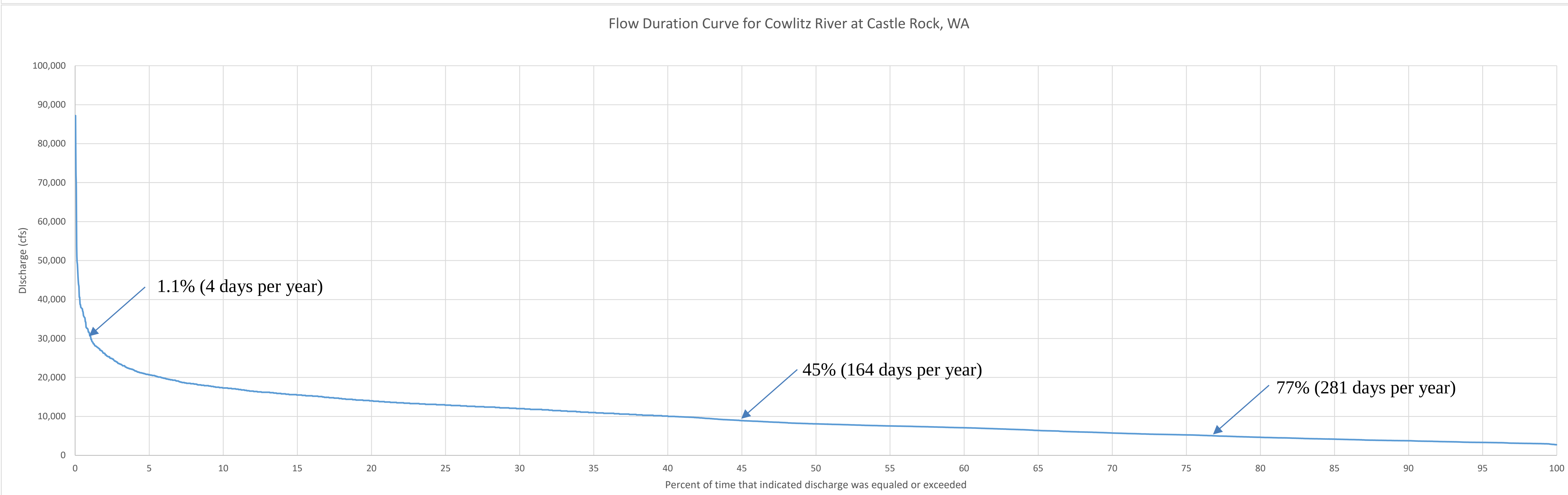




**Figure 3. Mean daily flows from 2006-2016 and associated flow duration curve**

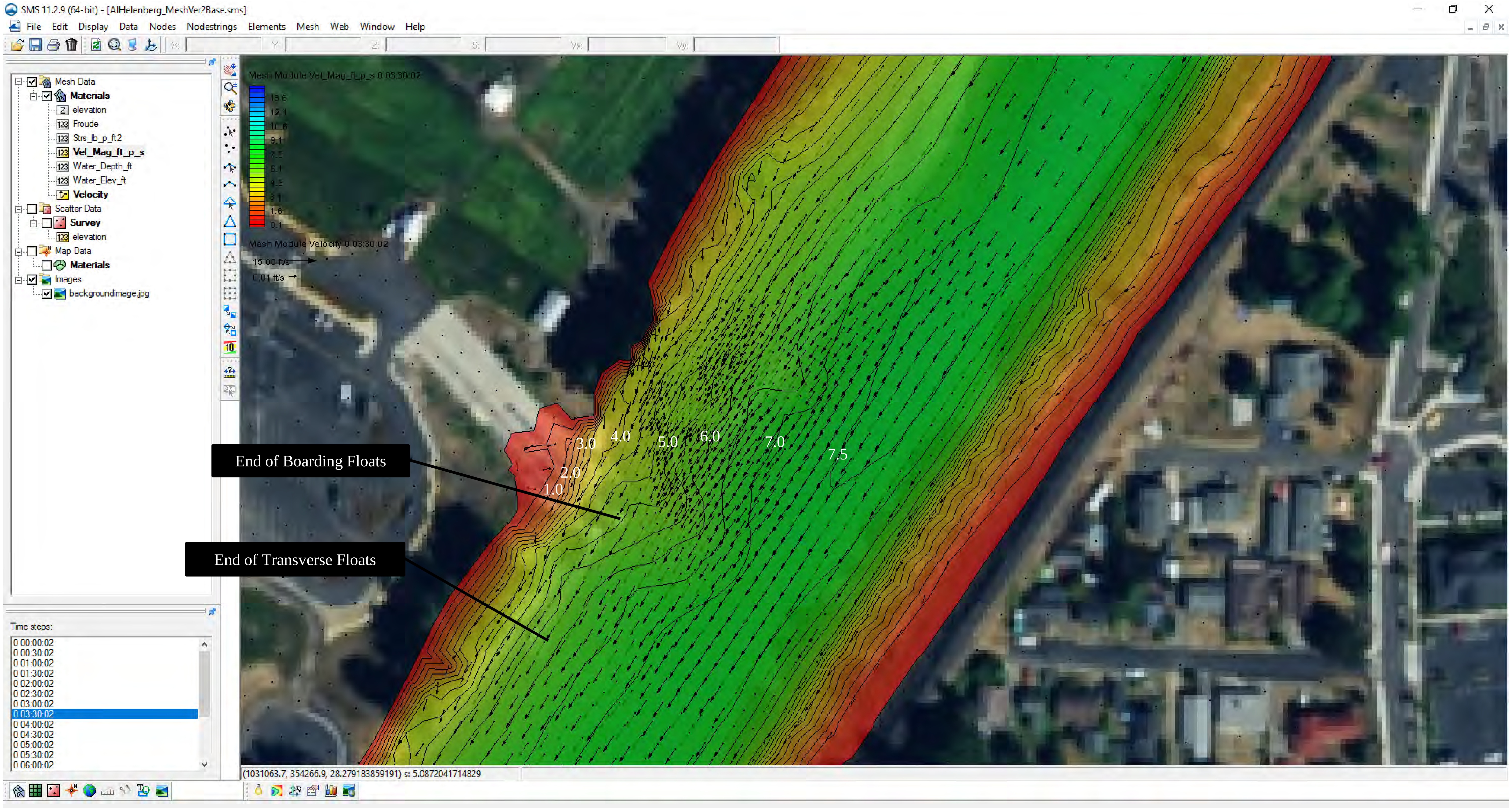



Figure 4. Existing conditions velocities (30,000 cfs)


**Figure 5. Existing conditions velocities (9,000 cfs)**


**Figure 6. Existing conditions velocities (5,000 cfs)**

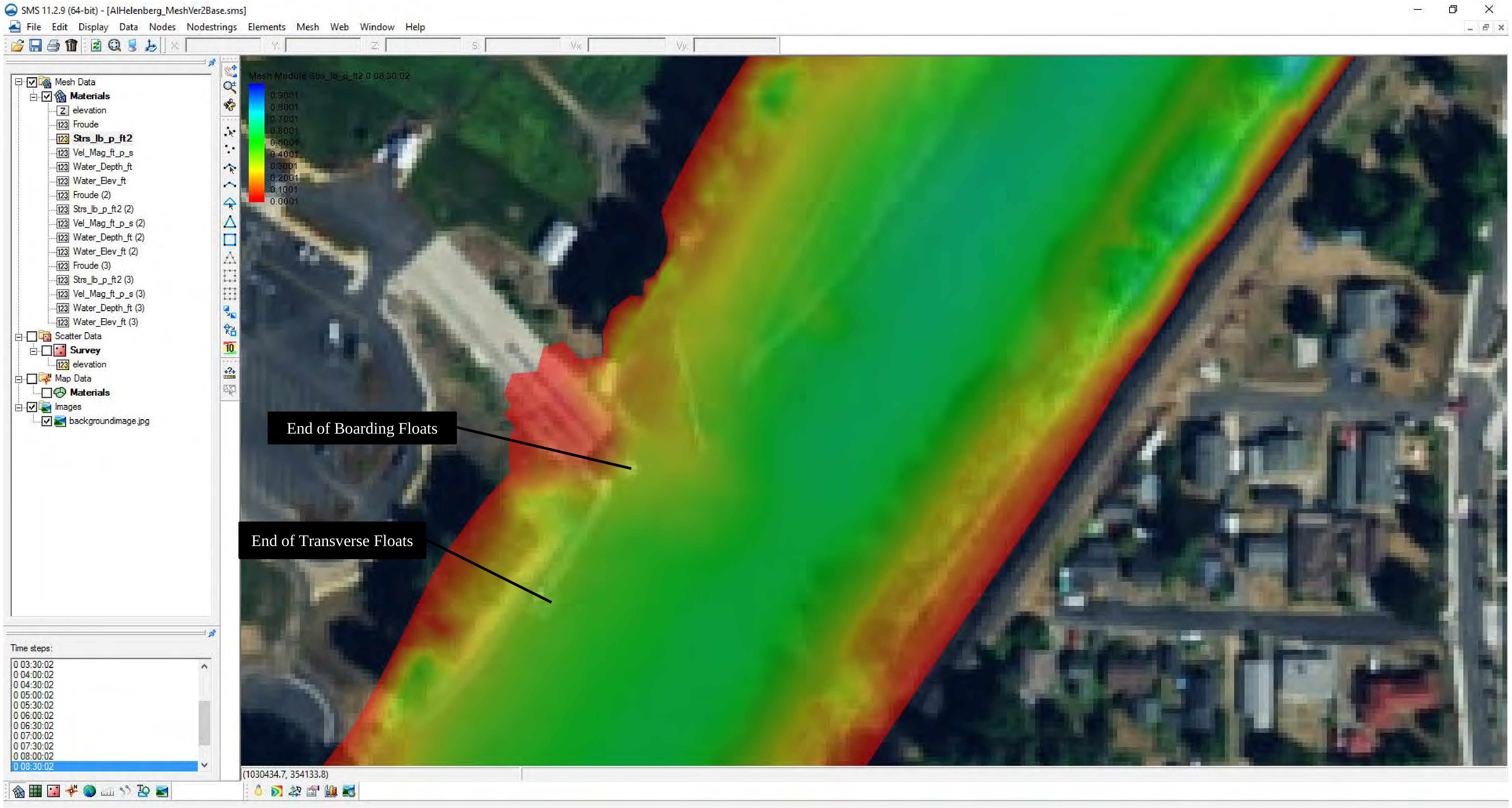



Figure 7. Existing conditions shear stresses (30,000 cfs)


Figure 8. Preferred Alternative (7d) velocities (30,000 cfs)


Figure 9. Preferred Alternative (7d) velocities (9,000 cfs)


**Figure 10. Preferred Alternative (7d) velocities (5,000 cfs)**

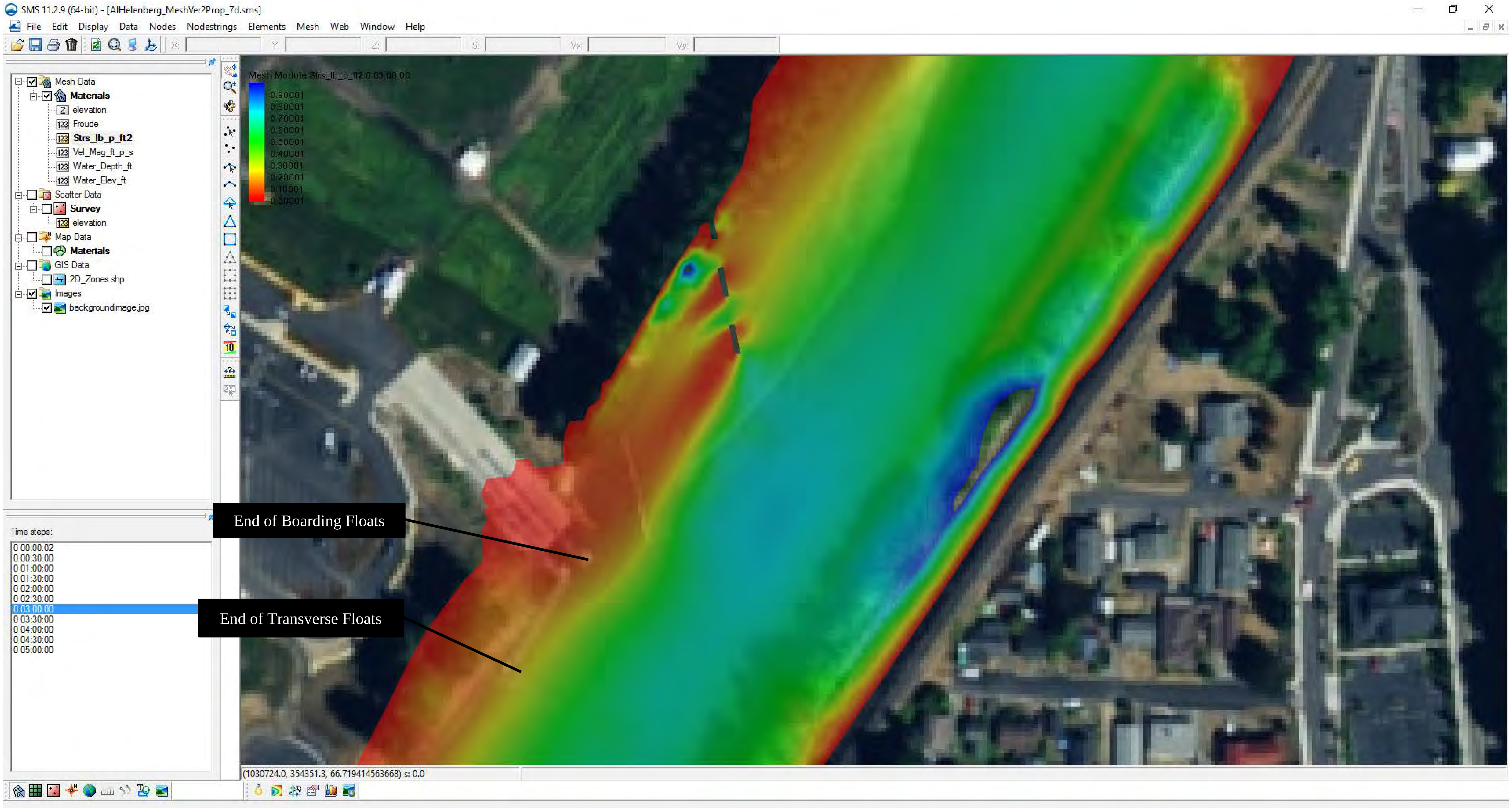



Figure 11. Preferred Alternative (7d) shear stresses (30,000 cfs)

# APPENDIX B
# PHOTOGRAPHIC LOG

Photo 1 – View of boat ramp from parking area

Photo 2 – View from top of ramp

Photo 3 – Riprap protection added after Dec 2015 high water

Photo 4 – Looking upstream along left bank


Photo 5 – Pile and log debris deflector


Photo 6 – Boarding floats (foreground) and transverse float (background)


Photo 7 – Boarding floats and marks from sediment deposition


Photo 8 – Looking upstream from end of boarding floats


Photo 9 – Looking downstream along transverse floats


Photo 10 – looking at base of ramp and debris from Dec 2015 high water


Photo 11 - Looking downstream at Hwy 411 bridge


Photo 12 – riprap protection along downstream bank

Photo 13 – Sediment deposits from December 9, 2015 high water

Photo 14 – Sediment deposits from December 9, 2015 high water


Photo 15 – December 9, 2015 high water

Photo 16 – Debris on floats during December 9, 2015 high water

# Technical Memo

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Expires 12/5/2020

**Name:** Tom Gower, PE, Project Manager

**Company:** Gibbs & Olson

**Date:** June 5, 2020

**From:** Hans Hadley, PE, CFM, Senior Hydraulic Engineer

**Subject:** Hydraulic Analysis of Castle Rock Boat Launch Safety Improvements Project for Conditional Letter of Map Revision

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## Introduction

WEST Consultants, Inc. (WEST) previously conducted a hydraulic alternatives analysis for a proposed boat launch velocity reduction structure to be located along the Cowlitz River in Castle Rock, WA (Cowlitz County). The City of Castle Rock has selected a preferred alternative and wishes to move forward with the design and permitting for the project. The project site is located approximately 300 feet upstream of the Al Helenberg Memorial Boat Launch located on the right (west) bank of the Cowlitz River which is located approximately at River Mile 17. The project is located in a FEMA regulatory floodway. As such, either a no-rise condition needs to be achieved or a Conditional Letter of Map Revision (CLOMR) should be submitted to FEMA. The proposed project site is shown in **Figure 1** (all figures provided in **Appendix A**). The design drawing for the project is provided in **Figure 2**. For consistency with the existing FEMA Flood Insurance Study, all elevations in this document are based on the NAVD 88 vertical datum, unless otherwise stated. All elevation data that were provided in NGVD 29 were converted to NAVD 88 by adding 3.36 ft, which is consistent with the effective Cowlitz County Flood Insurance Study (FIS) (FEMA 2015).

## Hydrology

The 1-percent annual chance flood discharge for the project reach is 97,000 cfs. This value was obtained from the effective FEMA HEC-RAS hydraulic model and is consistent with the hydrology published in the effective FIS. Flows at Castle Rock are partially regulated by Mossy Rock Dam.

## Hydraulics

Duplicate Effective Model (DEM)

A Duplicate Effective Model (DEM) was developed for the Cowlitz River using a HEC-RAS 1-dimension steady state hydraulic model. The starting water surface elevations for the downstream boundary are based on the effective FEMA HEC-RAS hydraulic model results for RS 0.0 (FEMA Station 17.15 (Cross Section AQ)). A comparison of the DEM and the effective base flood water surface elevations are provided in **Table 1**. No issues were found in the DEM that would warrant the development of a Corrected Effective Model (CEM). As seen in the table, the water surface elevations are less than the 0.1 ft allowable difference between the effective model and the DEM. It should be noted that, within the project reach, there were four interpolated cross sections in the effective HEC-RAS model.

**Table 1. Comparison of Effective Model and DEM base flood water surface elevations**

| Effective River Station | DEM River Station | FEMA XS Letter | Regulatory | | | Floodway | | |
|---|---|---|---|---|---|---|---|---|
| | | | Effective (ft) | DEM (ft) | Difference (ft) | Effective (ft) | DEM (ft) | Difference (ft) |
| 17.15 | 0 | AQ | 54.28 | 54.28 | 0.00 | 54.48 | 54.48 | 0.00 |
| 17.2125 | 430 | -- 1/ | 54.22 | 54.22 | 0.00 | 54.42 | 54.42 | 0.00 |
| 17.275 | 860 | -- 1/ | 54.24 | 54.24 | 0.00 | 54.44 | 54.44 | 0.00 |
| 17.3375 | 1290 | -- 1/ | 54.85 | 54.85 | 0.00 | 55.02 | 55.03 | +0.01 |
| 17.4 | 1727 | AR | 55.55 | 55.55 | 0.00 | 55.70 | 55.71 | +0.01 |
| 17.475 | 2107 | -- 1/ | 54.89 | 54.89 | 0.00 | 55.06 | 55.06 | 0.00 |

1. Interpolated cross section in effective HEC-RAS model

Existing Conditions Model (ECM)

An Existing Conditions Model (ECM) was developed by updating the DEM to include additional cross section data for the project site and updating the existing cross section data with newer data. Hydraulic cross section locations for the ECM are shown in **Figure 3**. Cross sections for the ECM were developed from available LiDAR topography (WSI, 2010) and bathymetry data collected in 2016 and 2019 by Gibbs & Olson, Inc. Floodway encroachment stations for the new cross sections were obtained from the floodway boundary provided in the FEMA DFIRM database. The ECM results for the base flood are provided in **Table 2**.

**Table 2. Comparison of DEM and ECM base flood water surface elevations**

| Effective River Station | ECM River Station | FEMA XS Letter | Regulatory | | | Floodway | | |
|---|---|---|---|---|---|---|---|---|
| | | | DEM (ft) | ECM (ft) | Difference (ft) | DEM (ft) | ECM (ft) | Difference (ft) |
| 17.15 | 0 | AQ | 54.28 | 54.28 | 0.00 | 54.48 | 54.48 | 0.00 |
| 17.2125 | 430 | -- [1,2/] | 54.22 | 54.53 | +0.31 | 54.42 | 54.71 | +0.29 |
| 17.275 | 860 | -- [1,2/] | 54.24 | 54.62 | +0.38 | 54.44 | 54.80 | +0.36 |
| -- | 1190 [3/] | -- | -- | 54.91 | -- | -- | 55.07 | -- |
| -- | 1247 [3/] | -- | -- | 54.94 | -- | -- | 55.11 | -- |
| 17.3375 | 1290 | -- [1,2/] | 54.85 | 54.97 | +0.12 | 55.03 | 55.13 | +0.10 |
| -- | 1349 [3/] | -- | -- | 54.96 | -- | -- | 55.12 | -- |
| 17.4 | 1727 | AR | 55.55 | 55.12 | -0.43 | 55.71 | 55.28 | -0.43 |
| 17.475 | 2107 | -- [1,2/] | 54.89 | 55.06 | +0.17 | 55.06 | 55.07 | +0.01 |

1. Interpolated cross section in effective HEC-RAS model
2. Geometry updated with new survey data
3. New cross section in project location (floodway encroachments based on mapped floodway boundary)

As seen in the table, the differences between the regulatory water surface elevations for the DEM and ECM range between -0.43 ft and +0.38 ft and the differences between the floodway water surface elevations range between -0.43 ft and +0.36 ft. These differences are attributed to the updated cross section geometry used in the ECM.

<u>Proposed Conditions Model (PCM)</u>

A Proposed Conditions Model (PCM) was developed by updating the ECM to include the two proposed velocity reduction structures. Hydraulic cross section locations for the PCM are the same as the ECM (see **Figure 3**). A comparison of the cross section geometry for the ECM and PCM is provided in **Appendix B**. Results for the PCM and ECM for with- and without-floodway are provided in **Table 3**. As seen in the table, a small rise in the water surface occurs upstream of the project site. Therefore, the project does not technically achieve a no-rise condition.

**Table 3. Comparison of ECM and PCM base flood water surface elevations**

| Effective River Station | PCM River Station | FEMA XS Letter | Regulatory | | | Floodway | | |
|---|---|---|---|---|---|---|---|---|
| | | | ECM (ft) | PCM (ft) | Difference (ft) | ECM (ft) | PCM (ft) | Difference (ft) |
| 17.15 | 0 | AQ | 54.28 | 54.28 | 0.00 | 54.48 | 54.48 | 0.00 |
| 17.2125 | 430 | -- [1,2] | 54.53 | 54.53 | 0.00 | 54.71 | 54.71 | 0.00 |
| 17.275 | 860 | -- [1,2] | 54.62 | 54.62 | 0.00 | 54.80 | 54.80 | 0.00 |
| -- | 1190 [3] | -- | 54.91 | 54.91 | 0.00 | 55.07 | 55.07 | 0.00 |
| -- | 1247 [3,4] | -- | 54.94 | 54.87 | -0.07 | 55.11 | 55.04 | -0.07 |
| 17.3375 | 1290 [4] | -- [1,2] | 54.97 | 54.89 | -0.08 | 55.13 | 55.06 | -0.07 |
| -- | 1349 [3] | -- | 54.96 | 54.99 | +0.03 | 55.12 | 55.15 | +0.03 |
| 17.4 | 1727 | AR | 55.12 | 55.15 | +0.03 | 55.28 | 55.31 | +0.03 |
| 17.475 | 2107 | -- [1,2] | 55.06 | 55.09 | +0.03 | 55.07 | 55.10 | +0.03 |

1. Interpolated cross section in effective HEC-RAS model
2. Geometry updated with new survey data
3. New cross section in project location (floodway encroachments based on mapped floodway boundary)
4. Cross section geometry modified to represent proposed conditions

## Results

**Table 4** summarizes the water surface elevation differences between the effective model and the PCM. As seen in the table, the results for the PCM do indicate that the project will cause a small rise in the water surface elevations compared to the effective model. These small increases only occur at unpublished cross sections. However, if a CLOMR were to be developed from the PCM results, it would show a decrease in the effective regulatory and floodway water surface elevations for Cross Section AR. As seen in **Figure 4**, the effective regulatory and floodway water surface elevations are currently 55.6 ft and 55.7 ft, respectively. For a CLOMR, the revised regulatory and floodway water surface elevations would be 55.2 ft and 55.4 ft, respectively. This decrease is due to the use of updated hydrographic and topographic data for the ECM. The DEM, ECM, and PCM model output is provided in **Appendix C**.

**Table 4. Comparison of Effective Model and PCM base flood water surface elevations**

| Effective River Station | ECM River Station | FEMA XS Letter | Regulatory | | | Floodway | | |
|---|---|---|---|---|---|---|---|---|
| | | | Effective (ft) | PCM (ft) | Difference (ft) | Effective (ft) | PCM (ft) | Difference (ft) |
| 17.15 | 0 | AQ | 54.28 | 54.28 | 0.00 | 54.48 | 54.48 | 0.00 |
| 17.2125 | 430 | -- 1,2/ | 54.22 | 54.53 | +0.31 | 54.42 | 54.71 | +0.29 |
| 17.275 | 860 | -- 1,2/ | 54.24 | 54.62 | +0.38 | 54.44 | 54.80 | +0.36 |
| -- | 1190 3/ | -- | -- | 54.91 | -- | -- | 55.07 | -- |
| -- | 1247 3,4/ | -- | -- | 54.87 | -- | -- | 55.04 | -- |
| 17.3375 | 1290 4/ | -- 1,2/ | 54.85 | 54.89 | +0.04 | 55.02 | 55.06 | +0.04 |
| -- | 1349 3/ | -- | -- | 54.99 | -- | -- | 55.15 | -- |
| 17.4 | 1727 | AR | 55.55 | 55.15 | -0.40 | 55.70 | 55.31 | -0.39 |
| 17.475 | 2107 | -- 1,2/ | 54.89 | 55.09 | +0.20 | 55.06 | 55.10 | +0.04 |

1. Interpolated cross section in effective HEC-RAS model
2. Geometry updated with new survey data
3. New cross section in project location (floodway encroachments based on mapped floodway boundary)
4. Cross section geometry modified to represent proposed conditions

Further, the flood profile developed from the PCM that would be used for a CLOMR would similarly show lower water surface elevations compared to the effective flood profile. As seen in **Table 5** and **Figure 5**, the flood profile developed from the PCM is lower in elevation than the effective flood profile over the entire project reach. It appears that the effective flood profile did not use all of the output data for the effective hydraulic model. As a result, the CLOMR flood profile that would be developed from the PCM would end up lowering the water surface elevations that are published in the FIS. In other words, the published effective water surface elevations along the project reach are conservatively higher than those that would be utilized for the CLOMR. Therefore, it is recommended that a CLOMR not be developed for the project and that FEMA Region 10 be made aware of the issue with the effective FEMA study. Again, the slightly higher water surface elevations that would result from the proposed project will remain below the published effective elevations. Also, no structures are located with the regulatory floodplain for this reach of the Cowlitz River.

**Table 5. Comparison of Effective Flood Profile with Effective Model and PCM base flood regulatory water surface elevations**

| Flood Profile Station | Effective River Station | ECM River Station | FEMA XS Letter | Regulatory | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | Effective Profile (ft) | Effective Model (ft) | Difference (ft) | PCM (ft) | Difference (ft) |
| 89,379 | 17.15 | 0 | AQ | 54.3 | 54.28 | -0.02 | 54.28 | -0.02 |
| 89,809 | 17.2125 | 430 | -- [1,2/] | 54.6 | 54.22 | -0.38 | 54.53 | -0.07 |
| 90,239 | 17.275 | 860 | -- [1,2/] | 54.9 | 54.24 | -0.66 | 54.62 | -0.28 |
| 90,569 | -- | 1190 [3/] | -- | 55.1 | -- | -- | 54.91 | -- |
| 90,626 | -- | 1247 [3,4/] | -- | 55.2 | -- | -- | 54.87 | -- |
| 90,669 | 17.3375 | 1290 [4/] | -- [1,2/] | 55.2 | 54.85 | -0.35 | 54.89 | -0.31 |
| 90,728 | -- | 1349 [3/] | -- | 55.3 | -- | -- | 54.99 | -- |
| 91,106 | 17.4 | 1727 | AR | 55.6 | 55.55 | -0.05 | 55.15 | -0.45 |
| 91,486 | 17.475 | 2107 | -- [1,2/] | 55.5 | 54.89 | -0.61 | 55.09 | -0.41 |

5. Interpolated cross section in effective HEC-RAS model
6. Geometry updated with new survey data
7. New cross section in project location (floodway encroachments based on mapped floodway boundary)
8. Cross section geometry modified to represent proposed conditions

## References


Federal Emergency Management Agency (FEMA). *Flood Insurance Study, Cowlitz County, Washington and Incorporated Areas, Volumes 1 - 3.* Flood Insurance Study Number 53015CV001A, 53015CV002A, and 53015CV003A. December 16, 2015.

Watershed Sciences, Inc. (WSI). *LiDAR Remote Sensing Data Collection: Columbia River Survey, Delivery 1*. Prepared for the US Army Corps of Engineers Portland District and obtained from DOGAMI (https://gis.dogami.oregon.gov/maps/lidarviewer/), 2010.

# APPENDIX A
# FIGURES

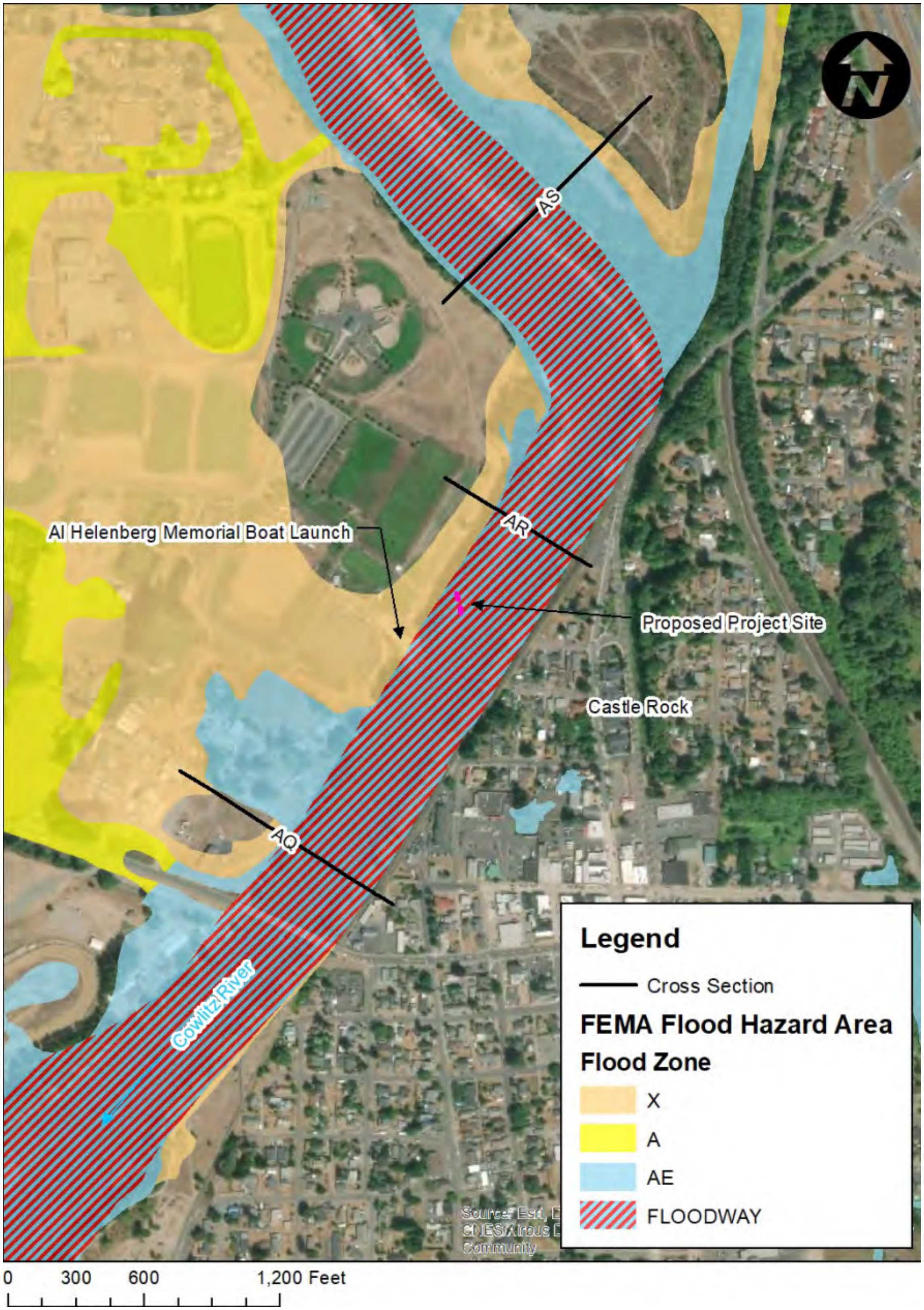
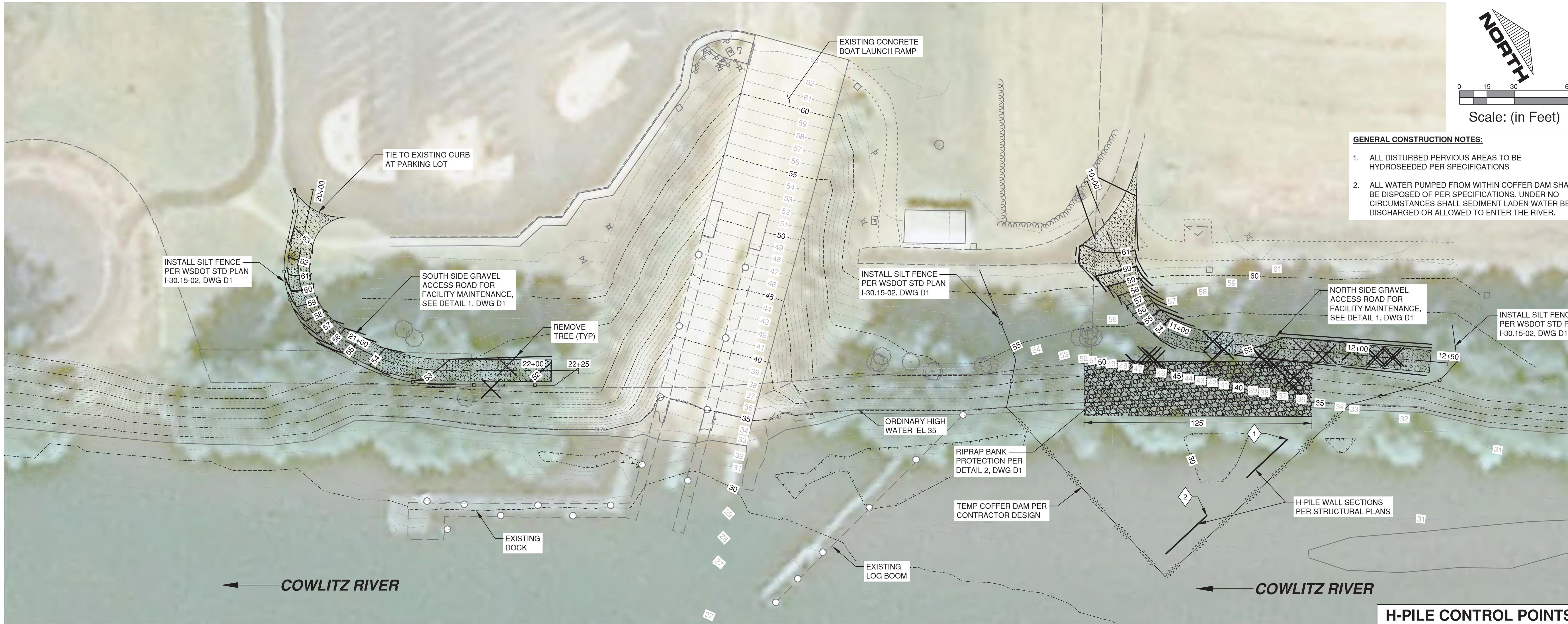


Figure 1. Project Location Map

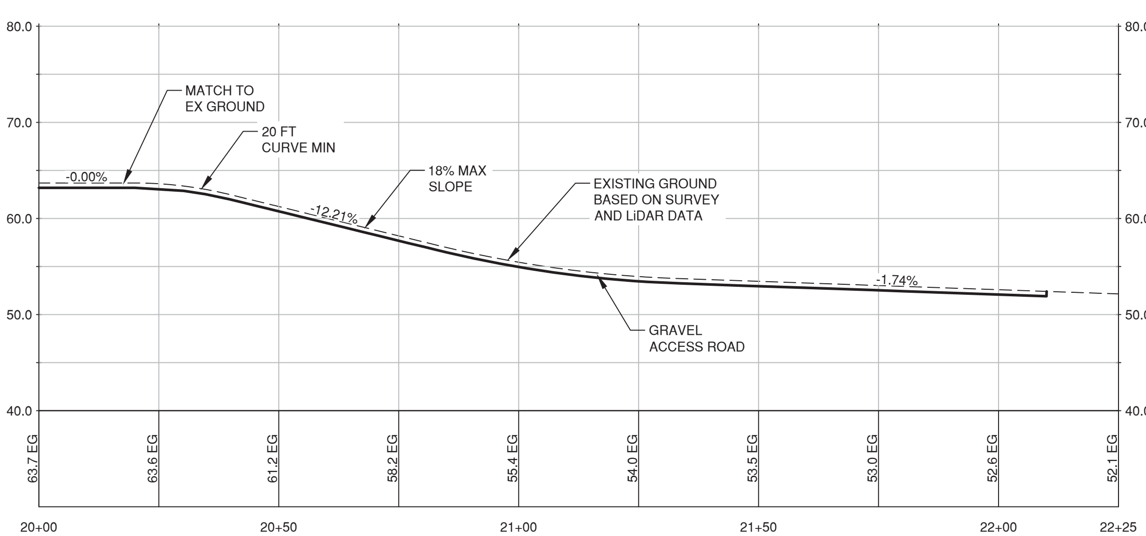



**GENERAL CONSTRUCTION NOTES:**

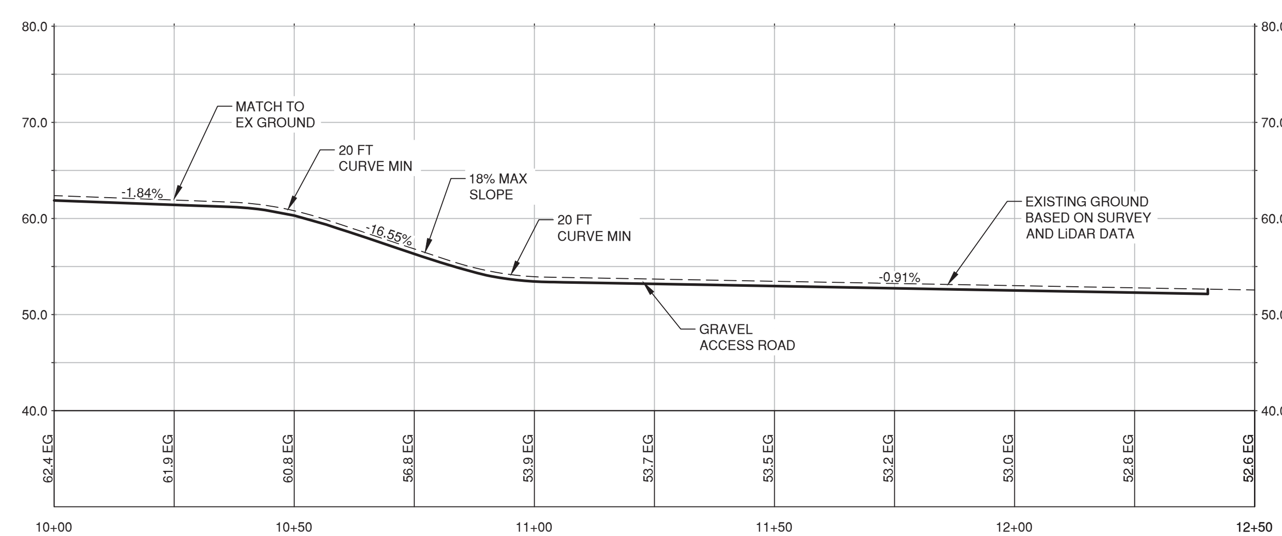
1. ALL DISTURBED PERVIOUS AREAS TO BE HYDROSEEDED PER SPECIFICATIONS
2. ALL WATER PUMPED FROM WITHIN COFFER DAM SHALL BE DISPOSED OF PER SPECIFICATIONS. UNDER NO CIRCUMSTANCES SHALL SEDIMENT LADEN WATER BE DISCHARGED OR ALLOWED TO ENTER THE RIVER.

**H-PILE CONTROL POINTS**

| POINT NO | NORTHING | EASTING | ANGLE |
|---|---|---|---|
| 1 | 354134.53 | 1030918.92 | S11°4'13"E |
| 2 | 354074.89 | 1030930.98 | S11°4'13"E |




## SOUTH ACCESS ROAD




## NORTH ACCESS ROAD

**RIGHT-OF-WAY DISCLAIMER**

THE RIGHT-OF-WAY AND/OR PROPERTY LINES SHOWN HEREON ARE BASED ON AVAILABLE INFORMATION, NOT ON A SURVEYED LOCATION AND ARE ONLY APPROXIMATE

Civil
Site Plan
----

City of Castle Rock
Boat Launch Safety Improvements
Castle Rock, Washington

Datum: NAD88-2011 NAVD 88

Survey Book: ----

Project Milestone:
**PRELIMINARY**

Date:
**03/27/2020**

THOMAS E. G... STATE OF WASHINGTON REGISTERED PROFESSIONAL ENGINEER
FOR REVIEW
03/27/2020

GIBBS & OLSON
www.gibbs-olson.com

Project Manager: TEG
Designed by: TEG
CAD by: KAR
Checked by: JJJ
Approved by: TEG

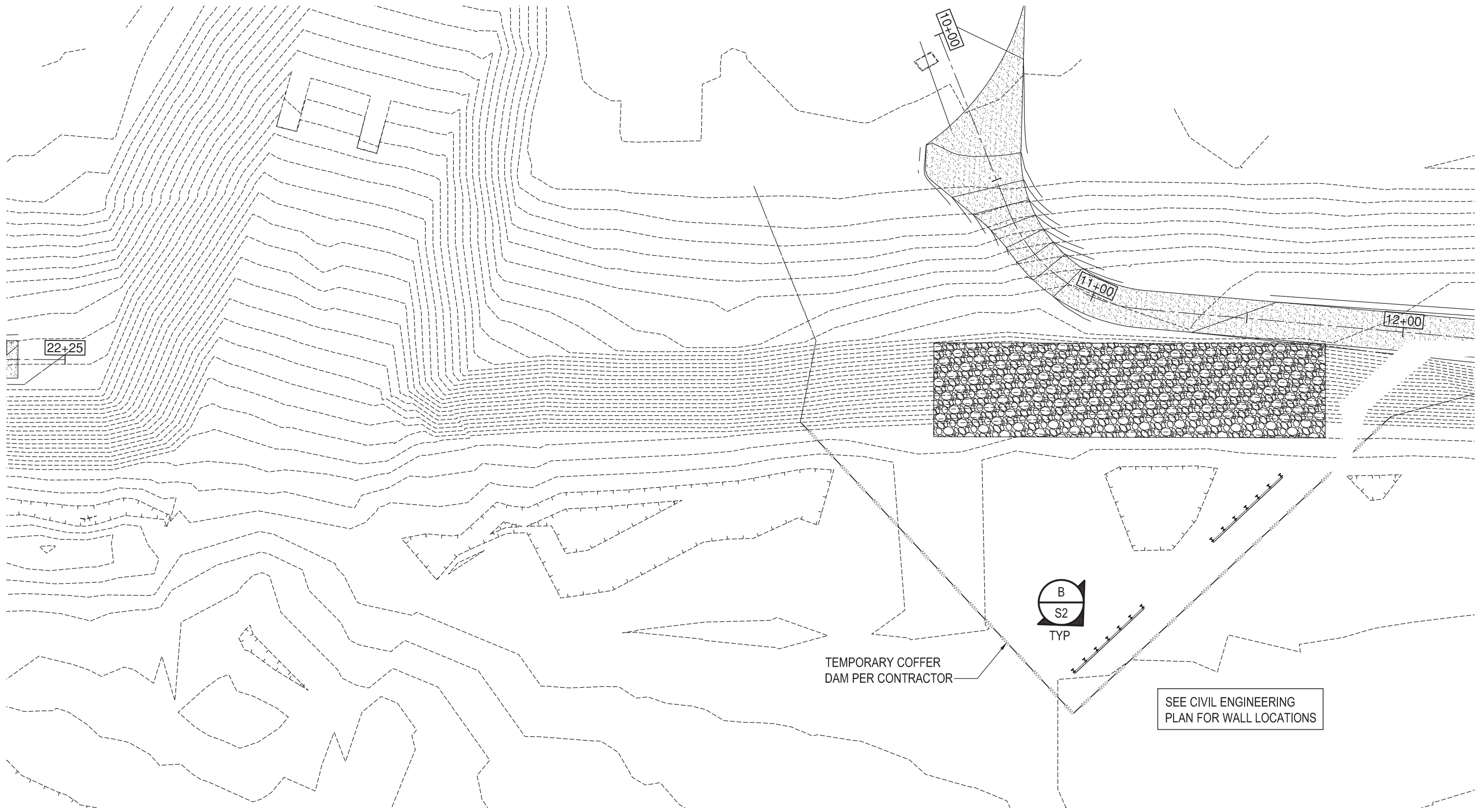
Project Number:
**0130.5014**

Drawing Number:
**C1**

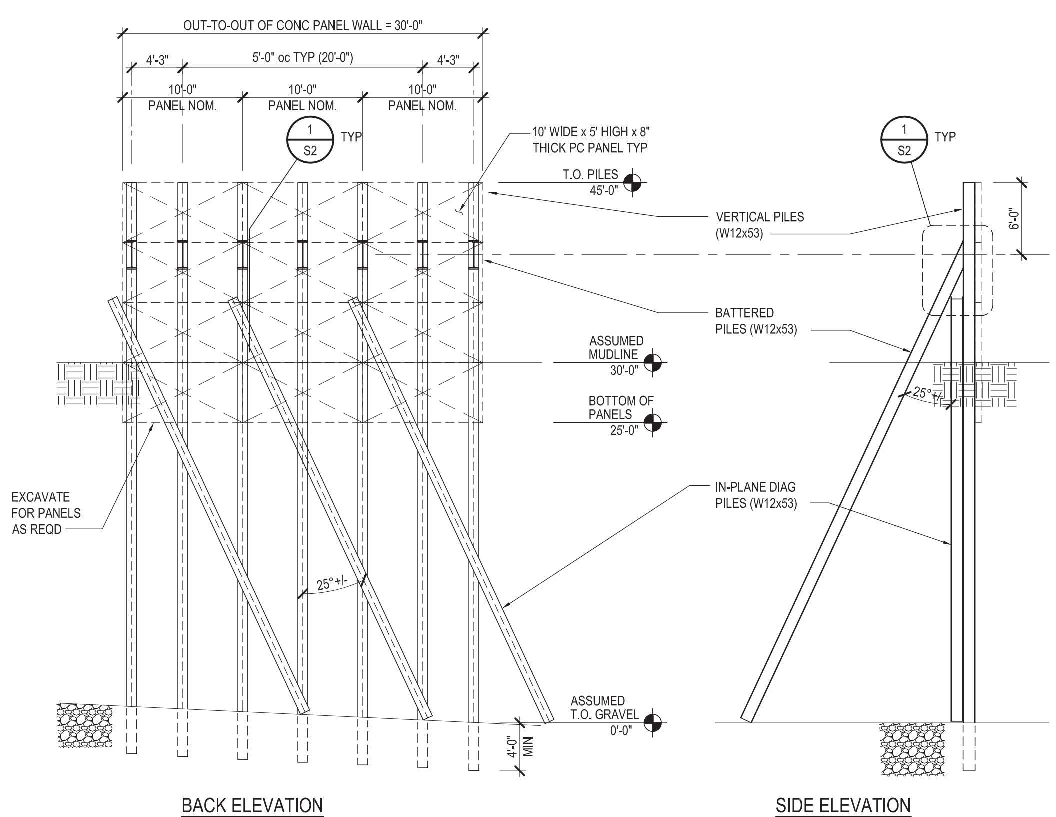
Sheet Number:
**3** of **6**

DRAWING: T:\PROJECTS\0130 CASTLE ROCK\5014 BOAT LAUNCH SAFETY IMPROVEMENTS\CONTRACT DRAWINGS\01305014 SITE CONTRACT DRAWING.DWG, LAYOUT TAB: C1, PLOT DATE: 5/13/2020 4:37:58 PM, DRAWING SAVE DATE: 3/27/2020 12:06:45 PM, PLOTTED BY: KROGERS
PROFILE: GIBBS & OLSON STANDARD -- C3D IMPERIAL 2020, PLOT DEVICE: GIBBS & OLSON -- DWG TO PDF.PC3, PLOT STYLE TABLE: GIBBS-OLSON STANDARD MONOCHROME.CTB, PAPER SIZE: GIBBS & OLSON -- PLANSHEET D SIZE (34.00 X 22.00 INCHES)

Figure 2 - Design Drawings

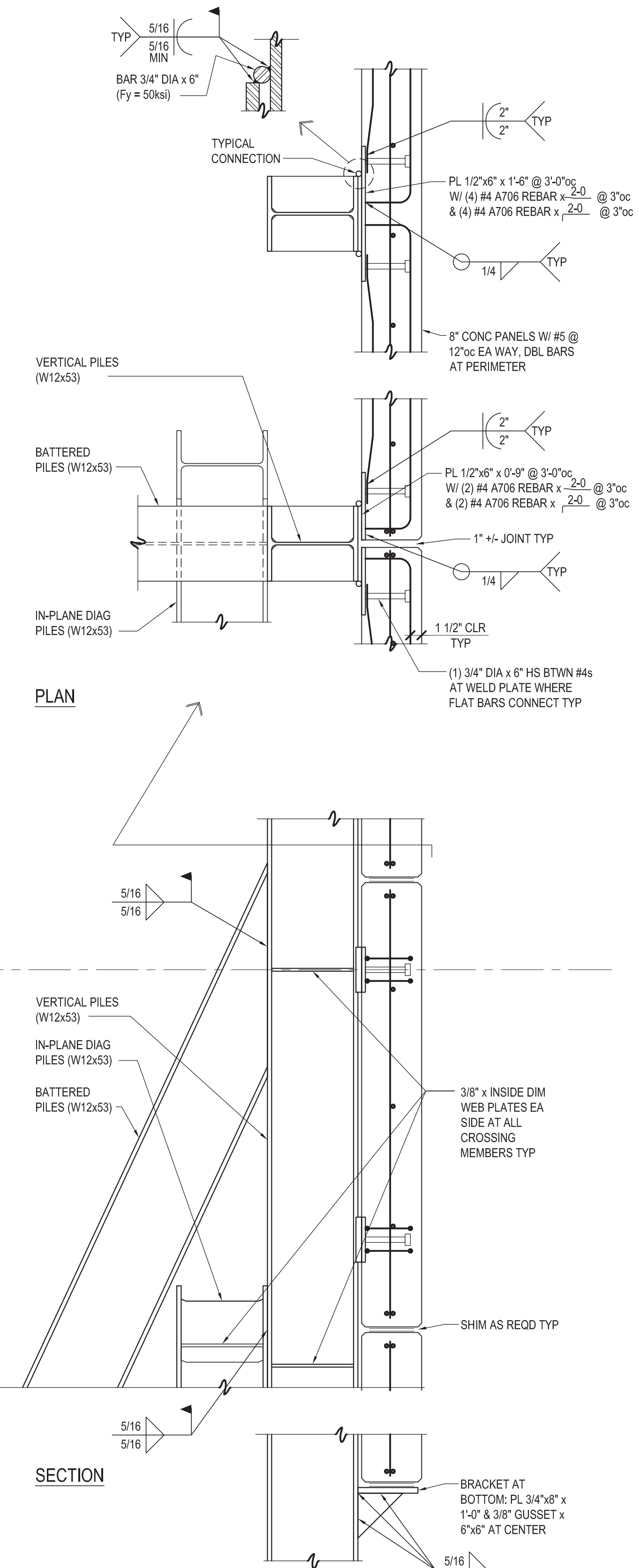



A
S2
## PARTIAL PLAN
1" = 30'-0"




B
S2
## WALL ELEVATION
1/8" =1'-0"

NOTES
1. BATTER & IN-LINE DIAGONAL PILES TO BE VIBRATORY DRIVEN TO REFUSAL AT GRAVEL LAYER. GRAVEL LAYER ASSUMED TO BE APPROXIMATELY 30 FEET BELOW MUD LINE. IF SHALLOWER GRAVEL LAYER PREVENTS PILE FROM BEING VIBRATORY DRIVEN TO REQUIRED DEPTH, CONTACT ENGINEER TO VERIFY IF ACCEPTABLE OR IF HAMMER DRIVING WILL BE REQUIRED.
2. SOLDIER PILES TO BE VIBRATORY DRIVEN TO REFUSAL AT THE ASSUMED GRAVEL LAYER APPROXIMATELY 30 FEET BELOW THE MUD LINE AND THEN HAMMER DRIVEN AN ADDITIONAL 4 FEET MINIMUM. USE DRIVING SHOE AT END OF THE PILES AND AIR BUBBLE CURTAIN FOR PORTION OF DRIVING WITH HAMMER. USE BUBBLE TUBING AS A PREFERRED METHOD TO CREATE A CURTAIN AROUND THE PILE.
3. SEE GEOTECHNICAL REPORT FOR INFORMATION CONCERNING SOILS AT INSTALLATION SITE.




1
S2
## PILE DETAIL
1"=1'-0"

NORTH

**RIGHT-OF-WAY DISCLAIMER**

THE RIGHT-OF-WAY AND/OR PROPERTY LINES SHOWN HEREON ARE BASED ON AVAILABLE INFORMATION, NOT ON A SURVEYED LOCATION AND ARE ONLY APPROXIMATE

Structural
Structural Plan
-

City of Castle Rock
Boat Launch Safety Improvements
Castle Rock, WA

Datum: NAD88-2001 NAVD 88

Survey Book: ----

Project Milestone:
**PRELIMINARY**

Date:
**03/27/2020**


GIBBS & OLSON

Project Manger: MH
Designed by: JG
CAD by: ATW
Checked by: JG
Approved by: MH

Project Number:
**0130.5014**

Drawing Number:
**S2**

Sheet Number:
**6 of 6**

Figure 2 - Design Drawings (continued)

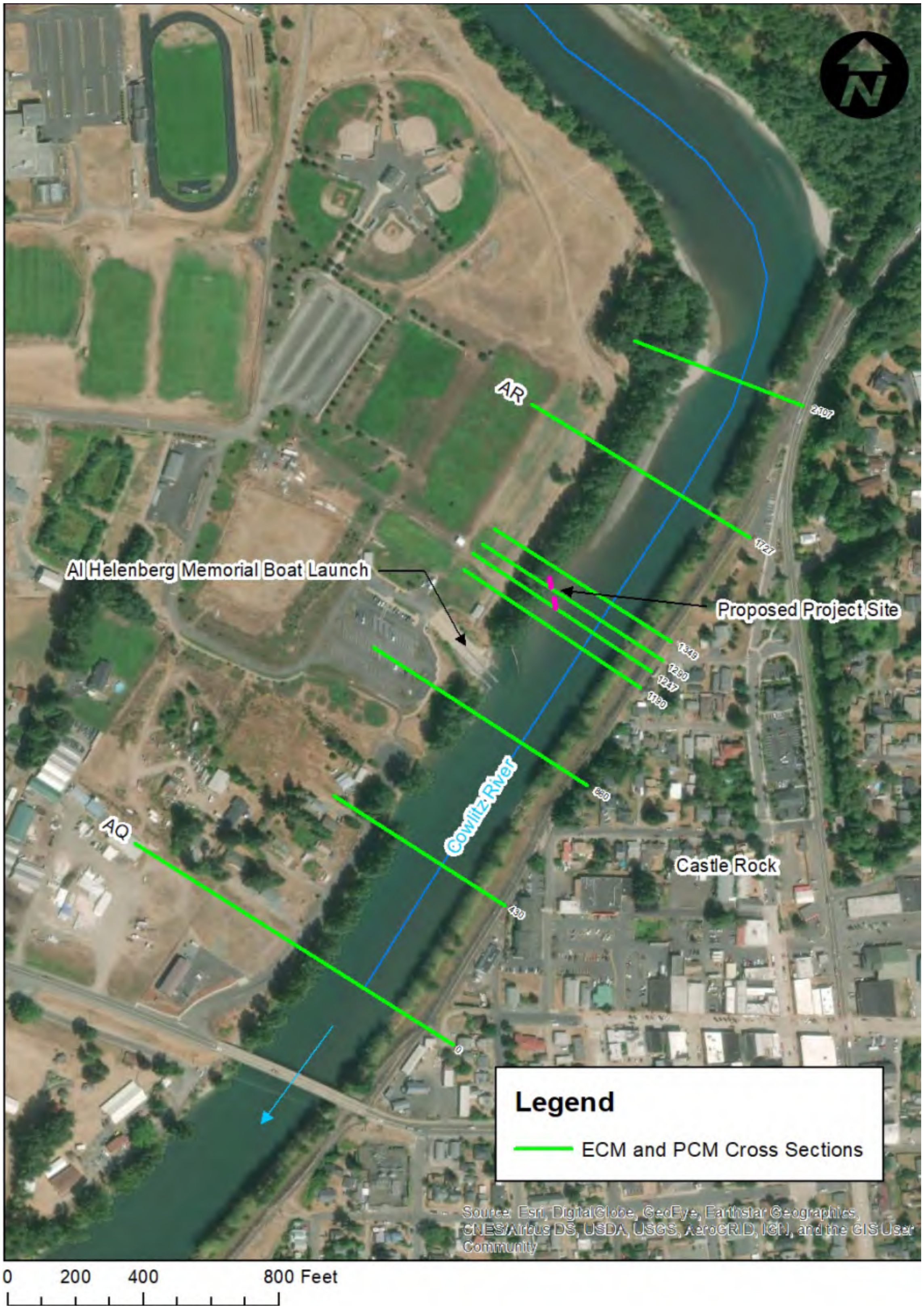
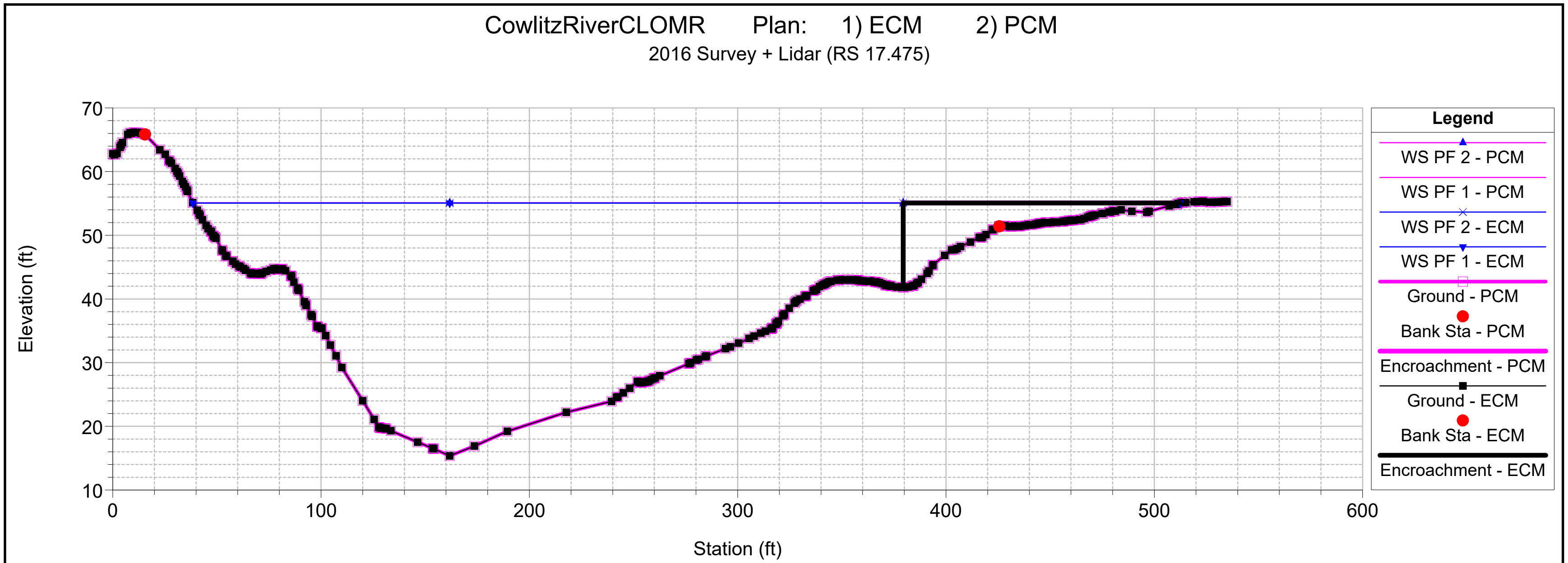
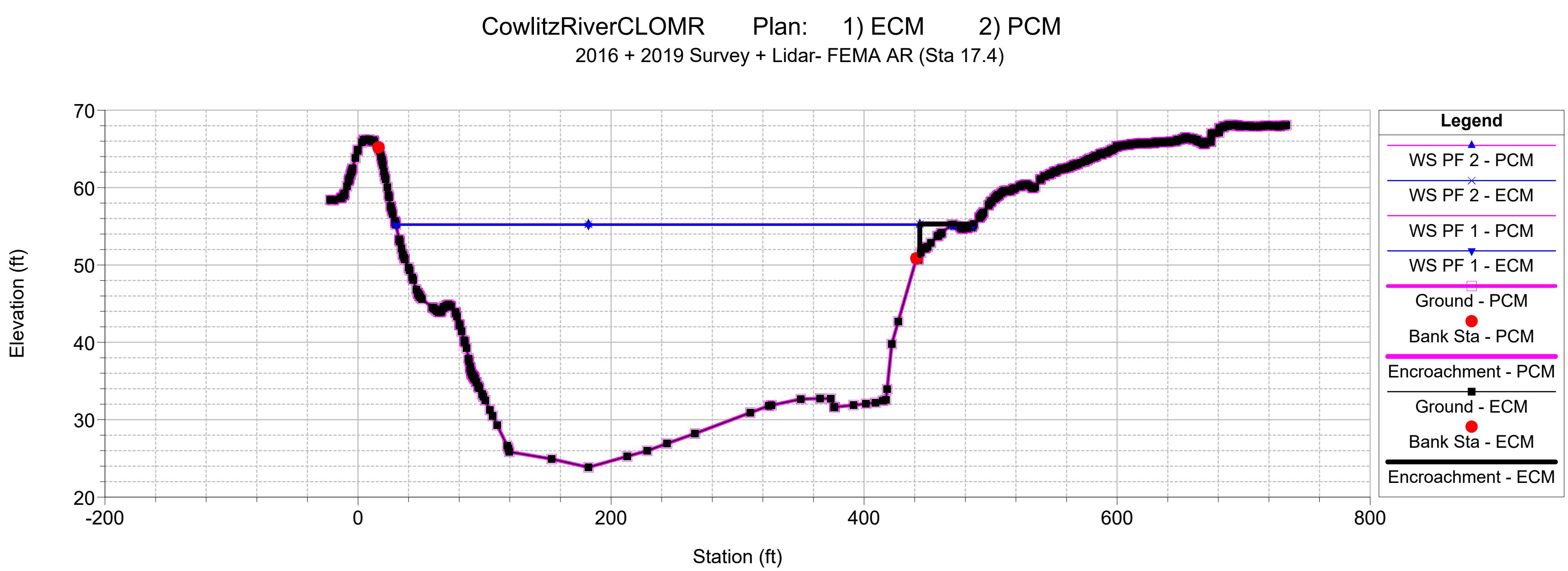



Figure 3. Hydraulic model cross section locations

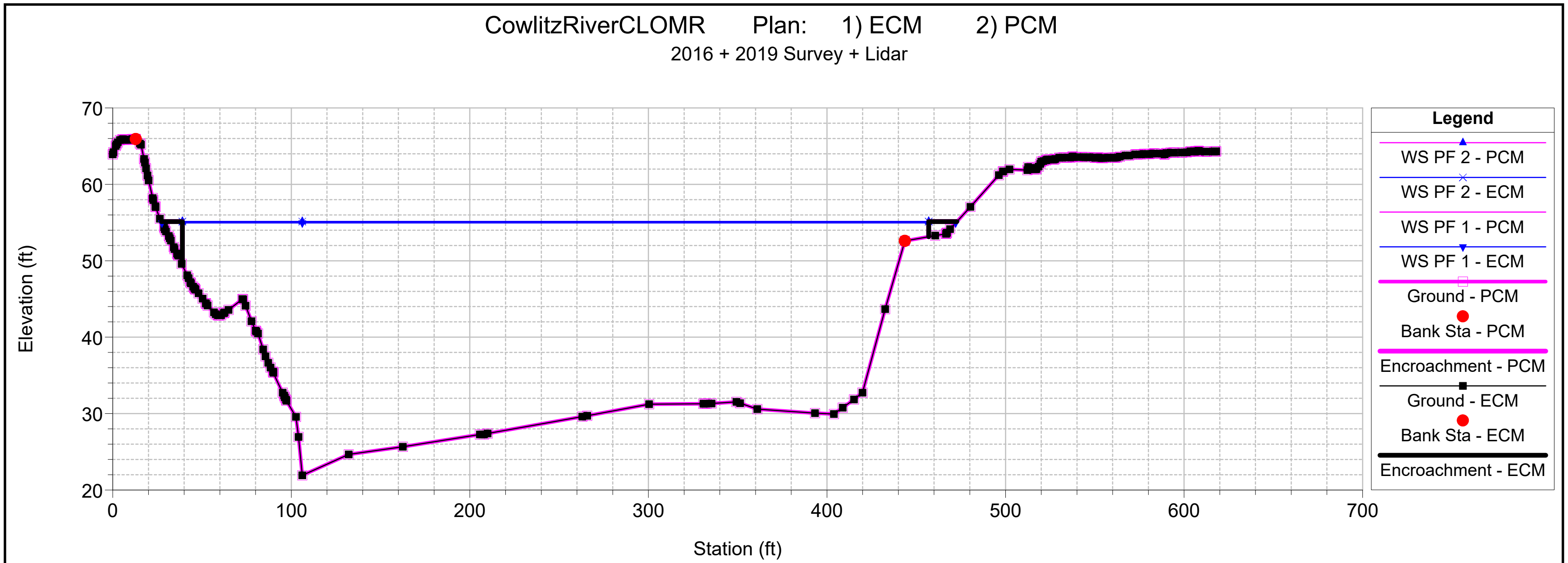
# APPENDIX B
# HYDRAULIC CROSS SECTIONS



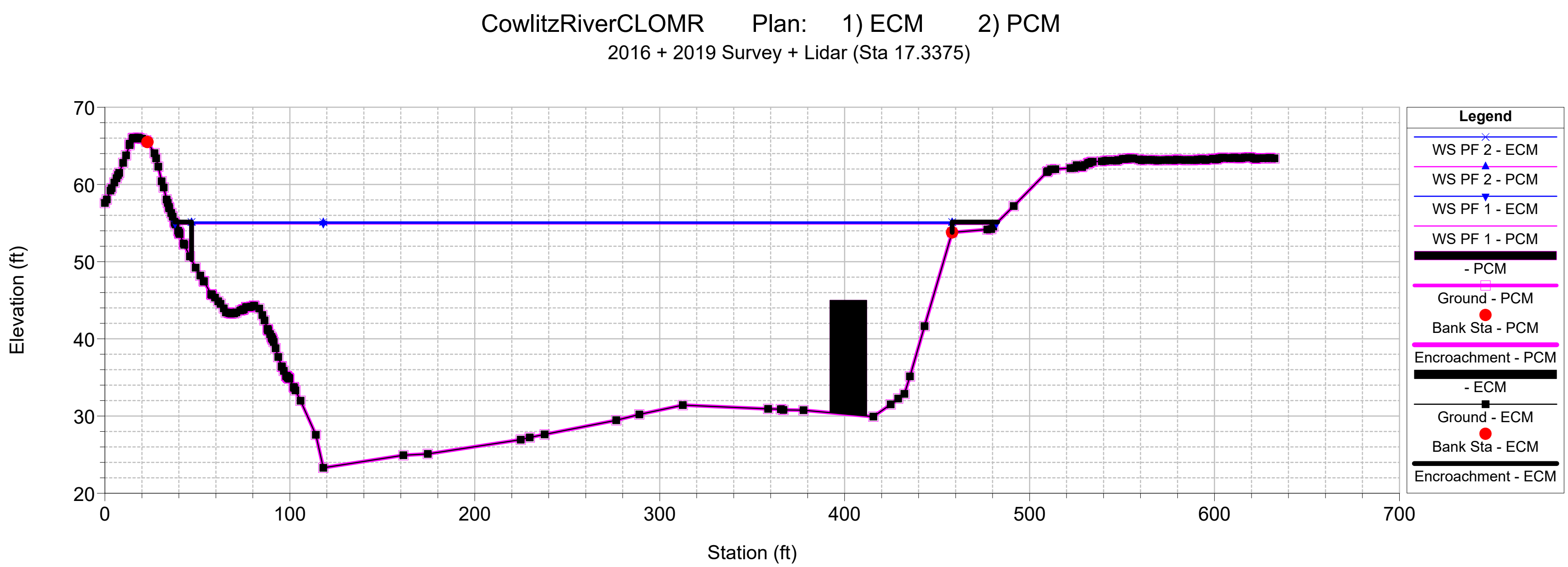
CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
2016 Survey + Lidar (RS 17.475)
Elevation (ft)
Station (ft)
Legend
WS PF 2 - PCM
WS PF 1 - PCM
WS PF 2 - ECM
WS PF 1 - ECM
Ground - PCM
Bank Sta - PCM
Encroachment - PCM
Ground - ECM
Bank Sta - ECM
Encroachment - ECM

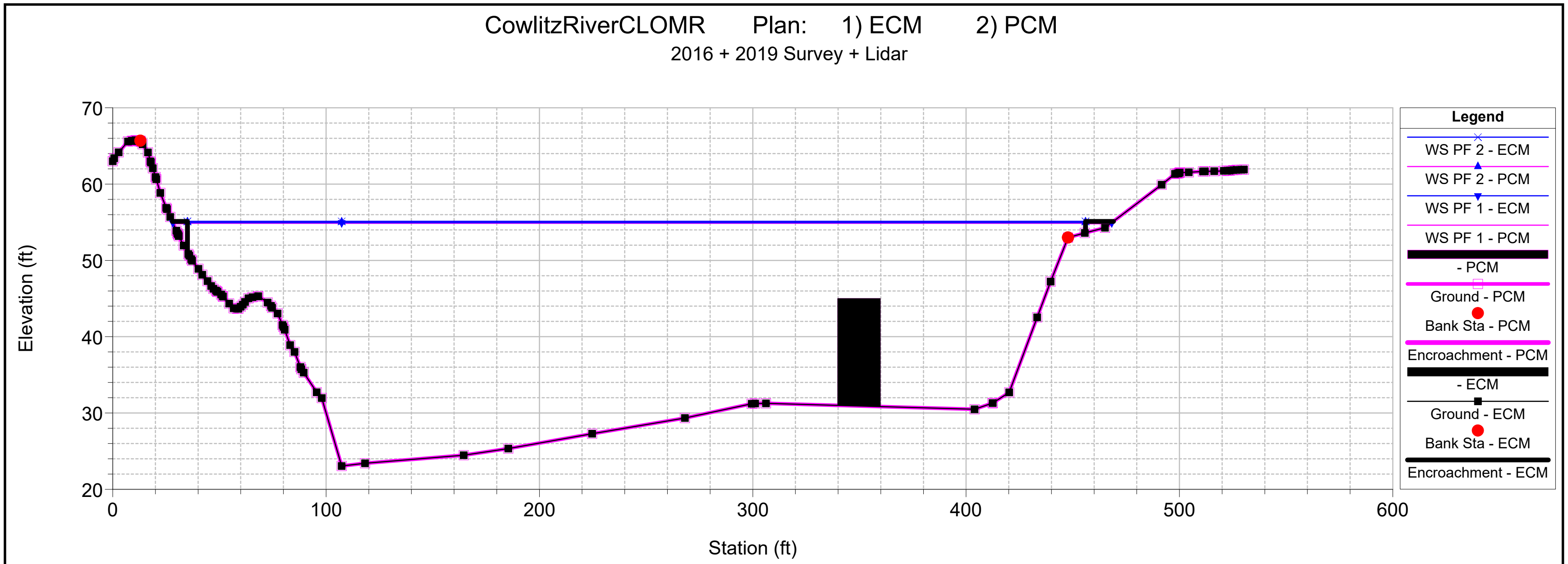
CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
2016 + 2019 Survey + Lidar- FEMA AR (Sta 17.4)
Elevation (ft)
Station (ft)
Legend
WS PF 2 - PCM
WS PF 2 - ECM
WS PF 1 - PCM
WS PF 1 - ECM
Ground - PCM
Bank Sta - PCM
Encroachment - PCM
Ground - ECM
Bank Sta - ECM
Encroachment - ECM

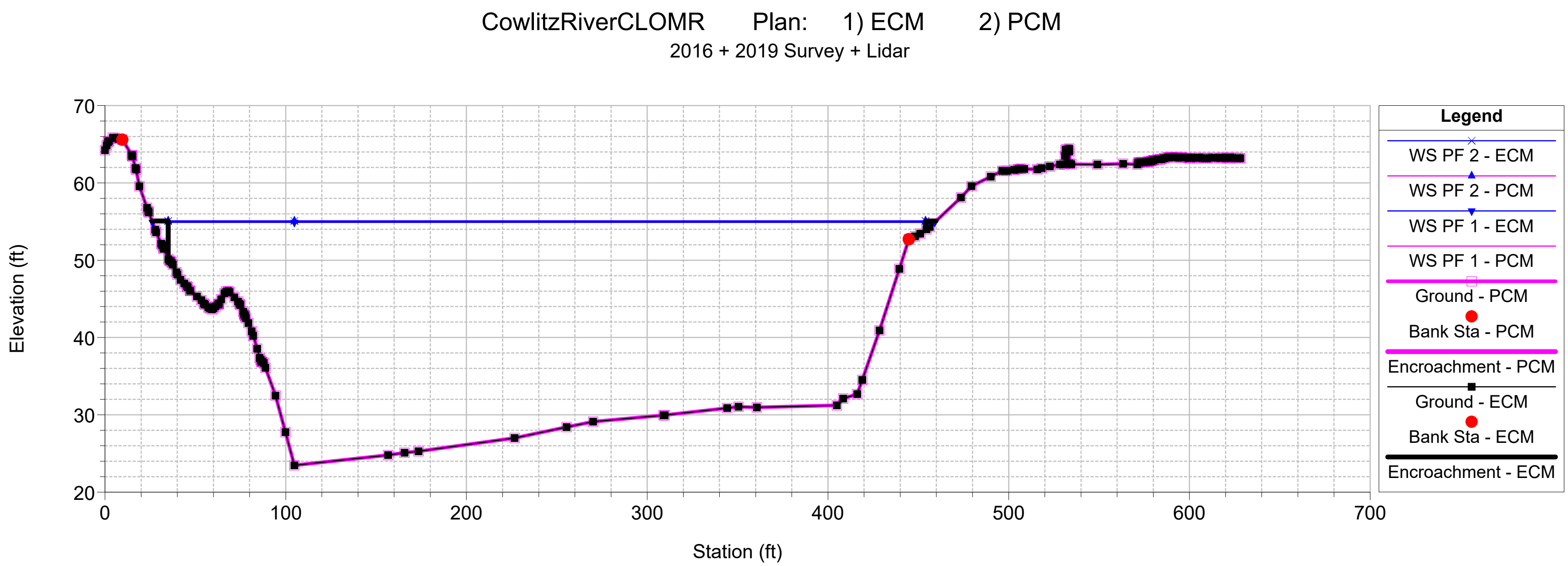


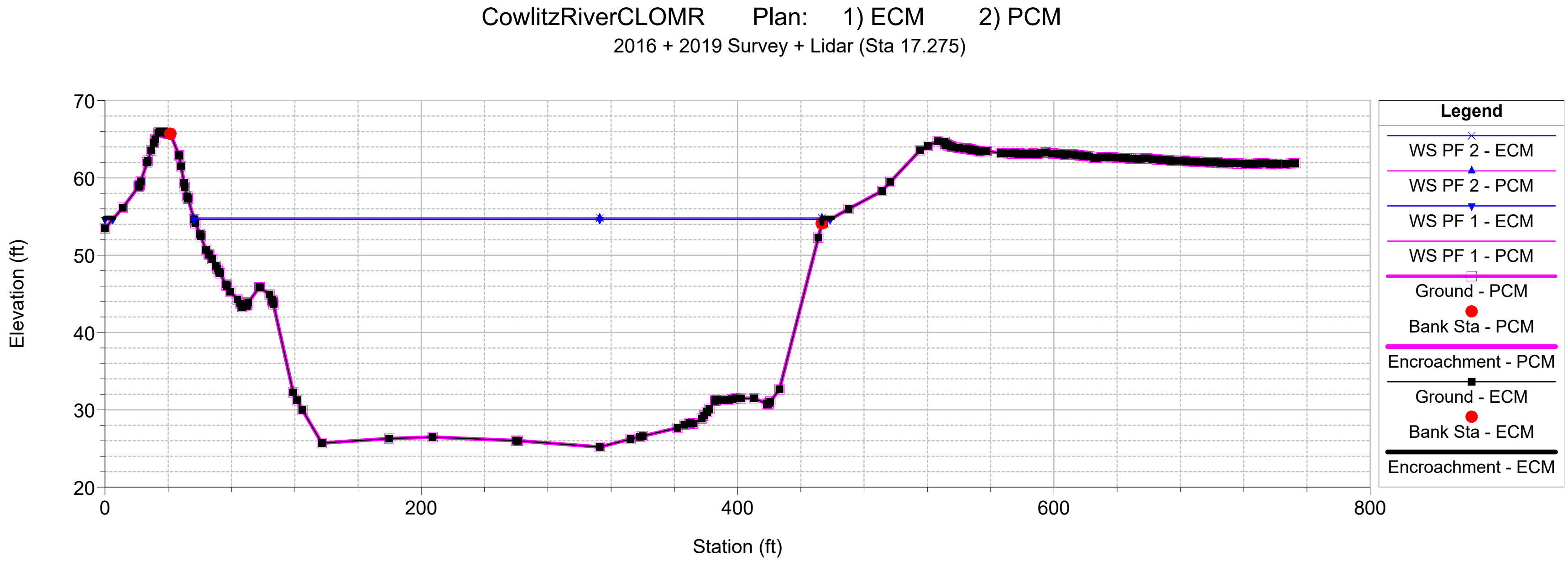
CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
2016 + 2019 Survey + Lidar
Elevation (ft)
Station (ft)
70
60
50
40
30
20
0
100
200
300
400
500
600
700
Legend
WS PF 2 - PCM
WS PF 2 - ECM
WS PF 1 - PCM
WS PF 1 - ECM
Ground - PCM
Bank Sta - PCM
Encroachment - PCM
Ground - ECM
Bank Sta - ECM
Encroachment - ECM

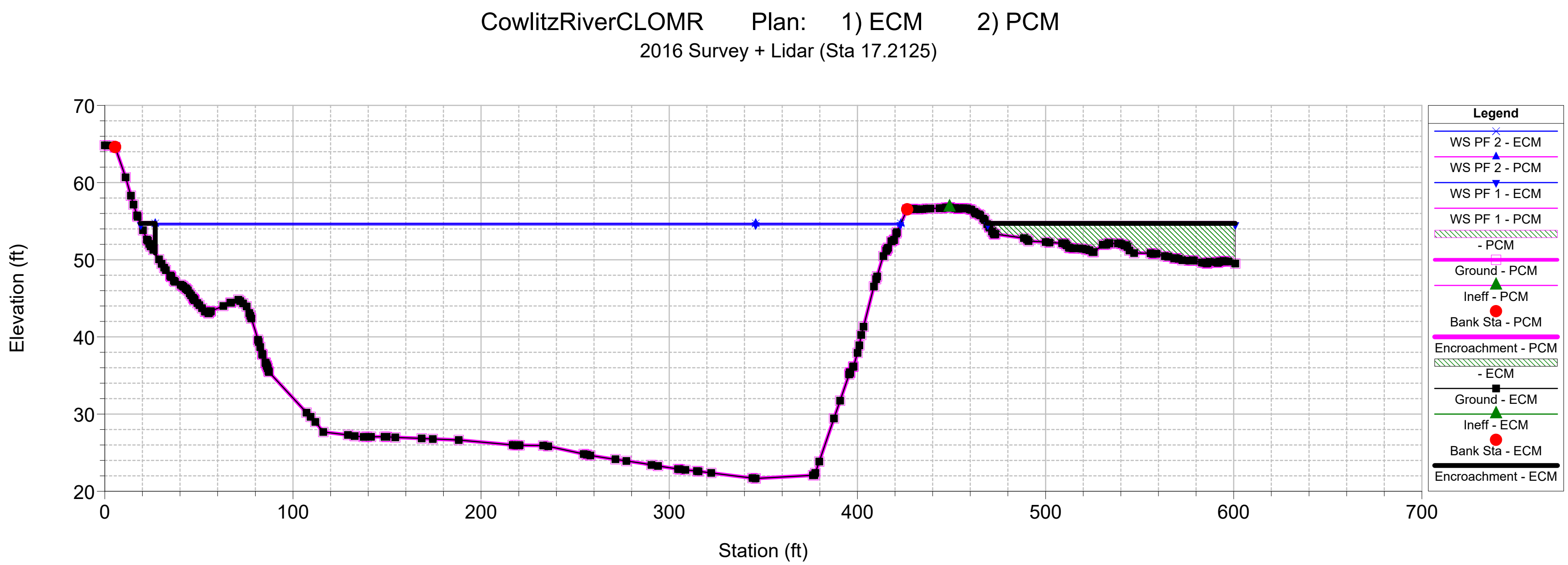



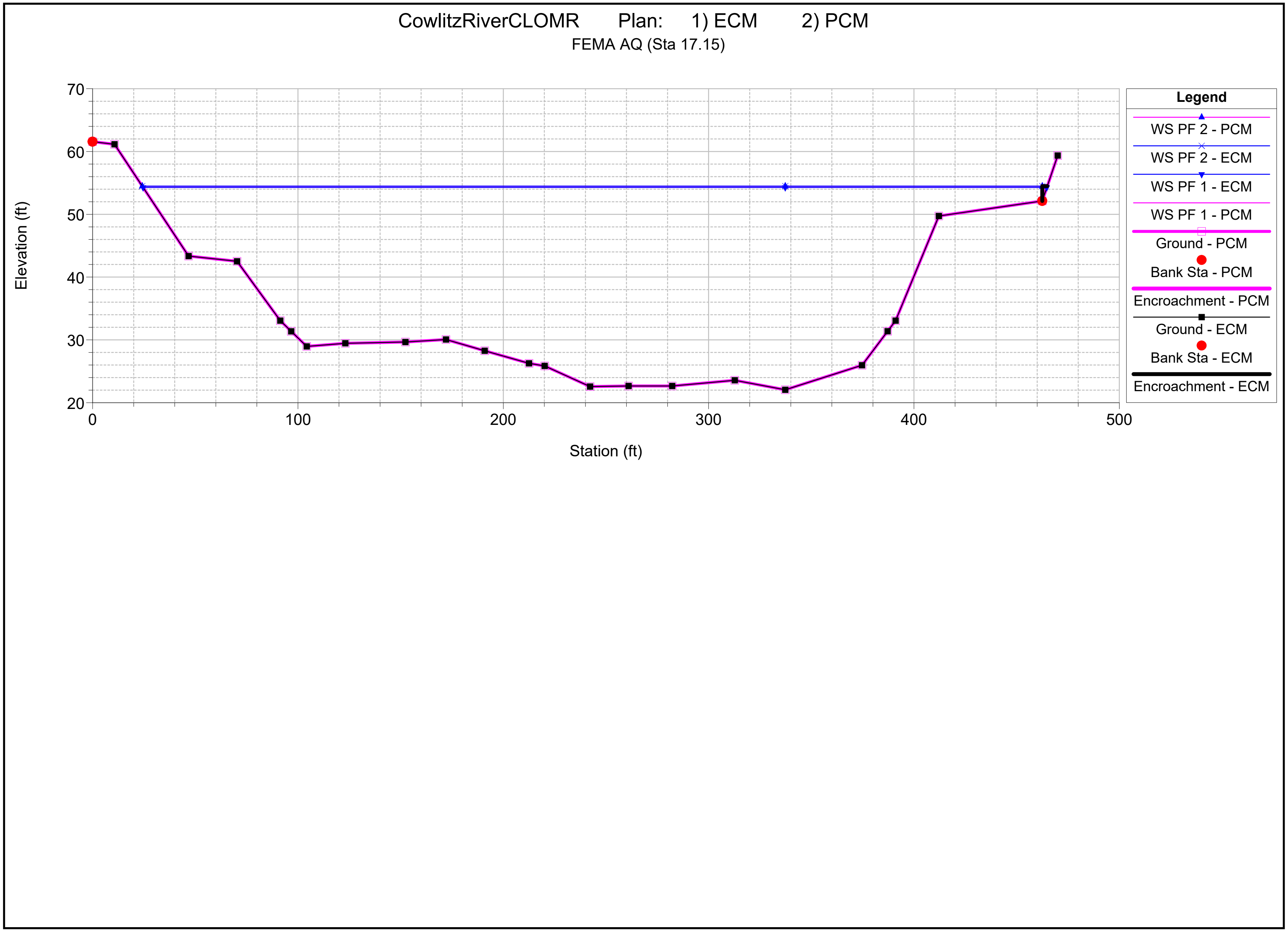
CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
2016 + 2019 Survey + Lidar (Sta 17.3375)
Elevation (ft)
Station (ft)
70
60
50
40
30
20
0
100
200
300
400
500
600
700
Legend
WS PF 2 - ECM
WS PF 2 - PCM
WS PF 1 - ECM
WS PF 1 - PCM
- PCM
Ground - PCM
Bank Sta - PCM
Encroachment - PCM
- ECM
Ground - ECM
Bank Sta - ECM
Encroachment - ECM


CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
2016 + 2019 Survey + Lidar
Elevation (ft)
Station (ft)
Legend
WS PF 2 - ECM
WS PF 2 - PCM
WS PF 1 - ECM
WS PF 1 - PCM
- PCM
Ground - PCM
Bank Sta - PCM
Encroachment - PCM
- ECM
Ground - ECM
Bank Sta - ECM
Encroachment - ECM


CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
2016 + 2019 Survey + Lidar
Elevation (ft)
Station (ft)
Legend
WS PF 2 - ECM
WS PF 2 - PCM
WS PF 1 - ECM
WS PF 1 - PCM
Ground - PCM
Bank Sta - PCM
Encroachment - PCM
Ground - ECM
Bank Sta - ECM
Encroachment - ECM


CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
2016 + 2019 Survey + Lidar (Sta 17.275)
Elevation (ft)
Station (ft)
70
60
50
40
30
20
0
200
400
600
800
Legend
WS PF 2 - ECM
WS PF 2 - PCM
WS PF 1 - ECM
WS PF 1 - PCM
Ground - PCM
Bank Sta - PCM
Encroachment - PCM
Ground - ECM
Bank Sta - ECM
Encroachment - ECM


CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
2016 Survey + Lidar (Sta 17.2125)
Elevation (ft)
Station (ft)
70
60
50
40
30
20
0
100
200
300
400
500
600
700
Legend
WS PF 2 - ECM
WS PF 2 - PCM
WS PF 1 - ECM
WS PF 1 - PCM
- PCM
Ground - PCM
Ineff - PCM
Bank Sta - PCM
Encroachment - PCM
- ECM
Ground - ECM
Ineff - ECM
Bank Sta - ECM
Encroachment - ECM



CowlitzRiverCLOMR Plan: 1) ECM 2) PCM
FEMA AQ (Sta 17.15)
Elevation (ft)
Station (ft)
70
60
50
40
30
20
0
100
200
300
400
500
Legend
WS PF 2 - PCM
WS PF 2 - ECM
WS PF 1 - ECM
WS PF 1 - PCM
Ground - PCM
Bank Sta - PCM
Encroachment - PCM
Ground - ECM
Bank Sta - ECM
Encroachment - ECM

# APPENDIX C
# HYDRAULIC MODEL RESULTS

**Duplicate Effective Model (DEM)**

| Reach | River Sta | Profile | Top Wdth Act (ft) | Area (sq ft) | Vel Total (ft/s) | W.S. Elev (ft) | Base WS (ft) | Prof Delta WS (ft) |
|---|---|---|---|---|---|---|---|---|
| Al Helenberg | 0 | PF 1 | 440.13 | 9618.58 | 10.08 | 54.28 | 54.28 | |
| Al Helenberg | 0 | PF 2 | 438.29 | 9703.77 | 10 | 54.48 | 54.28 | 0.2 |
| | | | | | | | | |
| Al Helenberg | 430 | PF 1 | 396.83 | 8529.32 | 11.37 | 54.22 | 54.22 | |
| Al Helenberg | 430 | PF 2 | 398.16 | 8609 | 11.27 | 54.42 | 54.22 | 0.2 |
| | | | | | | | | |
| Al Helenberg | 860 | PF 1 | 404.18 | 7759.31 | 12.5 | 54.24 | 54.24 | |
| Al Helenberg | 860 | PF 2 | 405.54 | 7840.59 | 12.37 | 54.44 | 54.24 | 0.2 |
| | | | | | | | | |
| Al Helenberg | 1290 | PF 1 | 410.03 | 8148.81 | 11.9 | 54.85 | 54.85 | |
| Al Helenberg | 1290 | PF 2 | 411.02 | 8222.28 | 11.8 | 55.03 | 54.85 | 0.18 |
| | | | | | | | | |
| Al Helenberg | 1727 | PF 1 | 415.25 | 8950.26 | 10.84 | 55.55 | 55.55 | |
| Al Helenberg | 1727 | PF 2 | 414.29 | 9015.62 | 10.76 | 55.71 | 55.55 | 0.16 |
| | | | | | | | | |
| Al Helenberg | 2107 | PF 1 | 363.13 | 6748 | 14.37 | 54.89 | 54.89 | |
| Al Helenberg | 2107 | PF 2 | 364.12 | 6811.16 | 14.24 | 55.06 | 54.89 | 0.17 |

**Existing Conditions Model (ECM)**

| Reach | River Sta | Profile | Top Wdth Act (ft) | Area (sq ft) | Vel Total (ft/s) | W.S. Elev (ft) | Base WS (ft) | Prof Delta WS (ft) |
|---|---|---|---|---|---|---|---|---|
| Al Helenberg | 0 | PF 1 | 440.13 | 9618.58 | 10.08 | 54.28 | 54.28 | |
| Al Helenberg | 0 | PF 2 | 438.29 | 9703.77 | 10 | 54.48 | 54.28 | 0.2 |
| | | | | | | | | |
| Al Helenberg | 430 | PF 1 | 403.64 | 10188.19 | 9.93 | 54.53 | 54.53 | |
| Al Helenberg | 430 | PF 2 | 396.29 | 9828.38 | 9.87 | 54.71 | 54.53 | 0.18 |
| | | | | | | | | |
| Al Helenberg | 860 | PF 1 | 407.1 | 9339.19 | 10.39 | 54.62 | 54.62 | |
| Al Helenberg | 860 | PF 2 | 397.23 | 9407.48 | 10.31 | 54.8 | 54.62 | 0.18 |
| | | | | | | | | |
| Al Helenberg | 1190 | PF 1 | 432.79 | 9702.87 | 10 | 54.91 | 54.91 | |
| Al Helenberg | 1190 | PF 2 | 419 | 9748.71 | 9.95 | 55.07 | 54.91 | 0.17 |
| | | | | | | | | |
| Al Helenberg | 1247 | PF 1 | 440.08 | 9743.73 | 9.96 | 54.94 | 54.94 | |
| Al Helenberg | 1247 | PF 2 | 421 | 9790.82 | 9.91 | 55.11 | 54.94 | 0.17 |
| | | | | | | | | |
| Al Helenberg | 1290 | PF 1 | 444.17 | 9750.84 | 9.95 | 54.97 | 54.97 | |
| Al Helenberg | 1290 | PF 2 | 411.2 | 9774.95 | 9.92 | 55.13 | 54.97 | 0.16 |
| | | | | | | | | |
| Al Helenberg | 1349 | PF 1 | 444.54 | 9622.18 | 10.08 | 54.96 | 54.96 | |
| Al Helenberg | 1349 | PF 2 | 418 | 9639.24 | 10.06 | 55.12 | 54.96 | 0.16 |
| | | | | | | | | |
| Al Helenberg | 1727 | PF 1 | 452.83 | 9564.86 | 10.14 | 55.12 | 55.12 | |
| Al Helenberg | 1727 | PF 2 | 414.19 | 9586.16 | 10.12 | 55.28 | 55.12 | 0.16 |
| | | | | | | | | |
| Al Helenberg | 2107 | PF 1 | 473.79 | 8728.87 | 11.11 | 55.06 | 55.06 | |
| Al Helenberg | 2107 | PF 2 | 340.9 | 8165.36 | 11.88 | 55.07 | 55.06 | 0.01 |

**Proposed Conditions Model (PCM)**

| Reach | River Sta | Profile | Top Wdth Act (ft) | Area (sq ft) | Vel Total (ft/s) | W.S. Elev (ft) | Base WS (ft) | Prof Delta WS (ft) |
|---|---|---|---|---|---|---|---|---|
| Al Helenberg | 0 | PF 1 | 440.13 | 9618.58 | 10.08 | 54.28 | 54.28 | |
| Al Helenberg | 0 | PF 2 | 438.29 | 9703.77 | 10 | 54.48 | 54.28 | 0.2 |
| | | | | | | | | |
| Al Helenberg | 430 | PF 1 | 403.64 | 10188.19 | 9.93 | 54.53 | 54.53 | |
| Al Helenberg | 430 | PF 2 | 396.29 | 9828.33 | 9.87 | 54.71 | 54.53 | 0.18 |
| | | | | | | | | |
| Al Helenberg | 860 | PF 1 | 407.1 | 9339.19 | 10.39 | 54.62 | 54.62 | |
| Al Helenberg | 860 | PF 2 | 397.23 | 9407.42 | 10.31 | 54.8 | 54.62 | 0.18 |
| | | | | | | | | |
| Al Helenberg | 1190 | PF 1 | 432.79 | 9702.87 | 10 | 54.91 | 54.91 | |
| Al Helenberg | 1190 | PF 2 | 419 | 9748.66 | 9.95 | 55.07 | 54.91 | 0.17 |
| | | | | | | | | |
| Al Helenberg | 1247 | PF 1 | 439.61 | 9429.18 | 10.29 | 54.87 | 54.87 | |
| Al Helenberg | 1247 | PF 2 | 421 | 9477.85 | 10.23 | 55.04 | 54.87 | 0.17 |
| | | | | | | | | |
| Al Helenberg | 1290 | PF 1 | 443.73 | 9421.91 | 10.3 | 54.89 | 54.89 | |
| Al Helenberg | 1290 | PF 2 | 411.2 | 9448.96 | 10.27 | 55.06 | 54.89 | 0.17 |
| | | | | | | | | |
| Al Helenberg | 1349 | PF 1 | 444.74 | 9637.18 | 10.07 | 54.99 | 54.99 | |
| Al Helenberg | 1349 | PF 2 | 418 | 9652.12 | 10.05 | 55.15 | 54.99 | 0.16 |
| | | | | | | | | |
| Al Helenberg | 1727 | PF 1 | 453.85 | 9579.75 | 10.13 | 55.15 | 55.15 | |
| Al Helenberg | 1727 | PF 2 | 414.23 | 9598.61 | 10.11 | 55.31 | 55.15 | 0.16 |
| | | | | | | | | |
| Al Helenberg | 2107 | PF 1 | 474.35 | 8744.52 | 11.09 | 55.09 | 55.09 | |
| Al Helenberg | 2107 | PF 2 | 340.96 | 8175.67 | 11.86 | 55.1 | 55.09 | 0.01 |