I-405 MP 21.94 Juanita Creek (WDFW ID 998602):

Preliminary Hydraulic Design Report

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1 Introduction

To comply with United States et al. vs. Washington et al. No. C70-9213 Subproceeding No. 01-1 dated March 29, 2013 (a federal permanent injunction requiring the State of Washington to correct fish barriers in Water Resource Inventory Areas [WRIAs] 1 through 23), the Washington State Department of Transportation (WSDOT) is proposing a project to provide fish passage at the Interstate 405 (I-405) crossing of Juanita Creek at milepost (MP) 21.94 within WSDOT's Northwest region. The existing structure at that location has been identified as a fish barrier by the Washington Department of Fish and Wildlife (WDFW) and WSDOT Environmental Services Office (ESO) (WDFW Site ID 998602) and has an estimated 1,309 linear feet of habitat gain.

The Juanita Creek restored stream connection discussed herein is part of a larger roadway project that will address fish barriers while providing a high-capacity roadway for the traveling public. Consistent with the long-term I-405 Master Plan (WSDOT 2002), the I-405 State Route (SR) 522 Vicinity to SR 527 Express Toll Lanes Improvement Project (Project) is intended improve transit reliability on I-405 and reduce congestion between the cities of Kirkland and Bothell.

The overall Project will include the following improvements:

- A dual express toll lanes (ETL) system will be created from MP 21.79 (south of I-405/SR 522 interchange) to MP 27.06 (just north of I-405/SR 527 interchange).
- The existing lanes will be restriped and I-405 will be resurfaced and widened I-405 to add one ETL in each direction, creating a dual ETL system.
- New direct access ramps will be constructed in the SR 522 interchange, and two inline transit stations will be constructed in the I-405 median. A new bridge will be constructed for I-405 northbound lanes using the reconfigured ramps. A bus stop and turnaround loop with pick-up and drop-off facilities will be constructed in the northwest quadrant.
- The northbound I-405 overcrossing will be widened at 228th Street SE.
- Just south of SR 527 at 17th Avenue SE, new direct access ramps will be constructed to the north, south, and east, and two inline transit stations will be constructed in the I-405 median.
- 17th Avenue SE and portions of 220th Street SE and SR 527 will be reconfigured to include a roundabout at the Canyon Park Park and Ride; bicycle and pedestrian improvements will also be made.
- Six fish barriers will be replaced with restored stream connections, and new noise and retaining walls will be constructed.
- Flow control and enhanced runoff treatment will be provided, based on the WSDOT *Highway Runoff Manual* (WSDOT 2019).

According to the federal injunction, and in order of preference, fish passage should be achieved by (1) avoiding the necessity for the roadway to cross the stream, (2) use of a full-span bridge, or (3) use of the stream simulation methodology. WSDOT evaluated the crossing using the stream simulation methodology because the permanent federal injunction allows for the use of the stream simulation method and bridge design method unless extraordinary circumstances exist on site. Juanita Creek does not meet all the criteria for the bridge design method, so the stream simulation methodology is the optimal choice.

The crossing is in King County, Washington. The crossing is in WRIA 8. The highway runs north to south at this location and is about 2.9 miles upstream from the confluence with Lake Washington. Juanita Creek generally flows east to west beginning 0.5 miles upstream of the I-405 crossing (see **Figure 1-1** for the vicinity map).

The proposed Juanita Creek project will replace the 358.6-foot-long, 48-inch-diameter corrugated steel pipe with a structure designed to accommodate a minimum hydraulic width of 23 feet. It is being recommended that the selected 30-foot structure be bottomless to reduce the potential of streambed material washout within the structure. The proposed structure is designed to meet the requirements of the federal injunction using the stream simulation criteria described in the 2013 WDFW *Water Crossing Design Guidelines* (WCDG) (Barnard et al. 2013) and commitments made between WSDOT, WDFW, the National Marine Fisheries Service, and the Muckleshoot Indian Tribe Fisheries Division (MITFD) during preliminary design and permitting. This design also meets the requirements of the WSDOT *Hydraulics Manual* (WSDOT 2023a).

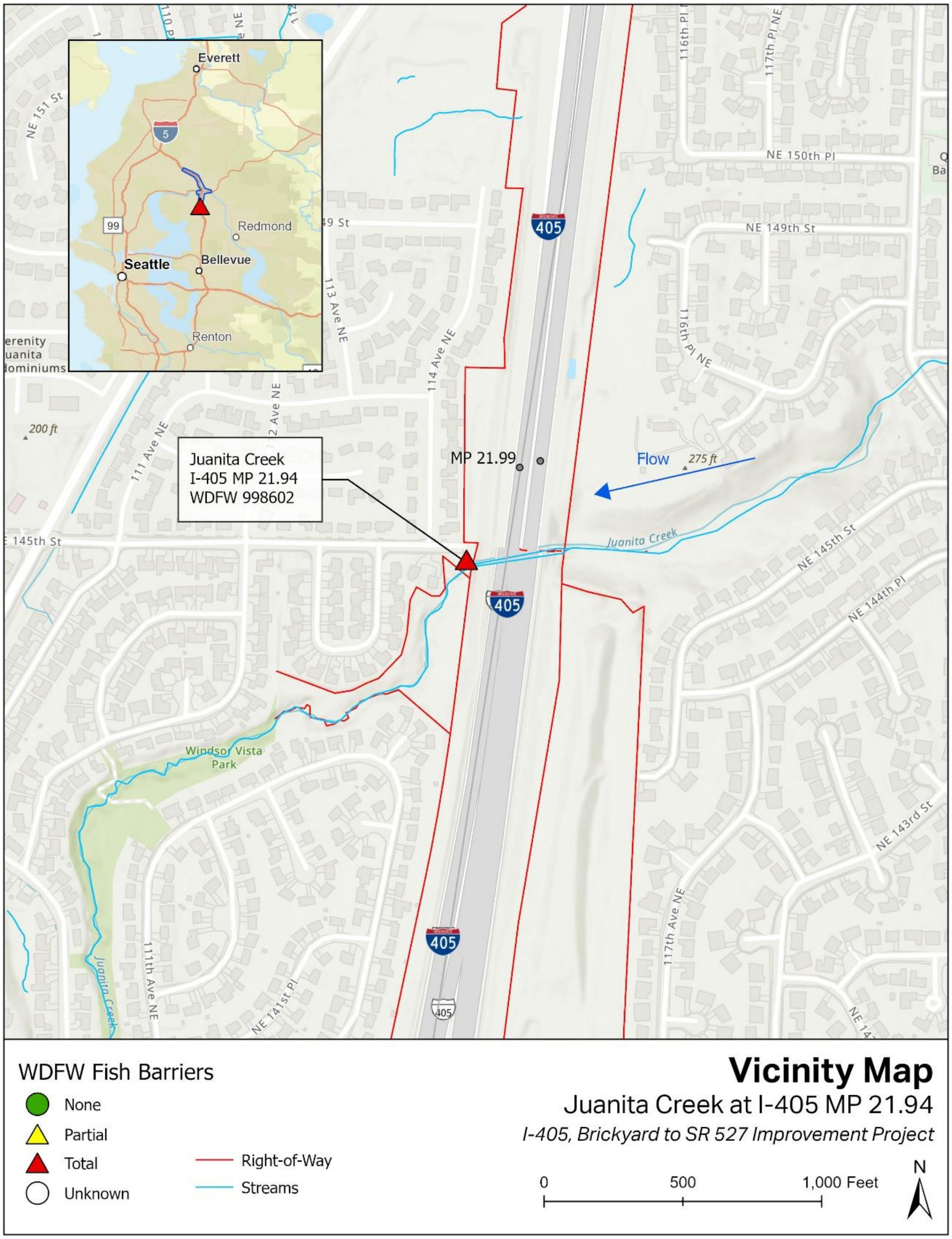


Figure 1-1: Vicinity map

2 Watershed and Site Assessment

The existing watershed was assessed in terms of land cover, geology, regulatory floodplains, fish presence, site observations, wildlife crossing priority, and geomorphology. Assessments were performed using a site visit and desktop research with resources such as the United States Geological Survey (USGS), Federal Emergency Management Agency (FEMA), and WDFW, along with past observations, maintenance, and fish passage evaluations.

2.1 Site Description

Juanita Creek contains flow year-round and originates approximately 0.5 miles east of I-405 in a forested area of low-density residential development. From its origin, the stream flows through two culverts located at 122nd Avenue NE and 119th Avenue NE. Downstream of the culvert at NE 122nd Avenue NE, the creek is conveyed in pipes until the outlet at 119th Avenue NE. Just downstream of 119th Avenue NE, the creek flows in a well-defined channel through a mixed forest valley. The stream crosses I-405 at MP 21.94 in a 48-inch corrugated steel pipe with a grated 4-foot concrete box (**Figure 2-1**). Juanita Creek outlets downstream of I-405 through a 48-inch corrugated steel pipe with a debris cage (**Figure 2-2**). Downstream of the crossing, the stream then flows through a vegetated area in the middle of a housing development dominated by Himalayan blackberry with some trees. An active beaver dam is present approximately 400 feet downstream of I-405 (**Figure 2-3**). The creek flows through a series of open channels and culverts (**Figure 2-4**), passing through residential areas, Edith Moulton Park, and commercial areas, until it finally reaches the northeast end of Lake Washington at Juanita Beach Park.

Figure 2-1: Inlet to Juanita Creek under I-405

Figure 2-2: Outlet to Juanita Creek

Figure 2-3: Beaver Dam downstream of I-405

Figure 2-4: Culvert downstream of the I-405 crossing

The existing culvert at I-405 is a complete barrier to fish because at the outlet of this crossing structure, a water surface drop of 2.62 feet impedes fish passage, in conjunction with the barrier at the inlet flow control structure. As a result of this inlet control structure detaining upstream flows, sediment and debris transport is inhibited crossing I-405 at this location. The total length of expect habitat gain is 1,309 linear feet.

The Juanita fish passage project location is not within a designated FEMA Flood Zone. The proposed structure is anticipated to improve upon the existing flooding conditions by replacing the existing regional detention facility with a fish passable structure. More discussion on floodplain impacts is provided in **Section 6**. Hydraulic modeling of existing conditions as part of preliminary analysis is included in **Section 5.2** and confirms the backwatered condition caused by this detention facility.

WDFW has identified 24 crossings between the I-405 crossing and Lake Washington, most of which have been identified as non-barriers for fish passage. Of these culverts near the project, two have been identified as complete barriers and five have been identified as partial barriers. The crossing at 119th Avenue NE upstream of I-405 is a full barrier, with the next upstream crossing at 122nd Avenue NE identified as partial barrier 431.76 (WDFW 2019a).

WSDOT Area 5 Maintenance was contacted to determine if there are ongoing maintenance problems at the existing structure due to large woody material (LWM) racking at the inlet or sedimentation. The maintenance representative indicated there was not a record of LWM blockage and/or removal or sediment removal at this crossing.

2.2 Watershed and Land Cover

Juanita Creek consists of approximately 0.5 miles of stream length upstream of the I-405 crossing and drains to the northeast end of Lake Washington approximately 2.9 miles downstream of the I-405 crossing. The contributing watershed upstream of the I-405 crossing has an area of 0.67 square miles, which receives annual precipitation of 41.2 inches (USGS 2019). Elevations within the contributing watershed range from 222 to 422 feet, referenced to the North American Vertical Datum of 1988 (NAVD 88). The watershed boundary shown in **Figure 2-5** was delineated using 2-foot contours generated from the 2021 King County Light

Detection and Ranging (LiDAR) dataset and City of Kirkland stormwater utility data. The 2021 King County impervious surface data layer (100-foot resolution; WGS 2023) was used to calculate existing impervious surface areas within the watershed. Land cover in the watershed, as documented by the 2021 National Land Cover database (NLCD 2021) and illustrated on **Figure 2-6** consists of a mix of impervious and pervious surfaces:

- Impervious, predominantly residential and commercial development (approximately 45.6 percent)
- Forested (approximately 10 percent)
- Other pervious surfaces (approximately 44.4 percent)

A summary of the land use for area surrounding Juanita Creek is provided in **Table 2-1**.

Table 2-1: Land cover

Land Cover Class	Basin Coverage (%)
Barren land	0.05
Deciduous forest	1.55
Developed, high intensity	8.74
Developed, low intensity	34.7
Developed, medium intensity	40.7
Developed, open space	5.79
Evergreen forest	6.52
Hay/pasture	0
Mixed forest	1.97
Open water	0
Shrub/scrub	0
Woody wetlands	0

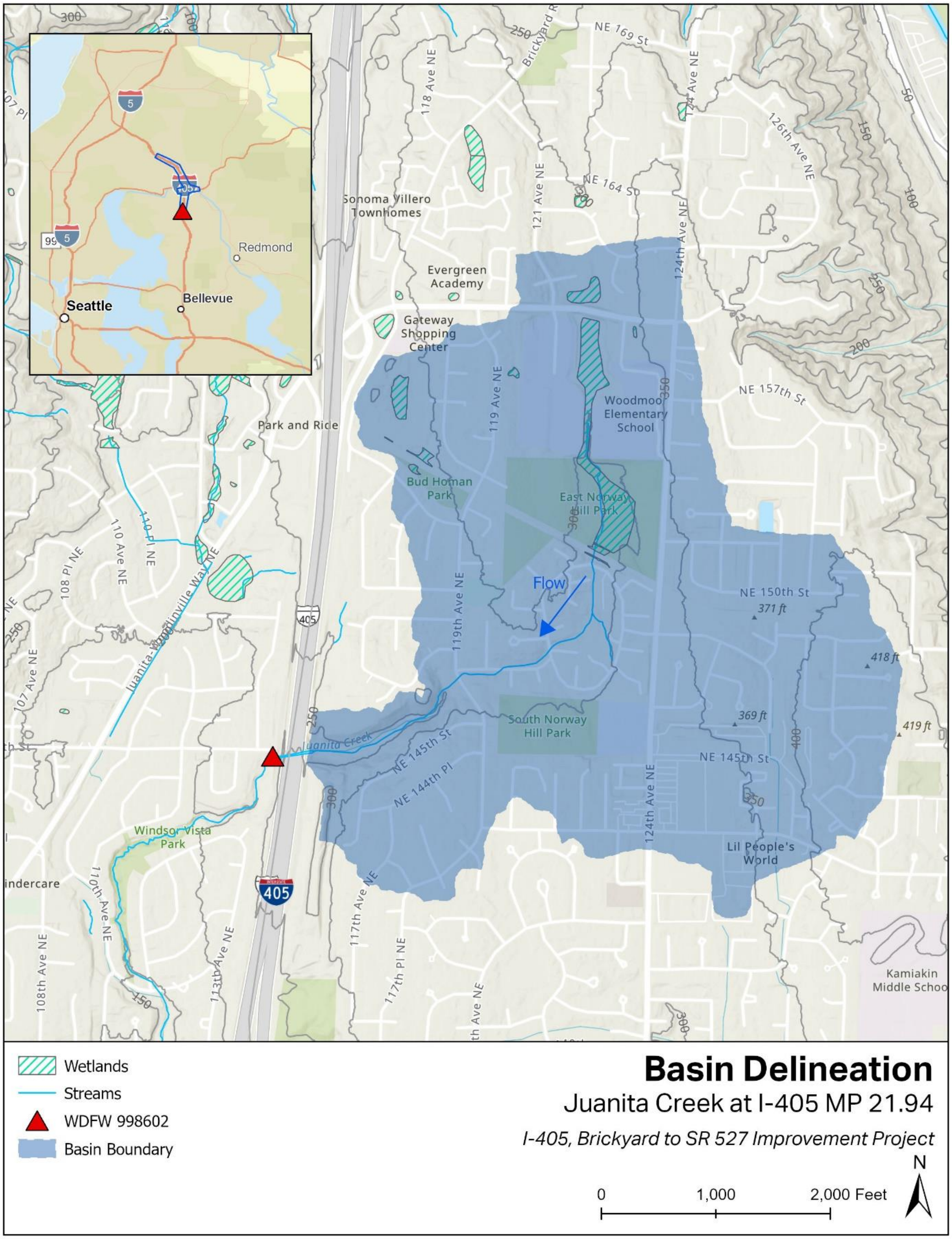


Figure 2-5: Watershed map

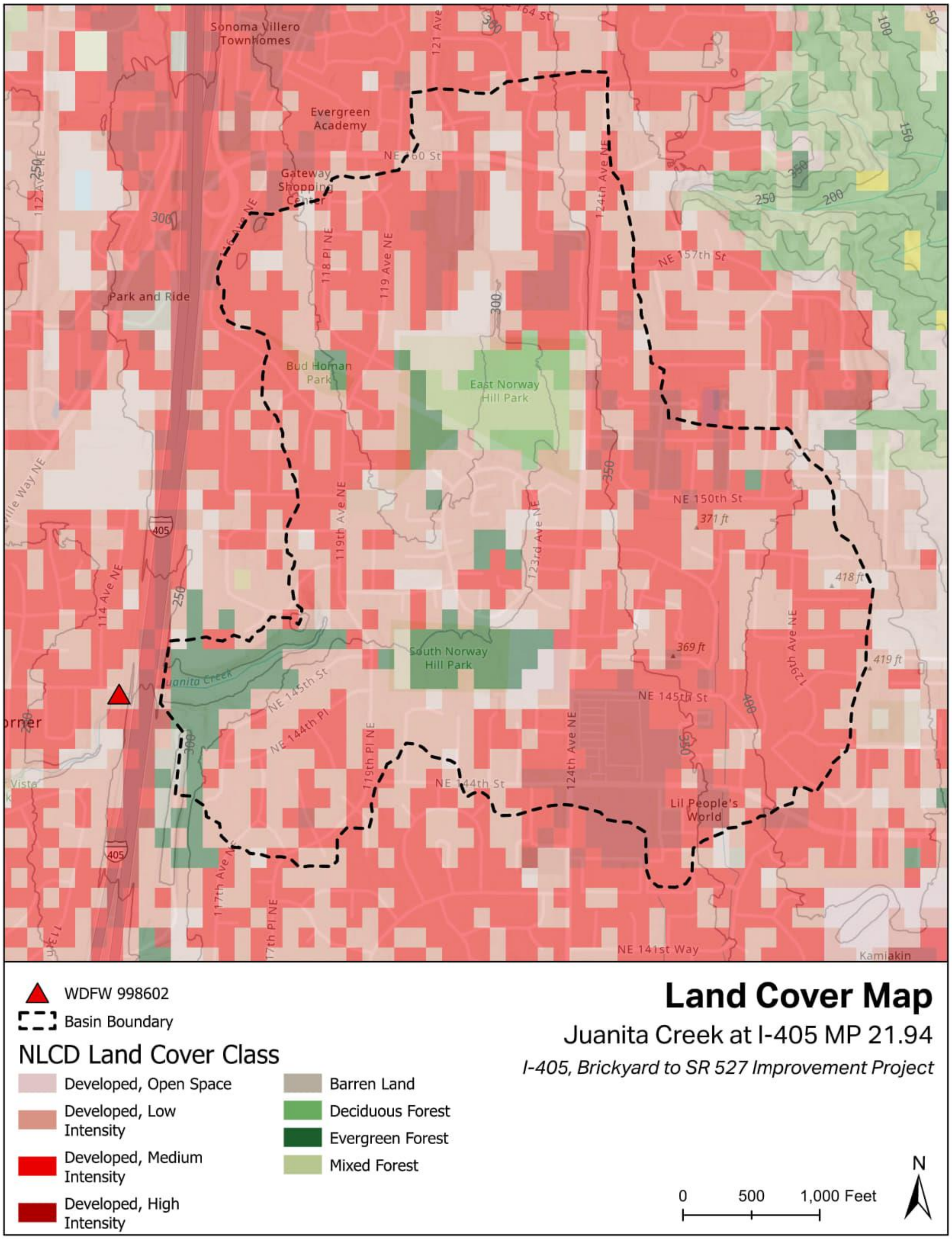


Figure 2-6: Land cover map (NLCD 2021)

2.3 Geology and Soils

Geology at the stream crossing was assessed using the USGS National Geologic Map Database (USGS 2022). USGS map (MF-1543) confirms that the area surrounding the Juanita Creek crossing is composed of advance outwash from the Vashon drift. The advance outwash underlies till and is typically composed of clean pebbly sand with an increasing gravel component closer to the ground surface. Some of the sediments associated with this outwash are stained by iron oxide precipitated from groundwater, indicating that the groundwater table in the area may be high. The soils are generally well drained and provide a stable foundation where not close to steep slopes (USGS 2023). **Figure 2-7** shows the geology in and around the Juanita Creek crossing.

Soil assessments conducted at the stream crossing were completed using United States Department of Agriculture's Natural Resources Conservation Service Gridded Soil Survey Geographic (gSURRGO) Database (USDA 2022). The predominant soil type found in the vicinity of the I-405 crossing is Alderwood gravelly sandy loam, a classification encompassing approximately 51.3 percent of the basin area. The specific soil variant can be attributed to historical outwash soils deposited from glacial activity and some alluvial soils, including Ragnar-Indianola association and Arents, Alderwood material upstream of the I-405 crossing, and Ragnar-Indianola association downstream of the crossing. **Figure 2-8** shows the soils map for the Juanita Creek watershed and crossing.

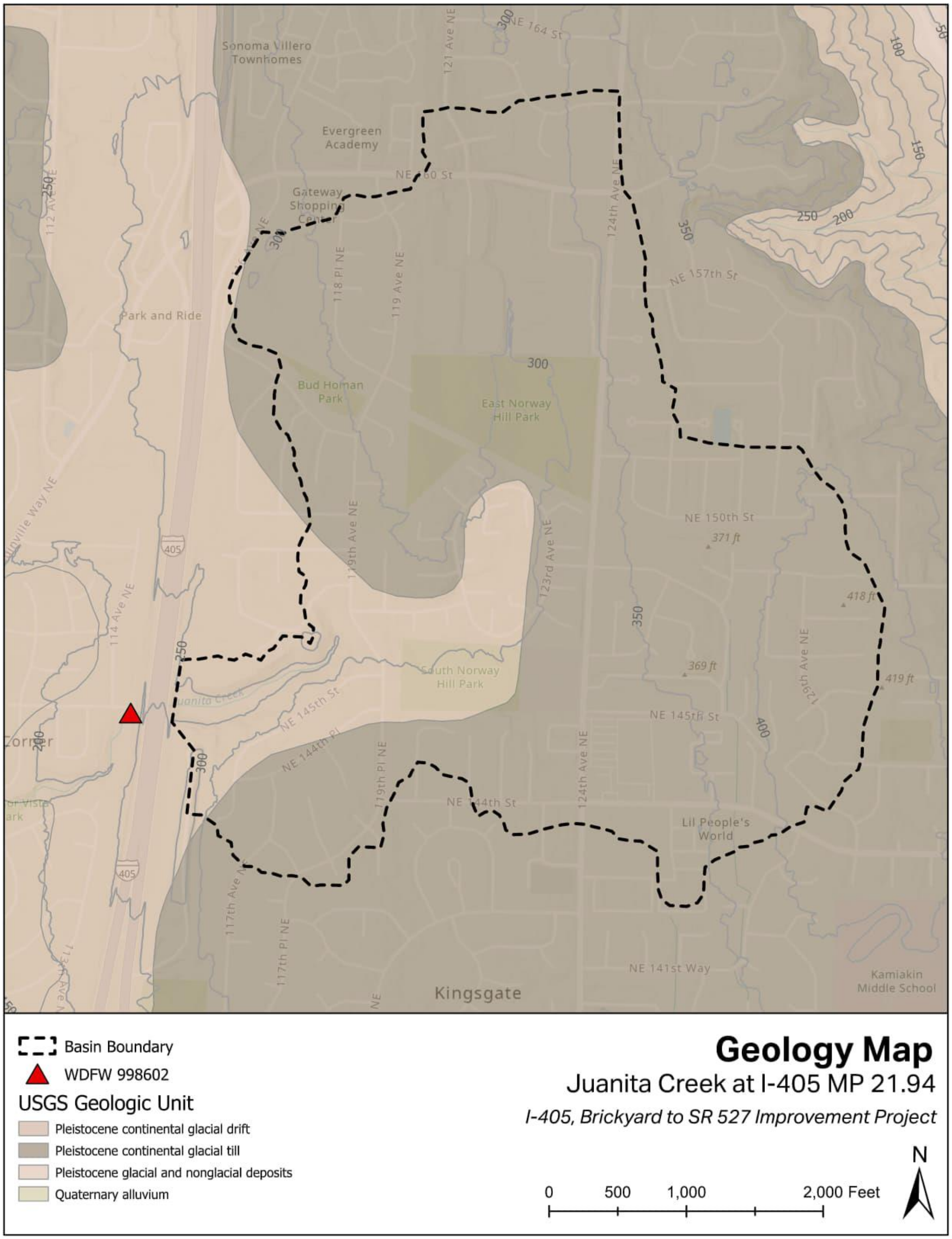


Figure 2-7: Geologic map

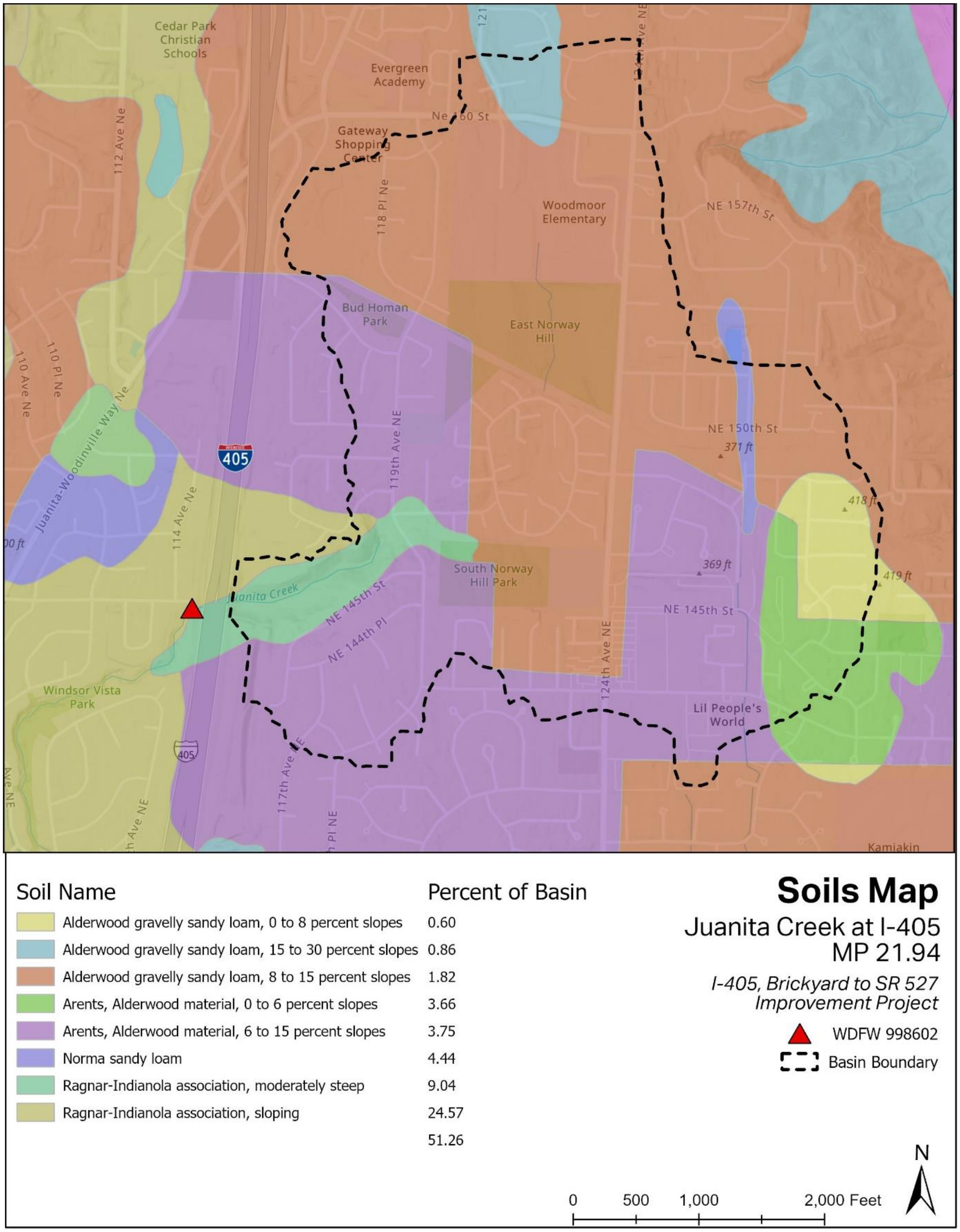


Figure 2-8: Soils map

2.4 Fish Presence in the Project Area

The WDFW online SalmonScape application documents sockeye salmon (*Oncorhynchus nerka*), coho salmon (*O. kisutch*), steelhead trout (*O. mykiss*), Chinook salmon (*O. tshawytscha*), and resident coastal cutthroat trout (*O. clarkii*) as present in Juanita Creek at the I-405 crossing. According to SalmonScape, coho salmon have been documented as spawning in Juanita Creek, and resident coastal cutthroat have been documented as present in the creek. Sockeye, Chinook, and steelhead are presumed present based on a lack of gradient barrier (WDFW 2019b). The Washington State Fish Passage mapping tool (WDFW 2019c) shows sockeye, coho, steelhead, and sea-run and resident cutthroat trout as potentially present in Juanita Creek based on suitable habitat.

Historically, Juanita Creek supported a small population of Chinook salmon throughout the basin (King County 2018a). According to the WDFW Spawning Ground Database (WDFW 2019e), Chinook salmon sightings have not been documented during stream surveys conducted annually since 1990. Spawning surveys conducted by King County did not identify any Chinook salmon adults spawning in Juanita Creek (King County 2002b). There are no available data confirming steelhead or sockeye presence in Juanita Creek, but presence is assumed based on the availability of suitable habitat.

Despite undesirable water quality conditions described in **Section 2.6.3** below (low dissolved oxygen and bacteria), migratory access is presumed to be the limiting factor for species to be found at the Juanita Creek I-405 crossing. **Table 2-2** shows the species potentially present within the project area.

Table 2-2: Native fish species potentially present within the project area

Species	Presence (Presumed, Modeled, or Documented)	Data Source	ESA Listing
Sockeye salmon (*Oncorhychus nerka*)	Presumed	SalmonScape (WDFW 2019b)	Not Warranted
Coho salmon (*O. kisutch*)	Documented	SalmonScape (WDFW 2019b)	Not Warranted
Steelhead trout (*O. mykiss*)	Presumed	SalmonScape (WDFW 2019b)	Threatened
Chinook salmon (*O. tshawytscha*)	Presumed	SalmonScape (WDFW 2019b)	Threatened
Resident coastal cutthroat trout (*O. clarkia*)	Documented	SalmonScape (WDFW 2019b)	Not Warranted

2.5 Wildlife Connectivity

Juanita Creek ranked medium priority for Ecological Stewardship and low priority for Wildlife-related Safety by WSDOT Headquarters (HQ) ESO. A wildlife connectivity memorandum will not be provided for this site, and additional width or height has not been recommended by WSDOT HQ ESO for wildlife connectivity purposes.

2.6 Site Assessment

The site was assessed through a desktop analysis, including a review of previously compiled information from WSDOT and through multiple site visits conducted by both WSDOT and AECOM.

2.6.1 Data Collection

WSDOT site assessments were conducted in June and December 2019 at the crossing within WSDOT right-of-way, as well as 300 feet upstream and downstream. On December 3, 2019, WSDOT, WDFW, and MITFD performed the site assessment at Juanita Creek to determine and concur on field-measured bankfull width, discuss possible alternatives for the proposed crossing, and observe existing site conditions. On December 3, 2023, AECOM completed a site assessment that included bankfull width measurements and existing site condition observations. Ten bankfull width measurements were taken in 2019, and three bankfull width measurements were taken in 2023. See **Section 2.7.2** for the bankfull width measurement summary. Three Wolman pebble counts were completed in 2019 near where bankfull widths were measured and specifically at the locations indicated in **Figure 2-9**. The pebble counts are discussed in **Section 2.7.3**. WSDOT performed stream surveys in October 2019 to gather the necessary survey data to build a surface model for analysis and design. The survey extended upstream approximately 300 feet to an existing pedestrian bridge of Juanita Creek and extended downstream approximately 700 feet. An additional stream survey was conducted in February 2024 to capture updated stream geometries and elevations in order to ensure the preliminary design ties into the existing stream and ensure accuracy of the hydraulic model. The WSDOT field report for WDFW Site ID 998602 is presented in **Appendix B**.

Figure 2-9: Reference reach, bankfull width, and pebble count locations

2.6.2 Existing Conditions

Juanita Creek, originating east of I-405, flows southwest through commercial development before entering forested areas. The stream features riffle-pool habitat and is bordered by red alder (Alnus rubra), bigleaf maple (Acer macrophyllum), western red cedar (Thuja plicata), and Douglas fir (Pseudotsuga menziesii).

Based on the WDFW *Fish Passage and Diversion Screening Inventory Database Report* provided in **Appendix J** (WDFW 2019d), the existing culvert at the I-405 crossing is a 358.6-foot-long corrugated metal pipe (CMP) with a slope of 3.0 percent. There is a water surface drop of 2.62 feet at the outlet, and there is no plunge pool present. At the outlet, the culvert downstream daylights onto rocks, which creates white water conditions and dissipates the energy (**Figure 2-2**). There is no apron on either the upstream or the downstream end. The culvert is not countersunk and does not contain sediment throughout its length. The culvert is not backwatered and does not slow the flow of water through its length. There is approximately 40 feet of road fill atop the culvert. The existing culvert is a complete barrier to fish because of the 2.6-foot water surface drop (WDFW 2019a).

There is an inline detention facility located just upstream of I-405 that influences channel shape and riparian and floodplain conditions. In this area, the stream channel is 5 to 7 feet wide and 1 foot deep. This detention facility area has a poorly defined channel where water spreads out and is very shallow. During a recent site visit, backwater was observed on the upstream side of the culvert, and the inlet was completely submerged. No spawning or rearing habitat is present within the first 170 feet of open channel upstream of the crossing. Just upstream of the poorly defined and shallow channel, the stream has incised down through accumulated sediments that were deposited due to the detention facility. Because of this incision, this reach of the stream is straight and offers minimal habitat for fish. Sediment substrate is dominated by fines and provides no benefit to fish in its current state.

Downstream of the crossing, the stream is dominated by angular riprap that provides no immediate fish habitat benefits. The riprap extends to the next downstream barrier, two property access culverts that are located approximately 70 feet downstream. No beneficial spawning or rearing habitat is present immediately downstream of the crossing.

2.6.3 Fish Habitat Character and Quality

There is no open channel habitat within the I-405 right-of-way. Upstream of the I-405 crossing, the stream flows through an inline detention facility that alters the natural channel characteristics and affects its connectivity to the floodplain and riparian area. Substrates are mostly fine materials. Downstream, the creek is confined to a narrow flow path between a housing development and the I-405 right-of-way.

Upstream of the I-405 culvert, Juanita Creek flows through a mixed forested area that is dominated by riffle and pool habitat and a wide vegetated riparian zone. As the stream flows through the inline detention facility, floodplain connectivity and riparian habitat are negatively affected. Downstream, the creek flows through a vegetated area in the middle of a housing development. This area is dominated by Himalayan blackberry (*Rubus armeniacus*), but the stream does receive some terrestrial nutrient input and shade. Substrates remain fine materials. A WDFW habitat survey detected juvenile salmonids throughout this downstream reach. According to residents, salmon and lamprey (*Petromyzontiformes)* are often seen downstream of the detention basin. Throughout its flow path, the channel has been armored in most segments of the stream. Channel complexity, floodplain connectivity, and riparian condition and function are heavily degraded due to bank armoring. There appears to be no habitat features that are likely to be used by fish species at any life stage in the current configuration.

Washington Department of Ecology (Ecology) lists impaired water bodies that do not meet water quality standards, which is referred as the 303(d) list. Juanita Creek is identified on the 303(d) list for dissolved oxygen and bacteria (WSDOE 2019). According to the City of Kirkland watershed report card (City of Kirkland 2023), the average temperature for Juanita Creek remains below 60°F throughout the year; however, low concentrations of dissolved oxygen have been recorded during summer months, and 24 pesticides were detected in the system.

2.6.4 Riparian Conditions, Large Wood, and Other Habitat Features

There are several pieces of LWM upstream of the I-405 crossing. The Juanita Creek system has good LWM recruitment potential in this area, as it flows through a steep wooded ravine.

Downstream of the crossing, the stream has some LWM available as it flows through short reaches with vegetated riparian habitat (**Figure 2-10**).The riparian buffer, while steep, is wide and intact (**Figure 2-11**). In general, however, the downstream reach is dominated by a developed riparian buffer with little LWM or potential for recruitment.

Figure 2-10: LWM downstream of I-405

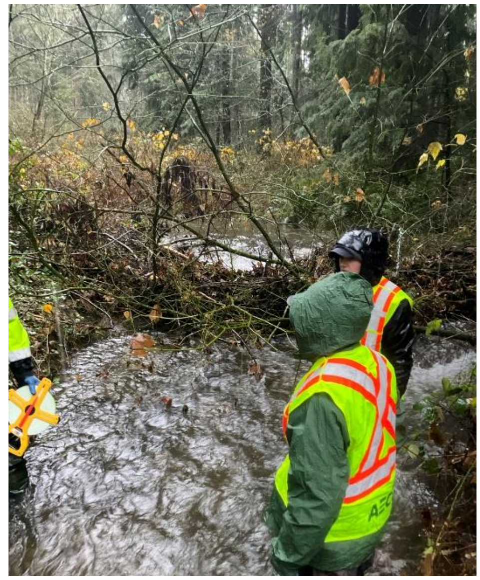


Figure 2-11: Riparian area upstream of I-405

2.7 Geomorphology

The following sections describe the geomorphic characteristics of Juanita Creek. The discussion of geomorphic information includes reference reach selection, channel geometry, sediment, and channel stability in both the vertical and lateral directions. The geomorphic characteristics of the channel were observed during the 2019 WSDOT and 2023 AECOM field investigations.

2.7.1 Reference Reach Selection

Upstream of the I-405 crossing, Juanita Creek is located in a forested reach within a drainage basin that is highly urbanized. The highly urbanized nature of the basin makes it difficult to find a reach that is representative of the historical natural conditions of the stream. A reference reach approximately 300 feet upstream of the crossing was selected (**Figure 2-9** and **Figure 2-12**). This reach is above the influence of the backwater caused by the existing inline detention facility and has similar gradient to the section of stream at the crossing. The gravels found in this reach are consistent with the materials found in the adjacent stream banks and are likely representative of the sediments routinely transported by the channel.

Figure 2-12: Reference reach, looking upstream

2.7.2 Channel Geometry

The reference reach channel can be described as a relatively confined, moderately sinuous urban stream. Historical aerial imagery suggests it is generally stable and is not subjected to extreme lateral migration. Minimal upstream channel incision is apparent, but some bank erosion is present, which is representative of the majority of lowland Washington streams. Existing cross sections are generally 1.0–1.5 feet in depth with steeper banks and are gently sloped down to the channel thalweg. The longitudinal slope of the reference reach was determined via a desktop investigation to be approximately 1.7 percent. **Figure 2-13** shows existing cross-section geometries upstream of the I-405 crossing.

Bankfull width measurements for Juanita Creek were determined in the field on December 3, 2019, by WSDOT, WDFW, and MITFD. Bankfull width measurements were obtained from 10 locations upstream of the I-405 crossing. These measurements were taken approximately 32.8 feet apart. Measurements upstream of I-405 ranged from 9.5 feet to 17.5 feet. Bankfull measurements are shown in **Table 2-3**. AECOM's three measurements were not included in the design because the average of 12.0 feet and the minimum hydraulic opening of the proposed structure of 23 feet were concurred with previously by WSDOT and other parties; however, AECOM's measurements confirm that the channel geometry has not altered greatly from 2019 to 2023.

Table 2-3: Bankfull width measurements

BFW Number	Width (ft)	Included in Design Average?	Site Assessment
1	10.5	Yes	WSDOT 2019
2	14.5 (12.8)	Yes (No)	WSDOT 2019 (AECOM 2023)
3	13.3 (11.6)	Yes (No)	WSDOT 2019 (AECOM 2023)
4	11.5 (10.4)	Yes (No)	WSDOT 2019 (AECOM 2023)

BFW Number	Width (ft)	Included in Design Average?	Site Assessment
5	10.0	Yes	WSDOT 2019
6	9.5	Yes	WSDOT 2019
7	11.5	Yes	WSDOT 2019
8	11.0	Yes	WSDOT 2019
9	17.5	Yes	WSDOT 2019
10	11.5	Yes	WSDOT 2019
Design Average	12.08		

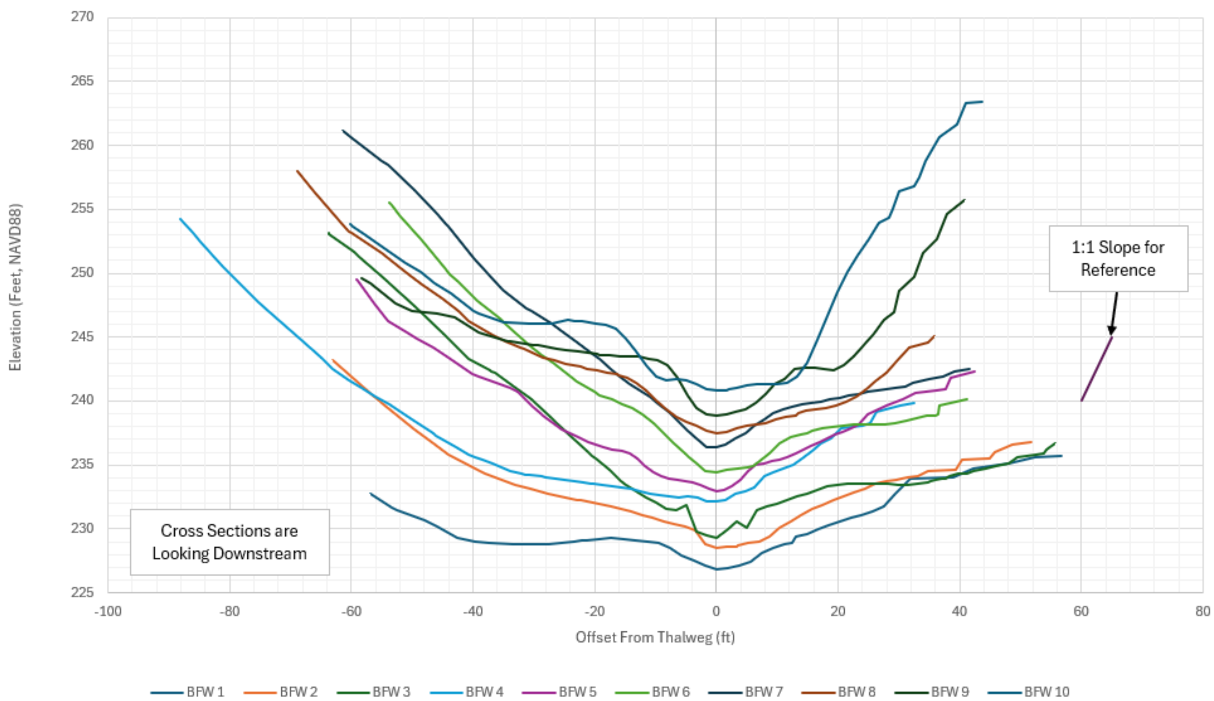


Figure 2-13: Existing cross-section examples

The bankfull widths reported in **Table 2-3** are understood to reflect a moderately confined channel in a forest reach of a highly urbanized basin. In coordination with WSDOT, WDFW and MITFD, a bankfull width of 12.0 feet has been accepted as the basis for completing the preliminary restored stream connection design.

2.7.2.1 Floodplain Utilization Ratio

The floodplain utilization ratio (FUR) is defined as the flood-prone width (FPW) divided by the bankfull width. The FPW is the water surface width at twice the bankfull depth, or the width at the 50-year to 100-year flood. A ratio under 3.0 is considered a confined channel and above 3.0 is considered an unconfined channel. The FUR was determined and confirmed by WSDOT using measurements upstream in a location that was outside of the culvert influence, and further detail was not necessary. The average FUR value upstream of the crossing was determined to be 2.11, which is considered unconfined. See **Table 2-4** for a summary of these metrics.

Table 2-4: FUR determination

Metric	Value
Average FUR Near Crossing	2.11
Bankfull Width (ft)	12.08
Slope Ratio	1.25

2.7.3 Sediment

On February 28, 2020, a field investigation of the sediment distribution upstream of Juanita Creek was conducted. Three Wolman pebble counts were conducted in the reference reach. Sediment in this valley is largely immobile due to the existing inline detention facility. Fines have accumulated over the years at the existing structure inlet. As shown on **Figure 2-14,** sediment included a wide distribution of gravels mixed with a smaller distribution of fines and cobbles (**Table 2-5**). The gradation curves for these pebble counts are found in **Figure 2-14**.

Table 2-5: Sediment properties near the project crossing

Particle Size	Pebble Count 1 Diameter (in.)	Pebble Count 2 Diameter (in.)	Pebble Count 3 Diameter (in.)	Average Diameter for Design (in.)
Included in average?	Yes	Yes	Yes	
$\mathbf{D_{16}}$	0.4	0.3	0.5	0.4
$\mathbf{D_{50}}$	0.9	0.9	1.3	1.0
$\mathbf{D_{84}}$	1.9	2.1	2.6	2.2
$\mathbf{D_{95}}$	3.5	3.3	3.5	3.5
$\mathbf{D_{100}}$	5.9	4.9	6.0	5.7

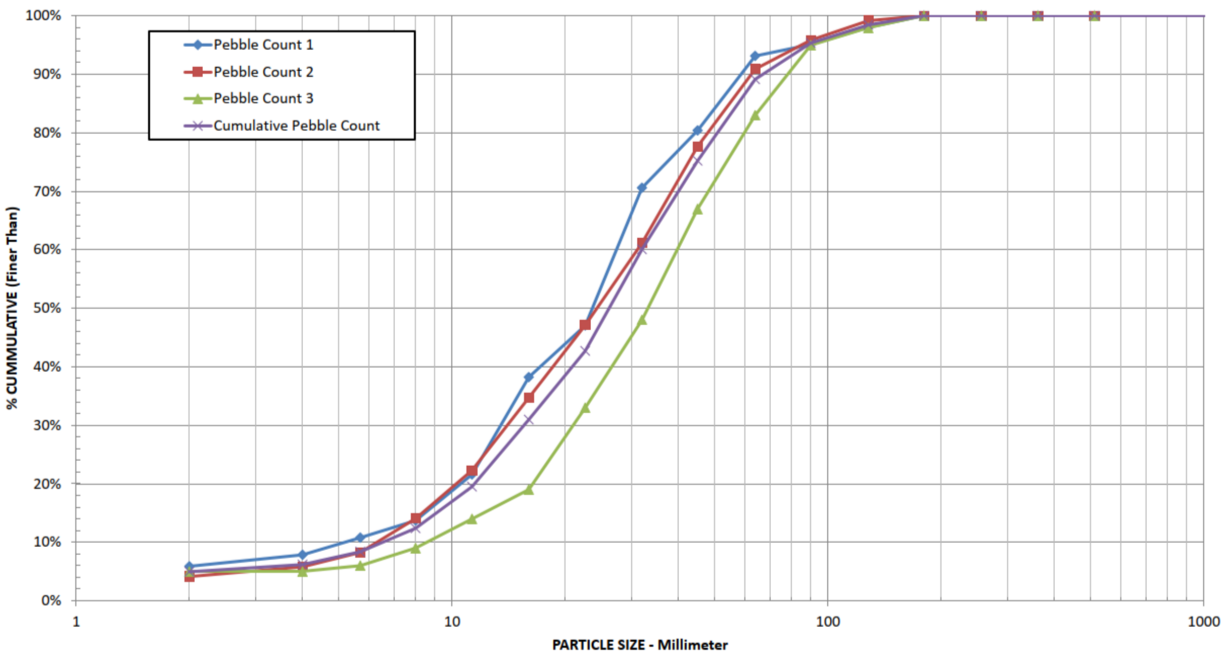


Figure 2-14: Sediment size distribution

Two grab samples were collected on December 3, 2023, one from the downstream side of I-405 located on the right bank of the outlet of property access culvert looking downstream and one from the upstream side of I-405 on the right bank looking downstream. The sediment samples were used for particle size distribution (ASTM D422) and liquid and plastic limits (ASTM D4318) tests completed by Otto Rosenau & Associates, Inc. The sediment samples consist of gray silty sand with trace organics. The particle size distributions can be found in **Appendix N.**

2.7.3.1 Vertical Channel Stability

This reference reach of Juanita Creek has a moderately low slope (approximately 2 percent) and appears to have minimal risk of major aggradation or degradation. There is some incision and erosion identified downstream of the I-405 crossing (**Figure 2-15**), but this does not appear to be a major issue within the project area.

Figure 2-15: Signs of erosion downstream of I-405

The longitudinal profile in **Figure 2-16** consists of LiDAR acquired from King County (WGS 2021) that has been supplemented with the survey completed in 2024. It shows the project location with the proposed crossing slope relative to the exiting ground elevations beginning at Lake Washington and extending upstream of I-405.

Sediment supply in the system is likely limited by the upstream culvert crossings. There is some potential in the immediate valley reach for sediment recruitment, but the presence of the undersized culverts upstream limits the ability for the sediment from further upstream in the system to migrate and travel through this reach. The I-405 crossing sits in a transition zone where the slope changes from steep to flat as Juanita Creek flows towards Lake Washington (**Figure 7-1**).

Aggradation is occurring in the current configuration due to the regional detention facility, but as this control structure is removed, the potential for aggradation upstream will be reduced. The proposed crossing will connect the upstream and downstream ends of the project with an improved channel alignment that maintains a slope of approximately 3 percent as discussed in **Section 4.1.3.**

As described in **Section 7.2**, Juanita Creek may be at risk for long-term degradation under proposed conditions because the Project would replace the existing I-405 culvert with a deformable channel bed. Preliminary equilibrium slope estimates and the potential for bed armoring were evaluated and indicate that degradation is expected to be minimal in the proposed condition.

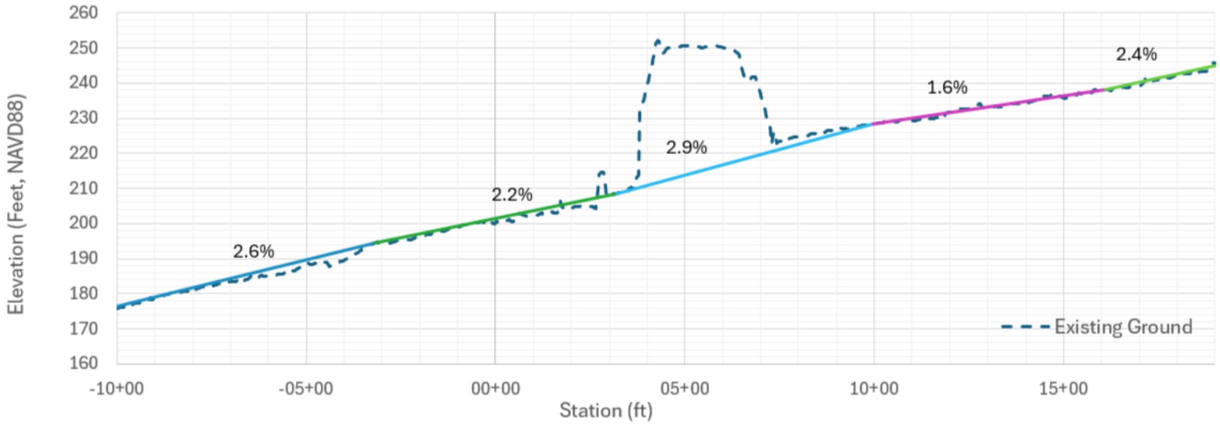


Figure 2-16. Watershed Longitudinal Profile

2.7.4 Channel Migration

Upstream of the I-405 crossing, Juanita Creek flows through a forested ravine with low to moderate channel migration potential until it reaches the inline detention facility. There is also limited channel migration potential immediately downstream of the crossing, as the stream flows through a narrow, vegetated area within a housing development. Further downstream, the creek continues to flow through pipes and developed areas with little opportunity for significant channel migration.

3 Hydrology and Peak Flow Estimates

WSDOT recognizes climate resilience as a component of the integrity of its structures and approaches the design of bridges and buried structures through a risk-based assessment beyond the design criteria. The largest risk to bridges and buried structures will come from increases in flow and/or sea level rise. The goal of fish passage projects is to maintain natural channel processes through the life of the structure and to maintain pass-ability for all expected life stages and species in a system.

WSDOT evaluates crossings using the mean percent change in 100-year flood flows from the WDFW Future Projections for Climate-Adapted Culvert Design program. All sites consider the projected 2080 percent increase throughout the design of the structure. **Appendix G** contains the projected increase information for the project site. The design flow for the crossing is 111 cubic feet per second (cfs) at the 100-year storm event. The projected increase for the 2080 100-year flow is 21.8 percent, yielding a projected 2080 100-year flow of 135 cfs.

In May 2021, WSDOT completed the preliminary Hydraulic Design (PHD) report, which details the design elements necessary to comply with the federal injunction as coordinated with the permitting agencies and affected tribe. Following the completion of the PHD Report, WSDOT coordinated with the City of Kirkland and its consultant, Northwest Hydraulic Consultants (NHC), to better understand the basin hydrology and floodplain impacts for existing and proposed conditions (City of Kirkland 2022). A technical memorandum documents the hydrology to be used for final design and the finding for potential downstream effects that result from the removal of the flow control structure (see **Appendix Q** for details). Based on NHC's field inspections, the original design ponding depths do not appear to match the actual performance within the regional pond. To address this, the City of Kirkland used current GIS data to update existing land cover for the study area and contracted NHC to perform a calibration of a 2012 King County Hydrological Simulation Program – FORTRAN (HSPF) model for Juanita Creek. NHC provided four model outputs for use in the design based on their updated and calibrated King County HSPF model. Their results, documented in **Table 3-1,** will be used as the basis for the final design directed by the City of Kirkland and WSDOT. The upstream subbasin of the I-405 culvert is called the "At I-405" and the "Above Norway Hill Trib" subbasin is one of the downstream subbasins that provides lateral flows to Juanita Creek. The 500-year discharge was not provided by this updated HSPF model but was extrapolated using the 2-to-100-year outputs. Also, the 2080 projected 100-year discharge was determined from the predicted percentage increase obtained from WDFW's Culvert and Climate Change web application.

Table 3-1: Peak flows for Juanita Creek at I-405

	Project Scenario	2-year	10-year	25-year	100-year	Extrapolated 500-year	2080 Projected 100-year
At I-405	Pre	25	32	35	41	51	50
	Post	40	65	81	111	174	135
Above Norway Hill Trib	Pre	30	39	44	51	65	62
	Post	45	74	94	129	205	157

4 Water Crossing Design

This section describes the water crossing design developed for I-405 MP 21.94 Juanita Creek, including channel design, minimum hydraulic opening, and streambed design.

4.1 Channel Design

The WCDG contains methodologies for five different types of crossings: No-Slope Culverts, Stream Simulation Culverts, Bridges, Temporary Culverts or Bridges, and Hydraulic Design Fishways (Barnard et al. 2013). The permanent federal injunction allows for the use of the stream simulation method and bridge design method unless extraordinary circumstances exist on site. According to the WCDG, the stream simulation method should be considered for a site if the following conditions are met:

- the FUR is greater than 3.0,
- the stream has a bankfull width of 10 to 15 feet,
- the channel is believed to be moderately stable, and
- the channel is believed to be moderately debris prone

This section describes the channel design developed for Juanita Creek, including channel planform and shape, alignment, and gradient.

4.1.1 Channel Planform and Shape

The channel shape was based on reference reach channel information described and shown in **Section 2.7.2,** with some engineering judgment modifications. The meander wavelengths of the original WSDOT PHD stream design were measured and compared to stable channel planforms based on various equations recommended by NRCS (2007). All meanders were measured to be within acceptable lengths, except the meander at the entrance to the proposed crossing; this meander was straightened because it was measured to be less than an appropriate length, as recommended by Soar and Thorne (2001) (see **Appendix O**).

The final riffle, left pool, and right pool typical cross sections conserve the same 12-foot bankfull width. The riffle typical section includes cross slopes of 8H:1V between the thalweg and toes of the banks, to increase depths for fish passage at low-flows fish passable channel, and bank slopes of the 2H:1V (see **Figure 4-1** and **Appendix O**). SMS modeling showed that 2-year water surface elevation (WSE) is spilling outside of the bankfull channel as desired.

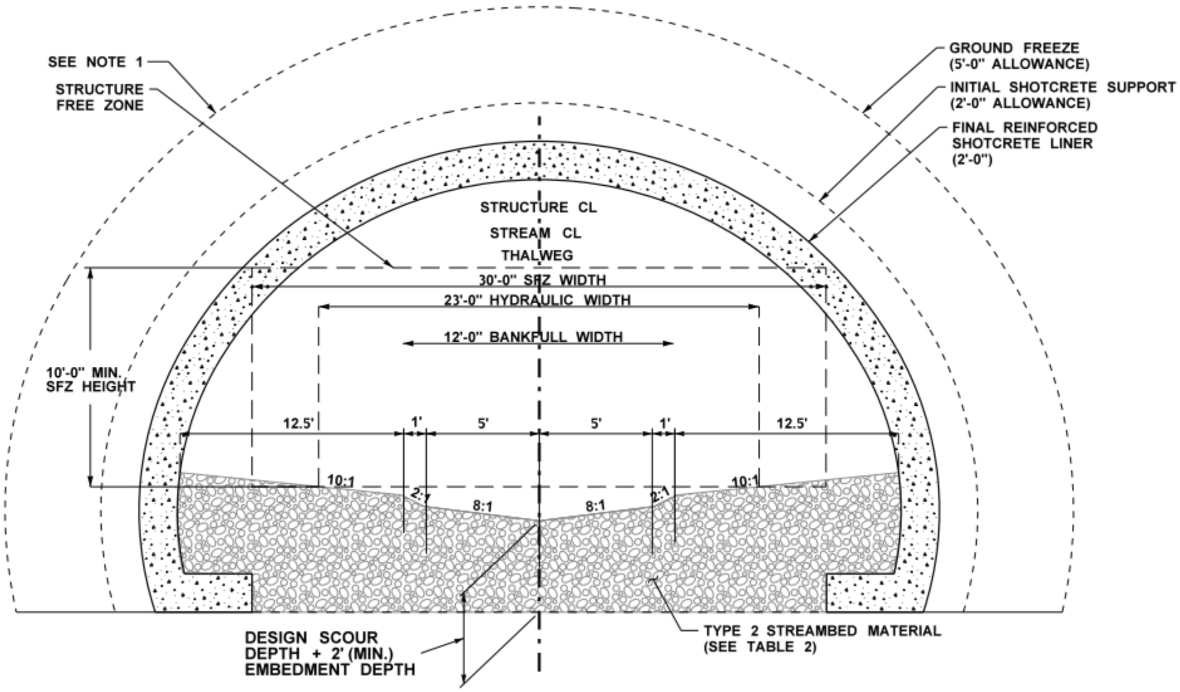


Figure 4-1: Design riffle cross section

The left and right typical pool sections are mirror images. As shown in **Figure 4-2,** pool bank slope will vary, but will not exceed a maximum slope of 2H:1V. Typical pool sections have a bankfull depth of 2.4 feet, providing a residual pool depth of approximately 1.3 feet below the elevation of the tailwater control at the downstream riffle crest thalweg.

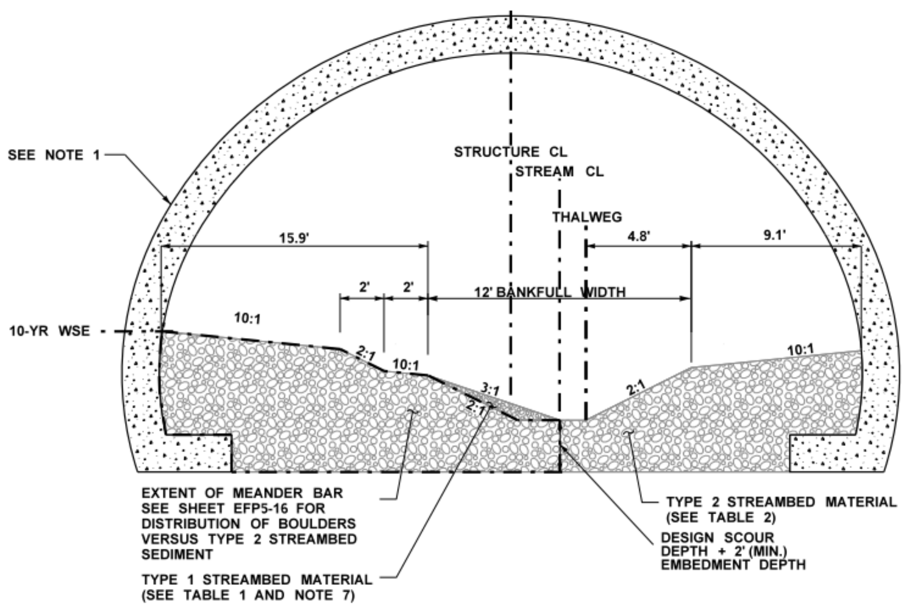


Figure 4-2: Design right pool cross section

Juanita Creek's existing channel upstream of I-405 flows into a stormwater detention facility surrounded by wetland vegetation. The stormwater detention facility drains through a 359-foot-long 48-inch CMP under I-405 to its downstream channel. The downstream channel flows for 70 feet and then enters two 25-foot-long, 24- and 36-inch culverts (both of which will be removed) and then flows into its channel again. The 48-inch CMP will be abandoned and replaced with a 37-foot-wide clear span shotcrete-lined tunnel offset 50 feet south from the existing culvert.

Upstream of the Juanita crossing, the existing channel will be preserved and used as high flow fish habitat. The proposed channel is lower and offset from the existing channel and existing 48-inch CMP crossing under I-405. The proposed channel's right bench will extend at a 10H:1V slope up to the existing channel. This right bank will vary 10 to 45 feet and be vegetated with wetland plants. The left bank is near a valley wall and will not have a bankfull bench. It will extend at a 2H:1V slope up to existing ground.

Through the proposed Juanita crossing, the left and right banks have benches at 10H:1V slopes. These benches vary between 5 to 13 feet wide based on the location of the meandering channel's centerline.

Downstream of the Juanita crossing, the existing channel will be filled to an elevation above the 100-year WSE to reduce flooding risk to adjacent properties. The downstream proposed channel's right bank has a 10-foot bench at 10H:1V side slopes. The left bank does not have a bench and ties into existing surface at 2H:1V slopes.

Updated 2024 existing stream survey was included in this phase of the modeling. A MicroStation InRoads corridor model was created with the typical cross sections at riffles and pools carving the proposed channel and benches. Channel meandering within the proposed alignment will provide increased habitat diversity. These meanders will be maintained through construction of meander bars and placed LWM that will enhance the instream channel diversity and encourage the stream to self-maintain this habitat over its design life. Step-pools will be incorporated upstream of the Juanita crossing in the next phase of design, as the current riffle slopes are undesirably steep for the system. Additional focus will also be placed on the existing abandoned channel's connections through the right bank log jam and the step-pools. This focus will prevent the abandoned existing channel's design from becoming a fish barrier.

The proposed typical cross sections create a low-flow channel with steeper 8H:1V slope on the riffle and "v" shaped pool cross sections. These changes will connect habitat features so the project does not produce a low-flow barrier. Additional input directed by the engineer in the field will be needed to ensure this low-flow channel is built. **Figure 4-3** illustrates the proposed channel section.

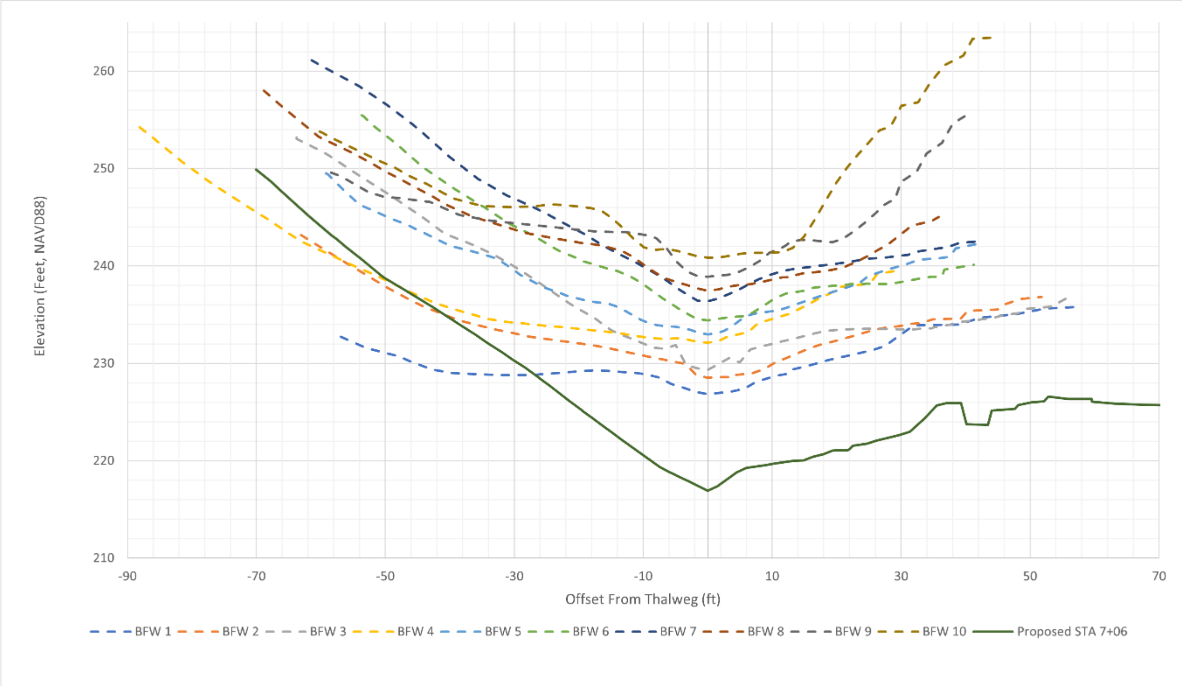


Figure 4-3: Proposed cross section superimposed with existing survey cross sections

4.1.2 Channel Alignment

The proposed channel alignment is approximately 697 feet in length, incorporating meanders to enhance habitat and mimic stream sinuosity. A factor that influenced the horizontal alignment of the proposed crossing included keeping the existing inline detention facility on-line during construction. A lateral separation from the existing structure was deemed necessary in the event that the City of Kirkland could not decommission the inline detention facility during construction due to their municipal stormwater treatment commitments. The meander configuration of the proposed alignment was driven by meander belt width data collected from the upstream reference reach and slightly smoothed out inside the crossing. The upstream reference reach meander belt width was measured to be 30 feet (**Figure 2-9**).

4.1.3 Channel Gradient

The WCDG recommends that the proposed culvert bed gradient not be more than 25 percent steeper than the stream gradient upstream of the crossing (WCDG Equation 3.1, Barnard et al. 2013). The slope ratio for the crossing and immediate upstream grading is shown in **Table 4-1**. The first 100 feet of existing stream slope upstream of the tie-in location exhibits an average slope of 2.4 percent. The roadway crossing of Juanita Creek was constructed at the location of a natural grade break, where a steep confined valley begins to flatten. As a result, minimal headcutting upstream of the proposed structure is expected. There is potential for long-term degradation of the channel bed, as discussed in **Section 7.2**.

Table 4-1: Channel gradient

Crossing location	Approximate Upstream Channel Grade	Proposed Structure Grade	Slope Ratio
I-405	2.4% (Existing)	3.0%	1.25
	3.0% (Proposed)		1.0

4.2 Minimum Hydraulic Opening

This section describes the minimum hydraulic width and vertical clearance; for discussion of the scour elevation, see **Section 7**. See **Figure 4-4** for an illustration of the minimum hydraulic opening.

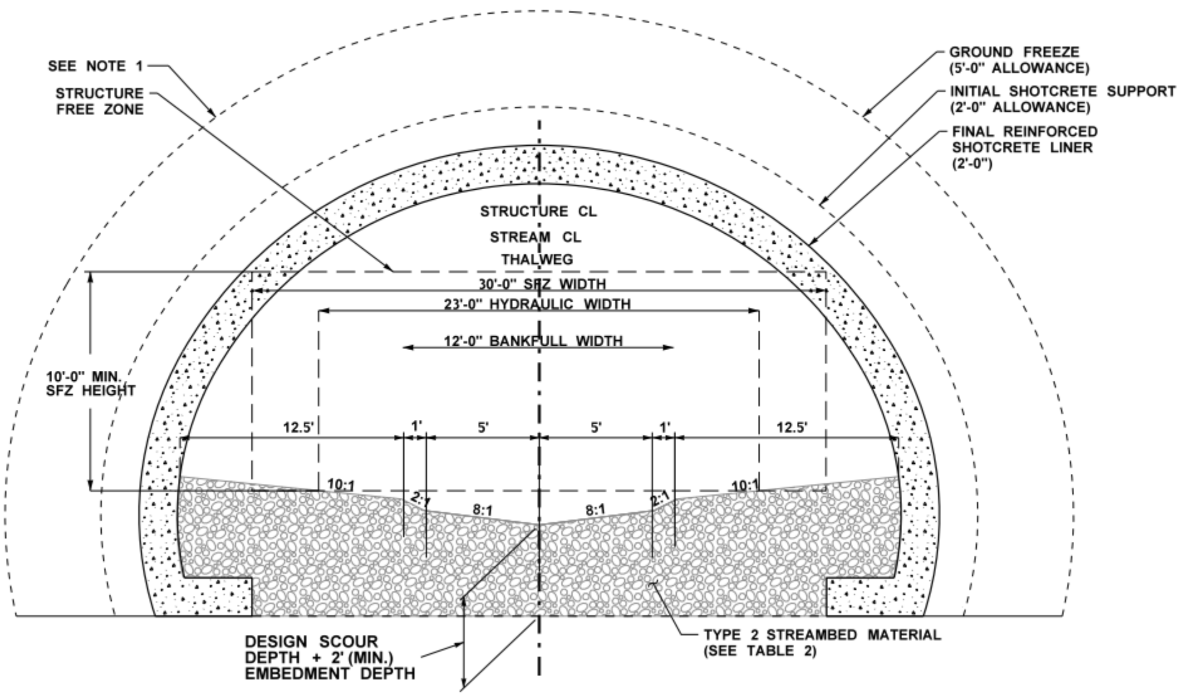


Figure 4-4: Minimum hydraulic opening illustration

4.2.1 Design Methodology

The proposed fish passage design was developed using the WCDG (Barnard et al. 2013) and the WSDOT *Hydraulics Manual* (WSDOT 2023a). Using the guidance in these two documents, the stream simulation design method was determined to be the most appropriate at this crossing. The following metrics aided in this conclusion:

- Bankfull width – 12 feet
- FUR – 2.11 (upstream), 1.31 (downstream)
- Slope ratio – maximum 1.25
- Length of crossing – 250 feet
- Channel stability – moderate

- Channel migration potential – low to moderate
- Climate Resiliency – negligible increase in WSEs

4.2.2 Hydraulic Width

The hydraulic width is defined as the minimum width perpendicular to the structure centerline that is necessary to convey stream design flow and allow for stream processes.

The starting point for the minimum hydraulic width determination of all WSDOT crossings is Equation 3.2 of the WCDG, rounded up to the nearest whole foot. For this crossing, a minimum hydraulic width of 17.0 feet was determined to be the minimum starting point as:

$$(1.2 * BFW) + 2 = 16.5 = 17.0$$

However, because this crossing is defined as a long culvert by WDFW (length-to-width ratio > 10), the required minimum hydraulic width is increased by 30 percent, to 23 feet.

Note that the structure free zone (SFZ) width is different from the hydraulic width. The SFZ width is the horizontal width perpendicular to the stream between the nearest surfaces of the structure. To allow for the implementation of meander bars and LWM, a SFZ width of 30 feet was agreed on by MITFD, WDFW, and WSDOT

Considering climate change resilience, the minimum hydraulic width was evaluated under current and 2080 100-year flow events. **Table 4-2** compares the discharge velocities from the 100-year and projected 2080 100-year events at three locations along the crossing. No size increase was determined to be necessary to accommodate climate change. For detailed hydraulic results, see **Section 5.4**.

Table 4-2: Average velocity comparison

Location	100-year Velocity (ft/s)	Projected 2080 100-year Velocity (ft/s)
Upstream of structure (STA 7+06)	3.1	3.4
Through structure (STA 5+22)	3.5	3.9
Downstream of structure (STA 2+98)	3.0	3.2

4.2.3 Vertical Clearance

The vertical clearance under a structure is made up of two considerations: freeboard and maintenance clearance. Both are discussed below, and results are summarized in **Table 4-3**. The minimum SFZ and maintenance heights are consistent with previous drafts of the PHD and design developed by the WSDOT engineer. The SFZ height does meet the Chapter 2 SFZ definition requirement. The proposed SFZ and maintenance clearance for Juanita Creek is 10 feet, SFZ height measured from highest point within the hydraulic width, as indicated in WSDOT Chapter 2: Technical Requirements for this project, and preliminary design developed by WSDOT.

According to the WSDOT *Hydraulics Manual*, a standard of 3 feet of freeboard is prescribed for all structures exceeding 20 feet in length and for all bridge structures unless an alternative is approved by HQ Hydraulics (WSDOT 2023a). In the current proposed configuration, the structure offers 9.6 feet of freeboard (relative to the SFZ height) at the 100-year event, satisfying the minimum requirements. The internal structure height is constrained to a minimum of 10 feet without necessitating alterations to the I-405 roadway profile. This freeboard condition is described in **Table 4-3**.

Table 4-3: Vertical clearances provided in preliminary design

Crossing Location	100-Year Design Freeboard Requirement (ft)	Proposed Internal Clearance from Top of Bank (ft)	Estimated Freeboard with Proposed Structure Height (ft)
I-405	3.0	10	9.6

4.2.3.1 Past Maintenance Records

WSDOT Area 5 Maintenance was contacted to determine whether there are ongoing maintenance problems at the existing structure because of LWM racking at the inlet or sedimentation. The maintenance representative indicated that there was no record of LWM blockage and/or removal or sediment removal at this crossing.

4.2.3.2 Wood and Sediment Supply

As mentioned in **Section 2.6.4**, the Juanita Creek system has good LWM recruitment potential both up and downstream of the I-405 crossing, as it flows through a steep wooded ravine. Future urban growth in the basin is unlikely under current City of Kirkland Zoning and Land Use ordinances. The riparian buffer, while steep, is wide and intact. Although minimal large wood is currently present in the system, the upper reach of this system has clear potential for significant wood recruitment. Currently, the inline regional detention facility is preventing large amounts of sediment from transporting through the system. Once removed, a portion of that accumulation will likely transport downstream.

4.2.4 Hydraulic Length

The maximum allowable hydraulic length determined by WSDOT for the I-405 crossing of Juanita Creek is 358.6 feet. If the hydraulic length is increased beyond 358.6 feet, the hydraulic width and vertical clearance will need to be reevaluated. The proposed crossing is 250 feet.

4.2.5 Future Corridor Plans

The existing structure is being replaced as part of the Project, as summarized in **Section 1**. Structure alignment and length have been designed to accommodate this future widening. As a result, there will be no requirement for subsequent lengthening or modifications to accommodate the specifications outlined in the master plan conditions. Therefore, this structure will be forward compatible.

4.2.6 Structure Type

The proposed structure type is an open-bottom, circular arch shaped, 37-foot wide by 10-foot high tunnel within the reinforced shotcrete final liner. The construction of the final liner will be facilitated with a ground freezing technique to stabilize the ground and the sequential excavation method (SEM) with initial shotcrete liner to create the underground space within which the final liner will be constructed. The thickness of the ground freeze allowance and initial and final shotcrete liners will be refined for final design. A bottomless structure is recommended by WSDOT HQ Hydraulics for the proposed crossing for multiple reasons. A bottomless structure will allow a natural bottom and will allow for natural channel adjustment. This type of structure will eliminate the risk of a plunge pool developing at the structure outlet, impeding fish passage.

4.3 Streambed Design

This section describes the streambed design developed for Juanita Creek at I-405 MP 21.94.

4.3.1 Bed Material

Existing condition hydraulics of Juanita Creek are influenced by the regional detention facility located under I-405. The proposed Juanita Creek structure under I-405 will not have such a facility, and velocities and shear stresses will be higher than existing conditions. Bed materials will need to be substantially coarser than current bed materials to resist scour and mobilization.

In order to model proposed hydraulic conditions and accurately size bed material, riffles and pools were graded into the terrain using MicroStation. While the average slope through the proposed fish passage structure is approximately 3 percent, riffle slopes in the structure are as high as 6 percent (**Appendix D**). Cross sections in the model were taken through three riffles and three pools within the structure, and average main channel shear stresses were estimated. The largest average shear stress of the three riffle sections and the largest average shear stress of the three pool sections were used for sediment sizing calculations.

PHD plans (**Appendix D**) currently show some riffles upstream of the structure steeper than 6 percent. These will be adjusted for the FHD and will likely be changed to step-pools instead of riffle-pools; thus, hydraulics in the upstream reach were not used for sediment sizing.

Three methods were used to size sediment: the Modified Shields Methodology (USDA 2008), the Bathurst Equation (Bathurst 1987), and the U.S. Army Corps of Engineers (USACE) riprap equation (Barnard 2013). The Modified Shields Methodology produced the most conservative results and was used for sediment sizing (**Appendix C**). **Table 4-4** provides a comparison of sediment sizes. Note that a ratio of D_{84}/D_{100} equal to 0.5 was used instead of the typical ratio of 0.4, so that the D_{100} could be kept to less than one quarter of the bankfull width.

Table 4-4: Comparison of observed, WSDOT PHD, and proposed riffle and meander bar streambed material

Sediment size	Existing Average Diameter (in.)	Previously Proposed Riffle-Pool System Diameter from WSDOT PHD (in.)	Proposed Diameter for Riffles and Meander Bars Based on Updated Hydraulics (in.)
$\mathbf{D_{16}}$	0.38	0.78	2.18
$\mathbf{D_{50}}$	1.01	3.89	6.96
$\mathbf{D_{84}}$	2.21	13.20	17.40
$\mathbf{D_{100}}$	5.69	18.00	34.80

The streambed sediment mix will not be increased by the WSDOT *Hydraulics Manual*-prescribed 20 percent, as shown in **Table 4-4**. The larger increase to the streambed sediment will account for this stream reach being returned to a natural flowing channel rather than a reach affected by backwatering of the regional detention facility.

Using the Modified Shields Methodology, the D_{50} and D_{84} for pools were calculated as 2.59 inches and 6.5 inches, respectively. However, because pools are intended to scour and fill, with most of their sediment moving through the system, the sediment gradation for pools was decreased to the standard WSDOT streambed sediment; a D_{50} of 1 inch was calculated to be mobile at the 2-year flow (**Appendix C**) (**Table 4-5**).

Table 4-5: Proposed pool streambed material (WSDOT streambed sediment)

Sieve Size (inches)	Percent Passing
2.5	99-100
2.0	85-100
1.0	50-82
0.5	28-68
0.017 (No. 40)	10-20
0.003 (No. 200)	5-10

To provide the needed channel stability and prevent entrainment along the culvert walls, meander bars will be included within the structure to prevent streambed sediment from being washed out. These meander bars will be composed of the same sediment gradation as the riffle sections (**Table 4-4**). In addition, mobile wood material will be arranged within the structure and LWM outside the structure to provide additional roughness, channel stability, and habitat variability.

In addition to the riffle gradation providing channel and general bedform stability, this bed material will also act as a profile stability measure that will limit long-term degradation of the channel bed. As discussed in **Section 7.2** in detail, this bed material will help to prevent LTD.

4.3.2 Channel Complexity

This section describes the channel complexity of the streambed design developed for Juanita Creek at I-405 MP 21.94.

Channel complexity within the structure will be created with the use of meander bars and LWM that partially obstruct low flows to create habitat diversity and act to prevent the stream from capturing the structure wall and avoid plane-bed configurations. At the PHD stage, it is typically only known what impact is necessary for the stream itself and not for constructability. Final LWM configuration design will be approved by the WSDOT HQ Hydraulics Office. LWM will be installed according to Chapter 10 of the *Hydraulics Manual* within the project limits, as defined for permitting. Channel complexity inside the crossing structures will include meander bars, riffles and pools, marginally mobile wood, and restored streambank shape as shown in the preliminary plans (**Appendix D**).

4.3.2.1 Design Concept

Multiple channel complexity elements will be incorporated into the channel design to meet the functional goals, as described in more detail in the following subsections and shown in **Figure 4-5**.

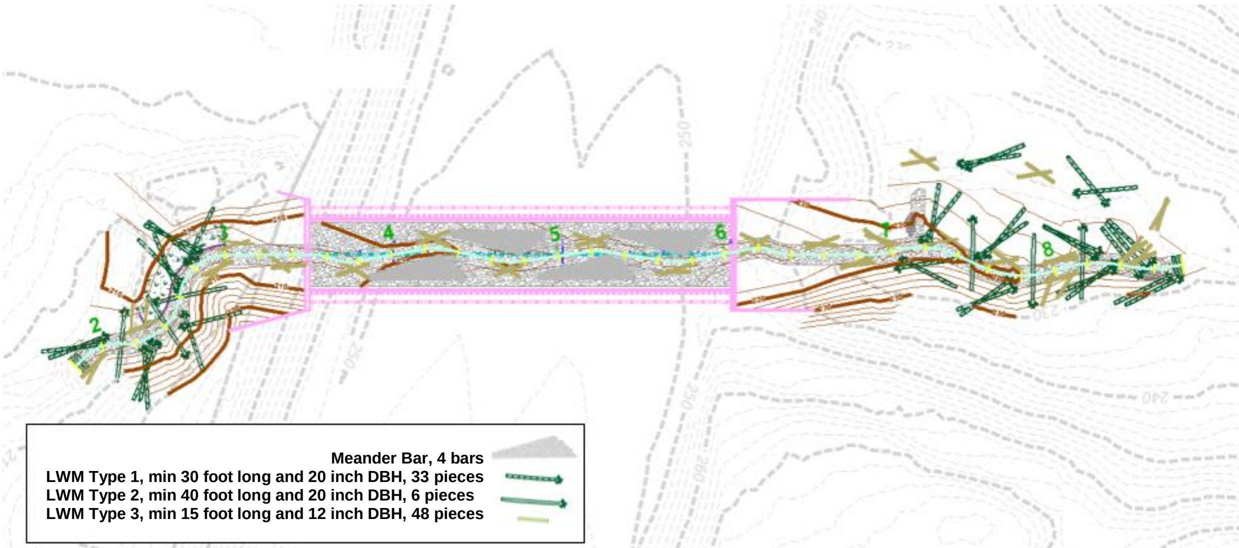


Figure 4-5: Conceptual layout of habitat complexity

4.3.2.1.1 Large Woody Material

LWM will be placed to emulate instream habitat functions in a manner that mimics natural wood loads, at the 75th percentile key-piece density levels found by Fox and Bolton (2007) (**Appendix F**) in other natural streams in the region. LWM can provide multiple benefits, including reducing overall channel creek velocities and associated scour due to increased roughness, supporting pool hydraulics, habitat complexity, and slope stability.

For Juanita Creek at I-405, a total of 87 pieces of LWM will be placed over the 697 feet of constructed channel. The total pieces will encompass 33 pieces of type 1, 6 pieces of type 2, and 48 pieces of mobile wood type 3. Type 1 and 2 are key pieces and will be buried and wedged between other pieces of LWM when feasible, so as not to require additional anchoring, or will be placed entirely above the 100-year WSE. The type 1 key pieces will have a minimum 20-inch diameter at breast height (DBH), be 30 feet long, and have rootwads attached for increased instream habitat complexity. Three type 1 key pieces and four type 3 mobile wood 15-foot logs will be incorporated into a right bank engineered log jam, designed to partially divert flows to use the existing channel as a side channel for habitat enhancement and refuge at high flows. Six 40-foot-long type 2 channel-spanning logs placed above the 100-year WSE have also been incorporated into the design. The remaining 44 mobile wood type 3 pieces will be a minimum 12-inch DBH and 15 feet long. They will not be buried and will be designed to be mobile above the 10-year design storm. This marginally mobile wood will be ballasted by key pieces of wood in some instances and will be placed strategically throughout the channel, both within and outside of the structure, to maximize instream habitat function. The LWM layout can be found in **Appendix D**.

Materials for LWM and LWM anchoring will follow the guidance in Chapter 10 of the WSDOT *Hydraulics Manual* and any environmental commitments noted in the contract.

4.3.2.1.2 Meander Bars

Channel meandering in the proposed structure will be maintained through construction of meander bars that will enhance the instream channel diversity and encourage the stream to self- maintain this habitat over its design life. The spacing of these meander bars will be based on the spacing of the meanders found in the existing stream channel and guided by the Rosgen Type B stream characteristics, which state that streams with moderately steep slopes (2 to 4 percent) have pool-to-pool spacing of approximately four to five bankfull widths (Rosgen 1996). Meander bars will be constructed to protrude into the low-flow channel to encourage meandering of the channel in low-flow conditions.

To maintain channel shape, the apex of the meander bar height is set to the 10-year flow depth, so the materials change the channel shape through the culvert structures rather than the typical channel section. The proposed meander bar material gradation (**Table 4-4**) follows the guidance in the WSDOT *Hydraulics Manual*. The document provides guidance for three components of the meander bar: (1) structure head; (2) structure tail; and (3) other considerations (such as small wood material or slash placed in front of structure head rocks to encourage racking and add additional habitat complexity).

Following the WSDOT *Hydraulics Manual* guidelines, the meander bar sediment sizing will have a D_{100} that is stable at the 100-year event. Materials in the bar tail are 50 percent by volume larger than the D_{84} of the observed streambed material. Finally, slash will be incorporated in the meander bar head to increase habitat complexity. Meander bar sediment sizing calculations are found in **Appendix C.**

4.3.2.1.3 Riffle Features

Riffle features will be built of coarser streambed material to help stabilize the pool-riffle sequences and overall channel grade and to help mitigate the risk of long-term degradation (see **Section 7.2**). They are not intended to be fixed grade control or rigid structures that do not deform over time.

The riffle feature gradation will include some larger material compared to the streambed sediment and will be constructed of rounded material (**Table 4-4**). Note that like meander bars, streambed material in riffle features is designed to be stable at the 100-year discharge. Thus, meander bars and riffle features have the same gradation.

Note that the hydraulic conditions of riffle sections assume a local slope that is steeper than the overall average channel grade because the pool sections are assumed to be flat. Riffle features will span the entire SFZ width, thereby providing increased stability along the longitudinal profile of the channel bed.

Sediment sizing calculations for riffle features are found in **Appendix C**.

4.3.2.1.4 Deformable Grade Control

Deformable grade control will be included at the upstream and downstream ends of the channel improvements to help mitigate the risk of long-term degradation (**Section 7.2**). They are not intended to be fixed-grade control or rigid structures that do not deform over time. Similar to the coarse band riffle features discussed above, within the specified gradation, the deformable grade control structure will include material larger than streambed sediment; this will provide additional protection against channel degradation. The deformable grade control structure will extend beyond the top of bank and will include woody debris that will be slightly exposed above finished grade. Material sizing calculations will be provided for the FHD.

4.3.2.1.5 Additional Habitat Features

In an effort to maximize the available salmon habitat in Juanita Creek, the upstream reach of the Juanita Creek fish passage design has been updated to use the existing Juanita Creek channel as a side channel in the ultimate configuration. The inclusion of the side channel and the proposed grading between the new Juanita Creek channel and the existing channel will promote wetland connectivity and provide increased habitat enhancement, additional rearing habitat, and high flow refuge. The downstream connection of this side channel is a short step-pool reach into the main stem of Juanita Creek to ensure fish do not get stranded in the existing channel during periods of low flow. This step-pool reach will be designed in a manner to avoid the potential for subsurface flow and any juvenile salmonids getting trapped in that subsurface flow. The hydraulic analysis included the implementation of an engineered log jam at the intersection where the proposed new channel diverges from the existing channel to help direct flow at the 2-year event and higher into this new side channel. Additional LWM has been included in the existing channel for added habitat diversity and cover.

In addition to the streambed materials that make up the main portion of the proposed channel, nourishment piles will be placed on the margins of the bankfull channel. These nourishment piles will consist of a gradation that supports salmonid spawning. Over time as flow encounters these piles, spawning-sized gravels will be washed into the main channel and will aid in replenishing any spawning-sized gravels that may be transported away from the project reach. As previously mentioned, the proposed channel must be significantly coarser than the existing material to provide adequate channel stability, and the use of these nourishment piles is aimed at providing additional finer gravels that will over time become useful for salmonid spawning. The gradation of these nourishment piles will be finalized for the FHD.

4.3.2.2 Stability Analysis

Large wood stability analysis will be completed for the FHD.

4.3.3 Planting Restoration

This section describes the planting restoration developed for Juanita Creek at I-405 MP 21.94. The development of the planting plan was guided by a close review of Juanita Creek – the existing vegetation, hydrology, climatic conditions, and ecological functions in its current condition. The planting plan focuses on the restoration of ecological processes and functions in the project area by placing native, locally occurring plants in the most species-appropriate microhabitats of the project site. This will allow them to thrive and provide key riparian vegetation functions such as erosion protection, bank stabilization, nutrient cycling and production, habitat creation, water retention, and many others. A proposed plan of the planting restoration for Juanita Creek at I-405 can be seen in **Figure 4-6**.

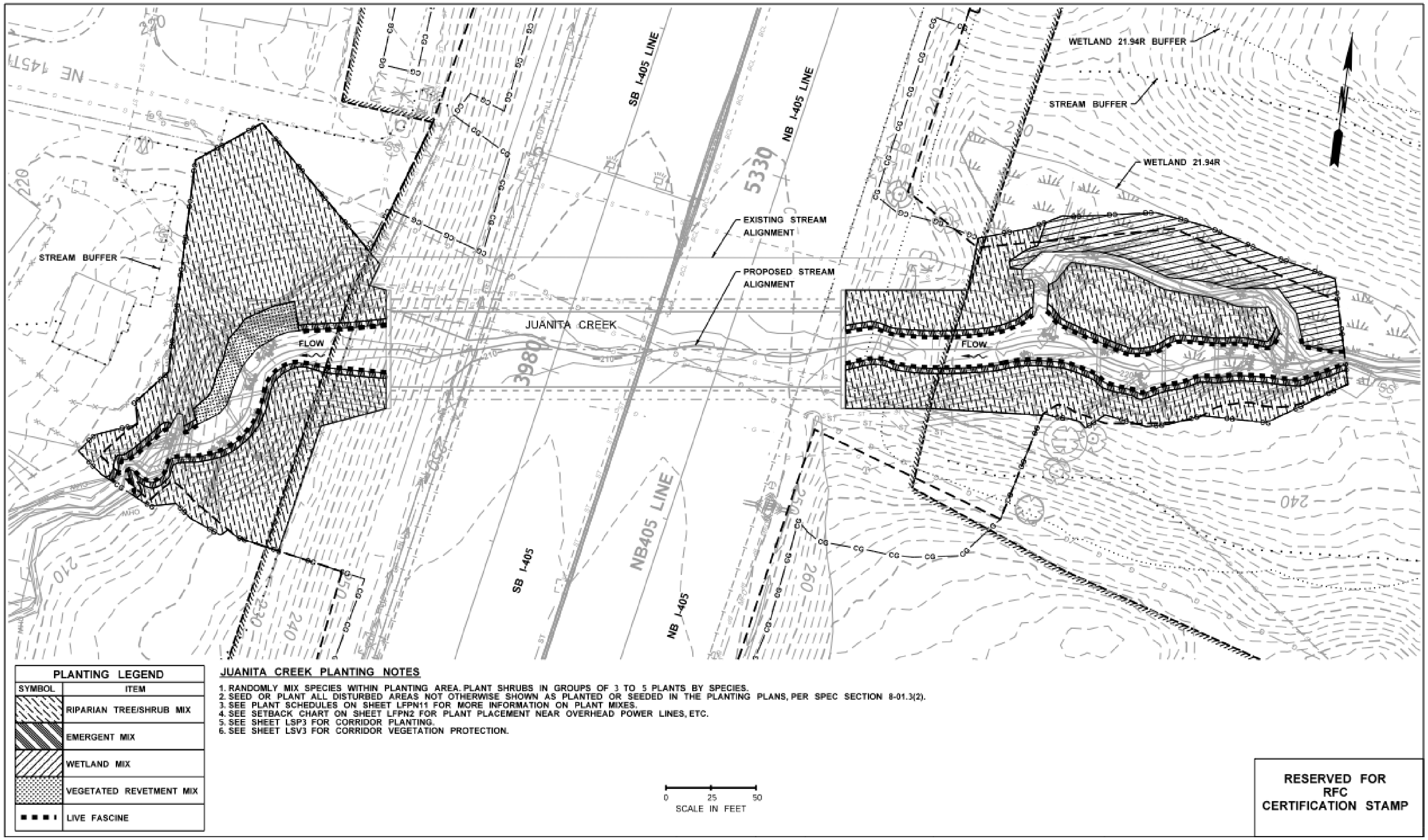


Figure 4-6: Proposed plan of planting restoration

4.3.3.1 Riparian Zones and Plant Selection

Riparian planting zones at the project site are defined as the lands adjacent to streams whose soils and vegetation are influenced by channelized water. They are the transition areas between aquatic and upland habitats with elements of both ecosystems. With this context, plant species selection and soil preparation were closely tied to the plant's role and function along the riparian gradient. The plant species that constitute the proposed planting grow naturally both at the project site and in its immediate vicinity. They were grouped into three zones based on water needs and degree of inundation, which are reflected by projected WSEs, including the ordinary high-water mark (OHWM), 2-year, 25-year, and 100-year flood lines. This information has been thoroughly studied and is well documented in the *Stream Habitat Restoration Guidelines* (Cramer 2012) for riparian habitat protection and restoration. The two planting zones are riparian tree/shrub zone and emergent wetland zone. The third zone will comprise live fascines with live staking, proposed for the purpose of bank stabilization. The proposed planting zones loosely correspond to upper and lower riparian areas. As described above, the delineation of planting zones was driven by the expected hydraulic, environmental, and substrate conditions along Juanita Creek.

4.3.3.2 Riparian Tree/Shrub Planting

The upper riparian zone consists of upland riparian trees and shrubs. Species selection for this zone closely coincides with roadside restoration efforts along the I-405 corridor due to similarities in elevation along the hydraulic gradient. Corridor planting types are determined by the I-405 Urban Design Criteria (WSDOT 2022b) for compliance with WSDOT standards. Only WSDOT-approved native plant species for sensitive areas are proposed within the stream buffer. Installation for this zone begins above the 25-year flood line. The plant mix consists of native species, both trees and shrubs, arranged in natural patterns to emulate surrounding riparian vegetation and corridor character. Heights and species of vegetation are also taken into consideration based on proximity to utilities, safety concerns, and visibility as it relates to the roadway and stream.

4.3.3.3 Live Fascines and Live Staking

Between the upper and lower riparian zones, streambank erosion control methods including live fascines with live staking are proposed. The primary live fascine is proposed along the 2-year flood line. The willow (*Salix* sp.) and ninebark (*Physocarpus* sp.) species proposed can withstand periods of inundation but require oxygenated soils for at least part of the growing season. These bank stabilization elements will provide important ecosystem benefits. They will also contribute to improved bank stability, enhance physical channel features, provide organic debris recruitment, and serve as a major source of nutrients to support in-stream fauna and flora.

4.3.3.4 Emergent Wetland Planting

The lower riparian zone consists of emergent wetland planting. Located between the 2-year and the 25-year flood lines, this zone is closest to the stream's base flow edge. The plant species proposed for this zone are well adapted to growing in permanently saturated soils. Along with emergent wetland shrubs, a diverse mix of grasses, sedges, and rushes are proposed to help infill between these plants and occupy smaller habitat niches. Emergent plant benefits include creation of important habitat and food sources for wildlife, filtering and trapping soil, and absorption of nutrients from runoff. Ultimately this planting zone will provide necessary roughness at the water's edge, without creating an inaccessible dense wall of planting at the foot of the bank.

4.3.3.5 Revegetation and Erosion Prevention in Bank Treatments

Establishment of the integrated plantings is critically important for the proper performance of each of the bank treatments. Detailed pre-installation planning and site preparation will be conducted to maximize the potential for survival of the plant materials installed and to enhance their ability to grow quickly. The following key restoration tasks will be completed to construct a good-quality, well-armored vegetated bank:

- Planting vegetation after grading work and soil preparation is complete
- Installing live fascines and live staking for streambank erosion control and overall bank stability.
- Installing irrigation equipment and emitters before the plants to ensure they receive adequate water immediately after installation and during the initial establishment period
- Planting during the rainy season to increase survival rates because newly installed riparian vegetation needs almost constant water for the first year in the ground
- Making sure that soil compaction does not exceed 80 percent relative density for a depth of 24 inches to promote good aeration, root growth, water-holding capacity, and natural composting processes

4.3.3.6 Irrigation Design

The temporary irrigation system used for establishment will be located outside of the OHWM to prevent the potential effects of high-water events and flood flows. The temporary irrigation zones are split into two main areas based on planting types and associated water requirements: an upland zone and a wetland zone. The upland irrigation zone will serve more drought-tolerant, non-aquatic plantings at a higher elevation. The wetland zone will serve plants that require periodic or permanently saturated soils, bordering the OHWM. The irrigation basis of design includes two points of connection occurring east and west of the I-405 crossing in the vicinity of Juanita Creek. Watering frequency will ultimately depend on soil texture, weather, and planting depth. During the 5-year plant establishment period, plants should generally be watered more frequently during the first growing season or until plants develop root systems capable of reaching a depth where the soils are permanently moist.

5 Hydraulic Analysis

The hydraulic analysis of the existing and proposed I-405 Juanita Creek crossing was performed using the U.S. Bureau of Reclamation's Sedimentation and River Hydraulics (SRH-2D) Version 3.6.5 computer program, a two-dimensional hydraulic and sediment transport numerical model (USBR 2024). Pre- and post-processing for this model was completed using Surface-Water Modeling Systems (SMS) Version 13.3.10 (Aquaveo 2024).

Two scenarios were analyzed for determining stream characteristics for Juanita Creek with the SRH-2D models: (1) existing conditions with the 48-inch CMP and (2) future conditions with the proposed 30-feet clear span structure.

It is worth noting that the preliminary hydraulic analysis is based on a structure length of 345 feet that has since been shortened to 250 feet. The plan sheets (**Appendix D**) reflect this shortened structure length, but the hydraulic model does not. For the next phase of design, the riffle pool sequence, stream alignment, and other features will be adjusted to fit this new crossing, which will likely result in hydraulic conditions very similar to what is reported in this section. Therefore, these results are valid for the purpose of preliminary analyses.

5.1 Model Development

This section describes the development of the model used for the hydraulic analysis and design.

5.1.1 Topographic and Bathymetric Data

The existing conditions SRH-2D surface data were established from a combination of two different field survey techniques. The first survey captured the entire Project vicinity and was obtained from an aerial LiDAR survey conducted in fall of 2021. More detailed supplemental field surveys were performed in fall of 2020 and spring of 2024, which more accurately captured open channel cross sections, culvert locations, and invert elevations. The InRoads digital terrain model (DTM) surface used as the basis of the existing conditions model is a merged version of these two surveys. All survey and LiDAR data are referenced against the NAVD 88 vertical datum.

Because the combined 2020 and 2024 field survey data did not provide coverage for the northeastern corner of the model domain that includes upland residential areas and part of 20th Avenue, 2021 King County West LiDAR data were used to fill this data gap (WGS 2023).

The proposed conditions SRH-2D terrain data consist of the MicroStation InRoads DTM created from typical riffle and pool cross sections described in **Section 4.1.1**. This surface is supplemented outside of the stream channel grading limits by the existing merged DTM that incorporates the 2024 survey. These surfaces were merged to create a single terrain for use in the SRH-2D proposed conditions model.

The existing and proposed DTM surfaces and 2021 King County LiDAR data were imported into SMS for mesh generation of existing and proposed conditions.

5.1.2 Model Extent and Computational Mesh

The existing and proposed conditions models have the same model extents with a total area of about 43 acres. The upstream limit of the computational mesh, on the northeast end, is approximately 1,300 feet upstream of the I-405 crossing along Juanita Creek. The downstream boundary of the mesh is approximately 1,370 feet downstream of the I-405 crossing.

The existing conditions mesh includes 49,865 elements (shown on **Figure 5-1**), and the proposed conditions mesh has 47,586 elements (shown on **Figure 5-2**: and **Figure 5-3**).

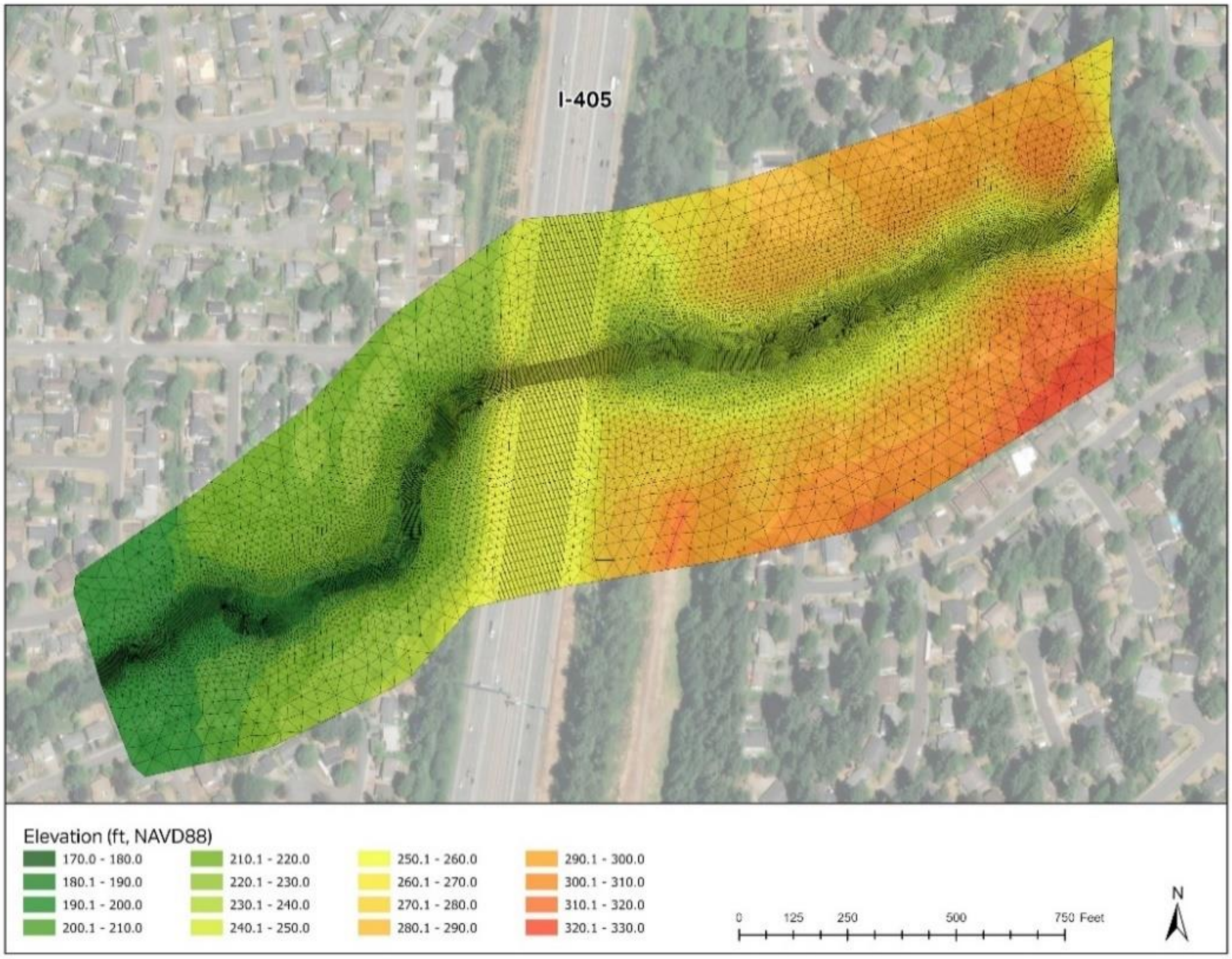


Figure 5-1: Existing conditions ground surface elevation mesh, model extents, boundary conditions

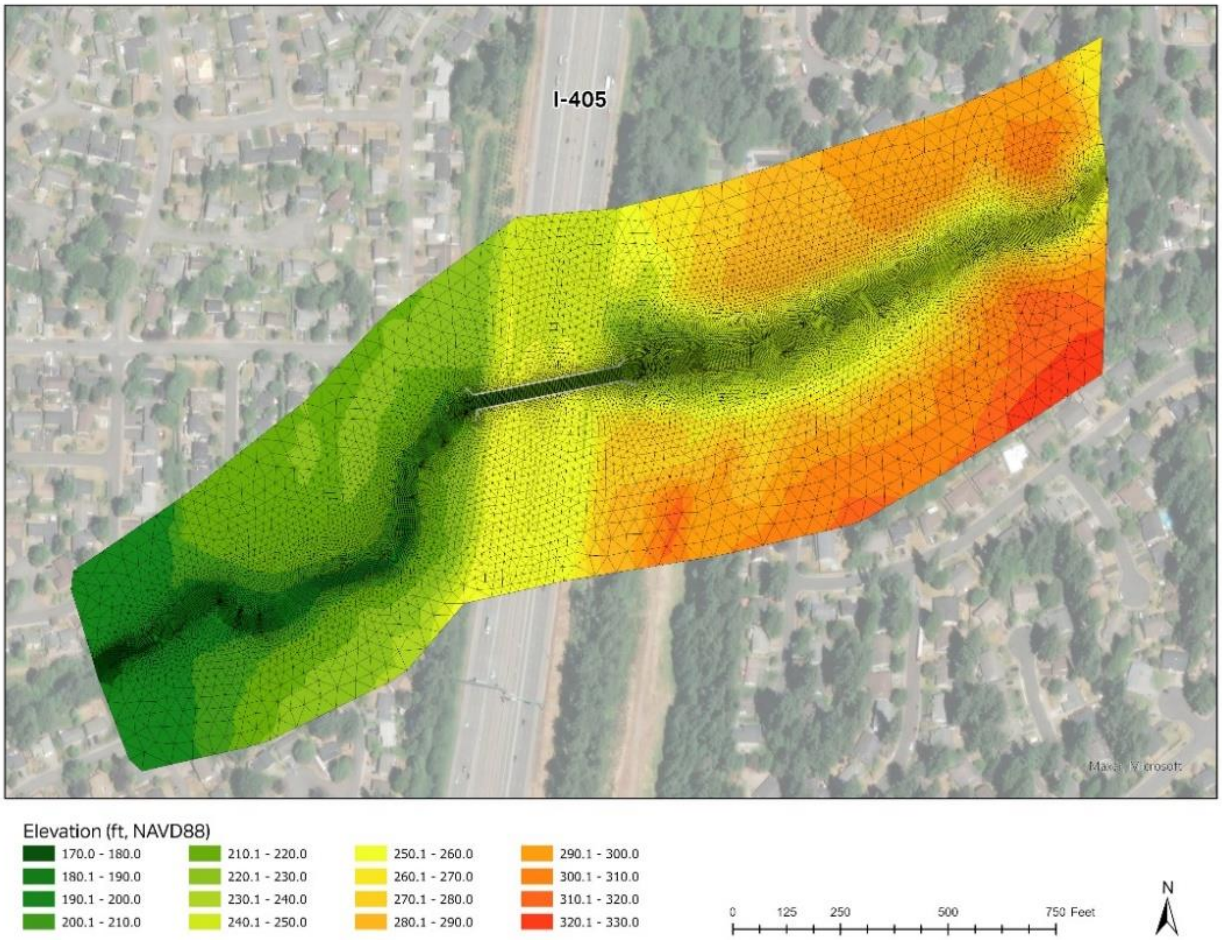


Figure 5-2: Proposed conditions ground surface elevation mesh, model extents, boundary conditions

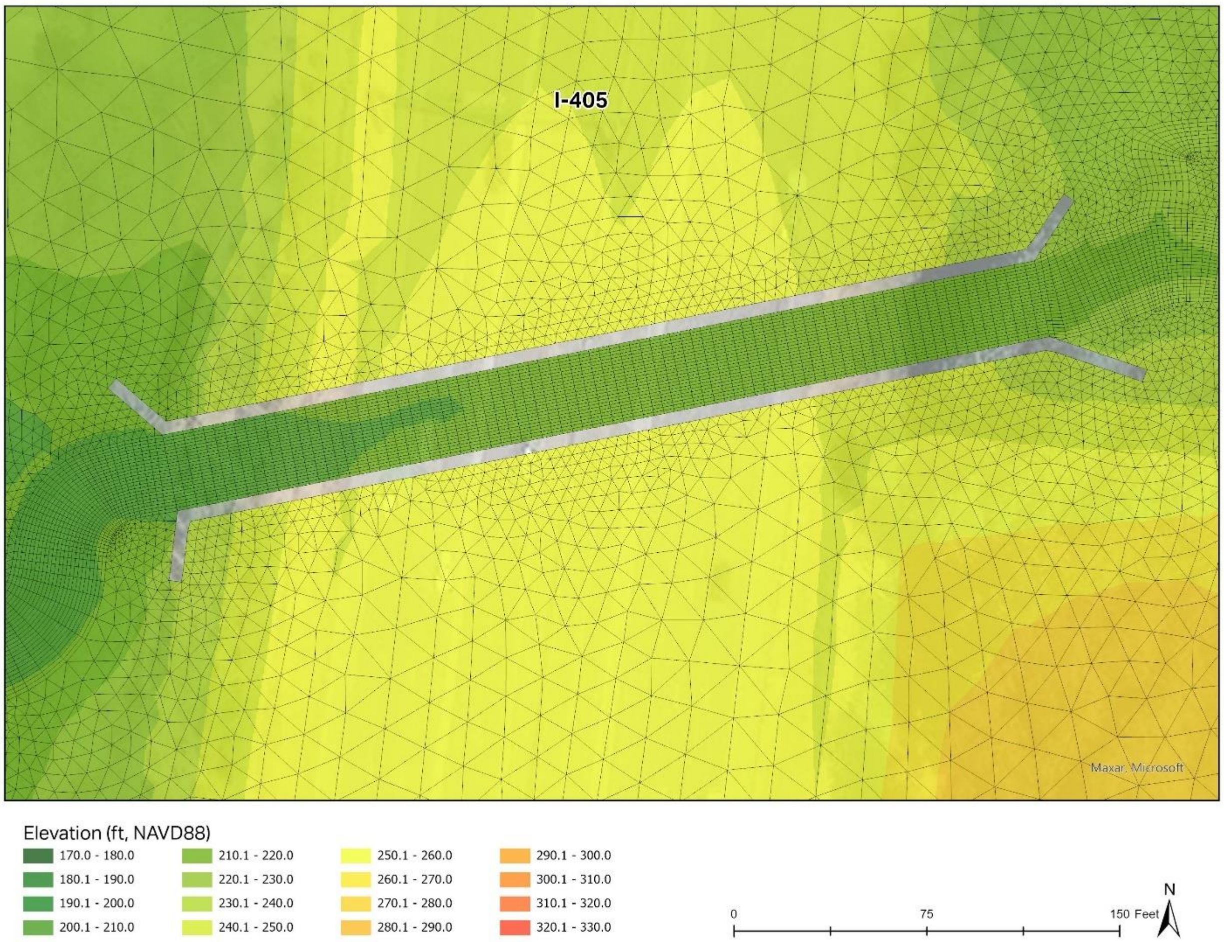


Figure 5-3: Proposed conditions ground surface elevation near I-405 crossing

5.1.3 Materials/Roughness

Manning's n values were determined by site observations, aerial topography, GIS land cover data, and standard empirical values. Calculations for Manning's roughness values are found in **Appendix E**, following guidance from Arcement and Schneider (1984) and Yochum, et al. (2014). **Table 5-1** summarizes the values used in the existing and proposed model.

The existing channel was assigned a Manning's roughness value of 0.065 due to the presence of thick vegetation, such as Himalayan blackberry, throughout the reach. This falls within the high end of the range provided for streams with regular sections and some weeds and medium brush on banks, as indicated in the WSDOT *Hydraulics Manual* reference for Manning's roughness coefficients.

The proposed channel inside the crossing will have a larger sediment size but less vegetation than the existing channel and was assigned a Manning's roughness value of 0.068. The proposed channel outside the crossing will have more obstruction due to more LWM and was assigned a Manning's roughness value of 0.078. LWM in the side channel will obstruct more of the flow than in the main channel, and thus it was assigned a Manning's n value of 0.08. In order estimate a Manning's n value for the step-pool, a stream with a similar slope was identified in Yochum (2014).

Figure 5-4 and **Figure 5-5** show the spatial distribution of Manning's n roughness values for existing and proposed condition models, respectively.

Table 5-1: Manning's n hydraulic roughness coefficient values used in the SRH-2D models

Existing		Proposed	
Material	**Manning's n**	**Material**	**Manning's n**
Existing Channel	0.065	Existing Channel	0.065
Overbank	0.080	Overbank	0.100
Pavement	0.016	Pavement	0.016
Development	0.060	Development	0.060
Existing Culverts	0.024	New Channel Inside Crossing	0.068
		New Channel Outside Crossing	0.078
		Side Channel	0.080
		Step-Pool	0.290

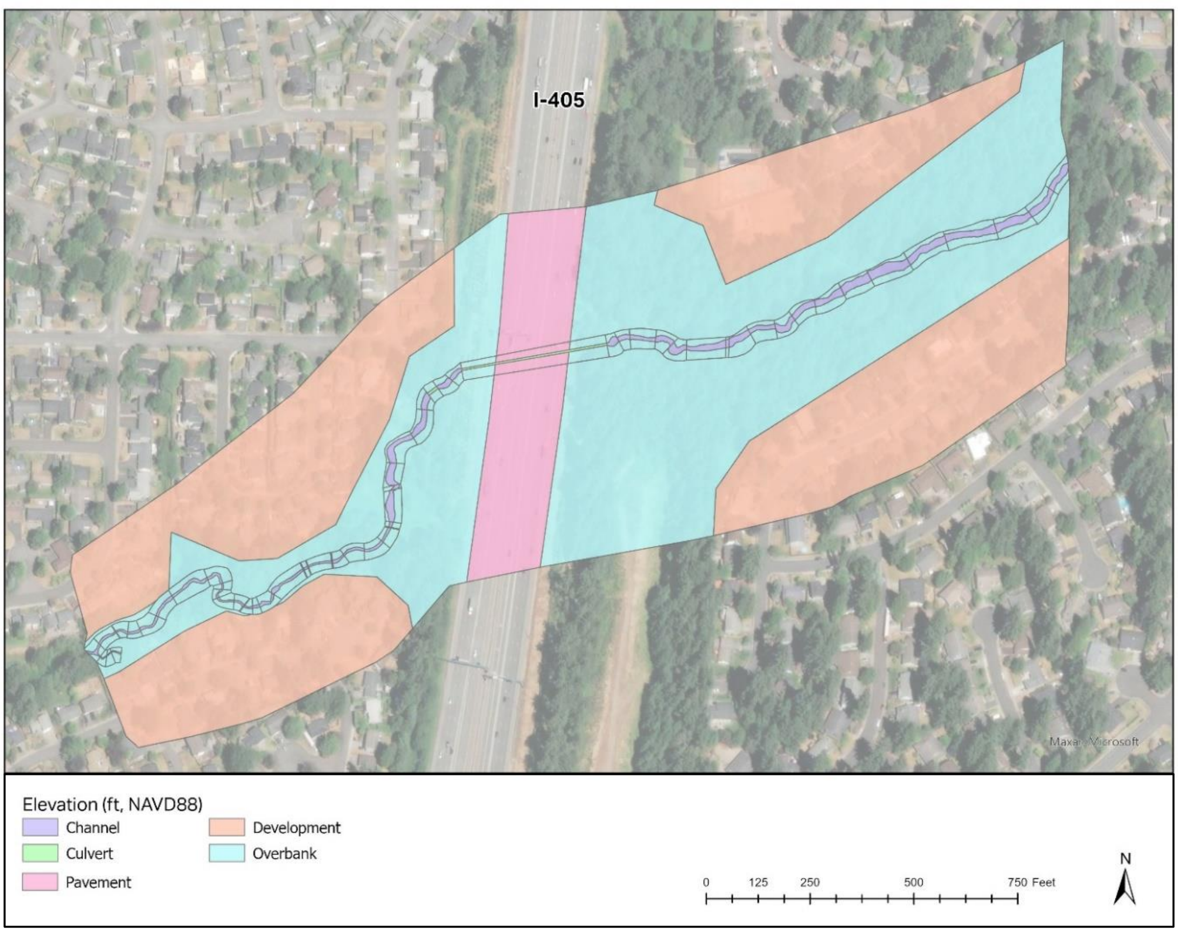


Figure 5-4: Spatial distribution of existing conditions roughness values in SRH-2D model

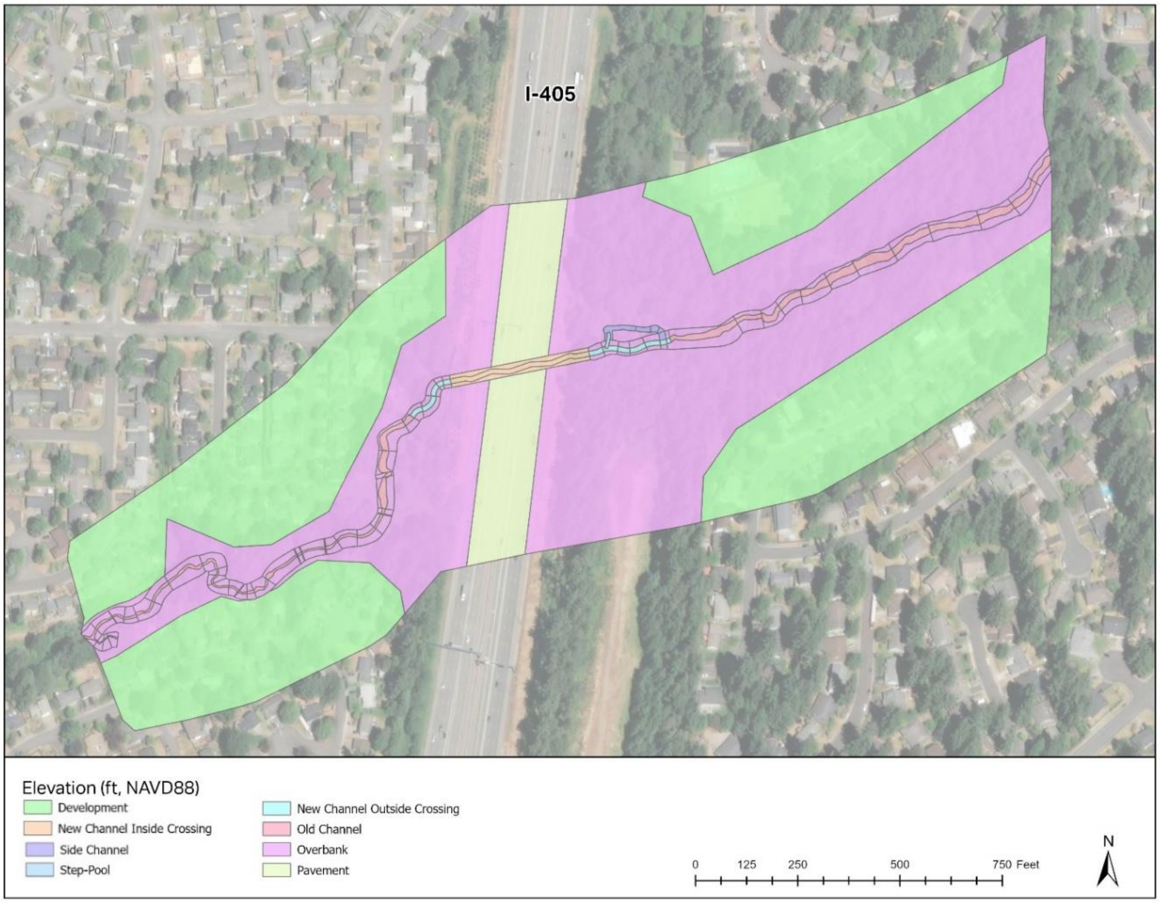


Figure 5-5: Spatial distribution of proposed conditions roughness values in SRH-2D model

5.1.4 Boundary Conditions

The existing conditions hydraulic model uses the boundary conditions as shown in **Figure 5-9**. An inflow boundary condition is placed 1,300 feet upstream of the crossing and flow was simulated as subcritical distributed via conveyance. An additional inflow immediately downstream of the existing I-405 crossing was added on the right bank as an inflow boundary condition for the model. The hydraulic model was run with one outflow boundary condition, downstream of the access road pipe crossing of Juanita Creek. A normal depth exit boundary was assigned using the SMS built-in normal depth calculator to efficiently calculate the expected WSE given the stream's cross-section area and gradient. The proposed hydraulic model uses the same boundary conditions, excluding the HY-8 boundaries, as shown in **Figure 5-10**

Each of the three existing structures along the surveyed reach of Juanita Creek were modelled within SMS using FHWA HY-8 for one-dimensional culvert hydraulics. Inputs for each structure follow the characteristics and geometry obtained in the stream survey.

Existing conditions model development included implementation of HY-8 internal boundary conditions to represent the existing 48-inch CMP at the I-405 crossing, as shown in **Figure 5-6**. There are two culverts (36-inch and 24-inch) 70 feet downstream of the I-405 culvert outlet, as shown in **Figure 5-7** and **Figure 5-8,** respectively. The existing culverts were modelled as conventional thin-edge projecting end treatments. The proposed conditions model uses the boundary conditions without the existing culvert boundaries, as shown in **Figure 5-10**.

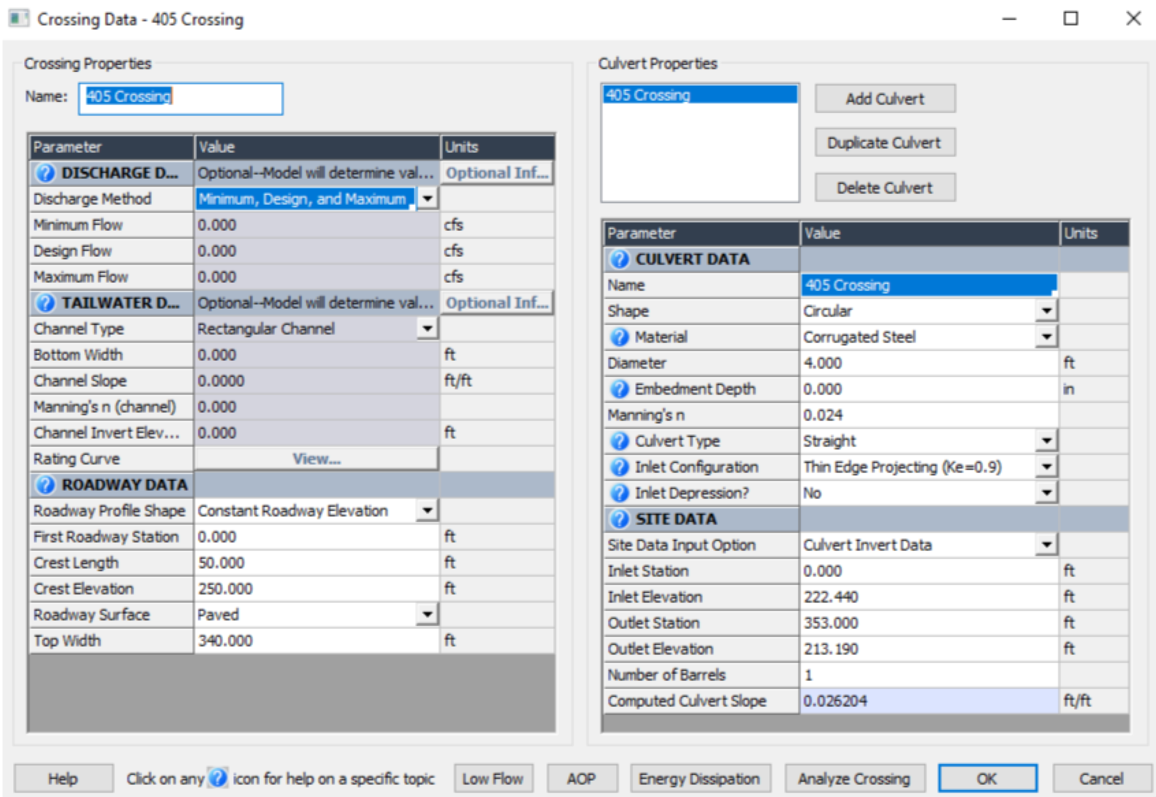


Figure 5-6: HY-8 I-405 crossing 48-inch culvert parameters

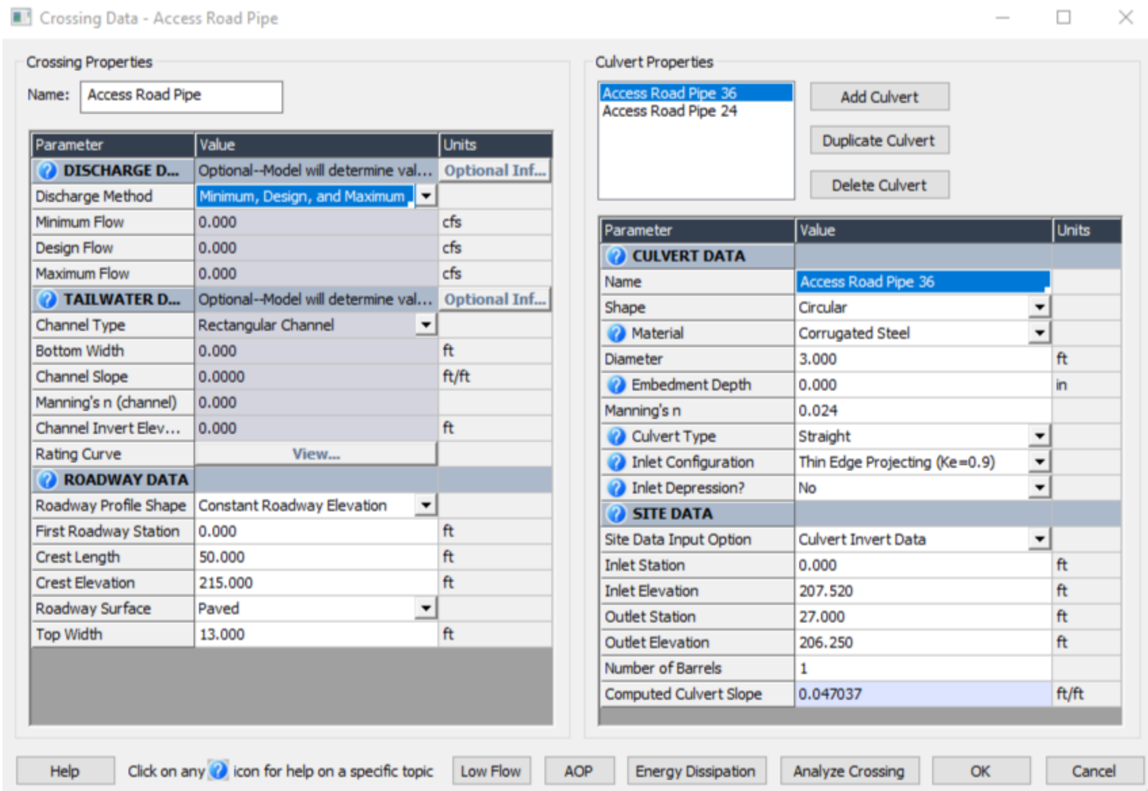


Figure 5-7: HY-8 property access crossing 36-inch culvert parameters

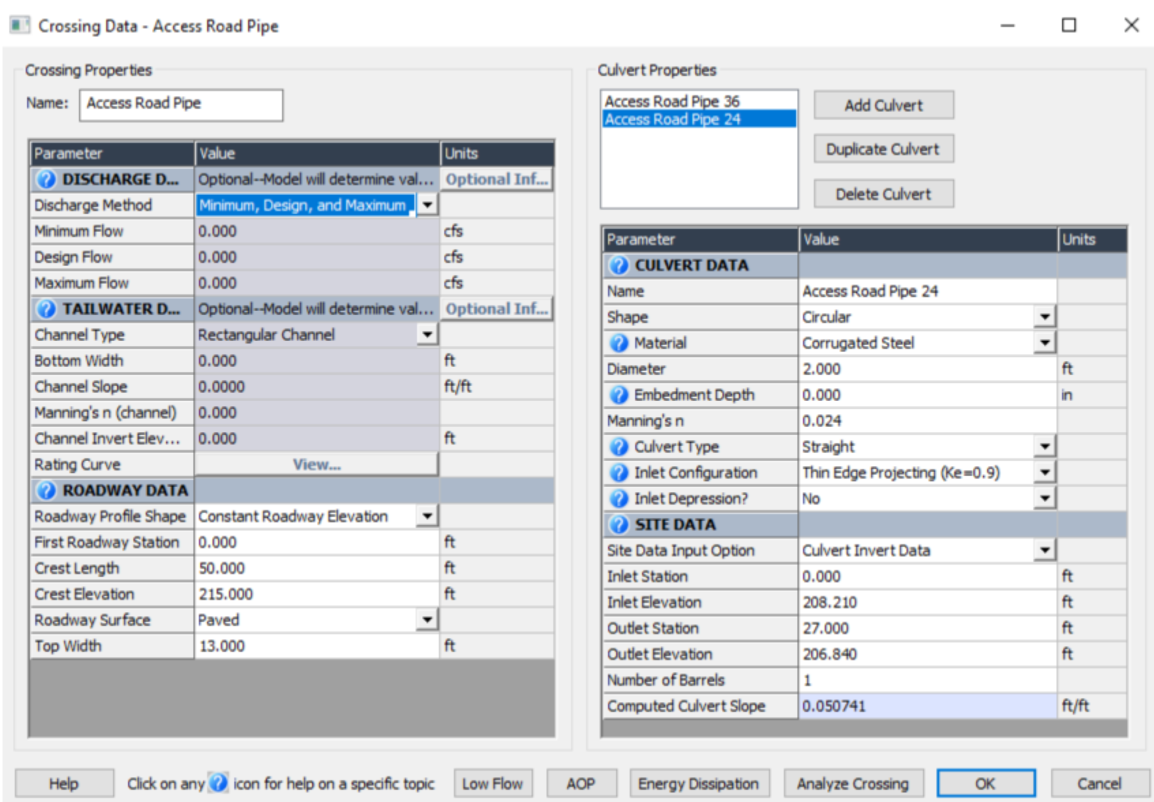


Figure 5-8: HY-8 property access crossing 24-inch culvert parameters

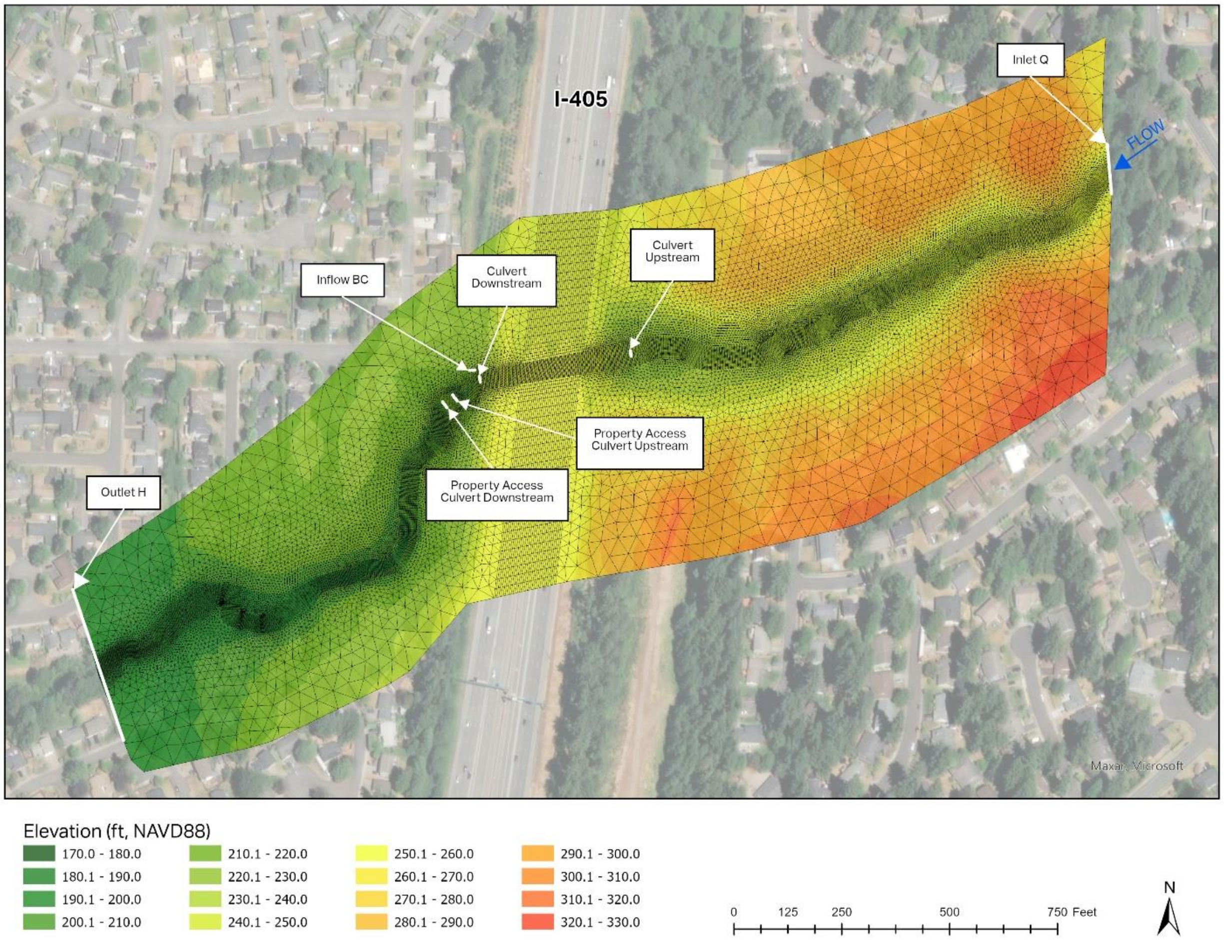


Figure 5-9: Existing conditions boundary conditions

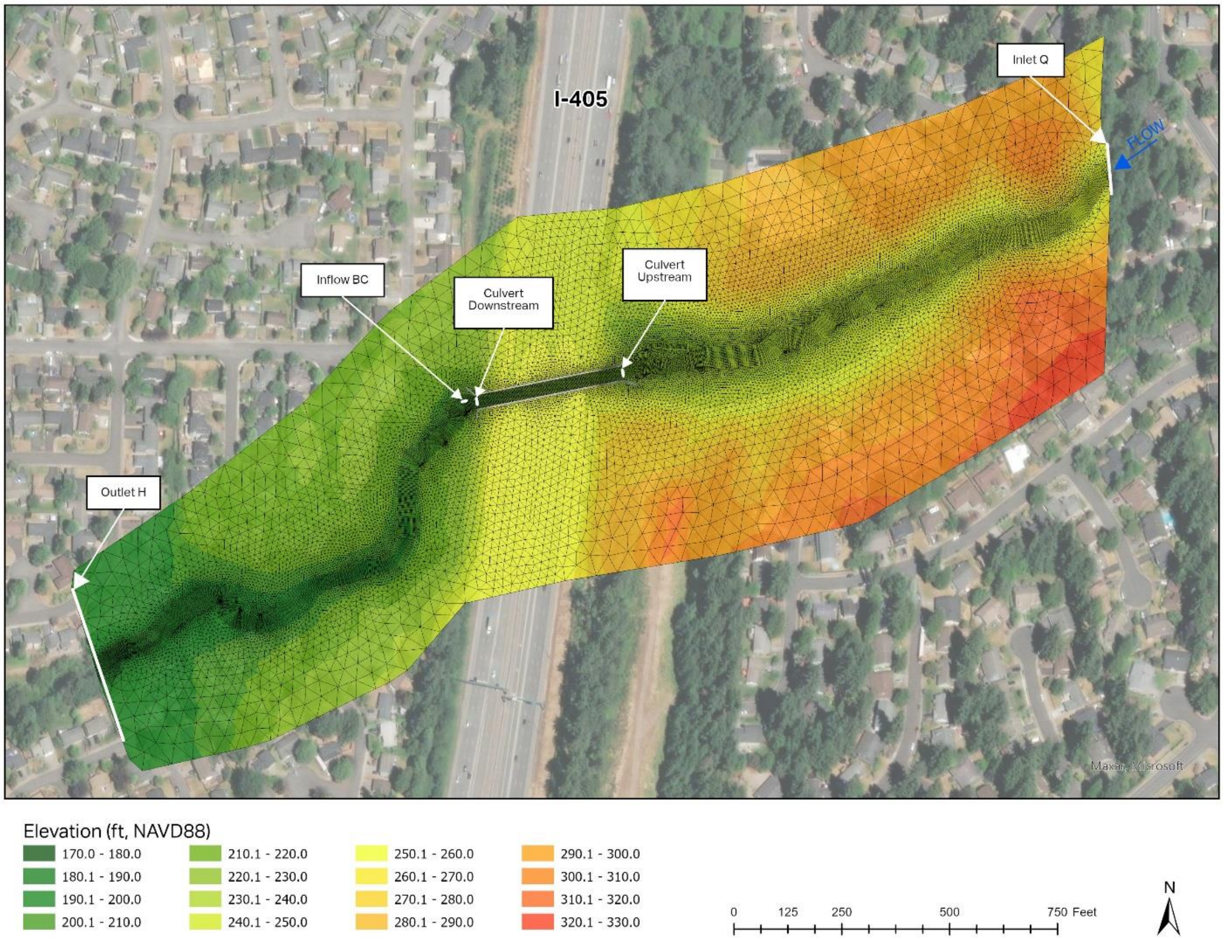


Figure 5-10: Proposed conditions boundary conditions

5.1.5 Model Run Controls

Model control for the existing conditions included a start time of 0 hours and an end time of 1 hour. Simulations were run with a 0.05-second time step, with an initial condition of dry. The output frequency was set to 6.0 minutes (0.1 hour), and the default parabolic turbulence was left as 0.7. The model was run as unsteady with three iterations per time step.

Model control for the proposed conditions included a start time of 0 hours and an end time of 4 hours. Simulations were run with a 0.5-second time step, with an initial condition of dry. The output frequency was set to 6.0 minutes (0.1 hour), and the default parabolic turbulence was left as 0.7. The model was run as unsteady with three iterations per time step.

Results showing the SRH-2D models' stability/continuity for the existing and proposed conditions can be found in **Appendix I**. The results display plots for net Q/inlet Q, mass balance, wet elements, monitor points WSE, monitor points z, and monitor lines.

5.1.6 Model Assumptions and Limitations

As stated at the beginning of this section, the modeled proposed condition that is reported and used for preliminary analyses represents a structure that will be shortened for the next phase of design. As the hydraulic model is updated with this shorter structure in the near future, design calculations and analyses will be updated accordingly.

The difficulties and uncertainties associated with the validation of the existing model stem from the lack of documented flood inundation within the ravine. The Juanita Creek reach between I-405 and NE 141st Street is relatively confined, and floodplain areas are limited. As a result, large storm events are confined to the ravine. This was confirmed during the hydraulic modelling effort for the *Juanita Creek Hydrology and Hydraulics Study H21 Report (*City of Kirkland 2022), as shown in the 100-year existing and proposed hydraulic model depth maps (**Appendix Q**). The model was developed by the WSDOT I-405 SR 167 Megaprogram staff and validated by peer review using an independent consultant expert in SRH-2D modelling.

5.2 Existing Conditions

This section presents the key results from the hydraulic analysis of the existing I-405 Juanita Creek crossing, modeled as a 48-inch-diameter CMP. Illustrations summarizing the WSE, velocity, and depths under the 2-,10- 25-, 100- and 500-year recurrence interval flows are provided in **Appendix H**. **Figure 5-11** depicts the SRH-2D existing conditions elevation mesh and location of cross sections used for results reporting. All results are presented relative to the NAVD88 datum. **Table 5-2** lists the existing conditions hydraulic results of the average WSE, maximum water depth, average velocity, and average shear stress at the cross sections shown on **Figure 5-11**. **Figure 5-12** shows the existing conditions water surface profiles, **Figure 5-13** provides a typical upstream existing channel cross section, and **Figure 5-14** presents the existing conditions 100-year velocity map. **Table 5-3** lists the existing conditions average channel and floodplain velocities.

The SRH-2D model of the existing conditions and the WSE figures indicate that the existing 48-inch CMP is insufficient for passing all recurrence interval flows, as significant backwatering occurs upstream for flows greater than the 2-year. No overtopping occurs at the downstream 36- and 24-inch culverts for all recurrence interval flows. Velocities and shear stresses are significant at the outlets of the culverts, which could cause scour immediately downstream.

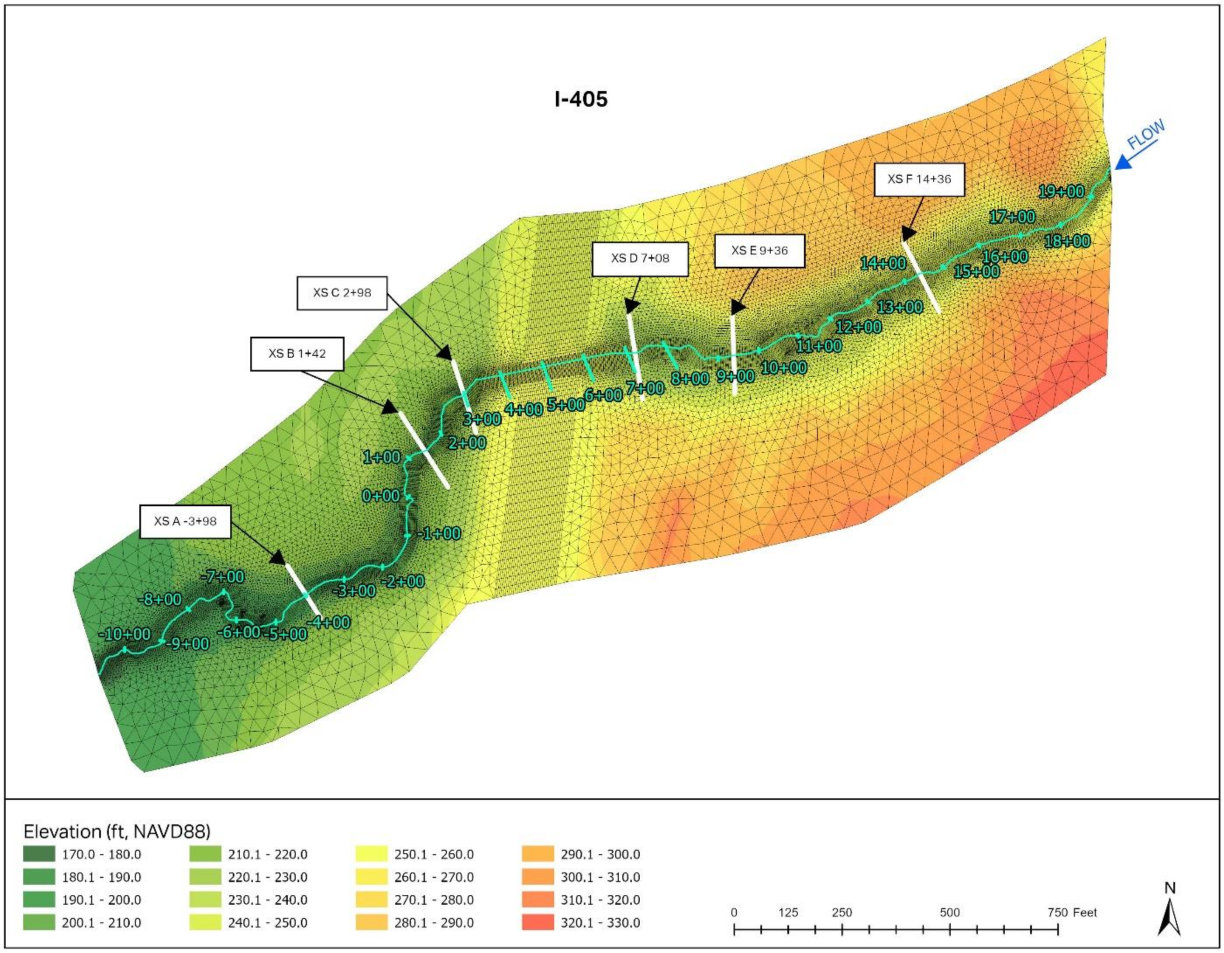


Figure 5-11: Locations of cross sections used for existing results reporting

Table 5-2: Average main channel hydraulic results for existing conditions

Hydraulic Parameter	Cross Section	2-year	100-year	500-year
Average WSE (ft)	DS -3+98 (A)	190.5	190.7	190.8
	DS 1+42 (B)	204.5	204.8	204.9
	DS 2+98 (C)	209.7	210.2	210.8
	US 7+06 (D)	225.1	225.6	225.9
	US 9+36 (E)	229.5	229.8	229.9
	US 14+36 (F)	238.1	238.4	238.6
Max depth (ft)	DS -3+98 (A)	0.8	1.0	1.1
	DS 1+42 (B)	1.1	1.4	1.6
	DS 2+98 (C)	1,2	1.7	2.3
	US 7+06 (D)	1.7	2.3	2.6
	US 9+36 (E)	1.1	1.4	1.5
	US 14+36 (F)	1.8	2.1	2.3
Average velocity (ft/s)	DS -3+98 (A)	2.70	3.45	2.96
	DS 1+42 (B)	1.63	1.88	2.02
	DS 2+98 (C)	1.09	1.15	0.96

Hydraulic Parameter	Cross Section	2-year	100-year	500-year
	US 7+06 (D)	1.21	1.68	1.64
	US 9+36 (E)	2.25	2.35	2.61
	US 14+36 (F)	1.68	2.17	2.41
Average shear (lb/SF)	DS -3+98 (A)	1.40	2.01	2.41
	DS 1+42 (B)	0.42	0.50	0.56
	DS 2+98 (C)	0.18	0.17	0.11
	US 7+06 (D)	0.76	0.55	0.45
	US 9+36 (E)	0.68	0.79	0.90
	US 14+36 (F)	0.33	0.49	0.59

Main channel extents were approximated.

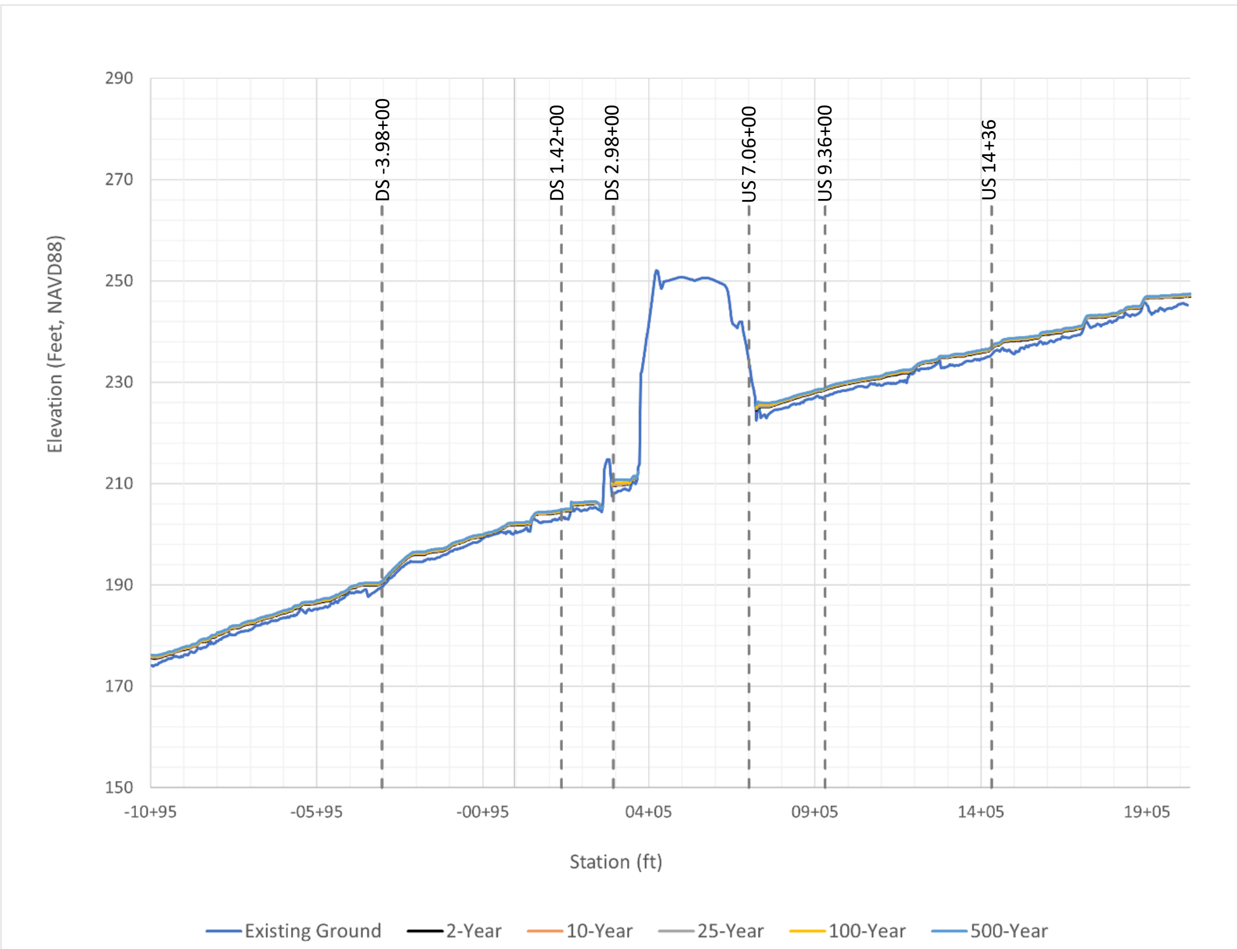


Figure 5-12: Existing conditions water surface profiles

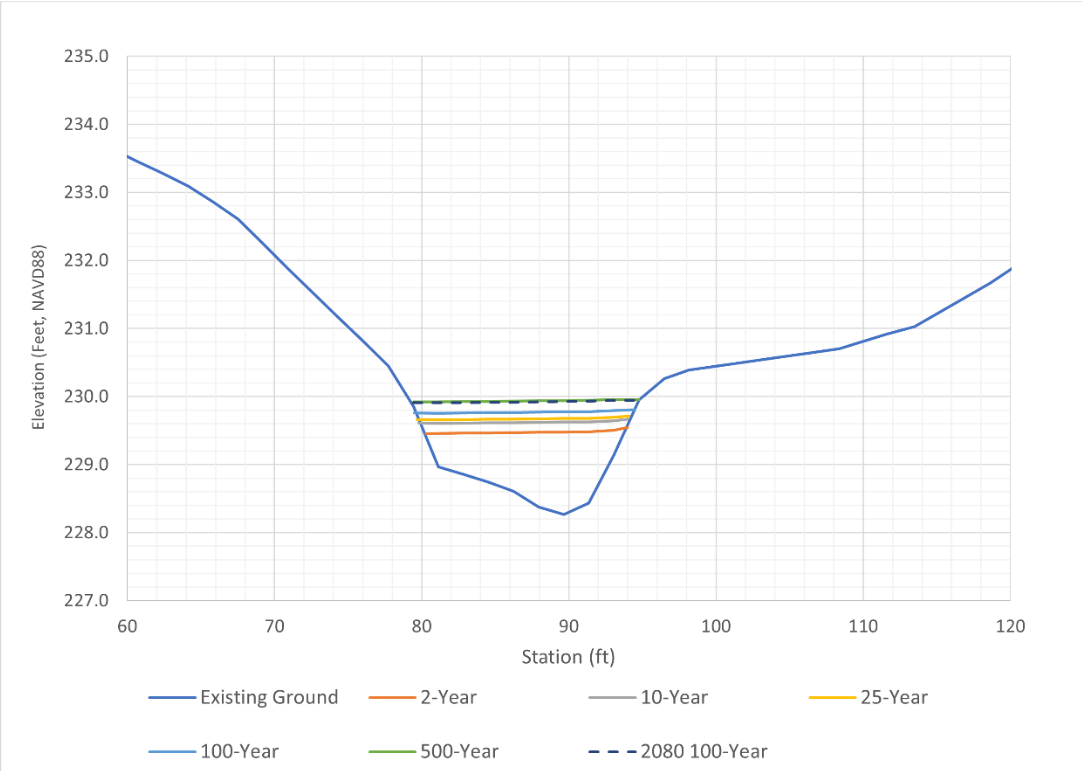


Figure 5-13: Typical upstream existing channel cross section (STA 9+36)

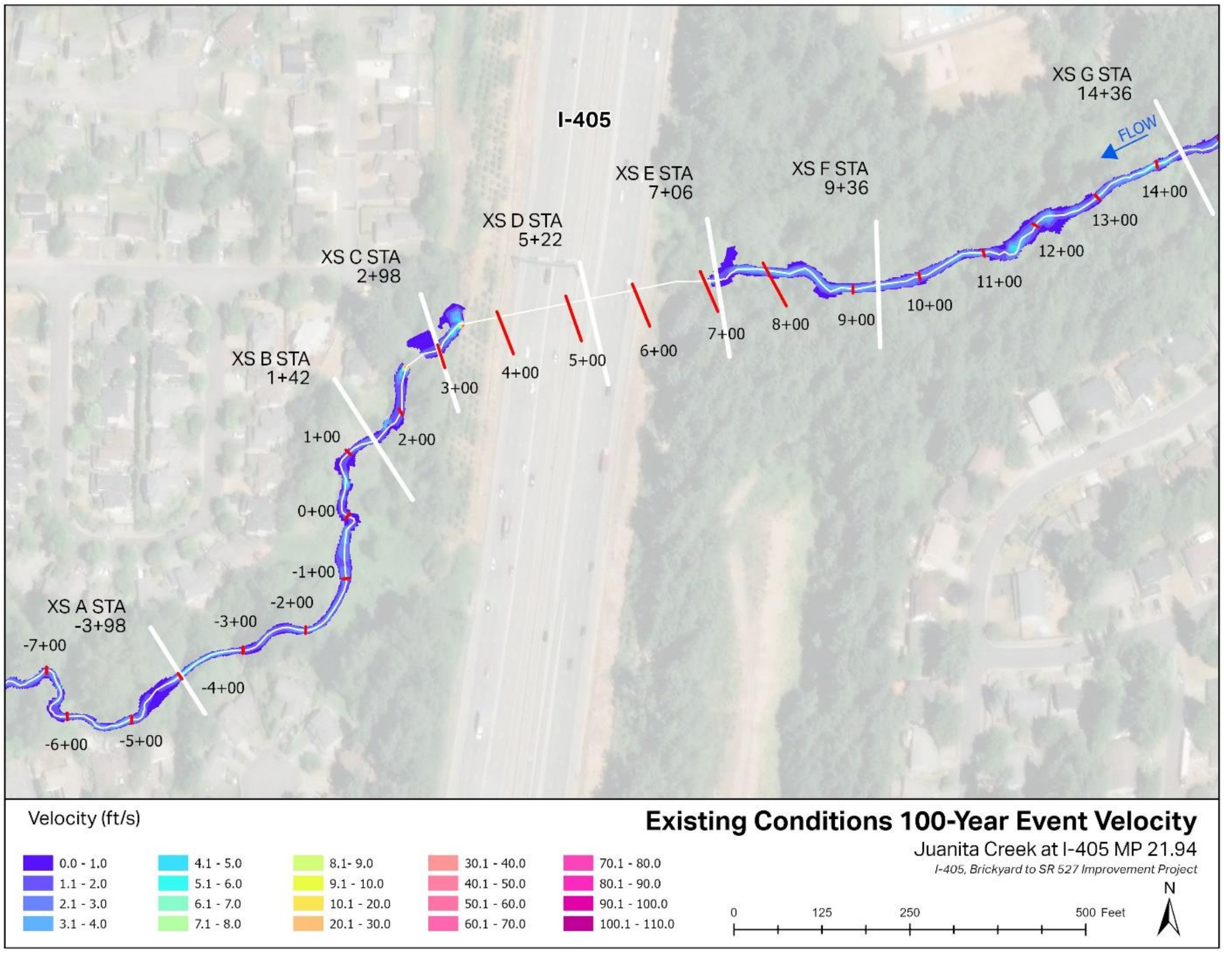


Figure 5-14: Existing conditions 100-year velocity map with cross-section locations

Table 5-3: Existing conditions average channel and floodplain velocities

Cross-section Location	Q100 Average Velocities Tributary Scenario (ft/s)		
	LOB[1]	Main Channel	ROB[1]
DS -3+98 (A)	2.2	3.8	NA
DS 1+42 (B)	0.2	2.0	1.6
DS 2+98 (C)	0.1	1.0	0.8
US 7+06 (D)	0.6	1.7	0.1
US 9+36 (E)	NA	2.6	2.6
US 14+36 (F)	0.9	2.4	1.0

Note:
[1] Right overbank (ROB)/left overbank (LOB) locations were approximated.

5.3 Natural Conditions

A natural conditions model was not required as the system is confined.

5.4 Proposed Conditions: 23-foot Minimum Hydraulic Width

The hydraulic width is defined as the width perpendicular to the creek beneath the proposed structure that is necessary to convey the design flow and allow for natural geomorphic processes. The hydraulic modeling assumes vertical walls at the edge of the minimum hydraulic width unless otherwise specified. See **Section 4.2.2** for a description of how the minimum hydraulic width was determined. **Figure 5-15** shows the locations of cross sections on the proposed alignment used for results reporting. **Table 5-4** lists the average main channel hydraulic results for the proposed conditions. **Figure 5-16** shows water surface profiles for the proposed conditions. **Figure 5-17** shows a typical section through the proposed structure. **Figure 5-18** presents the 100-year velocity map for the proposed conditions, and **Table 5-5** lists the proposed conditions average channel and floodplain velocities.

The SRH-2D model of the proposed conditions and the WSE figures indicate the proposed 30-foot-span structure easily passes flow through the project site without overtopping or creating a buildup of headwater upstream. The culvert immediately downstream of the I-405 culvert outlet is removed in proposed conditions, so there is no overtopping or flooding. Proposed velocities and shear stresses are inherently higher in the proposed conditions due to the lack of impoundment caused by existing culverts. Although there is an increase in these values, they are expected given the proposed channel geometry and do not pose any ill effects to fish migration upstream. Flow is present at all discharge events in the proposed side channel design located on the right bank upstream of the I-405 crossing while looking downstream. Flow begins to spill over into the benches at the beginning of the side channel for all recurrence interval flows.

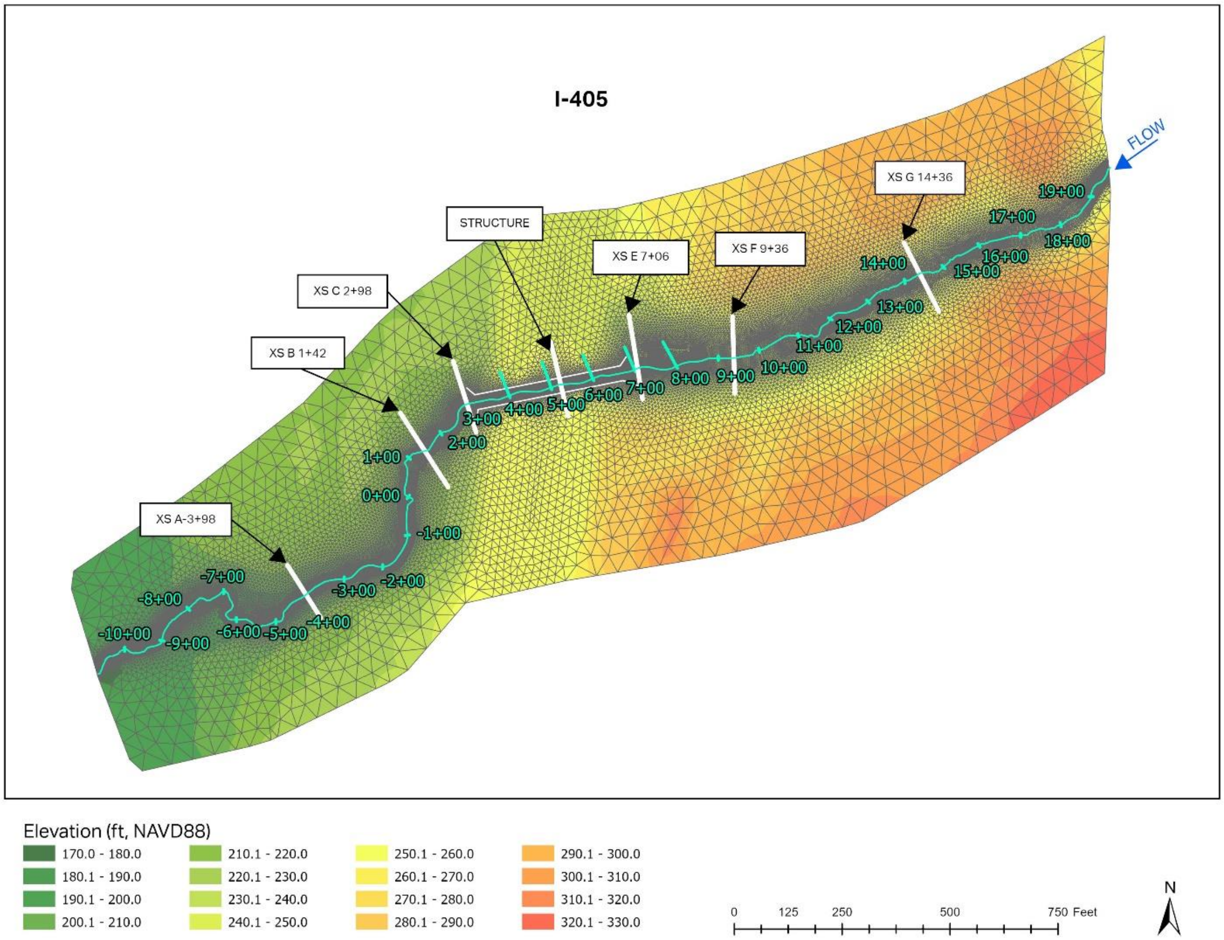


Figure 5-15: Locations of cross sections on proposed alignment used for results reporting

Table 5-4: Average main channel hydraulic results for proposed conditions

Hydraulic Parameter	Cross Section	2-year	100-year	Projected 2080 100-year	500-year
Average WSE (ft)	DS -3+98 (A)	191.0	191.8	192.0	192.3
	DS 1+42 (B)	205.2	206.3	206.5	206.9
	DS 2+98 (C)	207.3	208.3	208.6	209.0
	Structure 5+22 (D)	214.1	214.8	214.9	215.1
	US 7+06 (E)	219.6	220.4	220.6	220.9
	US 9+36 (F)	229.7	230.6	230.7	230.9
	US 14+36 (G)	238.4	239.3	239.5	239.8
Max depth (ft)	DS -3+98 (A)	1.4	1.9	2.1	2.7
	DS 1+42 (B)	1.8	1.5	1.8	3.5
	DS 2+98 (C)	2.4	1.9	2.2	4.1
	Structure 5+22 (D)	2.4	1.2	1.4	3.5
	US 7+06 (E)	2.6	1.7	1.9	3.9
	US 9+36 (F)	1.5	1.5	1.6	2.7
	US 14+36 (G)	2.2	1.0	1.2	3.6
	DS -3+98 (A)	4.3	5.9	6.2	6.7

Hydraulic Parameter	Cross Section	2-year	100-year	Projected 2080 100-year	500-year
Average velocity (ft/s)	DS 1+42 (B)	2.2	3.1	3.4	3.8
	DS 2+98 (C)	2.1	3.0	3.2	3.4
	Structure 5+22 (D)	2.0	3.5	3.9	4.3
	US 7+06 (E)	1.9	3.1	3.4	3.8
	US 9+36 (F)	2.6	3.9	4.3	4.7
	US 14+36 (G)	1.9	3.2	3.6	4.0
Average shear (lb/SF)	DS -3+98 (A)	3.6	5.7	6.2	6.9
	DS 1+42 (B)	0.8	1.3	1.4	1.7
	DS 2+98 (C)	0.7	1.2	1.3	1.4
	Structure 5+22 (D)	0.5	1.3	1.5	1.8
	US 7+06 (E)	0.6	1.4	1.6	1.8
	US 9+36 (F)	0.8	1.5	1.8	2.1
	US 14+36 (G)	0.4	1.0	1.2	1.4

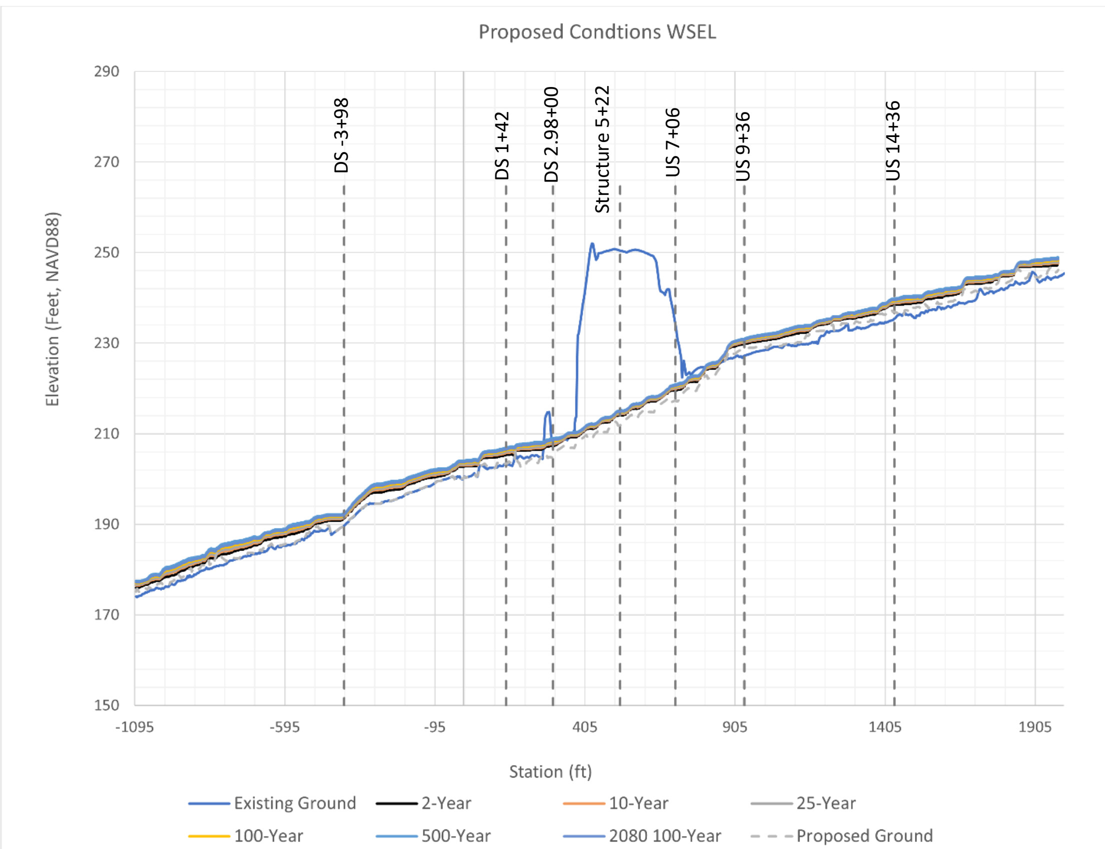


Figure 5-16: Proposed conditions water surface profiles

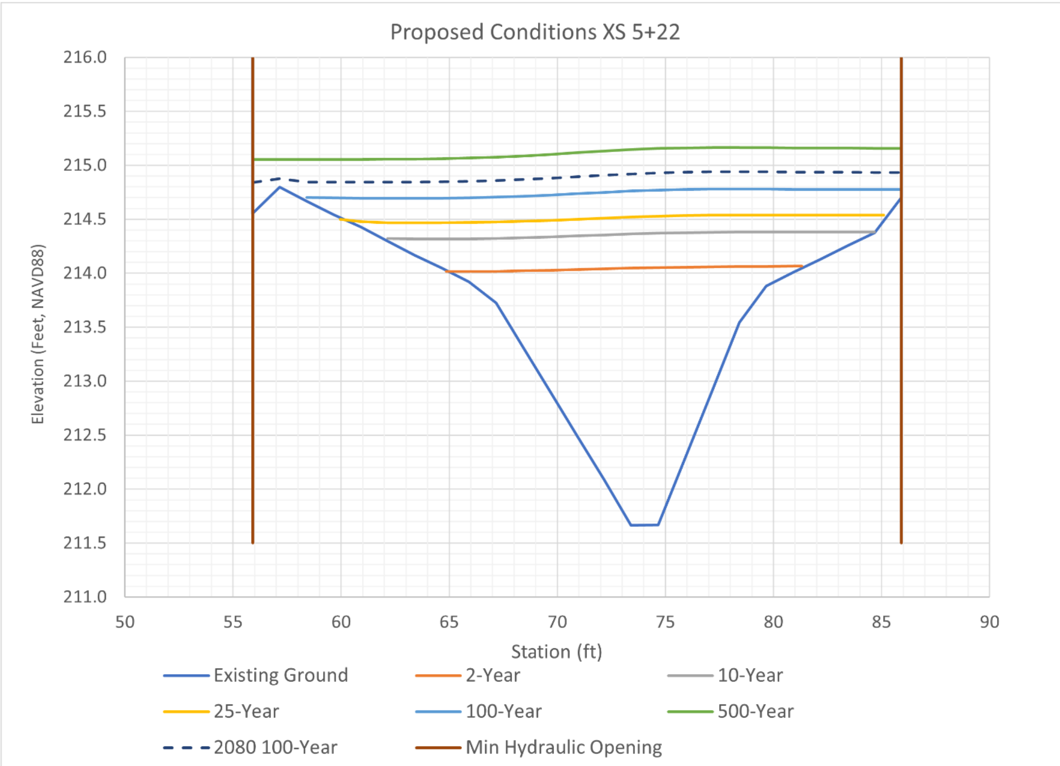


Figure 5-17: Typical section through proposed structure (STA 5+22)

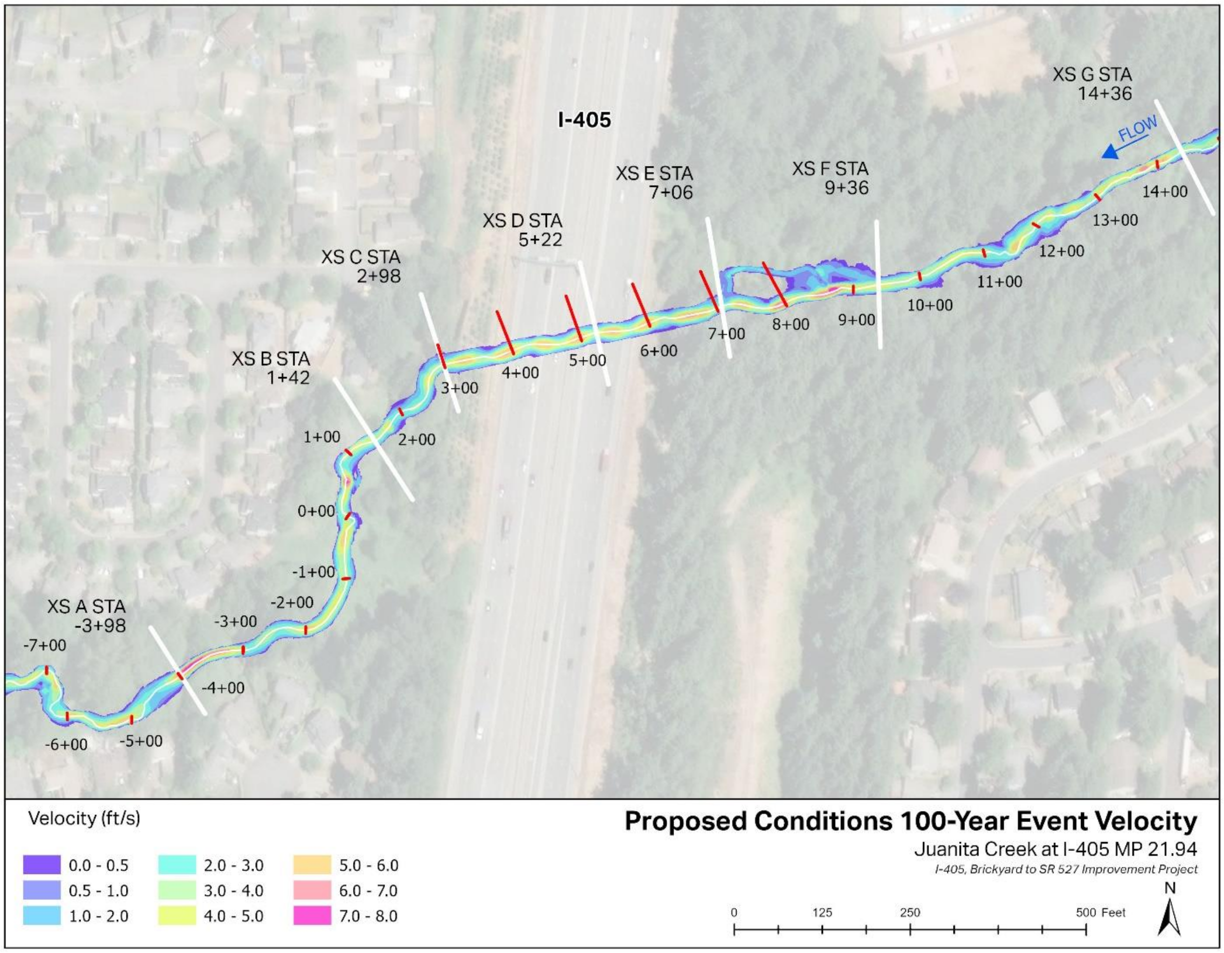


Figure 5-18: Proposed conditions 100-year velocity map

Table 5-5: Proposed conditions average channel and floodplains velocities

Cross-section Location	Q100 Average Velocities (ft/s)			2080 Q100 Average Velocity (ft/s)		
	LOB[1]	Main Channel	ROB[1]	LOB[1]	Main Channel	ROB[1]
DS -3+98 (A)	2.9	5.9	3.8	3.26	6.2	4.3
DS 1+42 (B)	1.0	3.1	0.0	1.16	3.4	0.2
DS 2+98 (C)	1.8	3.0	0.7	1.75	3.2	0.8
Structure 5+22 (D)	1.7	3.5	1.0	1.84	3.9	1.4
US 7+06 (E)	1.9	3.1	1.0	2.21	3.4	1.3
US 9+36 (F)	1.7	3.9	1.0	1.56	4.3	1.1
US 14+36 (G)	0.7	3.2	2.4	0.6	3.6	2.1

Note:
[1] Right overbank (ROB)/left overbank (LOB) locations were approximated.

6 Floodplain Evaluation

This project is not within a FEMA special flood hazard area (SFHA); see **Appendix A** for the floodplain insurance rate map. The existing and proposed project conditions were evaluated to determine whether the project would cause a change in flood risk.

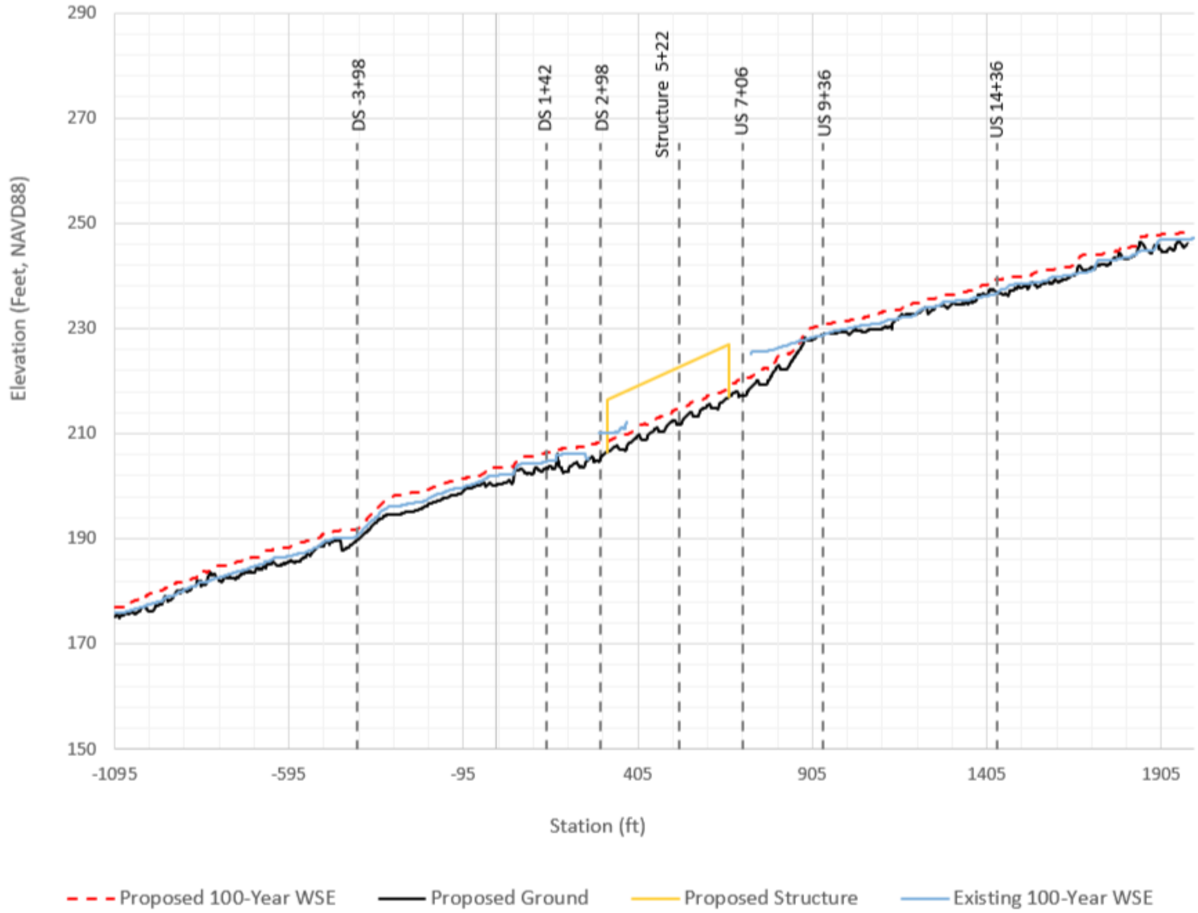
It is expected that after the proposed structure is built, and the regional detention facility is removed, that there will be a reduction in the amount of floodplain utilized upstream of the crossing, as the proposed design will not detain as much of the storm flows. Downstream, there will likely be an increase in floodplain inundation.

With the removal of the existing inline detention facility, upstream floodplain storage will no longer occur in high- flow events. If uncertainty surrounding the loss of detention from the removal of the detention facility is resolved, the risk to downstream infrastructure is considered minimal due to the relative low magnitude of expected flows and the confined nature of the downstream channel.

6.1 Water Surface Elevations

A flood risk assessment will be developed during later stages of the design. **Figure 6-1** compares existing and proposed 100-year water surface profiles along the proposed alignment. **Figure 6-2** and **Figure 6-3** show the 100-year WSE change from existing to proposed conditions.

The proposed model suggests that in the vicinity of the crossings, WSEs will lower due to the larger hydraulic opening, allowing high-flow events to pass unobstructed. The removal of the existing structure and the implementation of LWM in the system will slightly increase WSEs in areas further downstream and upstream, outside the vicinity of the proposed structure, reaching a new system equilibrium. Restoring this section of stream to natural conditions should not increase WSEs to a level considered to pose an adverse risk to downstream infrastructure.



Note: Existing and proposed conditions 100-year flows at the upstream BC are 41 cfs and 111 cfs, respectively (Table 3-1).

Figure 6-1: Existing and proposed conditions 100-year water surface profile comparison along proposed alignment

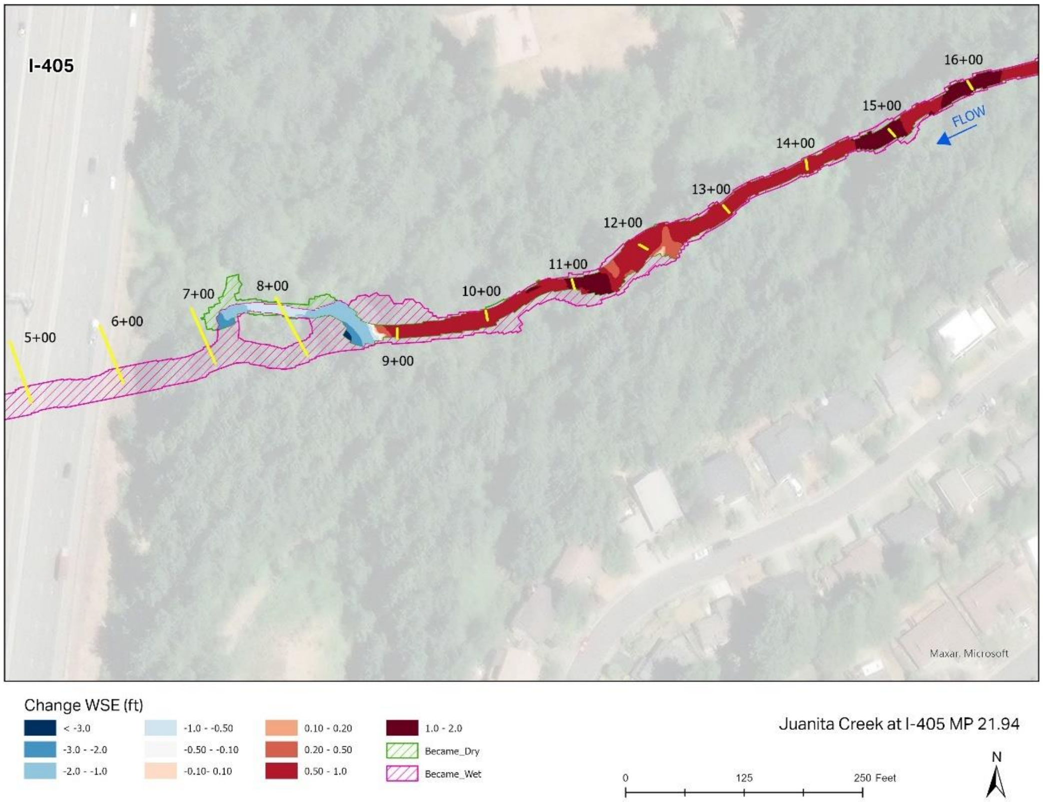


Figure 6-2: Upstream 100-year WSE change from existing to proposed conditions

Figure 6-3: Downstream 100-year WSE change from existing to proposed conditions

7 Preliminary Scour Analysis

Scour components considered in the analysis include long-term degradation, contraction scour, and local scour. The potential for lateral migration was also assessed to evaluate total scour at the proposed highway infrastructure. These various scour components will be discussed in the following sections. All calculations are included in **Appendix K.**

It is worth noting that the preliminary scour analysis is based on the preliminary fish passage structure length, and the structure has since been shortened. The shortened structure length is reflected in the plan sheets (**Appendix D**) but not in the hydraulic modeling results. Scour analysis will be updated when the hydraulic model is updated for the FHD.

7.1 Lateral Migration

The risk of lateral migration within the project reach was assessed based on the variables and evaluations described in this section and on the conditions described in **Section 2.7.4**. The assessment shows that there is low to moderate potential for lateral migration within the project reach.

Historical aerial imagery available through the King County GIS Spatial Data Warehouse and Google Earth Pro was reviewed for potential evidence of bend meander or channel widening. The available imagery dates from 1990 to the present. The resolution of available imagery is not refined enough to document small to moderate changes in the Juanita Creek channel and does not show any significant realignment of the channel.

A rapid assessment of stream stability was completed following the procedure outlined in the Federal Highway Administration's (FHWA's) *Hydraulic Engineering Circular No. 20 Stream Stability at Highway Structures* (HEC-20) (FHWA 2012). The existing conditions were used to assign a score for each of the 13 stability indicators listed in HEC-20 Table 5.5. The rapid assessment is used to classify overall channel stability as either excellent, good, fair, or poor, based on the stream channel classification and the summation of the 13 indicator scores. Observations made during a site visit in December 2023, along with field observations reported in the 2022 WSDOT PHD, were used to inform the rapid assessment scoring. Characteristics of the stream that contribute to instability are the urbanization of the watershed, channel confinement, adjacent infrastructure limiting floodplain interaction, sandy bank soil texture, steep bank slope angles, and the upstream distance from meander impact point to stream alignment at the crossing. All other indicators rate as fair or good for channel stability. The overall score of 69 indicates that Juanita Creek has good stream stability. A fractional score based on the vertical and lateral stability indicators is used to determine the dominant direction of instability. For Juanita Creek, the vertical rating of 0.43 and a lateral rating of 0.47 indicate that vertical instability is slightly dominant. See **Appendix P** for the scoring assessment.

Although the risk for lateral migration is considered low to moderate, the risk in relation to the structure is assumed to occur for the purposes of scour analysis and estimation of total scour. The geotechnical data available at this time are not sufficient to exclude the risk of lateral migration from the scour analysis until detailed geotechnical data (i.e., competent bedrock, geotechnical evaluation for soil erodibility, stream power versus soil erodibility, etc.) is available to support the assessment of no lateral migration being anticipated over the life (75+ years) of the proposed structure.

Slash material will be layered into the lifts of streambed sediment to provide channel stability.

7.2 Long-term Degradation of the Channel Bed

For Juanita Creek, the design is required to allow for the channel to naturally regrade, which could come in the form of long-term degradation (LTD) or aggradation.

LTD of the channel bed was evaluated using two equilibrium slope methods and a downstream grade control feature as the basis for base level control. The two equilibrium slope methods used are the Meyer-Peter Muller (MP) and the Schoklitsch (SCH) methods. The D_{50} and D_{90} from the pebble counts (as described in **Section 2.7.3** and **Appendix N**) were used to estimate the equilibrium slope (Pemberton and Lara 1984). Hydraulic information used in these equations comes from the 2-year hydraulic results. This is because the dominant discharge is responsible for affecting the ultimate shape and conditions of the channel. This dominant discharge that influences the channel geometry and function is the bankfull flow that is often estimated as the 2-year flow (Pemberton and Lara 1984).

Because there are no identifiable and definitive grade control points caused by non-erodible geologic features or other permanent structures, Juanita Creek's confluence with an unnamed tributary near the southwest corner of Edith Moulton Park in the City of Kirkland was used as the base level control. This location is approximately 1.3 miles downstream of the I-405 crossing and represents the limit at which an equilibrium slope calculation may be considered applicable. Equilibrium slope equations such as the Meyer-Peter Muller and Schoklitsch methods use reach-specific hydraulic parameters including bankfull width, dominant discharge, and sediment size. As Juanita Creek flows towards Lake Washington, this tributary confluence is the first significant flow change that greatly influences stream conditions such as discharge and bankfull width; therefore, using a base level control any further downstream of this location may be considered inappropriate. Plotted on **Figure 7-1** is the entire profile extending to Lake Washington, along with the two equilibrium slopes using the tributary confluence as the base level control. As shown on the figure, the average of these two methods produces the blue line that is above the ground elevation line and results in no LTD at the I-405 crossing. The averaged line is taken as the recommended approach in the *Technical Guideline for Bureau of Reclamation, Computing Degradation and Local Scour* (Pemberton and Lara 1984). Because the profile begins to flatten out as Juanita Creek approaches Lake Washington, moving the base level control farther downstream would also result in zero LTD.

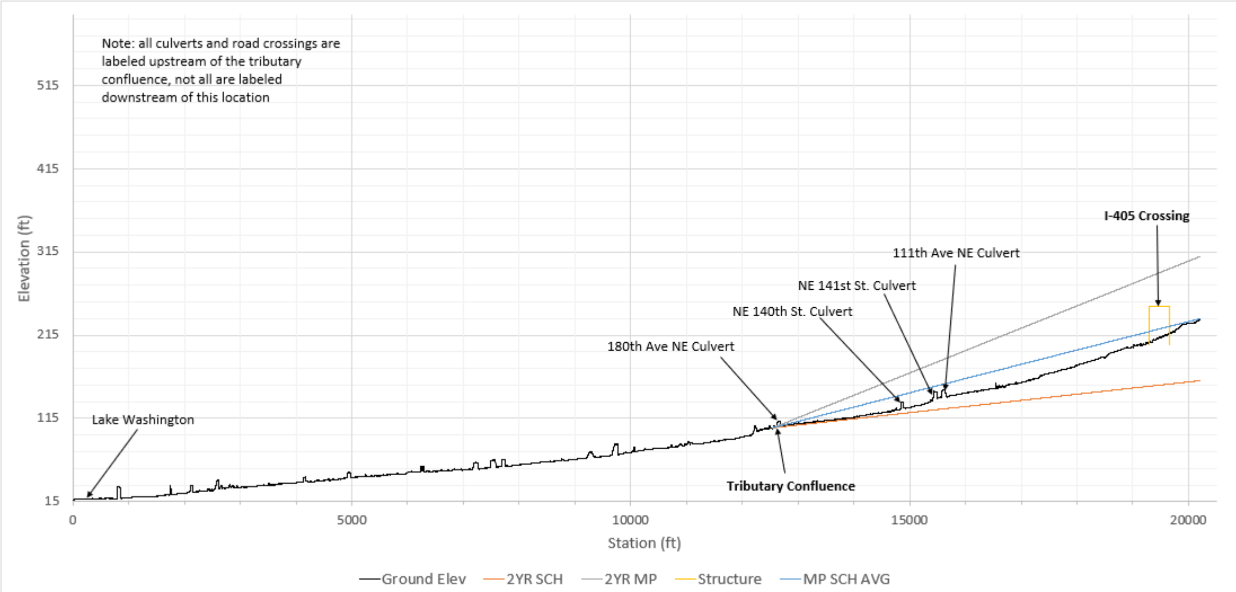


Figure 7-1. LTD Profile

Even though the equilibrium slope approach discussed above estimated no LTD for the Juanita crossing, an additional approach was considered for comparison. The Technical Guideline for Bureau of Reclamation, Computing Degradation and Local Scour recommends analyzing the ability for a streambed to develop a natural armor layer that would arrest any degradation to the depth at which this armor layer forms (Pemberton and Lara 1984). This approach is recommended for situations where a minimum of 10 percent of the material is coarse enough to resist transport. This in turn could limit degradation to a depth shallower than that calculated using equilibrium slope methods. This armor layer approach used the existing stream bed gradation (D_{50} = 1.01 inches), calculated from the pebble count data taken upstream of I-405; this material is likely less coarse than material downstream due to the in-line detention facility, and thus produces a conservative estimate of armoring depth. The armoring depth from this analysis is 0.88 feet and would likely be less if data were collected for material downstream of I-405. This result indicates that a coarse armor layer would develop at a depth of 0.88 feet, at which point LTD would be arrested.

A degradation of 0.88 feet would be applied to total scour using existing conditions as degradation or incision is initiated downstream. For the proposed design, in order to provide a stable bed, the streambed material needs to be coarser than existing material as described in **Section 4.3.** This difference in size creates a more stable streambed and also acts as a profile stability measure. In order to quantify the effects of this coarser streambed, the same armor layer depth approach was used. Using the design riffle and meander bar gradation (D_{50} = 6.96 inches), which will make up the majority of the streambed (and will be placed beneath the finer pool streambed sediment to a depth of 5 feet), the armoring depth for the proposed I-405 crossing is 0.14 feet (**Table 7-1**). This indicates that the riffle and meander bar sediments will act as an armor layer and not degrade significantly. Downstream of the proposed grading limits, if a scour depth of 0.88 feet developed over time and began to headcut up to the downstream end of the proposed streambed, the proposed riffle material will stop the headcut from migrating further upstream to the structure. The proposed streambed material outside of the structure is placed to a depth of 3 feet and therefore would be protected down to the LTD depth of 0.88 feet and an additional 2.12 feet. If this condition (or greater degradation) were to occur, it would also not likely result in a fish passage barrier (due to water drop) because the proposed riffle material would respond by creating a ramp connecting the degraded portion of the stream to the proposed streambed. This would effectively function as a steeper riffle. The proposed bed material acting as a profile stability measure from this analysis has implications on applicable structural total scour.

The armoring depth approach produced a more conservative estimate than the equilibrium slope approach for calculating LTD, and thus was chosen for the final LTD estimate. A LTD depth of 0.14 feet will be included in the total scour estimate. All calculations for evaluating LTD are found in **Appendix K.2** and **K.3.**

Table 7-1: LTD Summary table

	Long-term Degradation Depth (ft)
Equilibrium slope	0.0
Depth to armor layer (downstream of grading extents)	0.88
Depth to armor layer including stability measures (proposed streambed)	**0.14***

*Included in estimate of total scour

7.3 Contraction Scour

Based on the structure width of 30 feet that was established during preliminary phases of this project, contraction scour was calculated using clear-water conditions because the critical velocity of existing sediment at the approach section is greater than that of average velocity at the approach section (5.5 and 5.3 feet per second, respectively). The 2-, 10-, 25-, 100-, 2080 100-, and 500-year events were evaluated for contraction scour, and no events resulted in a positive scour depth. This calculation was performed as a separate check on contraction scour than what is discussed in **Section 7.4.3,** where abutment and contraction scour are evaluated jointly using the National Cooperative Highway Research Program 24-20 (NCHRP) method.

Within the SRH-2D hydraulic model, all flow events were evaluated using the same bridge scour coverage in the FHWA Hydraulic Toolbox, per SRH-2D and SMS best practices. FHWA Hydraulic Toolbox scour calculations are found in **Appendix M.**

The sediment size used for preliminary contraction scour analysis at the contracted section was the designed riffle sediment, as seen in **Section 4.3.1**. This is because a straight riffle bedform is designed to be present at the entrance of the culvert and is expected to be relatively stable over time. The design will allow sediment transport and deformability as is beneficial and required for fish passage projects, but the bedforms are designed to be stable and not rapidly wash through the structure to provide reliable fish passage and resistance to degradation of the streambed.

7.4 Local Scour

7.4.1 Scour at LWM

LWM is expected to create local scour holes, which are beneficial for creating habitat diversity and complexity but can potentially destabilize logs or expose structural elements to scour. Caution needs to be taken when placing LWM near structural elements such as culvert wing walls. Per the WSDOT *Hydraulics Manual* Chapter 10-6.5, reliable methods for estimating scour at LWM placements have not yet been developed in either the engineering or scientific communities. In some cases, equations developed for bridge piers and abutments have been used to predict scour, but these are overly conservative for gravel bed streams found in much of Washington and may not accurately represent the unique geometry of LWM. Scour analysis for LWM projects will therefore often rely heavily on engineering judgment and lessons learned from practical experience.

As the LWM layout and design is finalized during the FHD process, local scour resulting from LWM will be evaluated. In order to estimate the potential scour from LWM, the *Hydraulic Engineering Circular No.18* (HEC-18) pier scour equation will be used at LWM structures and at various flows. It is worth noting that the HEC-18 pier scour equation is based on the Colorado State (CSU) equation (FHWA 2012); after development of this approach, Landers et al. concluded that the CSU equation frequently over-predicted the observed scour in laboratory studies, resulting in maximum scour depths (FHWA 2012). When applying the HEC-18 equation to geometries like LWM that differ from the pier shapes that the equation was developed for, caution needs to be taken with the resulting scour depths. Thus, the less conservative Julien 2002 equation will also be evaluated (USBR 2015). The U.S. Bureau of Reclamation *Bank Stabilization Guidelines* (2015) outline methods for the design of engineered log jams for bank stabilization that include the evaluation of local scour using the Julien equation, which generally provides less conservative results than the HEC-18 method.

The results of these calculations will be used in the FHD design to ensure that LWM is placed far enough from structural features like wing walls so these are not undermined.

7.4.2 Pier Scour

There are no existing piers present in the project vicinity and the proposed work does not include any new piers. Therefore, pier scour was not evaluated.

7.4.3 Abutment Scour

Abutment scour was estimated using the NCHRP 24-20 approach for all flood events up to the scour design flood and scour check flood using a clear-water condition. The NCHRP 24-20 method jointly evaluates contraction scour and abutment scour by applying an amplification factor to contraction scour. As outlined in HEC-18, these results (separate from those found in Section 7.3) represent total scour at the abutment and would not be added to contraction scour if the calculation in Section 7.3 resulted in positive scour depths (FHWA 2012).

Abutment scour equations estimate scour depths of 1.6 and 1.8 feet at the 2080 projected 100-year event and the 500-year event, respectively. See **Appendix K.1** for the hydraulic toolbox report and abutment scour detailed information.

Abutment scour is evaluated at the entrance of the proposed culvert. For the PHD, the preliminary wing walls were incorporated into the hydraulic model geometry but will be updated for the FHD as the design of these wing walls is finalized and the overall length of the crossing is shortened. In the FHWA Hydraulic Toolbox, a vertical wall abutment with wing walls was assumed as the abutment type.

7.4.4 Bend Scour

Bend scour was evaluated with two methods within the I-405 fish passage culvert and details are included in **Appendix K.3**. Because Juanita Creek is a gravel bed stream, bend scour equations such as the Maynord Equation and Wattanabe Equation are not as applicable as those that were developed for use with gravel bed channels (Cramer 2012). Therefore, the Thorne Equation and the USACE Equation were evaluated for Juanita Creek, as these were developed for use in gravel streams (USACE 1994). As seen in **Figure 7-2**, bend scour was calculated at each bend within the proposed crossing.

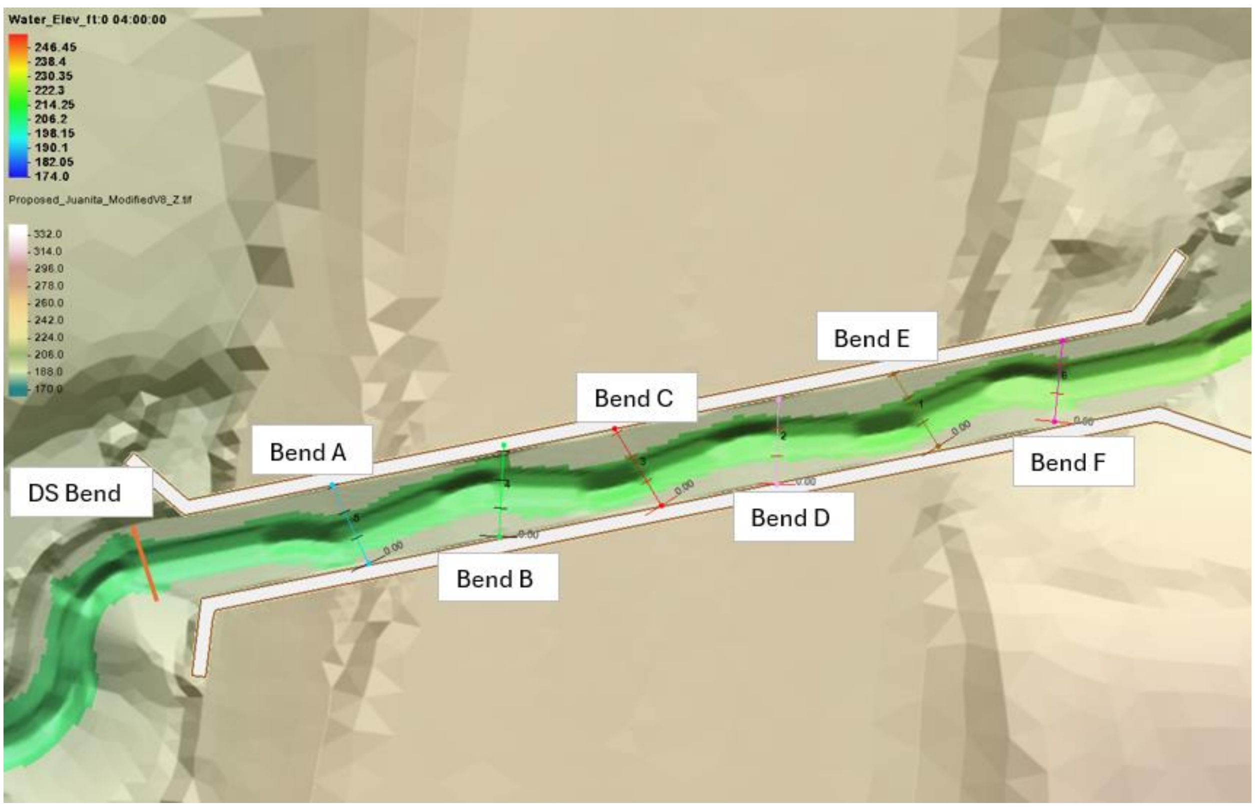


Figure 7-2. Bend scour locations

As seen in Table 7-2, resulting scour values for each method produce similar results. The overall maximum bend scour estimate is 1.4 feet at the furthest upstream Bend F**,** using the USACE Equation.

Table 7-2: Bend scour results

	Bend A	Bend B	Bend C	Bend D	Bend E	Bend F	DS Bend
Thorne Equation Scour Depth (ft)	0.9	1.2	1.1	1.2	1.1	1.3	1.0
USACE Equation Scour Depth (ft)	0.7	1.3	1.0	1.1	1.0	1.4	0.8

Scour depths for bend scour are not included in the total scour estimate because riffles and pre-scoured pools are included in the design of Juanita Creek's bedform. The residual pool depth (difference in depth between the riffle typical section and pool typical section) is 1.27 feet, which was designed to provide adequate energy dissipation and resting pools for fish. Because this depth is less than, or approximately equal to, the anticipated bend scour, it is not expected that scour at pools due to bends will remove material much further below the already graded pool bottoms. Bend scour is not anticipated to interact with changing bedforms because the relative locations of pools and riffles are designed to be stable over time and generally hold their locations. Furthermore, the bend scour estimates are less than the total scour and are unlikely to create a condition where this local scour extends below the total scour value because of the general stability of bedform locations.

The bend downstream of the I-405 crossing (denoted by DS Bend) was evaluated for bend scour because the stream at this location meanders and the outside of the bend is near existing fences and properties. Some property owners have already installed their own rock protection measures likely due to concerns of erosion. The more conservative estimated bend scour at this location is approximately 1 foot using the Thorne Equation. For the final design, this scour depth will guide the design of the vegetated revetment and its required depth of protection.

7.5 Total Scour

For Juanita Creek**,** there are three components of total scour: long-term degradation, contraction scour, and local scour in the form of abutment scour.

For Juanita Creek, all flows were evaluated up to the 500-year flow, and only the 2080 100-year and the 500-year flow produced scour. The scour design flood is represented by the 2080 100-year event and resulted in a total scour of 1.7 feet. The scour check flood is the 500-year event and produces a deeper scour of 1.8 feet. This 1.9-foot depth is applied below the channel thalweg and horizontally to the structure walls due to the risk of lateral migration over the lifespan of the crossing structure. Per the WSDOT Hydraulics Manual (WSDOT 2023a), a minimum scour depth of 3 feet shall be used for all 3 sided water crossing structures. Because the estimated total scour is less than 3 feet, the minimum value is reported for total scour.

Calculated total depths of scour for the scour design flood and scour check flood at the proposed Juanita Creek crossing are summarized in **Table 7-3**, and shown on the drawings, which are provided in **Appendix D.**

Table 7-3: Scour analysis summary

Calculated Scour Components and Total Scour for Juanita Creek		
Long-term degradation (ft)	0.14	
	Scour design flood	**Scour check flood**
HEC-18 contraction scour (ft)	0	0
NCHRP 24-20 contraction and abutment scour (ft)	1.6	1.8
Total depth of scour (ft)	1.7	1.9
WSDOT minimum scour depth (ft)	3	

8 Scour Countermeasures

As described in **Section 7.5**, the preliminary scour analysis shows no contraction scour and minimal local abutment scour. However, the scour analysis did not consider the various channel complexity features that were added to mitigate the potential for long-term degradation (see **Sections 4.3.2** and **7.5**).

No countermeasures are recommended to protect the abutments from scour at the Juanita Creek crossing of I-405. If scour countermeasures are later determined to be necessary as part of the FHD, a specialty report will document the risk to the infrastructure and rationale for protection. Implementation of scour countermeasures will follow the guidance of HEC-23 Volumes 1 and 2 (Lagasse et al. 2009).

Downstream of the I-405 crossing, vegetated revetments are proposed on the right bank of Juanita Creek to protect adjacent properties. Detailed design will be conducted for the FHD (see **Appendix D**).

9 Summary

Table 9-1 presents a summary of the results of this PHD Report.

Table 9-1: Report summary

Stream Crossing Category	Element	Value	Report Location
Habitat gain	Total length	1,309 LF	2.1 Site Description
Bankfull width	Reference reach found?	Yes	2.7.1 Reference Reach Selection
	Design BFW	12 feet	2.7.2 Channel Geometry
	Concurrence BFW	12 feet	2.7.2 Channel Geometry
Floodplain utilization ratio (FUR)	Flood-prone width	12.08 feet	2.7.2.1 Floodplain Utilization Ratio
	Average FUR	2.11	2.7.2.1 Floodplain Utilization Ratio
Channel morphology	Existing	See link	2.7.2 Channel Geometry
	Proposed	See link	4.3.2 Channel Complexity
Hydrology/design flows	100-year flow existing/proposed	41 cfs/111 cfs	3 Hydrology and Peak Flow Estimates
	2080 100-year flow existing/proposed	50 cfs/135 cfs	3 Hydrology and Peak Flow Estimates
	2080 100-year used for design	Yes	3 Hydrology and Peak Flow Estimates
	Dry channel in summer	No	2.1 Site Description
Channel geometry	Existing	See link	2.7.2 Channel Geometry
	Proposed	See link	4.1.1 Channel Planform and Shape
Channel slope/gradient	Existing culvert	3.0%	2.6.2 Existing Conditions
	Reference reach	1.7%	2.7.1 Reference Reach Selection
	Proposed	3.0%	4.1.3 Channel Gradient
Hydraulic width	Existing	12 feet	2.7.2 Channel Geometry
	Proposed	23 feet	4.2.2 Hydraulic Width
	Added for climate resilience	Yes	4.2.2 Hydraulic Width
Vertical clearance	Required freeboard	3 feet	4.2.3 Vertical Clearance
	Required freeboard applied to 100-year or 2080 100-year	2080 100 yr	4.2.3 Vertical Clearance
	Maintenance clearance	10 feet	4.2.3 Vertical Clearance
	Low chord elevation	See link	4.2.3 Vertical Clearance
Crossing length	Existing	358.6 feet	2.6.2 Existing Conditions
	Proposed	250 feet	4.2.4 Hydraulic Length
Structure type	Recommendation	Yes	4.2.6 Structure Type
	Type	Buried (bottomless)	4.2.6 Structure Type
Substrate	Existing	See link	2.7.3 Sediment
	Proposed	See link	4.3.1 Bed Material
	Coarser than existing?	Yes	4.3.1 Bed Material
Channel complexity	LWM for bank stability	Yes	4.3.2 Channel Complexity

Stream Crossing Category	Element	Value	Report Location
	LWM for habitat	Yes	4.3.2 Channel Complexity
	LWM within structure	Yes	4.3.2 Channel Complexity
	Meander bars	4	4.3.2 Channel Complexity
	Boulder clusters	4	4.3.2 Channel Complexity
	Coarse bands	No	4.3.2 Channel Complexity
	Mobile wood	Yes	4.3.2 Channel Complexity
Floodplain continuity	FEMA mapped floodplain	No	6 Floodplain Evaluation
	Lateral migration	No	2.7.4 Channel Migration
	Floodplain changes?	Yes	6 Floodplain Evaluation
Scour	Analysis	See link	7 Preliminary Scour Analysis
	Scour countermeasures	Determined at FHD	8 Scour Countermeasures
Channel degradation	Potential?	0.14 feet	7.2 Long-term Degradation of the Channel Bed
Channel degradation	Allowed?	Yes	7.2 Long-term Degradation of the Channel Bed

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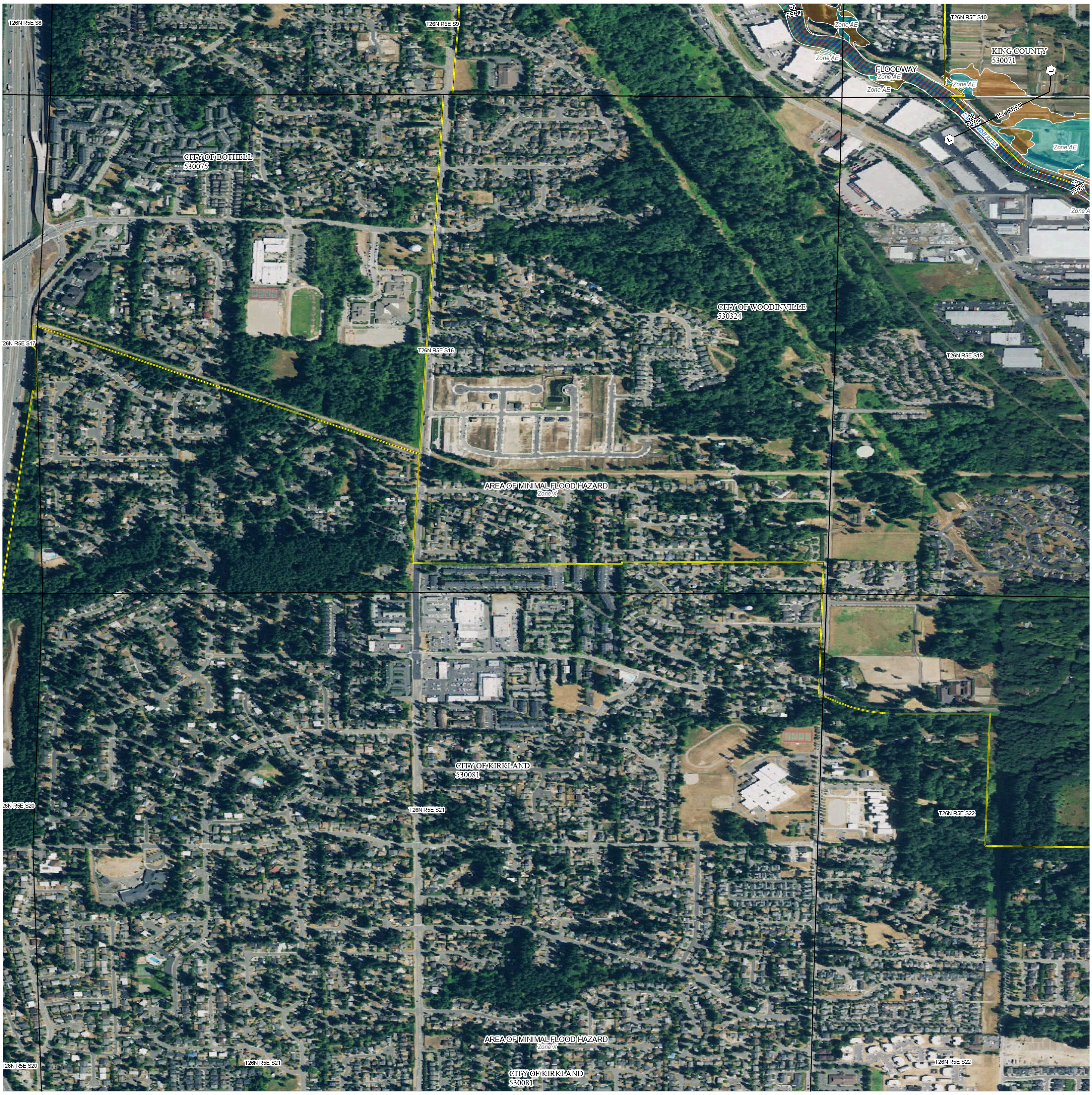
Appendix Q: Juanita Creek Hydrology and Hydraulics Study H21 Report

Appendix A: FEMA Floodplain Map

122°11'15.48"W 47°45'6.61"N



122°9'22.13"W 47°43'N

FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR DRAFT FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee See Notes *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 Cross Sections with 1% Annual Chance
- 17.5 Water Surface Elevation
- 8 Coastal Transect
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary

NOTES TO USERS

For information and questions about this Flood Insurance Rate Map (FIRM), available products associated with this FIRM, including historic versions, the current map date for each FIRM panel, how to order products, or the National Flood Insurance Program (NFIP) in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Flood Map Service Center website at https://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website.

Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM Index. These may be ordered directly from the Flood Map Service Center at the number listed above.

For community and countywide map dates, refer to the Flood Insurance Study Report for this jurisdiction.

To determine if flood insurance is available in this community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Basemap information shown on this FIRM was provided in digital format by USDA, Farm Service Agency (FSA). This information was derived from NAIP, dated April 11, 2018.

This map was exported from FEMA's National Flood Hazard Layer (NFHL) on **6/18/2024 1:57 PM** and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time. For additional information, please see the Flood Hazard Mapping Updates Overview Fact Sheet at https://www.fema.gov/media-library/assets/documents/118418

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards. This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date.

SCALE

Map Projection:
GCS, Geodetic Reference System 1980;
Vertical Datum: NAVD88
For information about the specific vertical datum for elevation features, datum conversions, or vertical monuments used to create this map, please see the Flood Insurance Study (FIS) Report for your community at https://msc.fema.gov

N

1 inch = 500 feet 1:6,000

0 250 500 1,000 1,500 2,000 Feet

0 50 100 200 300 400 Meters

FEMA
National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM
FLOOD INSURANCE RATE MAP

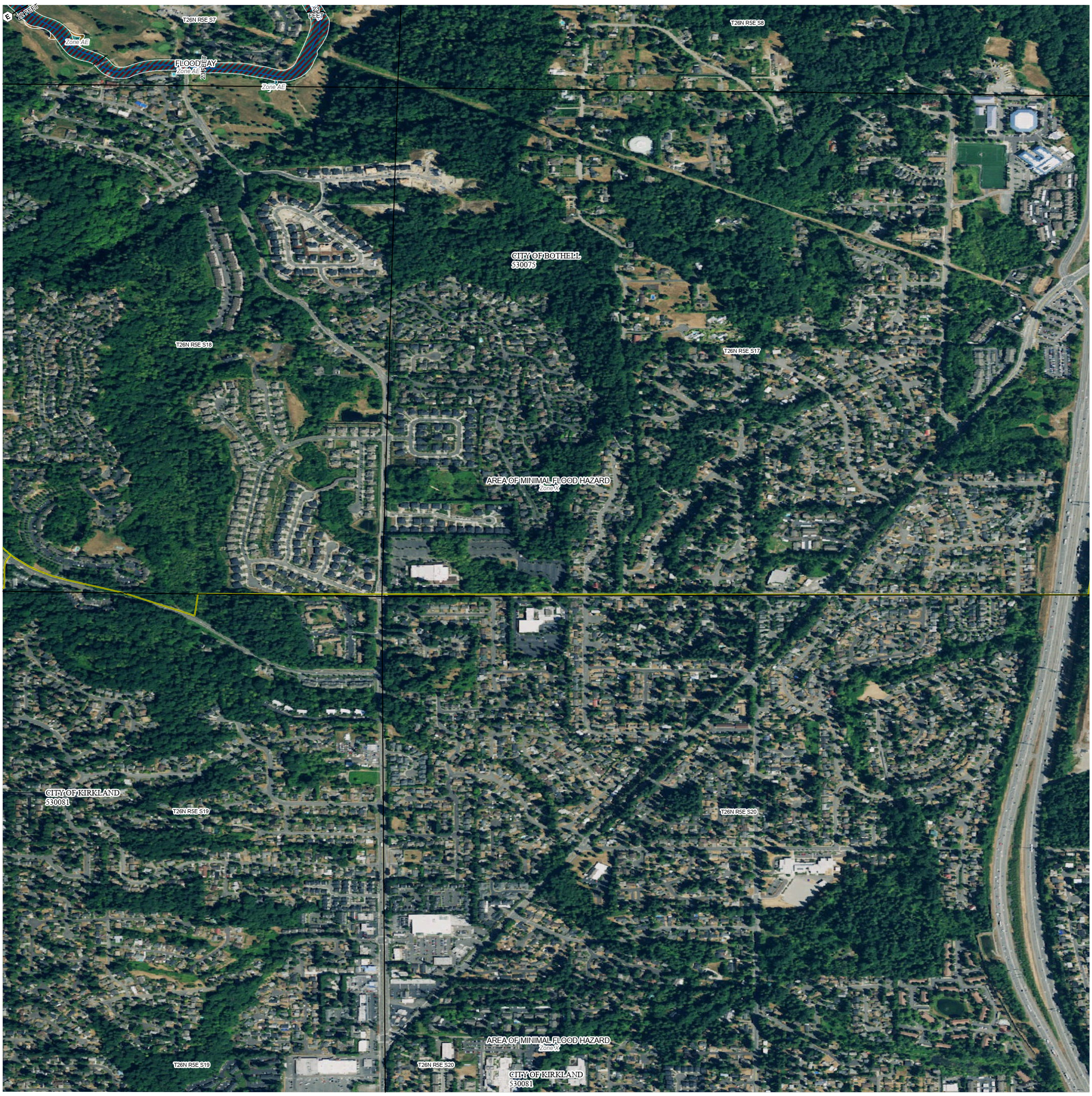
PANEL 356 OF 1700

Panel Contains:

COMMUNITY	NUMBER	PANEL
CITY OF WOODINVILLE	530324	0356
KING COUNTY	530071	0356
CITY OF BOTHELL	530075	0356
CITY OF KIRKLAND	530081	0356

MAP NUMBER
53033C0356H
EFFECTIVE DATE
August 19, 2020

122°13'7.98"W 47°45'6.61"N



122°11'14.63"W 47°43'N

FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR DRAFT FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS	Without Base Flood Elevation (BFE) *Zone A, V, A99*
	With BFE or Depth *Zone AE, AO, AH, VE, AR*
	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes *Zone X*
	Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN Area of Minimal Flood Hazard *Zone X*
	Effective LOMRs
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
OTHER FEATURES	B 20.2 Cross Sections with 1% Annual Chance
	17.5 Water Surface Elevation
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
	Limit of Study
	Jurisdiction Boundary

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SCALE

Map Projection:
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Vertical Datum: NAVD88
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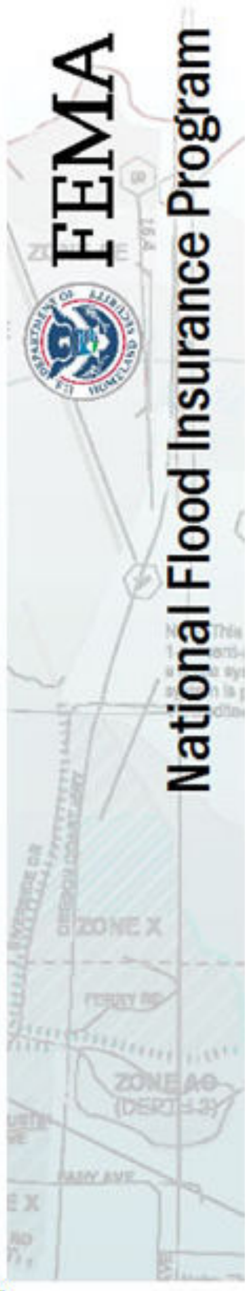
N

1 inch = 500 feet 1:6,000

0 250 500 1,000 1,500 2,000 Feet

0 50 100 200 300 400 Meters



NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

PANEL **352** OF **1700**

Panel Contains:

COMMUNITY	NUMBER	PANEL
CITY OF KENMORE	530336	0352
CITY OF BOTHELL	530075	0352
CITY OF KIRKLAND	530081	0352

MAP NUMBER
53033C0352H
EFFECTIVE DATE
August 19, 2020

Appendix B: Hydraulic Field Report Form

WSDOT Hydraulics Section	Hydraulics Field Report	Project Number:
	Project Name: Brickyard to SR 527 Improvement Project	Date: June/Oct/Dec 2019
	Project Office: I-405 Corridor Program – Mega Projects	Time of Arrival:
	Stream Name: Juanita Creek	Time of Departure:
WDFW ID Number: Site ID 998602	Purpose of Site Visit: Stream Data Collection	Prepared By: Alex Strom, PE
State Route/MP: I-405 MP 21.94	Weather: Overcast all visits	

Meeting Location:
Multiple Locations during visits, mainly accessed from roadside shoulder.

Attendance List:

Name	Organization	Role
Alex Strom	I-405 Corridor Program	Senior WR Engineer- Lead
Maki Dalzell	I-405 Corridor Program	Senior Wetland Biologist
Shelby Reibel	I-405 Corridor Program	Wetland Biologist
Joe Airoldi	I-405 Corridor Program	WR Engineer
Rob Woeck	I-405 Corridor Program	Environmental Director
Martin Fox	MITFD	Fisheries Division
Miles Penk	WDFW	Area Habitat Biologist

Bankfull Width:

Bankfull width measurements were taken during multiple site visits upstream of the I-405 crossing. They were taken upstream of the culvert influence, approx. 300 feet upstream of the existing culvert inlet. During the site visit that included the MITFD and WDFW, 10 measurement were taken approximately 10 meters apart. Measurements ranged from 9.5 feet to 17.5 feet, with an average bankfull width of 12 feet, that was agreed to by MITFD and WDFW.

Reference Reach:

The reference reach is located where the bankfull measurements were taken, upstream of the crossing approximately 300 feet. The Juanita Creek reference reach is located in a forested reach within a highly urbanized basin. The reference reach is above the influence of the backwater caused by the existing inline detention facility at the I-405 crossing. It has similar gradient to the section of stream at the crossing. The reference reach channel is relatively confined, moderate sinuosity, and limited floodplain area and connectivity. The lack of floodplain was discussed during multiple site visits, but other areas that had more floodplain area and connectivity were visited, but these other reaches were not similar enough in slope to be considered as an alternate reference reach. Historical imagery shows that this reach is generally stable and has not likely been subjected to extreme lateral migrations. It was decided that this location was the most suitable reference reach available.

Data Collection:

Alex Strom, Joe Airoldi, Maki Dalzell, and Shelby Reibel were the data collectors. Bankfull width, sediment samples, wetland delineations, riparian habitat, instream habitat, overstory, and understory data and measurements were collected during all site visits. This data was collected from the stream inlet upstream, past the reference reach. The riparian, wetland and over and understory data were also collected downstream of the crossing.

Observations:

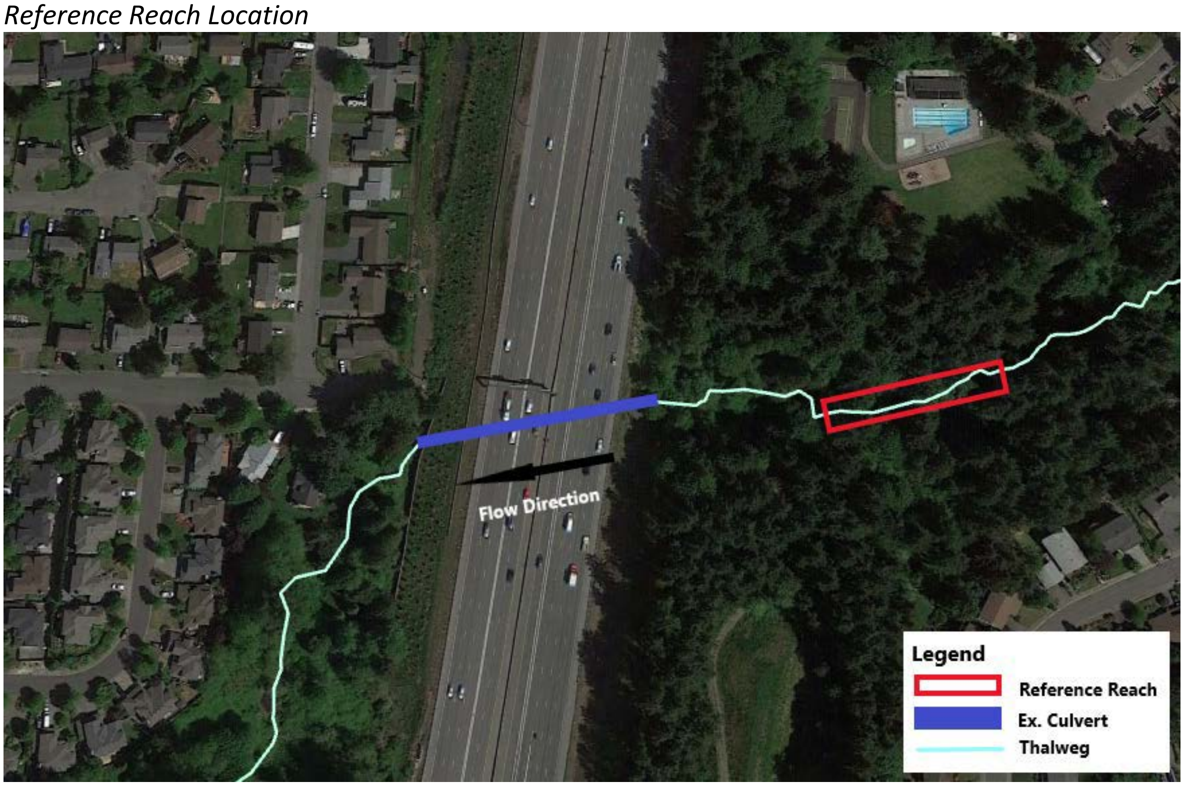
The upstream reaches of Juanita Creek flows in a well-defined channel through a mixed forest valley that contains red alder, big leaf maple, Western red cedar, and Douglas fir. Instream habitat consists of riffles, and sporadic, small pools. Gravels and cobbles make up the bulk of the substrate, except for just upstream of the crossing which is dominated by gravels and sands. This is due to the inline detention facility at the I-405 crossing. There is not much LWM engaged in the channel currently, but the creek has high recruitment potential. Very small pieces were encountered along the upstream reaches. There was only one location upstream of the crossing that contained a Fox and Bolton-defined key piece of LWM. It was located in a temporary natural barrier. It was not evident if the temporary barrier was beaver-influenced, or natural recruitment (see photo). This was located in the upper quarter of the reference reach.

Pebble Counts/Sediment Sampling:

Sediment samples were taken in the reference reach (see photo). These were Wolmann pebble counts. They were taken in riffle sections and seemed very representative of sediment within the reach.

Photos:

Reference Reach Location



Bankfull and Sediment Sample Locations

LWM in Natural Temporary Barrier

Appendix C: Streambed Material Sizing Calculations

Juanita Shear Stress - 100yr

Riffles

Arc 1

	Distance	Value	Mean
1	0.00	0.61	
2	0.76	0.81	
3	2.07	1.34	
4	3.39	1.88	
5	4.70	2.62	
6	6.02	3.51	
7	7.34	3.93	0.37
8	8.65	4.15	0.40
9	9.97	4.54	0.43
10	11.28	4.72	0.46
11	12.60	4.76	0.47
12	13.92	4.68	0.47
13	15.24	4.46	0.46
14	16.55	4.03	0.42
15	17.87	3.31	0.37
16	19.19	2.76	0.30
17	20.51	2.27	
18	21.83	1.67	
19	23.15	1.19	
20	24.47	0.70	
21	25.13	0.58	
	US Avg	4.08	4.17
	US Max	4.76	

Arc 3

	Distance	Value	Mean
1.00	0.00	0.58	
2.00	0.19	0.63	
3.00	1.44	1.08	
4.00	2.69	1.58	
5.00	3.94	2.18	
6.00	5.19	2.82	
7.00	6.44	3.35	0.31
8.00	7.69	3.72	0.35
9.00	8.94	3.94	0.38
10.00	10.19	4.05	0.40
11.00	11.44	4.08	0.41
12.00	12.69	4.06	0.41
13.00	13.94	3.98	0.40
14.00	15.19	3.81	0.39
15.00	16.44	3.54	0.37
16.00	17.69	3.13	0.33
17.00	18.94	2.58	
18.00	20.19	2.03	
19.00	21.44	1.56	
20.00	22.69	1.13	
21.00	23.94	0.73	
22.00	25.19	0.40	
23.00	25.34	0.38	
	Mid Avg	3.68	3.75
	Mid Max	4.08	

Arc 5

	Distance	Value	Mean
1.00	0.00	0.62	
2.00	0.62	0.77	
3.00	1.87	1.22	
4.00	3.12	1.75	
5.00	4.37	2.38	
6.00	5.62	3.01	
7.00	6.87	3.52	0.33
8.00	8.13	3.87	0.37
9.00	9.38	4.08	0.40
10.00	10.63	4.16	0.41
11.00	11.88	4.15	0.42
12.00	13.13	4.05	0.41
13.00	14.38	3.88	0.40
14.00	15.63	3.61	0.37
15.00	16.88	3.26	0.34
16.00	18.13	2.81	0.30
17.00	19.38	2.30	
18.00	20.63	1.80	
19.00	21.88	1.37	
20.00	23.13	0.98	
21.00	24.38	0.61	
22.00	25.16	0.43	
	DS Avg	3.67	3.75
	DS Max	4.16	

Pools

Arc 2

	Distance	Value	Mean
1.00	0.00	0.08	
2.00	0.95	0.13	
3.00	2.20	0.26	
4.00	3.45	0.46	
5.00	4.70	0.71	
6.00	5.95	0.94	0.07
7.00	7.20	1.11	0.09
8.00	8.45	1.24	0.11
9.00	9.70	1.31	0.12
10.00	10.95	1.31	0.12
11.00	12.20	1.25	0.12
12.00	13.45	1.16	0.11
13.00	14.70	1.06	0.10
14.00	15.95	0.91	0.09
15.00	17.20	0.70	0.07
16.00	18.45	0.47	0.05
17.00	19.70	0.26	
18.00	20.95	0.12	
19.00	22.20	0.05	
20.00	22.85	0.03	
	US Avg	1.01	1.05
	US Max	1.31	

Arc 4

	Distance	Value	Mean
1.00	0.00	0.16	
2.00	0.58	0.19	
3.00	1.83	0.33	
4.00	3.08	0.57	
5.00	4.33	0.90	
6.00	5.58	1.20	0.12
7.00	6.83	1.40	0.14
8.00	8.08	1.51	0.16
9.00	9.33	1.59	0.17
10.00	10.58	1.65	0.18
11.00	11.83	1.68	0.19
12.00	13.08	1.65	0.18
13.00	14.33	1.55	0.18
14.00	15.58	1.36	0.16
15.00	16.83	1.08	
16.00	18.08	0.78	
17.00	19.33	0.54	
18.00	20.58	0.36	
19.00	21.83	0.24	
20.00	23.08	0.13	
21.00	23.12	0.13	
	Mid Avg	1.45	1.48
	Mid Max	1.68	

Arc 6

	Distance	Value	Mean
1.00	0.00	0.08	
2.00	0.22	0.08	
3.00	1.47	0.18	
4.00	2.72	0.36	
5.00	3.97	0.61	
6.00	5.22	0.87	
7.00	6.47	1.08	0.10
8.00	7.72	1.21	0.11
9.00	8.97	1.27	0.12
10.00	10.22	1.29	0.13
11.00	11.47	1.29	0.13
12.00	12.72	1.27	0.13
13.00	13.97	1.18	0.12
14.00	15.22	1.02	0.11
15.00	16.47	0.80	0.09
16.00	17.72	0.55	0.07
17.00	18.97	0.33	
18.00	20.22	0.17	
19.00	21.47	0.07	
20.00	22.72	0.03	
21.00	23.97	0.01	
22.00	24.95	0.01	
	DS Avg	1.07	1.11127
	DS Max	1.29	

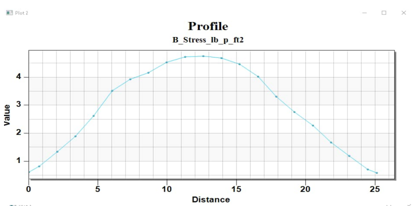
Riffle

Upstream Arc/Max Shear Stress/Most Steep Riffle

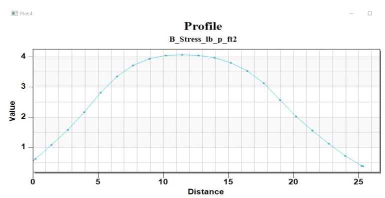
	Distance	Value	
1.00	8.37	2.83	
2.00	10.31	3.85	0.34
3.00	12.25	5.40	0.46
4.00	14.17	7.20	0.63
5.00	16.09	8.82	0.80
6.00	18.00	9.97	0.93
7.00	19.91	10.23	1.00
8.00	21.83	10.88	1.05
9.00	23.77	9.22	1.01
10.00	25.70	4.60	0.69
11.00	27.64	1.41	0.30
12.00	29.58	0.32	
13.00	31.52	0.35	
14.00	33.46	0.42	
15.00	35.40	0.46	
16.00	37.34	0.44	
17.00	39.28	0.42	
18.00	40.36	0.43	
	DS Avg	6.76	7.218928
	DS Max	10.88	

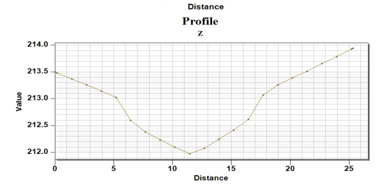
Riffles

Arc 1



Arc 3





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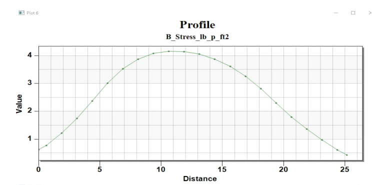
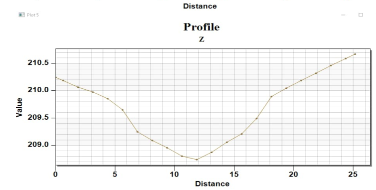
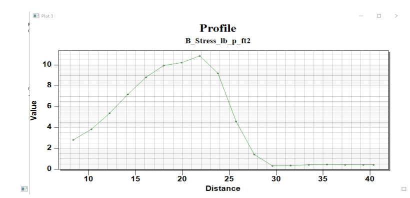
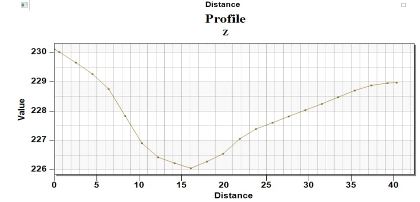




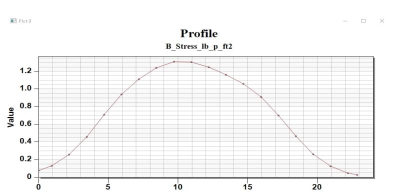
Upstream Arc/Max Shear Stress/Most Steep Riffle

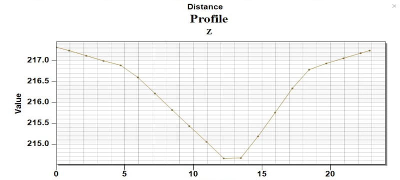




Pools

Arc 2





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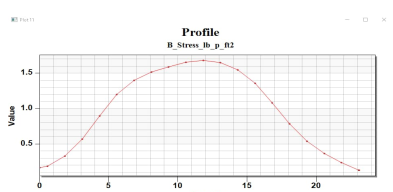
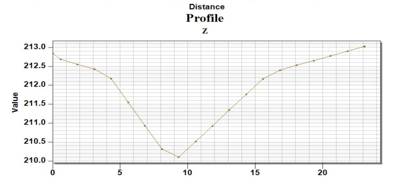
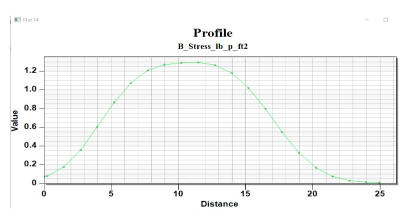
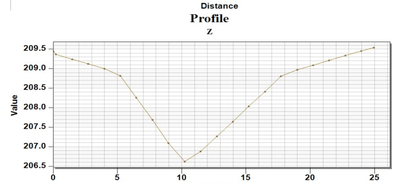
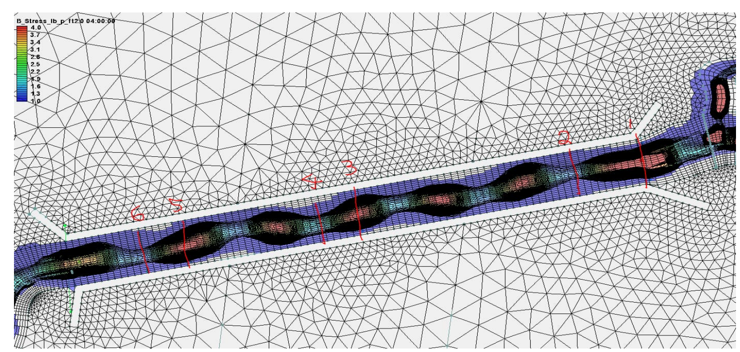




Arc 6







Juanita Shear Stress - 2yr

Pools

Arc 2

	Distance	Value	Mean
1	0.00		
2	0.95		
3	2.20	0.04	
4	3.45	0.02	
5	4.70	0.08	
6	5.95	0.24	0.01
7	7.20	0.40	0.03
8	8.45	0.53	0.04
9	9.70	0.60	0.05
10	10.95	0.61	0.05
11	12.20	0.55	0.05
12	13.45	0.48	0.05
13	14.70	0.39	0.04
14	15.95	0.28	0.03
15	17.20	0.18	0.02
16	18.45	0.10	0.01
17	19.70	0.04	
18	20.95	0.01	
19	22.20	0.01	
20	22.85		
	US Avg	0.37	0.40
	US Max	0.61	

Arc 4

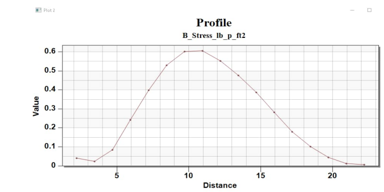
	Distance	Value	Mean
1.00	0.00		
2.00	0.58		
3.00	1.83	0.00	
4.00	3.08	0.05	
5.00	4.33	0.19	
6.00	5.58	0.39	0.03
7.00	6.83	0.57	0.05
8.00	8.08	0.68	0.07
9.00	9.33	0.75	0.08
10.00	10.58	0.78	0.08
11.00	11.83	0.77	0.09
12.00	13.08	0.68	0.08
13.00	14.33	0.54	0.07
14.00	15.58	0.34	0.05
15.00	16.83	0.14	
16.00	18.08	0.05	
17.00	19.33		
18.00	20.58		
19.00	21.83		
20.00	23.08		
21.00	23.12		
	Mid Avg	0.57	0.60
	Mid Max	0.78	

Arc 6

	Distance	Value	Mean
1.00	0.00		
2.00	0.22		
3.00	1.47	0.03	
4.00	2.72	0.02	
5.00	3.97	0.04	
6.00	5.22	0.18	
7.00	6.47	0.37	0.03
8.00	7.72	0.50	0.04
9.00	8.97	0.57	0.05
10.00	10.22	0.58	0.06
11.00	11.47	0.57	0.06
12.00	12.72	0.52	0.05
13.00	13.97	0.43	0.05
14.00	15.22	0.29	0.04
15.00	16.47	0.16	0.02
16.00	17.72	0.09	0.01
17.00	18.97	0.04	
18.00	20.22	0.01	
19.00	21.47	0.01	
20.00	22.72		
21.00	23.97		
22.00	24.95		
	DS Avg	0.39	0.41
	DS Max	0.58	

Pools

Arc 2



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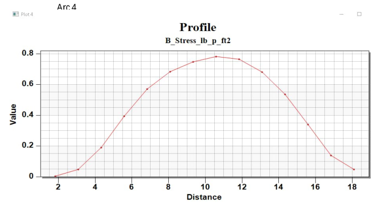
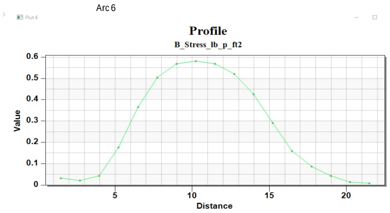
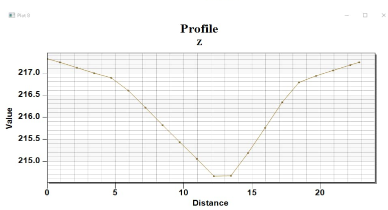
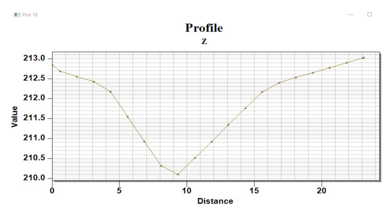
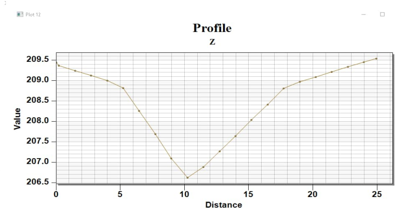


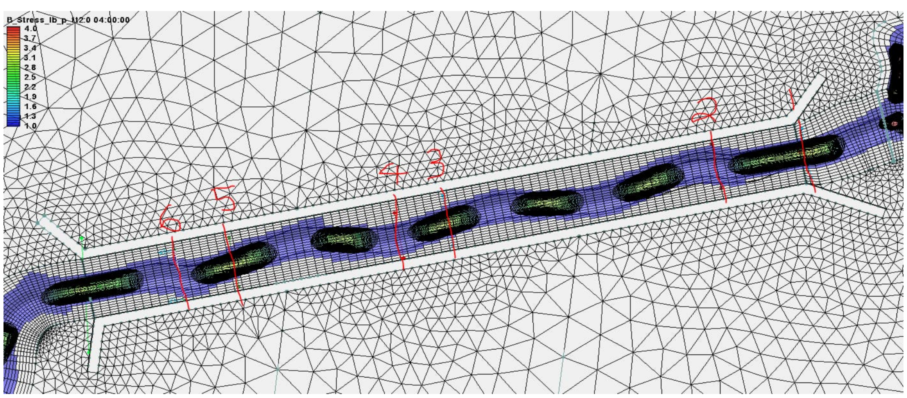
Arc 6











1. Modified Critical Shear Stress Approach for Sediment Sizing   Reference: fsm91_054564.pdf (usda.gov)

			Pebble Count	WSDOT PHD Design Gradation In Body Text	WSDOT Standard Spec Large (D84 = 2", D50 = 1")	WSDOT Standard Spec Small (D84 = 1", D50 = 0.5")	Not Mobile at 100yr average - Riffle[1]	Not Mobile at 100yr max - Riffle	Not Mobile at 100yr average - Pool[2]	Not Mobile at 100yr max - Pool	Not Mobile at 2yr average - Pool[3]	Upstream Arc/Max Shear Stress/Most Steep Riffle Average - 100yr[4]
Critical shear stress	τ_{c84}	(lb/sq-ft)	0.43	2.50	0.48	0.22	4.23	4.81	1.52	1.69	0.61	7.24
Shield's parameter	τ^*_{D50}		0.04	0.05	0.05	0.04	0.05	0.05	0.05	0.05	0.05	0.05
	D_{16}	(ft)	0.02	0.14	0.02	0.01	0.18	0.21	0.07	0.07	0.03	0.36
	D_{50}	(ft)	0.08	0.32	0.08	0.04	0.58	0.66	0.22	0.23	0.10	1.14
	D_{84}	(ft)	0.18	1.100	0.17	0.08	1.45	1.65	0.54	0.58	0.240	2.850
	D_{100}	(ft)	0.37	2.20	0.34	0.16	2.90	3.30	1.35	1.16	0.60	7.13
	D_{16}	(in)	0.28	1.65	0.26	0.12	2.18	2.48	0.81	0.87	0.36	4.28
	D_{50}	(in)	1.01	3.89	0.96	0.48	6.96	7.92	2.59	2.78	1.15	13.68
	D_{84}	(in)	2.21	13.20	2.04	0.96	17.40	19.80	6.5	6.96	2.9	34.2
	D_{100}	(in)	4.42	26.40	4.08	1.92	34.80	39.60	16.2	13.92	7.2	85.5
100-year max shear stress - riffle	4.76	(lb/sq-ft)	Motion	Motion	Motion	Motion	Motion	No Motion	Motion	Motion	Motion	No Motion
100-year average shear stress - riffle	4.17	(lb/sq-ft)	Motion	Motion	Motion	Motion	No Motion	No Motion	Motion	Motion	Motion	No Motion
100-year max shear stress - pool	1.68	(lb/sq-ft)	Motion	Motion	Motion	Motion	Motion	No Motion	Motion	No Motion	Motion	No Motion
100-year average shear stress - pool	1.48	(lb/sq-ft)	Motion	Motion	Motion	Motion	Motion	No Motion	No Motion	No Motion	Motion	No Motion
2-year average shear stress - pool	0.60	(lb/sq-ft)	Motion	No Motion	Motion	Motion	No Motion	No Motion	No Motion	No Motion	No Motion	No Motion
Upstream Arc/Max Shear Stress/Most Steep Riffle	7.22	(lb/sq-ft)	Motion	Motion	Motion	Motion	Motion	Motion	Motion	Motion	Motion	No Motion

[1]A ratio of D84/D100 = 0.5 was used instead of the typical 0.4, so that the D100 could be kept to less than one quarter of the bankfull width (BFW =12'; D100 = 34.8")

[2]While the model indicates that pool sediment size would need to have a D50=2.59" for it not to be mobile, because pools are intended to scour and fill, with most of their sediment moving through the system, the sediment gradation for pools was decreased to the standard WSDOT streambed sediment.

[3]WSDOT streambed sediment (D50=1") will be mobile in pools at 2-yr flow, since D50 needs to be 1.15" for no motion.

[4]Evidence that upstream end of the Juanita grading extents are too steep for a riffle-pool system, because D50 would need to be 13.7" to be stable at the 100-yr; thus proposing step-pool system for FHD.

Find critical shear stress for D 84

where:

τ_{ci} is the critical shear stress at which the sediment particle of interest begins to move (lb/ft² or N/m²).

τ_{D50} is the dimensionless Shields parameter for D_{50} particle size (this value can either be obtained from table E.1, or the value 0.045 can be used for a poorly sorted channel bed).

D_{50} is the diameter (ft or m) of the median or 50th percentile particle size of the channel bed .

D_i is the diameter (ft or m) of the particle size of interest. For stream simulation the particle size of interest is typically D_{84} and/or D_{95}.

Assuming $\gamma_s = 165$ lb/ft³ and $\gamma = 62.4$ lb/ft³, equation 5 can be simplified to:

Equation E.6

$$\tau_{ci} = 102.6\, \tau^*_{D50}\, D_i^{0.3}\, D_{50}^{0.7}$$

Compare critical shear stress to average shear stress.
If critical shear stress is greater than average shear stress, then the particle is stable.

Table E.1—Shield's parameter for different particle sizes. Modified from Julien 1995

Particle size classification	Particle size, D (mm)	Angle of repose, ϕ (degrees)	Shield's parameter, τ*	Critical shear stress, τ_c (lb/ft²)
very large boulders	> 2,048	42	0.054	37.37
large boulders	1,024-2,048	42	0.054	18.68
medium boulders	512-1,024	42	0.054	9.34
small boulders	256-512	42	0.054	4.67
large cobbles	128-256	42	0.054	2.34
small cobbles	64-128	41	0.052	1.13
very coarse gravels	32-64	40	0.050	0.54
coarse gravels	16-32	38	0.047	0.25
medium gravels	8-16	36	0.044	0.12
fine gravels	4-8	35	0.042	0.057
very fine gravels	2-4	33	0.039	0.026

The equation used to determine the Shields parameter for gravels, cobbles, and boulders is $\tau^* = 0.06 \tan\phi$.

The Shield's parameter and critical shear stress values are for the smallest number in the particle-size interval.

mm		ft		τ*
>	<	>	<	
2048		6.719		0.054
1024	2048	3.360	6.719	0.054
512	1024	1.680	3.360	0.054
256	512	0.840	1.680	0.054
128	256	0.420	0.840	0.054
64	128	0.210	0.420	0.052
32	64	0.105	0.210	0.05
16	32	0.052	0.105	0.047
8	16	0.026	0.052	0.044
4	8	0.013	0.026	0.042
2	4	0.007	0.013	0.039

Table 13: Comparison of Observed and Proposed Streambed Material

	Existing	Riffle-Pool System
	Average Diameter (in)	Proposed Diameter (in)
D_{16}	0.38	0.78
D_{50}	1.01	3.89
D_{84}	2.21	13.20
D_{100}	5.69	18.00

$D_{84}/D_{100} = 0.4$ **Equation 3.6**

$D_{84}/D_{50} = 2.5$ **Equation 3.7**

$D_{84}/D_{16} = 8.0$ **Equation 3.8**

7-2.5.3.3.1 Threshold-of-Motion Analysis

A threshold-of-motion (incipient motion) analysis is used to determine if a sediment particle of interest will mobilize under specific hydraulic conditions. For example, this analysis could determine if a particle of interest is mobilized during a specific flow. Alternatively, it could be used to determine what hydraulic forces would be required to mobilize a particle of interest. Common methods used include the unit discharge method (Bathurst 1987), which identifies a stable D_{84} particle size given a flow rate of interest. This method is typically used for channels with gradients over 4 percent. For shallower slopes, the modified Shields approach (USDA 2008) is used to determine sediment mobility. WSDOT is currently working to incorporate another method of assessing the threshold of sediment transport and scour (the erodibility index) based on the work presented in HEC-18 and Annandale (2006). This work will be included in the next *Hydraulics Manual* update.

9-03.11(1) Streambed Sediment

Streambed sediment shall meet the following requirements for grading. If the Contractor proposes an alternate gradation, the Contractor shall submit a Type 2 Working Drawing consisting of 0.45 power maximum density curve of the proposed gradation. The alternate gradation shall closely follow the maximum density line and have Nominal Aggregate Size of no less than 1½ inches or no greater than 3 inches. The exact point of acceptance will be determined by the Engineer.

Sieve Size	Percent Passing
2½"	99-100
2"	85-100
1"	50-82
1/2"	28-68
No. 40	10-20
No. 200	5.0-10.0

All percentages are by weight. The portion of sediment retained on ½" sieve shall not contain more than 0.2 percent wood waste.

2. USACE Riprap Design Approach for Sediment Sizing

EM 1110-2-1601

Average Slope

D30	0.33	ft	
	S		0.019
	q		8.5
	g		32.2
D84	0.49	ft	

$$D_{30} = 1.95S^{0.555}(1.25q)^{2/3}/g^{1/3}$$ **Equation 6.4**

Where:

D_{30}	=	the dimension of the intermediate axis of the 30th percentile particle
S	=	the bed slope
q	=	the unit discharge
g	=	acceleration due to gravity.

Riffle Slope

D30	0.59	ft	
	S		0.061
	q		7.8
	g		32.2
D84	0.89	ft	
D84	10.66	in	
D50	4.27		

2013 WCDG, pg 126
for slopes 2 -20%, and unit discharge is low

D84 = 1.5*D30

3. Bathurst Equation Approach for Sediment Sizing

References:

Bathurst, J.C. (1987) Critical Conditions for Movement in Steep Boulder-Bed Streams. Int. Assoc. of Hydraulical Sciences Pub. Vol. 165.

$$D_{84} = 3.54 * S^{0.747} * \frac{(1.25 * q_c)^{\frac{2}{3}}}{g^{\frac{1}{3}}}$$

$$cfs = \frac{ft^3}{s}$$

Cross Section Name/Station:

Flow Event:		100	yr
Energy Slope (S) - ft/ft:	S =	0.061	ft/ft
100-yr Flow in Main Channel (Q):	Q =	93.79	cfs
Stream Width (W):	W =	12.1	ft
Specific Discharge (q_c) - (cfs/ft):	q_c =	7.8	ft^2/s

$$q_c = \frac{Q}{W}$$

$$D_{84} = 3.54 * S^{0.747} * \frac{(1.25 * q_c)^{\frac{2}{3}}}{g^{\frac{1}{3}}}$$

D_{84} = 0.63 ft
7.52 in

$$D_{16} = \frac{D_{84}}{8}$$

D_{16} = 0.08 ft
0.94 in

$$D_{50} = \frac{D_{84}}{2.5}$$

D_{50} = 0.25 ft
3.01 in

$$D_{100} = \frac{D_{84}}{0.4}$$

D_{100} = 1.57 ft
18.80 in

Table 1: High Woodlands HSPF Peak Flow Rate (in cfs) for Various Storm Frequencies Based on 15- minute time step

	Project Scenario	2-year	10-year	25-year	100-yr
At I-405	Pre	25	32	35	41
	Post	40	65	81	111
Above Norway Hill Trib	Pre	30	39	44	51
	Post	45	74	94	129
At 141st	Pre	65	101	124	166

		Flow			Width			Area		
Reach	Station	Left	Chan	Right	Left	Chan	Right	Left	Chan	Right
reach_2	690.658	35.26	69.55	34.08	9.22	5.79	7.05	9.9	11.93	6.38
reach_2	1231.12	4.19	123	0	5.42	17.85	0	2.91	37.79	0
reach_2	1387.56	9.83	98.53	10.06	4.35	12.12	10.8	4.94	32.45	14.21
reach_2	1611.65	11.06	93.79	4.15	9.51	11.94	6.06	4.95	25.4	3.36
reach_2	1795.37	10.08	91.1	9.73	4.29	12.12	11.33	3.79	28.83	6.58
reach_2	2023.38	4.98	101.04	6.49	2.34	13.27	12.99	2.09	25.57	3.74
reach_2	2528.78	2.3	94.07	14.79	4.08	12.38	3.54	2.35	28.08	5.41

Table 2: Bankfull width measurements

BFW #	Width (ft)	Included in Design Average	Location Measured	Concurrence Notes
BFW 1	10.5	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 2	14.5	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 3	13.3	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 4	11.5	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 5	10.0	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 6	9.5	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 7	11.5	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 8	11.0	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 9	17.5	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
BFW 10	11.5	Yes	Reference Reach	Stakeholder concurred on 12/03/2019
Design Average	12.08			Stakeholder concurred in Fall 2019

Appendix D: Stream Plan Sheets, Profile, Details

Stream Plan Sheets, Profile, Details

I-405 MP 21.79 Juanita Creek (WDFW ID): 998602 Preliminary Hydraulic Design Report

To be submitted as a separate attachment.

Appendix E: Manning's Calculations

Project	I-405 Brickyard	Date	4/25/2024
SR Route		Designer	YR
Stream Crossing	Juanita	Checked By	AK

Source: WSDOT Hydraulics Manual Chapter 10 Open-Channel Flow

Chapter 4 ***Open-Channel Flow***

Table 4-2 Manning's Roughness Coefficients for Stream Channels

Stream Channels	Manning's n
Minor streams (surface width at flood stage less than 100 feet):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks	0.035–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by 0.01–0.02	
2. Irregular sections, with pools, slight channel meander; increase values given in 1a–e above 0.01–0.02	
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07
Floodplains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030–0.035
b. High grass	0.035–0.05
2. Cultivated areas:	
a. No crop	0.03–0.04
b. Mature row crops	0.035–0.045
c. Mature field crops	0.04–0.05
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees:	
a. Winter	0.05–0.06
b. Summer	0.06–0.08
5. Medium to dense brush:	
a. Winter	0.07–0.11
b. Summer	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. Cleared land with tree stumps, 100–150 per acre:	
a. No sprouts	0.04–0.05
b. With heavy growth of sprouts	0.06–0.08
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10–0.12
b. Flood depth reaches branches	0.12–0.16

Major streams (surface width at flood stage more than 100 feet): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited if possible. The value of n for larger streams of most regular sections, with no boulders or brush, may be in the range of 0.028–0.033.

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May 2023

0.065 Used to compare to composite value calculated below

Arcement and Schneider - *Guide For Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains*

Source: https://pubs.usgs.gov/wsp/2339/report.pdf

Existing Channel

Cowan's(1956) Procedure

$$n=(n_b+n_1+n_2+n_3+n_4)m \qquad (3)$$

where
n_b = a base value of n for a straight, uniform, smooth channel in natural materials,
n_1 = a correction factor for the effect of surface irregularities,
n_2 = a value for variations in shape and size of the channel cross section,
n_3 = a value for obstructions,
n_4 = a value for vegetation and flow conditions, and
m = a correction factor for meandering of the channel.

Table 1. Base values of Manning's n
[Modified from Aldridge and Garrett, 1973, table 1; —, no data]

Bed material	Median size of bed material (in millimeters)	Base n value: Straight uniform channel[1]	Base n value: Smooth channel[2]
Sand channels			
Sand[3]	0.2	0.012	—
	.3	.017	—
	.4	.020	—
	.5	.022	—
	.6	.023	—
	.8	.025	—
	1.0	.026	—
Stable channels and flood plains			
Concrete	—	0.012–0.018	0.011
Rock cut	—	—	.025
Firm soil	—	0.025–0.032	.020
Coarse sand	1–2	0.026–0.035	—
Fine gravel	—	—	.024
Gravel	2–64	0.028–0.035	—
Coarse gravel	—	—	.026
Cobble	64–256	0.030–0.050	—
Boulder	>256	0.040–0.070	—

[1] Benson and Dalrymple (1967).
[2] For indicated material; Chow (1959).
[3] Only for upper regime flow where grain roughness is predominant.

	Existing Channel	Proposed Channel Outside Culvert	Proposed Channel Inside Culvert	Side Channel	Step Pool	
	0.031	0.034	0.05	0.031		
	0.008	0.008	0.008	0.008		
	0.003	0.003	0.003	0.003		
	0.005	0.015	0.005	0.02		in PHD: note that wood placement isn't final, so level of obstruction still being determined
	0.018	0.018	0.002	0.018		
	0.065	0.078	0.068	0.08	0.29	FINAL

Table 2. Adjustment values for factors that affect the roughness of a channel
[Modified from Aldridge and Garrett, 1973, table 2]

Channel conditions		n value adjustment[1]	Example
Degree of irregularity (n_1)	Smooth	0.000	Compares to the smoothest channel attainable in a given bed material.
	Minor	0.001–0.005	Compares to carefully dredged channels in good condition but having slightly eroded or scoured side slopes.
	Moderate	0.006–0.010	Compares to dredged channels having moderate to considerable bed roughness and moderately sloughed or eroded side slopes.
	Severe	0.011–0.020	Badly sloughed or scalloped banks of natural streams; badly eroded or sloughed sides of canals or drainage channels; unshaped, jagged, and irregular surfaces of channels in rock.
Variation in channel cross section (n_2)	Gradual	0.000	Size and shape of channel cross sections change gradually.
	Alternating occasionally	0.001–0.005	Large and small cross sections alternate occasionally, or the main flow occasionally shifts from side to side owing to changes in cross-sectional shape.
	Alternating frequently	0.010–0.015	Large and small cross sections alternate frequently, or the main flow frequently shifts from side to side owing to changes in cross-sectional shape.
Effect of obstruction (n_3)	Negligible	0.000–0.004	A few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
	Minor	0.005–0.015	Obstructions occupy less than 15 percent of the cross-sectional area, and the spacing between obstructions is such that the sphere of influence around one obstruction does not extend to the sphere of influence around another obstruction. Smaller adjustments are used for curved smooth-surfaced objects than are used for sharp-edged angular objects.
	Appreciable	0.020–0.030	Obstructions occupy from 15 to 50 percent of the cross-sectional area, or the space between obstructions is small enough to cause the effects of several obstructions to be additive, thereby blocking an equivalent part of a cross section.
	Severe	0.040–0.050	Obstructions occupy more than 50 percent of the cross-sectional area, or the space between obstructions is small enough to cause turbulence across most of the cross section.
Amount of vegetation (n_4)	Small	0.002–0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrowweed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
	Medium	0.010–0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemmy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees in the dormant season, growing along the banks, and no significant vegetation is evident along the channel bottoms where the hydraulic radius exceeds 2 ft.
	Large	0.025–0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8- to 10-year-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft; bushy willows about 1 year old intergrown with some weeds along side slopes (all vegetation in full foliage), and no significant vegetation exists along channel bottoms where the hydraulic radius is greater than 2 ft.
	Very large	0.050–0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; bushy willow trees about 1 year old intergrown with weeds along side slopes (all vegetation in full foliage), or dense cattails growing along channel bottom; trees intergrown with weeds and brush (all vegetation in full foliage).

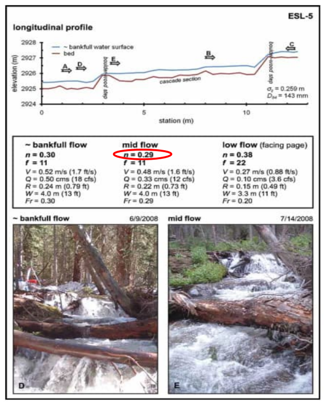
Degree of meandering[2] (m)	Appreciable	1.15	Ratio of the channel length to valley length is 1.2 to 1.5.
	Severe	1.30	Ratio of the channel length to valley length is greater than 1.5.

[1] Adjustments for degree of irregularity, variations in cross section, effect of obstructions, and vegetation are added to the base n value (table 1) before multiplying by the adjustment for meander.
[2] Adjustment values apply to flow confined in the channel and do not apply where downvalley flow crosses meanders.

Channel n Values 7

Calculation for Step Pool Manning's n:

	Manning's for step pool:	0.29
	slope of step pool	0.15
	Station	Elevation (ft)
1	0	222.896
2	0.137	222.921
3	0.176	222.893
4	7.873	221.729
5	8.146	221.694
6	8.146	221.694
7	8.188	221.694
8	38.369	217.127
9	38.433	217.091
10	38.645	216.964
11	38.818	216.868
12	38.818	216.868
13	39.066	216.918
14	39.086	216.923
15	39.33	216.972
16	39.43	216.97



Citation:

Yochum, Steven E.; Comiti, Francesco; Wohl, Ellen; David, Gabrielle C. L.; Mao, Luca. 2014. Photographic guidance for selecting flow resistance coefficients in high-gradient channels. Gen. Tech. Rep. RMRS-GTR-323. Fort Collins, CO: U.S. Department of Agriculture, Forest Service, Rocky Mountain Research Station. 91 p.

https://www.fs.usda.gov/rm/pubs/rmrs_gtr323.pdf

Appendix F: Large Woody Material Calculations

WSDOT Large Woody Material for stream restoration metrics calculator

State Route# & MP	*I 405*		Key piece volume	*1.310*	*yd^3*
Stream name	*Juanita*		Key piece/ft	*0.0335*	*per ft stream*
length of regrade[a]	*696.85*	ft	Total wood vol./ft	*0.3948*	*yd^3/ft stream*
Bankfull width	*12*	ft	Total LWM[c] pieces/ft stream	*0.1159*	*per ft stream*
Habitat zone[b]	*Western WA*				

Taper coeff.	-0.01554
LF_{rw}	1.5
H_{dbh}	4.5

yes

no

Log type	*Diameter at midpoint (ft)*	*Length(ft)[d]*	*Volume (yd^3/log)[d]*	*Rootwad?*	*Qualifies as key piece?*	*No. LWM pieces*	*Total wood volume (yd^3)*
A	1.50	*30*	1.96	yes	yes	33	64.80
B	1.50	*40*	2.62	yes	yes	6	15.71
C	0.88	*15*	0.34	no	no	48	16.22
D			0.00				0.00
E			0.00				0.00
F			0.00				0.00
G			0.00				0.00
H			0.00				0.00
I			0.00				0.00
J			0.00				0.00
K			0.00				0.00
L			0.00				0.00
M			0.00				0.00
N			0.00				0.00
O			0.00				0.00
P			0.00				0.00

DBH based on mid point diameter (ft)	$D_{root\ collar\ (ft)}$	L/2-Lrw (ft)
1.63	1.70	12.75
1.71	1.78	17.75
1.00	0.98	6.18
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0
	0.00	0

Log volume for stability calcs (yd^3, per log)	
rootwad	bole
0.51	1.82
0.51	2.47
0.00	0.34
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00
0.00	0.00

	No. of key pieces	Total No. of LWM pieces	Total LWM volume (yd^3)
Design	39	87	96.7
Targets	23	81	275.1
	surplus	surplus	deficit

[a] includes length through crossing, regardless of structure type

[b] choose one of the following Forest Regions in the drop-down menu (if in doubt ask HQ Biology). See also the Forest Region tab for additional information

Western Washington lowl	(generally <4,200 ft. in elevation west of the Cascade Crest)
Alpine	(generally > 4,200 ft. in elevation and down to ~3,700 ft. in elevation east of the Cascade crest)
Douglas fir-Ponderosa pin	(mainly east slope Cascades below 3,700 ft. elevation)

[c]LWM (Large Woody Material), also known as LWD (Large Woody Debris) is defined as a piece of wood at least 10 cm (4") diam. X 2 m (6ft) long (Fox 2001).

[d]includes rootwad if present

Key piece volume

BFW class (ft)	volume (yd3)
0-16	1.31
17-33	3.28
34-49	7.86
50-66	11.79
67-98	12.77
99-164	13.76
165-328	14.08

adapted from Fox and Bolton (2007), Table 5

Key Piece density lookup table

Habitat zone	BFW class (feet)	75th percentile (yd3/ft stream)
Western WA	0-33	0.0335
	34-328	0.0122
Alpine	0-49	0.0122
	50-164	0.0030
Douglas Fir/Pond. Pine (much of eastern WA)	0-98	0.0061

adapted from Fox and Bolton (2007), Table 4

Total Wood Volume lookup table

Habitat zone	BFW class (feet)	75th percentile (yd3/ft stream)
Western WA	0-98	0.3948
	99-328	1.2641
Alpine	0-10	0.0399
	11-164	0.1196
Douglas Fir/Pond. Pine	0-98	0.0598

adapted from Fox and Bolton (2007), Table 4

Number of LWM pieces lookup table

Habitat zone	BFW class (feet)	75th percentile (per/ft stream)
Western WA	0-20	0.1159
	21-98	0.1921
	99-328	0.6341
Alpine	0-10	0.0854
	11-98	0.1707
	99-164	0.1921
Douglas Fir/Pond.	0-20	0.0884
	21-98	0.1067

adapted from Fox and Bolton (2007), Table 4

FROM WSDOT PHD:

Region:	Western Washington	
Stream Name:	**Juanita Creek**	
Coordinated Bankfull Width:	12	feet
Stream Length:	692	feet

Per Foot of Stream Length (Fox and Bolton)			
	75th percentile	median	25th percentile
Number of Pieces:	**0.12**	0.09	0.08
Number of Key Pieces:	**0.03**	0.02	0.01
Volume of Wood (yd^3):	**0.40**	0.21	0.12
Volume of Key Pieces (yd^3):		1.31	

For Proposed Design Stream Length (Fox and Bolton)			
	75th percentile	median	25th percentile
Number of Pieces:	**81.0**	62.0	55.0
Number of Key Pieces:	**24.0**	13.0	9.0
Volume of Wood (yd^3):	**277.0**	146.0	84.0

Proposed Design Metrics			
	75th percentile	median	25th percentile
Number of Pieces:	--	71.0	--
Number of Key Pieces:	**29.0**	--	--
Volume of Wood (yd^3):	--	111.7	--

Tables from Fox and Bolton, 2007 - LWD Density in Stream Channels

TABLE 4.—Distributions of large woody debris (number of pieces, volume [m^3], and number of key pieces, all per 100 m of channel) by region and bank-full width (BFW) class. Large wood debris is defined as a pieces exceeding 10 cm in diameter and 2 m in length. Data are portrayed visually in Figure 6.

Region	BFW class	75th percentile	Median	25th percentile
	Number of pieces			
Western Washington	0–6 m	>38	29	<26
	>6–30 m	>63	52	<29
	>30–100 m	>208	106	<57
Alpine	>0–3 m	>28	22	<15
	>3–30 m	>56	35	<25
	>30–50 m	>63	34	<22
DF–PP forest zone	0–6 m	>29	15	<5
	>6–30 m	>35	17	<5
	Volume			
Western Washington	0–30 m	>99	51	<28
	>30–100 m	>317	93	<44
Alpine	>0–3 m	>10	8	<3
	>3–50 m	>30	18	<11
DF–PP forest zone	0–30 m	>15	7	<2
	Number of key pieces			
Western Washington	0–10 m	>11	6	<4
	>10–100 m	>4	1.3	<1
Alpine	>0–15 m	>4	2	<0.5
	>15–50 m	>1	0.3	<0.5
DF–PP forest zone	0–30 m	>2	0.4	<0.5

TABLE 5.—Minimum volume required for key pieces of large woody debris, by bank-full width (BFW) class.

BFW class	Minimum volume (m^3)
0–5 m	1.00[a]
5–10 m	2.50[a]
10–15 m	6.00[a]
15–20 m	9.00[a]
20–30 m	9.75
30–50 m	10.50[b]
50–100 m	10.75[b]

[a] Current WFPB (1997) definition.
[b] Piece must have an attached rootwad.

grouping them into the state regions based on the descriptive analysis. Through the descriptive analysis, the forest zones grouping did not substantially increase the variability; thus, we believe little was lost while gaining utility in simplification. Therefore, we chose state regions as the best single regional indicator for predicting instream wood loads in relation to various forms of climate-induced disturbance. Tree age, as influenced by natural fire history, increases with wetter climates. Because the adjacent riparian trees influence instream wood loads, the characteristics of riparian trees, as influenced by fire recurrence, vary by forest zones.

Log Count and Volumes

Log Type	*DBH*	*Length*	*Stem Volume*	*Out of 100-yrWSE?*	*Key Piece?*	*Log Quantity*	*Total Rootwad Volume*	*Total Wood Volume*
Unit of Measure	**ft**	**ft**	**yd3**				**yd3**	**yd3**
A	1.5	30	1.96	No	yes	29	14.80	71.75
B	1.5	40	2.62	Yes	no	6	3.06	18.77
C	1	15	0.44	No	no	36	5.45	21.15
D			0.00		no		0.00	0.00
E			0.00		no		0.00	0.00
F			0.00		no		0.00	0.00
G			0.00		no		0.00	0.00
H			0.00		no		0.00	0.00
I			0.00		no		0.00	0.00
J			0.00		no		0.00	0.00
Design Totals					29	71		111.7

Appendix G: Future Projections for Climate-Adapted Culvert Design

Future Projections for Climate-Adapted Culvert Design

Project Name:

Stream Name:

Street Name:

Culvert coordinates: 47.7345, -122.1829

Grid ID 47.71875_-122.15625

Ecoregion Pacific Maritime Mountains

Projected mean percent change in bankfull flow:

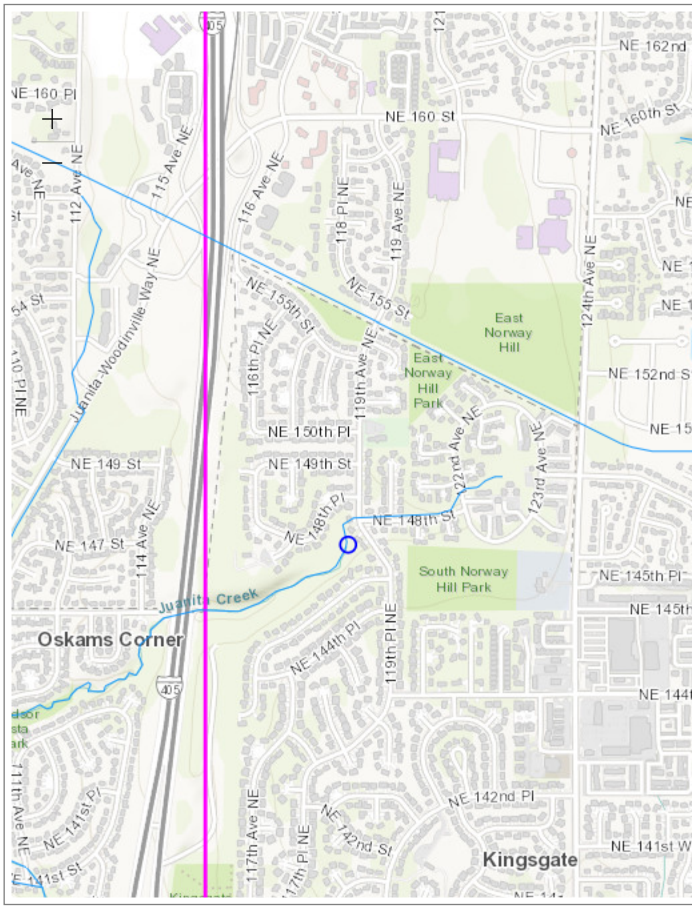
2040s: 10.5% **2080s:** 13.6%

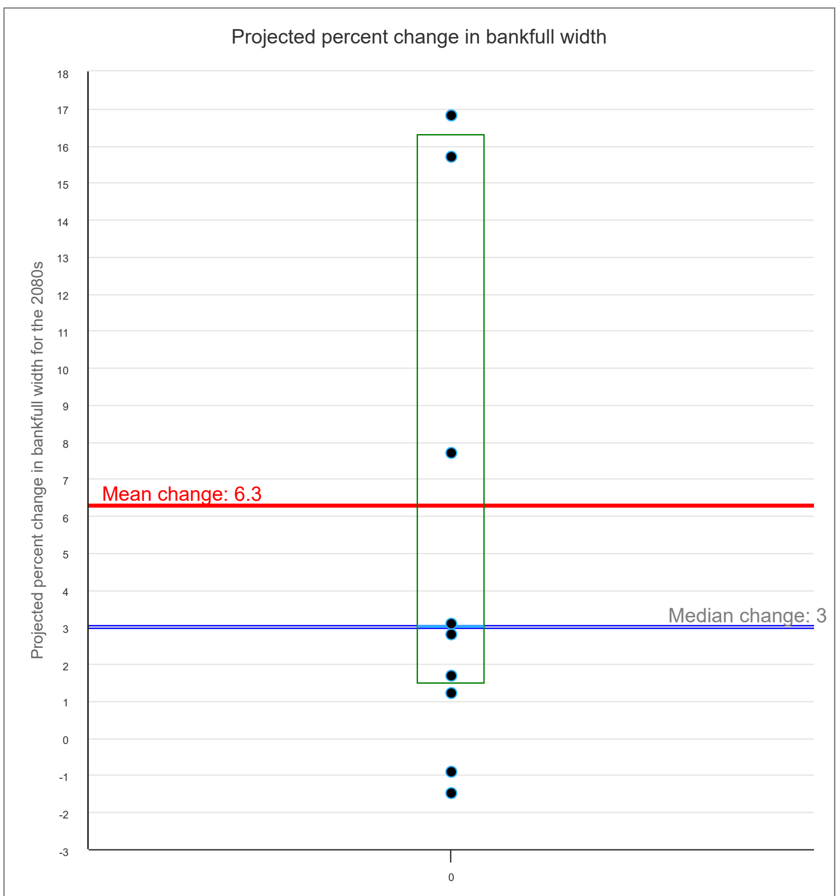
Projected mean percent change in bankfull width:

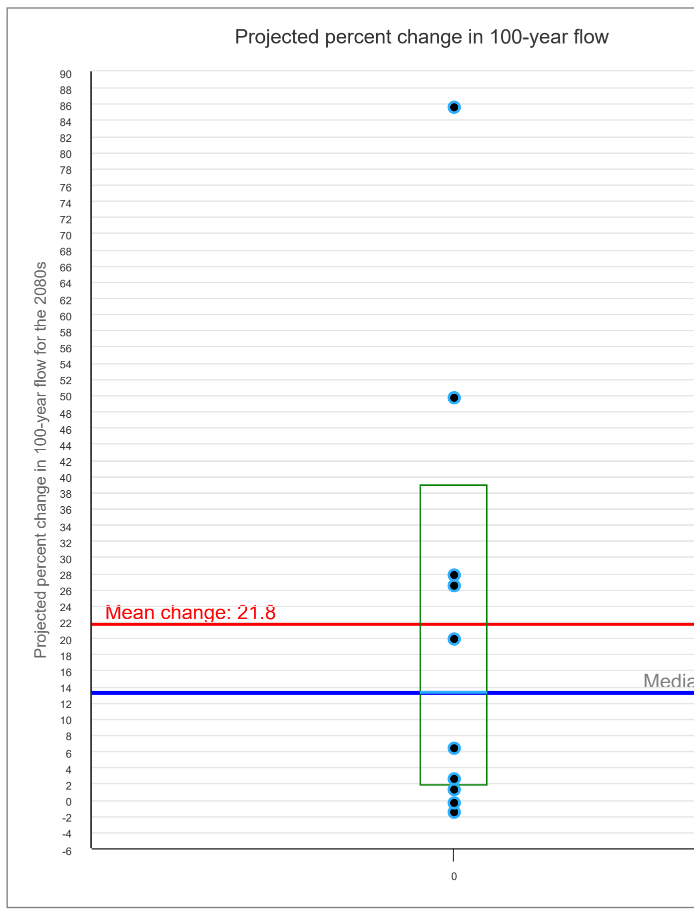
2040s: 5% **2080s:** 6.3%

Projected mean percent change in 100-year flood:

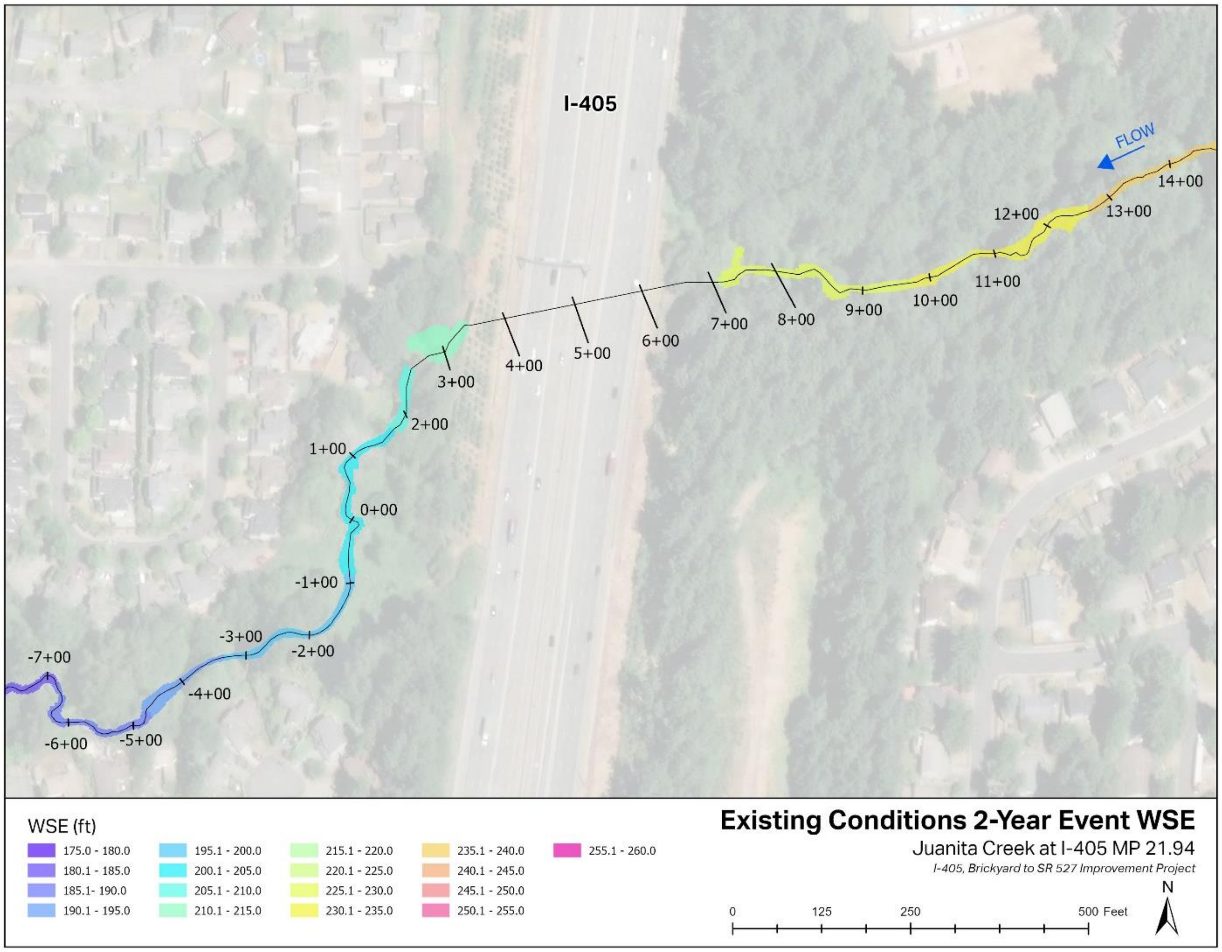
2040s: 11.3% **2080s:** 21.8%





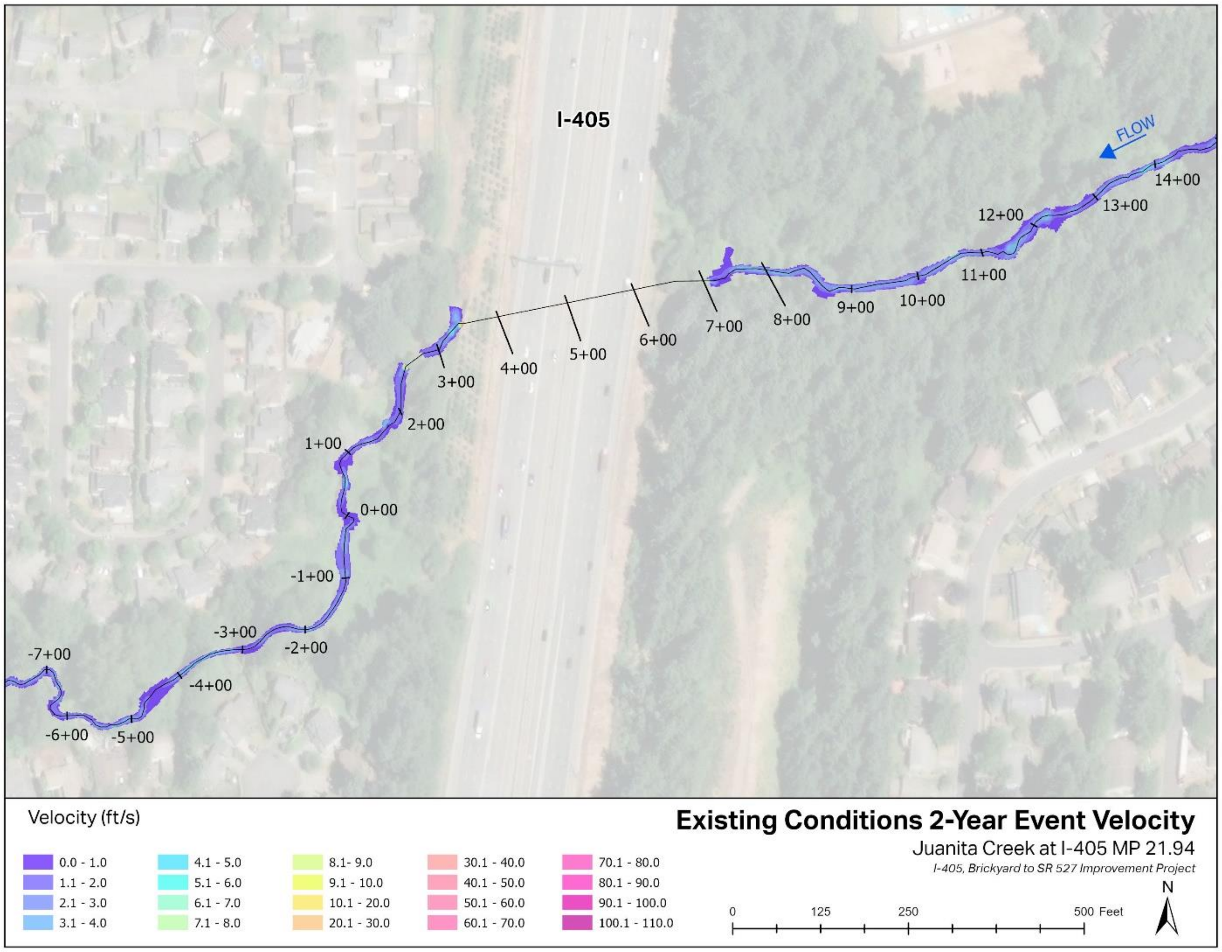


Appendix H: SRH-2D Model Results



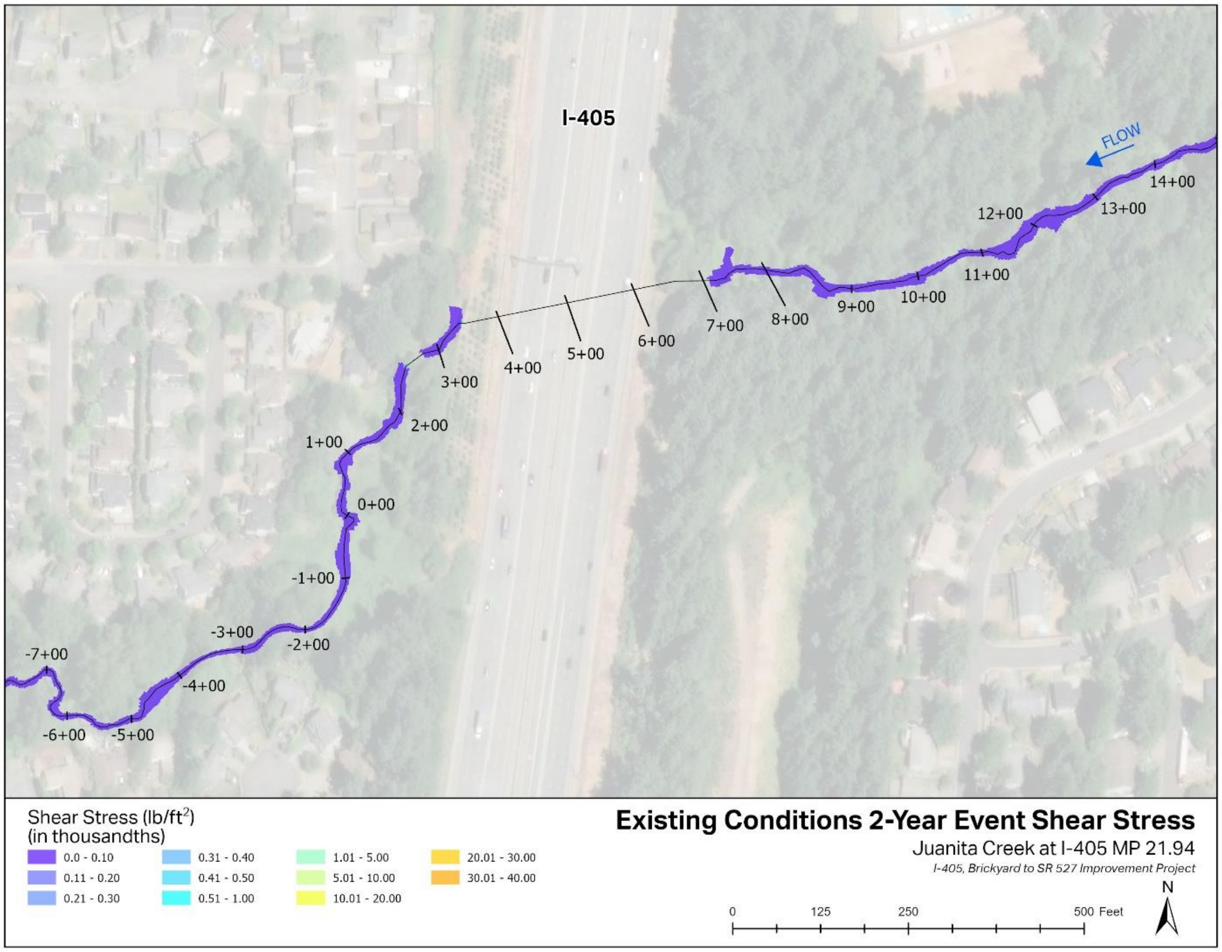
<u>Juanita Creek</u>

<u>Existing Condition – 2 Year Event WSE</u>



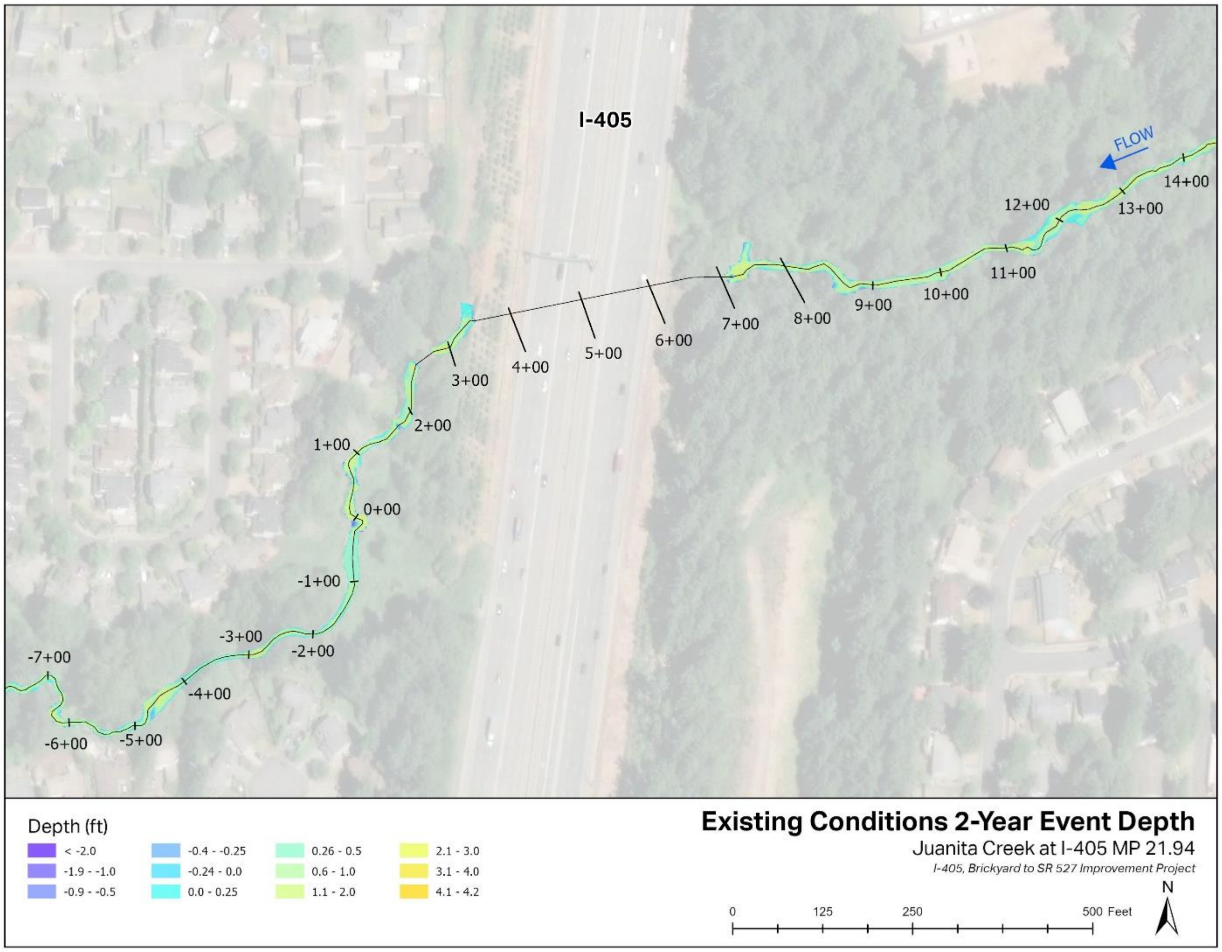
Juanita Creek

Existing Condition – 2 Year Event Velocity



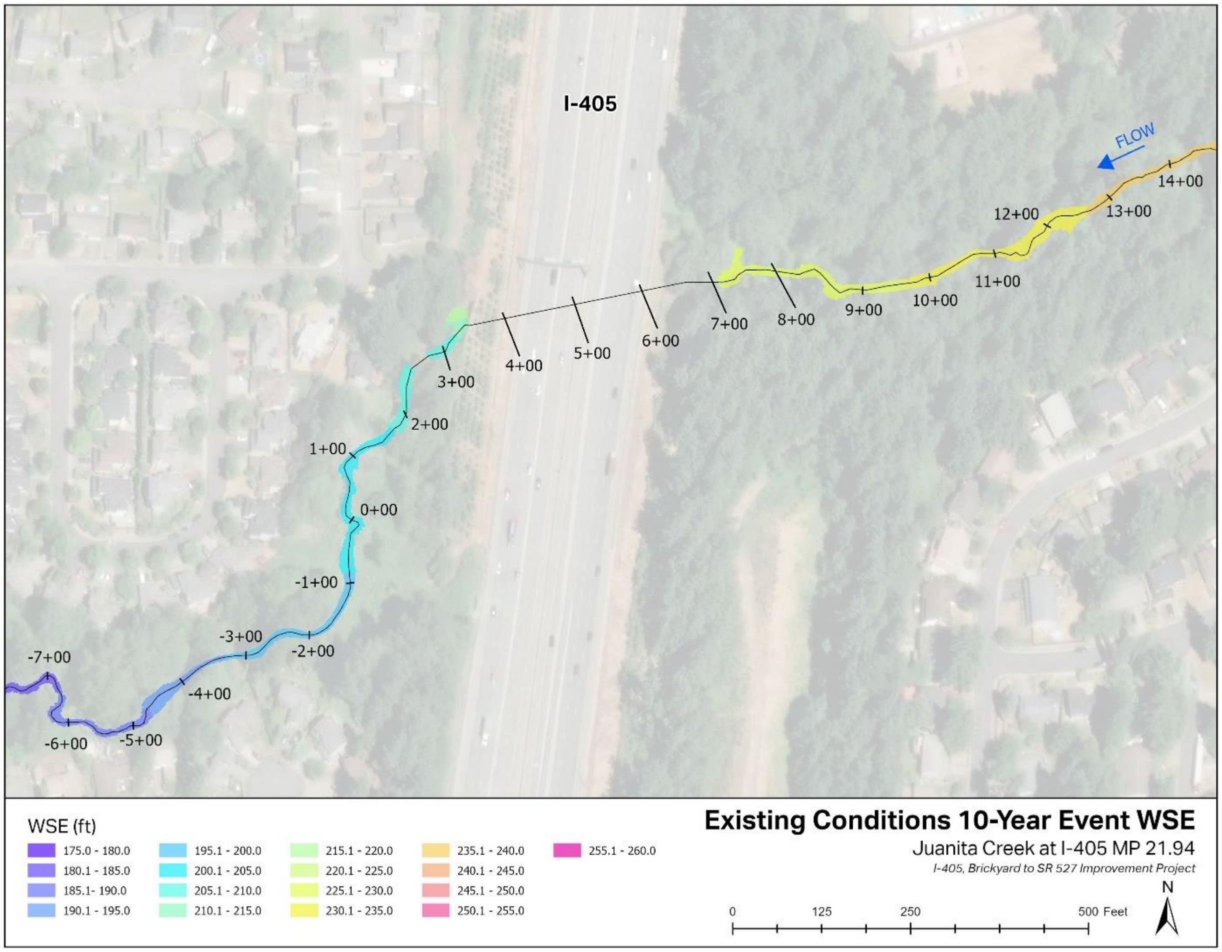
Juanita Creek

Existing Condition – 2 Year Event Shear Stress



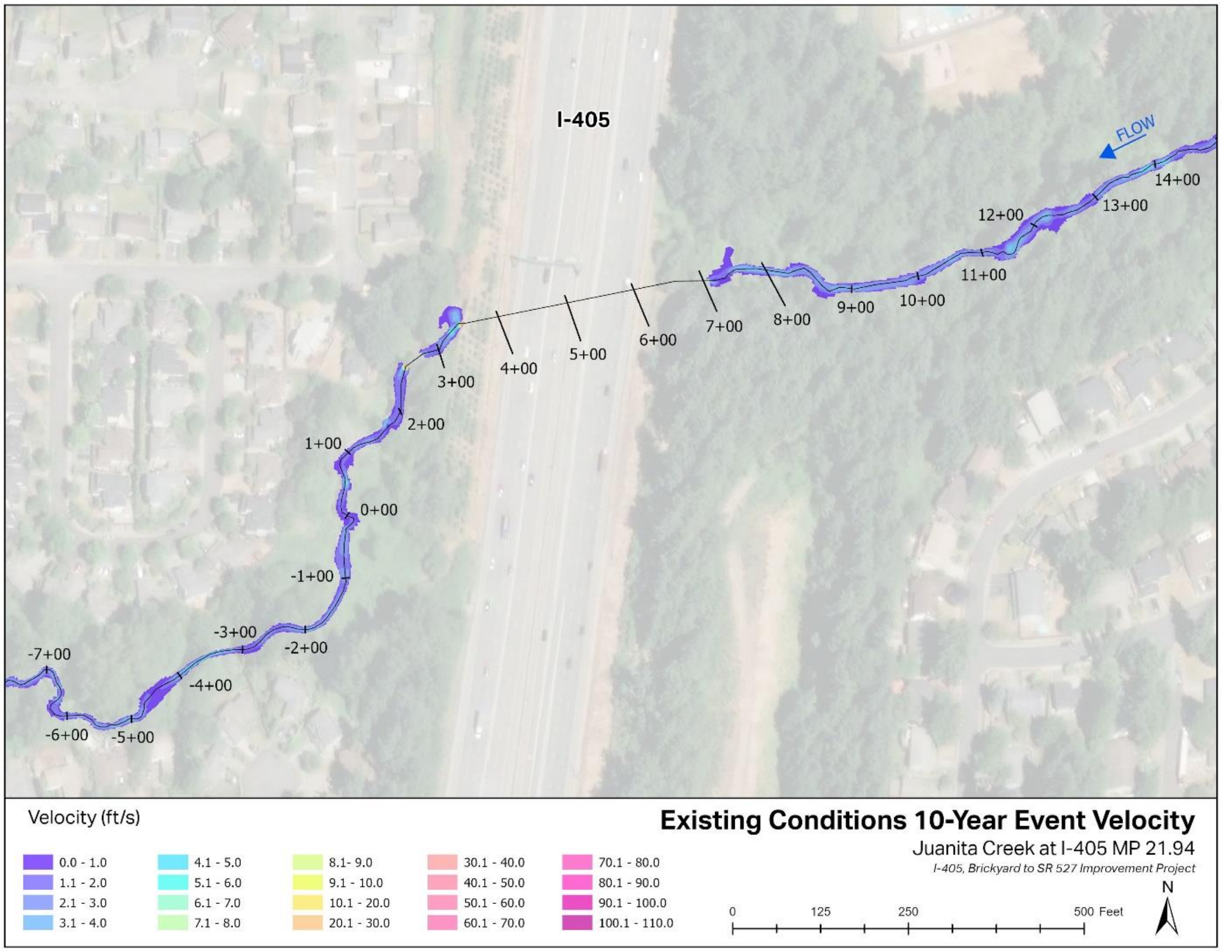
Juanita Creek

Existing Condition – 2 Year Event Depth



Juanita Creek

Existing Condition – 10 Year Event WSE



Juanita Creek

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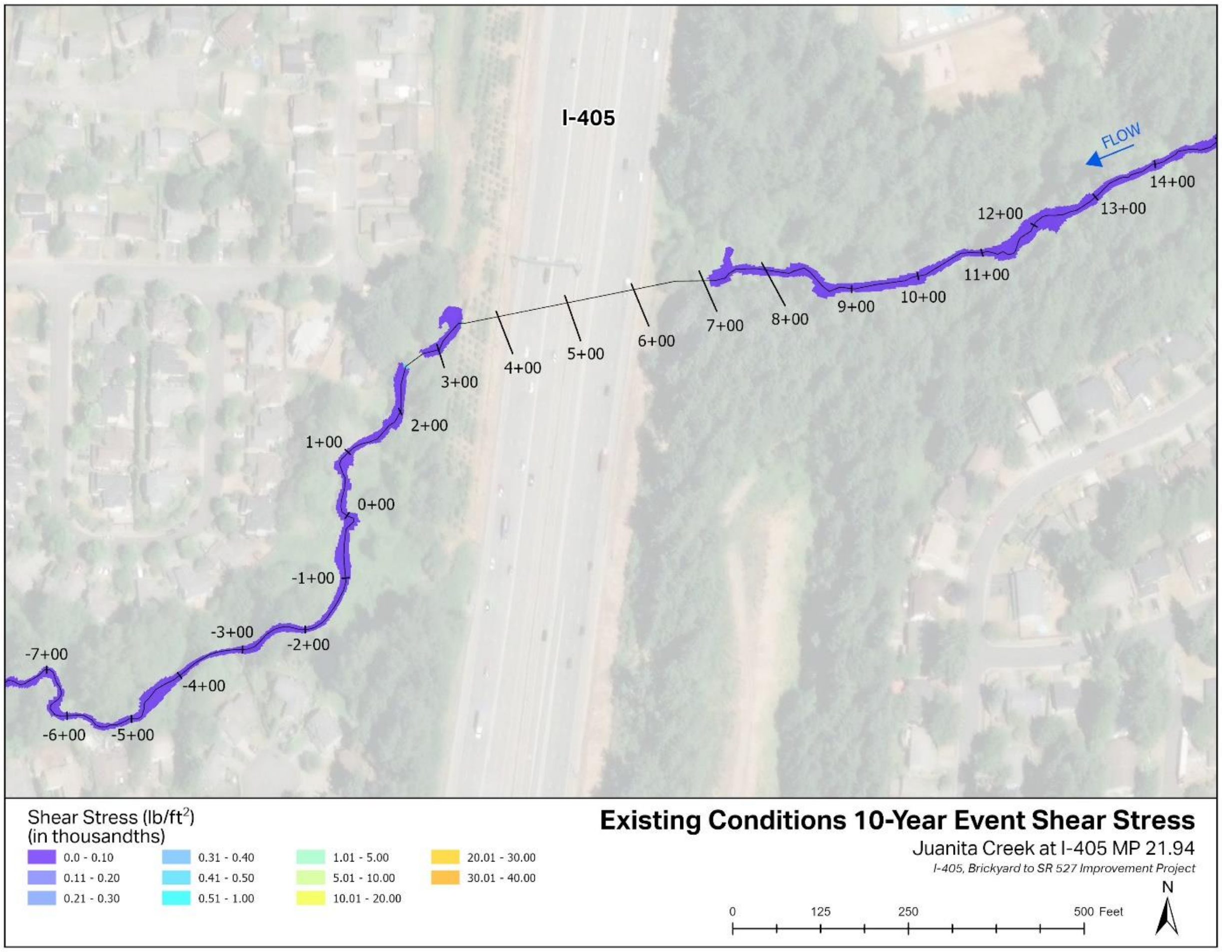

Juanita Creek

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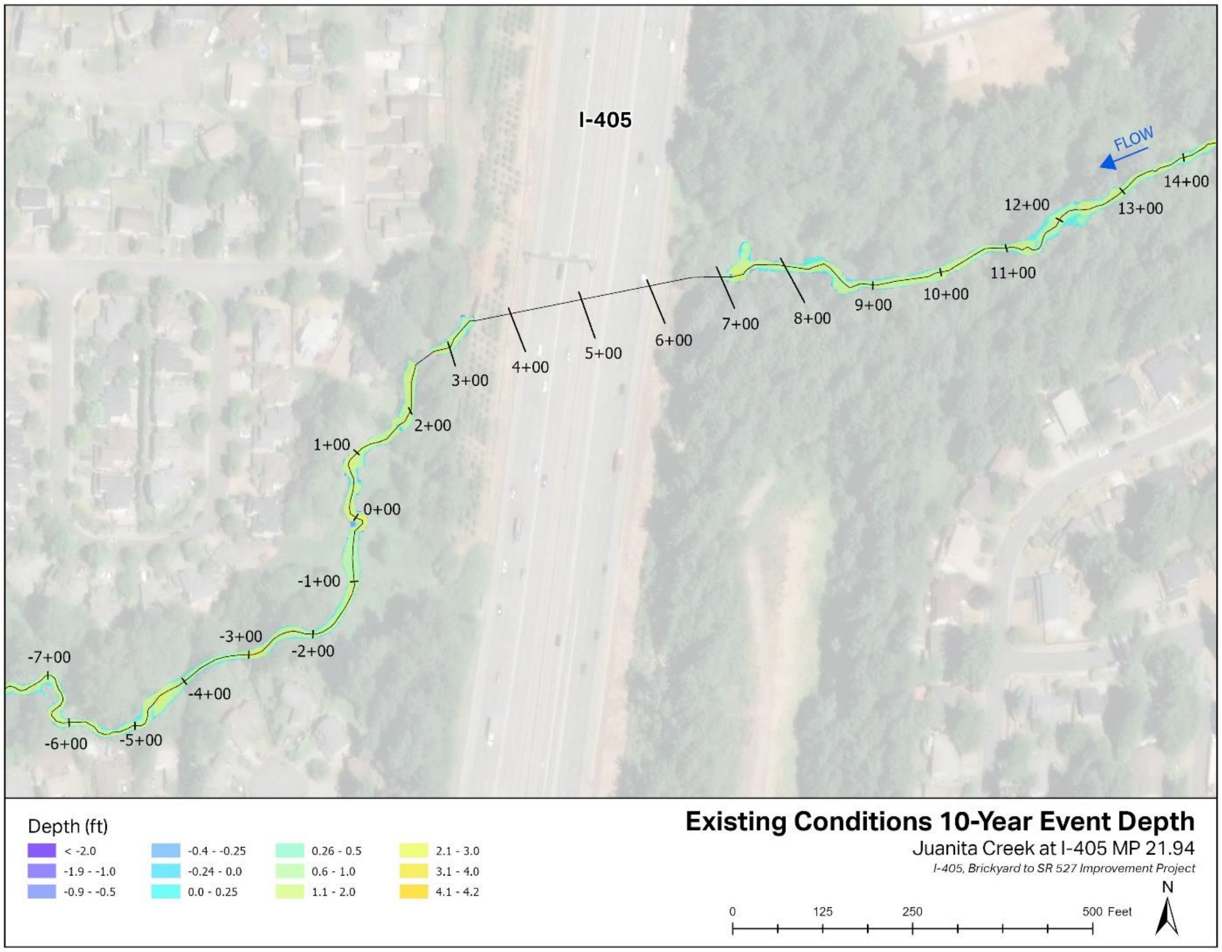

<u>Juanita Creek</u>

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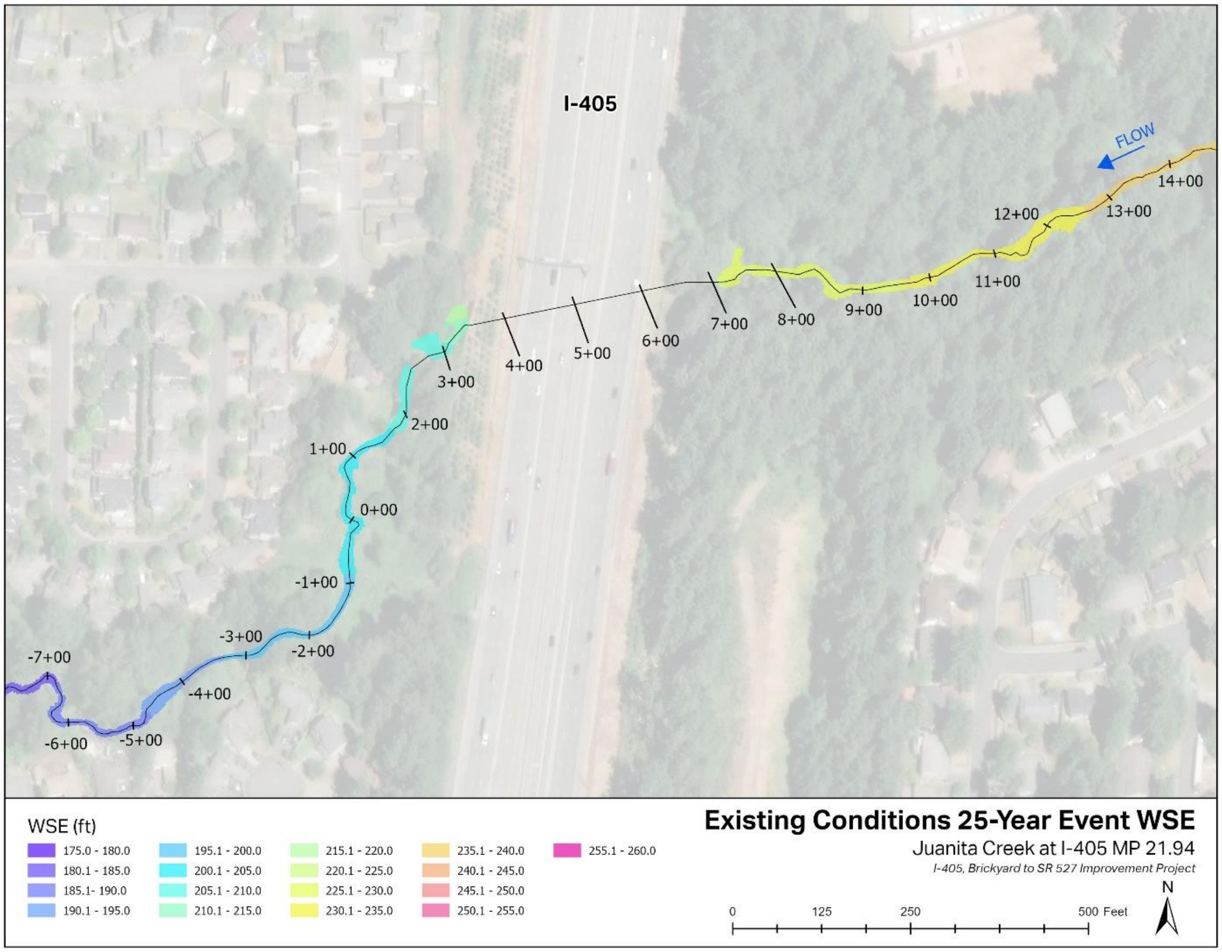
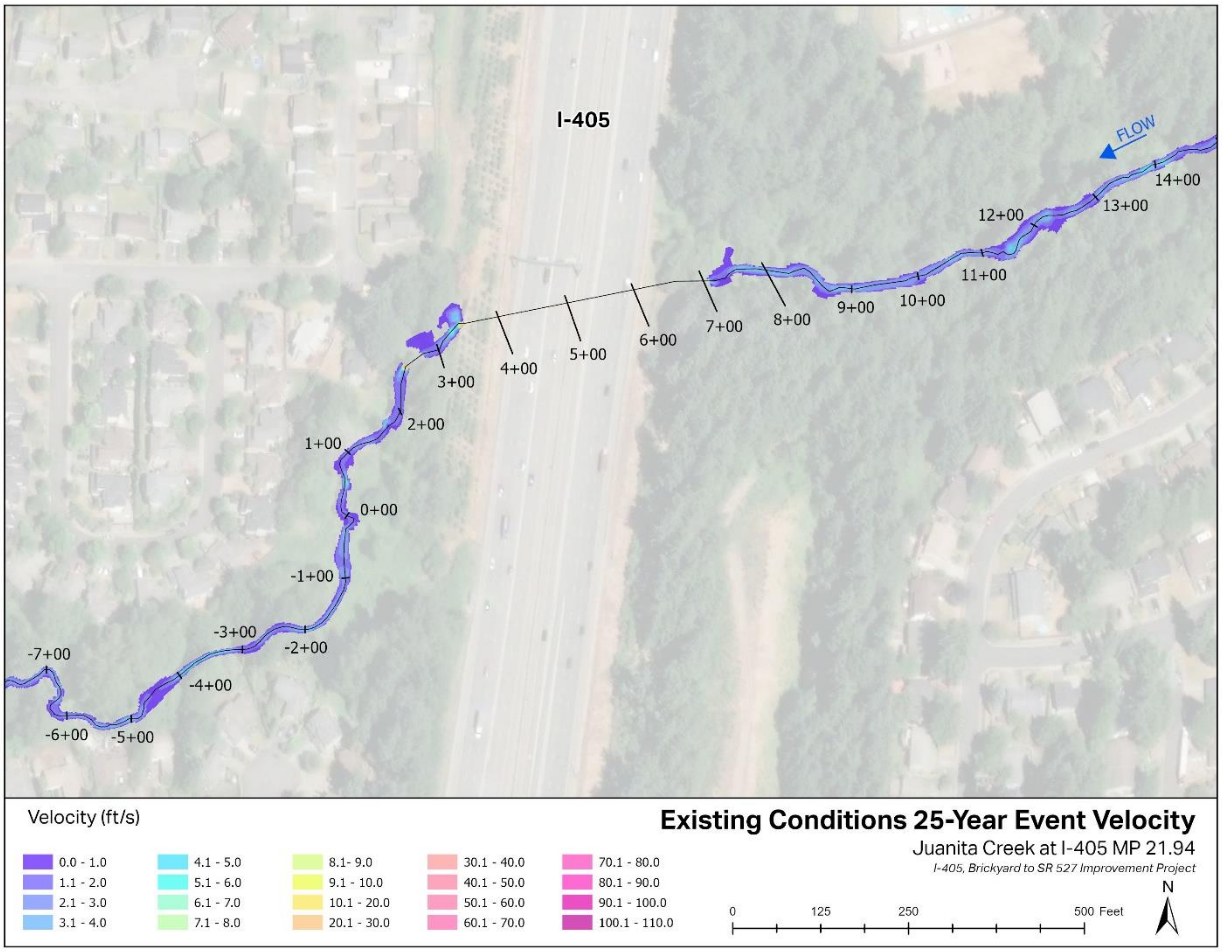


<u>Juanita Creek</u>

<u>Existing Condition – 25 Year Event WSE</u>



<u>Juanita Creek</u>

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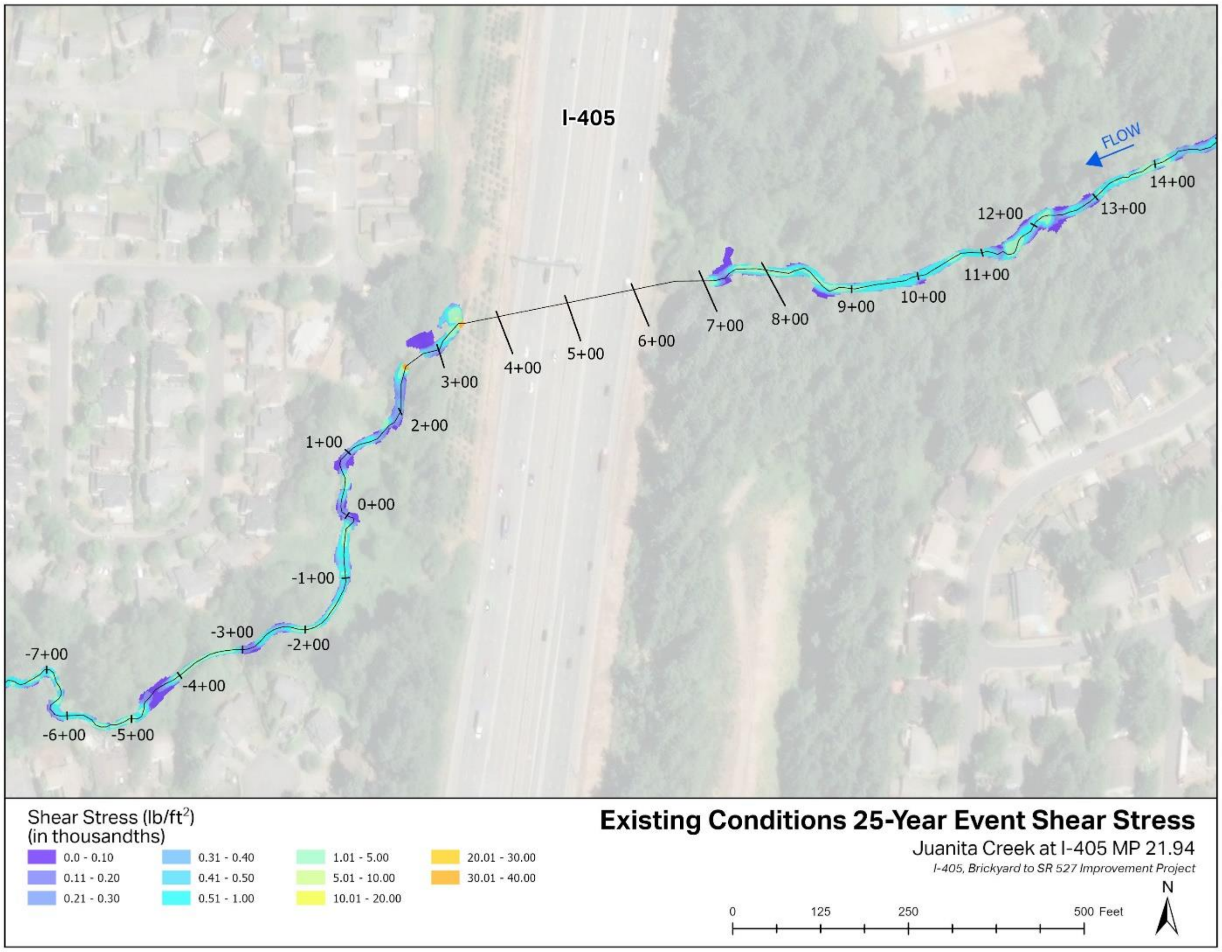

Juanita Creek

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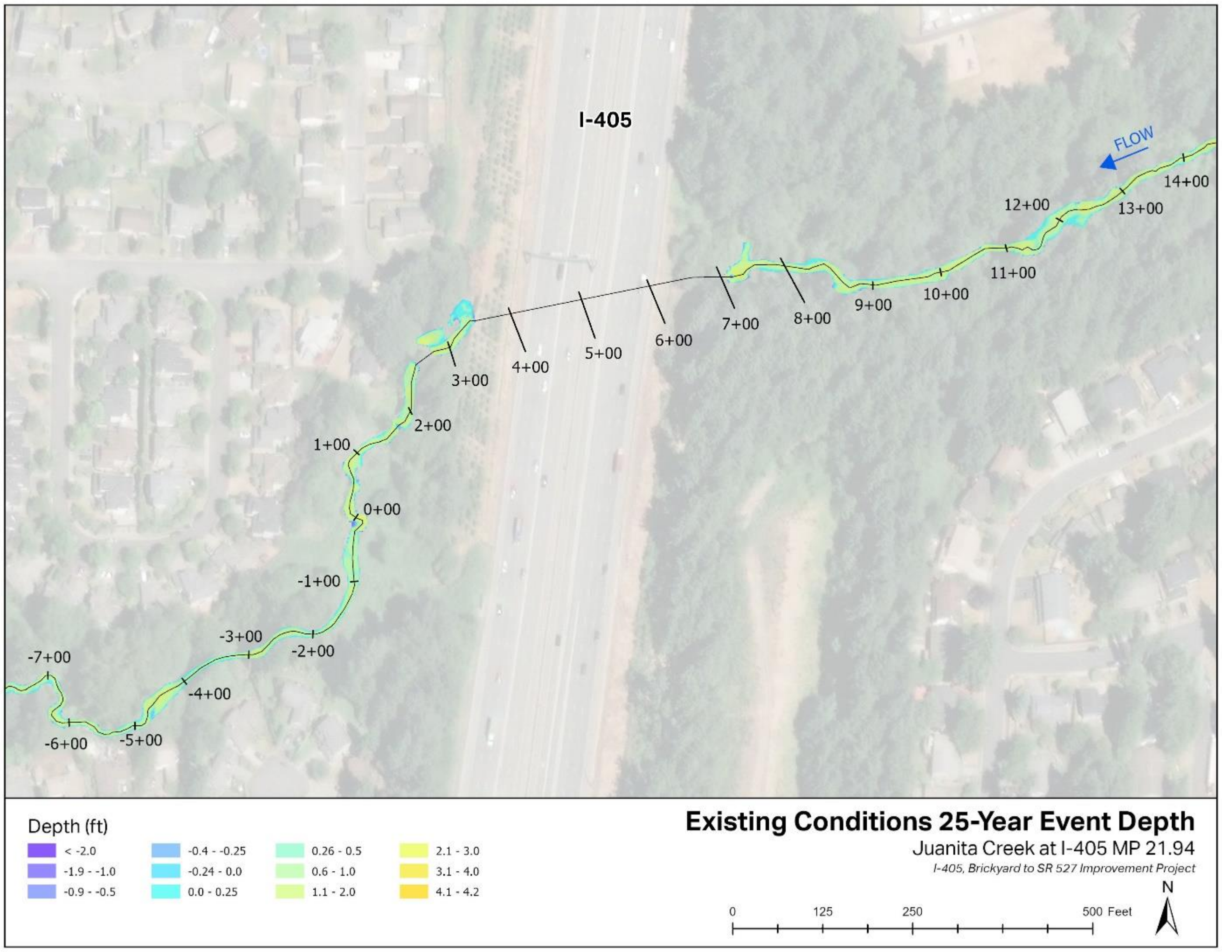

Juanita Creek

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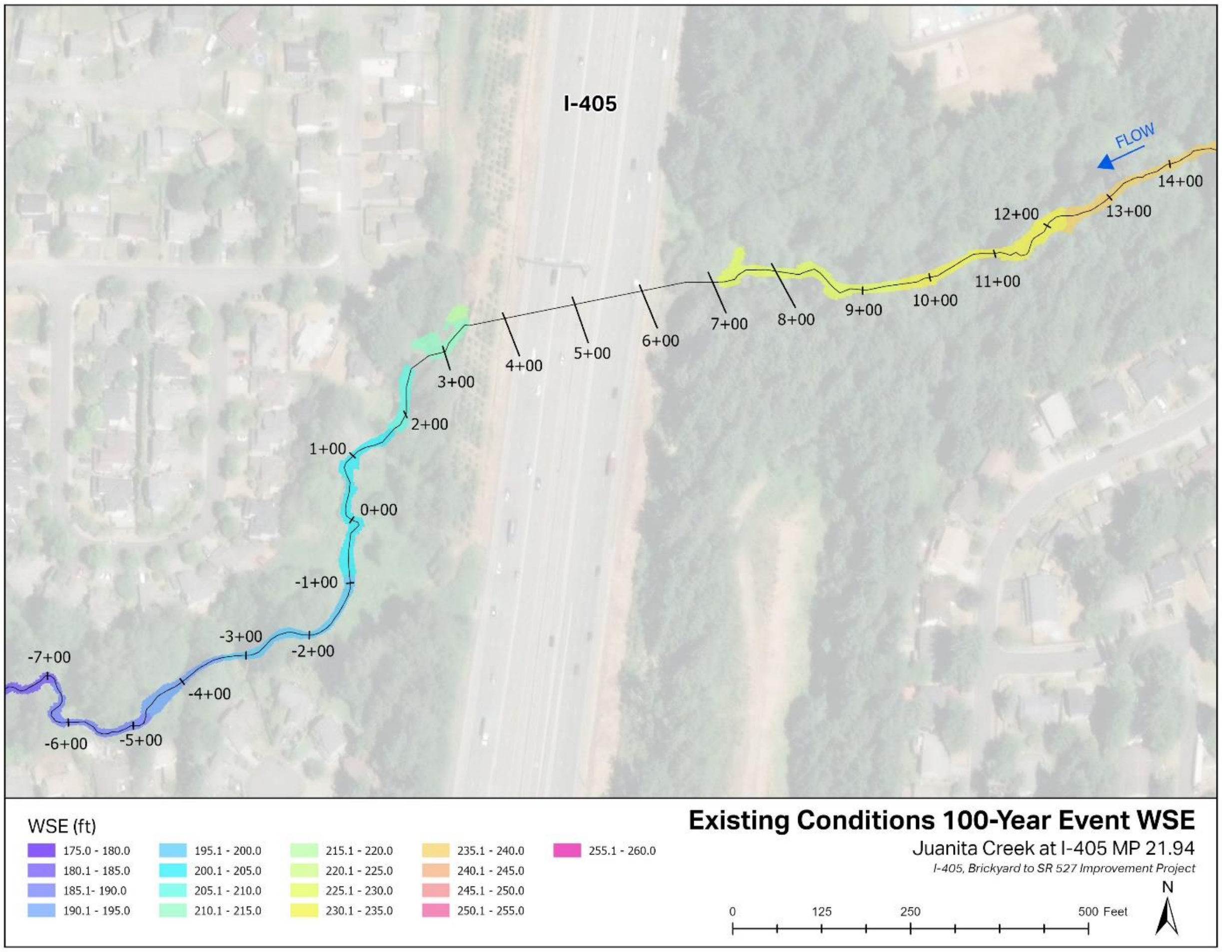
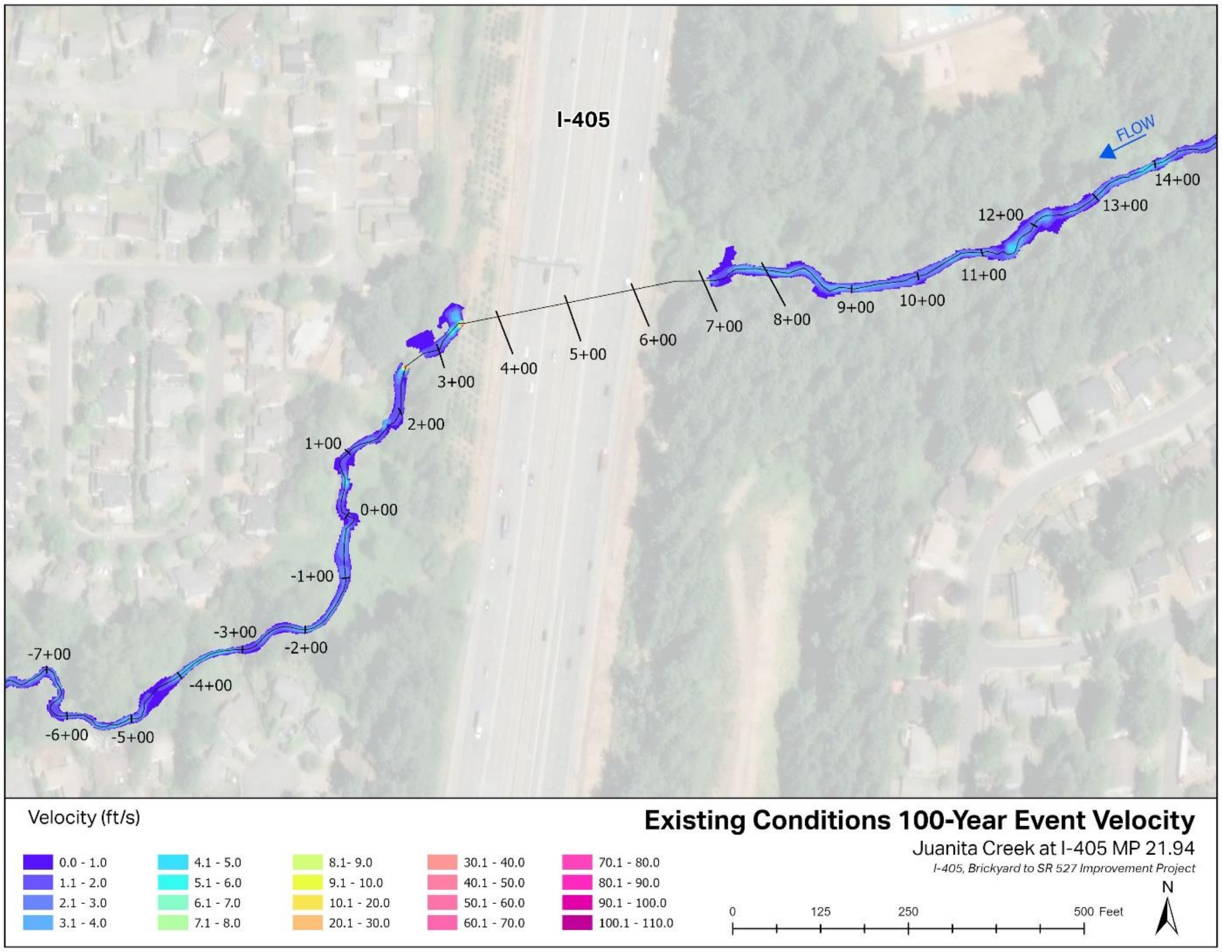


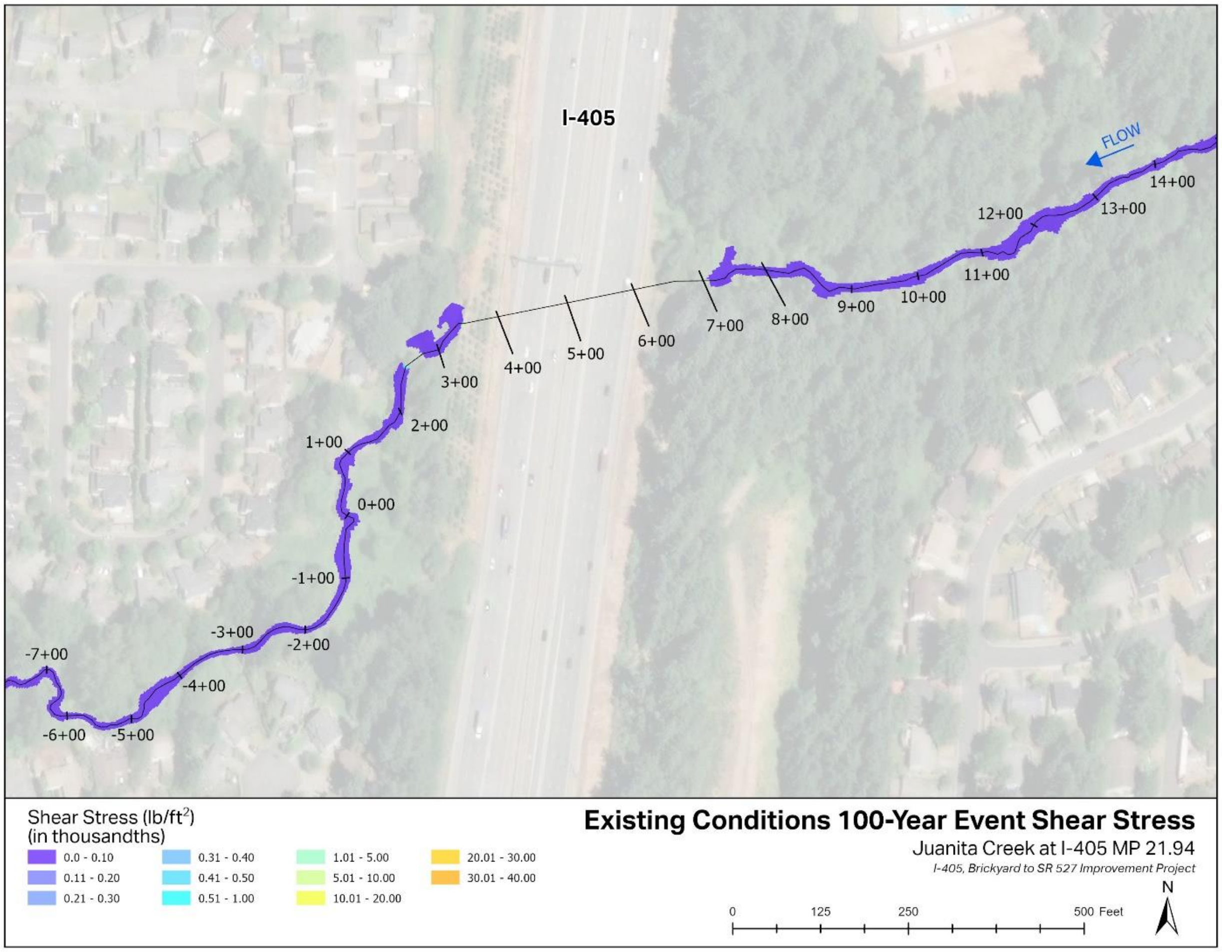
<u>Juanita Creek</u>

<u>Existing Condition – 100 Year Event WSE</u>



Juanita Creek

Existing Condition – 100 Year Event Velocity



Juanita Creek

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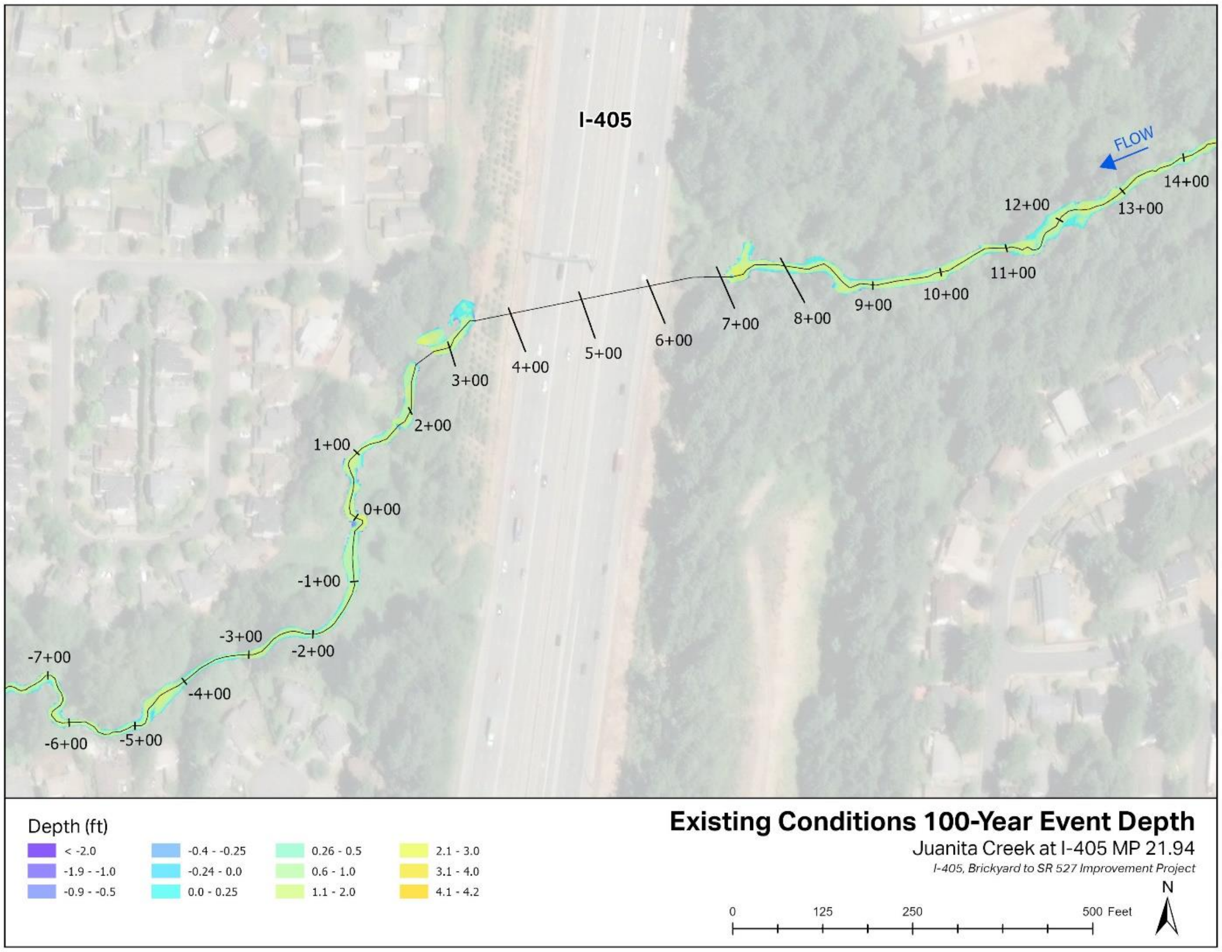
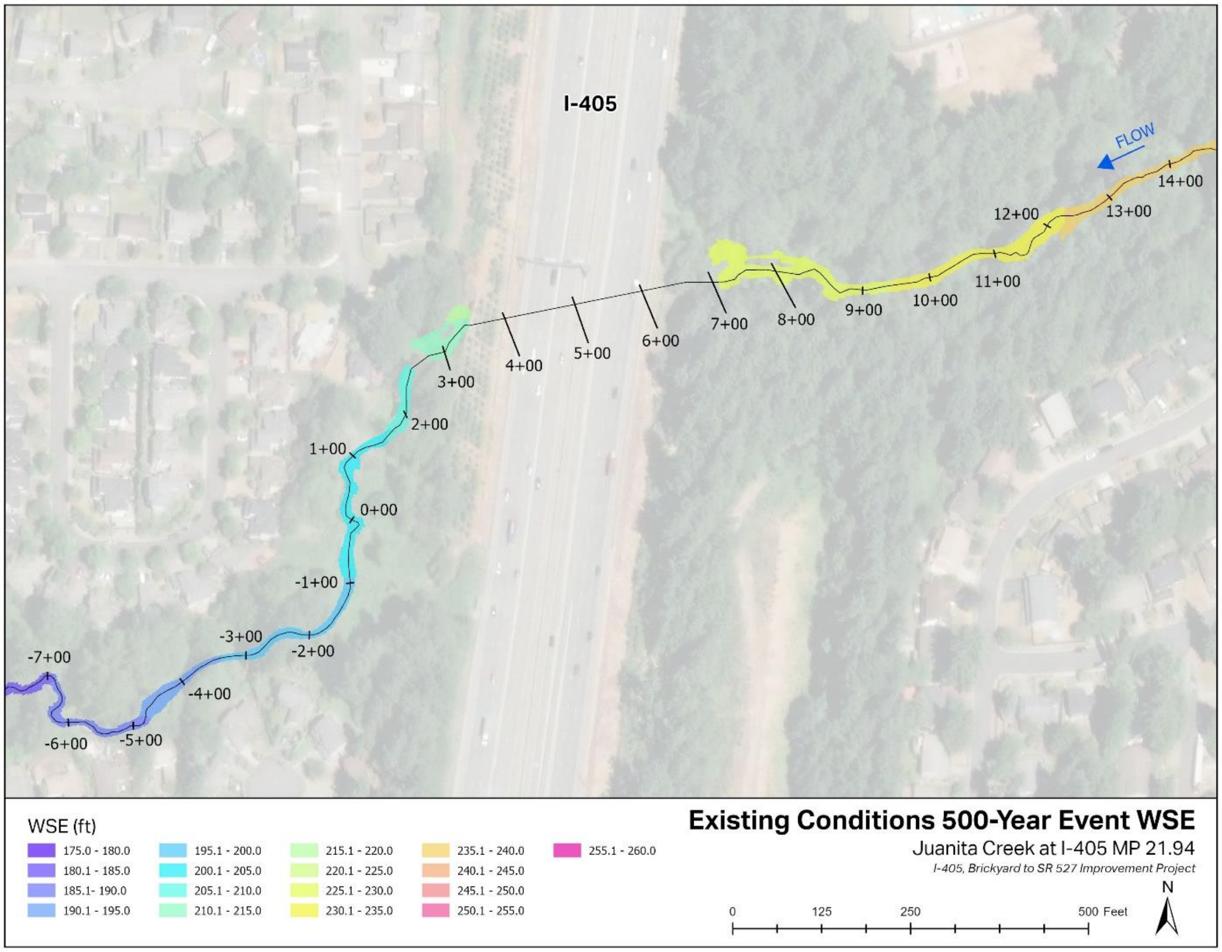


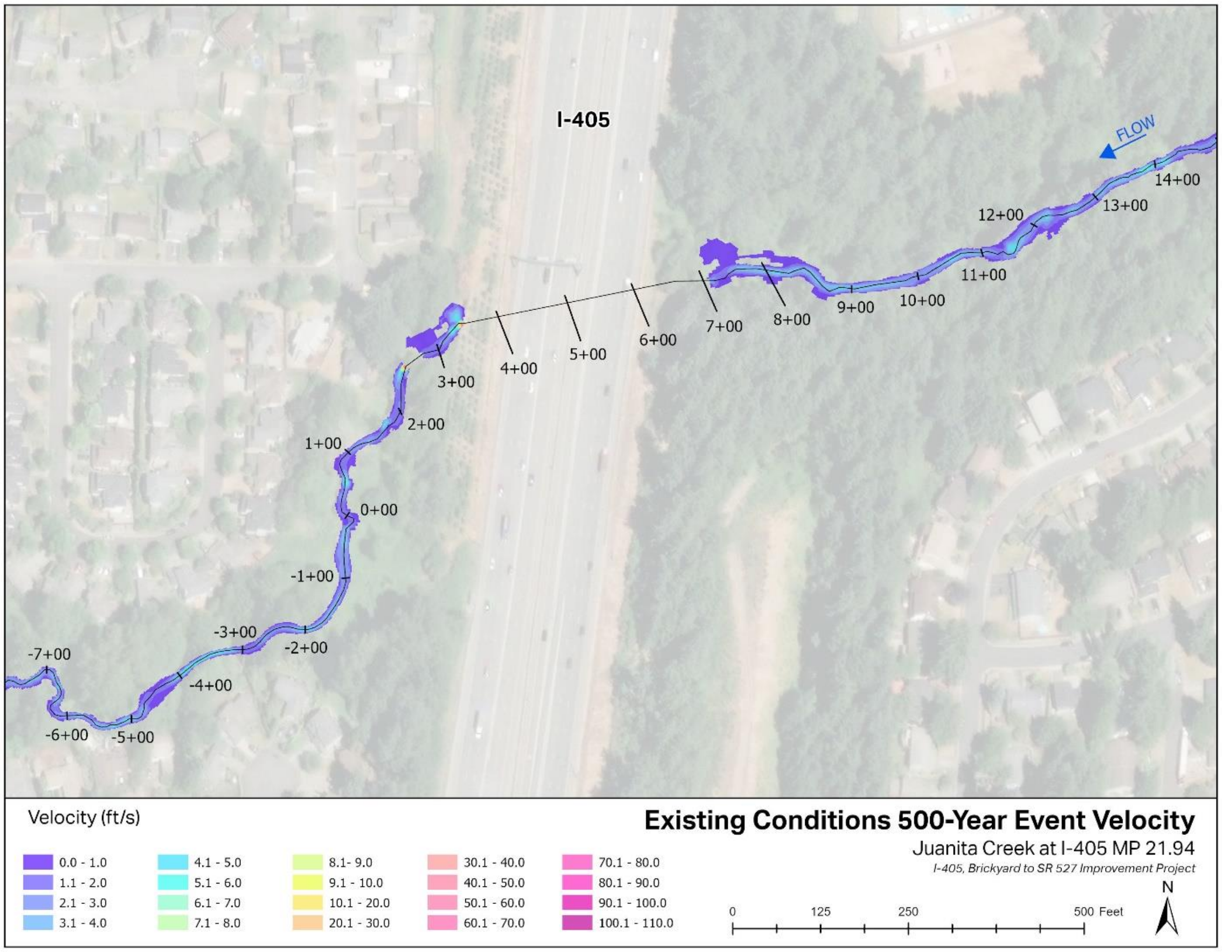
Juanita Creek

Existing Condition – 100 Year Event Depth



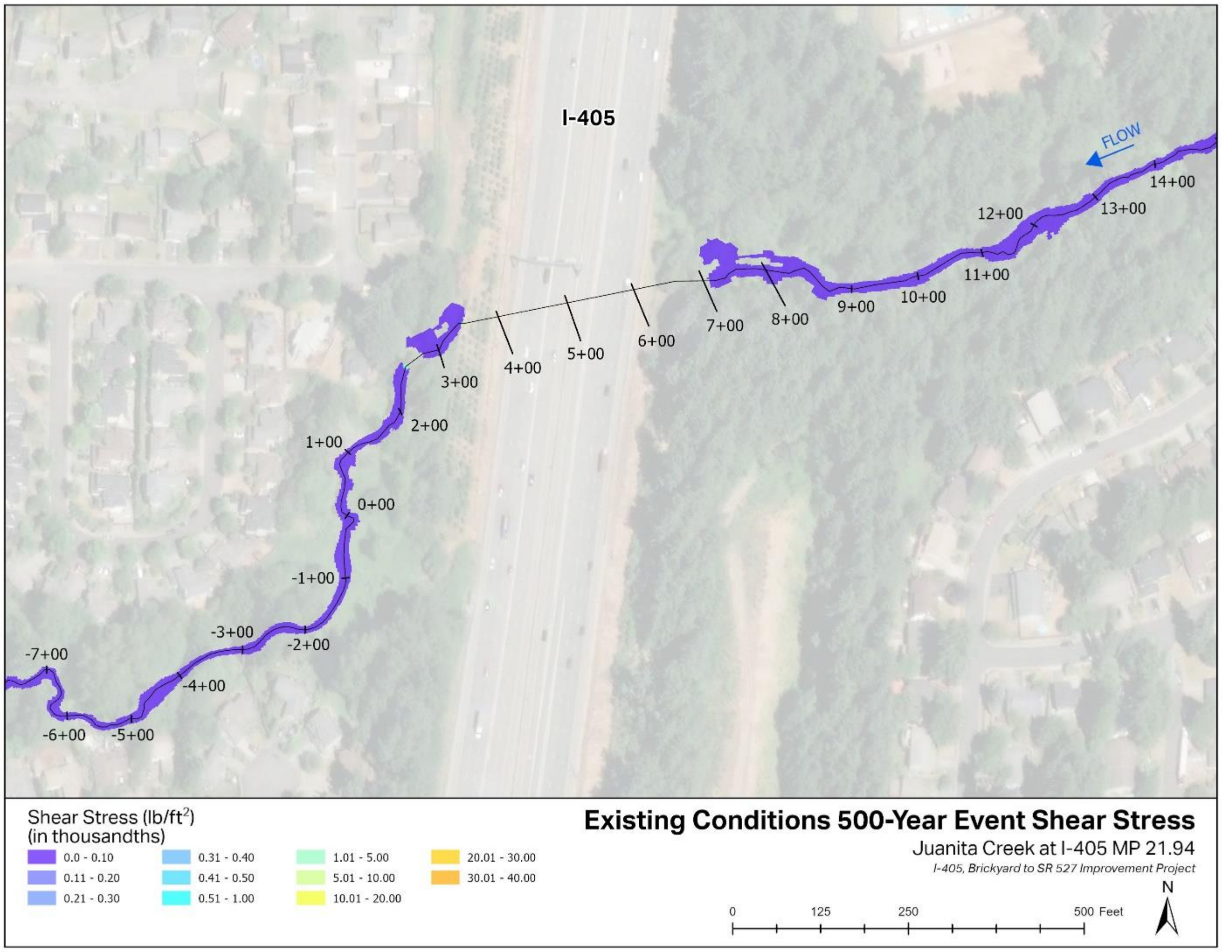
Juanita Creek

Existing Condition – 500 Year Event WSE



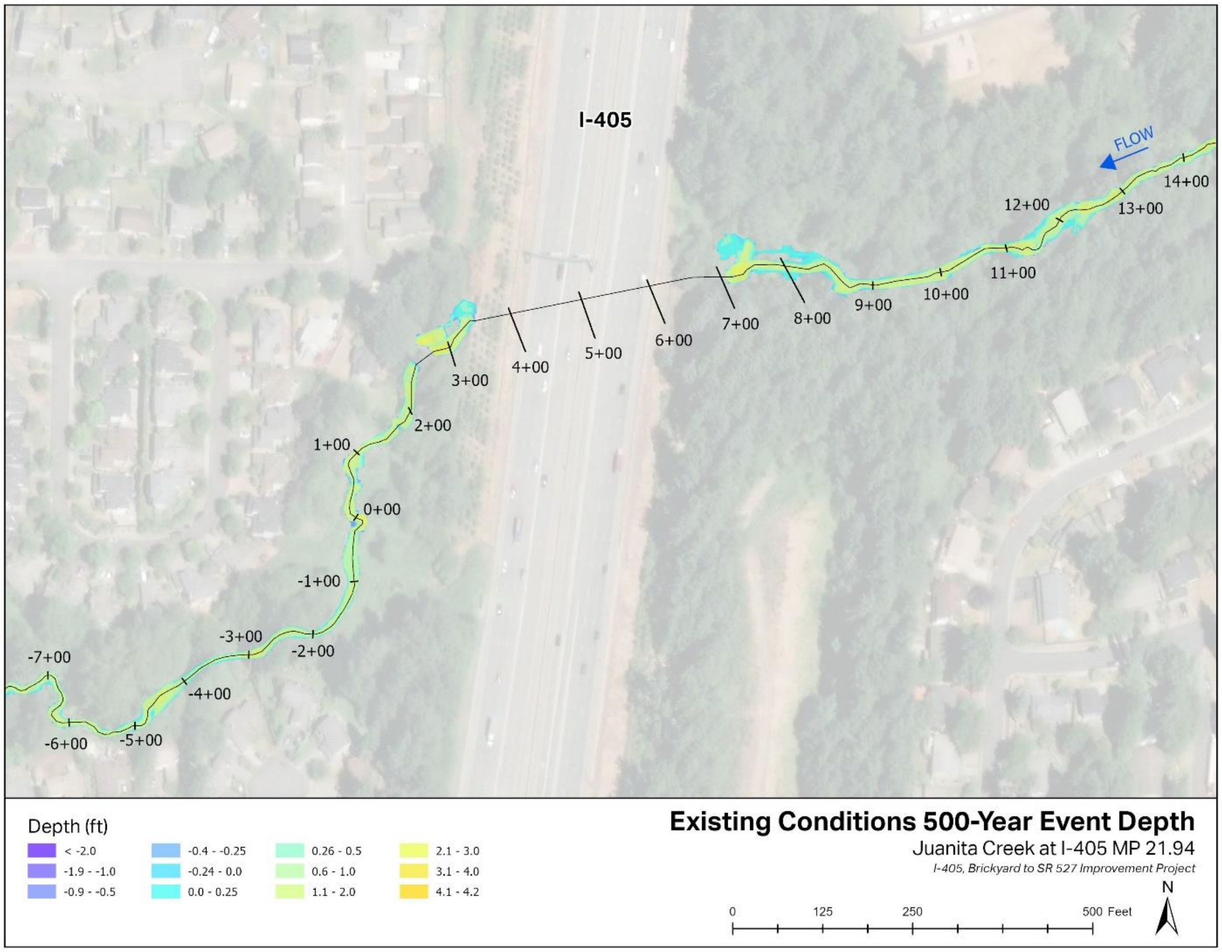
<u>Juanita Creek</u>

<u>Existing Condition – 500 Year Event Velocity</u>



Juanita Creek

Existing Condition – 500 Year Event Shear Stress



Juanita Creek

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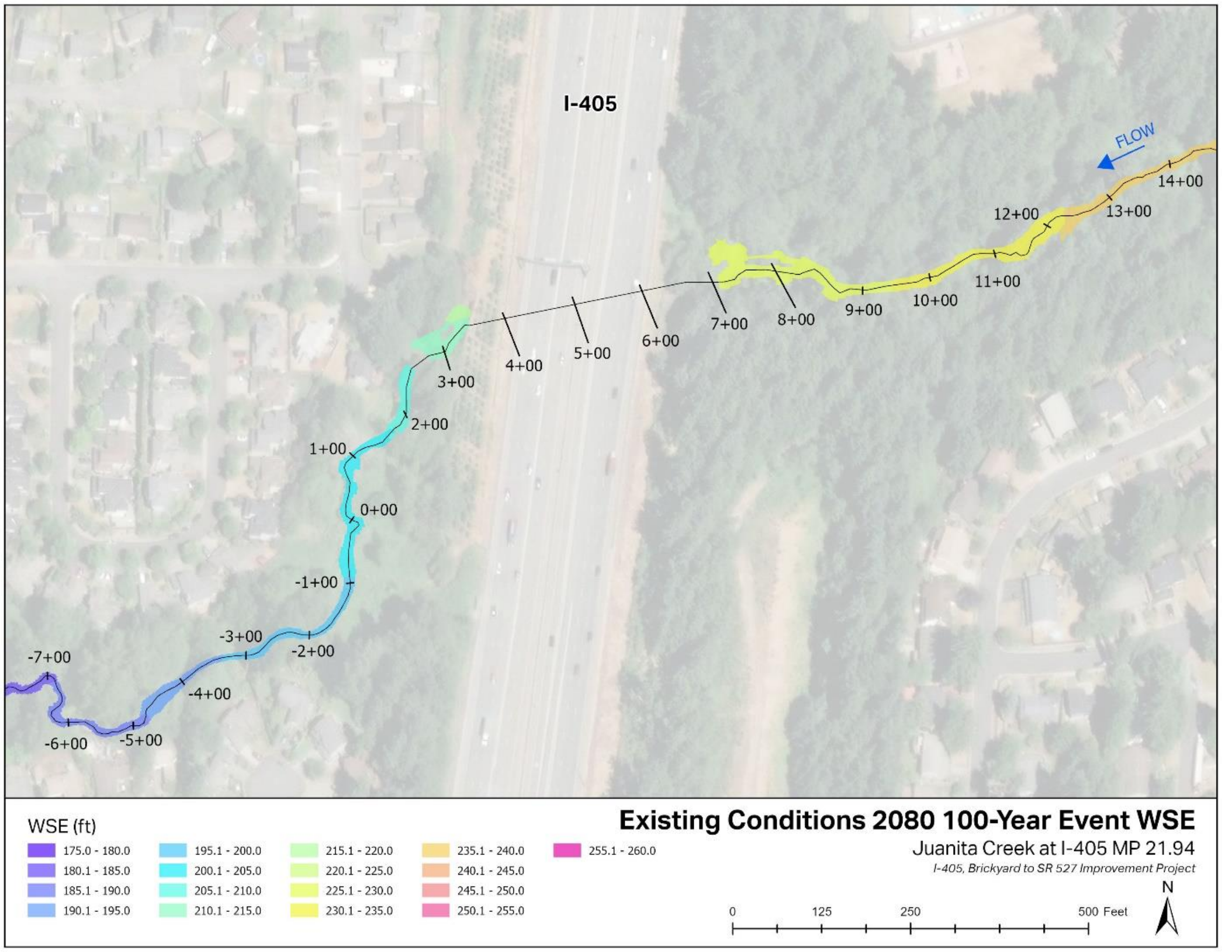
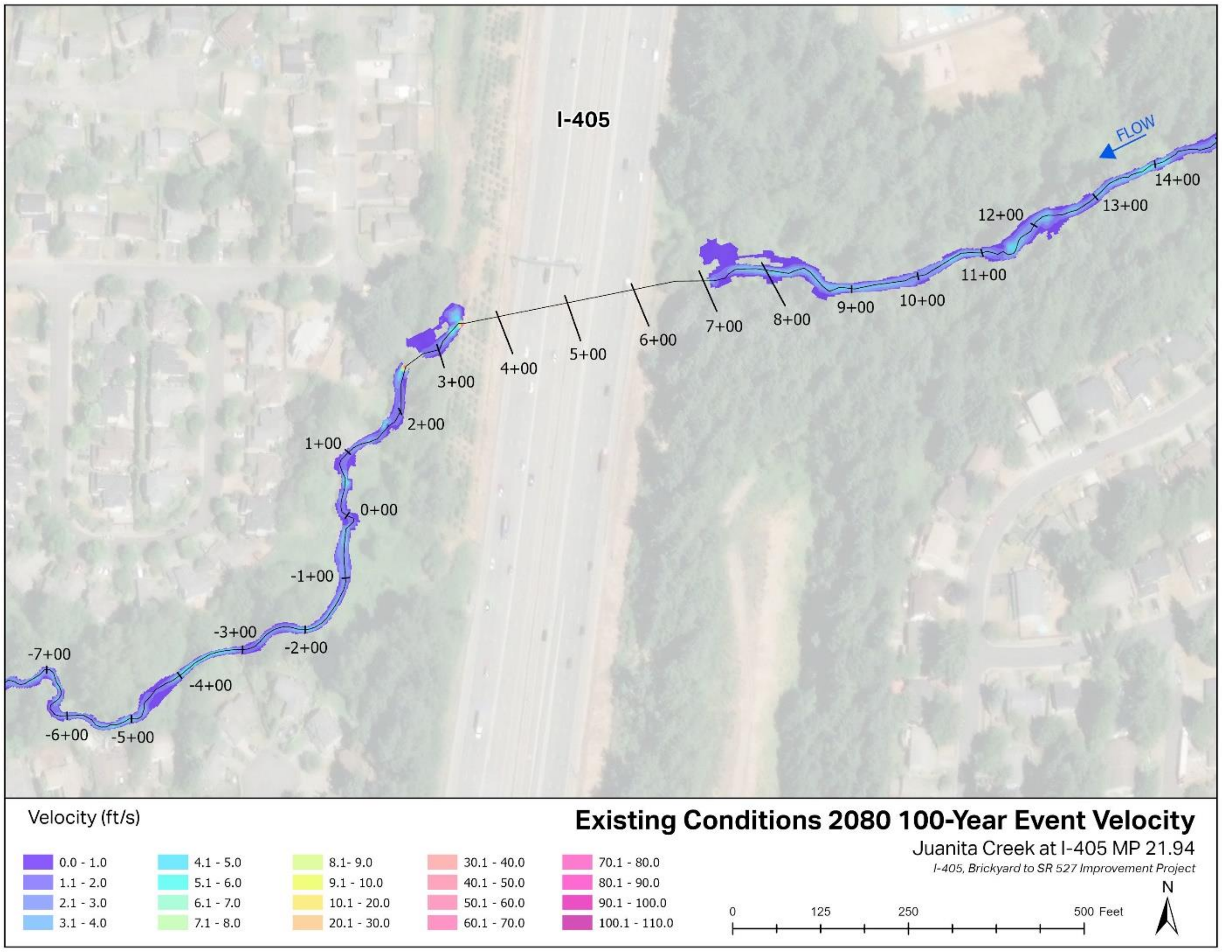


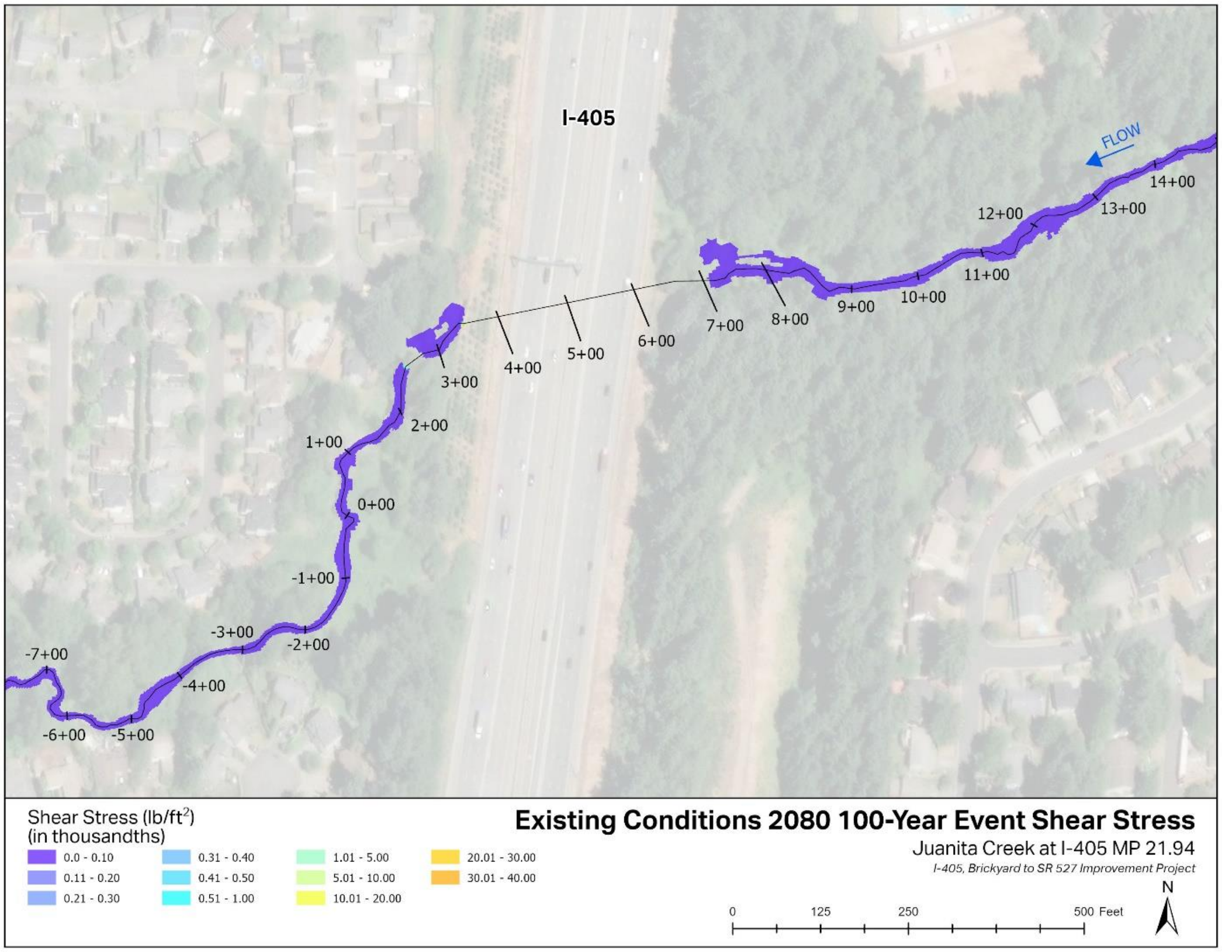
Juanita Creek

Existing Condition – 2080 100 Year Event WSE



Juanita Creek

Existing Condition – 2080 100 Year Event Velocity



<u>Juanita Creek</u>

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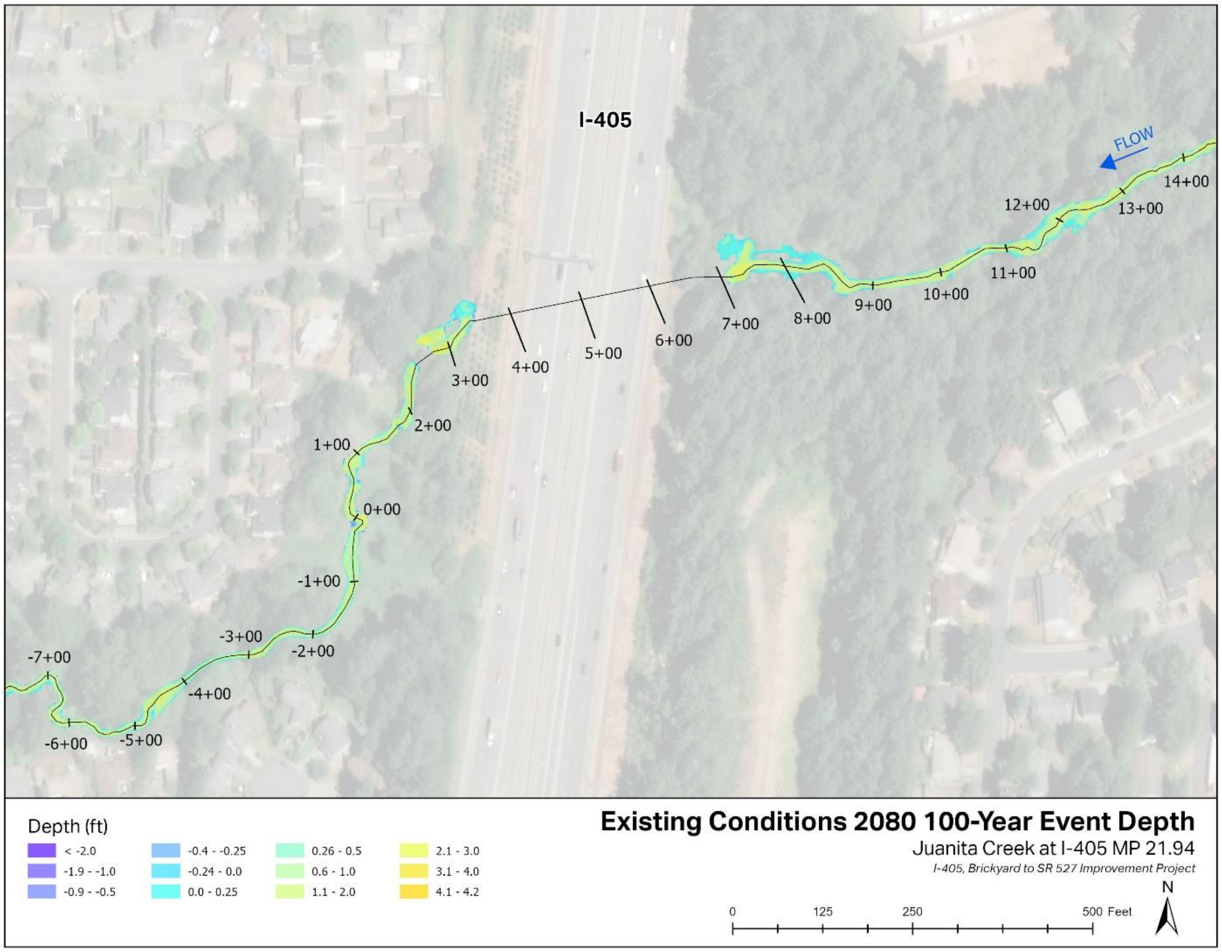
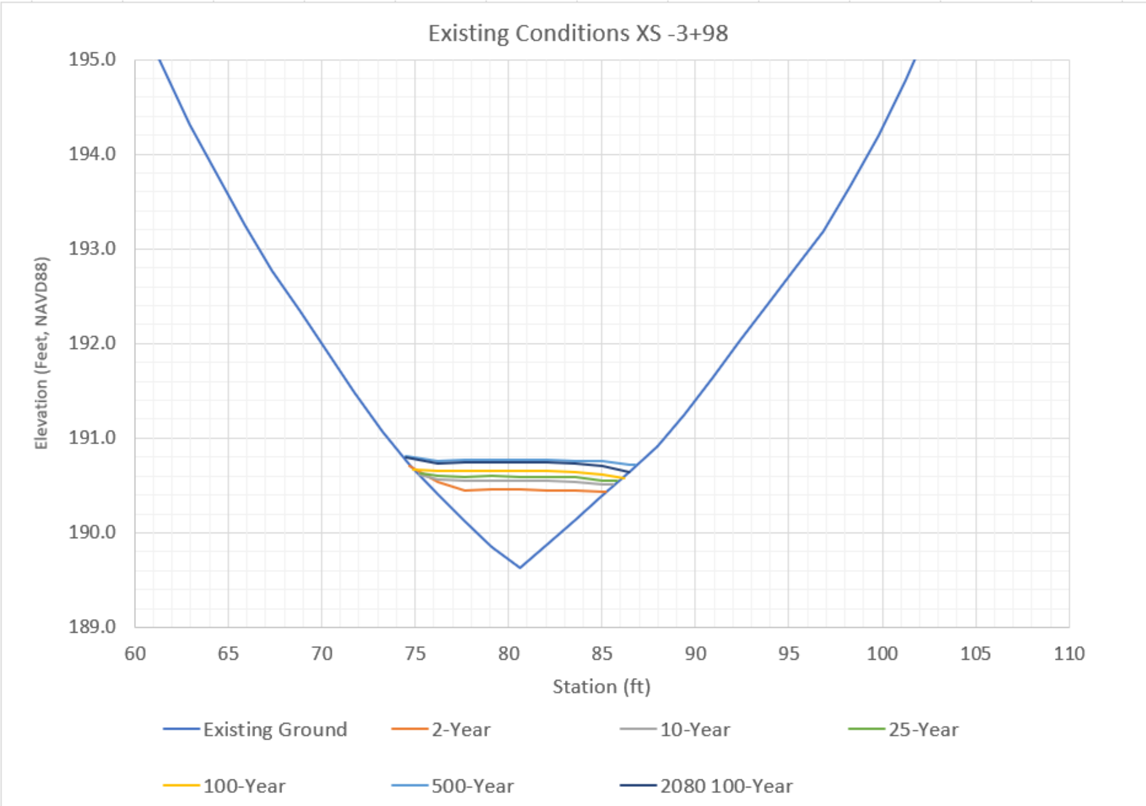
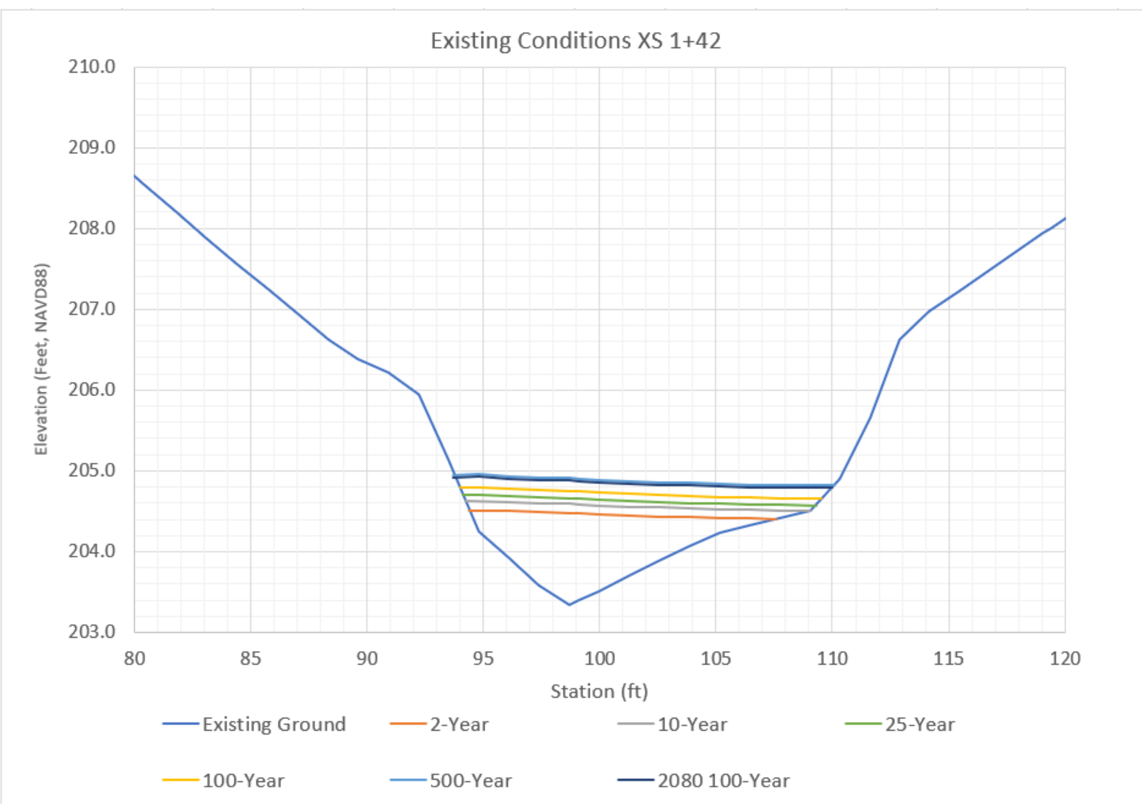


<u>Juanita Creek</u>

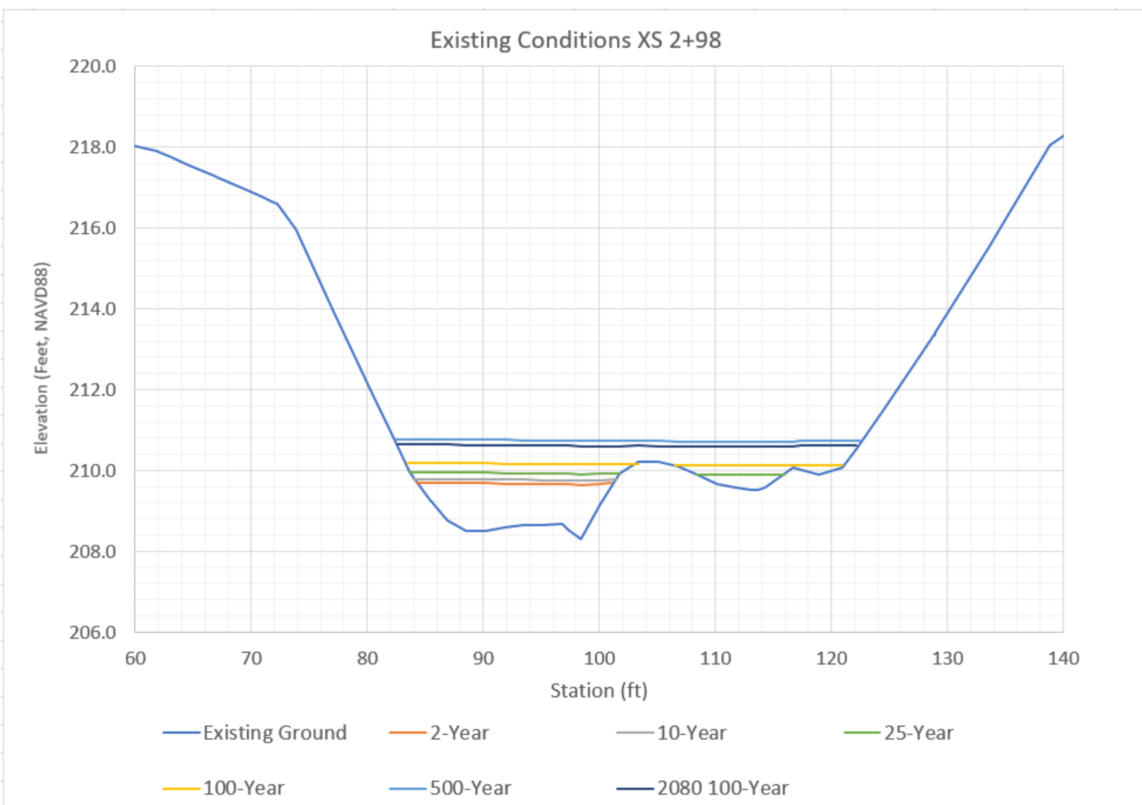
<u>Existing Condition – 2080 100 Year Event Depth</u>

Note: All cross sections are looking downstream



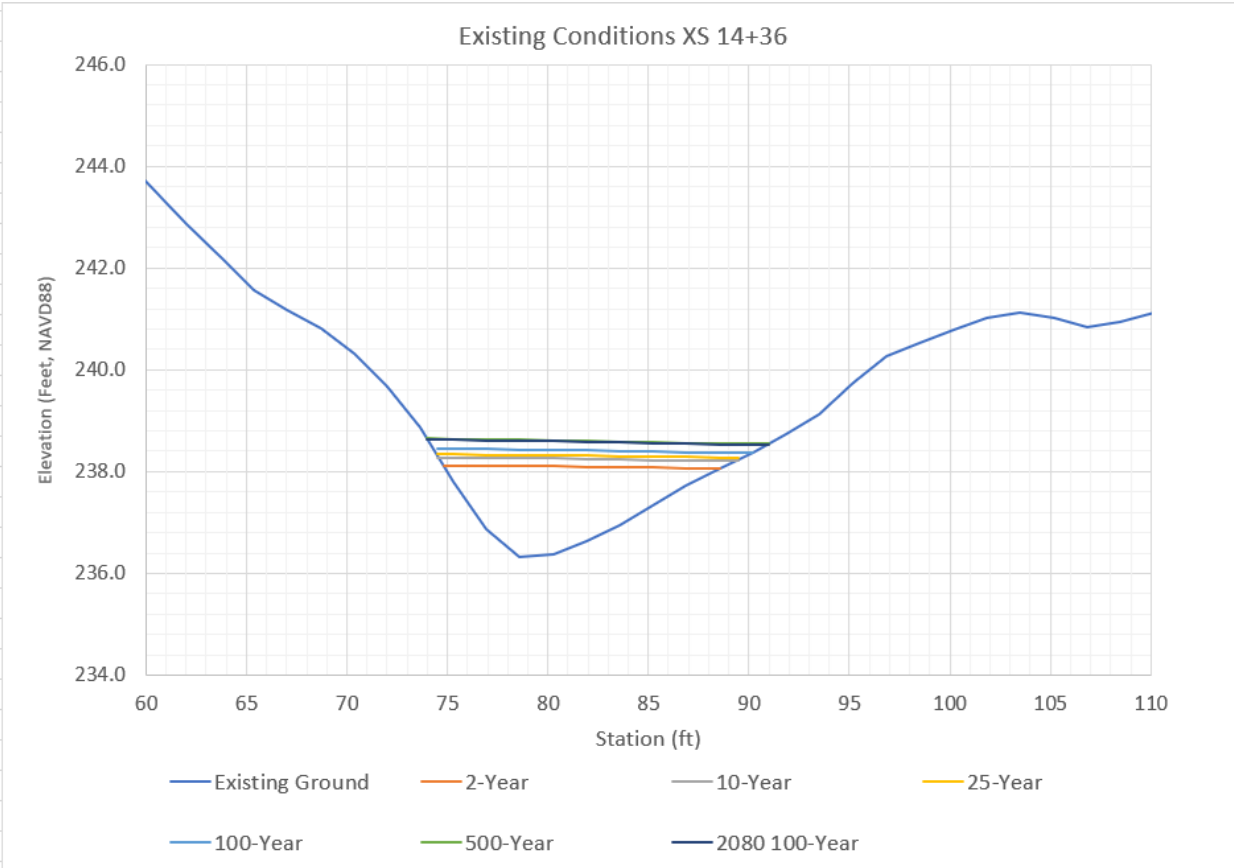


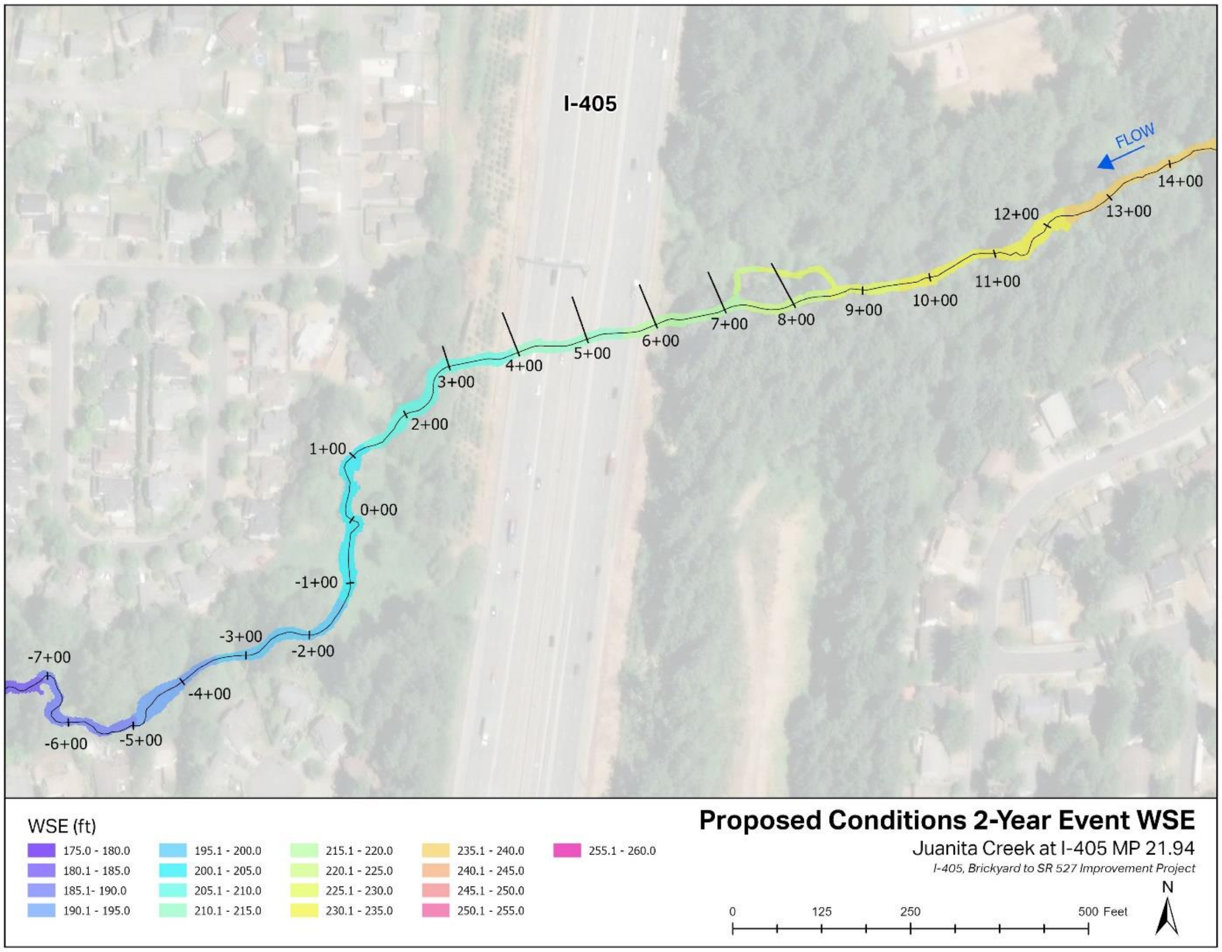

Existing Conditions XS 2+98
Elevation (Feet, NAVD88)
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
Series5
500-Year
2080 100-Year


Existing Conditions XS 2+98
Elevation (Feet, NAVD88)
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year


Existing Conditions XS 7+06
Elevation (Feet, NAVD88)
232.0
231.0
230.0
229.0
228.0
227.0
226.0
225.0
224.0
223.0
222.0
65
85
105
125
145
165
185
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year

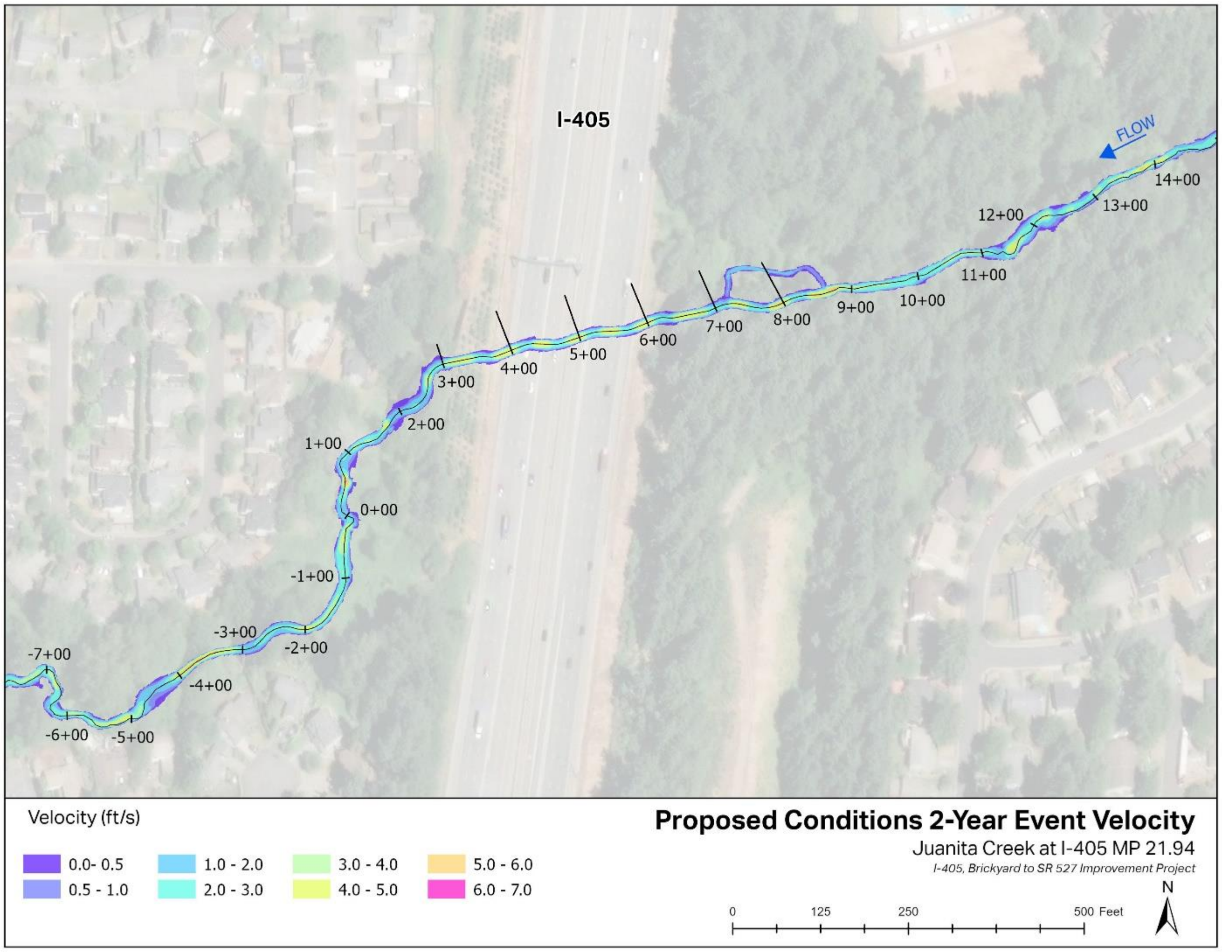

Existing Conditions XS 9+36
Elevation (Feet, NAVD88)
235.0
234.0
233.0
232.0
231.0
230.0
229.0
228.0
227.0
60
70
80
90
100
110
120
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year


Existing Conditions XS 14+36
246.0
244.0
242.0
240.0
238.0
236.0
234.0
Elevation (Feet, NAVD88)
60
65
70
75
80
85
90
95
100
105
110
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year



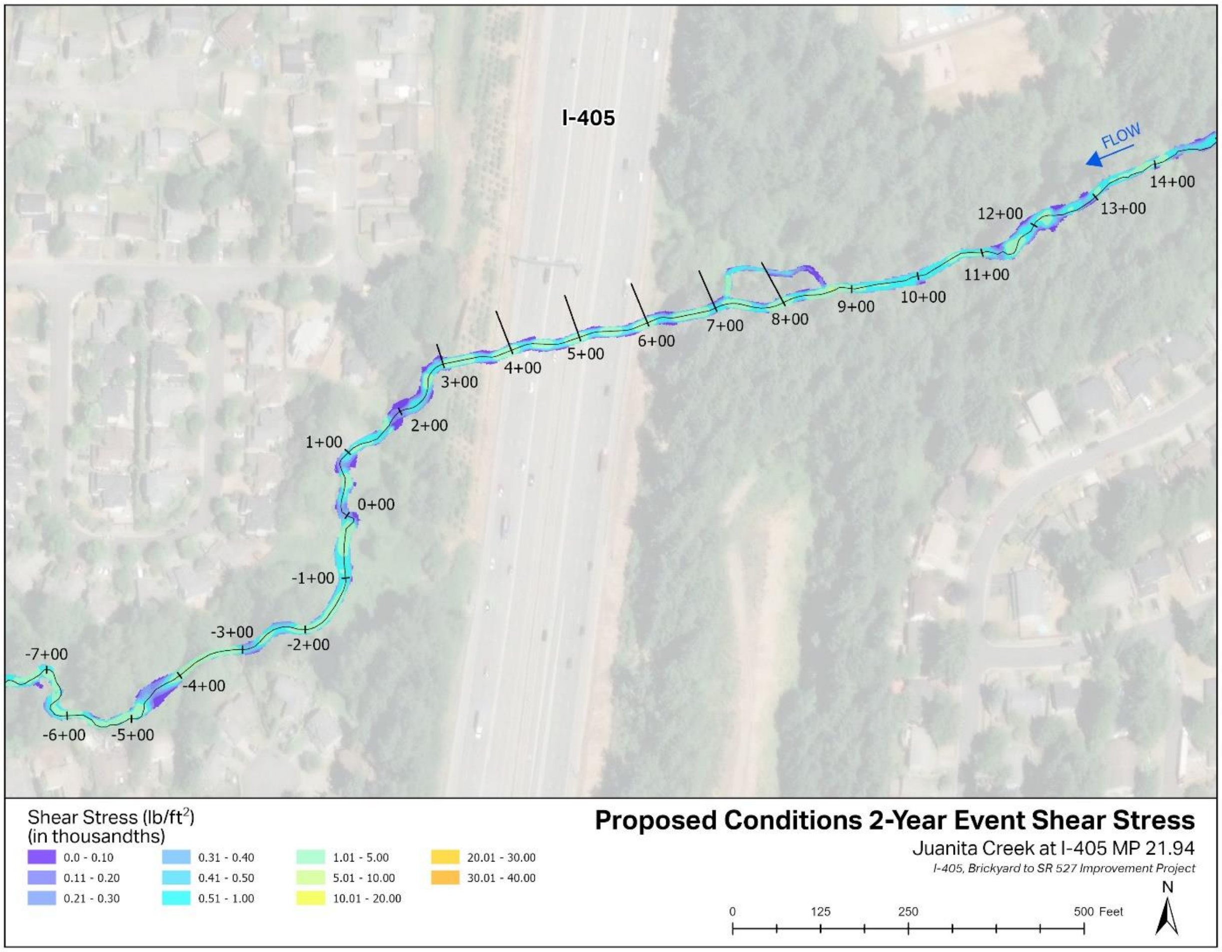
Juanita Creek

Proposed Condition – 2 Year Event WSE



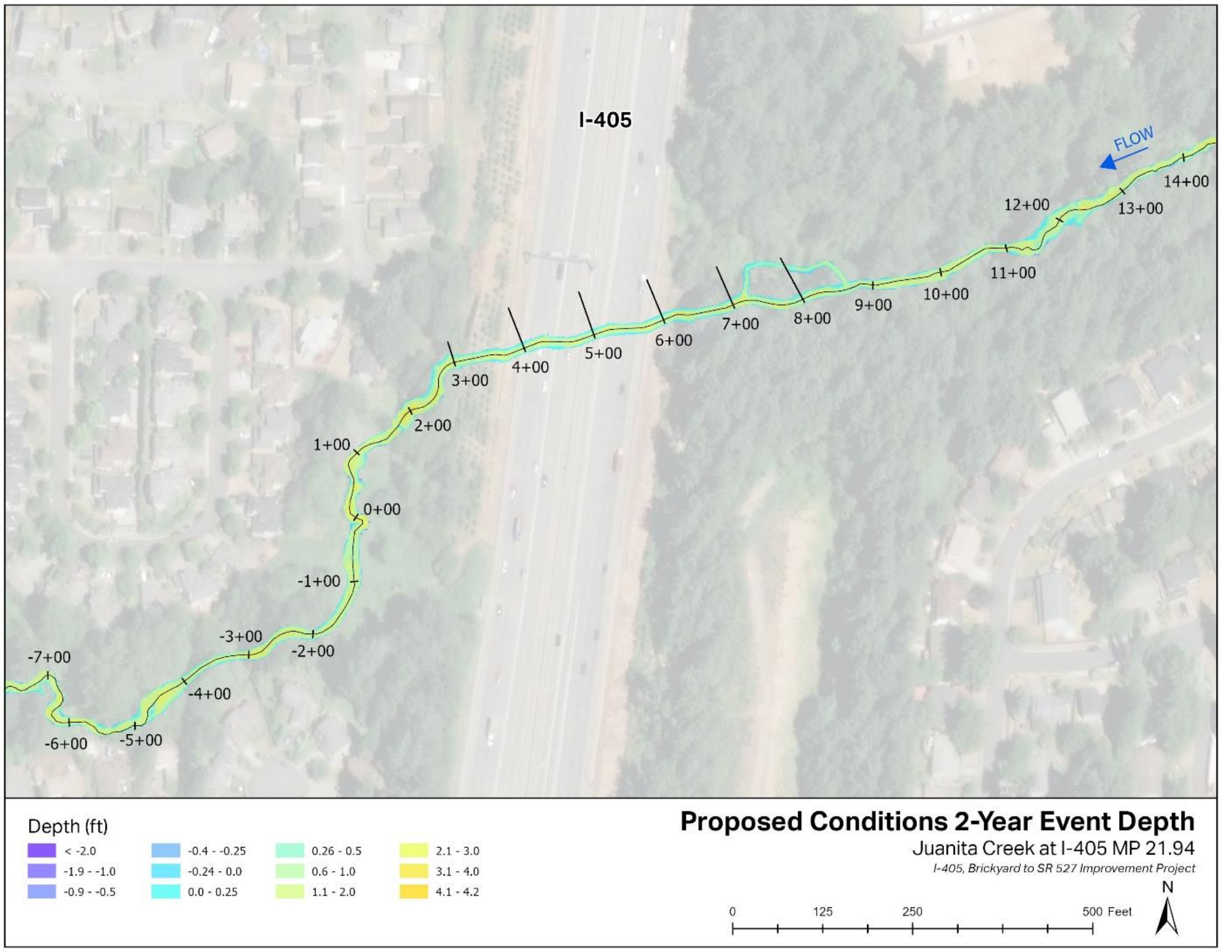
Juanita Creek

Proposed Condition – 2 Year Event Velocity



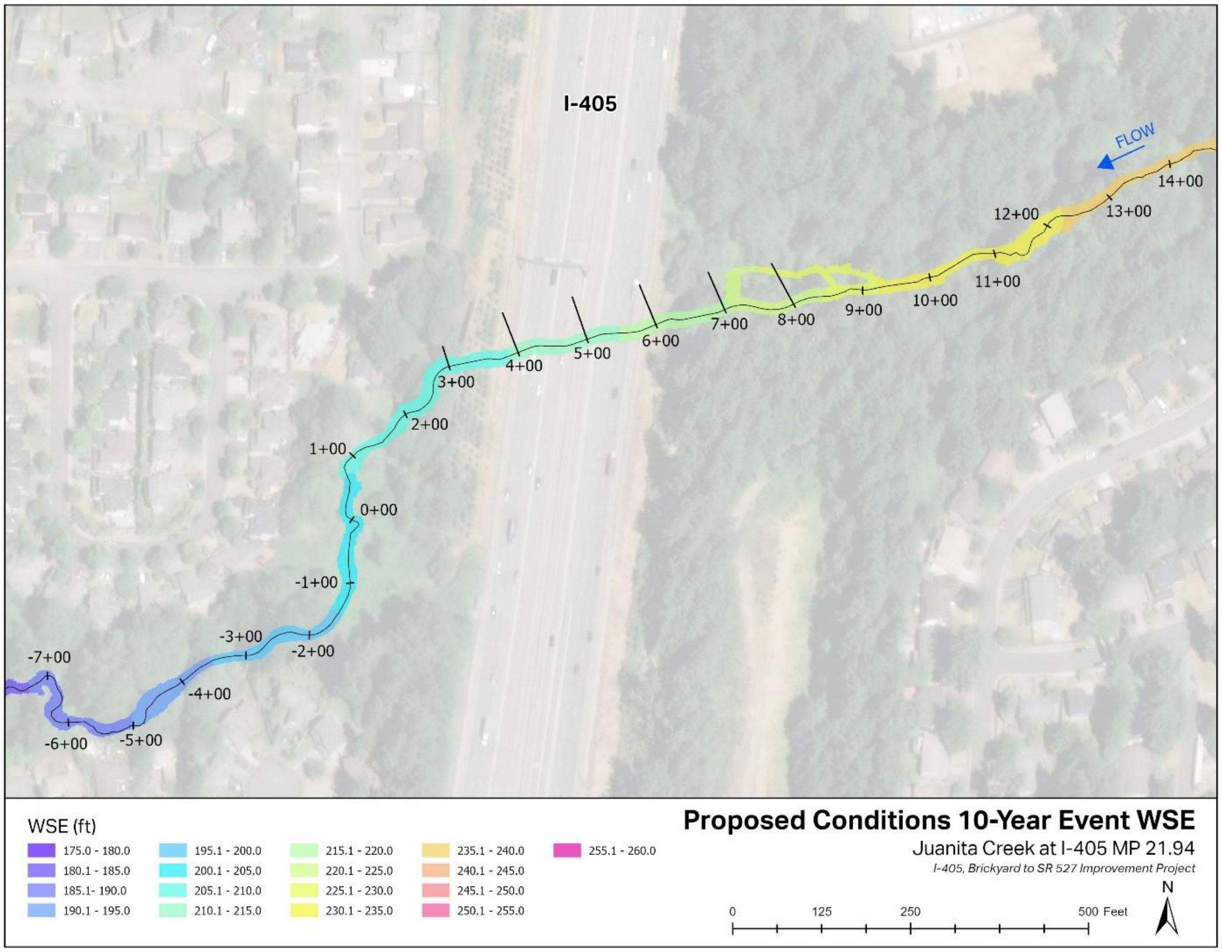
Juanita Creek

Proposed Condition – 2 Year Event Shear Stress



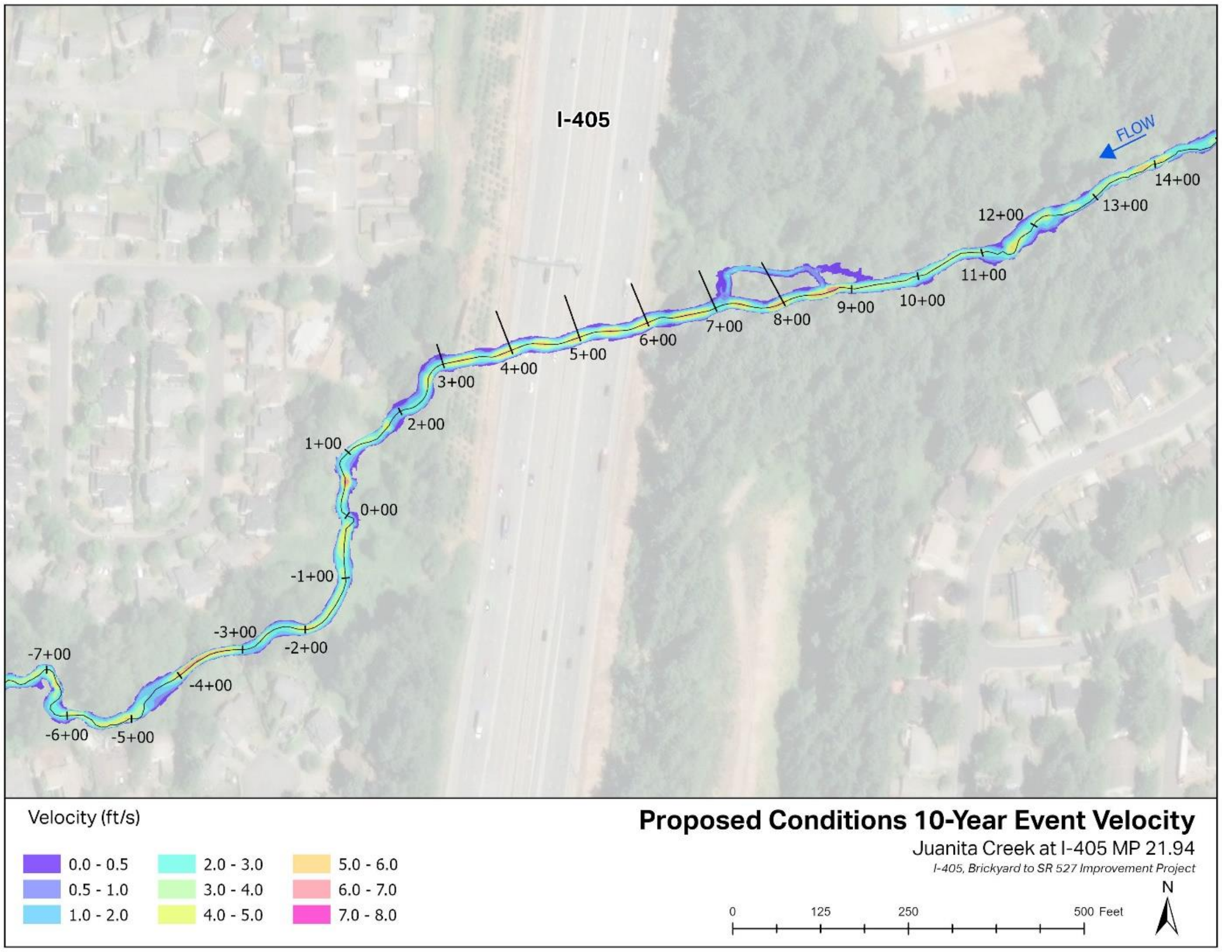
Juanita Creek

Proposed Condition – 2 Year Event Depth



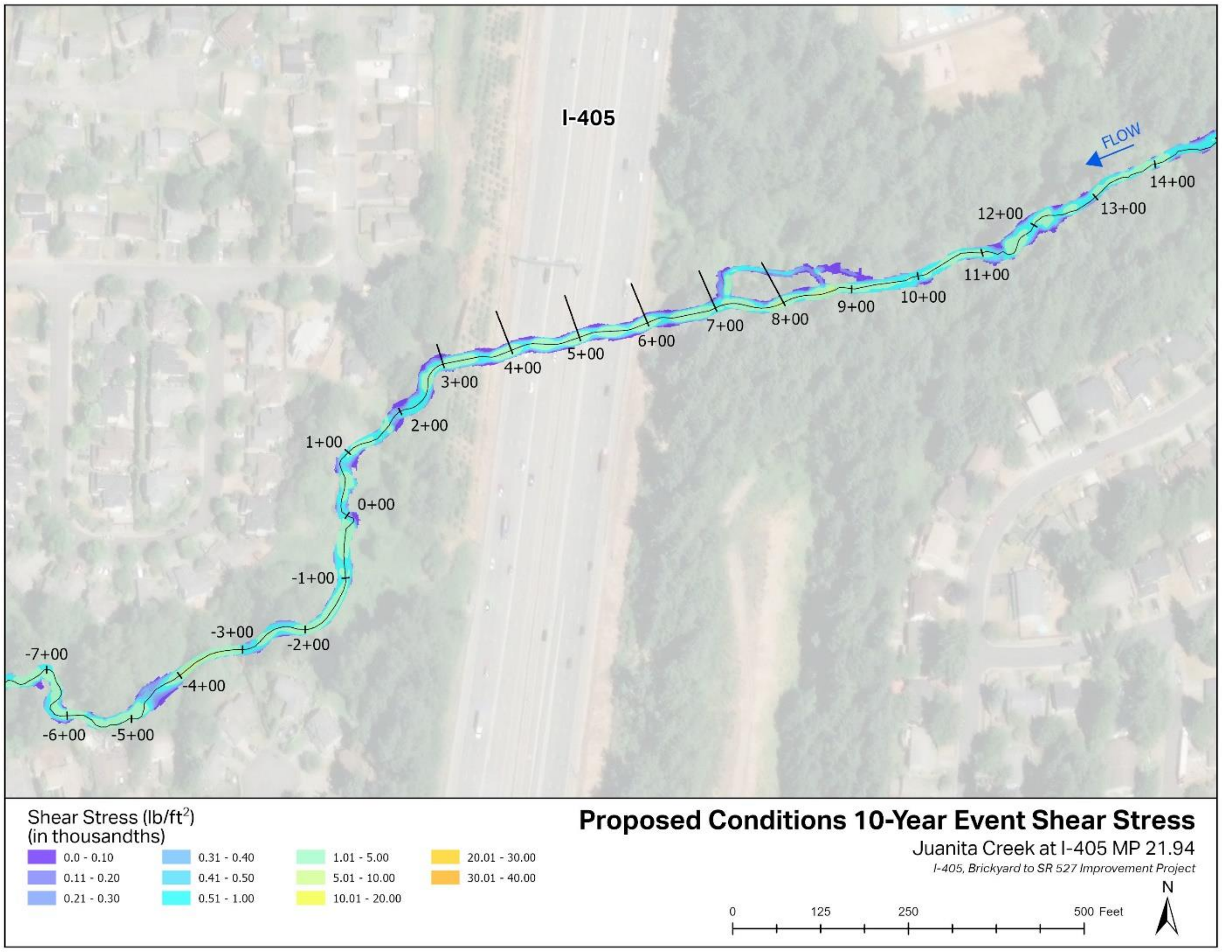
Juanita Creek

Proposed Condition – 10 Year Event WSE



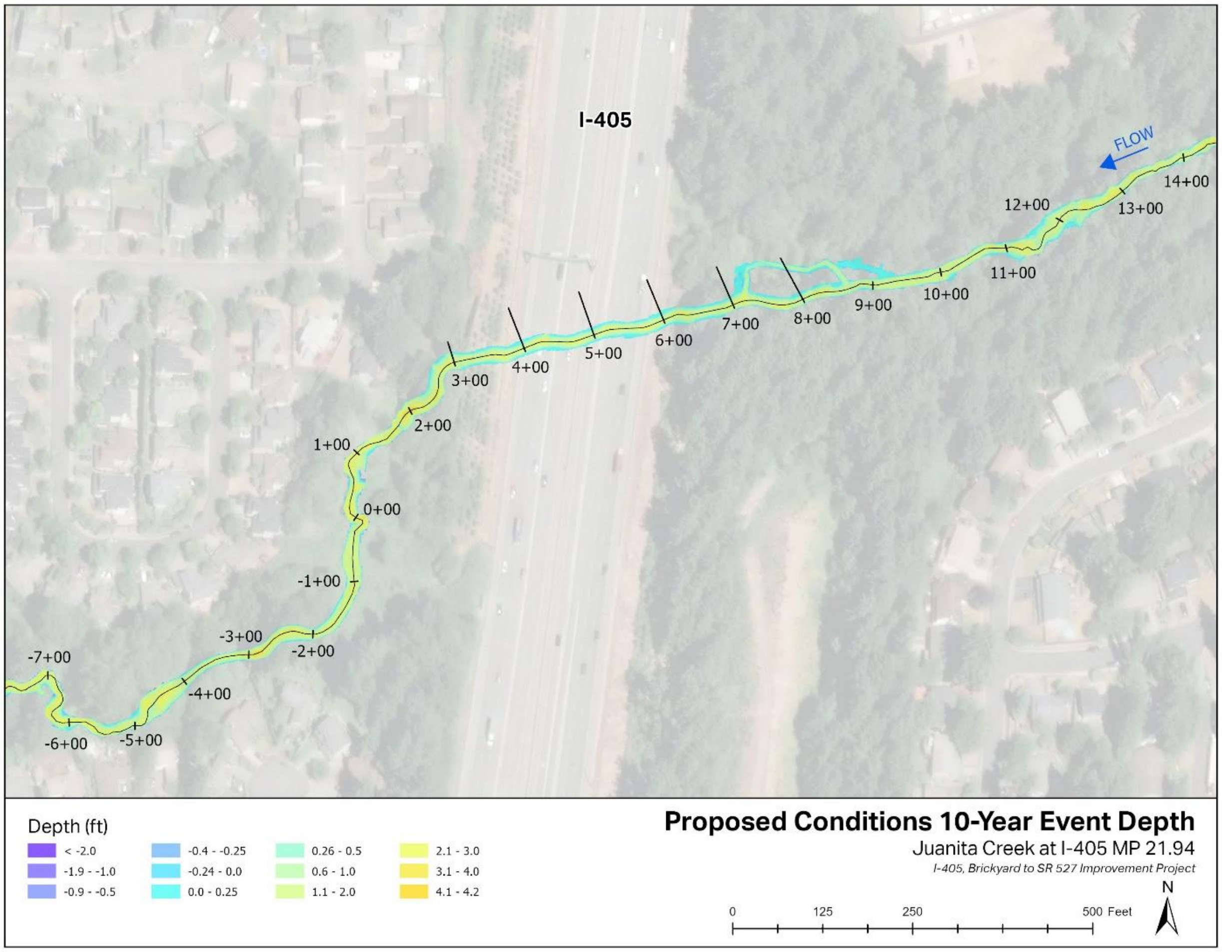
Juanita Creek

Proposed Condition – 10 Year Event Velocity



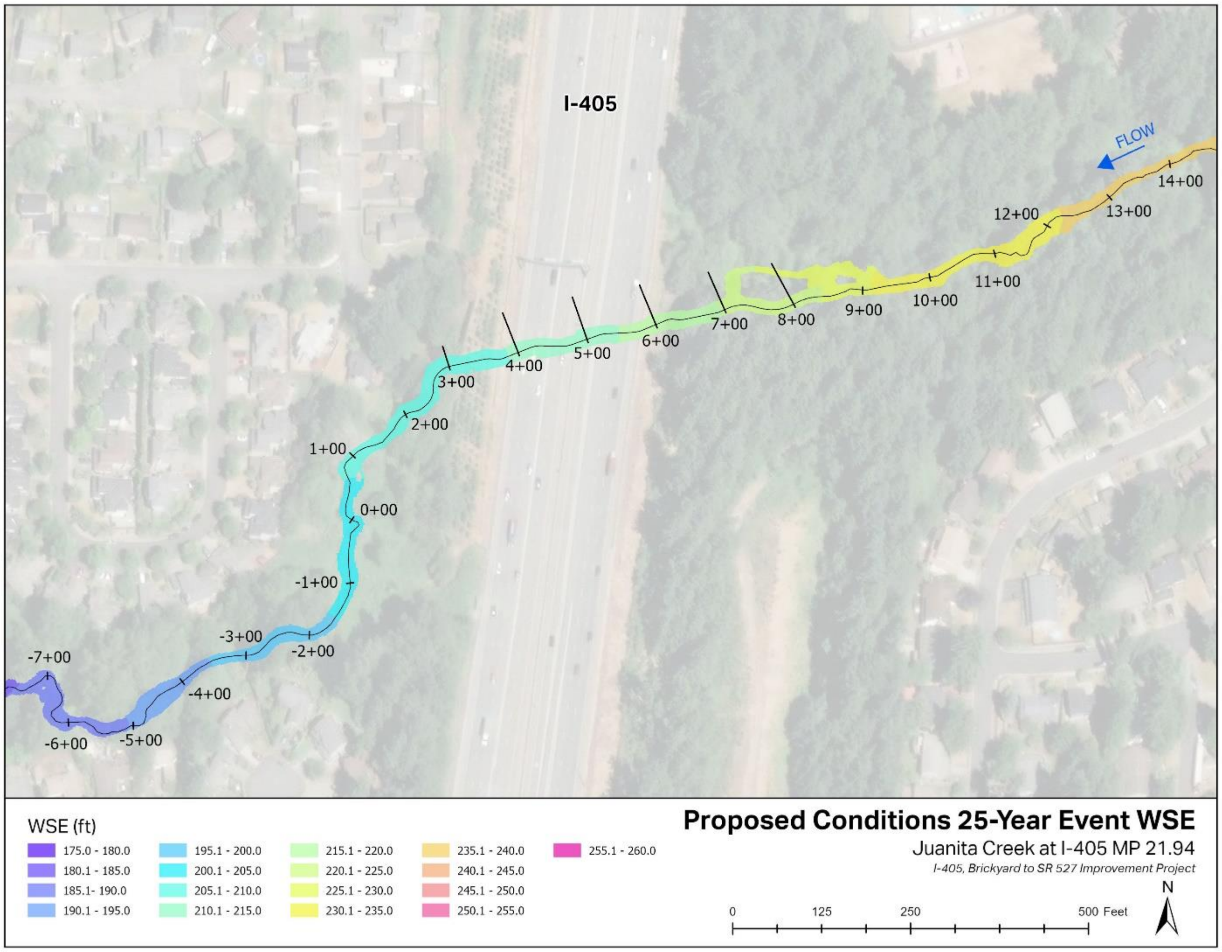
Juanita Creek

Proposed Condition – 10 Year Event Shear Stress



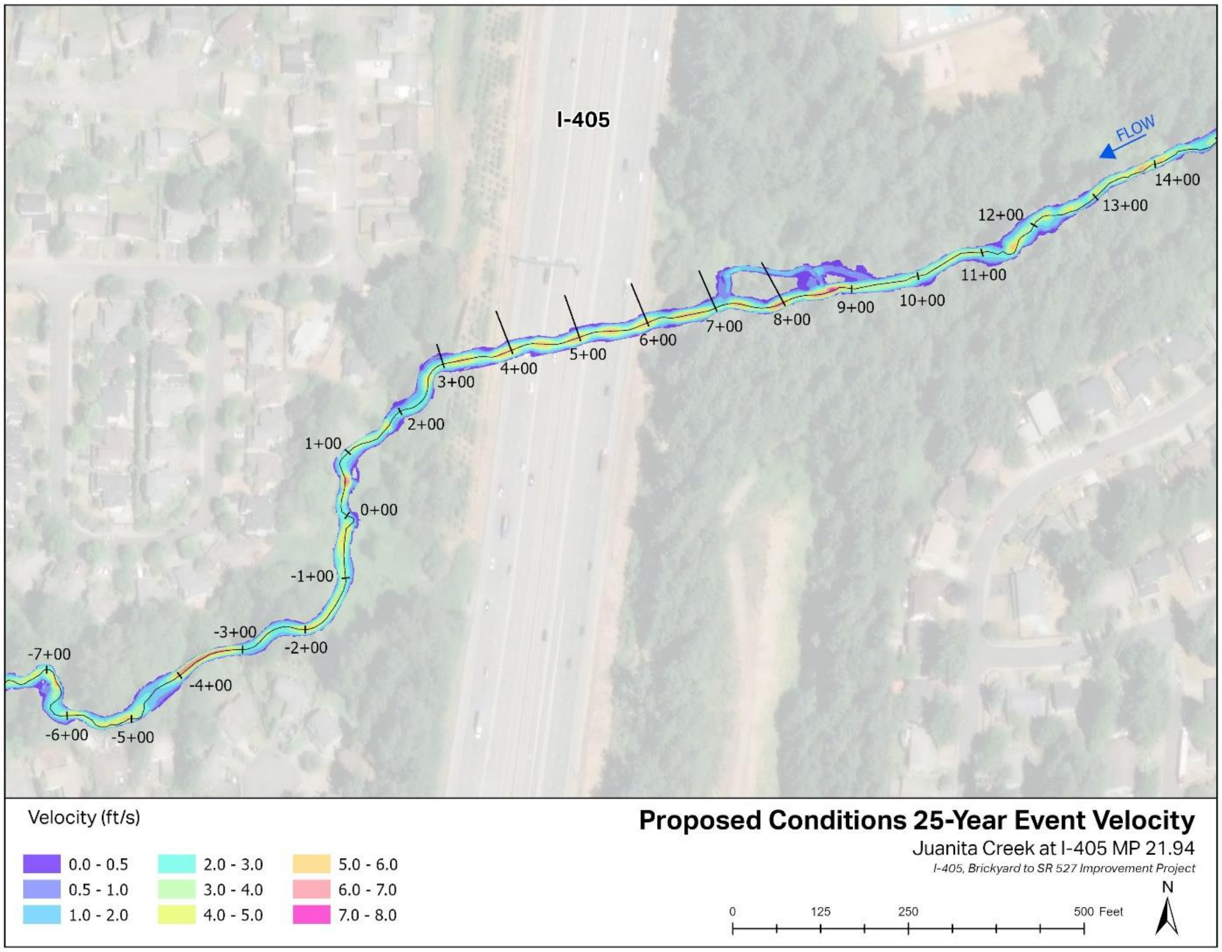
Juanita Creek

Proposed Condition – 10 Year Event Depth



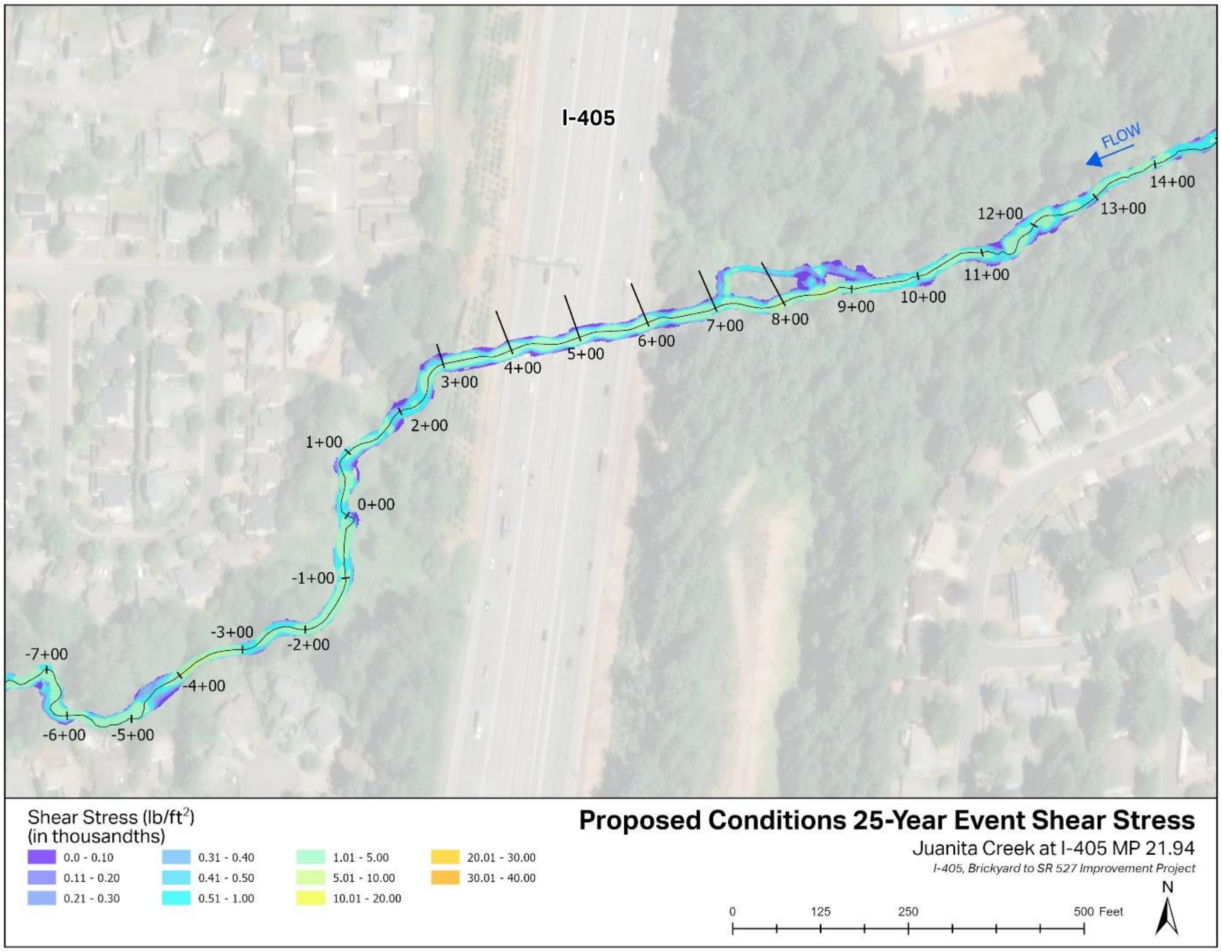
Juanita Creek

Proposed Condition – 25 Year Event WSE



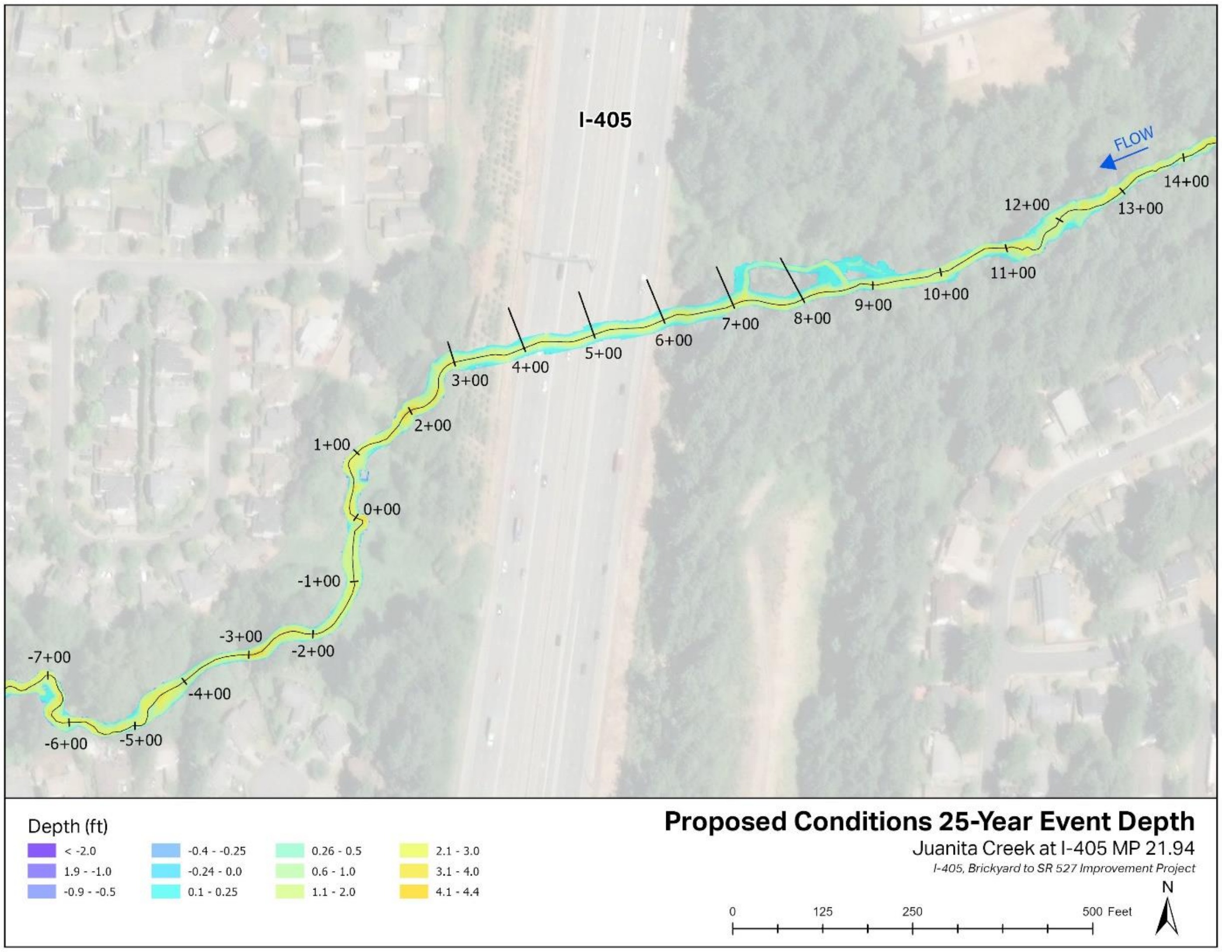
Juanita Creek

Proposed Condition – 25 Year Event Velocity



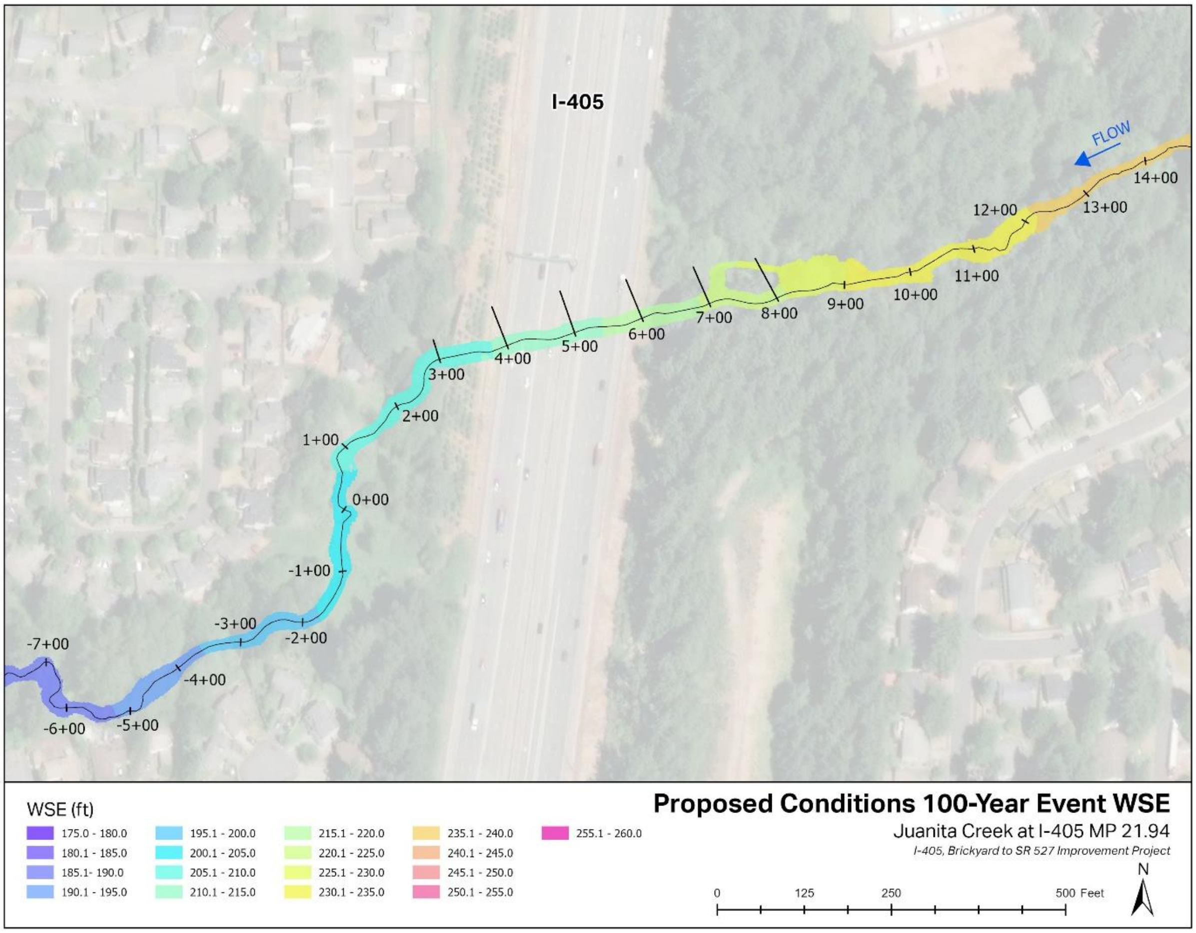
Juanita Creek

Proposed Condition – 25 Year Event Shear Stress



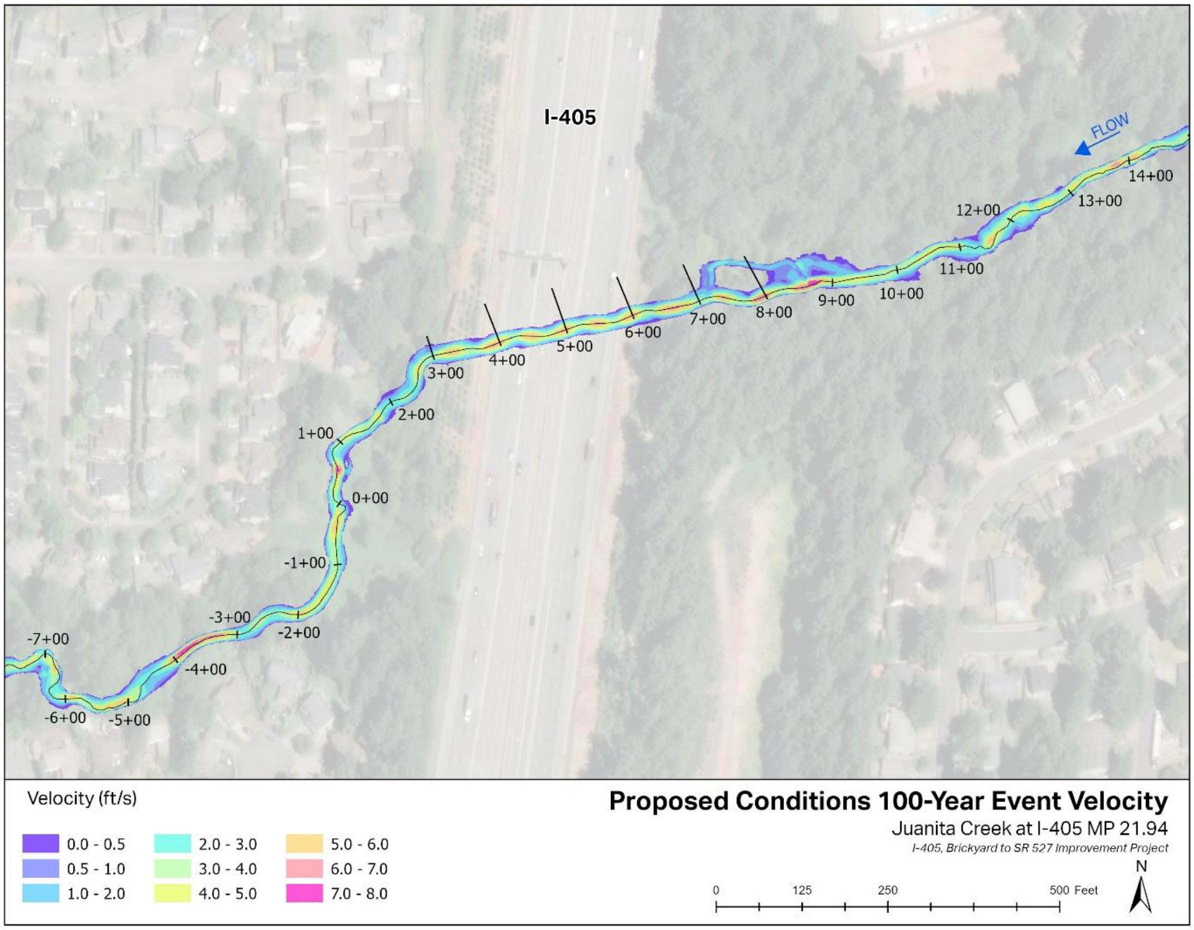
Juanita Creek

Proposed Condition – 25 Year Event Depth



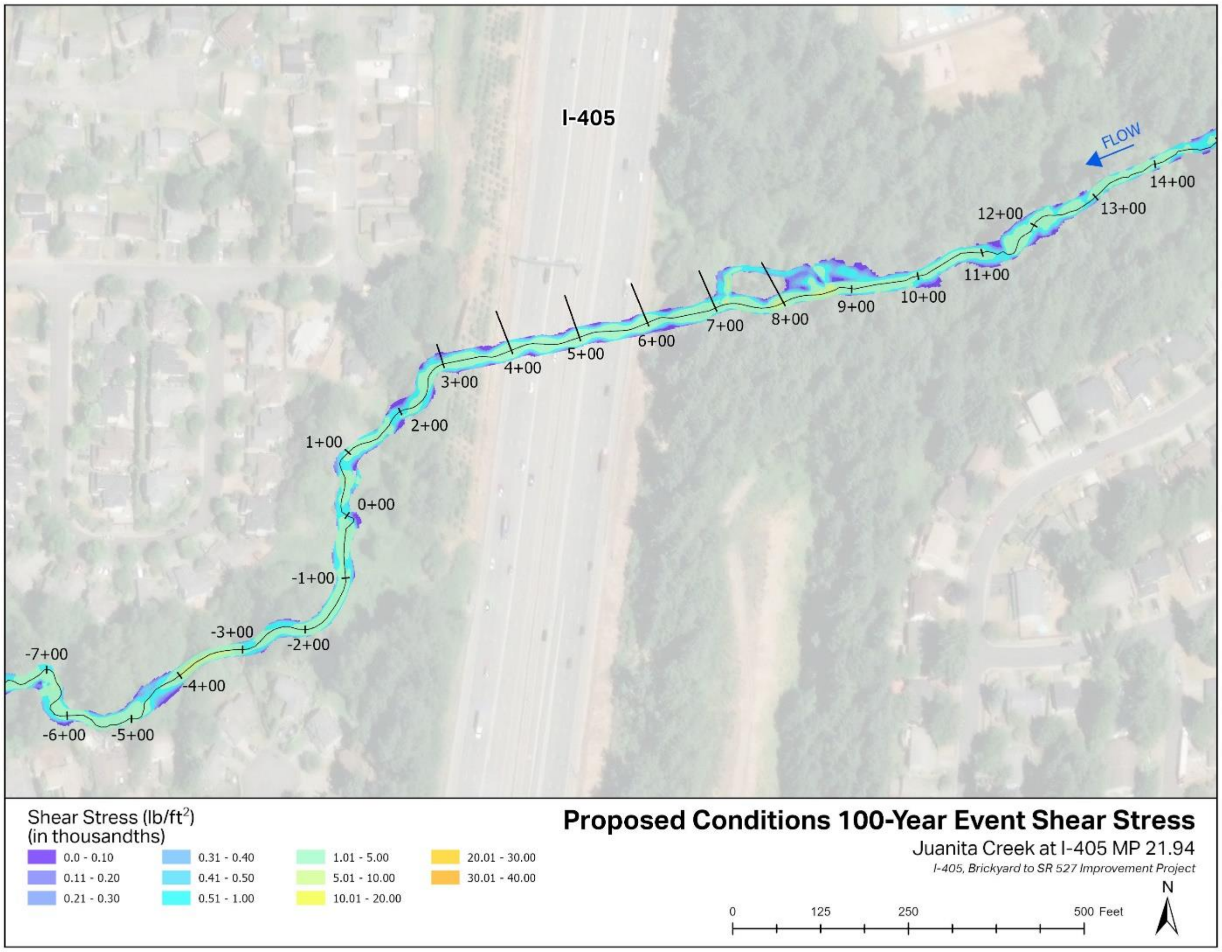
Juanita Creek

Proposed Condition – 100 Year Event WSE



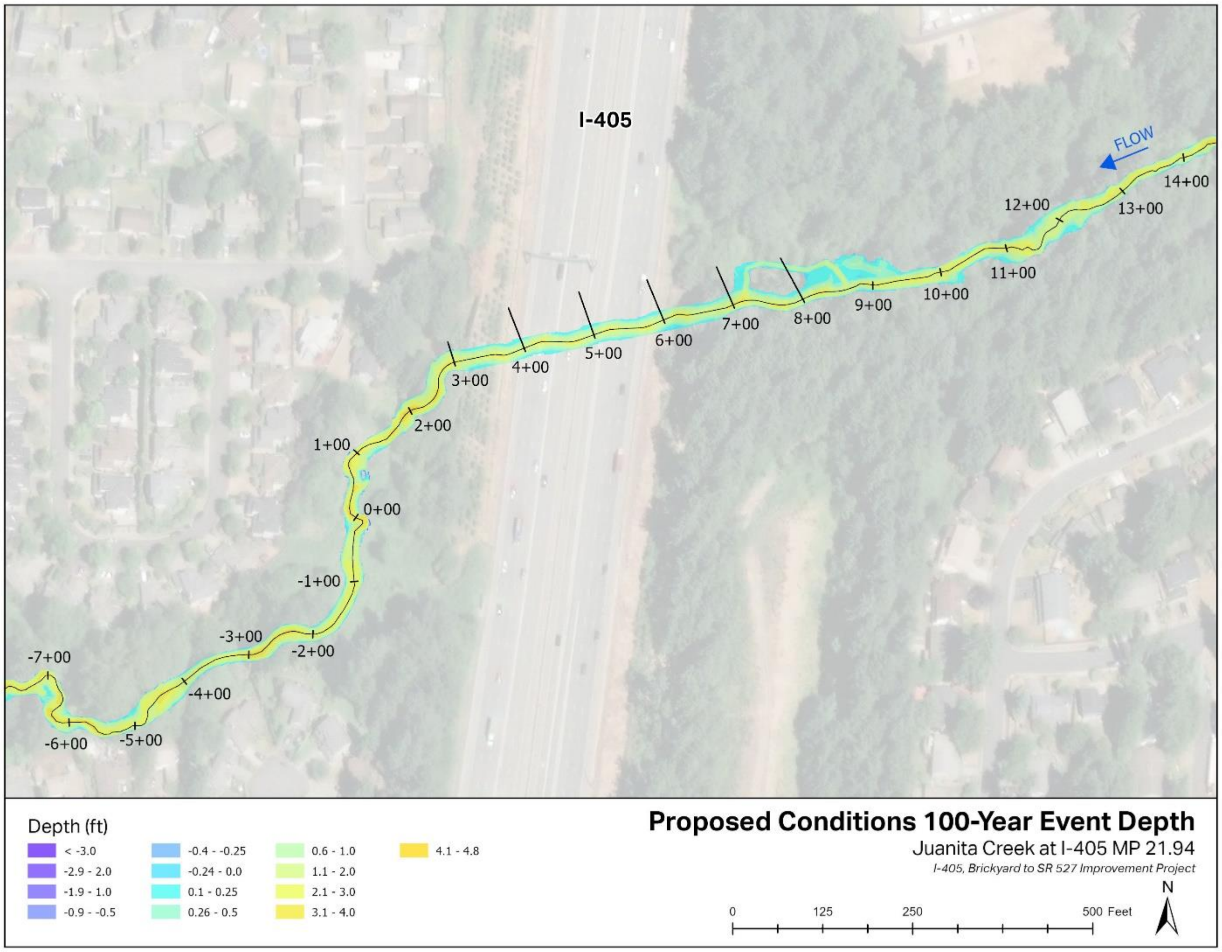
Juanita Creek

Proposed Condition – 100 Year Event Velocity



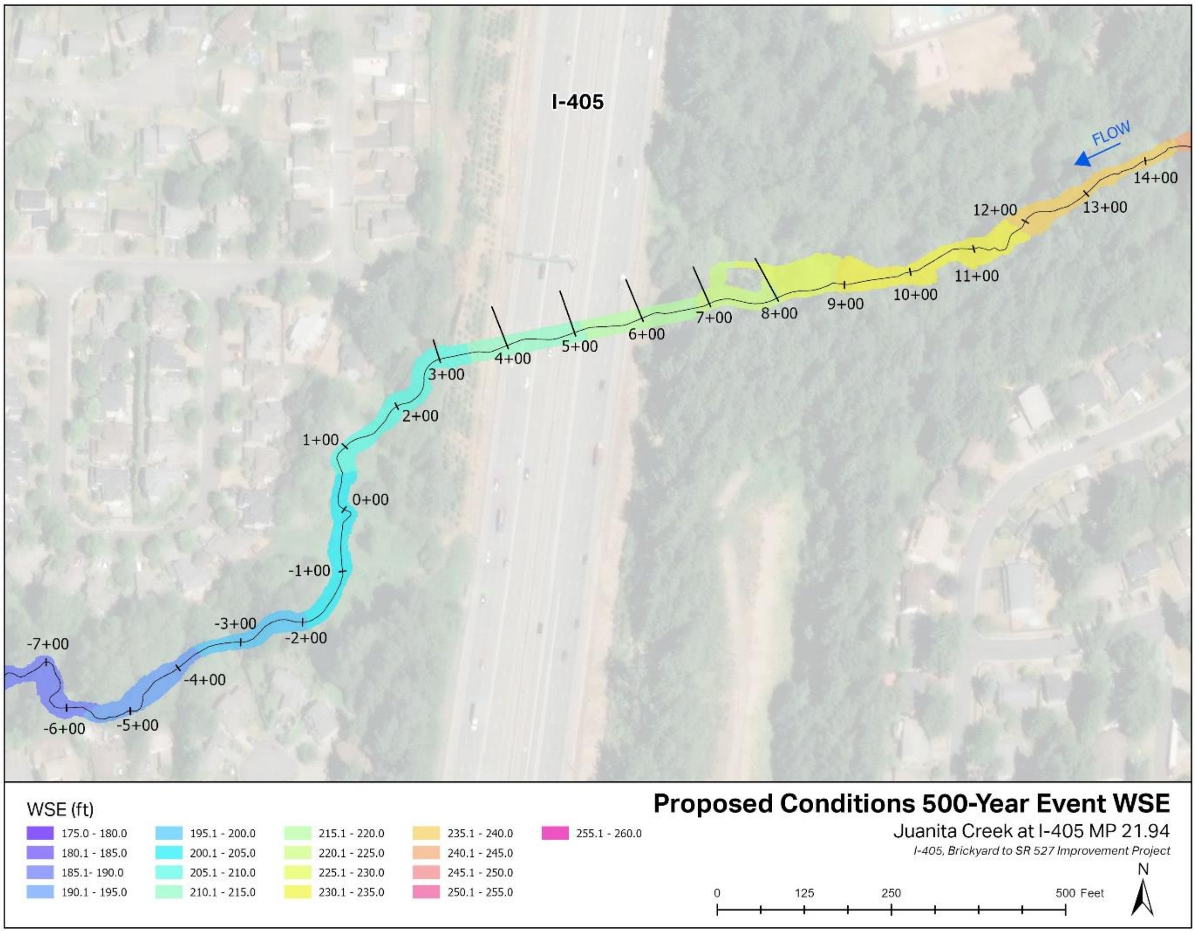
Juanita Creek

Proposed Condition – 100 Year Event Shear Stress



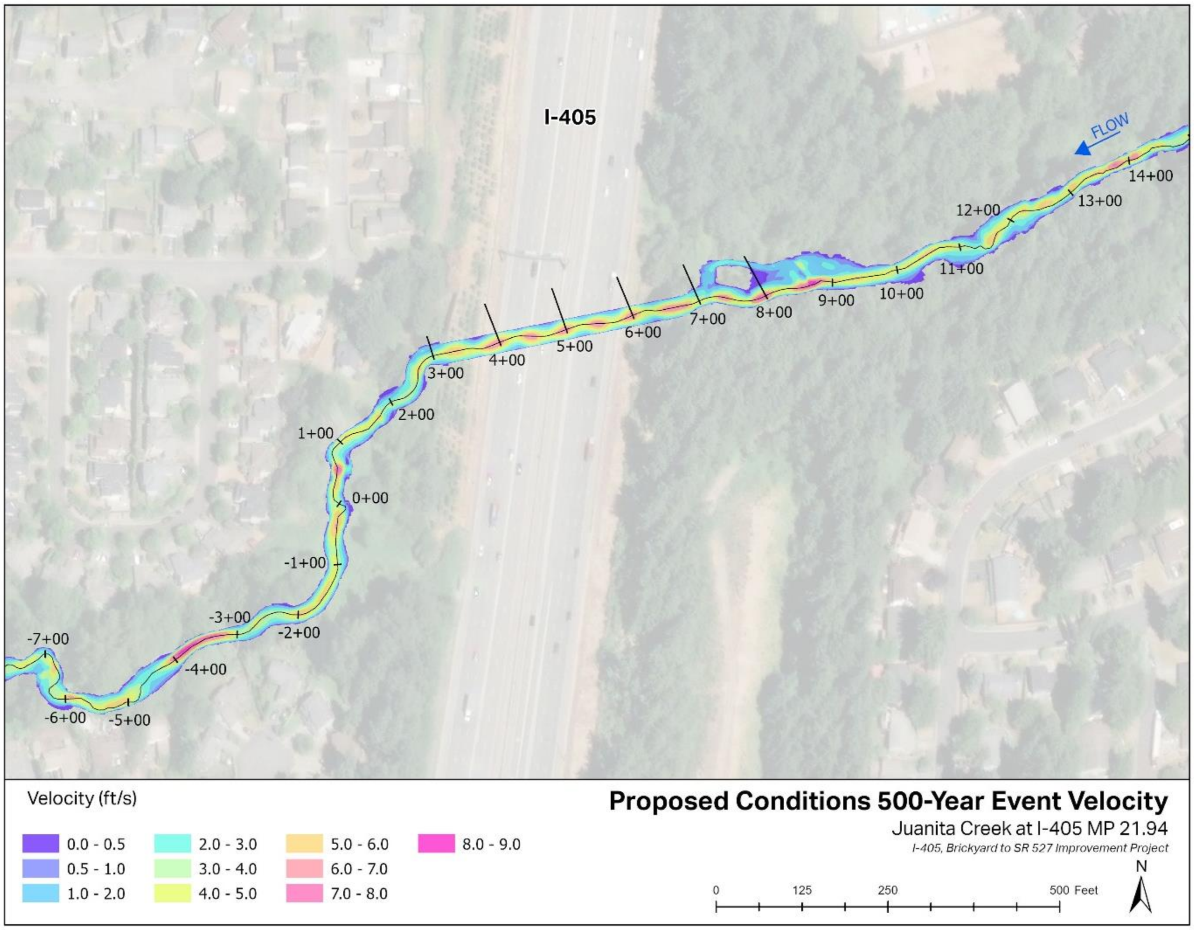
Juanita Creek

Proposed Condition – 100 Year Event Depth



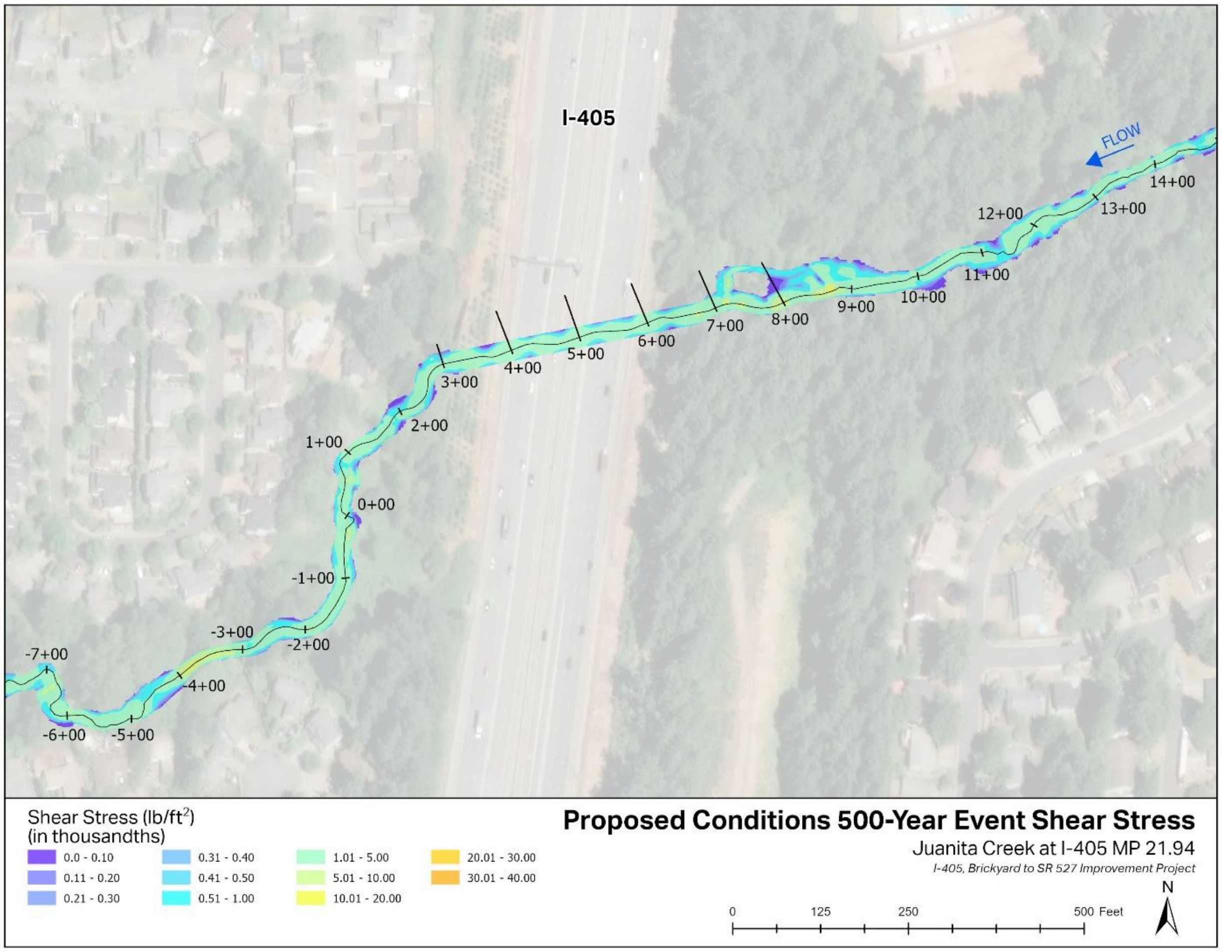
Juanita Creek

Proposed Condition – 500 Year Event WSE



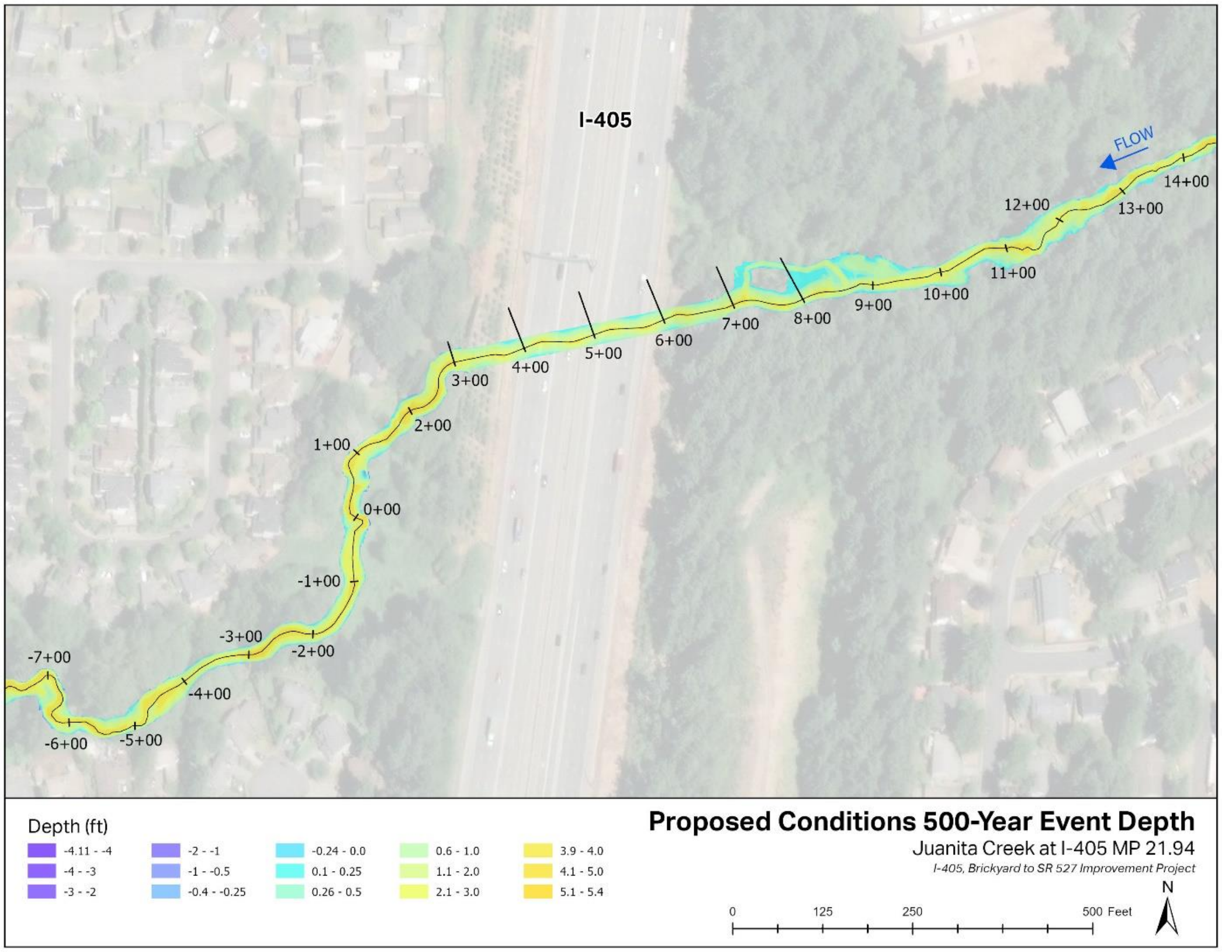
Juanita Creek

Proposed Condition – 500 Year Event Velocity



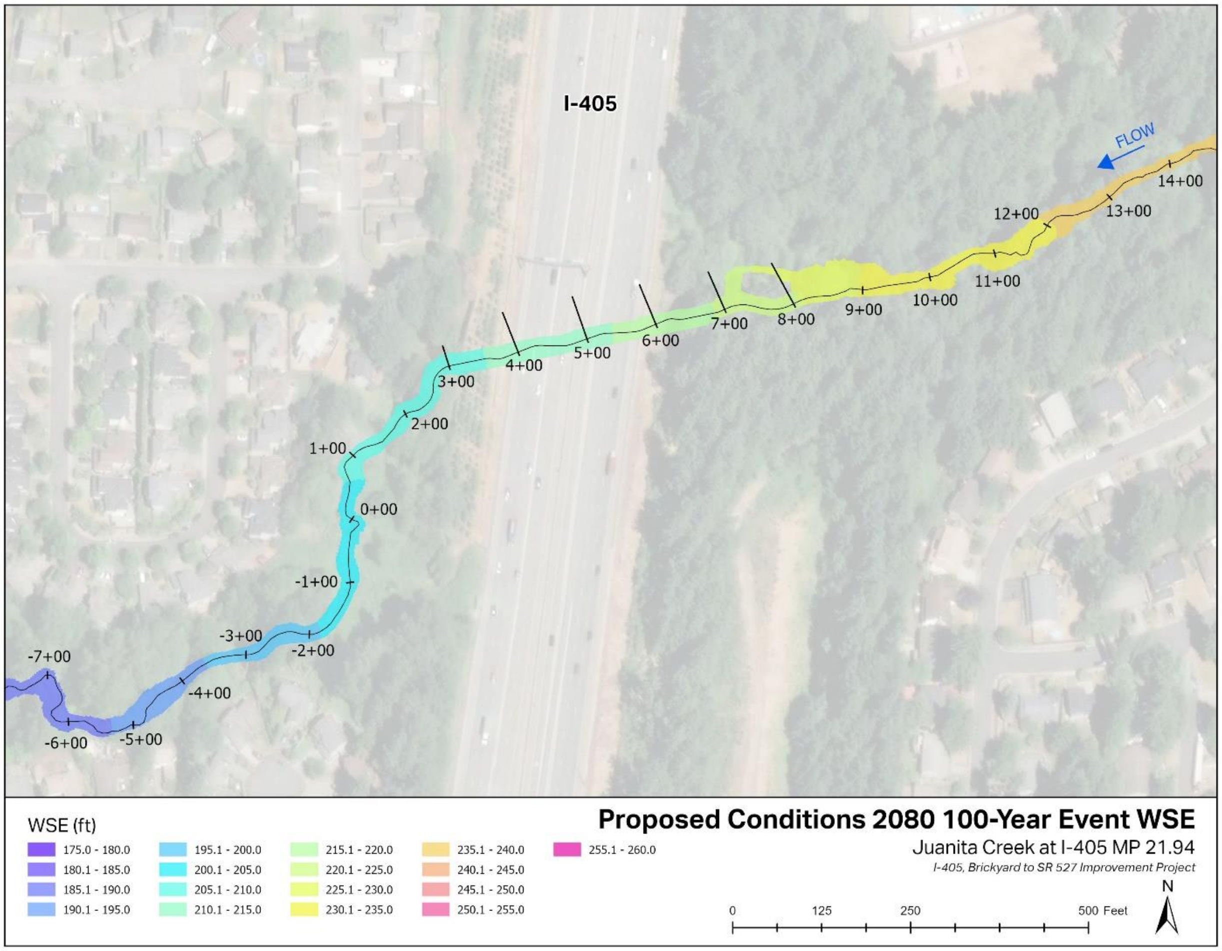
Juanita Creek

Proposed Condition – 500 Year Event Shear Stress



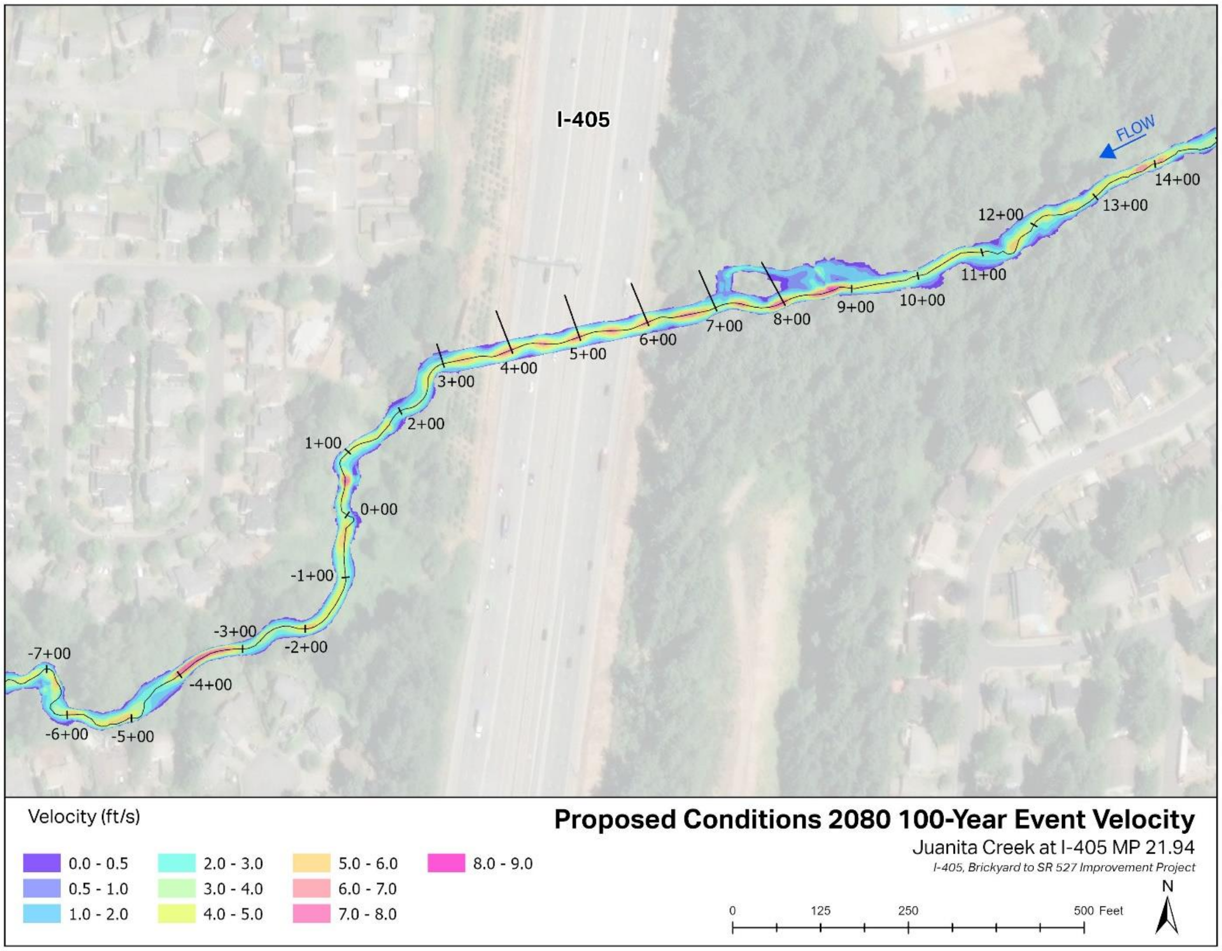
Juanita Creek

Proposed Condition – 500 Year Event Depth



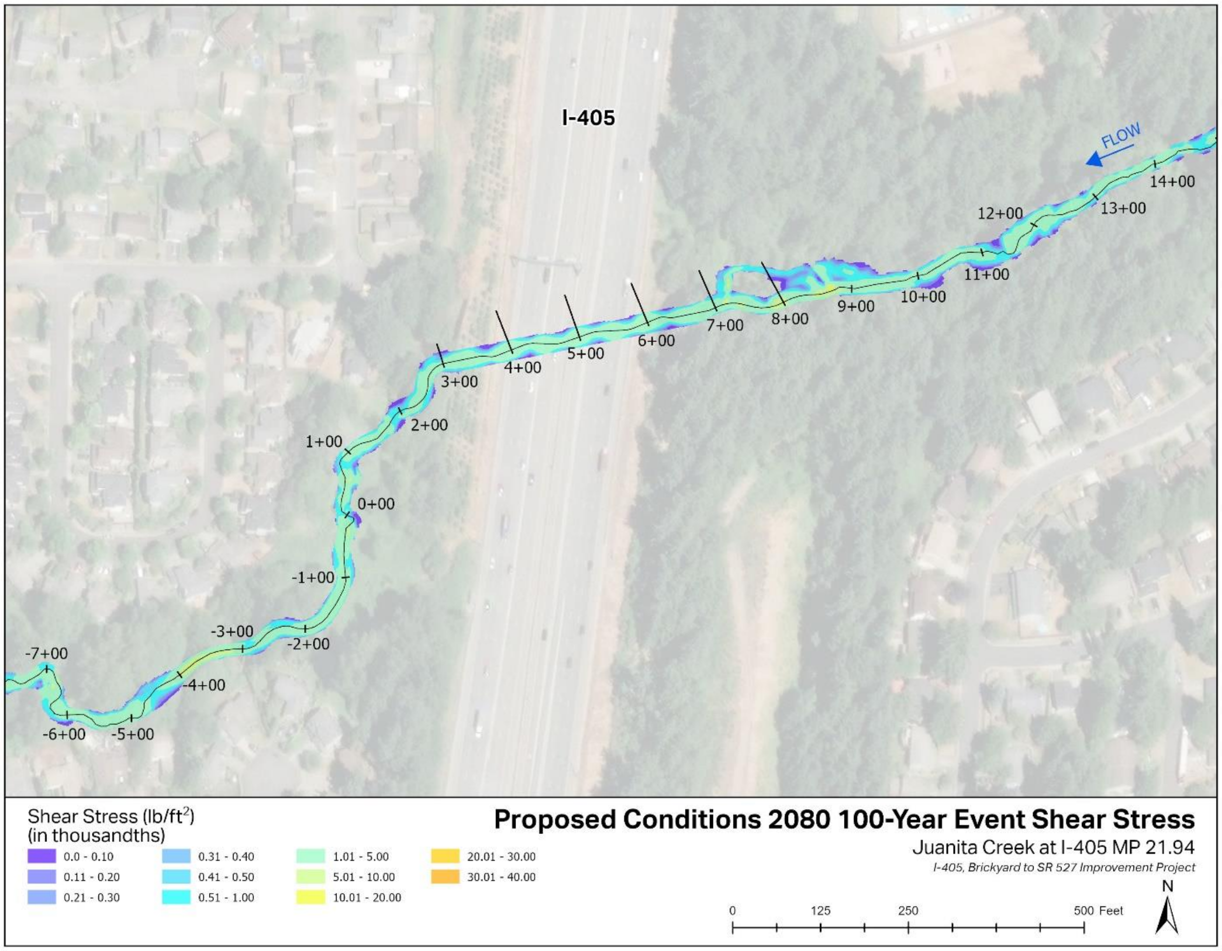
Juanita Creek

Proposed Condition – 2080 100 Year Event WSE



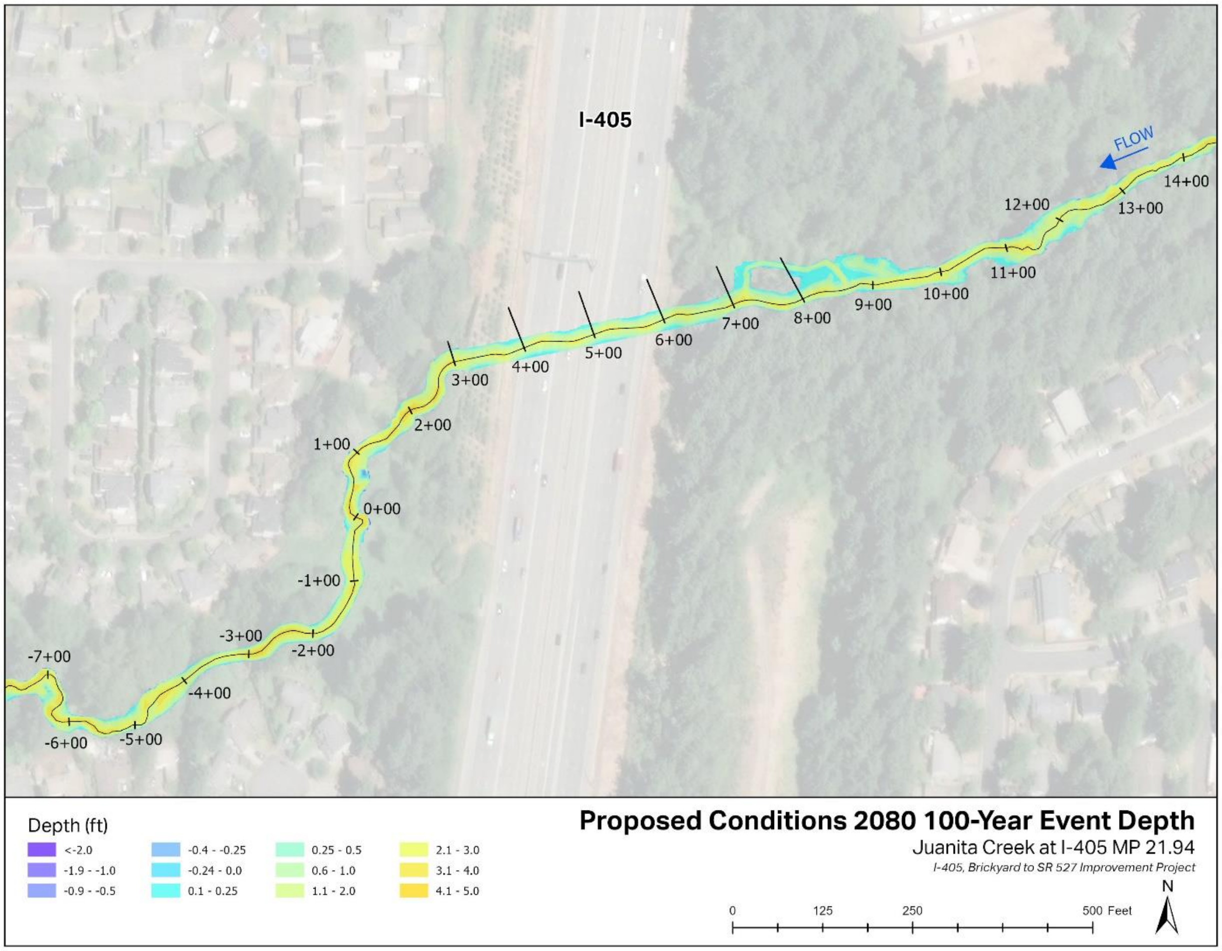
Juanita Creek

Proposed Condition – 2080 100 Year Event Velocity



Juanita Creek

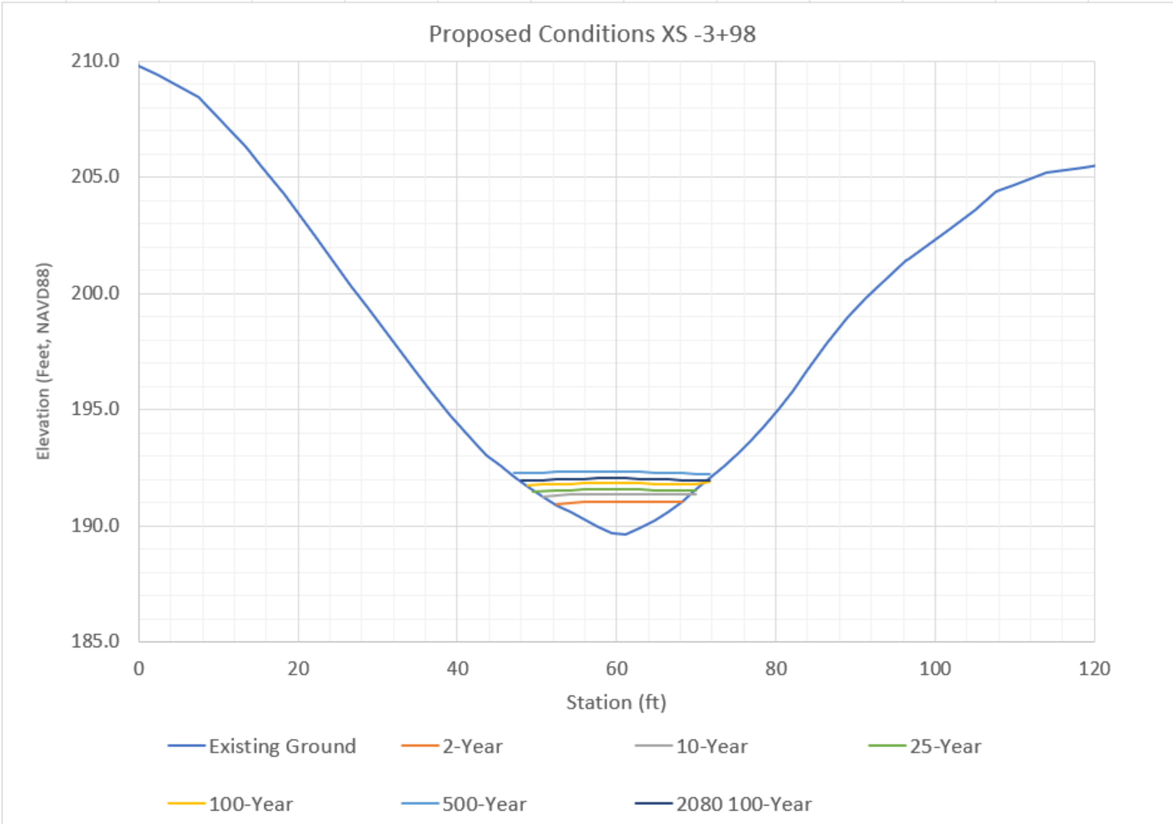
Proposed Condition – 2080 100 Year Event Shear Stress

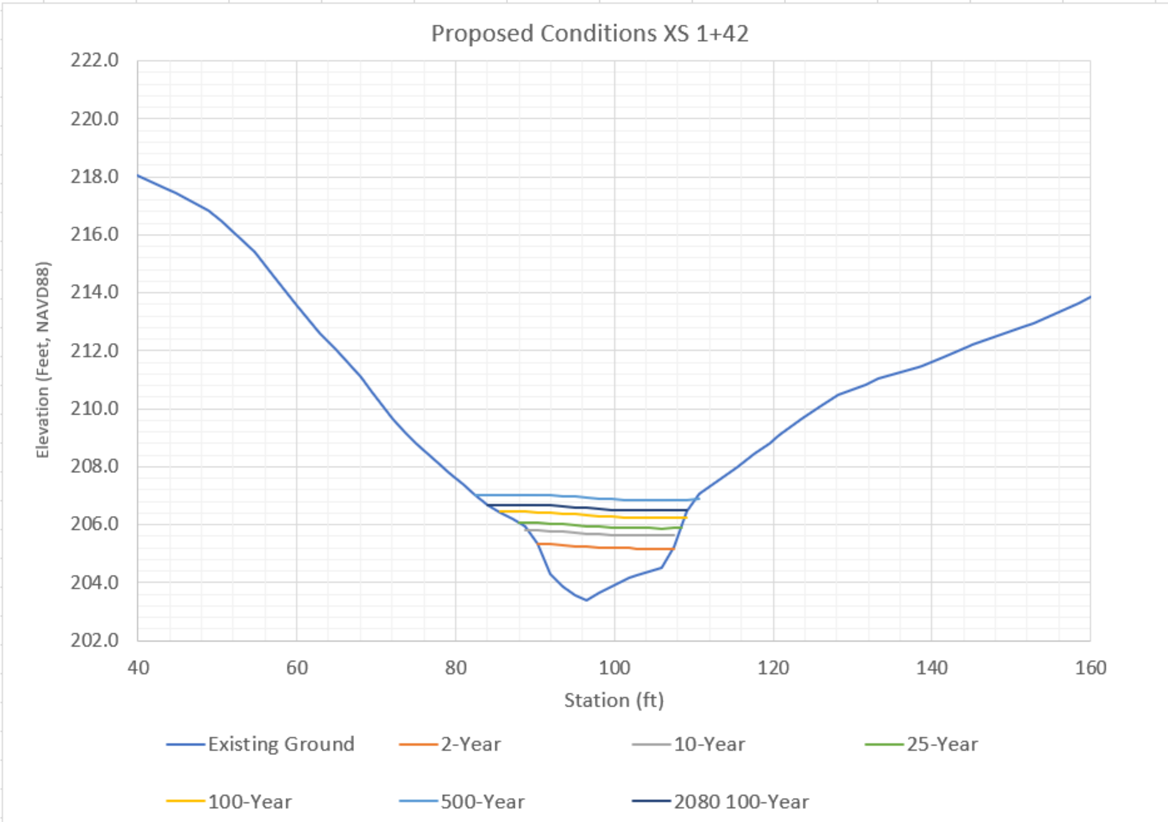


Juanita Creek

Proposed Condition – 2080 100 Year Event Depth

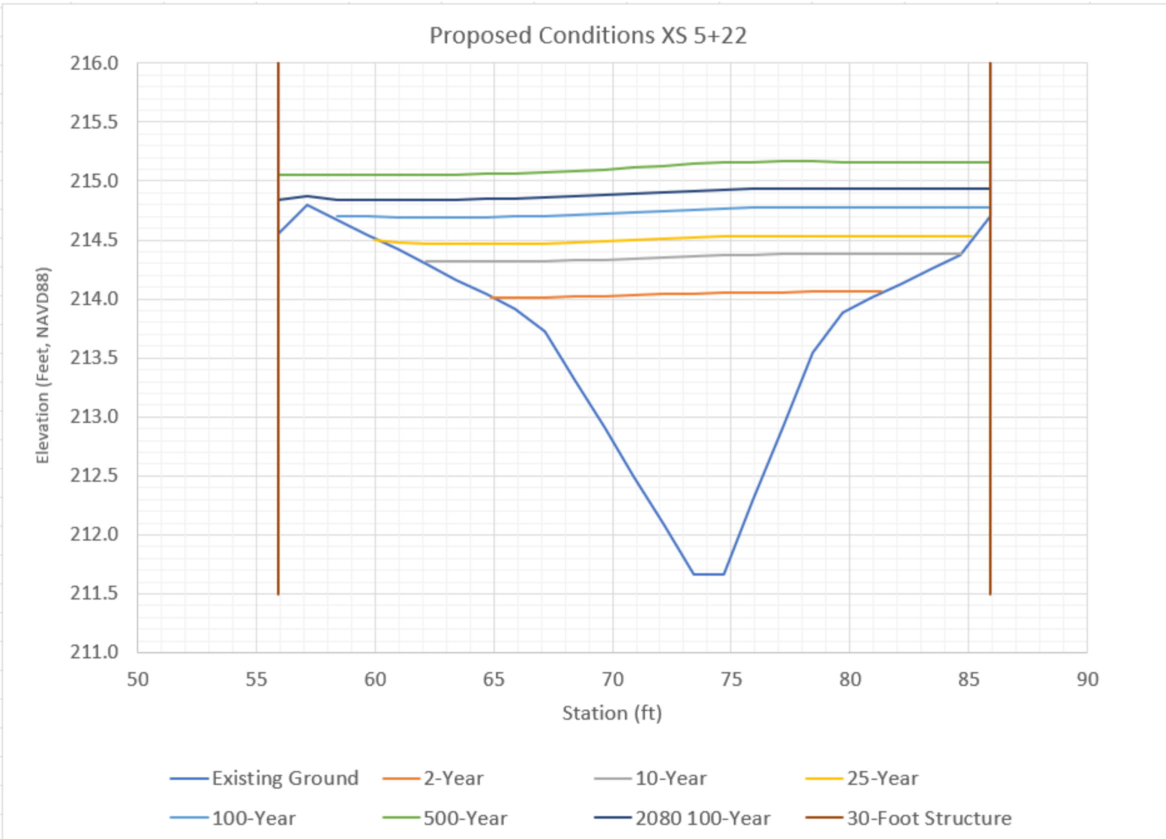
Note: All cross sections are looking downstream



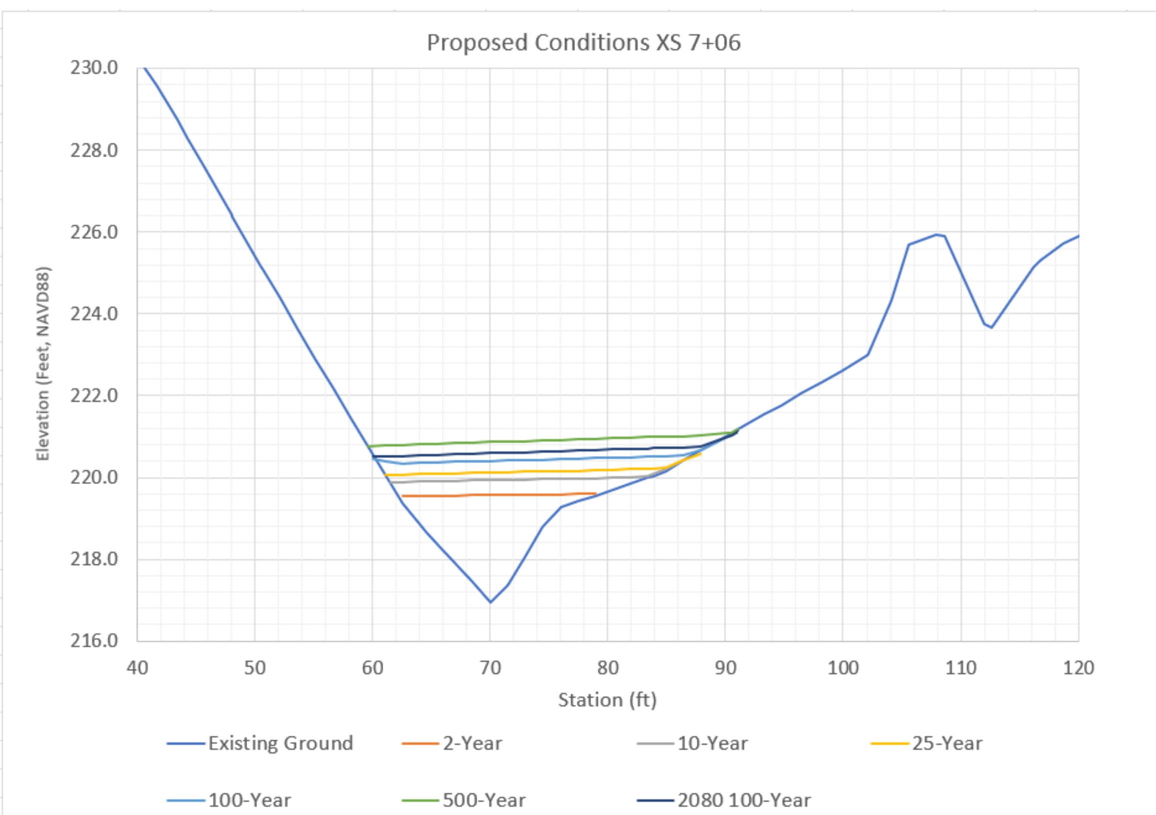


Proposed Conditions XS 1+42
Elevation (Feet, NAVD88)
222.0
220.0
218.0
216.0
214.0
212.0
210.0
208.0
206.0
204.0
202.0
40
60
80
100
120
140
160
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year

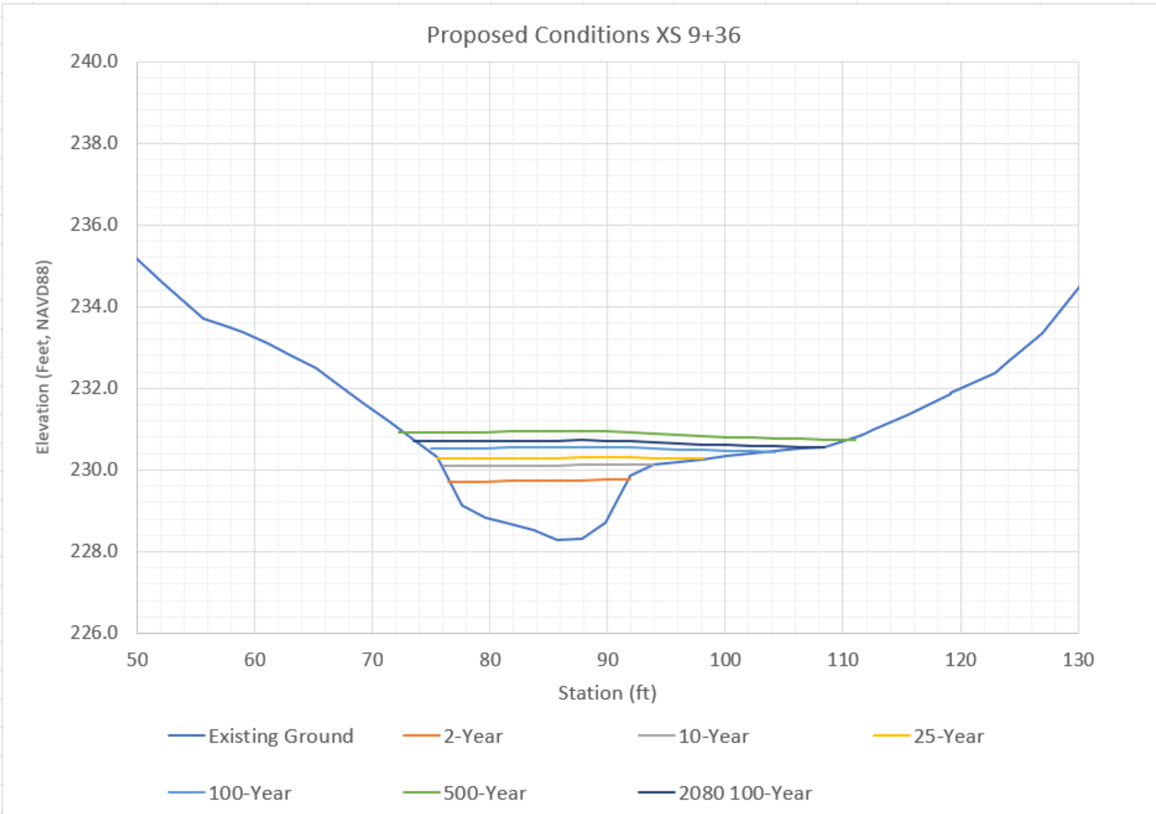
Proposed Conditions XS 2+98
Elevation (Feet, NAVD88)
230.0
225.0
220.0
215.0
210.0
205.0
200.0
0
20
40
60
80
100
120
140
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year



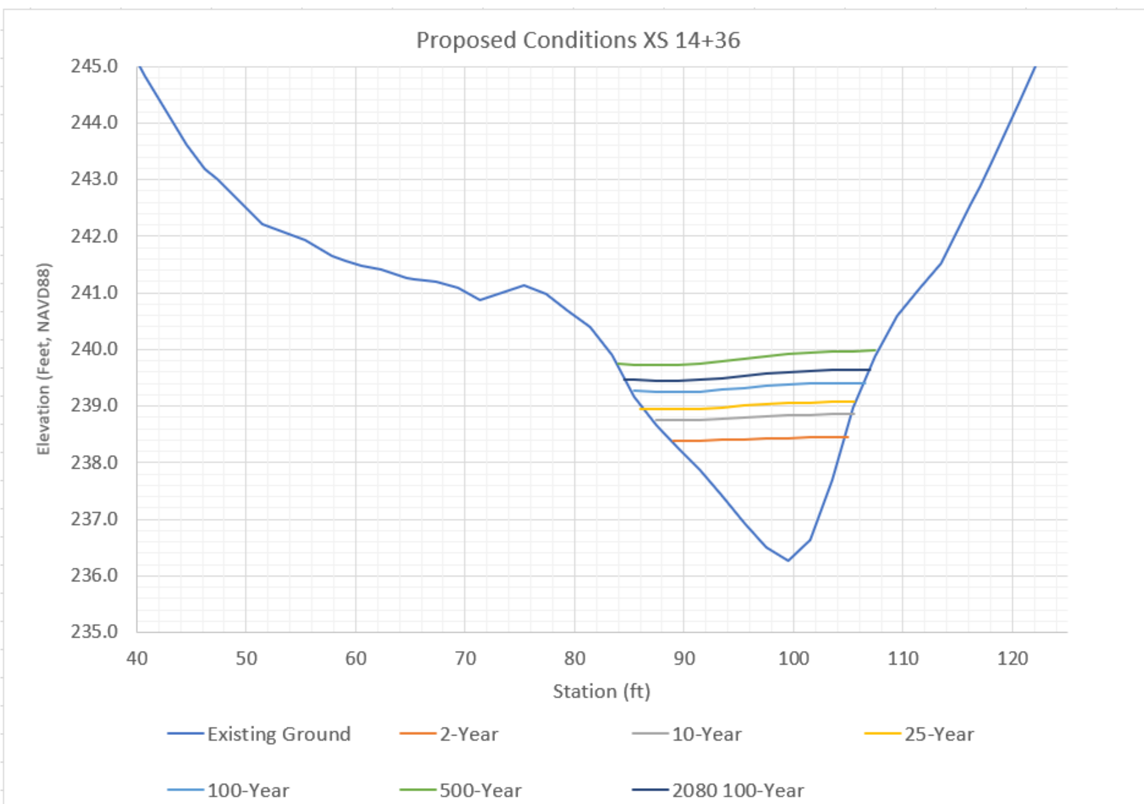
Proposed Conditions XS 5+22
Elevation (Feet, NAVD88)
216.0
215.5
215.0
214.5
214.0
213.5
213.0
212.5
212.0
211.5
211.0
50
55
60
65
70
75
80
85
90
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year
30-Foot Structure



Proposed Conditions XS 7+06
Elevation (Feet, NAVD88)
230.0
228.0
226.0
224.0
222.0
220.0
218.0
216.0
40
50
60
70
80
90
100
110
120
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year

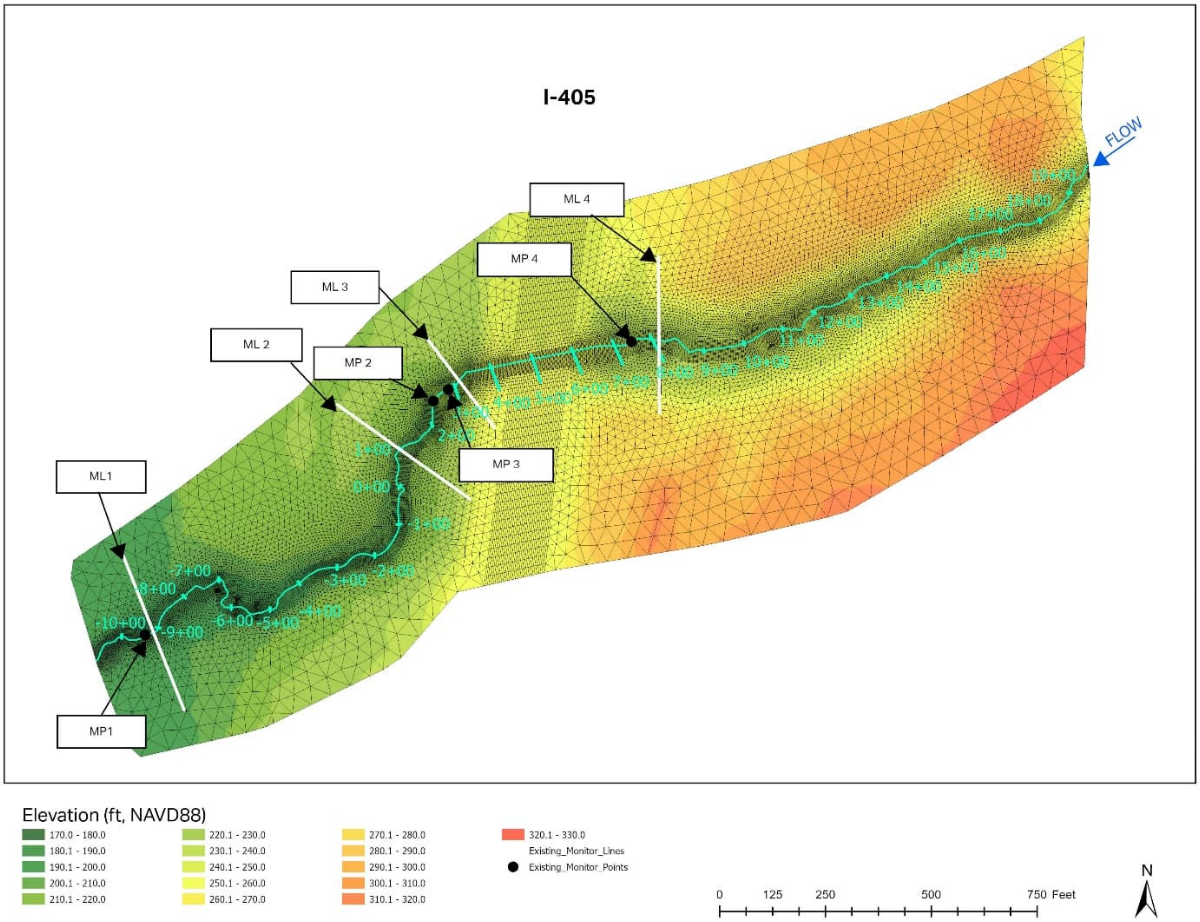


Proposed Conditions XS 9+36
240.0
238.0
236.0
234.0
232.0
230.0
228.0
226.0
Elevation (Feet, NAVD88)
50
60
70
80
90
100
110
120
130
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year



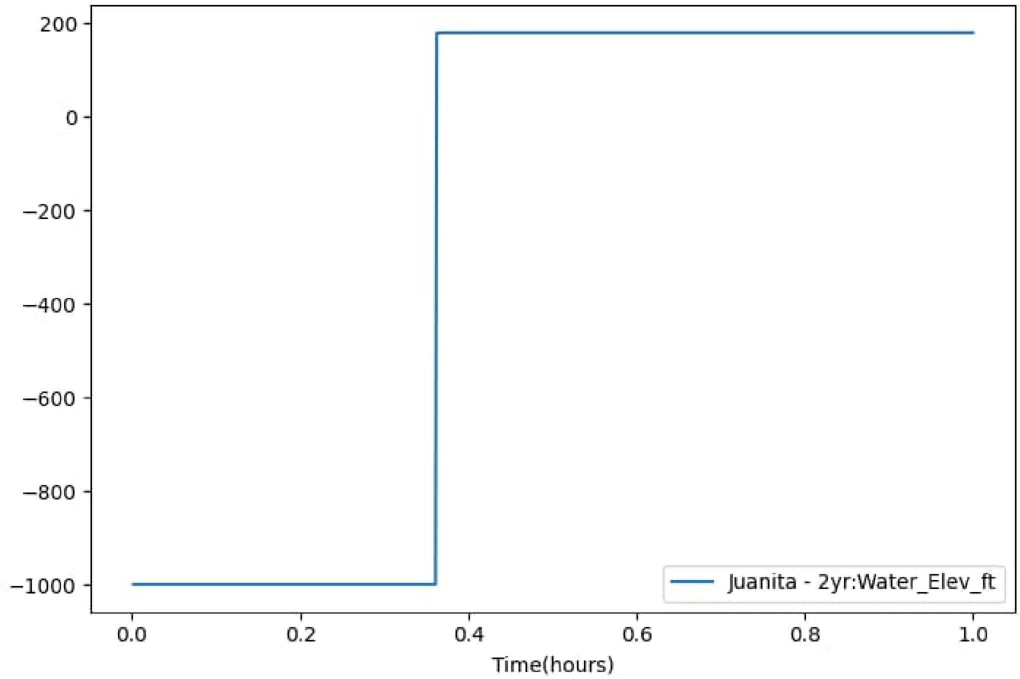
Proposed Conditions XS 14+36
245.0
244.0
243.0
242.0
241.0
240.0
239.0
238.0
237.0
236.0
235.0
Elevation (Feet, NAVD88)
40
50
60
70
80
90
100
110
120
Station (ft)
Existing Ground
2-Year
10-Year
25-Year
100-Year
500-Year
2080 100-Year

Appendix I: SRH-2D Model Stability and Continuity

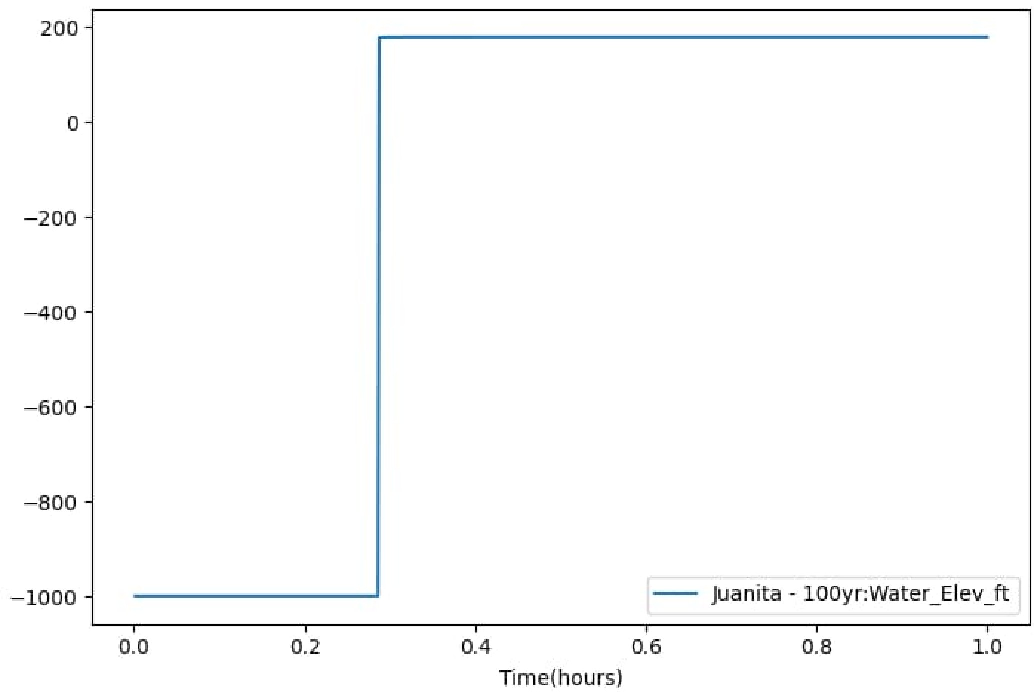


Existing Conditions SMS Monitoring Point WSE Plots

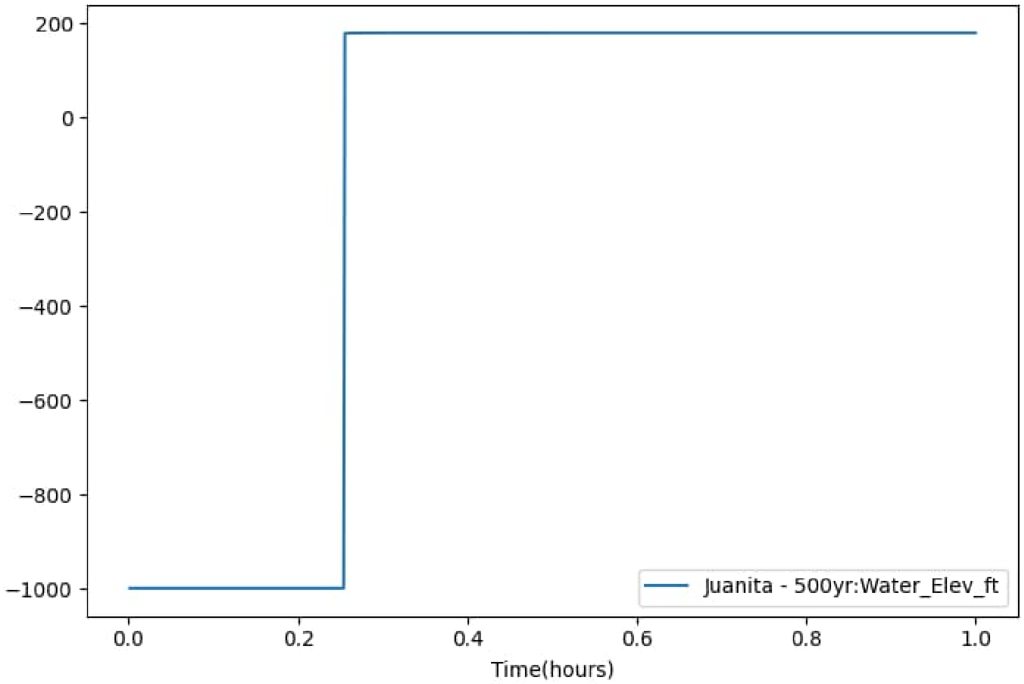
Note: MP2 and ML2 are downstream of the access road culvert and MP3 and ML3 are in between the access road culvert and the I-405 culvert. This area is also near the internal sink boundary condition. The complexity of the model in this region creates some oscillating flow plots at ML 3, but WSEs and other results are reasonable and stable in all areas of the model.



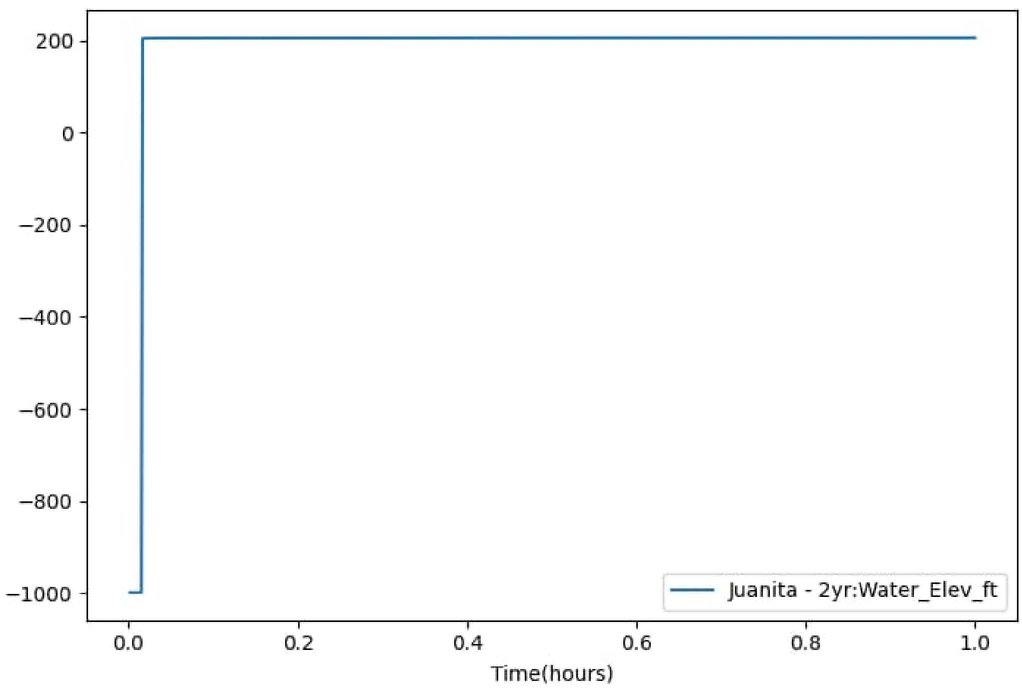
Existing Conditions Monitor Point 1 (MP 1) 2-Year WSE Plot



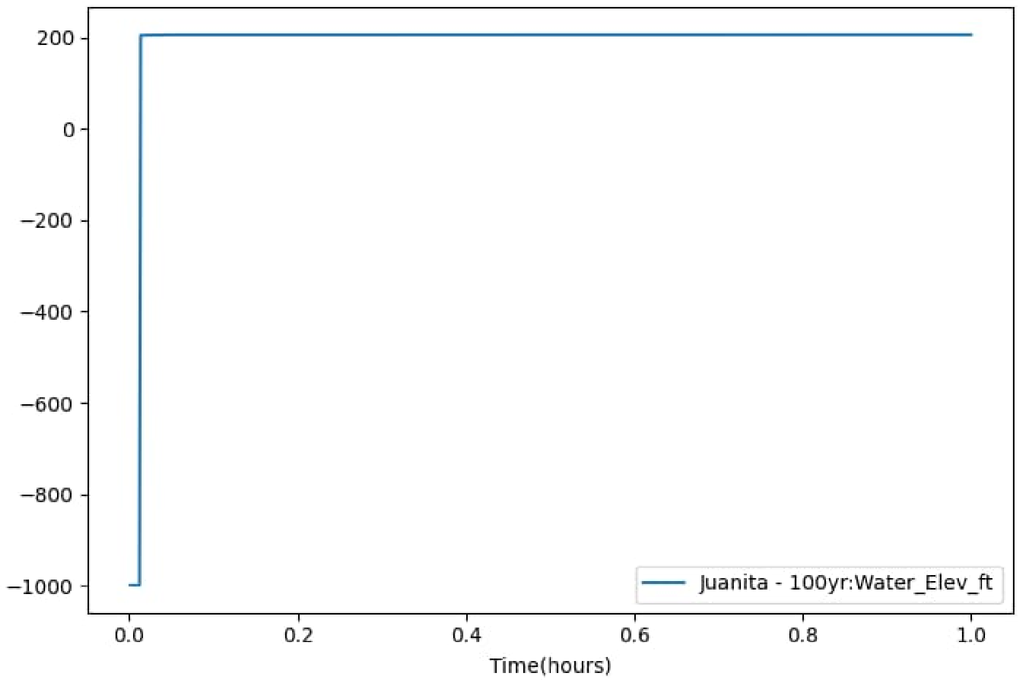
Existing Conditions Monitor Point 1 (MP 1) 100-yr WSE Plot



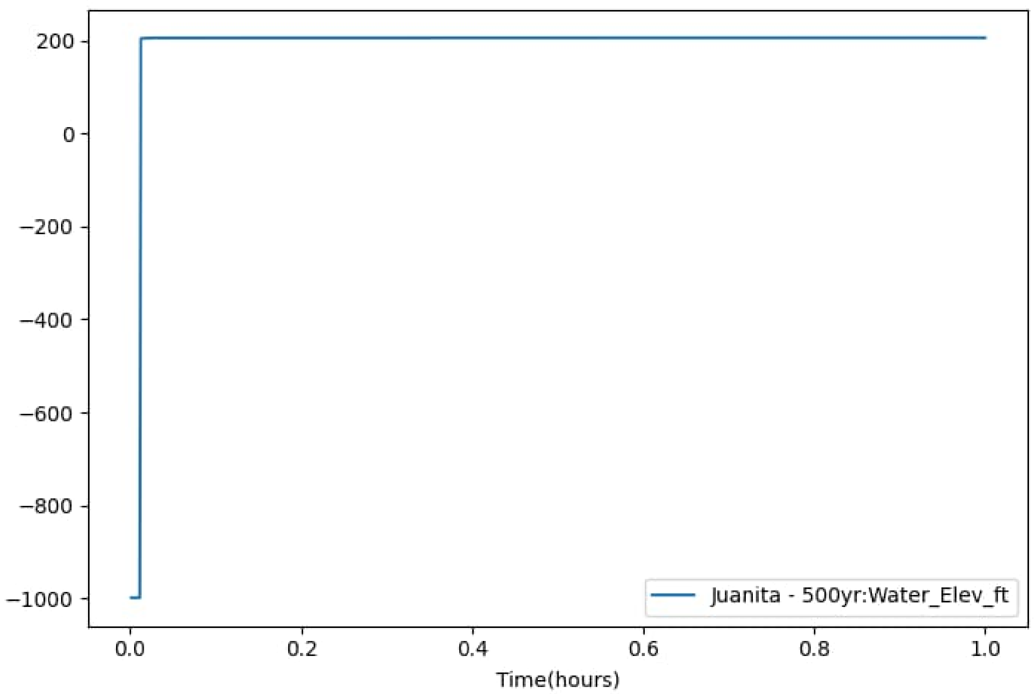
Existing Conditions Monitor Point 1 (MP 1) 500-Year WSE Plot



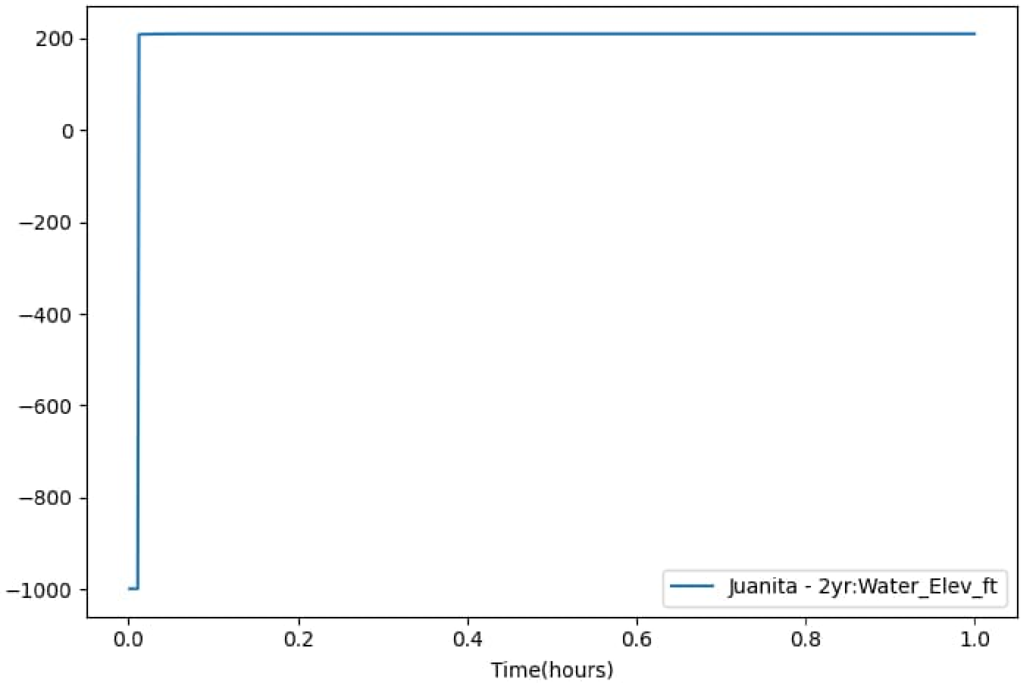
Existing Conditions Monitor Point 2 (MP2) 2-Year WSE Plot



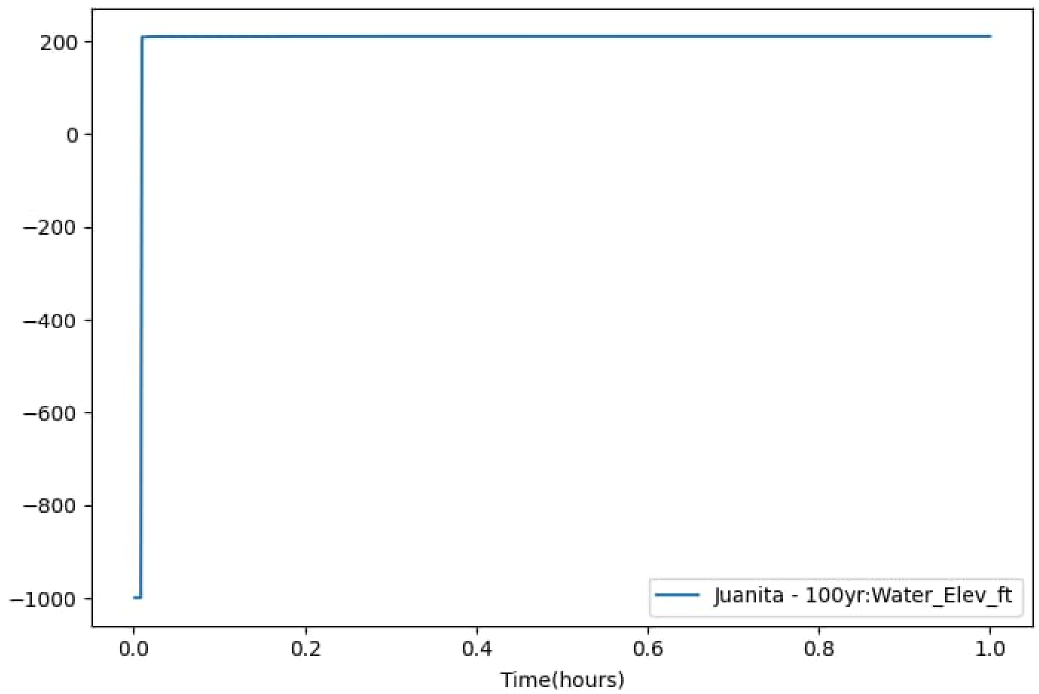
Existing Conditions Monitor Point 2 (MP2) 100-Year WSE Plot



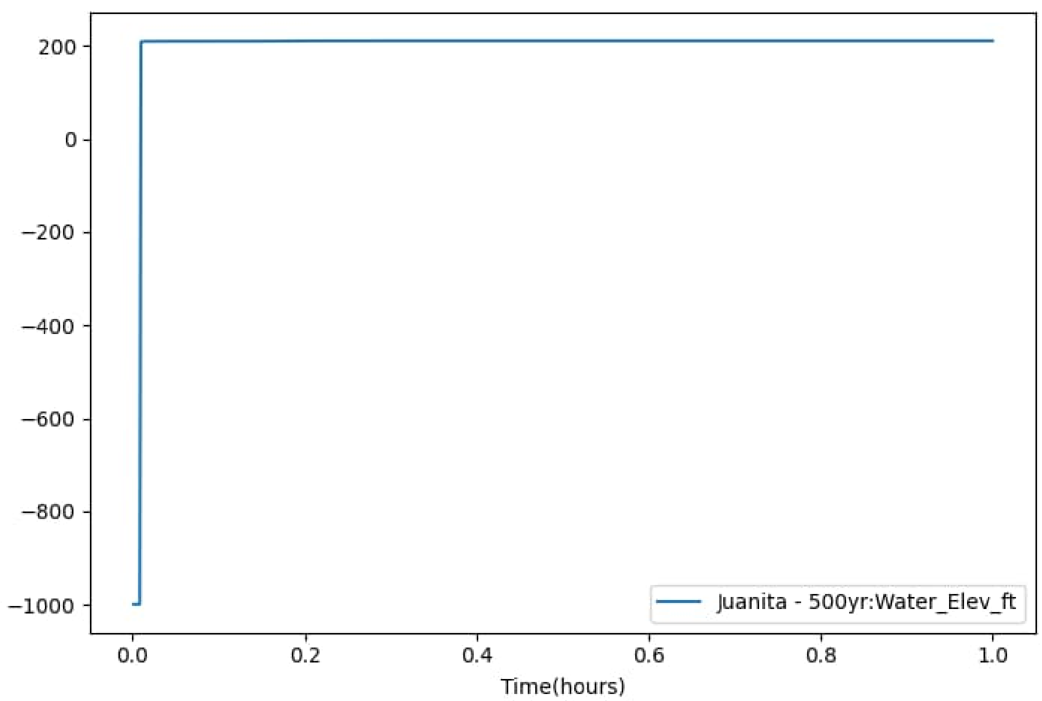
Existing Conditions Monitor Point 2 (MP2) 500-Year WSE Plot



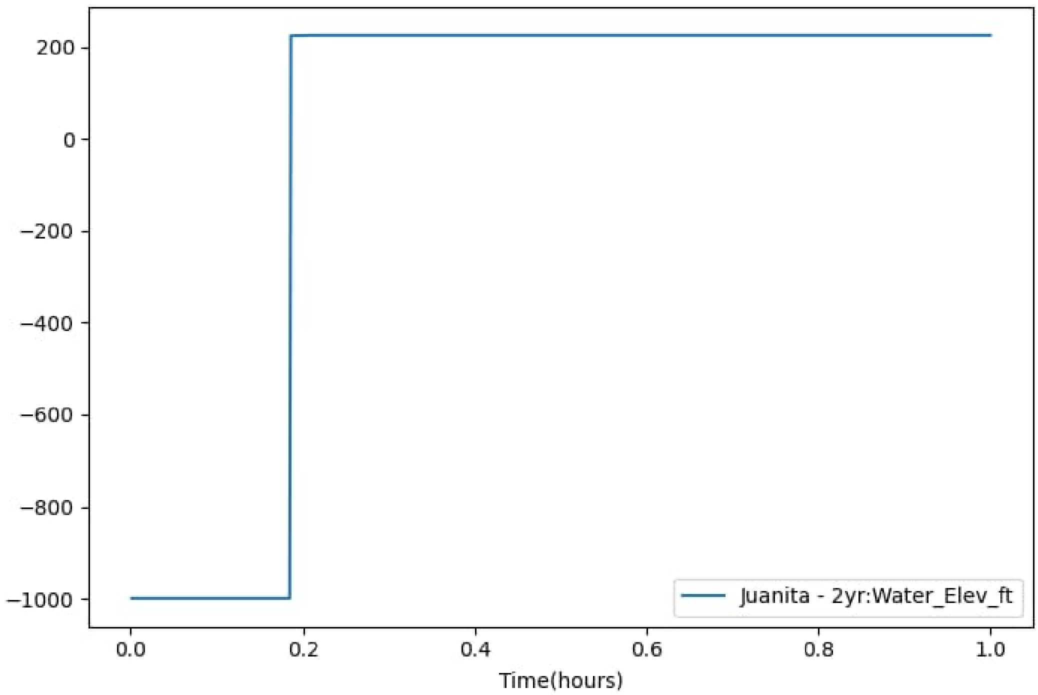
Existing Conditions Monitor Point 3 (MP3) 2-Year WSE Plot



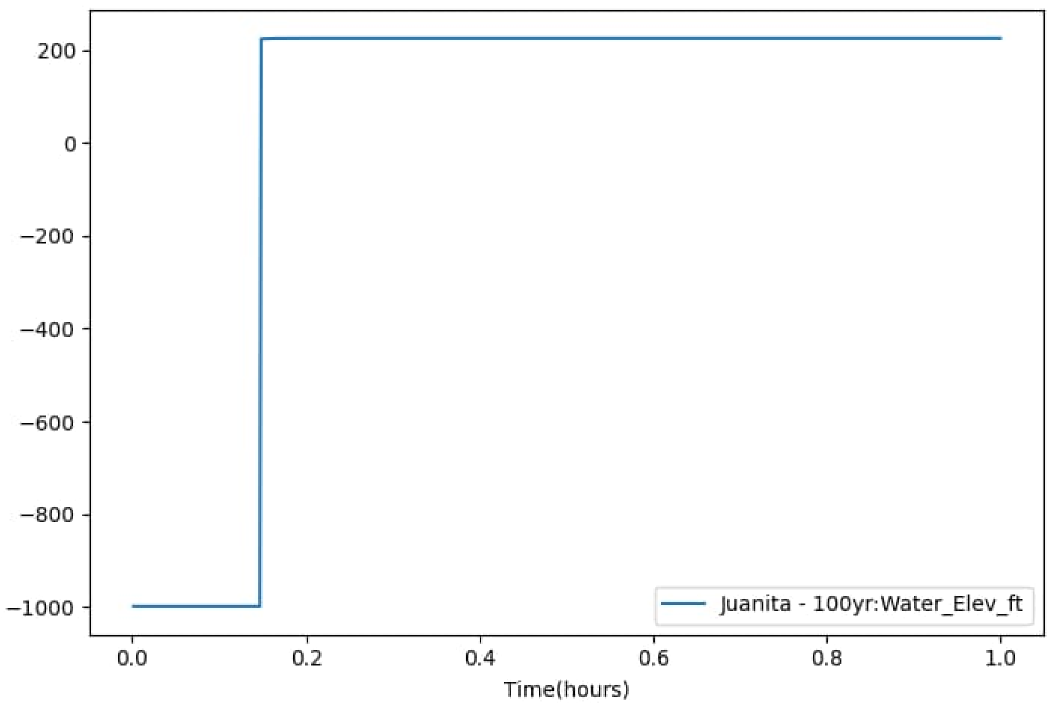
Existing Conditions Monitor Point 3 (MP3) 100-Year WSE Plot



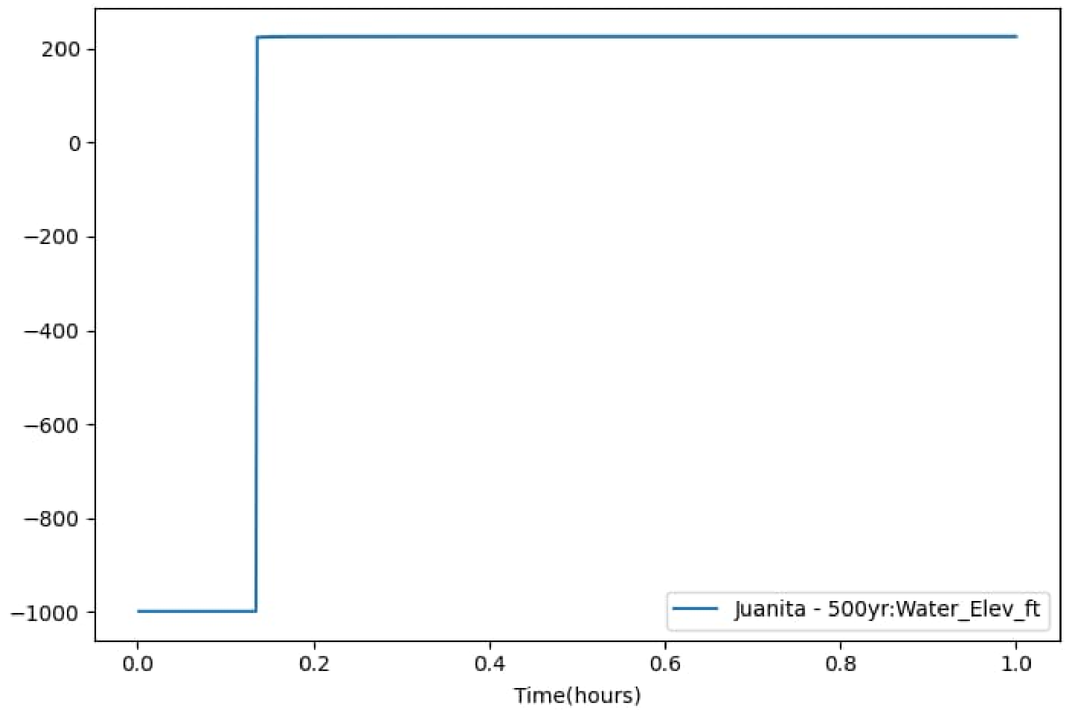
Existing Conditions Monitor Point 3 (MP3) 500-Year WSE Plot



Existing Conditions Monitor Point 4 (MP4) 2-Year WSE Plot

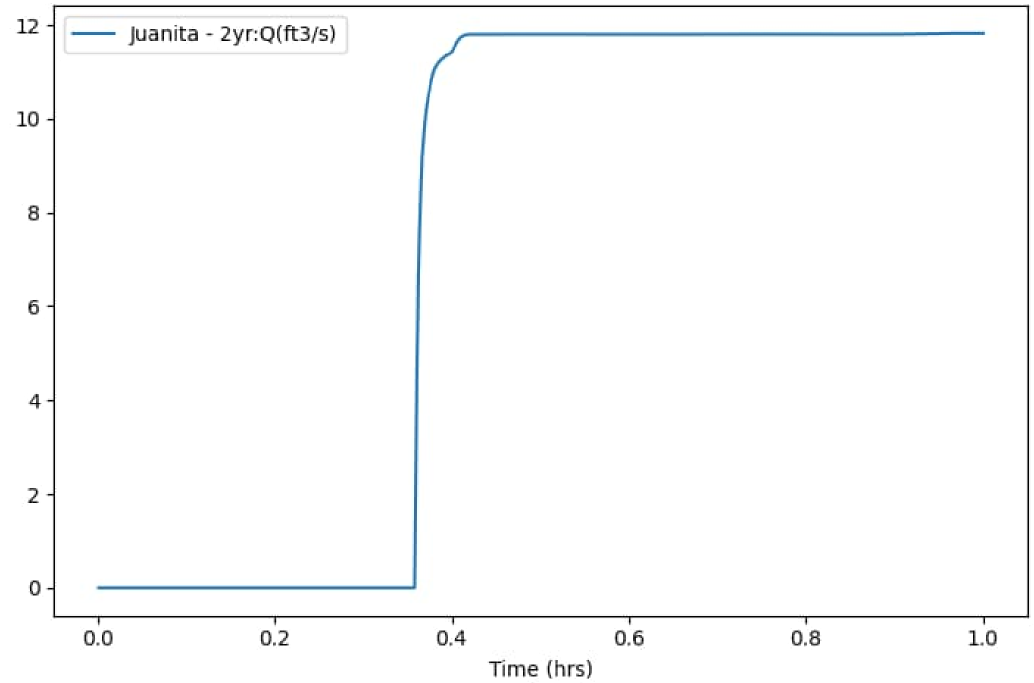


Existing Conditions Monitor Point 4 (MP4) 100-Year WSE Plot

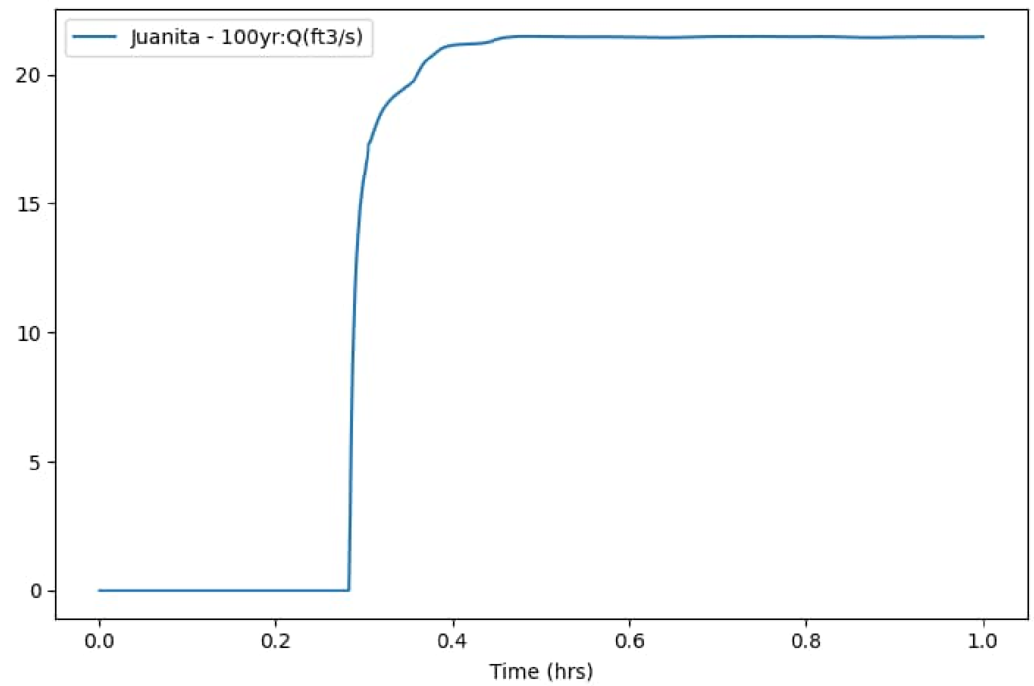


Existing Conditions Monitor Point 4 (MP4) 100-Year WSE Plot

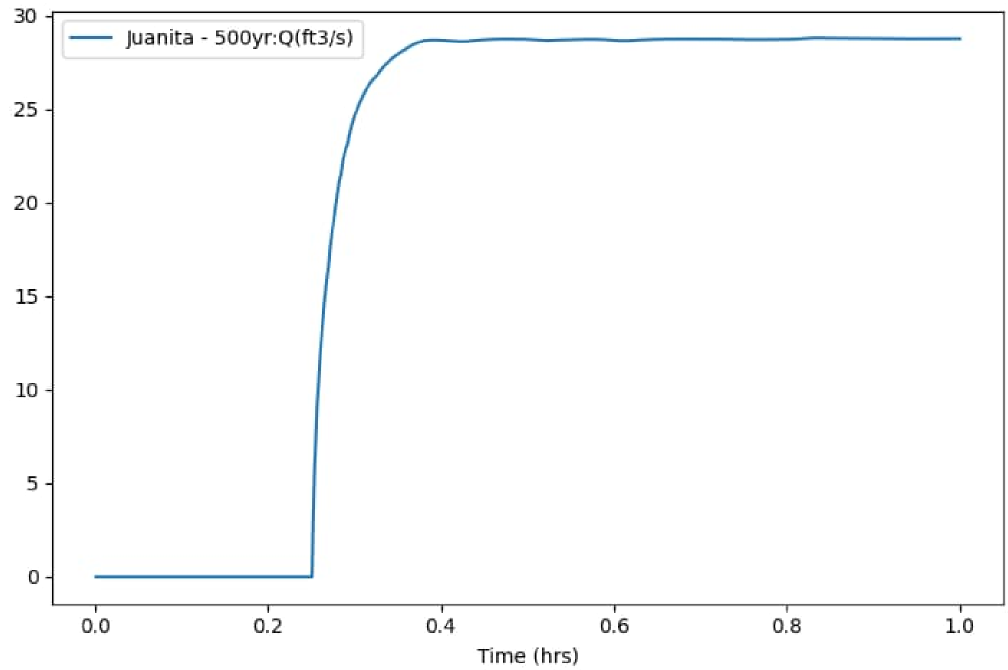
Existing Conditions SMS Monitoring Line Flow Plots



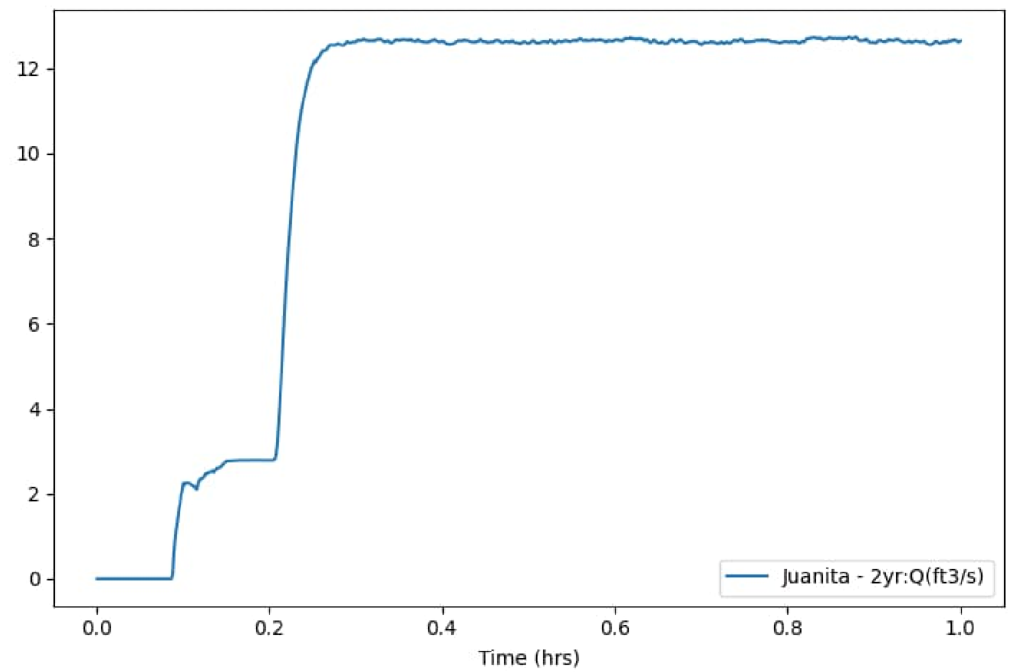
Existing Conditions Monitor Line 1 (ML 1) 2-Year Flow Plot



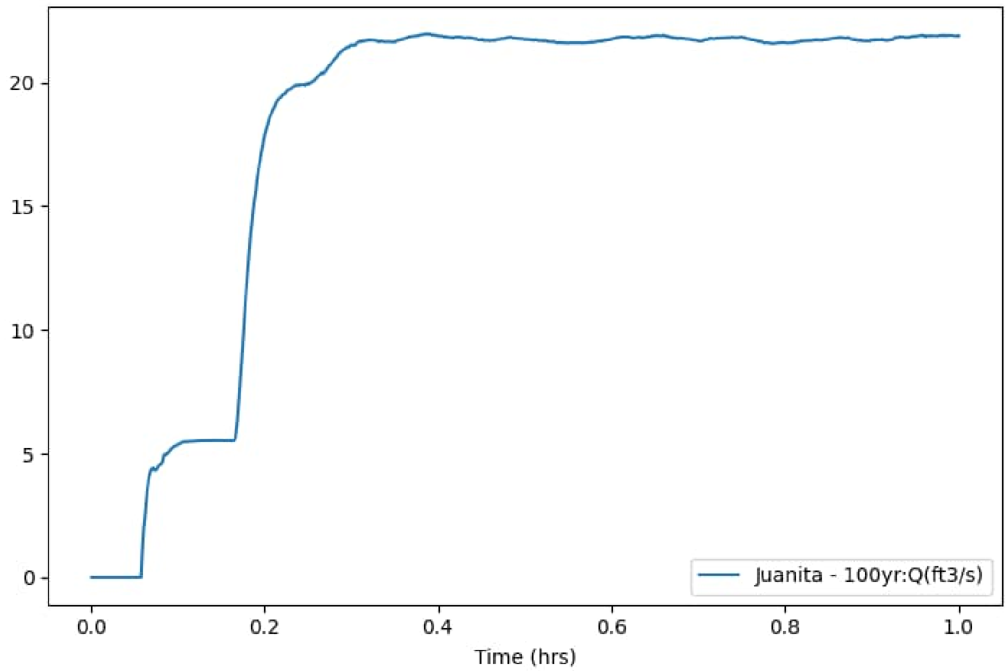
Existing Conditions Monitor Line 1 (ML 1) 100-Year Flow Plot



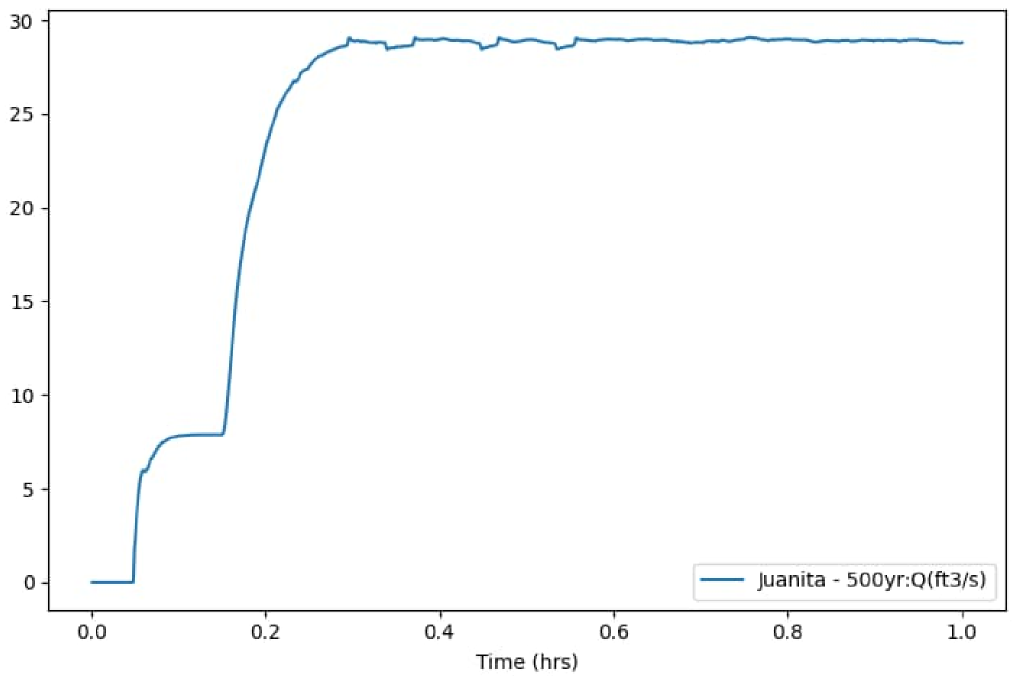
Existing Conditions Monitor Line 1 (ML 1) 500-Year Flow Plot



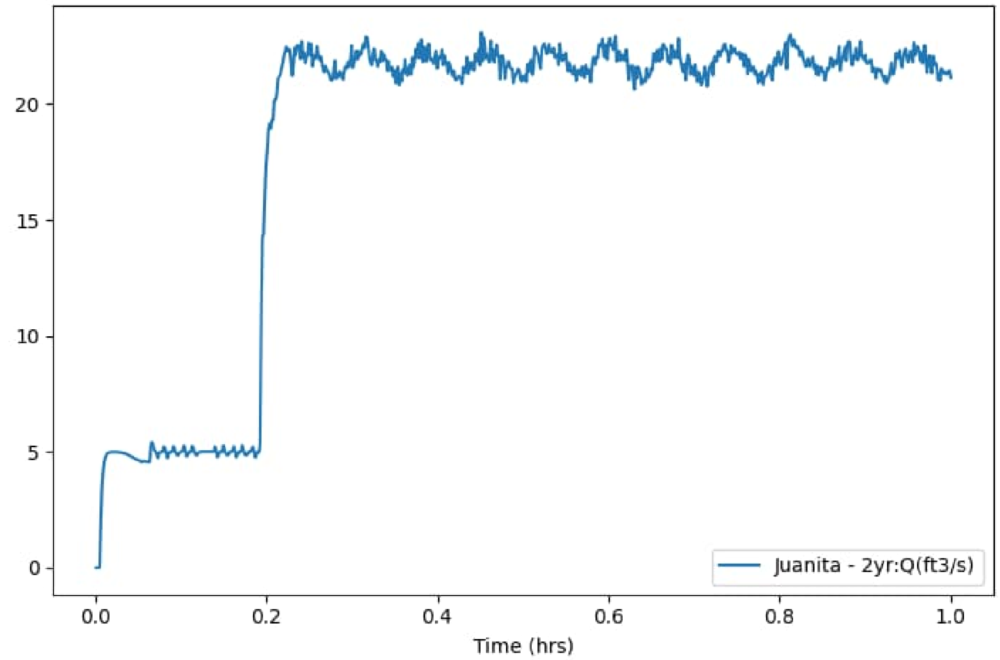
Existing Conditions Monitor Line 2 (ML 2) 2-Year Flow Plot



Existing Conditions Monitor Line 2 (ML 2) 100-Year Flow Plot

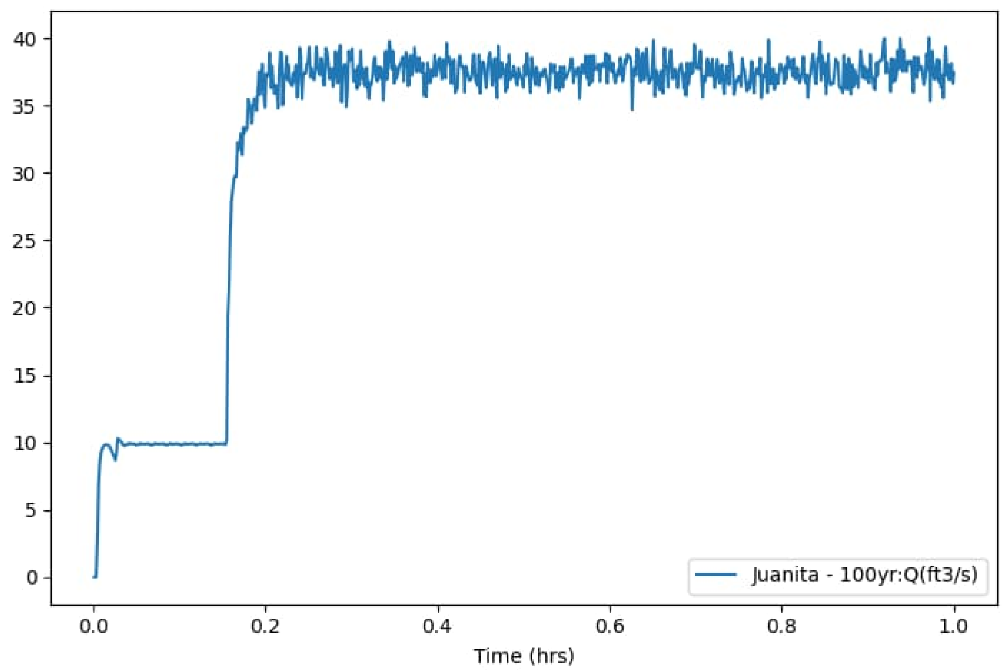


Existing Conditions Monitor Line 2 (ML 2) 500-Year Flow Plot

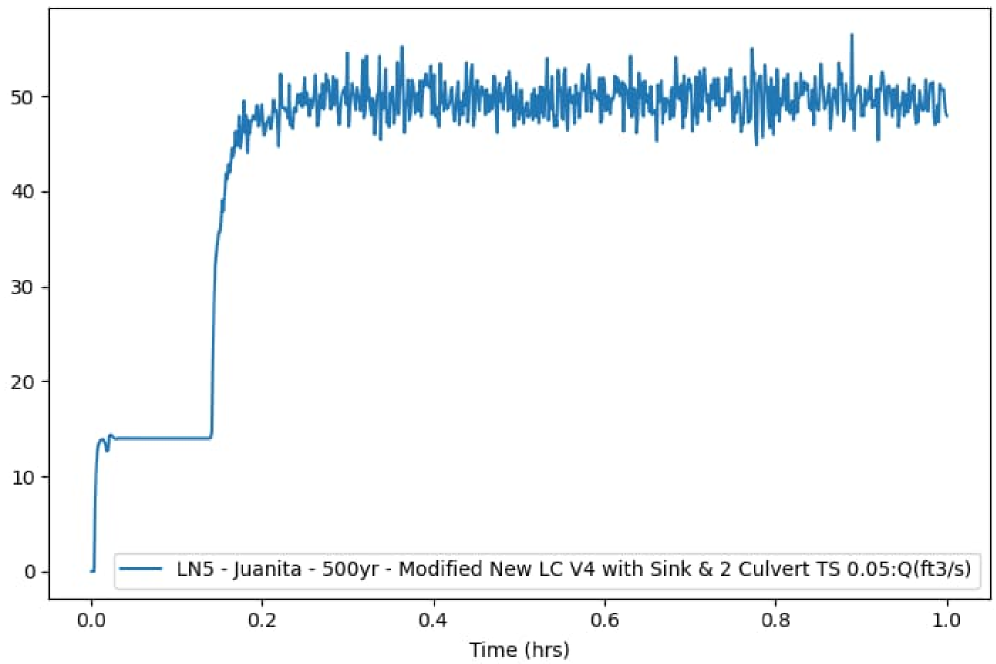


Note: Monitor Line 3 is immediately downstream of the I-405 culvert and the additional inflow boundary condition, and upstream of the 24" and 36" culverts, and thus there is some minor instability evident. The time step for the Existing Conditions model was decreased to 0.05 sec to help resolve this issue.

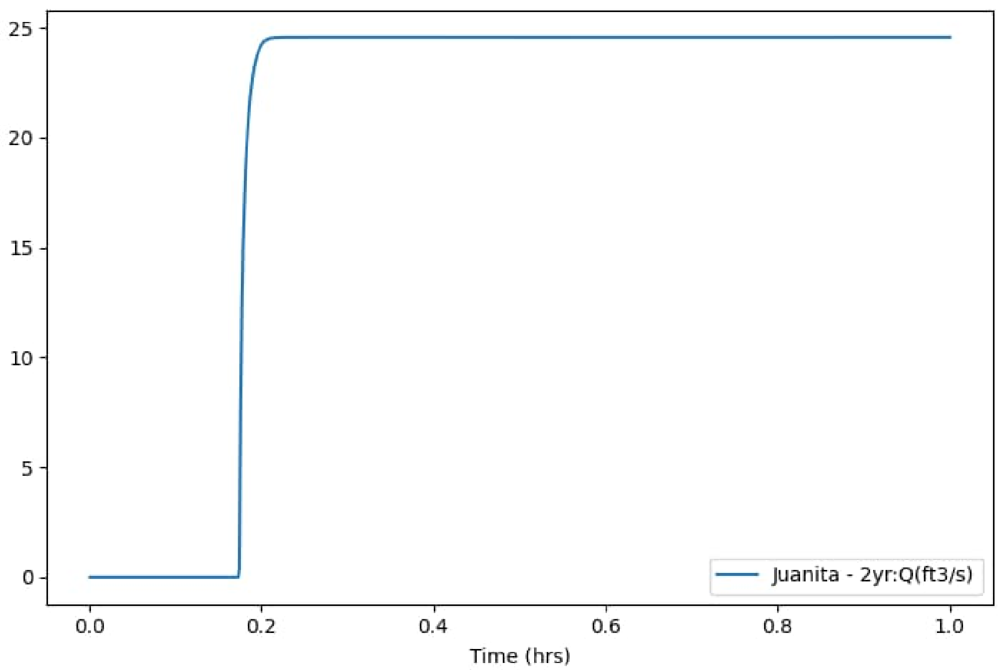
Existing Conditions Monitor Line 3 (ML 3) 2-Year Flow Plot



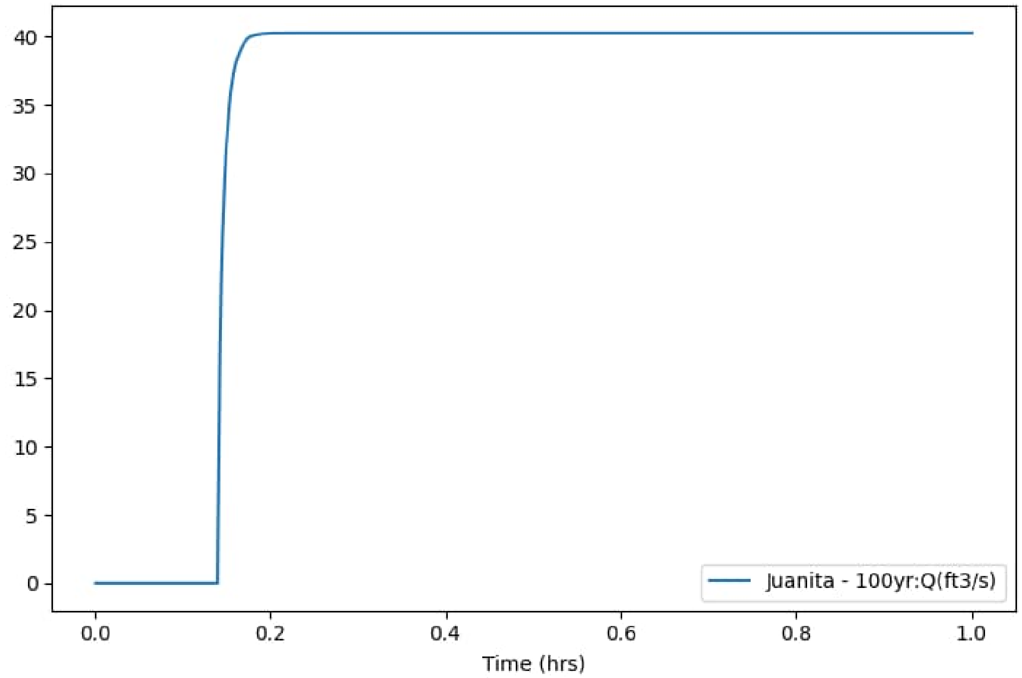
Existing Conditions Monitor Line 3 (ML 3) 100-Year Flow Plot



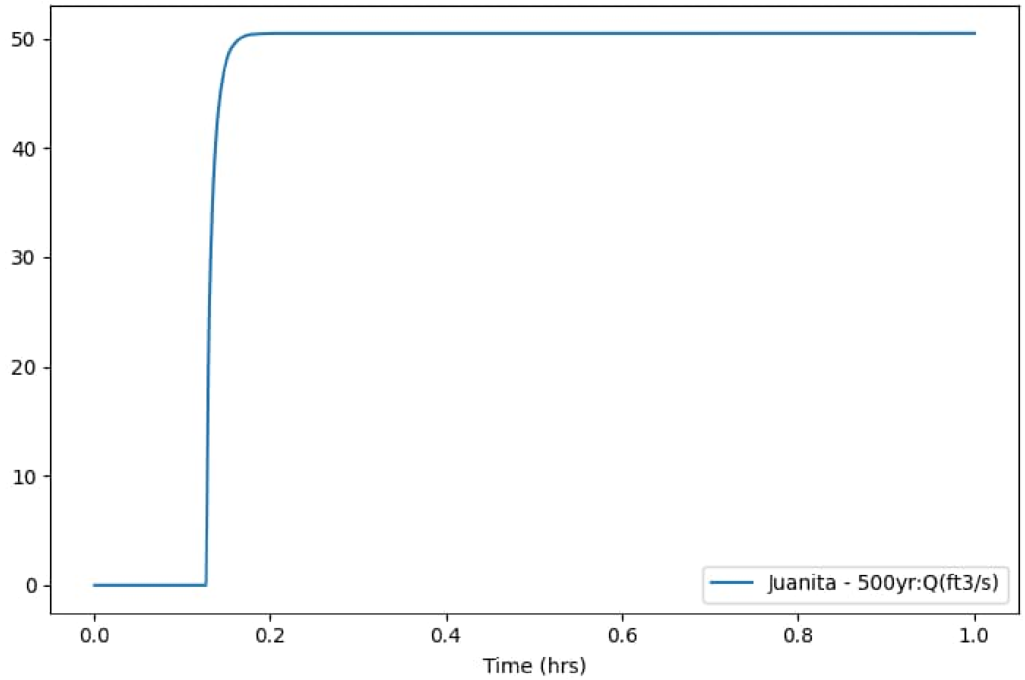
Existing Conditions Monitor Line 3 (ML 3) 500-Year Flow Plot



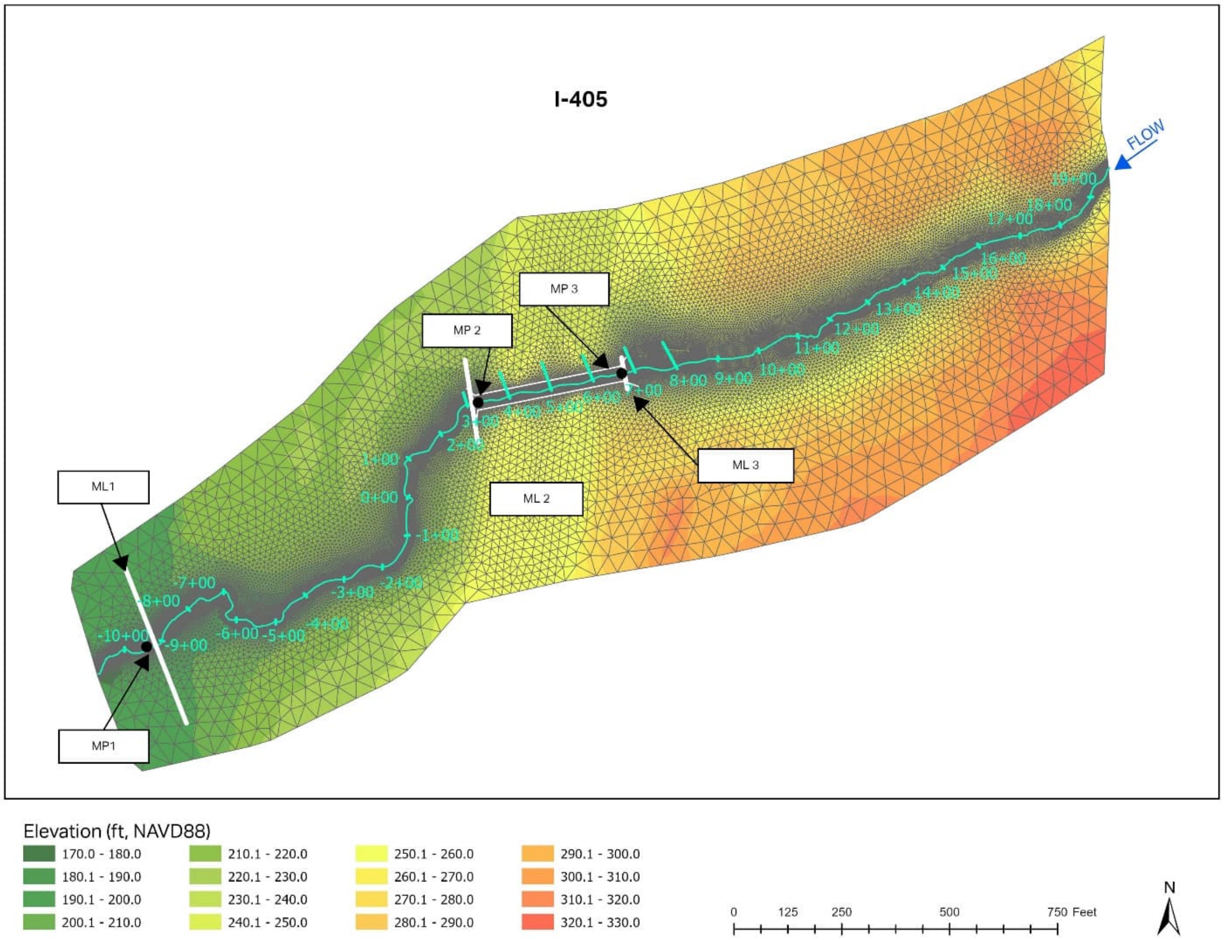
Existing Conditions Monitor Line 4 (ML 4) 2-Year Flow Plot



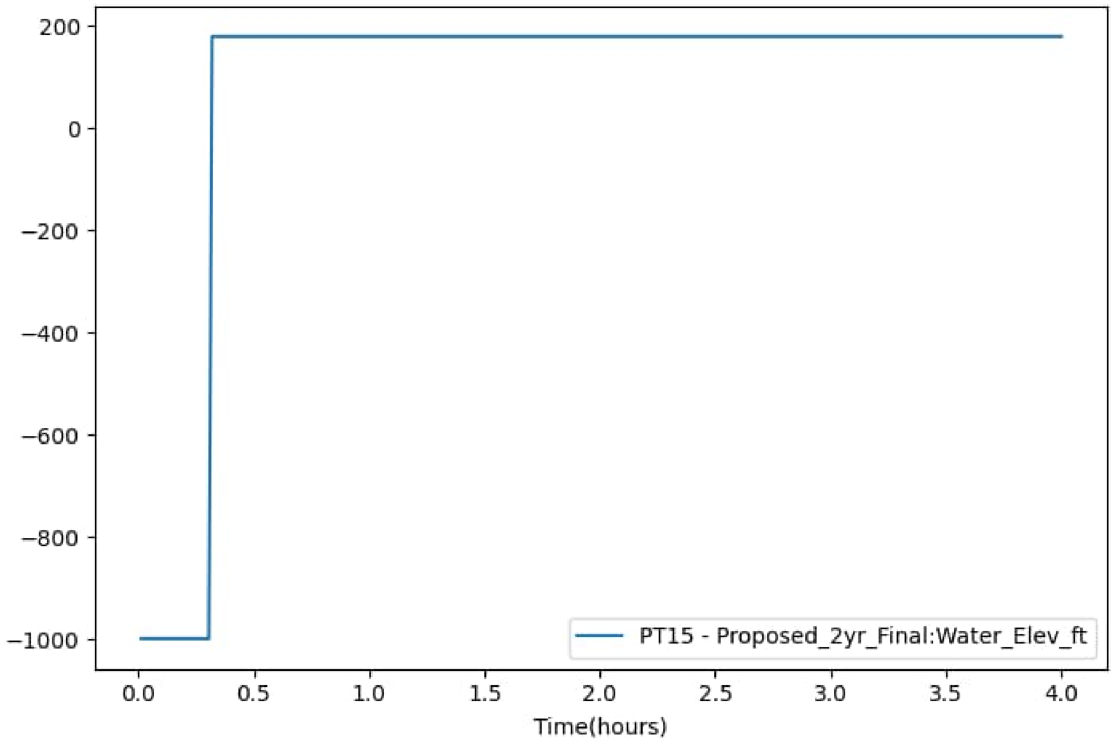
Existing Conditions Monitor Line 4 (ML 4) 100-Year Flow Plot



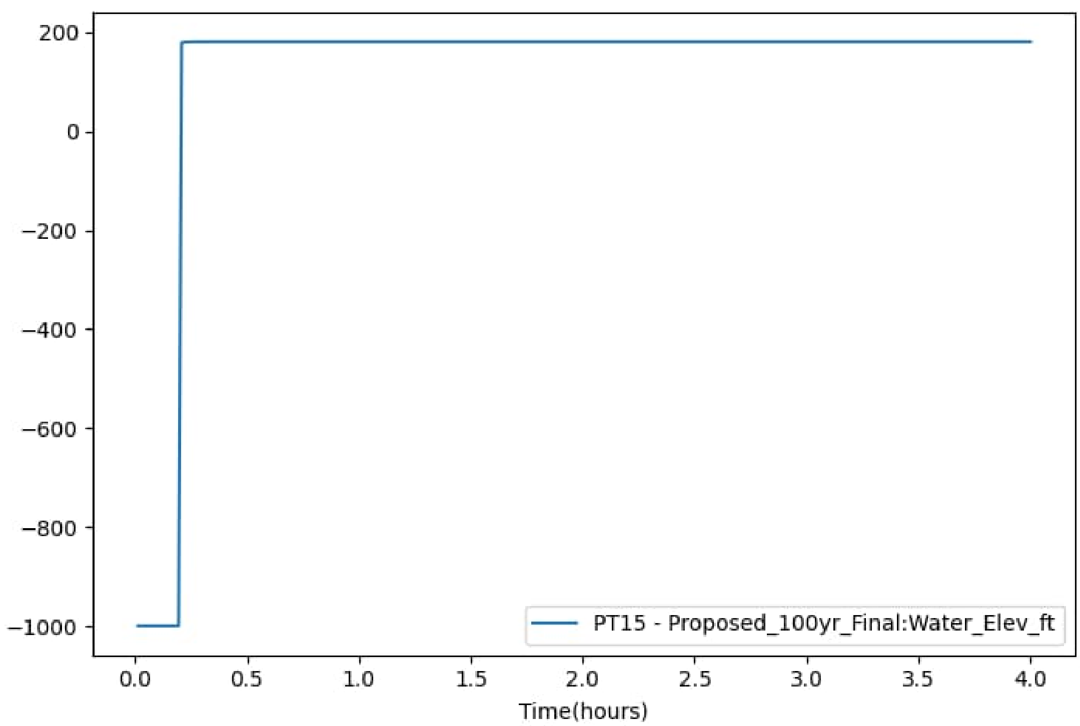
Existing Conditions Monitor Line 4 (ML 4) 500-Year Flow Plot



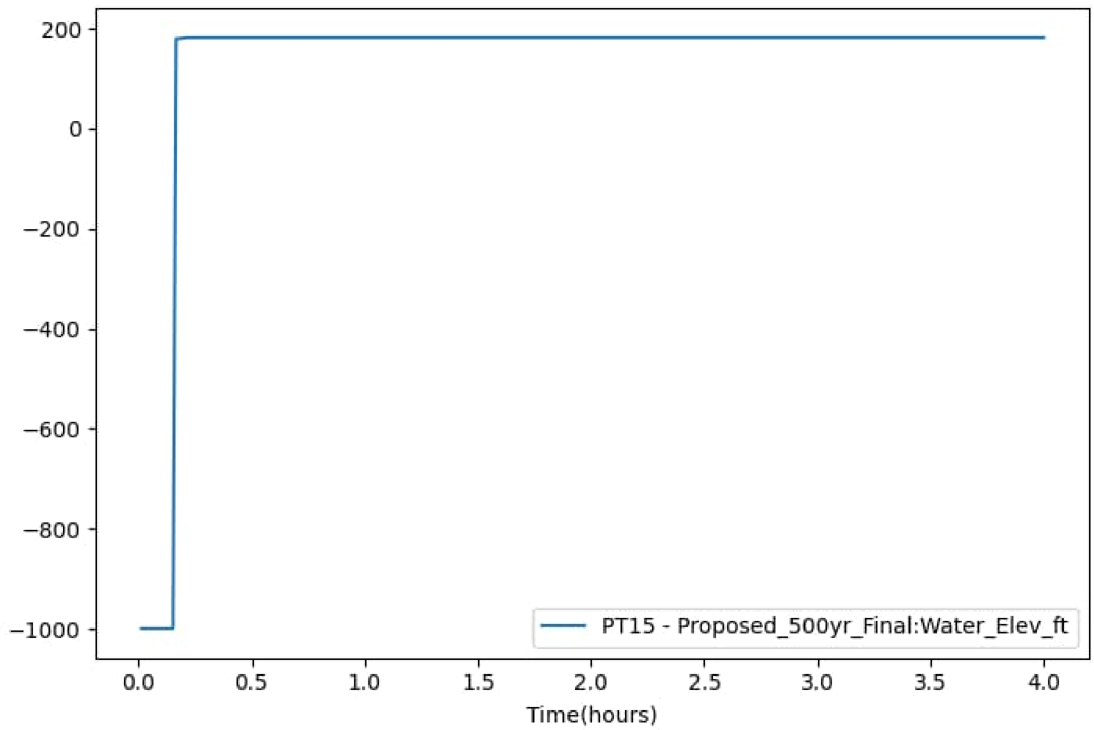
Proposed **Conditions SMS Monitoring** Point WSE Plots



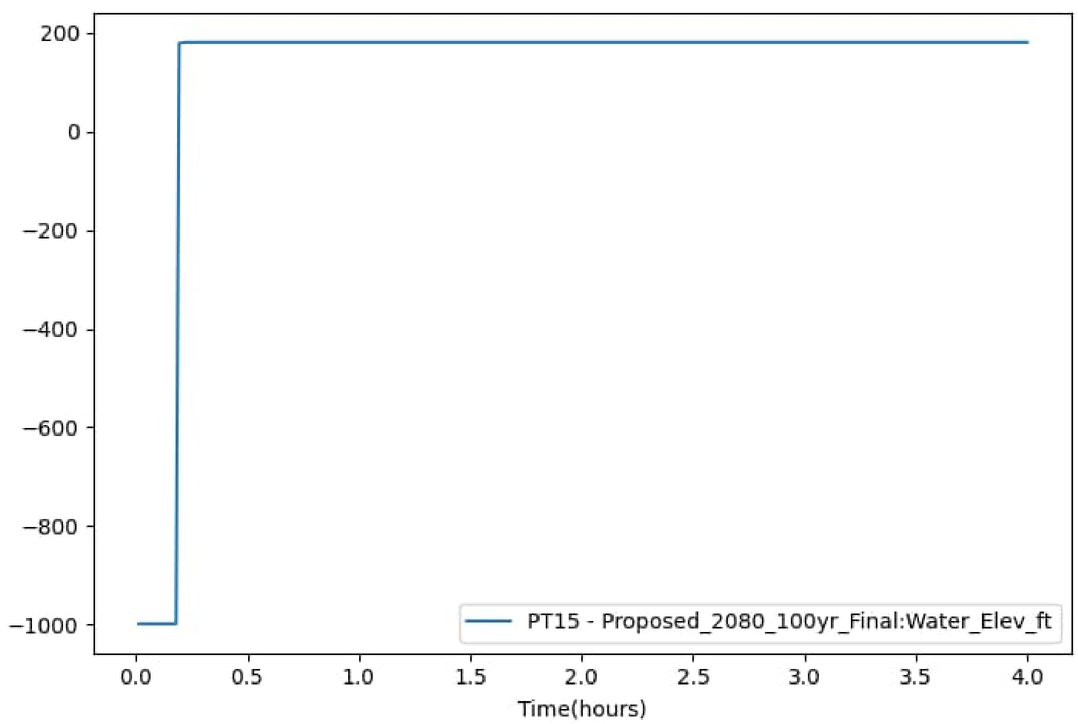
Proposed Conditions Monitor Point 1 (MP 1) 2-Year WSE Plot



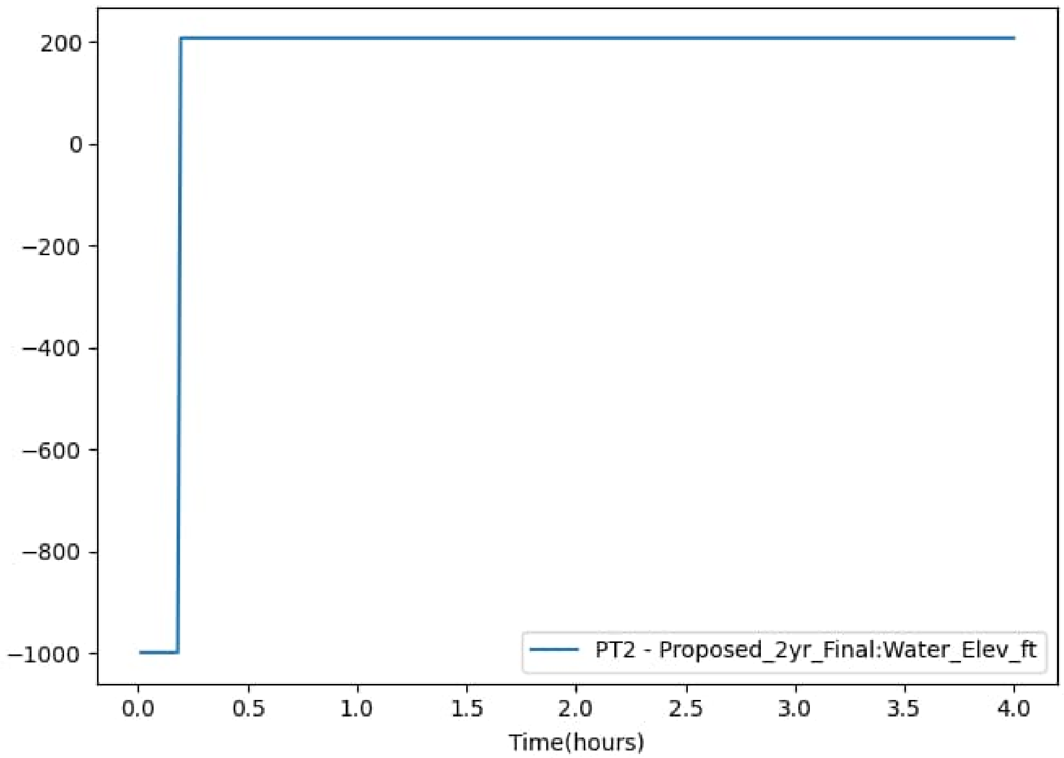
Proposed Conditions Monitor Point 1 (MP 1) 100-Year WSE Plot



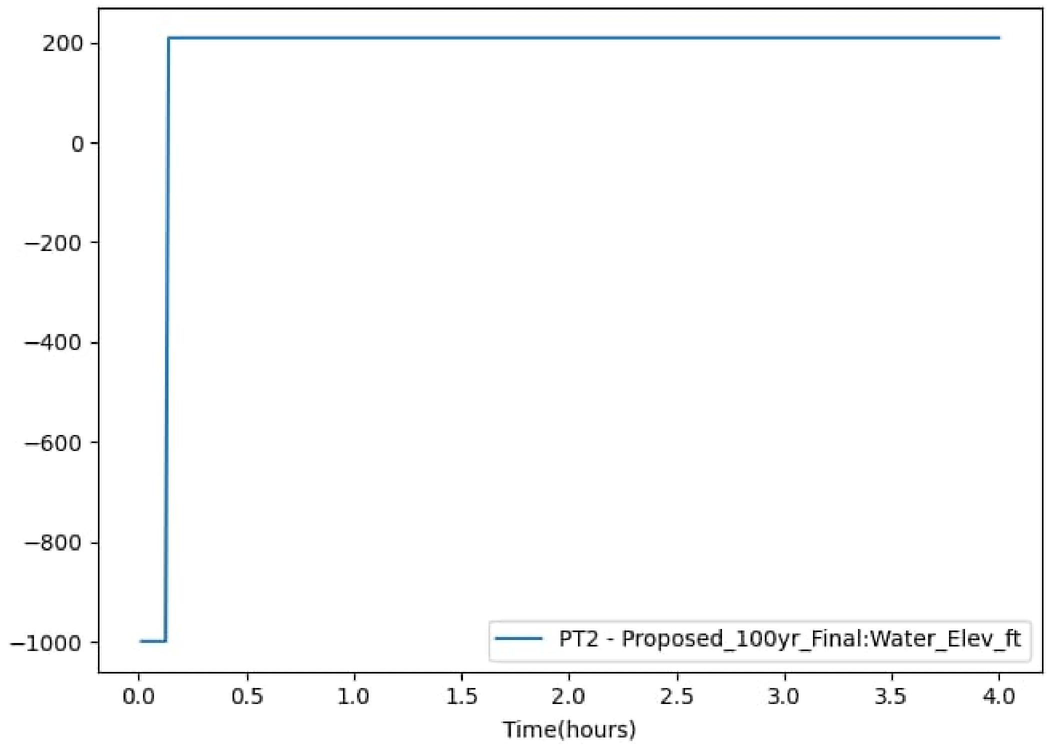
Proposed Conditions Monitor Point 1 (MP 1) 500-Year WSE Plot



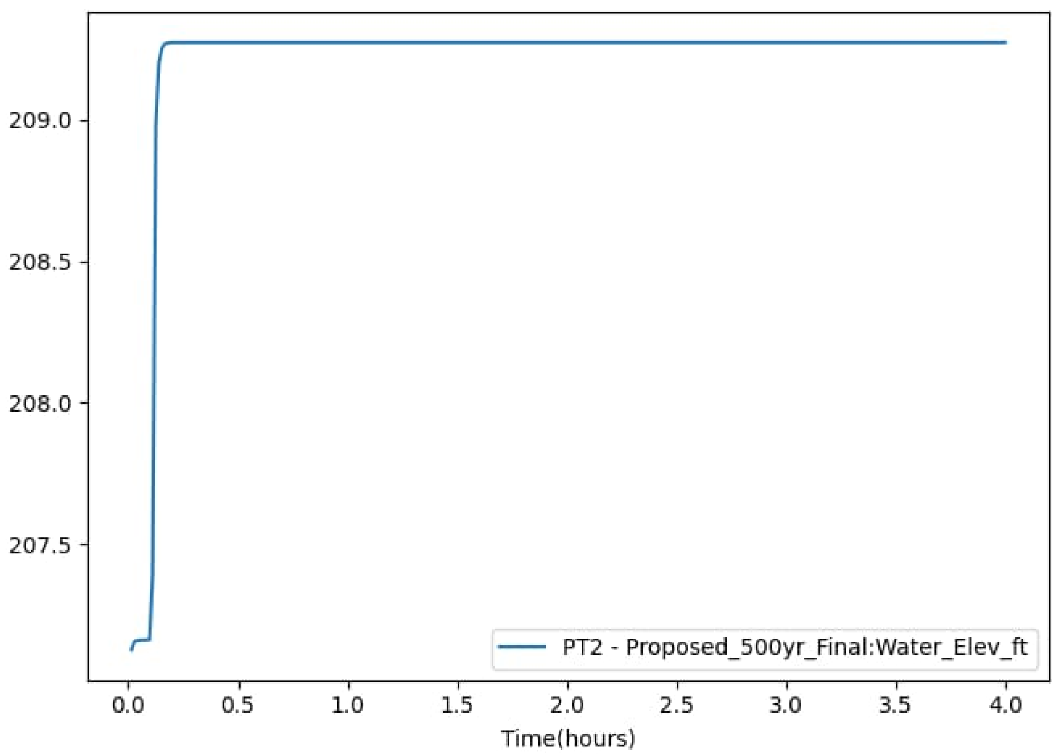
Proposed Conditions Monitor Point 1 (MP 1) 2080 100-Year WSE Plot



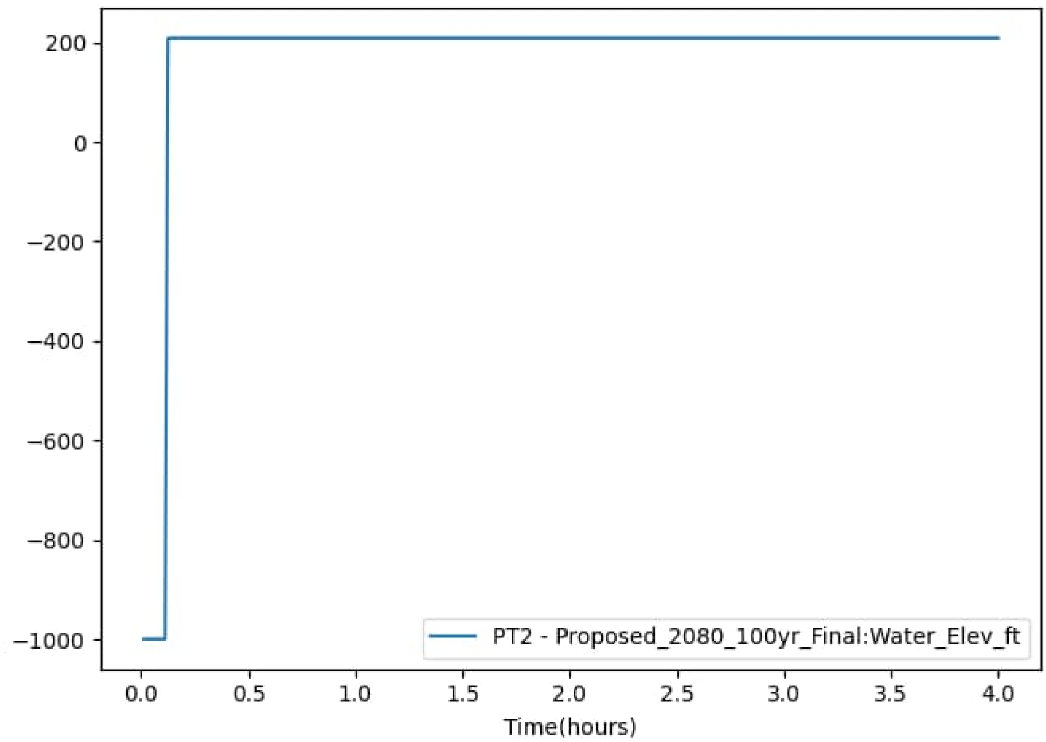
Proposed Conditions Monitor Point 2 (MP 2) 2-Year WSE Plot



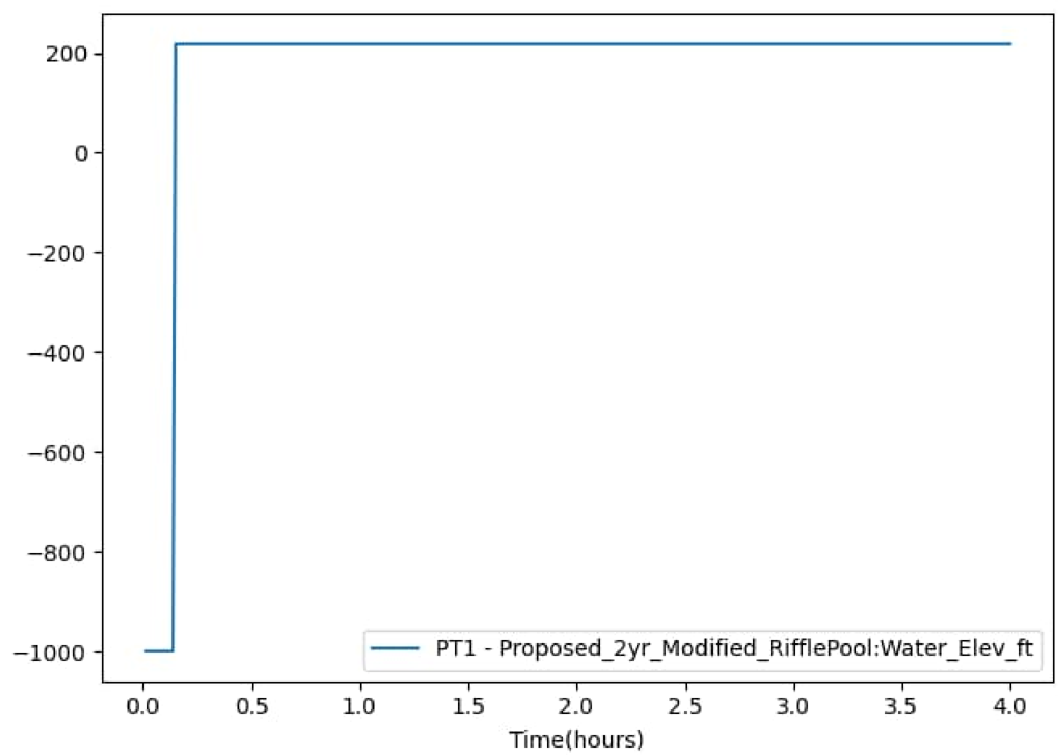
Proposed Conditions Monitor Point 2 (MP 2) 100-Year WSE Plot



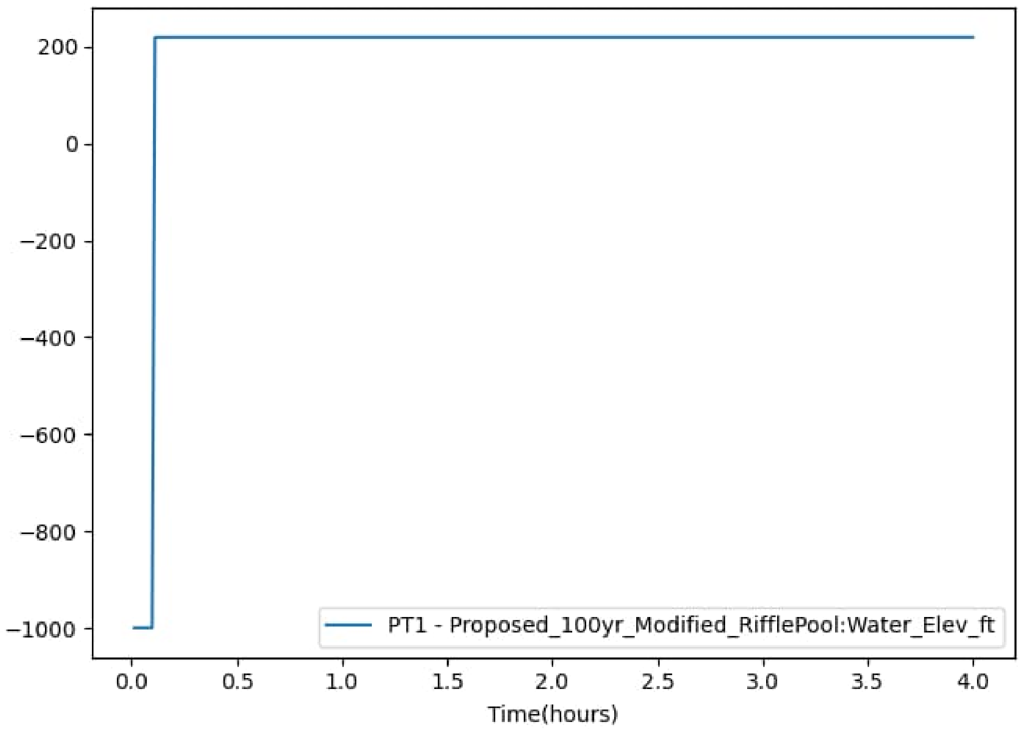
Proposed Conditions Monitor Point 2 (MP 2) 500-Year WSE Plot



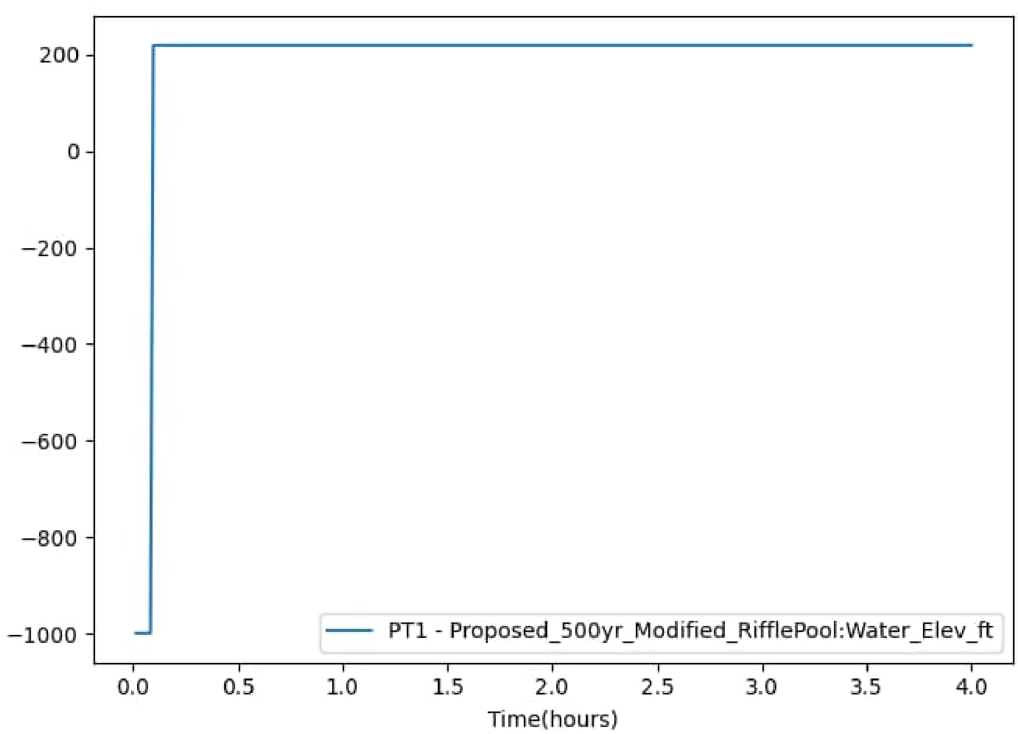
Proposed Conditions Monitor Point 2 (MP 2) 2080 100-Year WSE Plot



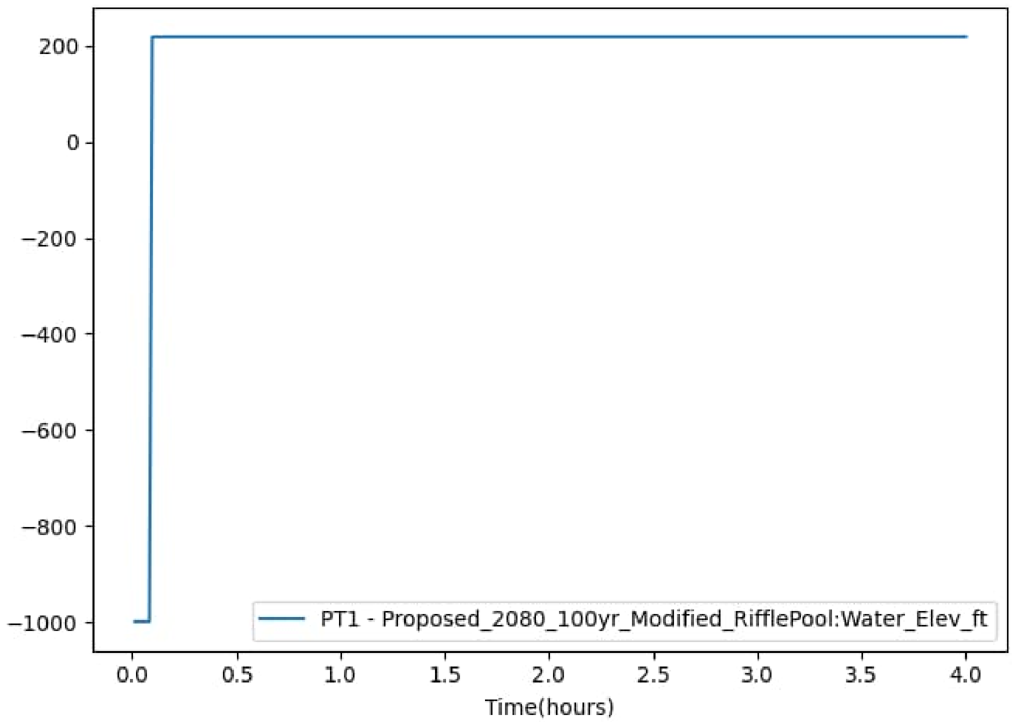
Proposed Conditions Monitor Point 3 (MP 3) 2-Year WSE Plot



Proposed Conditions Monitor Point 3 (MP 3) 100-Year WSE Plot

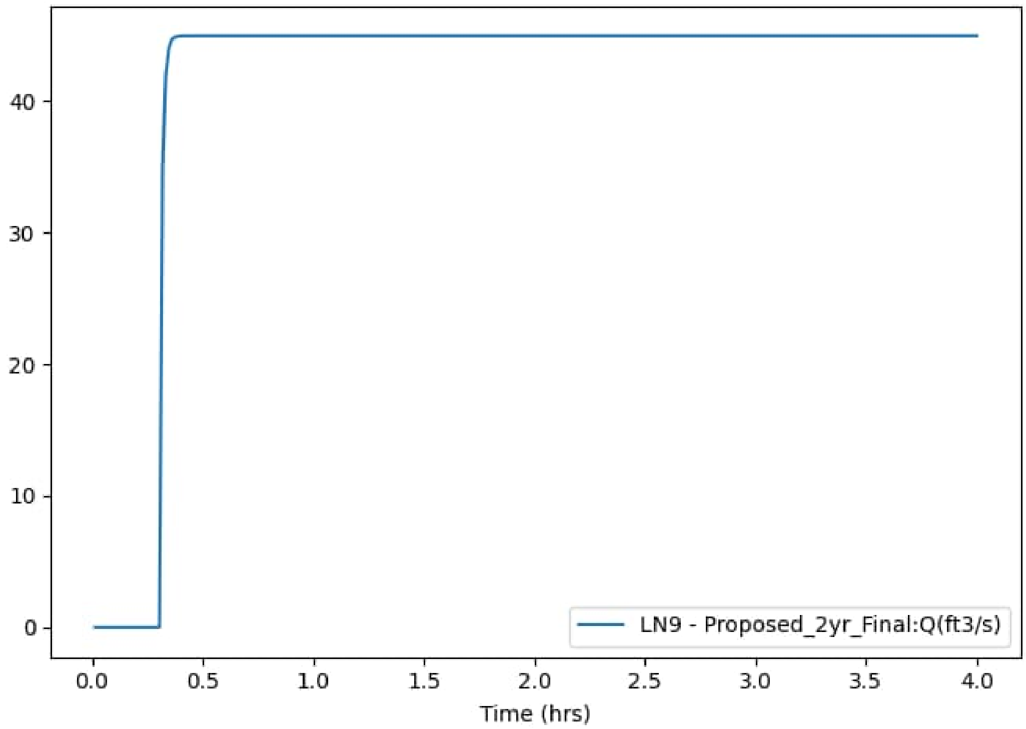


Proposed Conditions Monitor Point 3 (MP 3) 500-Year WSE Plot

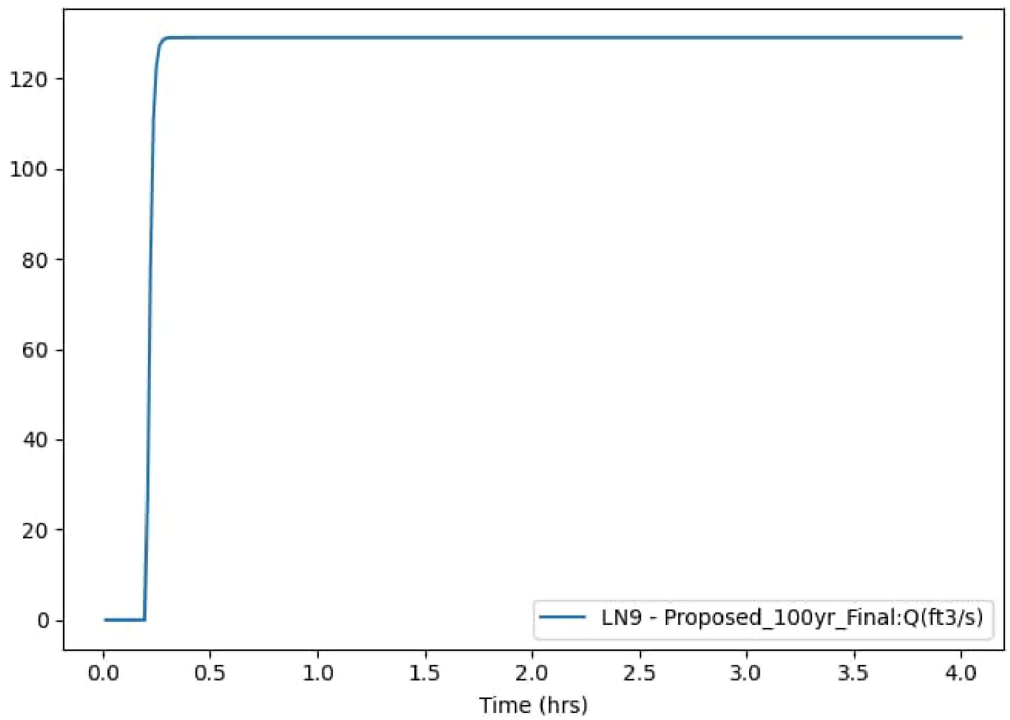


Proposed Conditions Monitor Point 3 (MP 3) 2080 100-Year WSE Plot

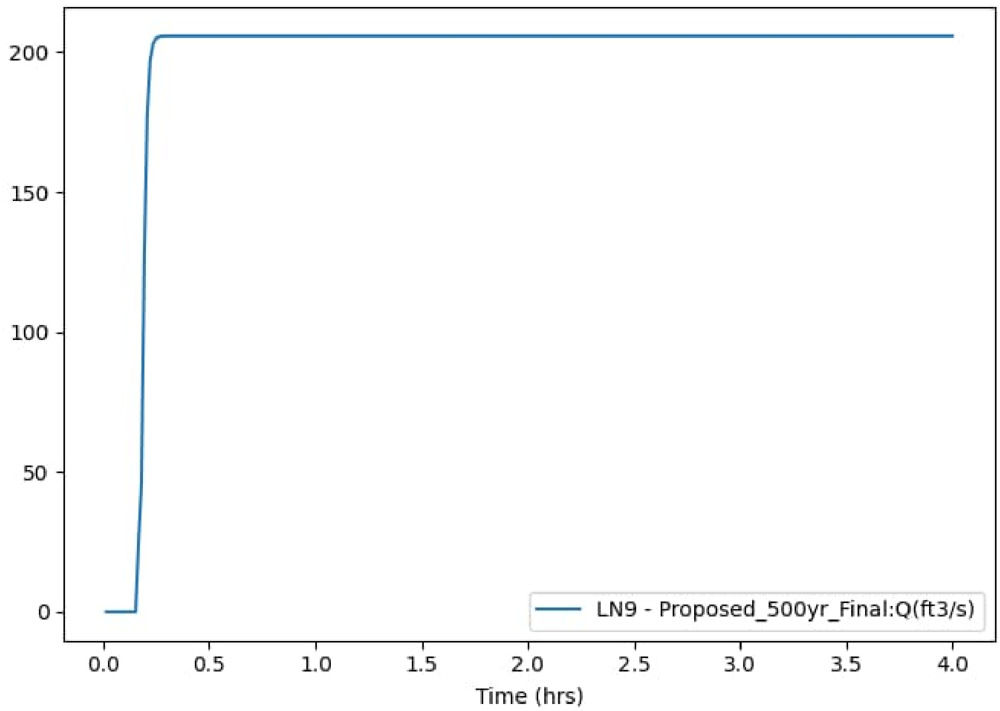
Proposed **Conditions SMS Monitoring** Line Flow Plots



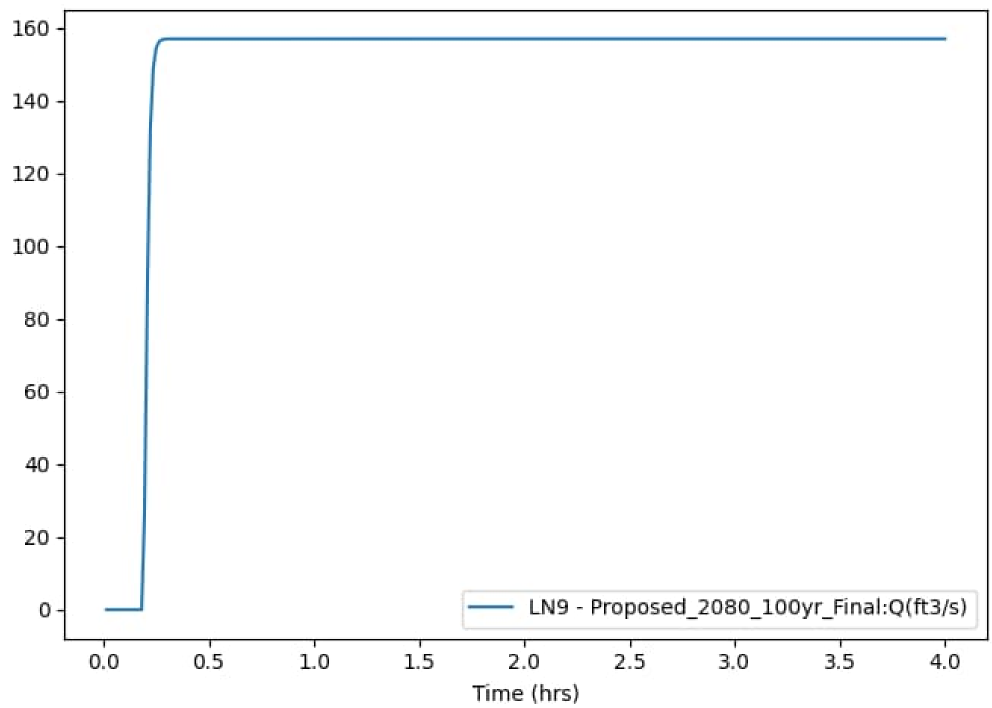
Proposed Conditions Monitor Line 1 (ML 1) 2-Year Flow Plot



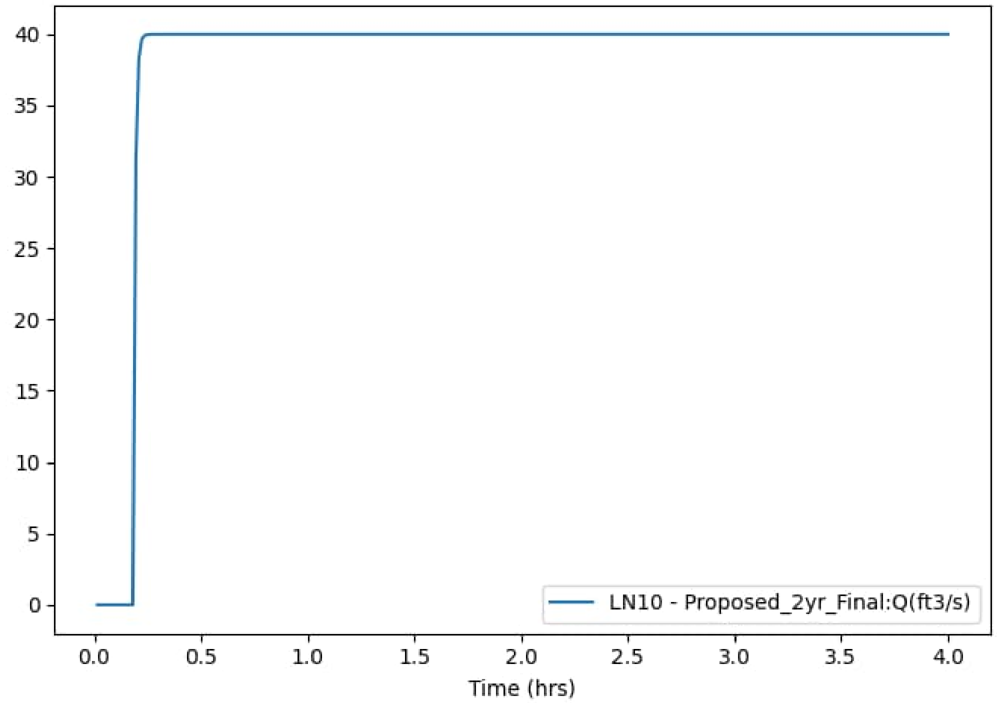
Proposed Conditions Monitor Line 1 (ML 1) 100-Year Flow Plot



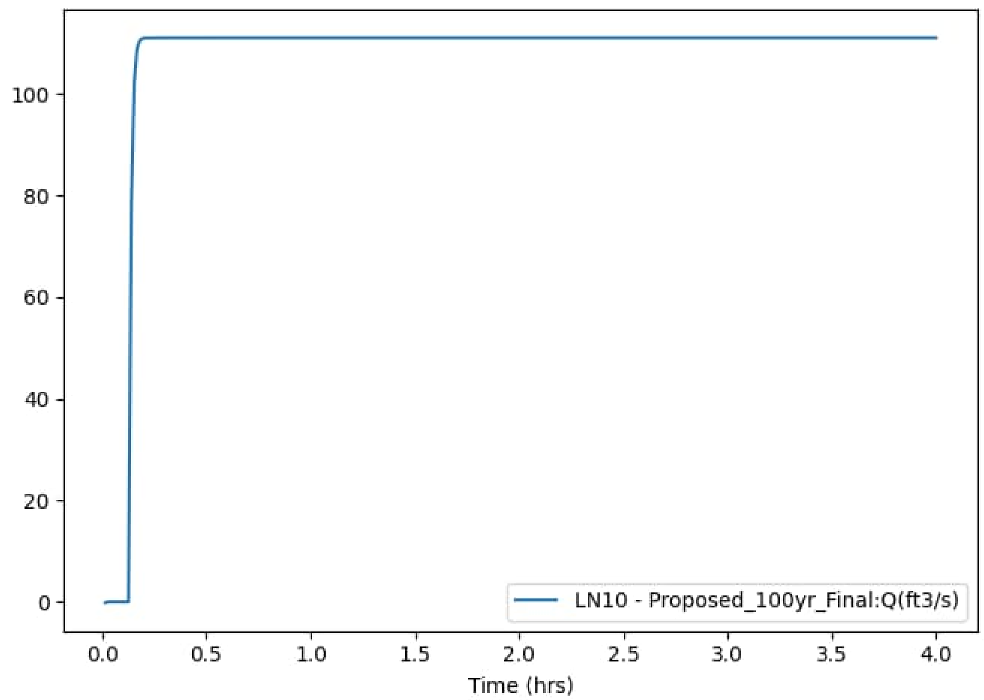
Proposed Conditions Monitor Line 1 (ML 1) 500-Year Flow Plot



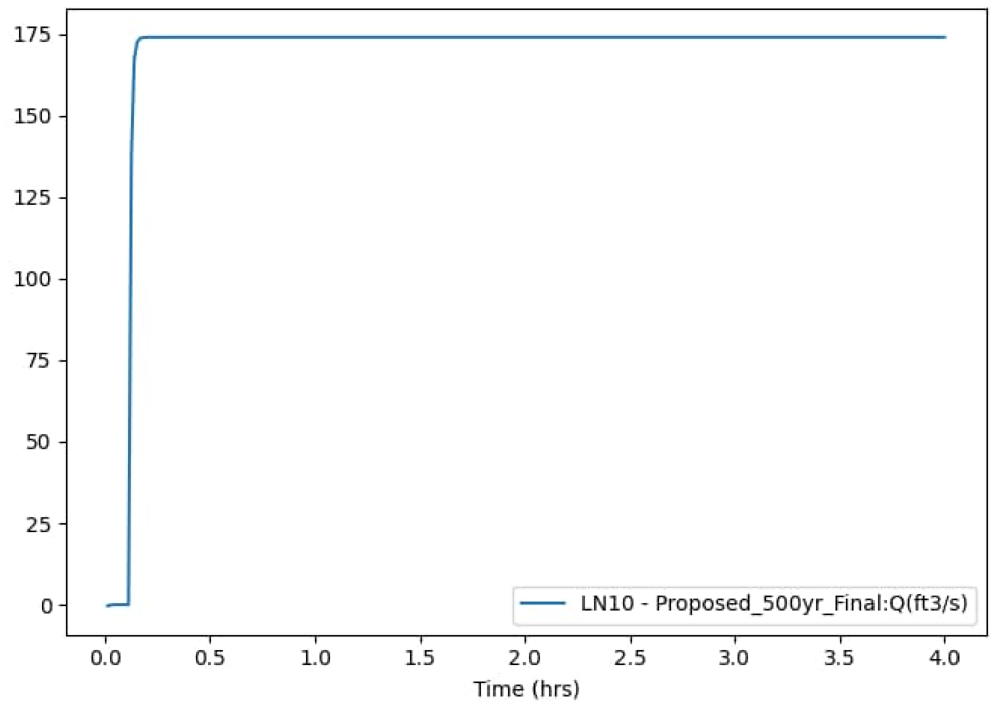
Proposed Conditions Monitor Line 1 (ML 1) 2080 100-Year Flow Plot



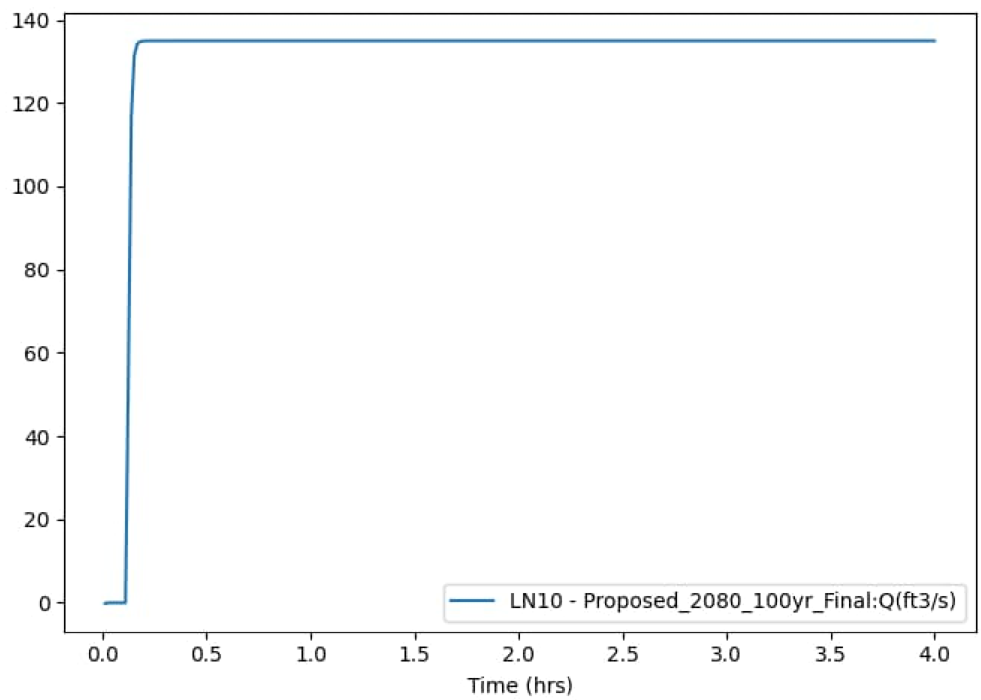
Proposed Conditions Monitor Line 2 (ML 2) 2-Year Flow Plot



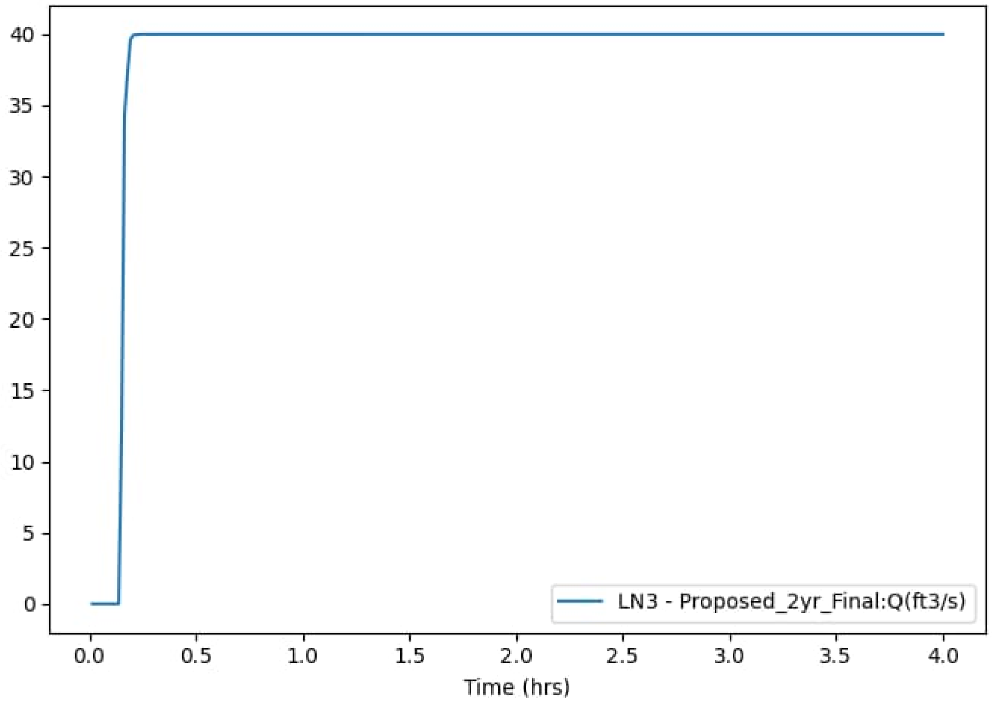
Proposed Conditions Monitor Line 2 (ML 2) 100-Year Flow Plot



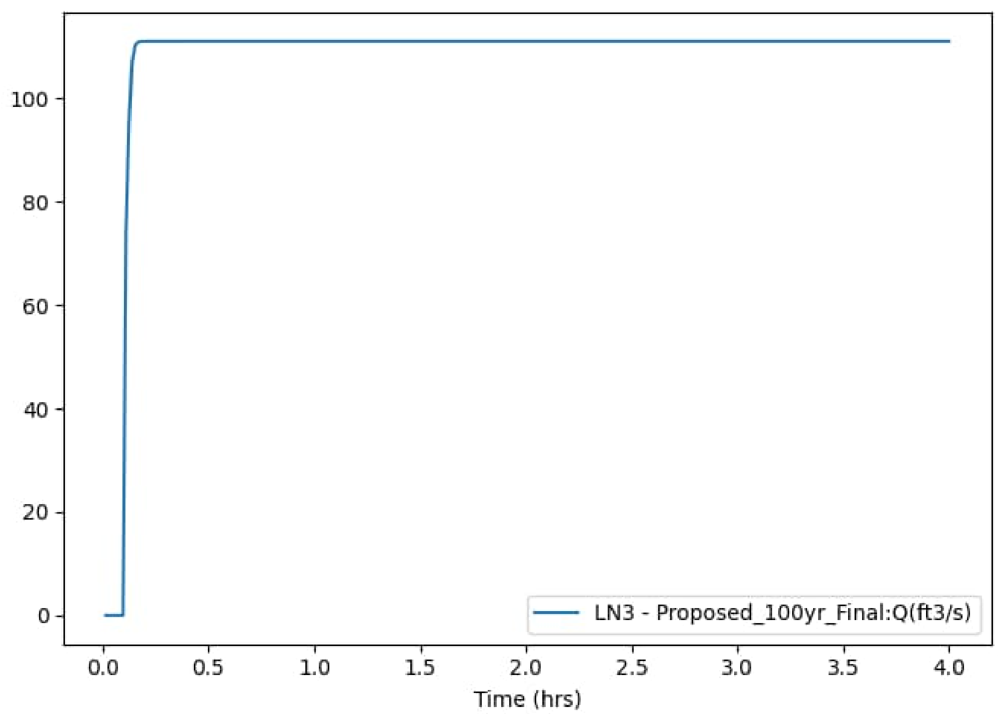
Proposed Conditions Monitor Line 2 (ML 2) 500-Year Flow Plot



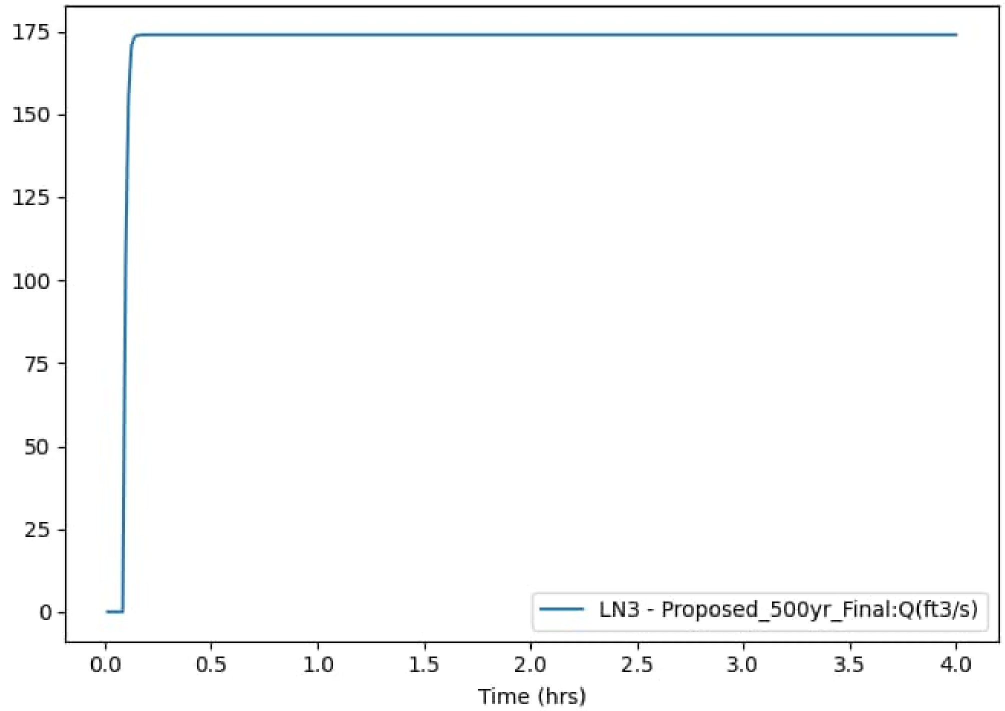
Proposed Conditions Monitor Line 2 (ML 2) 2080 100-Year Flow Plot



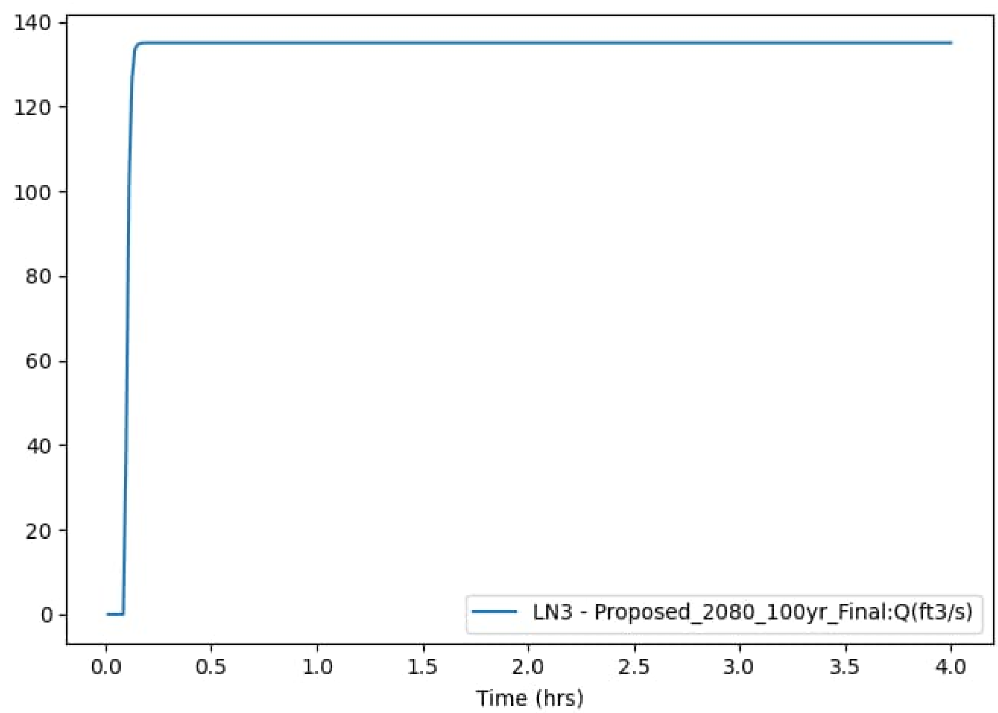
Proposed Conditions Monitor Line 3 (ML 3) 2-Year Flow Plot



Proposed Conditions Monitor Line 3 (ML 3) 100-Year Flow Plot



Proposed Conditions Monitor Line 3 (ML 3) 500-Year Flow Plot



Proposed Conditions Monitor Line 3 (ML 3) 2080 100-Year Flow Plot

Appendix J: Reach Assessment

Washington Department of Fish and Wildlife

Fish Passage & Diversion Screening Inventory Database
Report Cover Sheet

The following report is extracted from the Washington Department of Fish and Wildlife's (WDFW) Fish Passage and Diversion Screening Inventory Database (FPDSI). WDFW makes every attempt to keep these reports in sync with FPDSI; however, the dynamic nature of the data and workflows associated with maintaining the database may result in short-term differences.

Users are encouraged to contact WDFW to discuss appropriate use of the data and how we can assist with fish passage barrier removal or inventory. Please visit the Fish Passage web site for contact information at: https://wdfw.wa.gov/species-habitats/habitat-recovery/fish-passage/about

Disclaimers:

- Data presented here represent a snapshot observation of conditions in a dynamic environment that is subject to change. Fish passage data are also collected from a variety of agencies and sources. Therefore, WDFW makes no guarantee concerning the data's content, accuracy, completeness, or the results obtained from use of the data. WDFW assumes no liability for the data represented here.
- These data are not an attempt to provide you with an official agency response as to the impacts of your project on fish and wildlife.
- Note that some fish passage features, habitats or species may occur in areas not currently known to the WDFW Fish Passage division, and may not be reflected in this database. A lack of data does not necessarily indicate that a feature, habitat, or species are not present.
- Unauthorized attempts to alter or modify these data are strictly prohibited.
- Bankfull width measurements included in these reports should not be used for fish passage crossing design. They are solely for assessment purposes.
- The barrier status reported in this document is based on the swimming abilities of adult salmonids. Passabilities are a qualitative value, and should not be interpreted as a quantitative calculation. Please see page 1-4 of the Fish Passage Inventory, Assessment and Prioritization Manual for further clarification: https://wdfw.wa.gov/publications/02061
- EXIF data presented with Image Reports may be erroneous due to camera battery failures and resetting of camera clock functions.

Abbreviations:

Most abbreviations in this report are defined in the Quick Reference Tables of the Fish Passage Inventory, Assessment, and Prioritization Manual. Additional commonly used abbreviations are defined as follows:

NFB = no potential salmonid use, **BB** = both banks, **LB** = left bank looking downstream, **RB** = right bank looking downstream, **US or U/S** = upstream, **DS or D/S** = downstream, **WSDrop** = water surface drop, **BFW** = bankfull width, **OHW** = ordinary high water, **SLW** = scour line width, **CMP** = corrugated metal pipe, Q_{fp} = fish passage flow, **V&D** = Velocity and Depth, **ROW** = Right of Way

The FPDSI database often uses default values such as '**-99.99**' or '**-999**' to represent null values.

WDFW Fish Passage and Diversion Screening Inventory Database

Site Description Report

Site ID 998602 **Project** WSDOT ☐ **Mitigated**

Geographic Coordinates

Latitude (WGS 84):	47.732951092
Longitude (WGS 84):	-122.188992351
East (NAD 83 HARN):	1,224,749.2
North (NAD 83 HARN)	879,588.6

Waterbody

Stream:	Juanita Cr
Tributary To:	Lake Washington
WRIA:	08.0230
River Mile:	-999.99
Fish Use Potential:	Yes
FUP Criteria:	Biological

General Location

Road Name:	I-405
Mile Post:	21.94
County:	King
WDFW Region:	4

Owner

Type:	State
Name:	Washington State Department of Transportation

PI Species

- ☑ Sockeye
- ☐ Pink
- ☐ Chum
- ☐ Chinook
- ☑ Coho
- ☑ Steelhead
- ☑ Sea Run Cutthroat
- ☑ Resident Trout
- ☐ Bull Trout

Associated Features

- ☑ Culvert
- ☐ Non-Culvert Xing
- ☐ Dam
- ☐ Other
- ☐ Natural Barrier
- ☐ Fishway
- ☐ Diversion

Location/Directions

Easiest to locate from 145th off of Juanita-Woodinville Way, 14th deadends near the DS end of pipe.

Site Comments

Stormwater retention pond at US end of culvert.

11/20/2021

WDFW Fish Passage and Diversion Screening Inventory Database

Level A Culvert Assessment Report

Site ID:	**998602**				
Latitude:	**47.732951092**	Stream:	**Juanita Cr**	WRIA:	**08.0230**
Longitude:	**-122.188992351**	Tributary To:	**Lake Washington**	Fish Use Potential:	**Yes**

Data Source: Washington Department of Fish and Wildlife

Field Crew: Gatchell; Stilwater   Review Date: 8/29/2012

Culvert Details / **Level A Parameters**

ID	Shape	Material	Span	Rise	Length	WDIC	Apron	WSDrop	Location	Countersunk	Backwater	Slope (%)	Sediment
1.1	RND	CST	1.22	1.22	109.30	0.03	NO	0.80	Outlet	No	No	3.00	

All dimensions in meters

Channel Description

Toe Width (m):	0.65
Average Width (m):	4.90
Culvert/Stream Width Ratio:	0.25

Plunge Pool

Length (m):	-999.99
Max Depth (m):	-99.99
OHW Width (m):	-999.99

Road

Fill Depth (m):	12.00

Assessment Results

		Tidal Influence:		Tidegate Present:	No
Barrier:	Yes	Passability (%):	0	Method:	Level A
Reason:	WS Drop	Fishway Present:	No	Recheck:	

Comments

US end there is a stormcontrol standpipe that could serve as a dam under high flows. Outfalls onto boulder, no plunge pool. Swinging gate at the DS end with stand pipe and small hydrauilc hose that runs under the pipe. Overlflow pipe (.76PCC) on LB

Potential Habitat Gain

Survey Type:	RSFS	Spawning (sq m):	294	Length (m):	399
Significant Reach:	Yes	Rearing (sq m):	269	**PI Total**	17.73

11/20/2021

WDFW Fish Passage and Diversion Screening Inventory Database

Habitat Survey Summary Report

Site ID:	**998602**				
Latitude:	**47.732951092**	Longitude:	**-122.188992351**	WRIA:	**08.0230**
Stream:	**Juanita Cr**	Tributary To:	**Lake Washington**	PI Total:	**17.73**

Survey Type: RSFS

Spreadsheet File(s):

998602.xls

Downstream Survey

Date: 8/29/2012 Crew: Gatchell; Stilwater Length (m): 5,048

Downstream Comments:

Creek is impacted by urban development, flows behind houses. Substrates are imbedded in places. Canopy deciduous trees, shrubs, blackberry, 60-90% cover. There are no less than three DS barriers and three unknown passability culverts.

Upstream Survey

Date: 8/29/2012 Crew: Gatchell; Stilwater Length (m): 399

Upstream Comments:

US of stormwater storage pond, flows through a beautiful forested ravine. Substrate mostly sand/gravel, gradient 1-2%. Passes under 2 wood footbridges. Overall a nice looking salmonid stream.

Potential Habitat Gain

Lineal (m): 399

Spawning Area (sq m): 294

Rearing Area (sq m): 269

Distribution

- (●) Anadromous
- ( ) Resident Only
- ( ) Unknown

Gain Direction (Resident Only):

Potential Species Benefit

☑ Sockeye / Kokanee	☐ Chinook	☑ Searun Cutthroat
☐ Pink	☑ Coho	☑ Resident Trout
☐ Chum	☑ Steelhead	☐ Bull Trout

11/20/2021

WDFW Fish Passage and Diversion Screening Inventory Database

Barrier Priority Index Report

Site ID: **998602**

Stream Juanita Cr **Trib To** Lake Washington **WRIA** 08.0230

Habitat (H) Estimatiom Method RSFS

	B	H	M	D	C	Species PI
Sockeye	1	409	2	1	1	7.04
Pink			2		1	0.00
Chum			2		1	0.00
Coho	1	369	2	1	1	2.46
Chinook	1	349	2	3	1	2.41
Steelhead	1	369	2	3	1	1.47
Searun Cutthroat	1	369	2	1	1	2.29
Resident Trout	1	448	1	1	1	2.06
Dolly/Bull Trout					1	0.00
					TOTAL PI	**17.73**

B = proportion of fish passage improvement (1, 0.67, 0.33).

H = potential habitat gain (square meters), spawning habitat for sockeye, pink and chum, rearing habitat for the rest.

M= mobility modifier (anadromous = 2, resident = 1).

D = stock condition modifier (critical = 3, depressed = 2, not 2 or 3 = 1).

C= repair cost modifier (<$100K = 3, $100K - $500K = 2, >$500K = 1).

11/20/2021

WDFW Fish Passage and Diversion Screening Inventory Database

Image Report - Active

Site ID:	**998602**				
Latitude:	**47.732951092**	Stream:	**Juanita Cr**	WRIA:	**08.0230**
Longitude:	**-122.188992351**	Tributary To:	**Lake Washington**	Fish Use Potential:	**Yes**

Associated Features

- [x] Culvert
- [ ] Dam
- [ ] Natural Barrier
- [ ] Diversion
- [ ] Non-Culvert Xing
- [ ] Other
- [ ] Fishway

11/20/2021

These data represent a snapshot of the Washington Department of Fish and Wildlife's current records. Due to the ongoing nature of assessment and inventory of these features, these data may not accurately represent conditions on the ground, and are subject to change.

Appendix K: Scour Calculations

Hydraulic Analysis Report

Project Data

Project Title: I-405 Brickyard – Juantia Creek Fish Passage Project

Designer: Yacoub Raheem, Curtis Jones

Project Date: Thursday, June 6, 2024

Project Units: U.S. Customary Units

Notes:

Bridge Scour Analysis:Bridge Scour Analysis

Notes:

Scenario: 2yr_Final

Contraction Scour Summary

Contraction & Long Term Scour is applied method due to greater scour.

Live Bed Contraction Scour Depth 0.22 ft

Applied Contraction Scour Elevation with LTD 0.22 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 217.39 ft

Right Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 217.39 ft

Main Channel Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.92 ft

D50 (D50): 177.698400 mm

Average Velocity Upstream (V): 3.19 ft/s

Computed Contraction Scour Condition: Clear Water

Input Parameters for Clear Water

Discharge in Contracted Section (Q): 39.70 cfs

Bottom Width in Contracted Section (W): 11.69 ft

Depth Prior to Scour in Contracted Section (y0): 0.82 ft

Input Parameters for Live Bed

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section (S1): 0.041260 ft/ft

Discharge in Contracted Section (Q2): 39.70 cfs

Discharge Upstream that is Transporting Sediment (Q1): 35.14 cfs

Bottom Width in Contracted Section (W2): 11.69 ft

Width Upstream that is Transporting Sediment (W1): 11.91 ft

Depth Prior to Scour in Contracted Section (y0): 0.82 ft

Unit Weight of Water (gamma w): 62.40 lb/ft^3

Unit Weight of Sediment (gamma s): 165.00 lb/ft^3

Results

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported (Vc): 9.21 ft/s

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material (Dm): 222.123000 mm

Average Depth in Contracted Section after Scour (y2): 0.39 ft

Scour Depth (ys): -0.43 ft

Results of Live Bed Method

k1 (k1): 0.64

Shear Velocity (V*): 1.11 ft/s

Fall Velocity (V*): 1.64 ft/s

Scour Depth (ys): 0.22 ft

Shear Applied to Bed by Live-Bed Scour (theta 0): 0.2954 lb/ft^2

Shear Required for Movement of D50 Particle (Tau c): 2.3328 lb/ft^2

Recommendations

Recommended Scour Condition: Clear Water

Recommended Scour Depth: 0.00 ft

Left Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.14 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 0.24 ft/s

Results

More input is required for complete calculations

Right Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.54 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 1.22 ft/s

Results

More input is required for complete calculations

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 91.49 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 2.95 cfs

Unit Discharge in the Constricted Area (q2): 3.40 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 0.92 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Right Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 88.51 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 2.95 cfs

Unit Discharge in the Constricted Area (q2): 3.40 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 0.92 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Scenario: 10yr_Final

Contraction Scour Summary

Contraction & Long Term Scour is applied method due to greater scour.

Live Bed Contraction Scour Depth 0.28 ft

Applied Contraction Scour Elevation with LTD 0.28 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 217.69 ft

Right Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 217.69 ft

Main Channel Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 1.22 ft

D50 (D50): 177.698400 mm

Average Velocity Upstream (V): 3.80 ft/s

Computed Contraction Scour Condition: Clear Water

Input Parameters for Clear Water

Discharge in Contracted Section (Q): 62.00 cfs

Bottom Width in Contracted Section (W): 11.69 ft

Depth Prior to Scour in Contracted Section (y0): 1.09 ft

Input Parameters for Live Bed

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section (S1): 0.040541 ft/ft

Discharge in Contracted Section (Q2): 62.00 cfs

Discharge Upstream that is Transporting Sediment (Q1): 55.10 cfs

Bottom Width in Contracted Section (W2): 11.69 ft

Width Upstream that is Transporting Sediment (W1): 11.91 ft

Depth Prior to Scour in Contracted Section (y0): 1.09 ft

Unit Weight of Water (gamma w): 62.40 lb/ft^3

Unit Weight of Sediment (gamma s): 165.00 lb/ft^3

Results

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported (Vc): 9.64 ft/s

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material (Dm): 222.123000 mm

Average Depth in Contracted Section after Scour (y2): 0.57 ft

Scour Depth (ys): -0.52 ft

Results of Live Bed Method

k1 (k1): 0.64

Shear Velocity (V*): 1.26 ft/s

Fall Velocity (V*): 1.64 ft/s

Scour Depth (ys): 0.28 ft

Shear Applied to Bed by Live-Bed Scour (theta 0): 0.4583 lb/ft^2

Shear Required for Movement of D50 Particle (Tau c): 2.3328 lb/ft^2

Recommendations

Recommended Scour Condition: Clear Water

Recommended Scour Depth: 0.00 ft

Left Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.27 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 1.00 ft/s

Results

More input is required for complete calculations

Right Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.63 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 1.83 ft/s

Results

More input is required for complete calculations

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 91.44 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 4.63 cfs

Unit Discharge in the Constricted Area (q2): 5.30 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.22 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Right Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 88.56 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 4.63 cfs

Unit Discharge in the Constricted Area (q2): 5.30 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.22 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Scenario: 25yr_Final

Contraction Scour Summary

Contraction & Long Term Scour is applied method due to greater scour.

Live Bed Contraction Scour Depth 0.29 ft

Applied Contraction Scour Elevation with LTD 0.29 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 217.84 ft

Right Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 217.84 ft

Main Channel Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 1.37 ft

D50 (D50): 177.698400 mm

Average Velocity Upstream (V): 4.11 ft/s

Computed Contraction Scour Condition: Clear Water

Input Parameters for Clear Water

Discharge in Contracted Section (Q): 74.66 cfs

Bottom Width in Contracted Section (W): 11.69 ft

Depth Prior to Scour in Contracted Section (y0): 1.22 ft

Input Parameters for Live Bed

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section (S1): 0.040967 ft/ft

Discharge in Contracted Section (Q2): 74.66 cfs

Discharge Upstream that is Transporting Sediment (Q1): 66.92 cfs

Bottom Width in Contracted Section (W2): 11.69 ft

Width Upstream that is Transporting Sediment (W1): 11.91 ft

Depth Prior to Scour in Contracted Section (y0): 1.22 ft

Unit Weight of Water (gamma w): 62.40 lb/ft^3

Unit Weight of Sediment (gamma s): 165.00 lb/ft^3

Results

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported (Vc): 9.83 ft/s

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material (Dm): 222.123000 mm

Average Depth in Contracted Section after Scour (y2): 0.67 ft

Scour Depth (ys): -0.56 ft

Results of Live Bed Method

k1 (k1): 0.64

Shear Velocity (V*): 1.34 ft/s

Fall Velocity (V*): 1.64 ft/s

Scour Depth (ys): 0.29 ft

Shear Applied to Bed by Live-Bed Scour (theta 0): 0.5548 lb/ft^2

Shear Required for Movement of D50 Particle (Tau c): 2.3328 lb/ft^2

Recommendations

Recommended Scour Condition: Clear Water

Recommended Scour Depth: 0.00 ft

Left Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.26 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 1.26 ft/s

Results

More input is required for complete calculations

Right Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.82 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 2.18 ft/s

Results

More input is required for complete calculations

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 91.24 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 5.62 cfs

Unit Discharge in the Constricted Area (q2): 6.39 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.37 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Right Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 88.76 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 5.62 cfs

Unit Discharge in the Constricted Area (q2): 6.39 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.37 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Scenario: 100yr_Final

Contraction Scour Summary

Contraction & Long Term Scour is applied method due to greater scour.

Live Bed Contraction Scour Depth 0.32 ft

Applied Contraction Scour Elevation with LTD 0.32 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 218.07 ft

Right Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 218.07 ft

Main Channel Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 1.60 ft

D50 (D50): 177.698400 mm

Average Velocity Upstream (V): 4.57 ft/s

Computed Contraction Scour Condition: Clear Water

Input Parameters for Clear Water

Discharge in Contracted Section (Q): 95.85 cfs

Bottom Width in Contracted Section (W): 11.69 ft

Depth Prior to Scour in Contracted Section (y0): 1.44 ft

Input Parameters for Live Bed

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section (S1): 0.040659 ft/ft

Discharge in Contracted Section (Q2): 95.85 cfs

Discharge Upstream that is Transporting Sediment (Q1): 87.02 cfs

Bottom Width in Contracted Section (W2): 11.69 ft

Width Upstream that is Transporting Sediment (W1): 11.91 ft

Depth Prior to Scour in Contracted Section (y0): 1.44 ft

Unit Weight of Water (gamma w): 62.40 lb/ft^3

Unit Weight of Sediment (gamma s): 165.00 lb/ft^3

Results

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported (Vc): 10.09 ft/s

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material (Dm): 222.123000 mm

Average Depth in Contracted Section after Scour (y2): 0.83 ft

Scour Depth (ys): -0.61 ft

Results of Live Bed Method

k1 (k1): 0.64

Shear Velocity (V*): 1.45 ft/s

Fall Velocity (V*): 1.64 ft/s

Scour Depth (ys): 0.32 ft

Shear Applied to Bed by Live-Bed Scour (theta 0): 0.7168 lb/ft^2

Shear Required for Movement of D50 Particle (Tau c): 2.3328 lb/ft^2

Recommendations

Recommended Scour Condition: Clear Water

Recommended Scour Depth: 0.00 ft

Left Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.47 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 1.74 ft/s

Results

More input is required for complete calculations

Right Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.85 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 2.56 ft/s

Results

More input is required for complete calculations

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.74 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 7.31 cfs

Unit Discharge in the Constricted Area (q2): 8.20 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.60 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Right Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 89.26 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 7.31 cfs

Unit Discharge in the Constricted Area (q2): 8.20 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.60 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Scenario: 2080_100yr_Final

Contraction Scour Summary

Contraction & Long Term Scour is applied method due to greater scour.

Live Bed Contraction Scour Depth 0.33 ft

Applied Contraction Scour Elevation with LTD 0.33 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 1.61 ft

Total Scour at Abutment 1.61 ft

Total Scour Elevation at Abutment 216.56 ft

Right Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.00 ft

Total Scour at Abutment 0.00 ft

Total Scour Elevation at Abutment 218.23 ft

Main Channel Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 1.76 ft

D50 (D50): 177.698400 mm

Average Velocity Upstream (V): 4.87 ft/s

Computed Contraction Scour Condition: Clear Water

Input Parameters for Clear Water

Discharge in Contracted Section (Q): 110.98 cfs

Bottom Width in Contracted Section (W): 11.69 ft

Depth Prior to Scour in Contracted Section (y0): 1.58 ft

Input Parameters for Live Bed

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section (S1): 0.040843 ft/ft

Discharge in Contracted Section (Q2): 110.98 cfs

Discharge Upstream that is Transporting Sediment (Q1): 101.78 cfs

Bottom Width in Contracted Section (W2): 11.69 ft

Width Upstream that is Transporting Sediment (W1): 11.91 ft

Depth Prior to Scour in Contracted Section (y0): 1.58 ft

Unit Weight of Water (gamma w): 62.40 lb/ft^3

Unit Weight of Sediment (gamma s): 165.00 lb/ft^3

Results

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported (Vc): 10.25 ft/s

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material (Dm): 222.123000 mm

Average Depth in Contracted Section after Scour (y2): 0.94 ft

Scour Depth (ys): -0.65 ft

Results of Live Bed Method

k1 (k1): 0.64

Shear Velocity (V*): 1.52 ft/s

Fall Velocity (V*): 1.64 ft/s

Scour Depth (ys): 0.33 ft

Shear Applied to Bed by Live-Bed Scour (theta 0): 0.8333 lb/ft^2

Shear Required for Movement of D50 Particle (Tau c): 2.3328 lb/ft^2

Recommendations

Recommended Scour Condition: Clear Water

Recommended Scour Depth: 0.00 ft

Left Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.63 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 2.09 ft/s

Results

More input is required for complete calculations

Right Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 1.03 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 2.89 ft/s

Results

More input is required for complete calculations

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.46 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 8.55 cfs

Unit Discharge in the Constricted Area (q2): 9.49 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.76 ft

Flow Depth Prior to Scour: 0.05 ft

Result Parameters

q2/q1: 1.11

Average Velocity Upstream: 4.87 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 10.25 ft/s

Scour Condition: Clear Water

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.64

Flow Depth including Contraction Scour: 1.01 ft

Maximum Flow Depth including Abutment Scour: 1.66 ft

Scour Hole Depth from NCHRP Method: 1.61 ft

Right Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 89.54 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 8.55 cfs

Unit Discharge in the Constricted Area (q2): 9.49 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.76 ft

Flow Depth Prior to Scour: -1.00 ft

Result Parameters

q2/q1: 0.00

Average Velocity Upstream: 0.00 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 0.00 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 0.00

Flow Depth including Contraction Scour: 0.00 ft

Maximum Flow Depth including Abutment Scour: 0.00 ft

Scour Hole Depth from NCHRP Method: 0.00 ft

Scenario: 500yr_Final

Contraction Scour Summary

Contraction & Long Term Scour is applied method due to greater scour.

Live Bed Contraction Scour Depth 0.35 ft

Applied Contraction Scour Elevation with LTD 0.35 ft

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 1.68 ft

Total Scour at Abutment 1.68 ft

Total Scour Elevation at Abutment 216.54 ft

Right Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 1.84 ft

Total Scour at Abutment 1.84 ft

Total Scour Elevation at Abutment 216.54 ft

Main Channel Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 1.98 ft

D50 (D50): 177.698400 mm

Average Velocity Upstream (V): 5.27 ft/s

Computed Contraction Scour Condition: Clear Water

Input Parameters for Clear Water

Discharge in Contracted Section (Q): 133.61 cfs

Bottom Width in Contracted Section (W): 11.69 ft

Depth Prior to Scour in Contracted Section (y0): 1.79 ft

Input Parameters for Live Bed

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section (S1): 0.040138 ft/ft

Discharge in Contracted Section (Q2): 133.61 cfs

Discharge Upstream that is Transporting Sediment (Q1): 124.43 cfs

Bottom Width in Contracted Section (W2): 11.69 ft

Width Upstream that is Transporting Sediment (W1): 11.91 ft

Depth Prior to Scour in Contracted Section (y0): 1.79 ft

Unit Weight of Water (gamma w): 62.40 lb/ft^3

Unit Weight of Sediment (gamma s): 165.00 lb/ft^3

Results

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported (Vc): 10.46 ft/s

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material (Dm): 222.123000 mm

Average Depth in Contracted Section after Scour (y2): 1.10 ft

Scour Depth (ys): -0.69 ft

Results of Live Bed Method

k1 (k1): 0.64

Shear Velocity (V*): 1.60 ft/s

Fall Velocity (V*): 1.64 ft/s

Scour Depth (ys): 0.35 ft

Shear Applied to Bed by Live-Bed Scour (theta 0): 1.0077 lb/ft^2

Shear Required for Movement of D50 Particle (Tau c): 2.3328 lb/ft^2

Recommendations

Recommended Scour Condition: Clear Water

Recommended Scour Depth: 0.00 ft

Left Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 0.79 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 2.54 ft/s

Results

More input is required for complete calculations

Right Bank Contraction Scour

Computation Type: Clear-Water and Live-Bed Scour

Input Parameters

Input Parameters for Scour Condition

Average Depth Upstream of Contraction (y1): 1.31 ft

D50 (D50): 0.000000 mm

Average Velocity Upstream (V): 3.27 ft/s

Results

More input is required for complete calculations

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 90.29 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 10.45 cfs

Unit Discharge in the Constricted Area (q2): 11.43 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.98 ft

Flow Depth Prior to Scour: 0.23 ft

Result Parameters

q2/q1: 1.09

Average Velocity Upstream: 5.27 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 10.46 ft/s

Scour Condition: Clear Water

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.61

Flow Depth including Contraction Scour: 1.19 ft

Maximum Flow Depth including Abutment Scour: 1.91 ft

Scour Hole Depth from NCHRP Method: 1.68 ft

Right Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment with wing wall

Angle of Embankment to Flow: 89.71 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 10.45 cfs

Unit Discharge in the Constricted Area (q2): 11.43 cfs/ft

D50: 177.698400 mm

Upstream Flow Depth: 1.98 ft

Flow Depth Prior to Scour: 0.06 ft

Result Parameters

q2/q1: 1.09

Average Velocity Upstream: 5.27 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 10.46 ft/s

Scour Condition: Clear Water

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.61

Flow Depth including Contraction Scour: 1.19 ft

Maximum Flow Depth including Abutment Scour: 1.91 ft

Scour Hole Depth from NCHRP Method: 1.84 ft

Scour Summary Table

Long Term Degradation

Contraction Scour

Parameter	2yr_Final	10yr_Final	25yr_Final	100yr_Final	2080_100yr_Final	500yr_Final	Units	Notes
Selected Contraction Computation Method	Clear-Water and Live-Bed Scour	Clear-Water and Live-Bed Scour	Clear-Water and Live-Bed Scour	Clear-Water and Live-Bed Scour	Clear-Water and Live-Bed Scour	Clear-Water and Live-Bed Scour		
Applied Contraction Scour Depth	0.00	0.00	0.00	0.00	0.00	0.00	ft	Clear Water
Clear Water Contraction Scour Depth	-0.43	-0.52	-0.56	-0.61	-0.65	-0.69	ft	
Live Bed Contraction Scour Depth	0.22	0.28	0.29	0.32	0.33	0.35	ft	
Streamb	216.47	216.47	216.47	216.47	216.47	216.47	ft	prior to

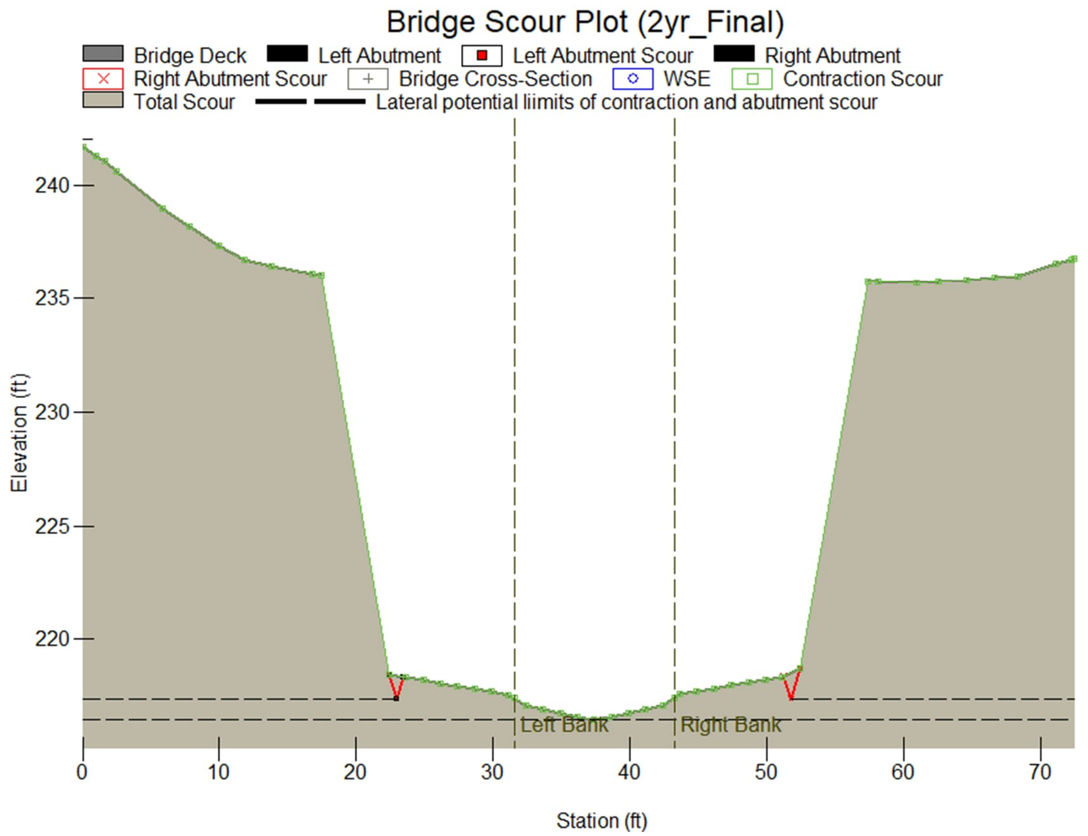
ed Thalweg Elevation								any scour
Applied Contraction Scour Elevation with LTD	216.47	216.47	216.47	216.47	216.47	216.47	ft	

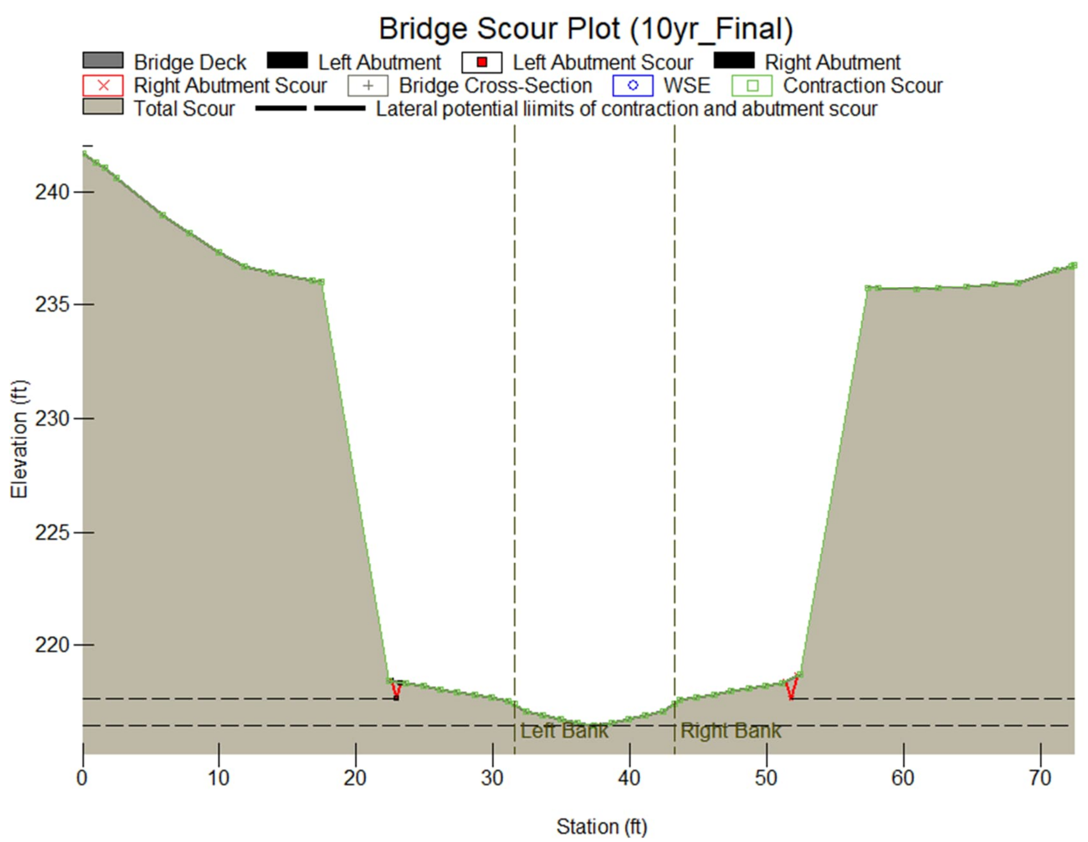
Local Scour at Piers

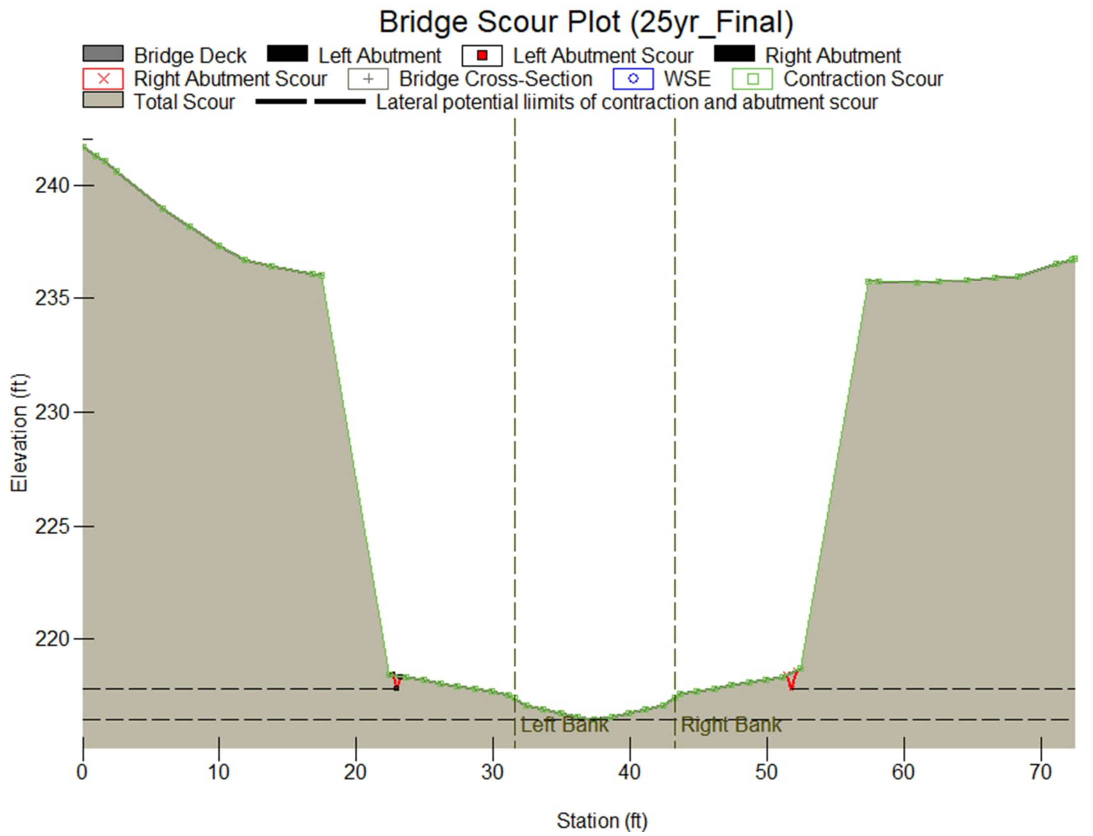
Local Scour at Abutments

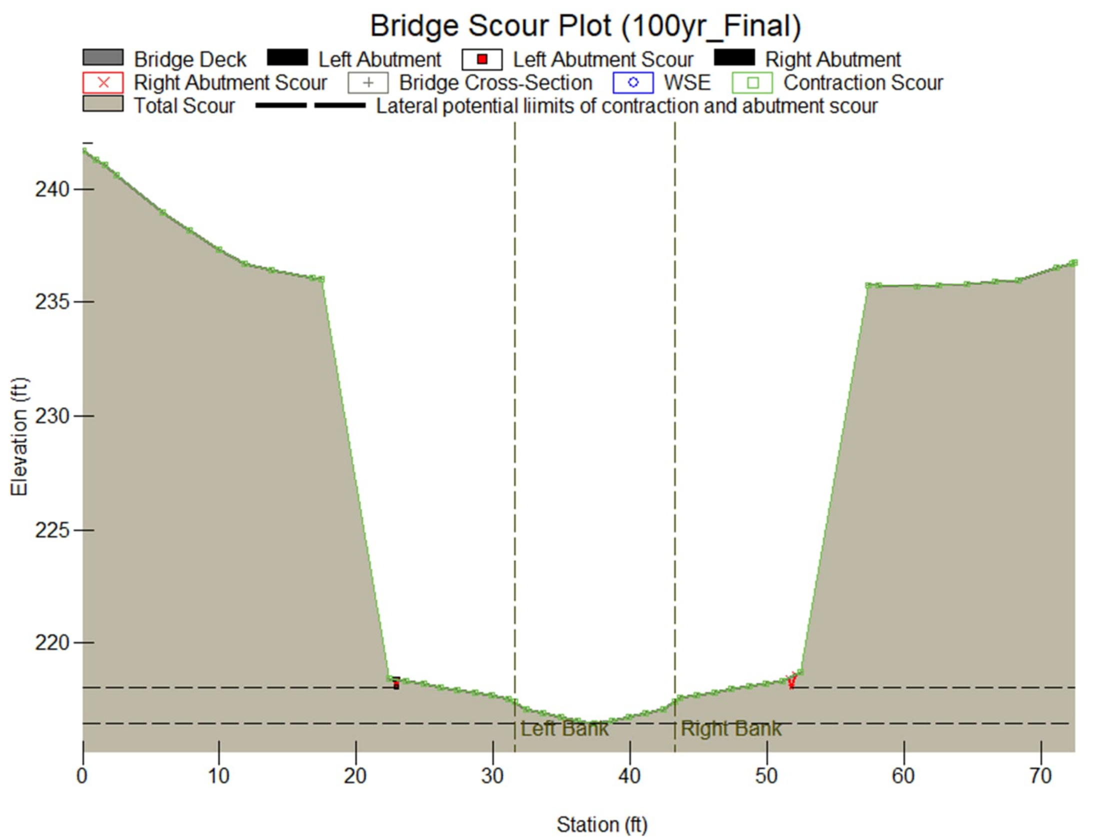
Parameter	2yr_Final	10yr_Final	25yr_Final	100yr_Final	2080_100yr_Final	500yr_Final	Units	Notes
Left Abutment								
Abutment Scour Depth	0.00	0.00	0.00	0.00	1.61	1.68	ft	NCHRP Method: Scour Condition A (includes LTD)
Max Flow Depth including Abutment Scour	0.00	0.00	0.00	0.00	1.66	1.91	ft	Including the long-term scour depth
Total Scour at Abutment	0.00	0.00	0.00	0.00	1.61	1.68	ft	
Local Streambed Elevation at	218.39	218.39	218.39	218.39	218.39	218.39	ft	prior to any scour

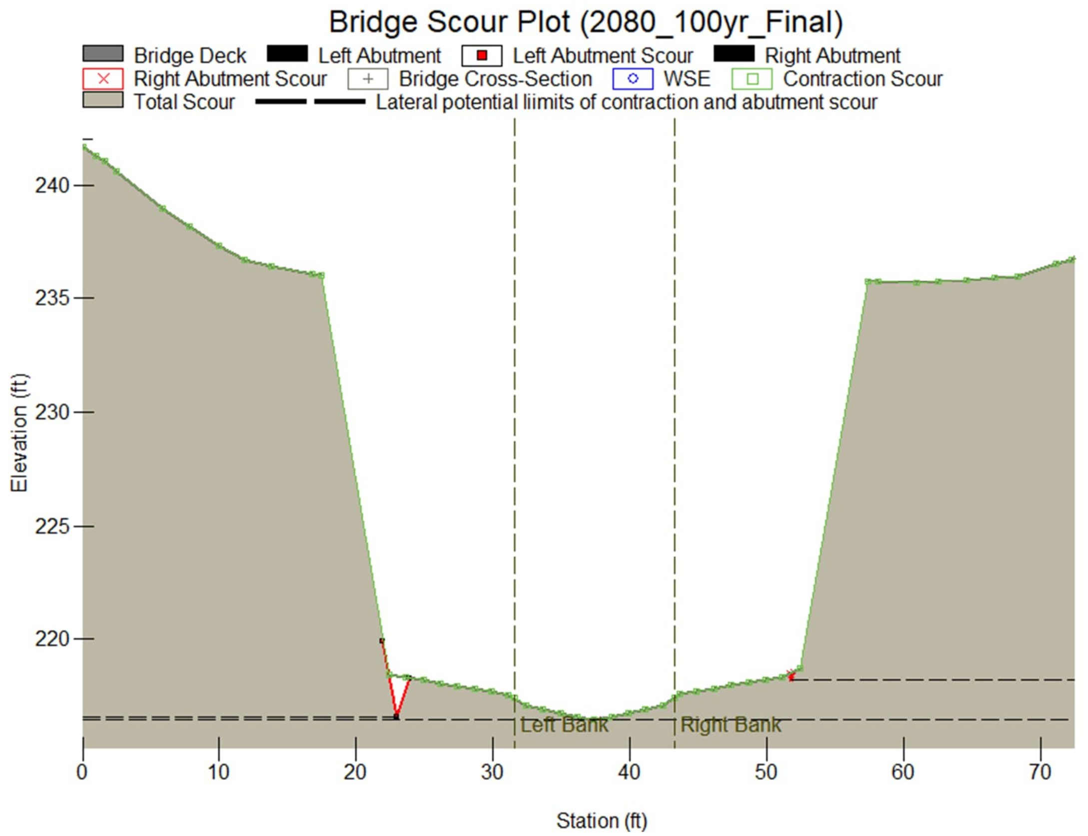
Abutment								
Total Scour Elevation at Abutment	217.39	217.69	217.84	218.07	216.56	216.54	ft	
Right Abutment								
Abutment Scour Depth	0.00	0.00	0.00	0.00	0.00	1.84	ft	NCHRP Method: Scour Condition A (includes LTD)
Max Flow Depth including Abutment Scour	0.00	0.00	0.00	0.00	0.00	1.91	ft	Including the long-term scour depth
Total Scour at Abutment	0.00	0.00	0.00	0.00	0.00	1.84	ft	
Local Streambed Elevation at Abutment	218.54	218.54	218.54	218.54	218.54	218.54	ft	prior to any scour
Total Scour Elevation at Abutment	217.39	217.69	217.84	218.07	218.23	216.54	ft	

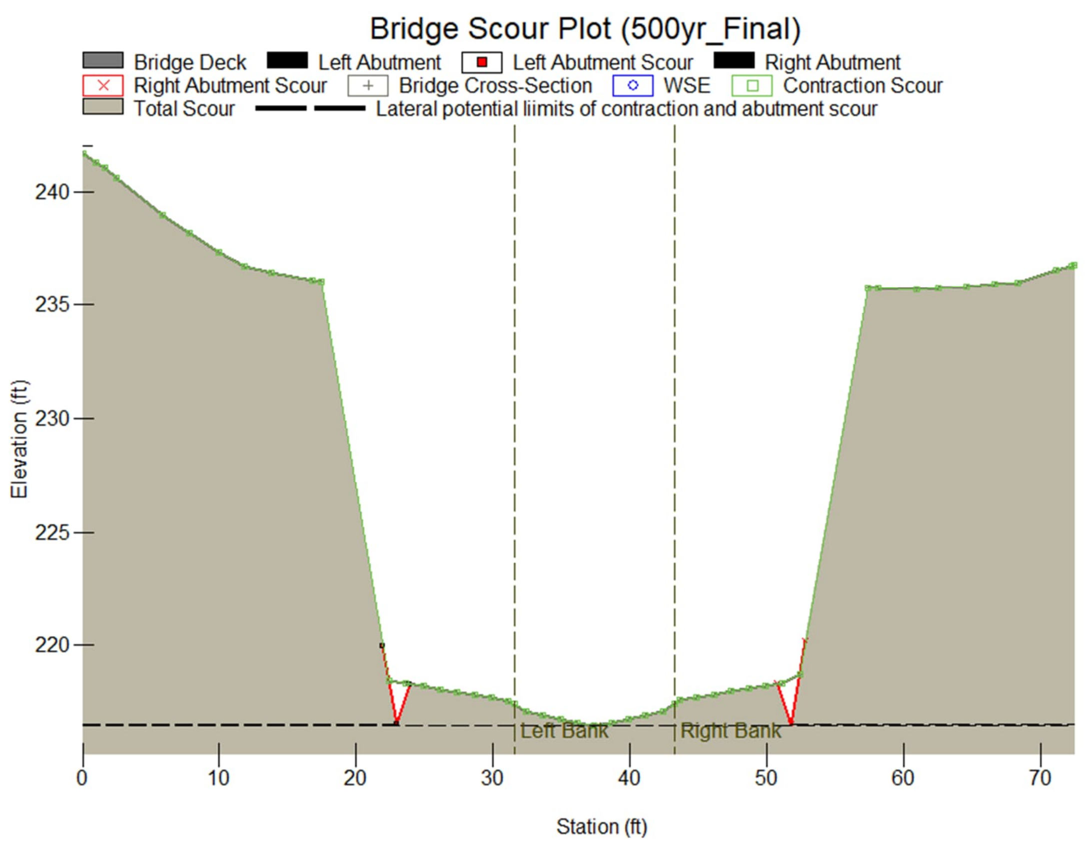

Bridge Scour Plot (2yr_Final)
Bridge Deck
Left Abutment
Left Abutment Scour
Right Abutment
Right Abutment Scour
Bridge Cross-Section
WSE
Contraction Scour
Total Scour
Lateral potential liimits of contraction and abutment scour
Elevation (ft)
240
235
230
225
220
Left Bank
Right Bank
0
10
20
30
40
50
60
70
Station (ft)

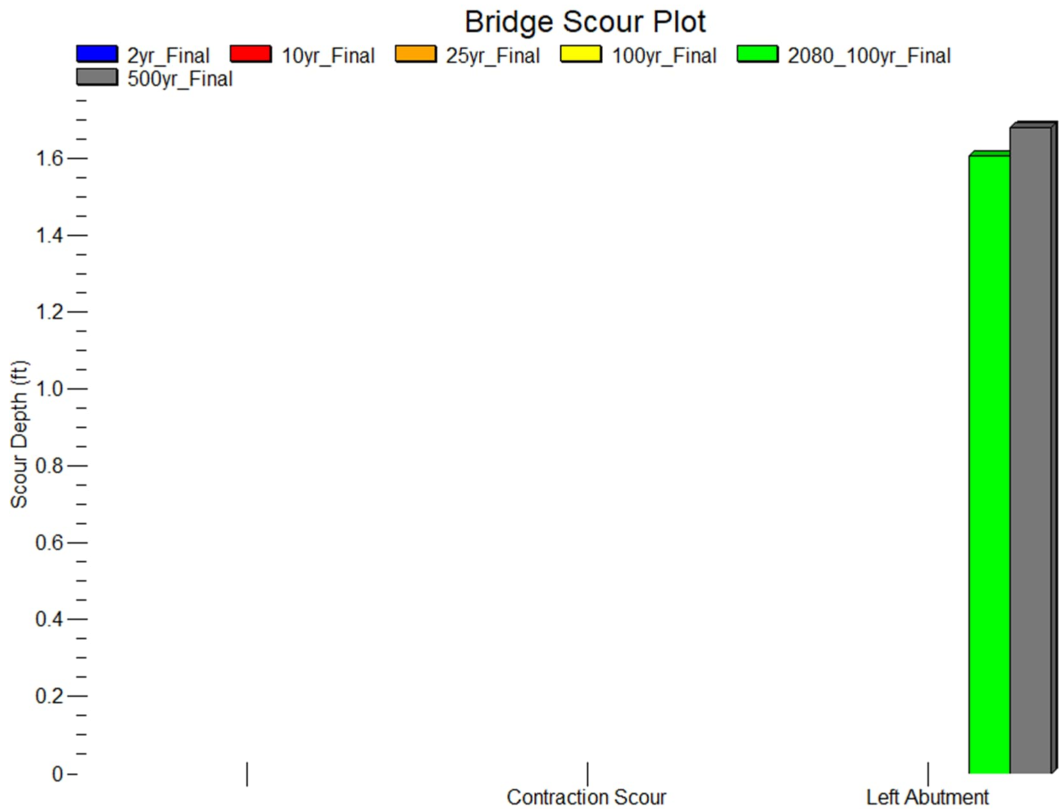

Bridge Scour Plot (10yr_Final)
Bridge Deck
Left Abutment
Left Abutment Scour
Right Abutment
Right Abutment Scour
Bridge Cross-Section
WSE
Contraction Scour
Total Scour
Lateral potential liimits of contraction and abutment scour
Elevation (ft)
240
235
230
225
220
Left Bank
Right Bank
0
10
20
30
40
50
60
70
Station (ft)


Bridge Scour Plot (25yr_Final)
Bridge Deck
Left Abutment
Left Abutment Scour
Right Abutment
Right Abutment Scour
Bridge Cross-Section
WSE
Contraction Scour
Total Scour
Lateral potential liimits of contraction and abutment scour
Elevation (ft)
240
235
230
225
220
Left Bank
Right Bank
0
10
20
30
40
50
60
70
Station (ft)


Bridge Scour Plot (100yr_Final)
Bridge Deck
Left Abutment
Left Abutment Scour
Right Abutment
Right Abutment Scour
Bridge Cross-Section
WSE
Contraction Scour
Total Scour
Lateral potential liimits of contraction and abutment scour
Elevation (ft)
240
235
230
225
220
Left Bank
Right Bank
0
10
20
30
40
50
60
70
Station (ft)


Bridge Scour Plot (2080_100yr_Final)
Bridge Deck
Left Abutment
Left Abutment Scour
Right Abutment
Right Abutment Scour
Bridge Cross-Section
WSE
Contraction Scour
Total Scour
Lateral potential liimits of contraction and abutment scour
Elevation (ft)
Station (ft)
Left Bank
Right Bank


Bridge Scour Plot (500yr_Final)
Bridge Deck
Left Abutment
Left Abutment Scour
Right Abutment
Right Abutment Scour
Bridge Cross-Section
WSE
Contraction Scour
Total Scour
Lateral potential liimits of contraction and abutment scour
Elevation (ft)
Station (ft)
Left Bank
Right Bank


Bridge Scour Plot
2yr_Final
10yr_Final
25yr_Final
100yr_Final
2080_100yr_Final
500yr_Final
Scour Depth (ft)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
Contraction Scour
Left Abutment

Long-Term Degradation

I-405 MP 21.94 Juanita Creek (WDFW ID): 998602 Preliminary Hydraulic Design Report

Background

The Washington State Department of Transportation (WSDOT) is proposing a project to provide fish passage at the Interstate 405 (I-405) crossing of Juanita Creek at milepost (MP) 21.94. The existing culvert has been identified as a fish barrier and will be replaced by a 30-foot-wide bottomless culvert designed using the Stream Simulation methodology.

Purpose

The objective of this assessment is to provide a quantitative measure of Juanita Creek's potential for long-term degradation.

Data Available

The following is a list of data/references used to prepare this calculation:

- WSDOT Hydraulics Manual (2022)
- Federal Highway Administration (FHWA) HEC-18 & HEC-20 (2012a; 2012b),
- AECOM preliminary streambed profile and geometry.
- USBR Computing Degradation and Local Scour – Technical Guideline for Bureau of Reclamation (Pemberton and Lara 1984)

Methodology

Long-term degradation is quantitatively analyzed using the equilibrium slope method as described below.

Long-term degradation

As described in HEC-18, the procedure to quantify long-term degradation are presented in HEC-20 (FHWA, 2012). Using the Meyer-Peter Muller equilibrium slope analysis equation, the slope at which shear stresses are too low to cause incipient motion of streambed particles is given as:

$$S_{eq} = 60.1\frac{\left(D_{50}^{\left(\frac{10}{7}\right)}\right)\left(n^{\left(\frac{9}{7}\right)}\right)}{\left(D_{90}^{\left(\frac{5}{14}\right)}\right)\left(q^{\left(\frac{6}{7}\right)}\right)} \qquad \text{(Eqn. 1)}$$

where,

$$D_{50} := median\ particle\ size\ (ft)$$

$$D_{90} := 90th\ percintile\ particle\ size\ (ft)$$

$$n := Manning's\ roughness\ coeffecient$$

$$q := channel\ discharge\ per\ unit\ width\ (ft^2/s)$$

Considering the recommended bankfull discharge as an initial estimate – understanding that a range of discharges are responsible for forming the channel – the input parameters are summarized in Table 1; where unit discharge is equal to the 2-yr discharge (45 cfs) divided by the bankfull width (12.08 ft).

Table 1: Equilibrium Slope Input Parameters for Bankfull Discharge at Juanita Creek

Parameter	D_{50}	D_{90}	n	q
Value	*1.01 inches = 0.0841 feet*	*3.21 inches = 0.2675 feet*	*0.065*	3.72 (ft^2/s)

Using equation 1, the equilibrium slope is 0.0270 ft/ft or 2.7%. This is checked by estimating the equilibrium slope using the Schoklitsh method. Inputs and the resulting equilibrium slope of 0.74% are shown in Table 2.

Table 2: Equilibrium Slope Inputs and Results for the Schoklitsh Method

Schoklitsh Method	Value	Comment
Reference: Pemberton & Lara, 1984		
K_s	0.00174	Constant
D	25.65	mm. Mean sediment bed diameter.
W_{bf}	12.1	feet. Bankfull width associated with channel-forming discharge.
Q	45	cfs. Dominant discharge.
S_L	**0.00740533**	$= K_s (D * W_{bf} / Q)^{3/4}$. Schoklitch Equilibrium Slope.

Once the equilibrium slope has been determined, the long-term degradation can be estimated using a geometric relationship between the design slope and the equilibrium slope. Assuming there is a base level control downstream that limits the long-term degradation – in this case the confluence of the next downstream tributary limits the applicability of the equilibrium slope due to change in channel characteristics, so this location was chosen as an example base level control. USGS Streamstats estimates watershed areas for Juanita Creek as 1.37 square miles and the tributary as 1.08 square miles which suggests this is location of significant change in channel size and flow. (USGS 2024).

$$Y_s = L(S_{design} - S_{eq}) \qquad \text{(Eqn. 2)}$$

where,

$$L := distance\ upstream\ of\ base\ level\ control\ (ft)$$

$$S_{design} := design\ slope$$

$$S_{eq} := equilibrium\ slope$$

The distance between the assumed base level control and the proposed structure outlet is ~7,100 ft.

Thus, using equation 2, and an upstream distance of 7,100 ft, the total degradation at the proposed inlet is 0 feet because the average of the Meyer-Peter Muller and Schoklitsch methods is above the ground profile as seen in Figure 1. Taking the average of several methods is recommended (Pemberton and Lara 1984).

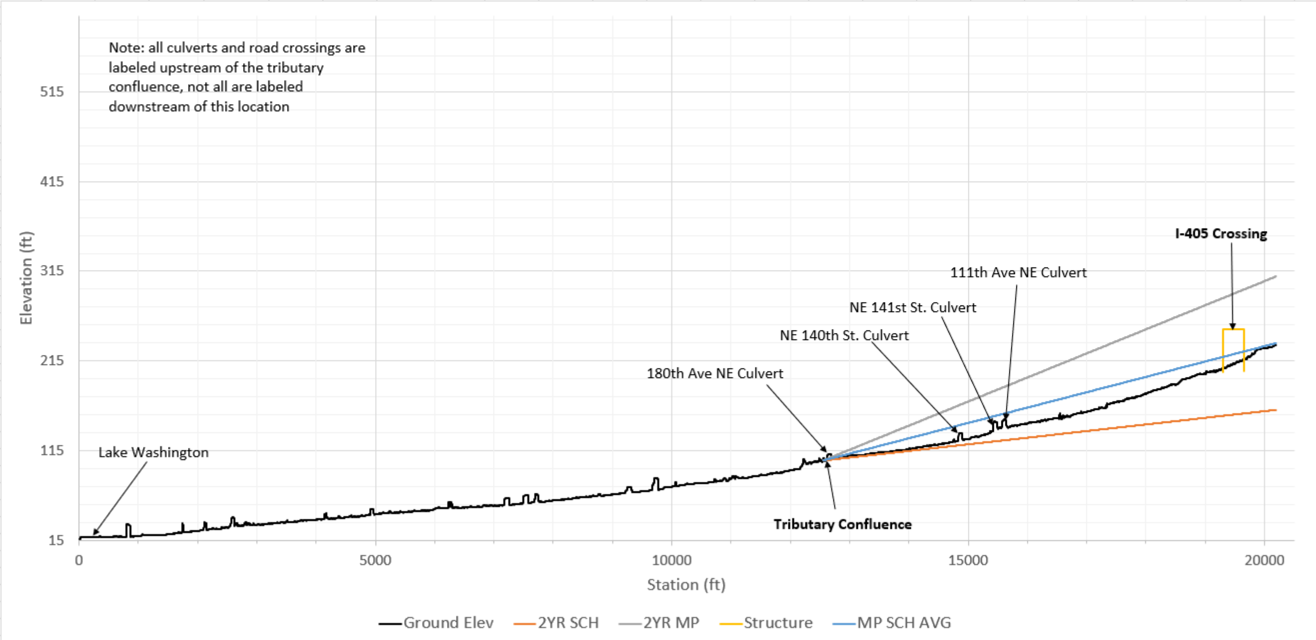


Figure 1. LTD Plot from the Juanita Creek Tributary confluence.

Armoring Depth

The armoring depth calculation results in 0.88 feet for the existing material and 0.14 feet for the proposed material. Calculations are as follows:

DEGRADATION LIMITED BY ARMORING

When the channel bed downstream from a dam contains more than 10 percent coarse material which cannot be transported under dominant flow conditions armoring will in time develop. The formation of an armoring layer at the maximum depth of degradation will depend on such factors as reservoir operations, the amount of armoring material available in the scour depth zone below streambed, and the distance to which this material extends downstream.

There are several ways to compute the size of bed material required for armoring and each method is regarded as a check on the others. Each method computes a different armoring size and some judgment may be required in selecting the lower size limitation of nontransportable material. Reclamation recommends the following methods to determine armoring size:

1. Meyer-Peter, Muller (bedload transport equation)
2. Competent bottom velocity
3. Lane's tractive force theory
4. Shields diagram
5. Yang incipient motion

Juanita Creek pebble count data

Table 3: Sediment properties upstream of project crossing

	Upstream Diameter (in)	Upstream Diameter (in)	Upstream Diameter (in)	Average Diameter (in)
D_{16}	0.35	0.51	0.34	0.38
D_{50}	0.91	1.31	0.94	1.01
D_{84}	1.96	2.59	2.10	2.21
D_{95}	3.48	3.54	3.34	3.47
D_{100}	5.96	5.98	4.95	5.69

mm
9.652
25.654
56.134
88.138
144.526

Meyer-Peter, Muller (Bedload Transport Equation)

Bedload transport equations provide a method to compute a nontransportable particle size representing coarse bed material capable of forming an armoring layer. To describe a nontransportable size, the Meyer-Peter, Muller (1948) bedload equation (Sheppard, 1960) for beginning transport of individual particle sizes, may be applied when rewritten in the form:

$$D_c = \frac{dS}{K\left(\frac{n_s}{D_{90}^{1/6}}\right)^{3/2}} \qquad (2)$$

where:

D_c = Individual particle size in millimeters
K = 0.19 inch-pound units (0.058 metric units)
d = Mean water depth at dominant discharge, ft (m)
S = Slope of energy gradient, ft/ft (m/m)
n_s = Manning's "n" for bed of stream
D_{90} = Particle size in millimeter at which 90 percent of bed material by weight is finer

K (in-lb)	0.19
d (ft)	1
S (ft/ft)	0.024
Ns	0.065
D90 (mm)	88.138
Dc (mm)	23.35492

Competent Bottom Velocity

Investigations show that the size of a particle plucked from a streambed is proportional to the velocity of flow near the bed. The particle starts to move at what is called the competent bottom velocity (Mavis and Laushey, 1948) which is approximately 0.7 times V_m, the mean channel velocity. The competent bottom velocity method for determining armoring size is computed from a relationship between mean channel velocity with armoring size by the equation:

$$D_c = 1.88\ V_m^2 \text{ inch-pound units} \qquad (3)$$

$$D_c = 20.2\ V_m^2 \text{ metric units}$$

where:

D_c = Armor size, mm
V_m = Mean channel velocity, ft/s (m/s)

Vm (ft/s)	2.23
Dc (mm)	9.349052

Lane's Tractive Force

The tractive force method is based on the results of a study by Lane (1952). He summarized the results of many studies in a relationship of critical tractive force versus the mean particle size diameter in millimeters, which is reproduced on figure 4. This method entails computing the critical tractive force (equation 4) using the channel hydraulics for dominant discharge. By selecting an appropriate curve on figure 4, usually the recommended set of "curves for canals with clear water in coarse noncohesive material," a critical tractive force gives the lower size limit of the nontransportable material, D_c.

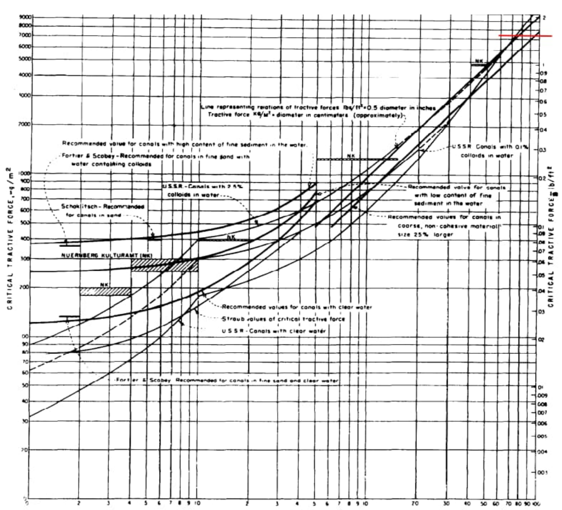
$$\tau_c = \tau_w dS \qquad (4)$$

where:

τ_c = Critical tractive force, lb/ft² (g/m²)
τ_w = Specific weight)(mass) of water, 62.4 lb/ft³ (1 t/m³)
d = Mean water depth, ft (m)
S = Slope, ft/ft (m/m)

Ywlb/ft^3	62.4
d(ft)	1
S(ft/ft)	0.024
Tc	1.4976
Dc(mm)	

N/A, recommended "canals with clear water" curve falls outside of the range of critical tractive force



Shields Diagram

Many investigators use the Shields diagram (Shields, 1936), figure 5, to define the initiation of motion for various particle sizes. In the process of armoring of a streambed for predominately gravel size material >1.0 mm and high Reynold's number R*>500, the Shields parameter given below provides a method for determining an armor size.

$$\tau_* = \frac{\tau_c}{\quad} = 0.06 \qquad (5)$$

T*	0.6
Tc	1.4976
Ys(lb/ft^3)	165
Yw(lb/ft^3)	62.4
d (ft)	1
S	0.024
Dc (ft)	0.243274854

$$\tau_* = \frac{\tau_c}{(\gamma_s - \gamma_w)D_c} = 0.06 \qquad (5)$$

(mm)	74.15017544

where:

τ_* = Dimensionless shear stress
τ_c = Critical shear stress = $\gamma_w dS$, lb/ft^2 (t/m^2)
γ_s = Specific weight (mass) of the particle
γ_w = Specific weight (mass) of water
D_c = Diameter of particle

Inch-pound units	Metric units
$\gamma_w = 62.4\ lb/ft^3$	$\gamma_w = 1.0\ t/m^3$
$\gamma_s = 165\ lb/ft^3$	$\gamma_s = 2.65\ t/m^3$
d = depth, ft	d = depth, m
S = slope, ft/ft	S = slope, m/m
D_c = size, ft	D_c = size, m

Inch-pound units

$$D_c = \frac{\gamma_w dS}{0.06\ (\gamma_s - \gamma_w)}$$

Yang Incipient Motion

Yang (1973) developed a relationship between dimensionless critical velocity, V_{cr}/w, and shear velocity Reynold's number, R_*, at incipient motion. Under rough regime conditions where $R_*>70$, the equation for incipient motion which is considered applicable to bed material size larger than about 2 mm by Reclamation is:

$$\frac{V_{cr}}{w} = 2.05 \qquad (6)$$

Vcr (ft/s)	4.83
Dc (ft)	0.153737451
(mm)	46.85917506

where:

V_{cr} = Critical average water velocity at incipient motion, ft/s (m/s)
w = Terminal fall velocity, ft/s (m/s)

The settling velocity by Rubey (1933) for material larger than 2 mm in diameter will approximate the fall velocity by:

$$w = 6.01\ D_c^{1/2} \text{ inch-pound units} \qquad (7)$$

$$w = 3.32\ D_c^{1/2} \text{ metric units}$$

Equations 6 and 7 can be combined to give:

$$D_c = 0.00659\ V_{cr}^2 \text{ inch-pound units} \qquad (8)$$

$$D_c = 0.0216\ V_{cr}^2 \text{ metric units}$$

Depth to Armor and Volume Computations

After determining the size of the material required to armor the streambed, from either an average of the five methods or a judgment decision on the best method, an estimate can be made of the probable vertical degradation before stabilization is reached. The armoring computations assume that an armoring layer will form as shown on figure 6 by the equations:

$$y_a = y - y_d \qquad (9)$$

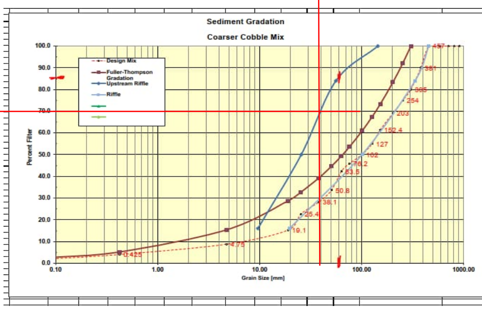
and

$$y_a = (\Delta p)\ y \qquad (10)$$

which are combined to:

$$y_d = y_a\left(\frac{1}{\Delta p} - 1\right) \qquad (11)$$

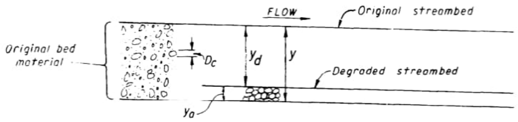
AVG All			
Dc (mm)	38.42833089		
	Ya 0.5 ft	3*D	
Ya (mm)	152.4	115.285	use smaller of values
P	0.3		
Yd (mm)	268.9983162		
Yd(ft)	0.882540408		



Fuller-Thompson Gradation	
Dmax =	12
D[in]	
12.000	100.00
10.000	92.12
8.000	83.32
6.000	73.20
5.000	67.44
4.000	61.00
3.000	53.59
2.500	49.37
2.000	44.65
1.500	39.23
1.000	32.69
0.750	28.72
No. 4 =	15.37
No. 40 =	5.19
No. 200 =	2.38

where:

y_a = Thickness of armoring layer
y = Depth from original streambed to bottom of the armoring layer
y_d = Depth from original streambed to top of armoring layer or the depth of degradation
Δp = Decimal percentage of original bed material larger than the armor size, D_c



y = Depth to bottom of the armoring layer
y_d = Depth of degradation
y_a = Armoring layer
D_c = Diameter of armor material
Δ_p = Decimal percentage of original bed material larger than D_c

Figure 6. - Armoring definition sketch.

The percentage of the bed material equal to or greater than the required armoring size, D_c, can be determined from the bed material size analysis curve from samples collected of the streambed material through the reach involved and at a depth through the anticipated scour zone. This size analysis gives the value Δp to be used in equation 11. The depth, y_a, of the required armoring may vary, depending on the limiting particle size, from a thickness of one particle diameter to three particle diameters or one and three times the armoring size, respectively. A rough guide for use in design is either three armoring particle diameters or 0.5 ft (0.15 m), whichever is smaller. Although armoring has been observed to occur with less than three particle diameters, variability of channel bed material and occurrence of peak discharges dictate the use of a thicker armor layer.

DEGRADATION LIMITED BY ARMORING

When the channel bed downstream from a dam contains more than 10 percent coarse material which cannot be transported under dominant flow conditions armoring will in time develop. The formation of an armoring layer at the maximum depth of degradation will depend on such factors as reservoir operations, the amount of armoring material available in the scour depth zone below streambed, and the distance to which this material extends downstream.

There are several ways to compute the size of bed material required for armoring and each method is regarded as a check on the others. Each method computes a different armoring size and some judgment may be required in selecting the lower size limitation of nontransportable material. Reclamation recommends the following methods to determine armoring size:

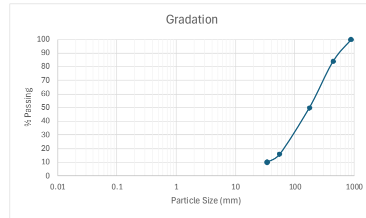
1. Meyer-Peter, Muller (bedload transport equation)
2. Competent bottom velocity
3. Lane's tractive force theory
4. Shields diagram
5. Yang incipient motion

Juanita Creek Proposed Material gradation

Sediment size	Existing Average Diameter (in)	Previously Proposed Riffle-Pool System Diameter from WSDOT PHD (in)	Proposed Diameter for Riffles and Meander Bars Based on Updated Hydraulics (in)
D_{16}	0.38	0.78	2.18
D_{50}	1.01	3.89	6.96
D_{84}	2.21	13.20	17.40
D_{100}	5.69	18.00	34.80

% p	in	mm
10		33.9
16	2.18	55.372
50	6.96	176.784
84	17.4	441.96
100	34.8	883.92

Interpolation	
10	33.94635
90	607.695



Meyer-Peter, Muller (Bedload Transport Equation)

Bedload transport equations provide a method to compute a nontransportable particle size representing coarse bed material capable of forming an armoring layer. To describe a nontransportable size, the Meyer-Peter, Muller (1948) bedload equation (Sheppard, 1960) for beginning transport of individual particle sizes, may be applied when rewritten in the form:

$$D_c = \frac{dS}{K\left(\frac{n_s}{D_{90}^{1/6}}\right)^{3/2}} \qquad (2)$$

where:

D_c = Individual particle size in millimeters
K = 0.19 inch-pound units (0.058 metric units)
d = Mean water depth at dominant discharge, ft (m)
S = Slope of energy gradient, ft/ft (m/m)
n_s = Manning's "n" for bed of stream
D_{90} = Particle size in millimeter at which 90 percent of bed material by weight is finer

K (in-lb)	0.19
d (ft)	0.8
S (ft/ft)	0.03
Ns	0.065
D90 (mm)	607.7
Dc (mm)	37.84512

Competent Bottom Velocity

Investigations show that the size of a particle plucked from a streambed is proportional to the velocity of flow near the bed. The particle starts to move at what is called the competent bottom velocity (Mavis and Laushey, 1948) which is approximately 0.7 times V_m, the mean channel velocity. The competent bottom velocity method for determining armoring size is computed from a relationship between mean channel velocity with armoring size by the equation:

$$D_c = 1.88\ V_m^2 \text{ inch-pound units} \qquad (3)$$

$$D_c = 20.2\ V_m^2 \text{ metric units}$$

where:

D_c = Armor size, mm
V_m = Mean channel velocity, ft/s (m/s)

Vm (ft/s)	2.37
Dc (mm)	10.55977

Lane's Tractive Force

The tractive force method is based on the results of a study by Lane (1952). He summarized the results of many studies in a relationship of critical tractive force versus the mean particle size diameter in millimeters, which is reproduced on figure 4. This method entails computing the critical tractive force (equation 4) using the channel hydraulics for dominant discharge. By selecting an appropriate curve on figure 4, usually the recommended set of "curves for canals with clear water in coarse noncohesive material," a critical tractive force gives the lower size limit of the nontransportable material, D_c.

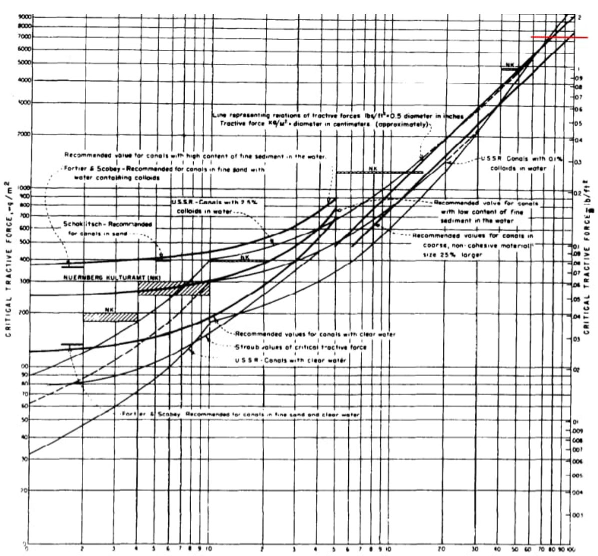
$$T_c = T_w dS \qquad (4)$$

where:

T_c = Critical tractive force, lb/ft² (g/m²)
T_w = Specific weight)(mass) of water, 62.4 lb/ft³ (1 t/m³)
d = Mean water depth, ft (m)
S = Slope, ft/ft (m/m)

Ywlb/ft^3	62.4
d(ft)	0.8
S(ft/ft)	0.03
Tc	
Dc(mm)	

N/A, recommended "canals with clear water" curve falls outside of the range of critical tractive force



<u>Shields Diagram</u>

Many investigators use the Shields diagram (Shields, 1936), figure 5, to define the initiation of motion for various particle sizes. In the process of armoring of a streambed for predominately gravel size material >1.0 mm and high Reynold's number R*>500, the Shields parameter given below provides a method for determining an armor size.

$$\tau_* = \frac{\tau_c}{(\tau_s - \tau_w)D_c} = 0.06 \qquad (5)$$

where:

τ_* = Dimensionless shear stress
τ_c = Critical shear stress = $\tau_w dS$, lb/ft^2 (t/m^2)
τ_s = Specific weight (mass) of the particle
τ_w = Specific weight (mass) of water
D_c = Diameter of particle

Inch-pound units	Metric units
τ_w = 62.4 lb/ft^3	τ_w = 1.0 t/m^3
τ_s = 165 lb/ft^3	τ_s = 2.65 t/m^3
d = depth, ft	d = depth, m
S = slope, ft/ft	S = slope, m/m
D_c = size, ft	D_c = size, m

T*	0.06
Tc	1.4976
Ys(lb/ft^3)	165
Yw(lb/ft^3)	62.4
d (ft)	0.8
S	0.03
Dc (ft)	0.243274854
(mm)	74.15017544

<u>Inch-pound units</u>

$$D_c = \frac{\tau_w dS}{0.06\ (\tau_s - \tau_w)}$$

<u>Yang Incipient Motion</u>

Yang (1973) developed a relationship between dimensionless critical velocity, V_{cr}/w, and shear velocity Reynold's number, R*, at incipient motion. Under rough regime conditions where R*>70, the equation for incipient motion which is considered applicable to bed material size larger than about 2 mm by Reclamation is:

$$\frac{V_{cr}}{w} = 2.05 \qquad (6)$$

where:

V_{cr} = Critical average water velocity at incipient motion, ft/s (m/s)
w = Terminal fall velocity, ft/s (m/s)

The settling velocity by Rubey (1933) for material larger than 2 mm in diameter will approximate the fall velocity by:

$$w = 6.01\ D_c^{1/2} \text{ inch-pound units} \qquad (7)$$

$$w = 3.32\ D_c^{1/2} \text{ metric units}$$

Vcr (ft/s)	9.16
Dc (ft)	0.552937904
(mm)	168.5354731

Equations 6 and 7 can be combined to give:

$$D_c = 0.00659\ V_{cr}^2 \text{ inch-pound units} \qquad (8)$$

$$D_c = 0.0216\ V_{cr}^2 \text{ metric units}$$

<u>Depth to Armor and Volume Computations</u>

After determining the size of the material required to armor the streambed, from either an average of the five methods or a judgment decision on the best method, an estimate can be made of the probable vertical degradation before stabilization is reached. The armoring computations assume that an armoring layer will form as shown on figure 6 by the equations:

$$y_a = y - y_d \qquad (9)$$

and

$$y_a = (\Delta p)\ y \qquad (10)$$

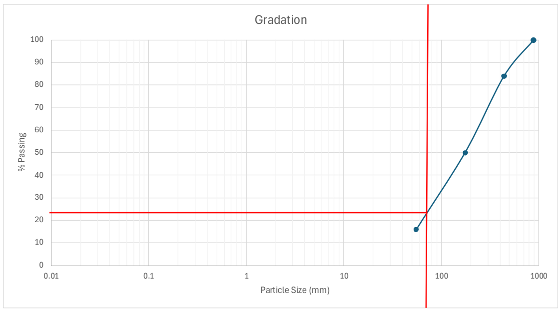
which are combined to:

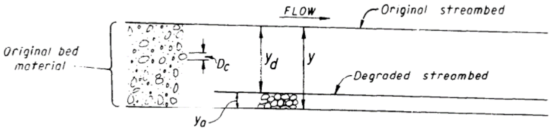
$$y_d = y_a \left(\frac{1}{\Delta p} - 1 \right) \qquad (11)$$

where:

y_a = Thickness of armoring layer
y = Depth from original streambed to bottom of the armoring layer
y_d = Depth from original streambed to top of armoring layer or the depth of degradation
Δp = Decimal percentage of original bed material larger than the armor size, D_c

	AVG All		
Dc (mm)	72.77263576		
	Ya 0.5 ft	3*D	
Ya (mm)	152.4	218.3179	use smaller of values
P	0.78		
Yd (mm)	42.98461538		
Yd(ft)	0.141025641		





y = Depth to bottom of the armoring layer

y_d = Depth of degradation

y_a = Armoring layer

D_c = Diameter of armor material

Δp = Decimal percentage of original bed material larger than D_c

Figure 6. - Armoring definition sketch.

The percentage of the bed material equal to or greater than the required armoring size, D_c, can be determined from the bed material size analysis curve from samples collected of the streambed material through the reach involved and at a depth through the anticipated scour zone. This size analysis gives the value Δp to be used in equation 11. The depth, y_a, of the required armoring may vary, depending on the limiting particle size, from a thickness of one particle diameter to three particle diameters or one and three times the armoring size, respectively. A rough guide for use in design is either three armoring particle diameters or 0.5 ft (0.15 m), whichever is smaller. Although armoring has been observed to occur with less than three particle diameters, variability of channel bed material and occurrence of peak discharges dictate the use of a thicker armor layer.

Thorne Bend Scour
Reference: Washington State Aquatic Habitat Guidelines Program - Integrated Streambank Protection Guidelines 2003
https://wdfw.wa.gov/sites/default/files/publications/00046/wdfw00046.pdf

Thorne Equation

$$d / y_1 = 1.07 - \log(R_c/W - 2) \quad \text{for } 2 < R_c/W < 22 \qquad \text{(EQUATION 8)}$$

Where: d = maximum depth of scour below local stream bed elevation
y_1 = average flow depth directly upstream of the bend
W = width of flow
R_c = radius of curvature at channel centerline

The width of flow in Equation 8 corresponds to the width of active flow. This width is subject to engineering judgement. However, it often corresponds to the bankfull top width for streams that are flowing near or above bankfull stage. English or metric units may be used.

S. Maynord reviewed bend scour estimates for natural, sand-bed channels and presented one bend-scour equation by W. Wattanabe and a second method of his own.[7] These two equations are listed below. They are useful for predicting scour depths on sand-bed streams and for determining conservative scour depths (for comparison to other methods) on streams with coarser bed materials.

Bend	A	B	C	D	E	F	DS Bend
Avg depth(ft)	0.667053933	0.868661	0.778378	0.82640531	0.77708083	0.92686746	0.750373
Rc	30	30	40	30	50	30	30
Design BFW	12.08	12.08	12.08	12.08	12.08	12.08	12.08
Rc/BFW	2.483443709	2.483444	3.311258	2.48344371	4.13907285	2.48344371	2.483444
scour depth (ft)	0.924306009	1.203664	0.741259	1.1451119	0.57486452	1.28431768	1.039757

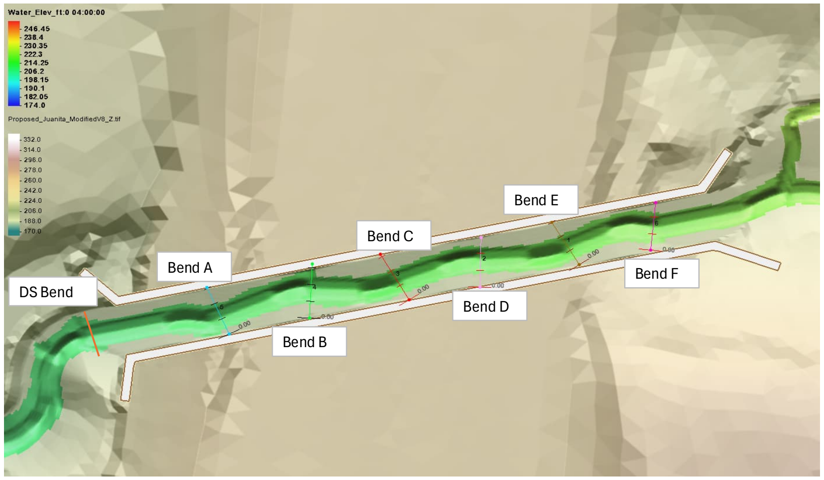
USACE Bend Scour
Reference: USACE Engineering and Design - Hydraulic Design of Flood Control Channels
https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1601.pdf

USACE EM 1110-2-1601

$$\Delta y = \begin{cases} \bar{D}_{US}\left(-1.51\log_{10}\left(\frac{Rc}{W}\right)+3.37\right)-D_{Max}\ for\ sand \\ \bar{D}_{US}\left(-1.62\log_{10}\left(\frac{Rc}{W}\right)+3.375\right)-D_{Max}\ for\ gravel \end{cases}$$

Bend	A	B	C	D	E	F	DS Bend
Avg depth(ft)	0.667054	0.868661	0.778378	0.826405	0.777081	0.926867	0.750373
rb	30	30	40	30	50	30	30
W	12.08	12.08	12.08	12.08	12.08	12.08	12.08
rb/W	2.483444	2.483444	3.311258	2.483444	4.139073	2.483444	2.483444
Dus	0.667054	0.868661	0.778378	0.826405	0.777081	0.926867	0.750373
Dmax	1.125	1.125	1.125	1.125	1.125	1.125	1.3
Scour depth	0.699401	1.250799	0.84633	1.135228	0.721046	1.409994	0.752279

	Cross Section							
	A	B	C	D	E	F		DS Bend
	Depth (ft)	Depth (ft)	Depth (ft)	Depth (ft)	Depth (ft)	Depth (ft)		Value
	-0.0397073	-0.0337193	0.0281002	-0.0217916	-0.0922847	0.1632816		0.022123
	0.1311465	0.041504	0.2097847	0.1106481	0.0869534	0.1902786		0.285512
	0.5582167	0.1449009	0.7220653	0.2939756	0.4434948	0.3552067		0.740236
	0.8248709	0.4226857	1.093478	0.655045	0.8767386	0.4947601		0.913857
	0.98189	0.9898725	1.2906021	0.8063402	1.0573125	0.7170968		1.084495
	1.1463449	1.1859709	1.4883718	1.0491794	1.211044	1.1704028		1.229587
	1.2663198	1.3430007	1.3417964	1.3551544	1.2988977	1.417029		1.229637
	1.1600603	1.5376734	1.0296757	1.511084	1.2108597	1.6626134		1.302247
	0.9788811	1.5970438	0.8754504	1.4371339	1.0710722	1.8930746		1.145189
	0.8230017	1.4704092	0.8550946	1.2179238	1.0666552	1.9298615		0.992073
	0.5545151	1.4692858	0.6988251	0.9054878	0.8737212	1.7367662		0.839692
	0.1270262	1.2960886	0.3740851	0.4389454	0.6258999	1.36509		0.531942
	0.0774004	1.0704825	0.111588	0.1359461	0.2072867	1.1445456		0.1588
	0.0420276	0.785449	-0.0041351	-0.0126865	0.0721151	0.8169843		0.029835
	-0.0651027	0.5827278			-0.0147951	0.4657908		
		0.4420967				0.2078546		
		0.2859662				0.0261101		
		0.1020823						
							AVG	
Avg depth(ft)	0.6670539	0.8686612	0.7783783	0.8264053	0.7770808	0.9268675	0.807408	0.750373

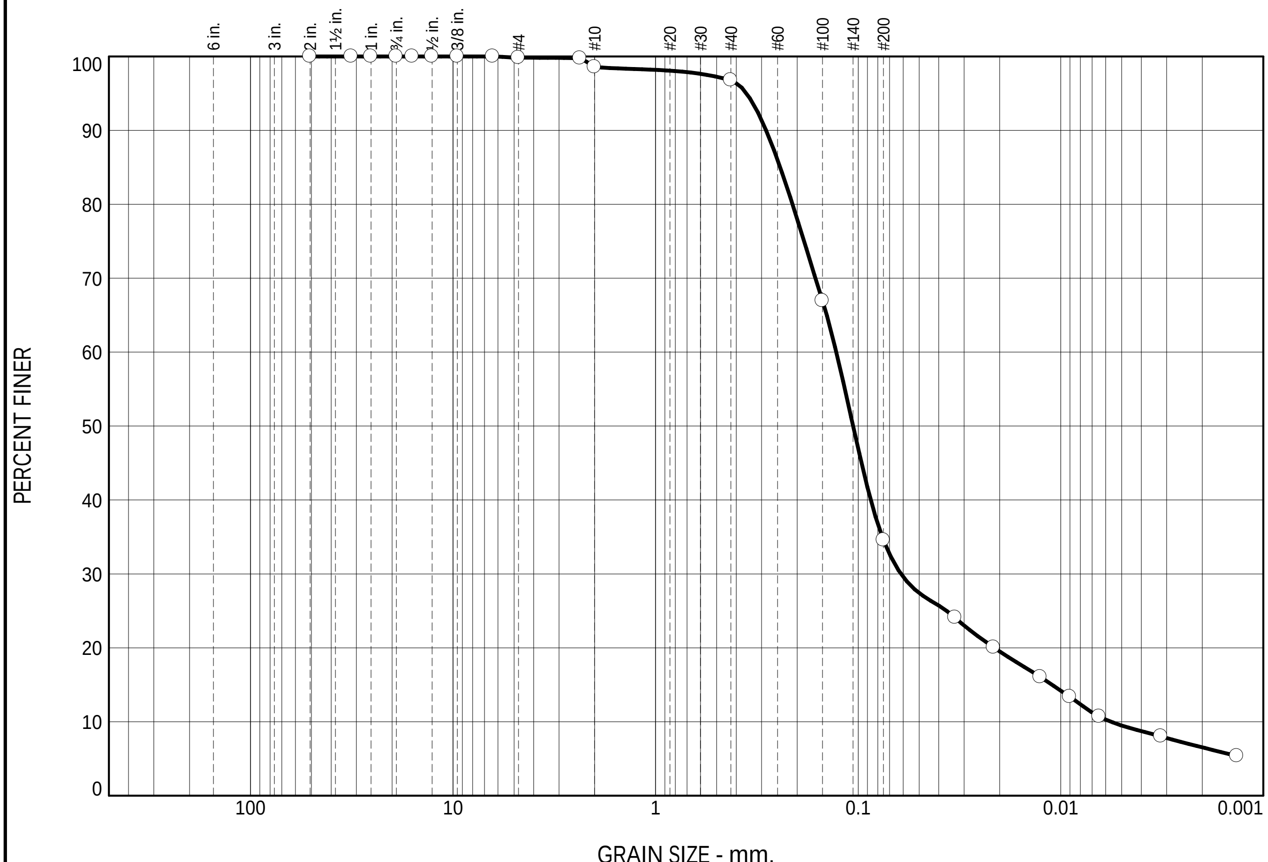


Appendix L: Floodplain Analysis *(FHD ONLY)*

Appendix M: Scour Countermeasure Calculations (FHD ONLY)

Appendix N: Particle Size Distribution Reports

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.2	1.2	1.8	62.2	25.1	9.5

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2	100.0		
1 1/4	100.0		
1	100.0		
3/4	100.0		
5/8	100.0		
1/2	100.0		
3/8	100.0		
1/4	100.0		
#4	99.8		
#8	99.8		
#10	98.6		
#40	96.8		
#100	66.9		
#200	34.6		
0.0332 mm.	24.1		
0.0214 mm.	20.1		
0.0126 mm.	16.1		
0.0090 mm.	13.4		
0.0065 mm.	10.7		
0.0032 mm.	8.0		
0.0013 mm.	5.4		

* (no specification provided)

Soil Description

ORA sample ID: 1384 Gray silty sand (Trace organic)

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.2848 D_{85}= 0.2426 D_{60}= 0.1282
D_{50}= 0.1059 D_{30}= 0.0614 D_{15}= 0.0110
D_{10}= 0.0056 C_u= 22.72 C_c= 5.22

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Test equipment ID: Set 5
Was sample soaked? Yes
As received MC: 52.9%

Location: Upstream Juanita
Sample Number: 1384

Date: 12/7/2023

OTTO ROSENAU & ASSOCIATES, INC.

Client: AECOM, Inc. - Seattle
Project: Par Creek Sediment Sampling
1405 - SR 522 Vicinity to SR 527 Express, Bothell
Project No: 23-0669 Figure 1384

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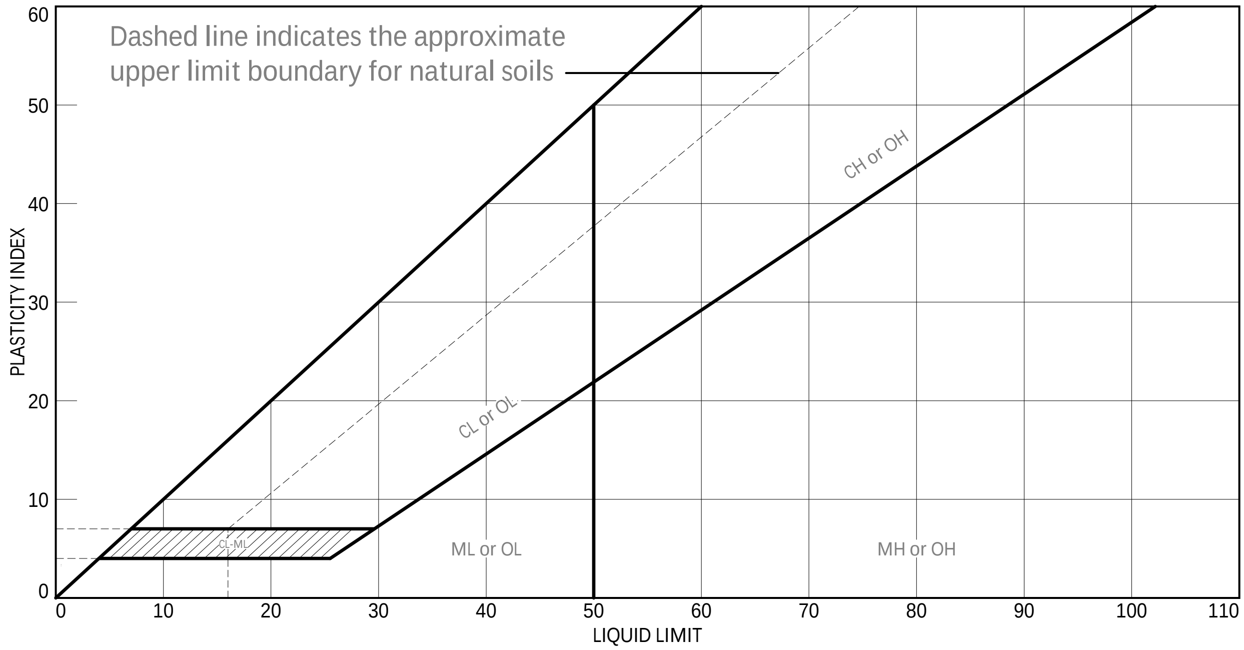
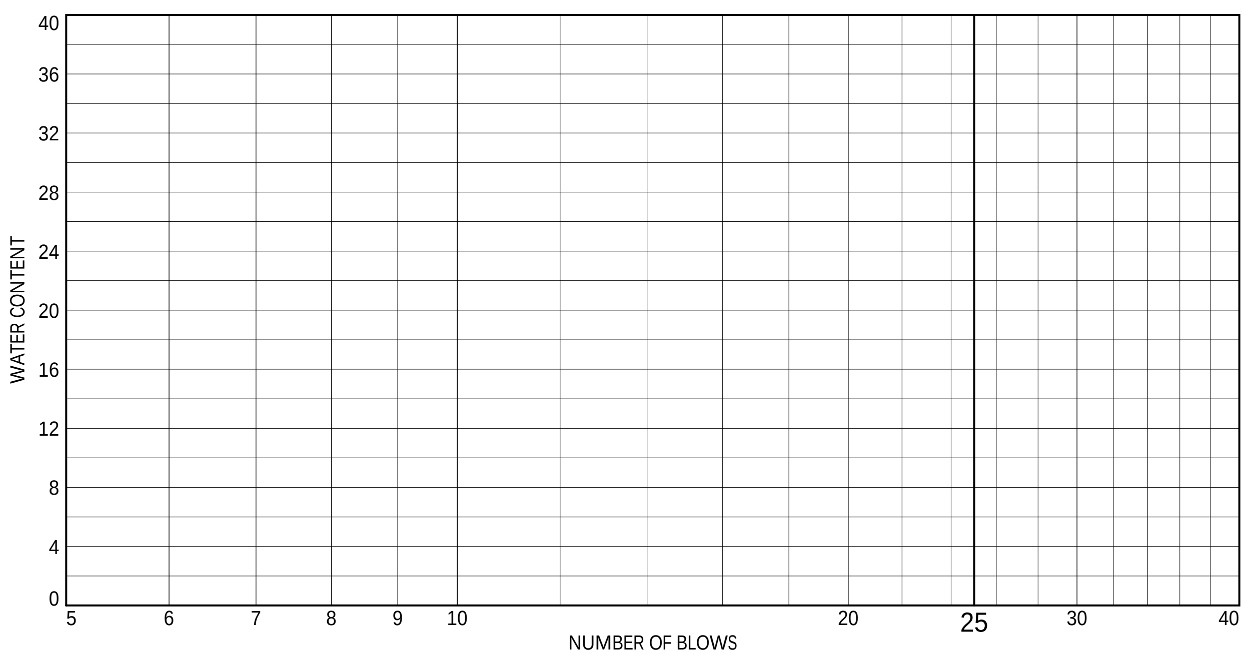
Anthony Coyne, P.E. Date: 2024.01.10 13:58:41 -08'00'

Tested By: Andy Duong Checked By: Anthony Coyne, P.E.

Note: These samples were taken from the banks of Juanita Creek, not the stream bed. These were not used for analysis in this PHD.

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	ORA sample ID: 1384 Gray silty sand (Trace organic)	NV	NP	NP	96.8	34.6	SM

Project No. 23-0669 Client: AECOM, Inc. - Seattle

Project: Par Creek Sediment Sampling

1405 - SR 522 Vicinity to SR 527 Express, Bothell

Location: Upstream Juanita

Sample Number: 1384

Remarks:

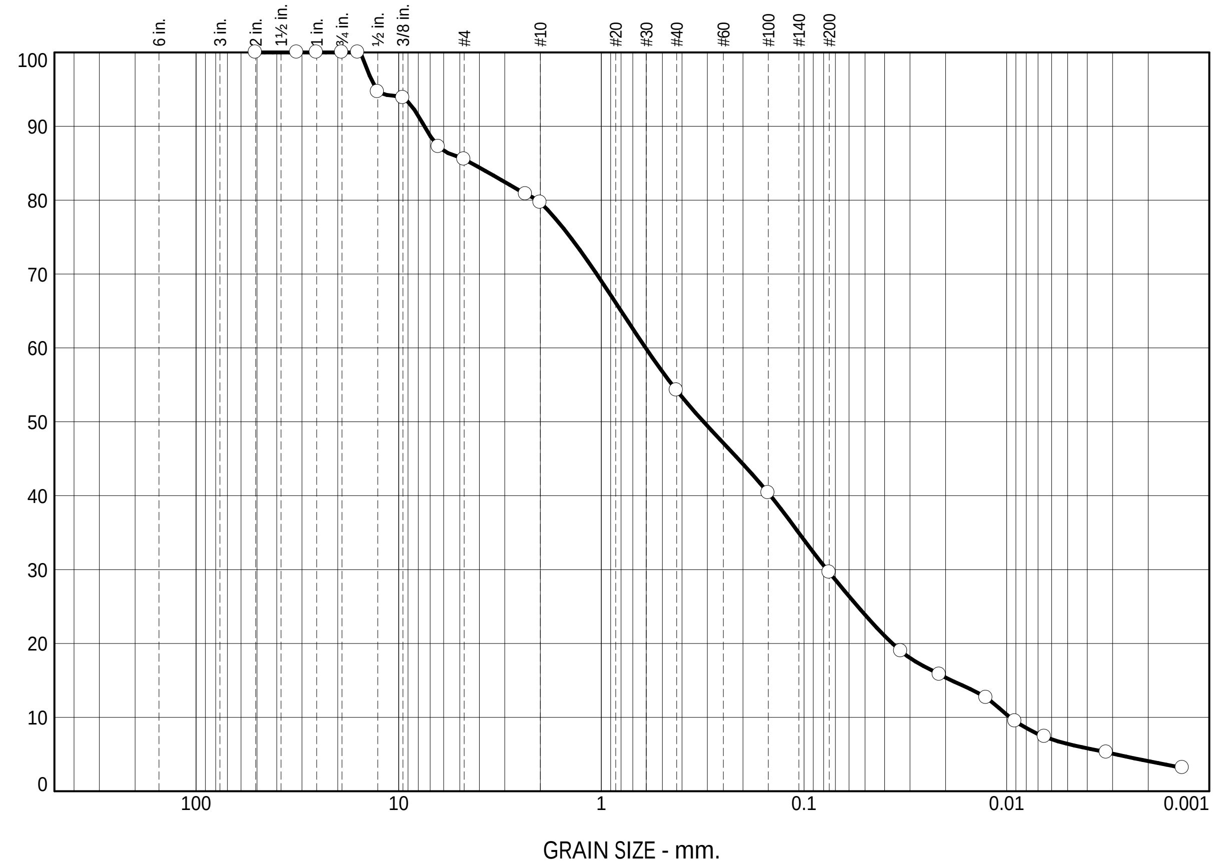
●COULD NOT ROLL SOIL INTO AN 1/8" THREAD.

OTTO ROSENAU & ASSOCIATES, INC.

Figure 1384

Tested By: Andy Duong Checked By: Anthony Coyne Anthony Coyne

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	14.5	5.8	25.4	24.7	23.2	6.4

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2	100.0		
1 1/4	100.0		
1	100.0		
3/4	100.0		
5/8	100.0		
1/2	94.7		
3/8	93.8		
1/4	87.2		
#4	85.5		
#8	80.8		
#10	79.7		
#40	54.3		
#100	40.4		
#200	29.6		
0.0332 mm.	19.0		
0.0214 mm.	15.8		
0.0126 mm.	12.6		
0.0091 mm.	9.5		
0.0065 mm.	7.4		
0.0032 mm.	5.3		
0.0014 mm.	3.2		

* (no specification provided)

Soil Description

ORA sample ID: 1381 Gray silty sand (Trace organic)

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 7.4704 D_{85}= 4.3703 D_{60}= 0.6042
D_{50}= 0.3128 D_{30}= 0.0771 D_{15}= 0.0187
D_{10}= 0.0096 C_u= 62.77 C_c= 1.02

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Test equipment ID: Set 5
Was sample soaked? Yes
As received MC: 41.1%

Location: Juanita downstream near yard
Sample Number: 1381

Date: 12/7/2023

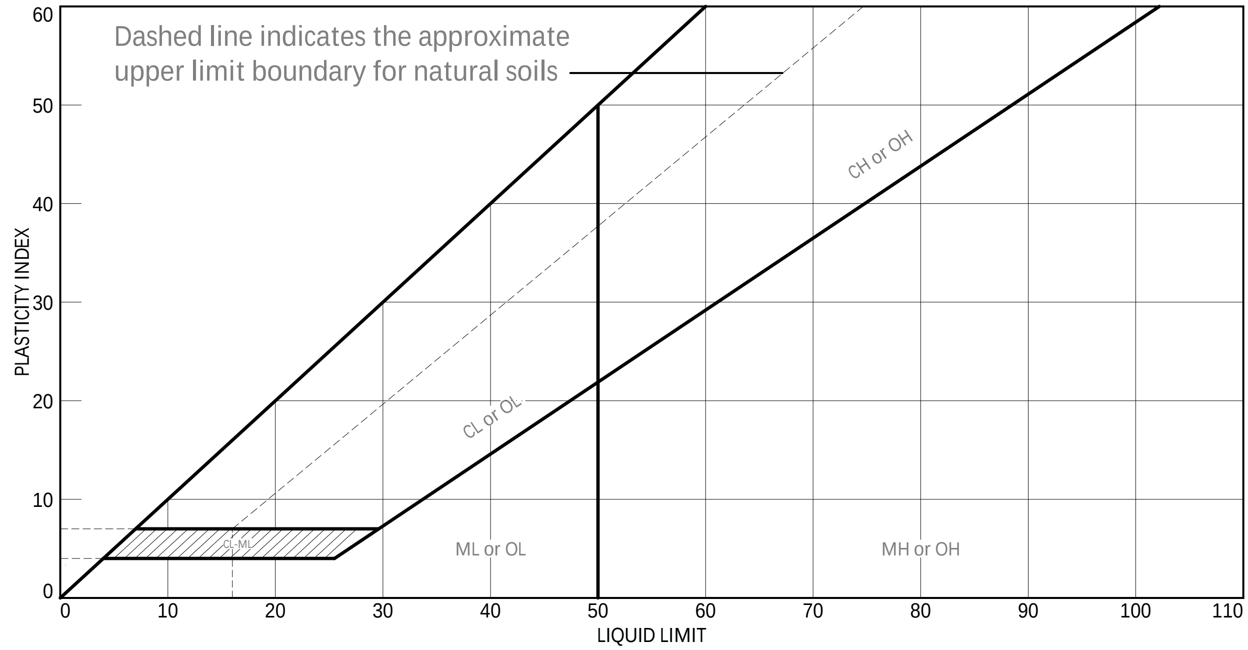
OTTO ROSENAU & ASSOCIATES, INC.

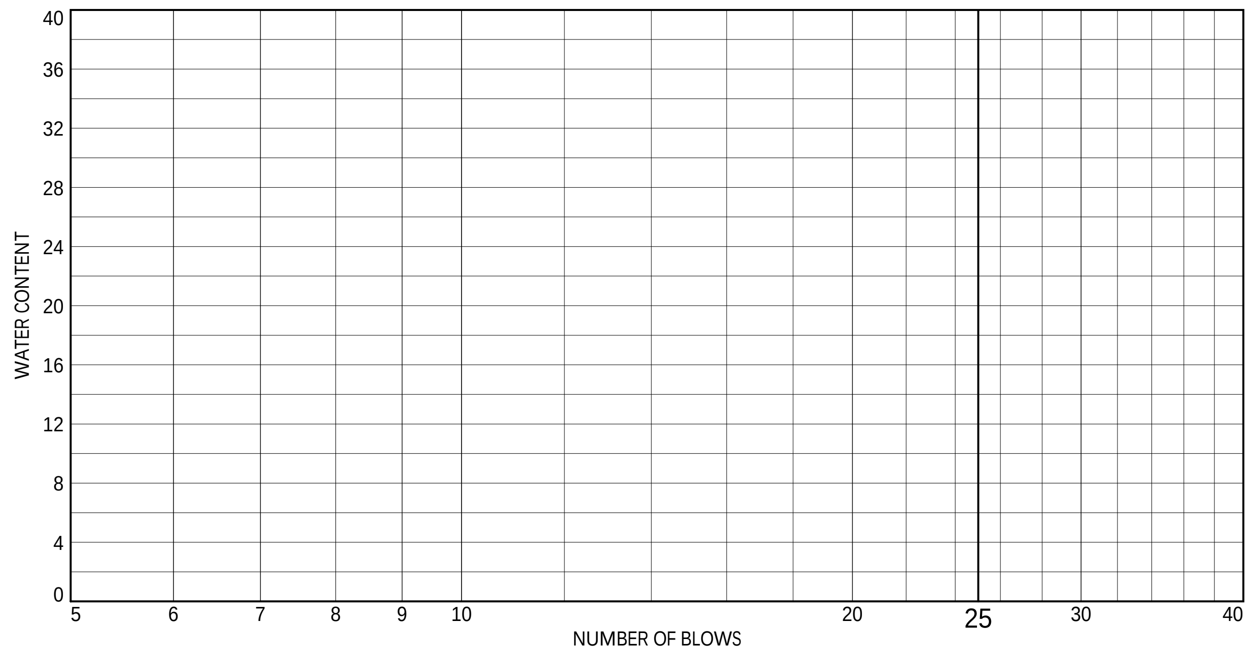
Client: AECOM, Inc. - Seattle
Project: Par Creek Sediment Sampling
1405 - SR 522 Vicinity to SR 527 Express, Bothell
Project No: 23-0669 Figure 1381

Tested By: Andy Duong Checked By: Anthony Coyne, P.E.

Anthony Coyne, P.E. Date: 2024.01.10 13:52:19 -08'00'

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	ORA sample ID: 1381 Gray silty sand (Trace organic)	NV	NP	NP	54.3	29.6	SM

Project No. 23-0669 Client: AECOM, Inc. - Seattle

Project: Par Creek Sediment Sampling

1405 - SR 522 Vicinity to SR 527 Express, Bothell

Location: Juanita downstream near yard

Sample Number: 1381

Remarks:

●COULD NOT ROO SOIL INTO AN 1/8" THREAD.

OTTO ROSENAU & ASSOCIATES, INC.

Figure 1381

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Tested By: Andy Duong Checked By: Anthony Coyne

Appendix O: Channel Design

Juanita Creek Planform Analysis

W (bankfull) (ft)	12.08
Qbf (bankfull discharge, 2-yr) (cfs)	40

Equation	Meander Wavelength (ft)	Citation
8 X BF	96.6	I-405 Brickyard RFP, Tbl 2.30-B
4.01*W	48.4	Thorne and Soar (2001); NEH Part 654, Ch 12, Fig 12-3 Lower 90% CI
26.07*W	314.9	Thorne and Soar (2001); NEH Part 654, Ch 12, Fig 12-3 Upper 90% CI
10.23*W	123.6	Thorne and Soar (2001)
11.26*W	136.0	Thorne and Soar (2001)
12.47*W	150.6	Thorne and Soar (2001)
10.9*W^1.01	135.0	Leopold and Wolman (1960)
6.06*W^0.99	71.4	Inglis (1941)
6*W	72.5	Yalin (1992)
30*Qbf^0.5	189.7	Dury (1965)
8.2*Qbf^0.62	80.7	Carlston (1965)

Table 1: High Woodlands HSPF Peak Flow Rate (in cfs) for Various Storm Frequencies Based on 15- minute time step

	Project Scenario	2-year	10-year	25-year	100-yr
At I-405	Pre	25	32	35	41
	Post	40	65	81	111
Above Norway Hill Trib	Pre	30	39	44	51
	Post	45	74	94	129
At 141st	Pre	65	101	124	166
	Post	81	135	172	242

Refer to Attachment 1 for additional storm frequencies evaluated in the HSPF model.

Wavelength Measurements of Meanders from WSDOT Proposed Terrain (starting downstream)	Within 90% CI of Thorne and Soar (2001) (>48.4)?
87.0	Y
96.8	Y
101.5	Y
74.1	Y
37.0	N
84.3	Y
86.0	Y
Average 88.28	

Part 654 Stream Restoration Design National Engineering Handbook. Channel Alignment and Variability Design. Ch 12

wavelength equation was 0.88 for a linear regression equation, with a variable exponent on W. This exponent was found not to be significantly different from 1.0, so the exponent was fixed at 1.0 for convenience. Only sites with sinuosities of at least 1.2 and bankfull widths between 1 meter and 1,000 meters were used in development of this regression equation. Within these constraints, meander wavelengths range between 10.4 meters and 19,368 meters, and sinuosity values range between 1.2 and 5.3. The equation, corrected for bias, is:

$$\lambda = 11.85W \qquad \text{(eq. 12–2)}$$

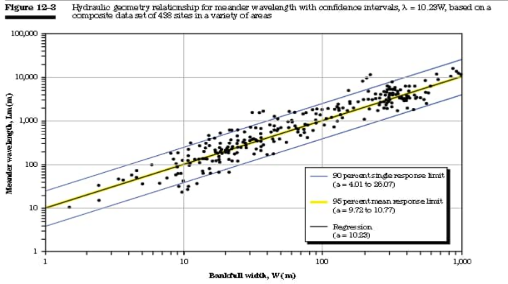
An unbiased hydrologic equation for meander wavelength suitable for engineering design, within 95 percent confidence limits on the mean response is:

$$\lambda = (11.26 \text{ to } 12.47)W \qquad \text{(eq. 12–3)}$$

According to Hey (1976) and Thorne (1997), twice the distance between successive riffles (or pools) in a straight channel should equal $4\pi W$, or 12.57W. This is based on the assumption that the average size of the largest macroturbulent eddies (or helical flow cell) is half the channel width. Equation 12–3 shows that the upper range of stable meander wavelengths is numerically very close to this value and similar to the coefficient of 12.34 given by Richards (1982). This corroborates the assertion by Leopold and Wolman (1957, 1960) that the matching of waveforms in bed topography and planform is related to the mechanics of the flow and, in particular, to the turbulent flow structures responsible for shaping the forms and features of meandering channels.

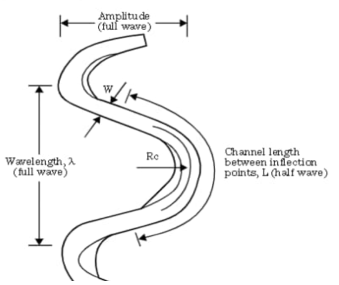
Table 12–1 shows the data sources (438 sites) used in the development of these equations.

Figure 12–3 Hydraulic geometry relationship for meander wavelength with confidence intervals, $\lambda = 10.23W$, based on a composite data set of 438 sites in a variety of areas



(210–VI–NEH, August 2007) 12–3

Figure 12–2 Planform descriptive variables



Hydraulic geometry for meander wavelength

...nposite relationship has been developed by ...e and Soar (2001), combining 9 data sets and 438 ... Their mean linear regression predictor for wave... ...n is:

$$\lambda = 10.23W \qquad \text{(eq. 12–1)}$$

...e:
= meander wavelength
= channel width in any consistent units of measurement

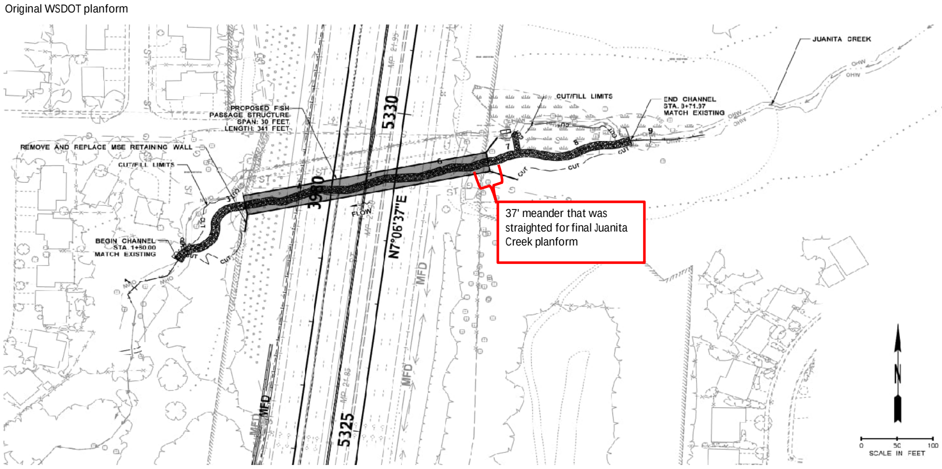
...tions of planform descriptive variables are ...n in figure 12–2. Confidence bands about this ...ion are shown in figure 12–3. The r^2 for the

Table 12–2 Hydraulic geometry relationships for meander wavelength

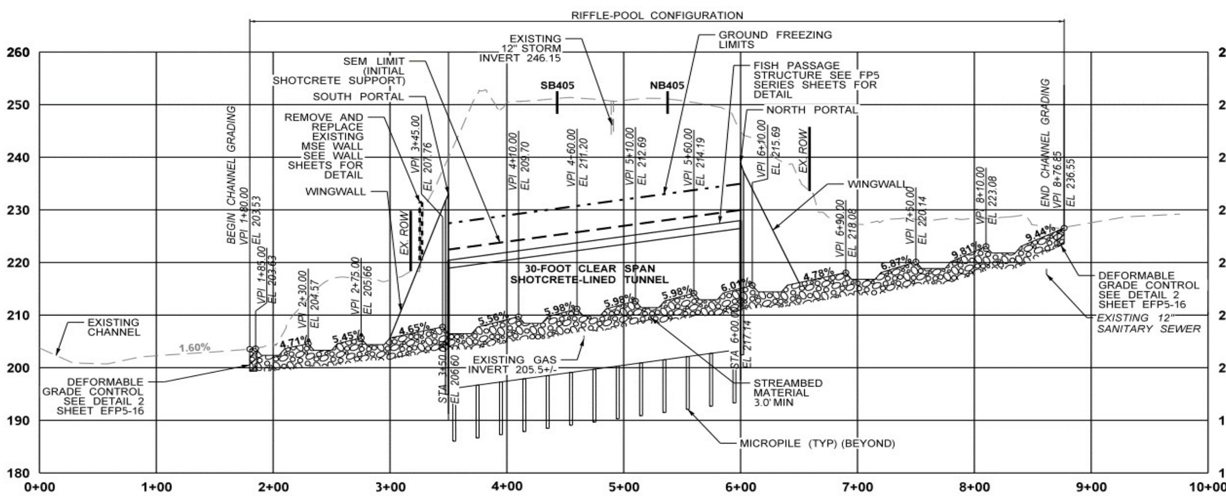
Author	Equation	Units
Leopold and Wolman (1960)	$\lambda = 10.9\ W^{1.01}$	ft
Inglis (1941)	$\lambda = 6.06\ W^{0.99}$	ft
Yalin (1992)	$\lambda = 6\ W$	length
Dury (1965)	$\lambda = 30\ Q_{bf}^{0.5}$	ft, ft^3/s
Carlston (1965)	$\lambda = 8.2\ Q_{bf}^{0.62}$	ft, ft^3/s
Carlston (1965)	$\lambda = 106.1\ Q_{ma}^{0.46}$	ft^3/s
Schumm (1967)	$\lambda = 1890\ Q_{ma}^{0.34}\ M^{0.74}$	ft, ft^3/s

Notes: λ = meander wavelength
W = width
Q_{bf} = bankfull discharge
Q_{ma} = mean annual discharge
M = silt-clay factor

Original WSDOT planform

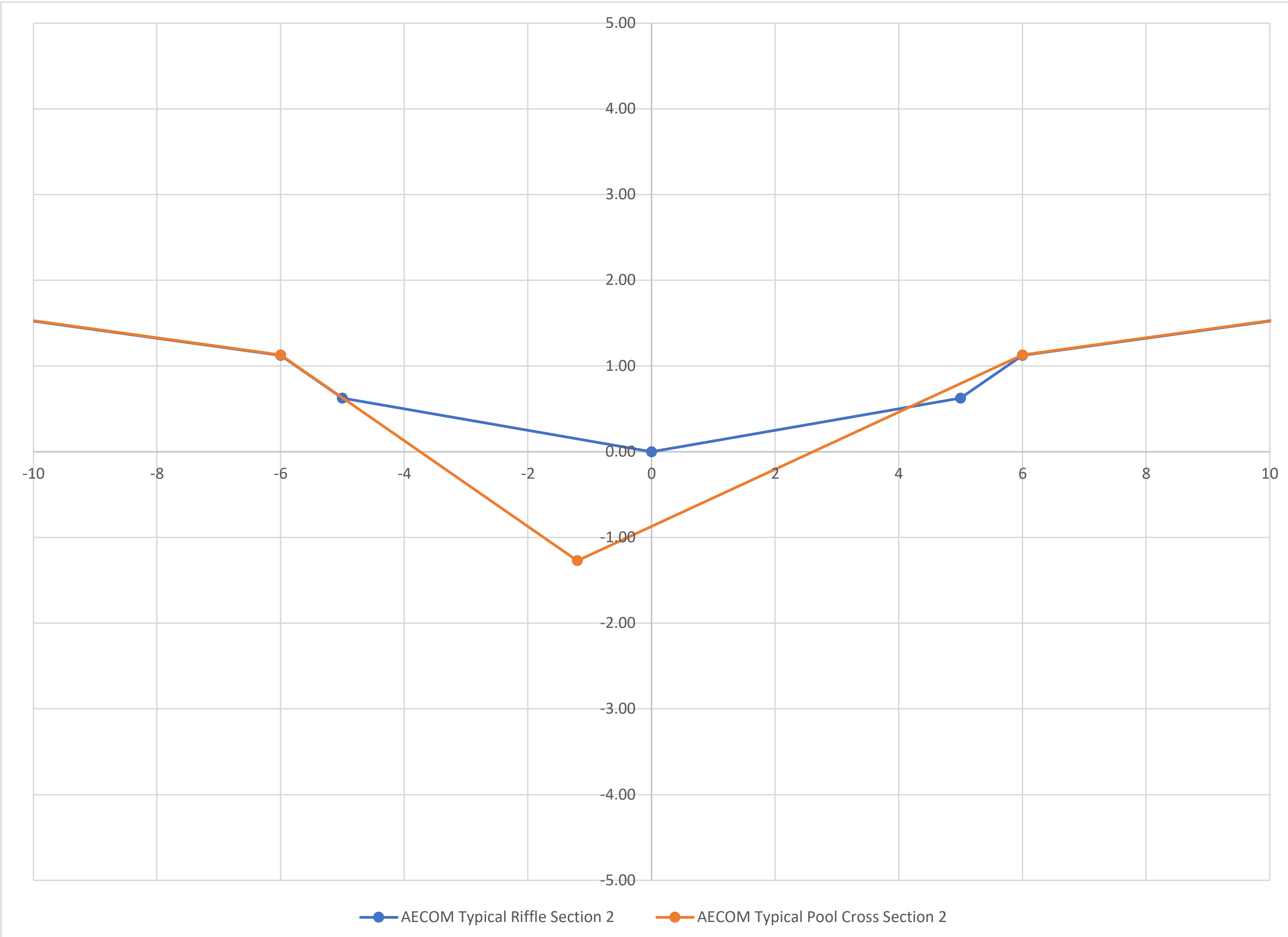


Feature	Sta Station	Elev	Slope	Length (ft)
end	180	203.53		
riffle	185	203.63	2.0%	5
Right pool	210	203.63		25
riffle	230	204.57	4.7%	20
Left pool	255	204.57		25
riffle	275	205.66	5.5%	20
Right pool	300	205.66		25
riffle	345	207.76	4.6%	45
Left pool	375	207.76		30
riffle	410	209.70	5.6%	35
Right pool	435	209.70		25
riffle	460	211.20	6.0%	25
Left pool	485	211.20		25
riffle	510	212.69	6.0%	25
Right pool	535	212.69		25
riffle	560	214.19	6.0%	25
Left pool	585	214.19		25
riffle	610	215.69	6.0%	25
Right pool	640	215.69		30
riffle	690	218.08	4.8%	50
Right pool	720	218.08		30
riffle	750	220.14	6.9%	30
Left pool	780	220.14		30
riffle	810	223.08	9.8%	30
Right pool	840	223.08		30
riffle	876.85	236.55	9.4%	36.85



JUANITA CREEK PROFILE

N.T.S.



5.00
4.00
3.00
2.00
1.00
0.00
-1.00
-2.00
-3.00
-4.00
-5.00
-10
-8
-6
-4
-2
0
2
4
6
8
10
AECOM Typical Riffle Section 2
AECOM Typical Pool Cross Section 2

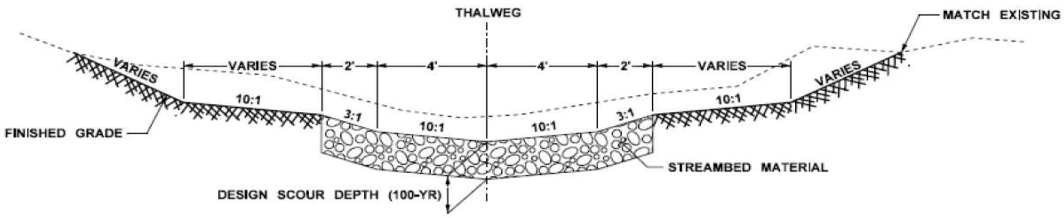


Figure 27: Design cross section

WSDOT Typical Riffle Section

Bankfull width = 12 ft.

STA	ELEV	SLOPE
-26	3.07	10 to 1
-6	1.07	3 to 1
-4	0.40	10 to 1
0	0.00	Thalweg
4	0.40	10 to 1
6	1.07	3 to 1
26	3.07	10 to 1

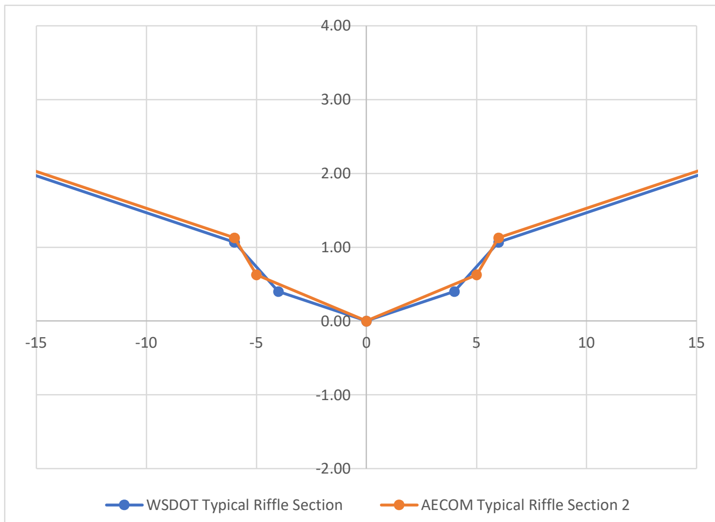
bkful depth
1.07

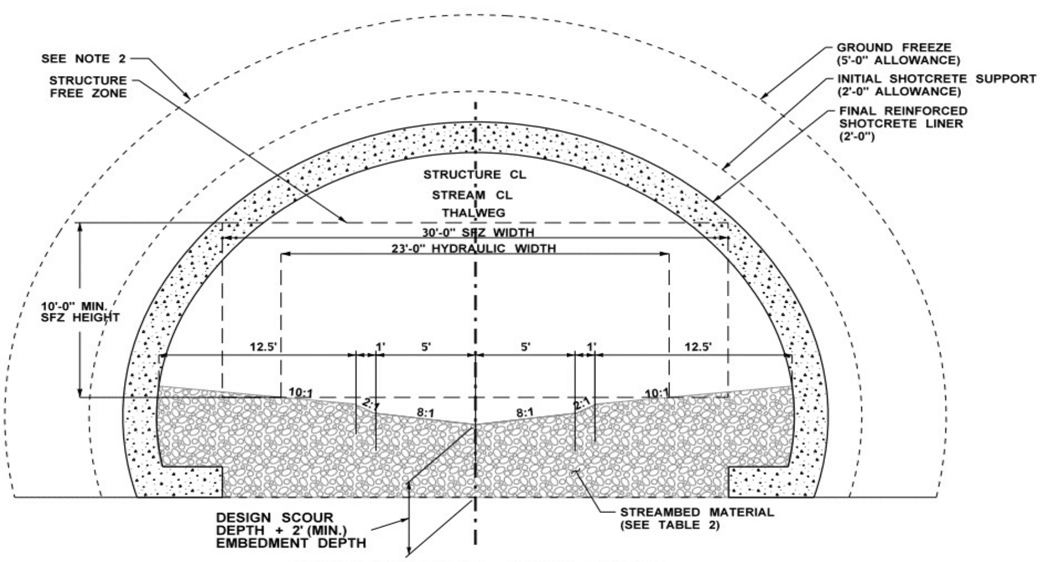
AECOM Typical Riffle Section 2

Bankfull width = 12 ft.

Sta	Elev	Slope
-26	3.13	10 to 1
-6	1.13	2 to 1
-5	0.63	8 to 1
0	0.00	Thalweg
5	0.63	8 to 1
6	1.13	2 to 1
26	3.13	10 to 1

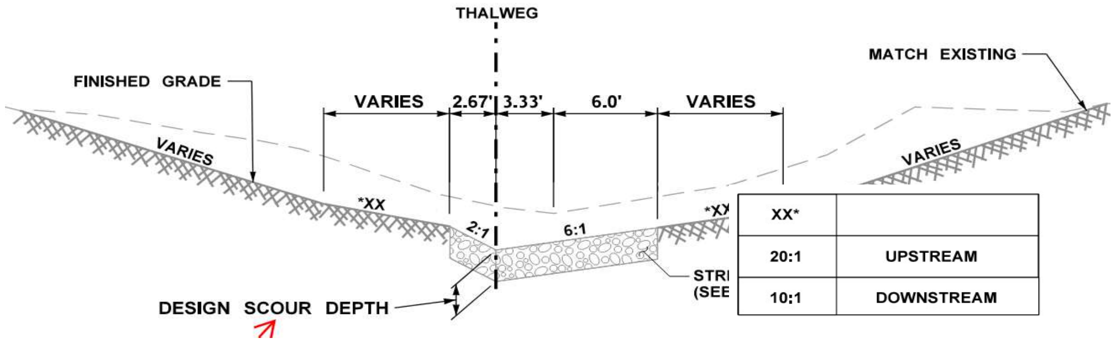
bkful depth
1.13





TYPICAL STRUCTURE - RIFFLE SECTION

N.T.S.



WSDOT Typical Pool Cross Section

Bankfull width = 12 ft.

STA	ELEV	SLOPE
-26.00	3.07	10 to 1
-6.00	1.07	2 to 1
-3.33	-0.27	Thalweg
6.00	1.29	6 to 1
26.00	3.29	10 to 1

Bkful Depth

1.34 to 1.56

Pool Depth - Riffle Depth

-0.27

AECOM Typical Pool Cross Section 2

Bankfull width = 12 ft.

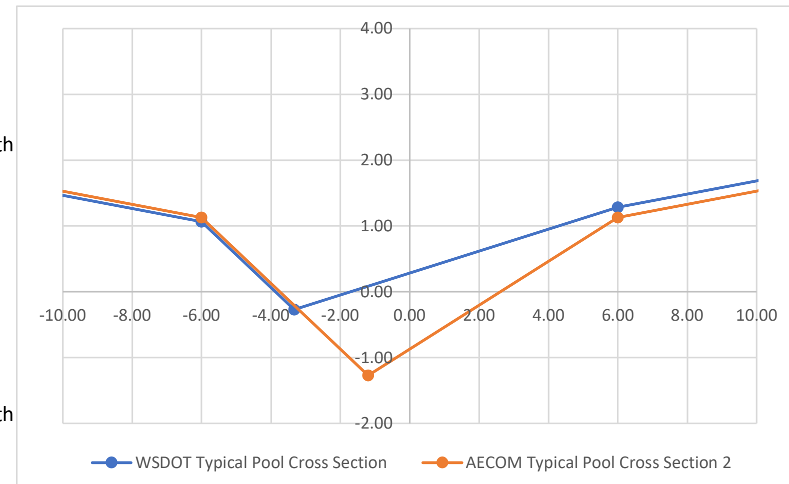
STA	ELEV	SLOPE
-26.0	3.13	10 to 1
-6.00	1.13	2 to 1
-1.20	-1.27	Thalweg
6.00	1.13	3 to 1
26.0	3.13	10 to 1

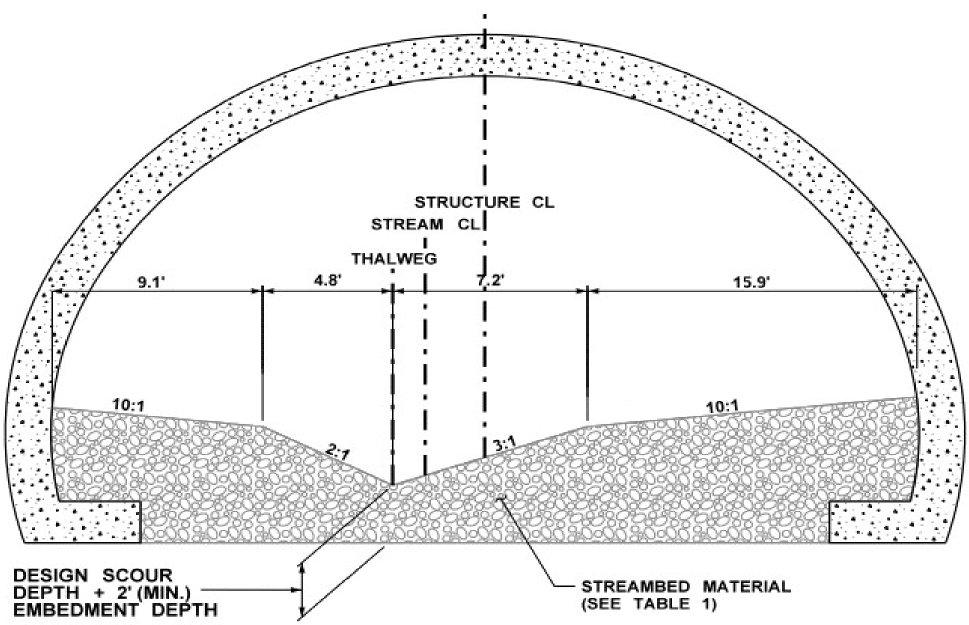
Bkful Depth

2.40

Pool Depth - Riffle Depth

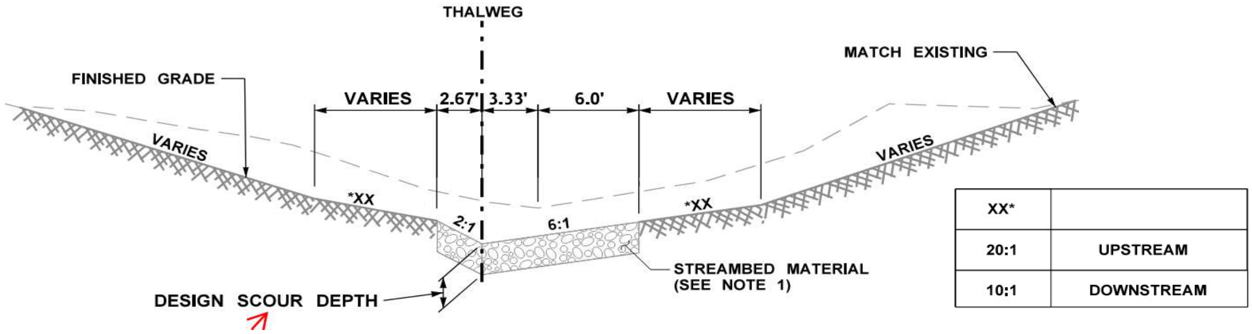
-1.27





TYPICAL STRUCTURE LEFT POOL SECTION

N.T.S

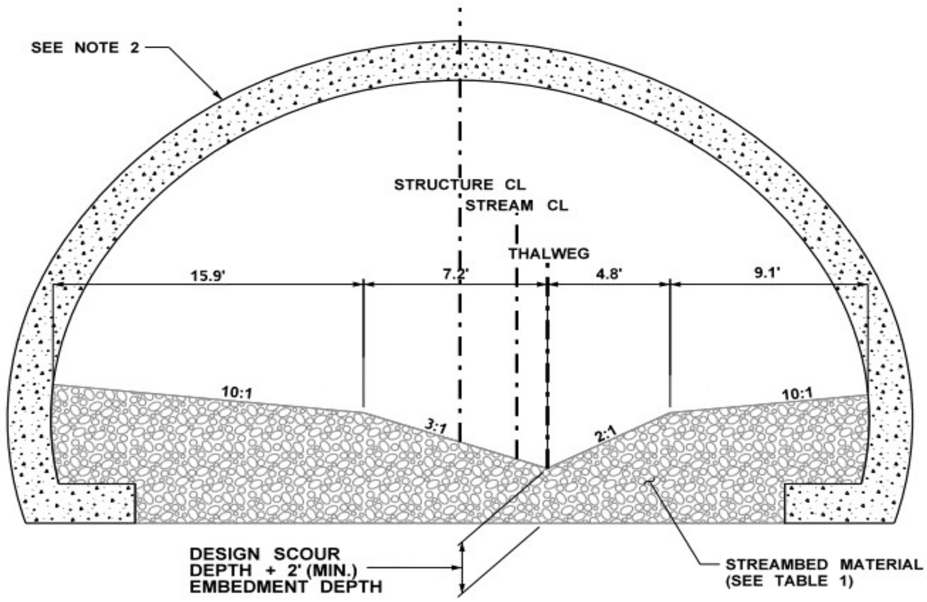


AECOM Typical Pool Cross Section 2

Bankfull width = 12 ft.

STA	ELEV	SLOPE	NOTE
-26.0	3.13	10 to 1	LOB
-6.00	1.13	2 to 1	LB
1.20	-1.27	Thalweg	Thalweg
6.00	1.13	3 to 1	RB
26.0	3.13	10 to 1	ROB

2.40
1.13
1.27



TYPICAL STRUCTURE RIGHT POOL SECTION
N.T.S.

Appendix P: Lateral Migration Rapid Assessment

Lateral Migration Rapid Assessment

I-405 MP 21.94 Juanita Creek (WDFW ID): 998602 Preliminary Hydraulic Design Report

Background

The Washington State Department of Transportation (WSDOT) is proposing a project to provide fish passage at the Interstate 405 (I-405) crossing of Juanita Creek at milepost (MP) 21.94. The existing culvert has been identified as a fish barrier and will be replaced by a 30-foot-wide bottomless culvert designed using the Stream Simulation methodology.

Purpose

The objective of this assessment is to provide a quantitative measure of Juanita Creek's potential for lateral migration.

Data Available

The following is a list of data/references used to prepare this calculation:

- Stream Stability at Highway Structures (FHWA, 2012),
- Field observations,
- WSDOT preliminary streambed profile and geometry (2021).

Methodology

The rapid assessment method was used from the Hydraulic Engineering Circular No. 20 (HEC-20) – Tables 5.5 through 5.8 – to rate the migration potential of Juanita Creek.

Results

The results of the rapid assessment indicate that Juanita Creek has low potential for lateral or vertical migration.

References

Lagasse, P.F., Zevenbergen, L.W., Spitz, W.J., and Arneson, L.A. (2012). *Stream Stability at Highway Structures, Fourth Edition.* Federal Highway Administration (FHWA) Publication No. FHWA-HIF-12-004. Hydraulic Engineering Circular No. 20.

WSDOT. 2021. *I-405 MP 26.87, SR 527 2.78 Queensborough Creek: Preliminary Hydraulic Desing Report*.

Attachments

Attachment A – Rapid Assessment for Juanita Creek

Attachment A – Rapid Assessment for Juanita Creek

A rapid stability assessment method should have the following characteristics: (1) it should be brief such that it can be completed quickly; (2) it should be simple in that extensive training is not required (although some training will be required); (3) it should be based on sound indicators as discussed in Section 5.3.3; and (4) it should be based on the needs of the bridge engineering community.

One way to assure that all aspects of channel stability are included is to start at the watershed or regional level and focus in on vertical and lateral aspects of the channel, following the concepts of Thorne (1998) (see Appendix C) and Montgomery and MacDonald (2002). Thus, at the broader level, watershed and floodplain activities and characteristics, flow habit, channel pattern and type, and entrenchment are selected as appropriate indicators. At the channel level, indicators such as bed material consolidation and armoring, bar development, and obstructions are used. Indicators of bank stability include bank material, angle, bank and riparian vegetation, bank (fluvial) cutting, and mass wasting (geotechnical failure). Finally, the position of the bridge relative to the channel can be indicated by meander impact point and alignment.

5.4.2 Rapid Assessment Method

The rapid channel stability assessment method uses a set of indicators, as determined from the literature and field observations. For each indicator, a rating is selected, the ratings are summed for a total score, and the score compared to stability definitions. In order to provide an appropriate level of sensitivity, the stability is based on stream type. Given that the Montgomery–Buffington classification method (Table 5.3) is based on processes as well as physical characteristics, this scheme was used to provide additional sensitivity to the method.

There are several assumptions implicit in this method of obtaining an overall rank. First, all indicators are weighted equally. This assumption was tested by assigning weights to each of the indicators and creating a weighted score for every bridge where observations were made. The results showed that the weighted indicators yielded the same results as the equally weighted indicators. Thus, there was no advantage in using weights. Second, this method implies that each indicator is independent of all others. While it is possible that some correlation exists between several of the indicators, an attempt was made to select indicators that independently describe various aspects of channel stability; thus, correlation effects were judged to be insignificant. Third, the summing of the ratings implies a linear scheme. The impact of this is not precisely known; however, given that weighted ratings provided no change in the overall results (Johnson 2005; FHWA 2006), it can be assumed that the linearity will also not affect the results significantly.

In collecting the data and observations for this method, it is desirable for the engineer or other inspector to view aerial photos of the bridge crossing and surrounding area and to walk some distance upstream and downstream from the bridge, rather than making all observations of the channel from the bridge itself. The appropriate distance, however, depends on several factors, such as uniformity of stream conditions; magnitude of disturbances along the banks, in the floodplain, or in the watershed; time available; and accessibility. Ideally, the observer should walk at least 10 channel widths upstream and downstream of the bridge. Although it is possible to establish stability conditions in a lesser distance, the more of the stream that is observed, the better the observer will understand the causes, processes, and rates of change, assuming that such observations are repeated at different times. Roads and bridges often divide property and sometimes divide geomorphic features or regions. Thus, conditions upstream and downstream of the bridge may be significantly different. In this case, it may be necessary to conduct separate analyses upstream and downstream.

5.4.3 Stability Indicators

The 13 indicators identified for this study are listed in Table 5.5 (FHWA 2006). For each indicator, a rating of poor, fair, good, or excellent can be assigned based on descriptors listed in the table. After a rating is assigned for each of the indicators, an overall score is obtained by summing the 13 ratings. This total score provides the overall relative stability of the channel. Table 5.6 provides the range of scores for Excellent, Good, Fair, and Poor ratings of stability for each of the three divisions of stream channels. The simplified data collection sheets of Appendix E assist in obtaining information necessary to score the stability indicators.

Occasionally, rating of each of the thirteen factors for a particular bridge will result in one factor which stands out as being much higher (worse) than the others. This situation is worth noting and making additional observations during future inspections.

5.4.4. Lateral and Vertical Stability

The indicators in Table 5.5 can be divided into those that indicate vertical stability and those that indicate lateral stability; vertical stability is described by indicators 4–6, while lateral stability is indicated by indicators 8 –13. Each of the lateral and vertical stability scores, based on summing the appropriate ratings, were normalized by the total number of points possible in each category so that they could be represented as a fraction and more readily compared. If the lateral fraction is greater than the vertical fraction, then it can be expected that channel instability is expressed primarily in the lateral direction. Lateral and vertical processes may be ongoing simultaneously or they may be occurring differentially; this is not indicated by the assessment method. If both fractions are relatively low, this suggests minimal instability in either direction.

As an example, if the lateral score is significantly higher than the vertical score (say for example, 0.93 versus 0.67), indicating that lateral instability is dominant. If, on the other hand, the vertical score fraction is greater than the lateral, then bed degradation is the dominant source of instability. If both scores are high, then the channel is unstable due to both lateral and vertical processes. For example, if a channel has lateral and vertical fractions of 0.86 and 0.92, this indicates that the channel is both degrading and widening.

Table 5.5. Stability indicators, descriptions, and ratings. Range of values in ratings columns provide possible rating values for each factor.

Stability Indicator	Ratings				
	Excellent (1-3)	Good (4-6)	Fair (7-9)	Poor (10-12)	
1. Watershed and floodplain activity and characteristics	Stable, forested, undisturbed watershed.	Occasional minor disturbances in the watershed, including cattle activity (grazing and/or access to stream), construction, logging, or other minor deforestation. Limited agricultural activities.	Frequent disturbances in the watershed, including cattle activity, landsliding, channel sand or gravel mining, logging, farming, or construction of buildings, roads, or other infrastructure. Urbanizatio over significant portion of watershed. 9	Continual disturbances in the watershed. Significant cattle activity, landsliding, channel sand or gravel mining, logging, farming, or construction of buildings, roads, or other infrastructure. Highly urbanized or rapidly urbanizing watershed.	36% impervious per PHD
2. Flow habit	Perennial stream with no flashy behavior 3	Perennial stream or ephemeral 1st order stream with slightly increased rate of flooding	Perennial or intermittent stream with flashy behavior	Extremely flashy; flash floods prevalent mode of discharge; ephemeral stream other than 1st order stream	
3. Channel pattern	Straight to meandering with low radius of curvature; primarily suspended load	Meandering, moderate radius of curvature; mix of suspended and bed loads; well maintai engineered channel 4	Meandering with some braiding; tortuous meandering; primarily bed load; poorly maintained engineered channel.	Braided; primarily bed load; unmaintained engineered channel.	
4. Entrenchment / channel confinement	Active floodplain exists at top of banks; no sign of undercutting infrastructure; no levees	Active floodplain abandoned, but is currently rebuilding; minimal channel confinement; infrastructure not exposed; levees are low and set well back from the river	Moderate confinement in valley or channel walls; some exposure of infrastructure; terraces exist; floodplain abandoned; levees are moderate in size and have minimal setback from the river. 7	Knickpoints visible downstream; exposed water lines or other infrastructure; channel width to top of banks ratio small; deeply confined; no active floodplain; levees are high and along the channel edge.	The bankfull width measurements reflect a moderately confined channel in a forest reach of a highly urbanized basin.
5. Bed material Fs = approximate portion of sand in the bed	Assorted sizes tightly packed, overlapping, and possibly imbricated. Most material > 4 mm. Fs < 20%	Moderately packed with some overlapping. Very small amounts of material < 4 mm. 20 < Fs < 50% 5	Loose assortment with no apparent overlap. Small to medium amounts of material < 4 mm. 50 < Fs < 70%	Very loose assortment with no packing. Large amounts of material < 4 mm. Fs > 70%	Observed sediment included a wide distribution of gravels mixed with a smaller distribution of fines and cobbles.
6. Bar development S = Slope W/Y = Width to Depth ratio	For S < 0.02 and W/Y > 12, bars are mature, narrow relative to stream width at low flow, well vegetated, and composed of coarse gravel to cobbles. For S > 0.02 and W/Y < 12, no bars are evident.	For S < 0.02 and W/Y > 12, bars may have vegetation and/or be composed of coarse gravel to cobbles, but minimal recent growth of bar evident by lack of vegetation on portic of the bar. For S > 0.02 ar W/Y < 12, no bars are evident. 5	For S < 0.02 and W/Y > 12, bar widths tend to be wide and composed of newly deposited coarse sand to small cobbles and/or may be sparsely vegetated. Bars forming for S > 0.02 and W/Y < 12.	Bar widths are generally greater than ½ the stream width at low flow. Bars are composed of extensive deposits of fine particles up to coarse gravel with little to no vegetation. No bars for S < 0.02 and W/Y > 12.	W/Y > 12 (12.08'/1.25' =9.664) S~2%. Some small sand/gravel bars.
7. Obstructions, including bedrock outcrops, armor layer, large woody debris jams, grade control, bridge bed paving, revetments, dikes or vanes, riprap	Rare or not present.	Occasional, causing cross currents and minor bank and bottom erosion. 5	Moderately frequent and occasionally unstable obstructions, cause noticeable erosion of the channel. Considerable sediment accumulation behind obstructions.	Frequent and often unstable causing a continual shift of sediment and flow. Traps are easily filled causing channel to migrate and/or widen.	Lots of down trees in the wetlands upstream. Channel complexity, floodplain connectivity, and riparian condition and function are heavily degraded due to bank armoring.

Table 5.5. Stability indicators, descriptions, and ratings. Range of values in ratings columns provide possible rating values for each factor.

Stability Indicator	Ratings				
	Excellent (1-3)	Good (4-6)	Fair (7-9)	Poor (10-12)	
8. Bank soil texture and coherence	Clay and silty clay; cohesive material	Clay loam to sandy clay loam; minor amounts of noncohesive or unconsolidated mixtures; layers may exist, but are cohesive materials.	Sandy clay to sandy loam; unconsolidated mixtures of glacial or other materials; small layers and lenses of noncohesive or unconsolidated mixtures 8	Loamy sand to sand; noncohesive material; unconsolidated mixtures of glacial or other materials; layers or lenses that include noncohesive sands and gravels	Soil type can be confimed by geotech - Gray silty sand.
9. Average bank slope angle (where 90° is a vertical bank) V = Vertical H = Horizontal	Bank slopes < 3H:1V (18°) for noncohesive or unconsolidated materials to < 1:1 (45°) in clays on both sides 3	Bank slopes up to 2H:1V (27°) in noncohesive or unconsolidated materials to 0.8:1 (50°) in clays on one or occasionally both banks	Bank slopes to 1H:1V (45°) in noncohesive or unconsolidated materials to 0.6:1 (60°) in clays common on one or both banks.	Bank slopes over 45° in noncohesive or unconsolidated materials or over (60°) in clays common on one or both banks	Bank slope angles vary by location. An average of 14.5 degrees
10. Vegetative or engineered bank protection	Wide band of woody vegetation with at least 90% density and cover. Primarily hard wood, leafy, deciduous trees with mature, healthy, and diverse vegetation located on the bank. Woody vegetation oriented vertically. In absence of vegetation, both banks are lined or heavily armored.	Medium band of woody vegetation with 70-90% plant density and cover. A majority of hard wood, leafy, deciduous trees with maturing, diverse vegetation located on the blank. Woody vegetation oriented 80-90° from horizontal with minimal root exposure. Partial lining or armoring of one or both banks. 4	Small band of woody vegetation with 50-70% plant density and cover. A majority of soft wood, piney, coniferous trees with young or old vegetation lacking in diversity located on or near the top of bank. Woody vegetation oriented at 70-80° from horizontal often with evident root exposure. No lining of banks, but some armoring may be in place on one bank.	Woody vegetation band may vary depending on age and health with less than 50% plant density and cover. Primarily soft wood, piney, coniferous trees with very young, old and dying, and/or monostand vegetation located off of the bank. Woody vegetation oriented at less than 70° from horizontal with extensive root exposure. No lining or armoring of banks.	
11. Bank Cutting	Little or none evident. Infrequent raw banks, insignificant percentage of total bank.	Some intermittently along channel bends and at prominent constrictions. Raw banks comprises minor portion of bank in vertical direction. 5	Significant and frequent on both banks. Raw banks comprise large portion of bank in vertical direction. Root mat overhangs.	Almost continuous cuts on both banks, some extending over most of the banks. Undercutting and sod-root overhangs.	Bank erosion was encountered in various locations along Juanita. On the downstream side scour was evident below the outlet of the culvert and lateral scour further downstream into a resident yard.
12. Mass wasting or bank failure	No or little evidence of potential or very small amounts of mass wasting. Uniform channel width over the entire reach.	Evidence of infrequent and/or minor mass wasting. Mostly healed over with vegetation. Relatively constant channel width and minimal scalloping of banks. 4	Evidence of frequent and/or significant occurrences of mass wasting that can be aggravated by higher flows, which may cause undercutting and mass wasting of unstable banks. Channel width quite irregular and scalloping of banks is evident.	Frequent and extensive mass wasting. The potential for bank failure, as evidenced by tension cracks, massive undercuttings, and bank slumping, is considerable. Channel width is highly irregular and banks are scalloped.	infrequent evidence of mass wasting, "undercut banks slumping into channel"
13. Upstream distance to bridge from meander impact point and alignment	More than 115 ft (35 m); bridge is well aligned with river flow	66-115 ft (20 - 35 m); bridge is aligned with flow	33 - 66 ft (10 - 20 m); bridge is skewed to flow or flow alignment is otherwise not centered beneath bridge 7	Less than 33 ft (10 m); bridge is poorly aligned with flow	

Juanita most consistent with Pool-Riffle from Table 3.5

Table 5.6. Overall Scores for Three Classifications of Channels.			
Category	Score, R		
	Pool-Riffle, Plane-Bed, Dune-Ripple, and Engineered Channels	Cascade and Step-Pool Channels	Braided Channels
Excellent	R < 49	R < 41	N/A
Good	49 ≤ R < 85	41 ≤ R < 70	R < 94
Fair	85 ≤ R < 120	70 ≤ R < 98	94 ≤ R < 129
Poor	120 ≤ R	98 ≤ R	129 ≤ R

1st score = 69
Revised score = ?
Result: **Good** for Pool-Riffle, **Good** for Step-Pool

Table 5.7. Stability Ratings for Streams in Figures 5.11 – 5.14.															
Stream	Indicator													Total	Rating Based on Table 5.5
	1	2	3	4	5	6	7	8	9	10	11	12	13		
Figure 5.11	12	4	6	12	11	10	10	12	12	11	12	12	3	127	Poor
Figure 5.12	9	12	10	7	11	8	3	11	8	10	6	7	8	110	Fair
Figure 5.13	8	2	4	5	3	5	5	8	5	2	2	2	6	57	Good
Figure 5.14	3	2	3	3	4	3	3	4	5	4	4	1	5	44	Excellent

Table 5.8. Lateral and Vertical Stability for Streams in Figures 5.11 – 5.14.				
Stream	Lateral	Vertical	Lateral Fraction	Vertical Fraction
Figure 5.11	62	33	0.86	0.92
Figure 5.12	50	26	0.69	0.72
Figure 5.13	25	13	0.35	0.36
Figure 5.14	23	10	0.32	0.28

Lateral Fraction = sum[4:6] = 7+5+5 = 17/36 = 0.47
Vertical Fraction = sum[8:13] = 8+3+4+5+4+7 = 31/72 = 0.43

Channel has equal (albeit low) potential for both vertical and lateral migration.

See 5.4.4

Appendix Q: Juanita Creek Hydrology and Hydraulics Study H21 Report

Culverts And Climate Change (wa.gov)	Date Accessed:	2/5/2024
Future Projection for Climate-Adapted Culvert Design		
Projected mean percent change in bankfull flow		
2040s:	10.5	
2080s:	13.6	
Projected mean percent change in bankfull width		
2040s:	5	
2080s:	6.3	
Projected mean percent change in 100-year flood:		
2040s:	11.3	
2080s:	21.8	

Table 1: High Woodlands HSPF Peak Flow Rate (in cfs) for Various Storm Frequencies Based on 15- minute time step

	Project Scenario	2-year	10-year	25-year	100-yr
At I-405	Pre	25	32	35	41
	Post	40	65	81	111
Above Norway Hill Trib	Pre	30	39	44	51
	Post	45	74	94	129
At 141st	Pre	65	101	124	166
	Post	81	135	172	242

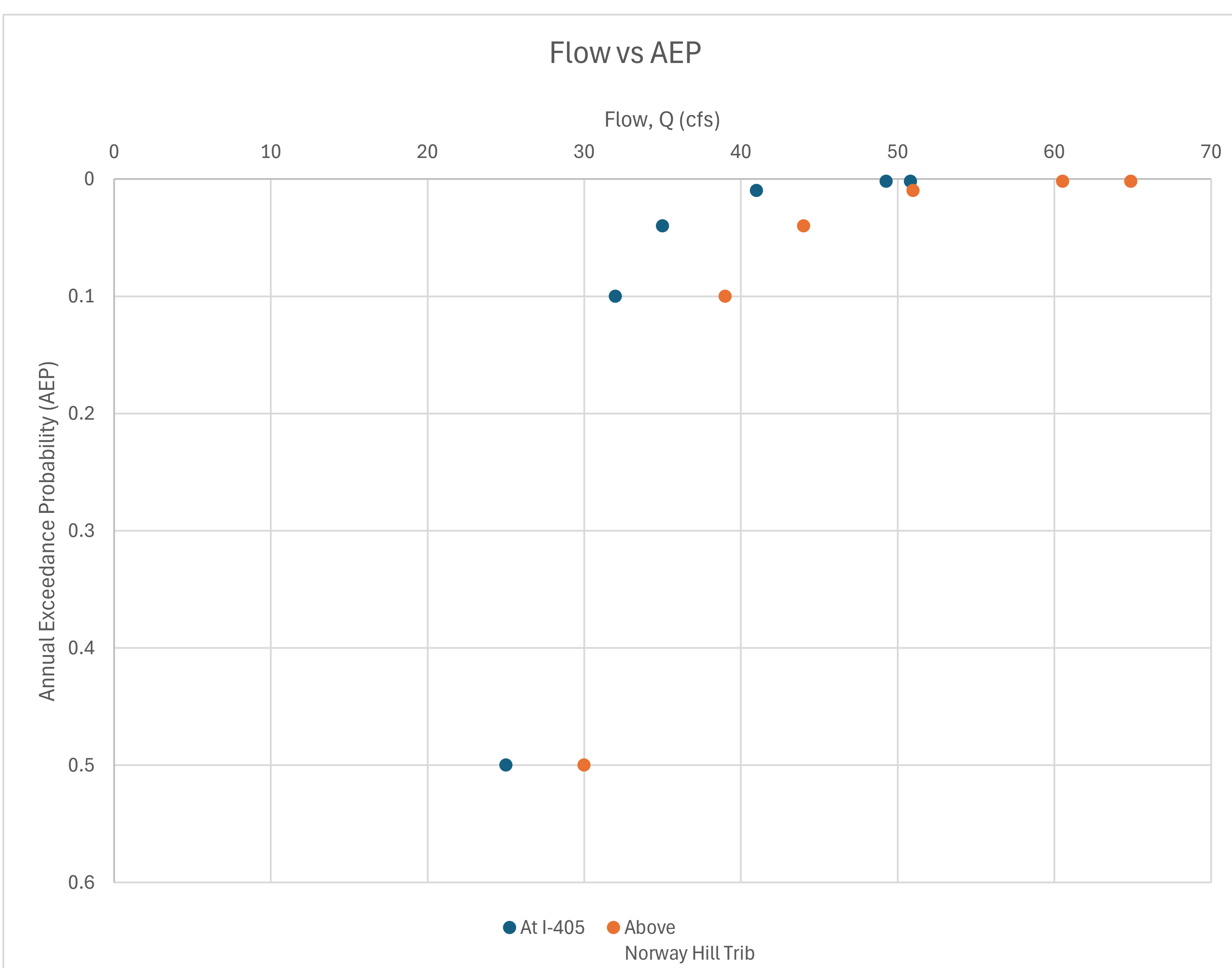
Refer to Attachment 1 for additional storm frequencies evaluated in the HSPF model.

Reference: H21 Report - I-405 Brickyard Park and Ride Iline Freeway Station BRT Project - Technical Memorandum -Juanita Creek Hydrology and Hydraulics Study

Juanita -Existing						
			At I-405		Above Norway Hill Trib	
	AEP	Log(RP)	Q (cfs)	Log(Q)	Q (cfs)	Log(Q)
2yr	0.5	-0.30103	25	1.39794	30	1.477121255
10yr	0.1	-1	32	1.50515	39	1.591064607
25yr	0.04	-1.39794	35	1.544068	44	1.643452676
100yr	0.01	-2	41	1.612784	51	1.707570176
500yr all	0.002	-2.69897	51	1.7061	65	1.812121348
500yr last 2	0.002	-2.69897	49	1.69256	61	1.782008288
100yr 2040			46		57	
100yr 2080			50		62	

Note: Green is what is used as an inflow for the 500-year modeled event

Note: Red values are for reference and are not used for extrapolation



I-405 Brickyard Park and Ride Inline Freeway Station BRT Project

Technical Memorandum

Juanita Creek Hydrology and Hydraulics Study

Purpose

This technical memorandum documents the hydrology that shall be used for final design of the Juanita Creek Fish Passage to be completed by the Design-Builder. The Design-Builder shall continue to coordinate with the City of Kirkland and WSDOT with respect to the hydrology of the Juanita Creek basin.

Introduction

To comply with United States, et al vs. Washington, et al No. C70-9213 Subproceeding No. 01-1 dated March 29, 2013 (a federal permanent injunction requiring the State of Washington to correct fish barriers in Water Resource Inventory Areas (WRIAs) 1-23), WSDOT is proposing a project to provide fish passage at the I-405 crossing of Juanita Creek at Mile Post (MP) 21.94. This existing structure on I-405 has been identified as a fish barrier by Washington Department of Fish and Wildlife (WDFW) and WSDOT Environmental Services Office (ESO) (Site ID 998602) and has an estimated 1,309 feet of habitat gain.

Per Section 9 of the injunction, WSDOT is required to design and build a restored stream connection at each identified barrier to pass all species of salmon at all life stages at all flows where the fish would naturally seek passage. To that end, WSDOT evaluated design options as defined in Section 9 of the injunction. WSDOT is proposing to replace the existing culvert with a crossing structure designed to restore the stream connection under the I-405 roadway and simulate natural stream functions that meets the terms of the injunction. In May 2021, WSDOT completed the Preliminary Hydraulic Design (PHD) Report which details the design elements necessary comply with the federal injunction as coordinated with the permitting agencies and affected tribe. See Appendix H08 for the Juanita Creek PHD Report.

The proposed project will remove the existing flow control structure that is connected to the existing culvert and provides outlet control for the City of Kirkland owned High Woodlands Regional Detention Facility on the main stem of Juanita Creek.

Following the completion of the PHD Report, WSDOT coordinated with the City of Kirkland and their Consultant, Northwest Hydraulic Consultants (NHC) to better understand the basin hydrology and floodplain impacts for existing and proposed conditions. This technical memorandum documents the hydrology to be used for final design and the findings for potential downstream effects that result from the removal of the flow control structure. Refer to **Figure 1** for an illustration of the downstream facilities that were surveyed as part of this study.

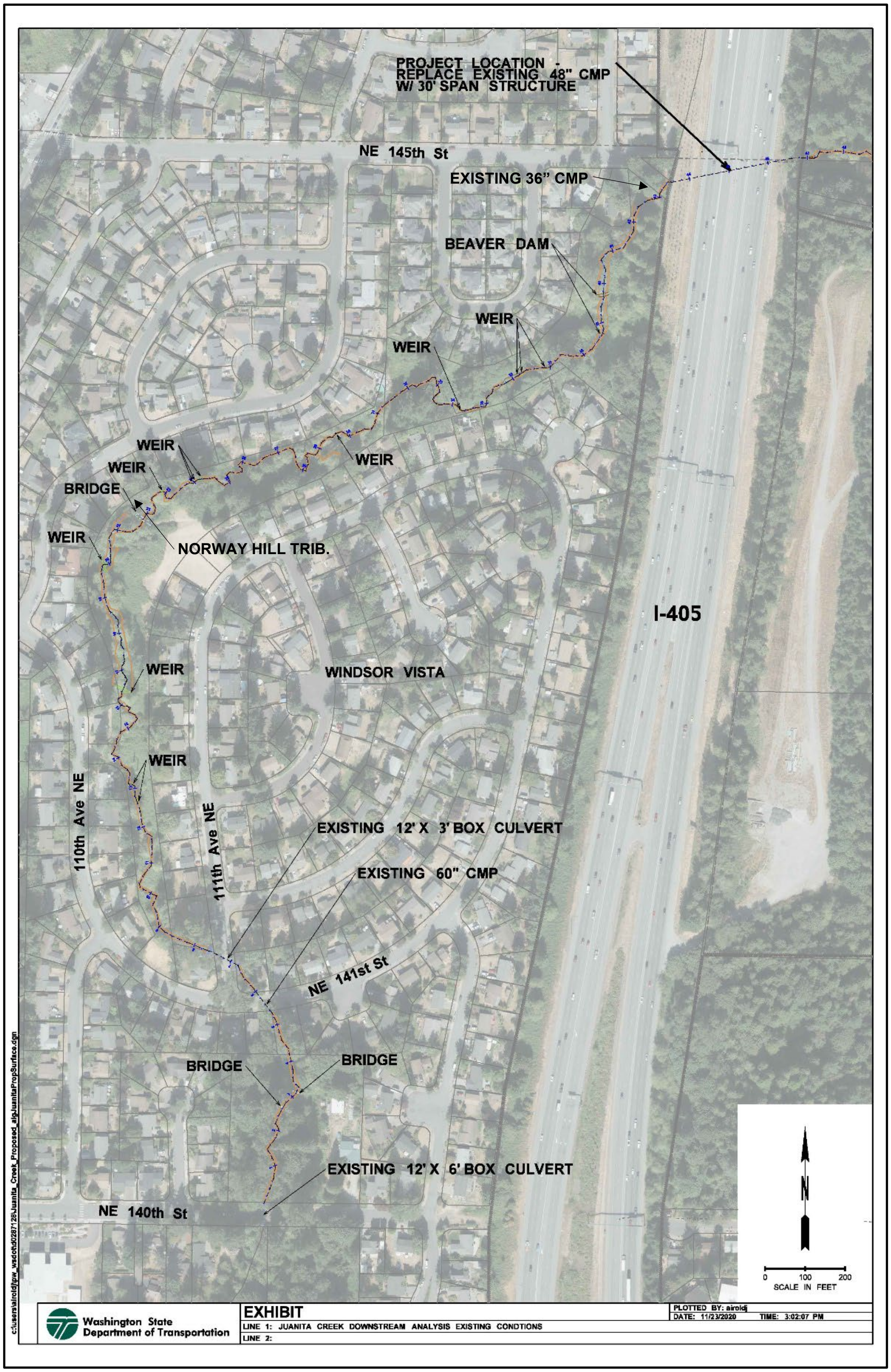


Figure 1: Vicinity Map and Channel Survey Limits

Analysis Approach

Hydrology

The study began with a 2019 to 2021 collaboration effort between WSDOT, the City of Kirkland and King County to understand the existing High Woodlands Regional Detention Facility features and hydrologic effect on flows downstream of I-405.

The existing 48-inch culvert is currently connected to a City of Kirkland owned control structure that was designed by King County in 1990 and completed by 1994. The "High Woodlands Regional Detention Pond Project #047007" utilizes the Juanita Creek ravine to the east of I-405 for detention storage while the control structure meters flow through the 48-inch culvert. The original King County design utilized a single-event Santa Barbara Unit Hydrograph (SBUH) model and documented a design intent to reduce flows downstream by varying amounts between 16% for the 2-year storm up to 28% for the 100-year storm event. Based on field inspections, the original design ponding depths do not appear to match the actual performance within the regional pond. This study looked at available data and utilized more current King County HSPF continuous hydrograph simulation model to better understand the existing function of the control structure and effects downstream.

To understand the hydraulic conditions for the site, WSDOT performed Juanita Creek flow monitoring between March 16 and June 24, 2020. Monitoring equipment was installed to determine existing peak flows at five (5) locations associated with the High Woodlands Regional Detention Pond. Inflow to the regional pond was monitored within the City of Kirkland storm sewer network locations entering from the south and east. Outflow was measured at the downstream (west) end of the WSDOT culvert, while the City also monitored water levels on the upstream side of the flow control structure.

King County Juanita Creek Rain Gauge (27u) was used to estimate rainfall in the basin.

Ground surveys were conducted in 2014 to provide insight into detention volume function provided by the pond. The control structure has steel plates to facilitate variable control opening settings, so the City of Kirkland maintenance crews measured the actual control structure opening in Fall 2019. City of Kirkland Maintenance Records 1993 to 2021 show that there was very little change in the flow control settings since this regional pond was annexed into the city. This information was used to create an accurate stage-storage discharge for the regional pond.

Using this information, the initial modeling found that the standard (regionally applicable) continuous hydrology models, MGSFlood and WWHM, did not predict the flows and pond function compared to observed storage. Working with the City of Kirkland and King County, we also found that the 2012 King County HSPF Juanita Creek Basin Plan models were not calibrated well to reflect the regional pond performance for the small storm data from the flow monitoring. To address this, the City of Kirkland used current GIS data to update existing land cover for the study area and contracted Northwest Hydraulics Consultants (NHC) to perform a calibration of a 2012 King County HSPF model for Juanita Creek. Their results, documented in **Table 1**, shall be used as the basis for the final design unless otherwise directed by the City of Kirkland and WSDOT.

Hydraulics

The hydraulic model of the I-405 Juanita Creek crossing was built using ground survey data that was collected in the Fall of 2020. This hydraulic model utilized U.S. Bureau of Reclamation's SRH-2D Version 3.2.3 software. This program is a two-dimensional hydraulic and sediment transport numerical model that uses the Federal Highway Administrations (FHWA) HY-8 for culvert hydraulic computations. Pre- and post-processing for this model was completed using Aquaveo's SMS Version 13.0.10. The hydraulic analysis performed for this scenario utilized the 100-yr design storm event as the base line for assessing potential downstream effects.

Hydrology and Peak Flow Estimates

An extensive investigation into the hydrology of the Juanita Creek subbasins was conducted to ensure all stakeholders came to an agreement on the existing and projected peak flows in Juanita Creek as a result of the removal of the High Woodlands Regional Detention Facility.

Peak flows for the study were determined by NHC and provided to the City of Kirkland for the hydraulic analysis. NHC provided four model outputs for use in the design based on their updated and calibrated King County HSPF model.

Table 1: High Woodlands HSPF Peak Flow Rate (in cfs) for Various Storm Frequencies Based on 15- minute time step

	Project Scenario	2-year	10-year	25-year	100-yr
At I-405	Pre	25	32	35	41
	Post	40	65	81	111
Above Norway Hill Trib	Pre	30	39	44	51
	Post	45	74	94	129
At 141st	Pre	65	101	124	166
	Post	81	135	172	242

Refer to Attachment 1 for additional storm frequencies evaluated in the HSPF model.

Table 1 peak flow rates were applied for the steady state hydraulic models.

NHC also generated a 100-year storm event hydrograph to serve as the basis for unsteady state analyses. **Table 3** is the 100-year flows during each 15-minute time step over a 6-hour hydrograph.

The map below (**Figure 2**) shows the basin setup where the yellow boundaries are the original King County model delineations while the shaded areas show the current boundaries from GIS data. Refer to **Table 2** that serves as a legend describing where those basin flows are applied to the Juanita Creek hydraulic model. Basin 1211 is at the top of the basin (inflow to the upstream of the existing pond) and all the subsequent numbers are lateral flows that were added in at the appropriate locations.

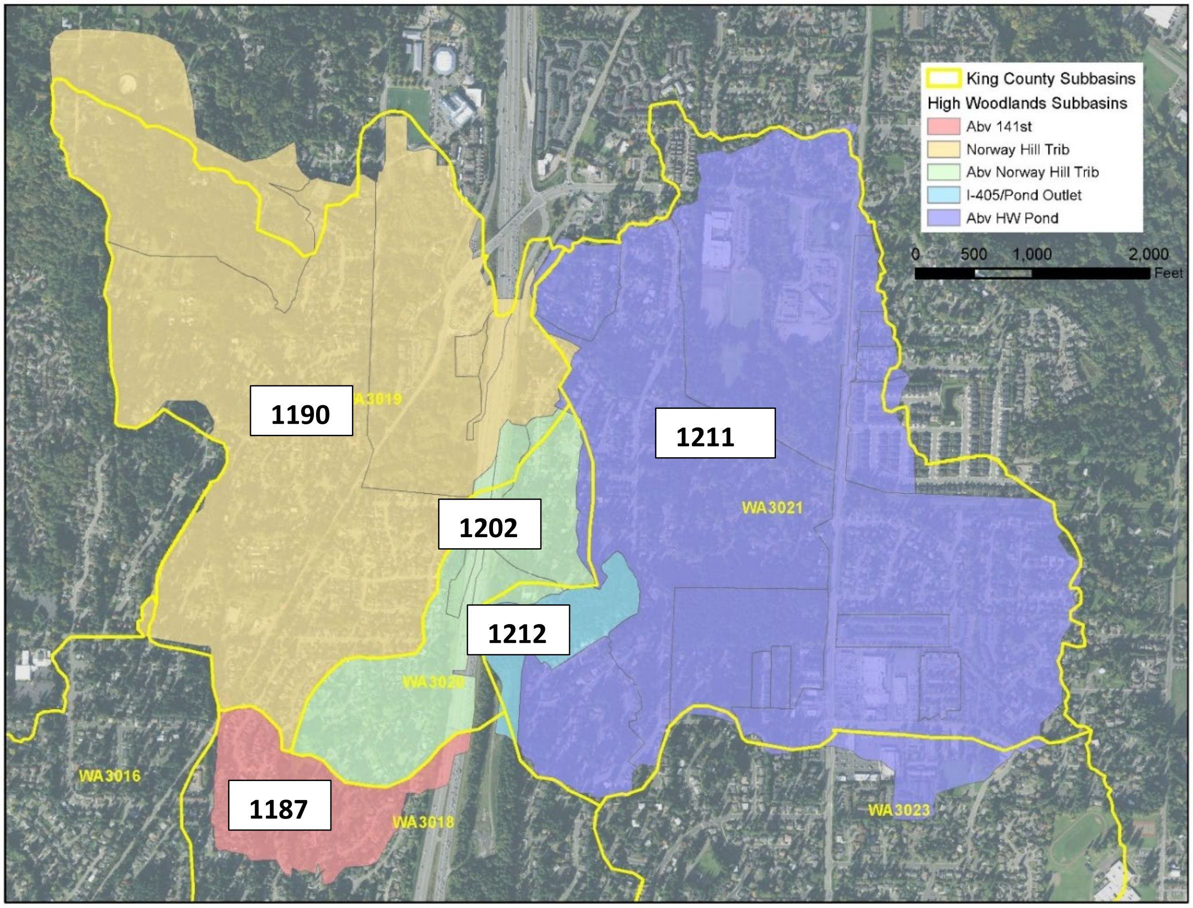


Figure 2: HSPF Basin Delineation Map for the Study Area

Table 2: Basin Identification and Application in the Unsteady State Hydraulics Model

Basin	Represents	Add at
1211	Inflow upstream of existing pond	upstream end of model
1212	Local pond area inflow	upstream of I-405 culvert
1202	Local inflow to reach d/s of I-405	downstream of I-405 culvert
1190	Norway Hill trib	
1187	Local inflow d/s Norway Hill trib	Downstream of Norway Hill trib

Table 3: HSPF hydrograph (in cfs) for the 100-year Storm Frequency Event Proposed Condition Based on 15- minute time step

Time	1211	1212	1202	1190	1187
0:15	12.79	0.21	2.59	9.69	1.53
0:30	15.89	0.22	2.86	11.59	1.66
0:45	16.73	0.22	2.88	12.22	1.66
1:00	14.94	0.19	2.35	10.45	1.27
1:15	12.66	0.17	1.98	8.91	1.04
1:30	11.14	0.16	1.76	7.79	0.90
1:45	18.16	0.36	3.78	15.17	2.38
2:00	25.95	0.42	5.09	21.81	3.19
2:15	27.43	0.34	4.55	20.53	2.72
2:30	22.93	0.24	3.21	15.00	1.78
2:45	17.34	0.20	2.48	11.38	1.33
3:00	13.09	0.17	1.82	8.69	0.90
3:15	11.00	0.17	1.66	7.44	0.82
3:30	12.63	0.21	2.25	9.08	1.25
3:45	15.34	0.22	2.72	10.98	1.54
4:00	71.27	2.43	15.39	77.78	10.68
4:15	112.49	2.94	19.58	108.79	13.07
4:30	75.39	1.30	11.56	60.68	7.09
4:45	51.24	0.76	7.64	38.14	4.73
5:00	34.55	0.32	4.14	20.85	2.23
5:15	26.62	0.29	3.59	16.52	2.02
5:30	21.36	0.22	2.54	11.84	1.28
5:45	15.59	0.20	1.91	9.03	0.89
6:00	12.99	0.19	1.78	7.89	0.83

Hydraulic Analysis

The Juanita Creek Downstream Analysis SRH-2D Hydraulic Model was developed for two scenarios; 1) existing conditions with the existing regional detention facility and 2) proposed condition without the regional detention facility. The differences in topography between the existing and proposed models are only in the vicinity of the proposed structure which is off alignment of the existing outflow pipe from the regional detention facility. WSDOT acquired stream bathymetry from the downstream limits of the survey obtained for the I-405 crossing and downstream of NE 140th Street along Juanita Creek. Encompassed in this survey was two city-owned stream crossing structures under 111th Ave NE and NE 141st Street. Over a linear mile of channel was surveyed for this model. Outside the immediate floodplain limits, lidar was used to supplement the survey to be used in mesh generation (King County 2016).

Existing Conditions - High Woodlands Regional Detention Facility Outflows

The existing conditions hydraulic model was created to simulate the existing function of the High Woodlands Regional Detention Facility on Juanita Creek upstream of I-405 as well as the existing hydraulics downstream to NE 140th Street (**Figure 3**). The hydraulic roughness values between channel banks were assigned a n-value of 0.065 for the entire length of the model. Overbank and floodplain areas were assigned a n-value of 0.08. Residential lawns and pavement were assigned n-values of 0.02 and 0.016, respectively. Each of the four existing structures along the surveyed reach of Juanita Creek were modelled within SMS using FHWA HY-8 for one-dimensional culvert hydraulics. Inputs for each structure follow the characteristics and geometry obtained in the stream survey.

The existing condition hydraulic model includes an inflow boundary condition at the head of the High Woodlands Ravine. Flow was simulated as subcritical distributed via conveyance. Additional inflows immediately downstream of the existing I-405 crossing and at the Norway Hill Tributary, upstream of the 111th Ave NE Juanita Creek crossing, round out the inflow boundary conditions for the model. The hydraulic model was run with one outflow boundary condition, upstream of the NE 140th Street crossing of Juanita Creek. A normal depth exit boundary was assigned using the SMS built-in normal depth calculator to efficiently calculation the expected water surface elevation given the streams cross section area and gradient.

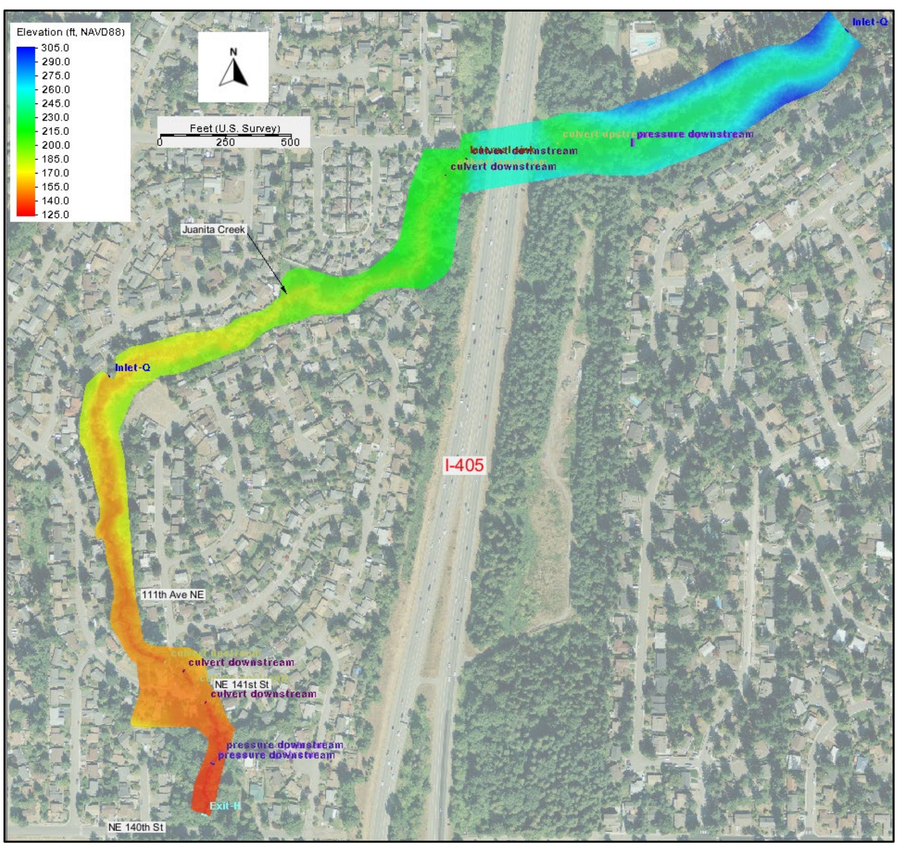


Figure 3: Existing Conditions Ground Surface Elevation Mesh, Model Extents, and Boundary Conditions

Proposed Conditions – Open Channel Flows Without Detention (Steady State)

In the proposed condition hydraulic model, the inline High Woodlands Regional Detention Facility was removed from the model and a proposed 30-foot-wide hydraulic opening was simulated as the Juanita Creek fish passage crossing of I-405. A property access culvert immediately downstream was removed from the model as it will be removed by the Design-Builder. The existing structures under 111th Ave NE and NE 141st Street were both modelled under the same conditions as the existing model. Mesh extents were the same as the existing conditions model and inflow and outflow boundary conditions are spatially the same (**Figure 4**).

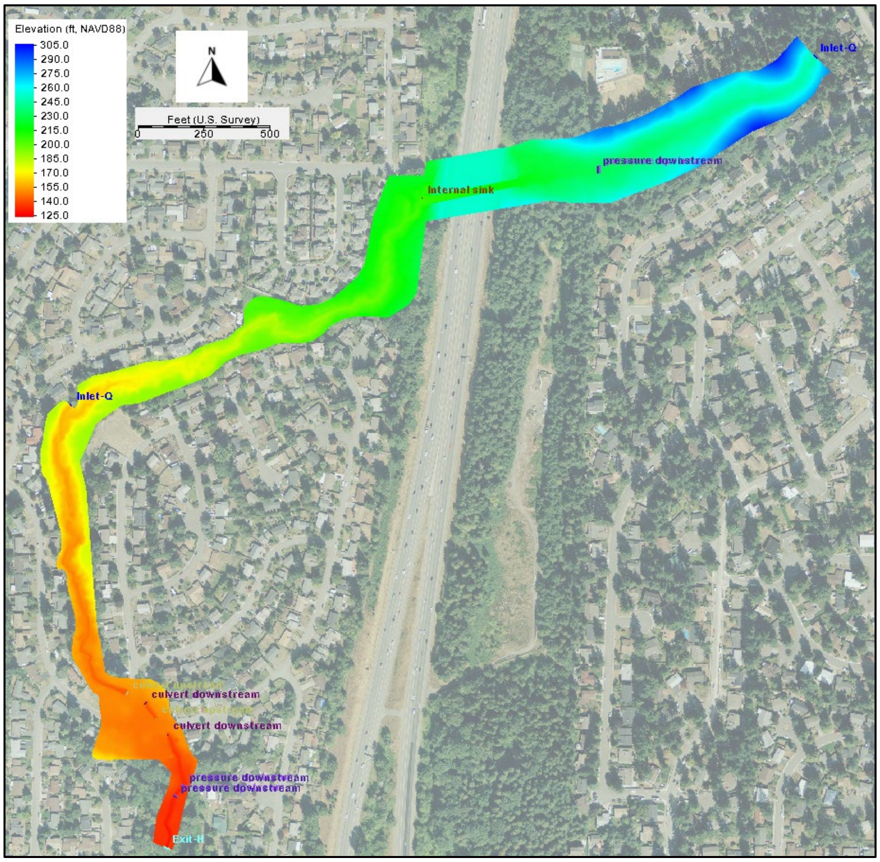


Figure 4: Proposed Conditions Ground Surface Elevation Mesh, Model Extents, and Boundary Conditions

Model Validation

The difficulties and uncertainties associated with the validation of the proposed model stem from the lack of documented observed existing flood inundation within the ravine. The Juanita Creek reach between I-405 and NE 141st Street is relatively confined, and floodplain areas are limited. As a result, large storm events are confined to the ravine. This was confirmed during the hydraulic modelling effort as shown in the Q100 existing depth map in **Figure 5**. **Figure 6** shows the Q100 proposed hydraulic model depth map. The model was developed by the WSDOT I-405 SR 167 Megaprogram staff and validated by peer review using an independent consultant expert in SRH-2D modelling.

Figure 5: Existing Conditions Q100 Extents

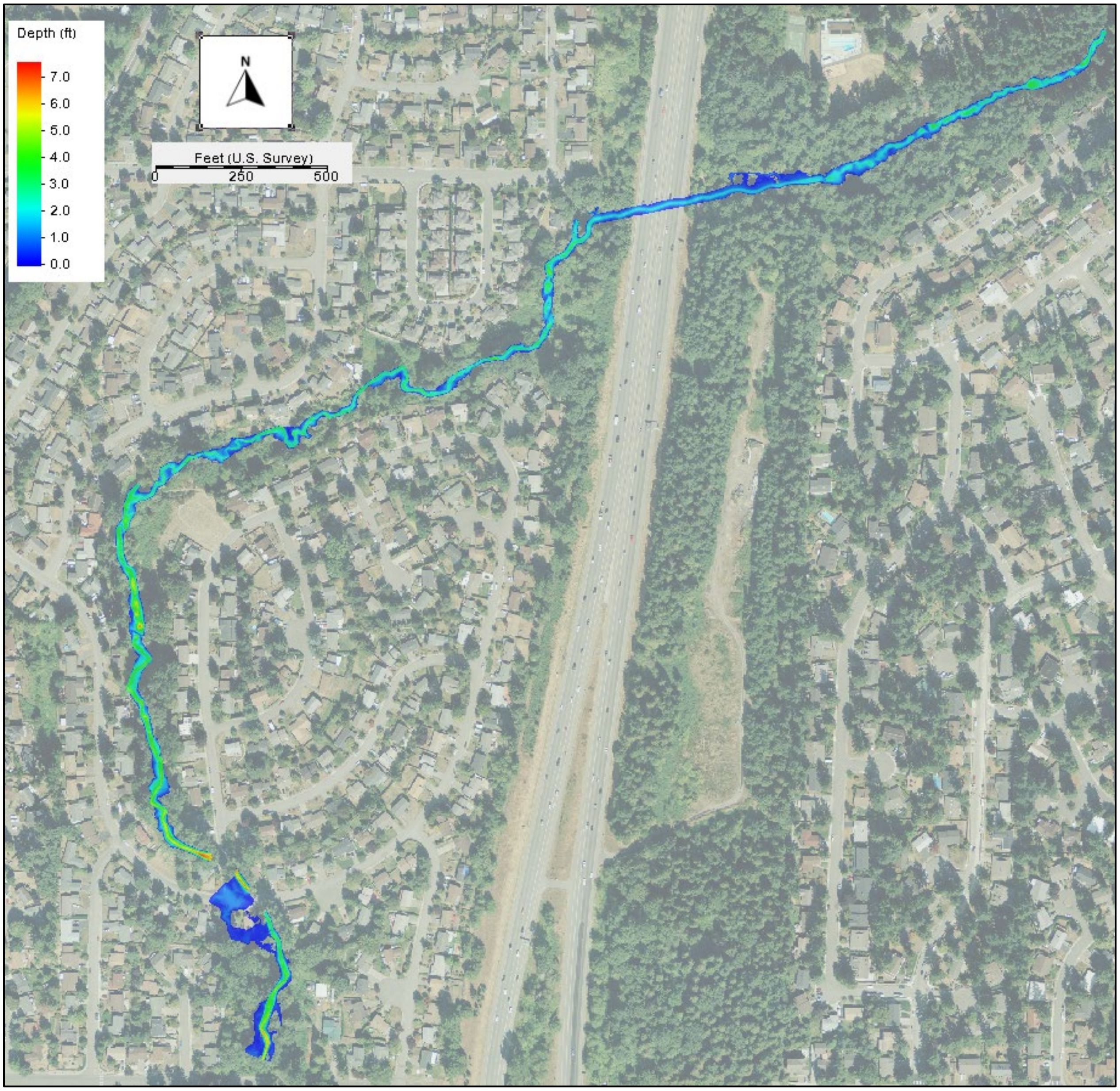


Figure 6: Proposed Conditions Q100 Extents

Analysis Results

After validating the existing and proposed conditions models, the flood inundation extents and resulting water surface elevations were compared to determine the effects to Juanita Creek downstream of I-405. Our comparison computed the spatial difference in water surface elevations between the existing and proposed scenarios and mapped the changes in inundation extents. **Figure 7** illustrates the alignment that was established to evaluate the change in water surface elevations along the longitudinal profile downstream of where the proposed crossing ties into the existing stream alignment downstream of I-405.

Figure 7 Juanita Creek Profile Line for Results Reporting

Over most of the study area the water surface elevations are expected to have a 0.5 to 1.25 foot increase downstream of I-405 as a result of this project at the 100-year event (**Figure 8**). The 100-year floodplain width would remain within the confined channel area, so this increase only has a minor change in flood width through a majority of the study area. See **Figures 8** through **Figure 11** for the steady 100-year design flow peak model findings. **Figure 11** for a plot of the relative change in water surface elevations comparing existing versus proposed results.

The areas immediately upstream of 111th Ave NE and NE 141st Street would have a greater flow depth increase of 1.5 feet. Both of these structures experience backwater from the NE 141st Street 60-inch corrugated metal pipe during existing conditions. With the additional peak flow in the proposed condition, the proposed hydraulic model shows that the backwater exceeds the channel capacity between 111th Avenue NE and NE 141st Street. The model predicts that the backwater spills into the adjacent local roadway and downstream through the private properties as illustrated by **Figure 9**.

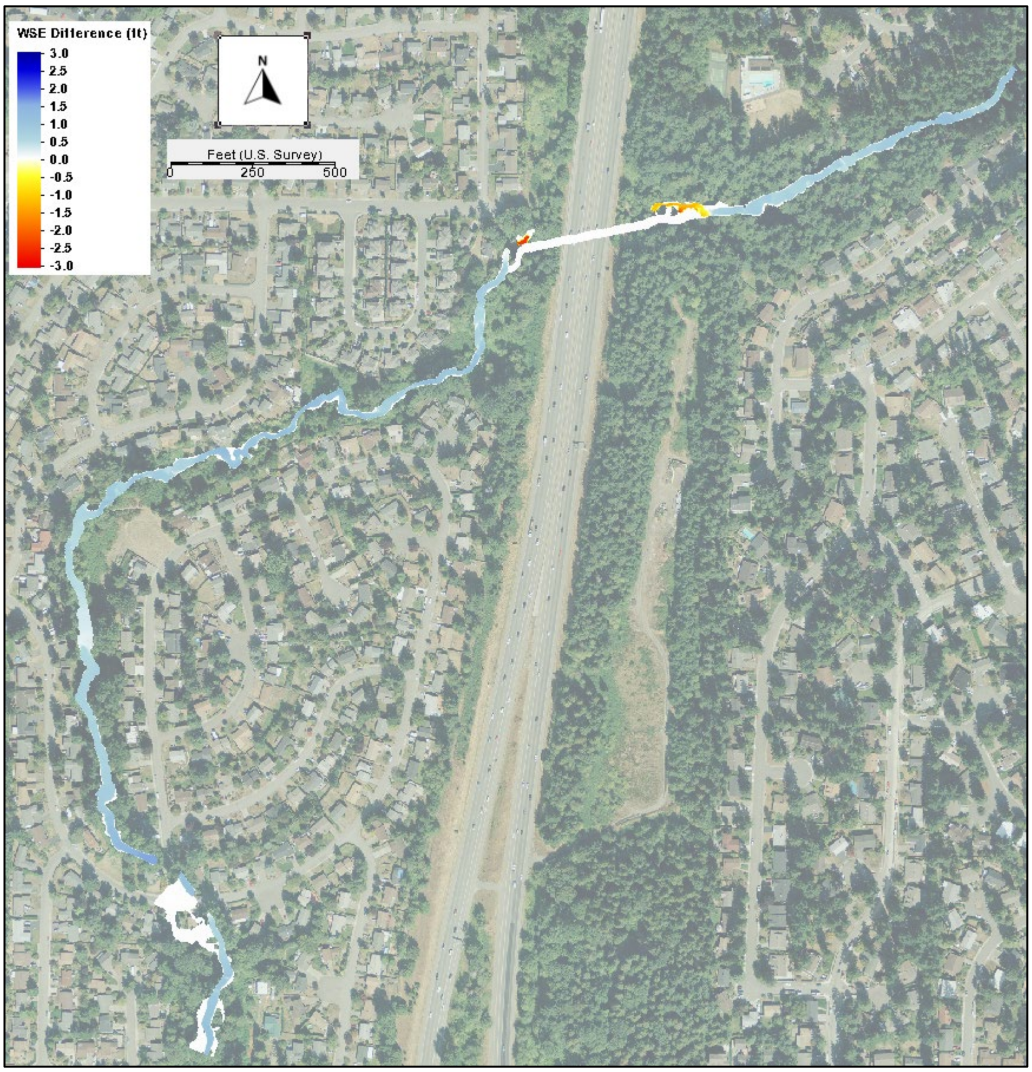


Figure 8: Comparison of Q100 Existing and Proposed Water Surface Elevation

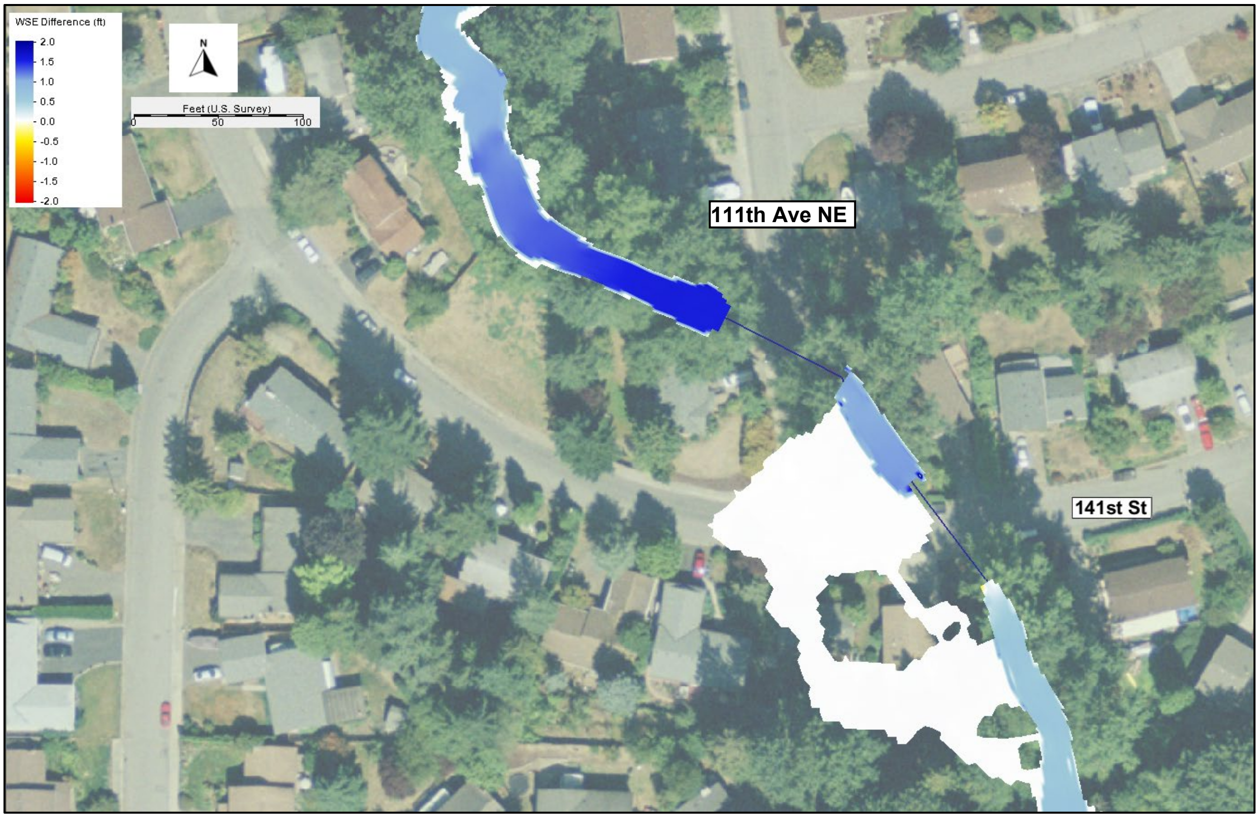


Figure 9: 111 Avenue NE and NE 141 Street Intersection Q100 Flood Inundation

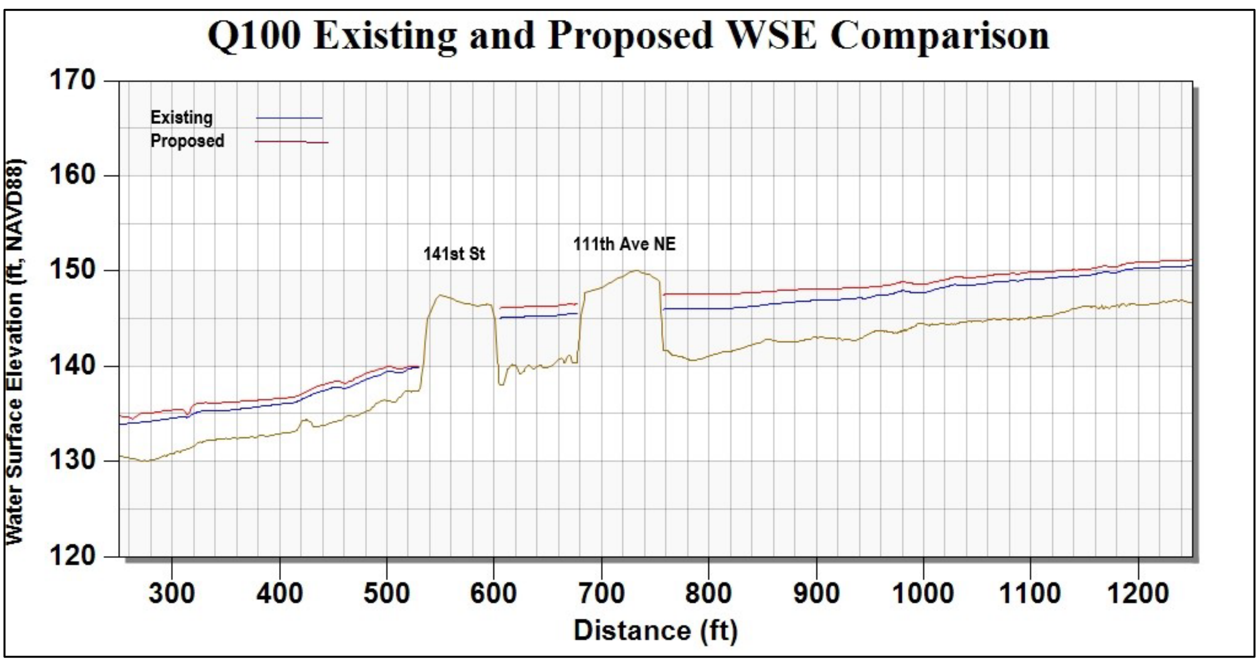


Figure 10: 141st St and 111th Ave Vicinity Q100 WSE Comparison

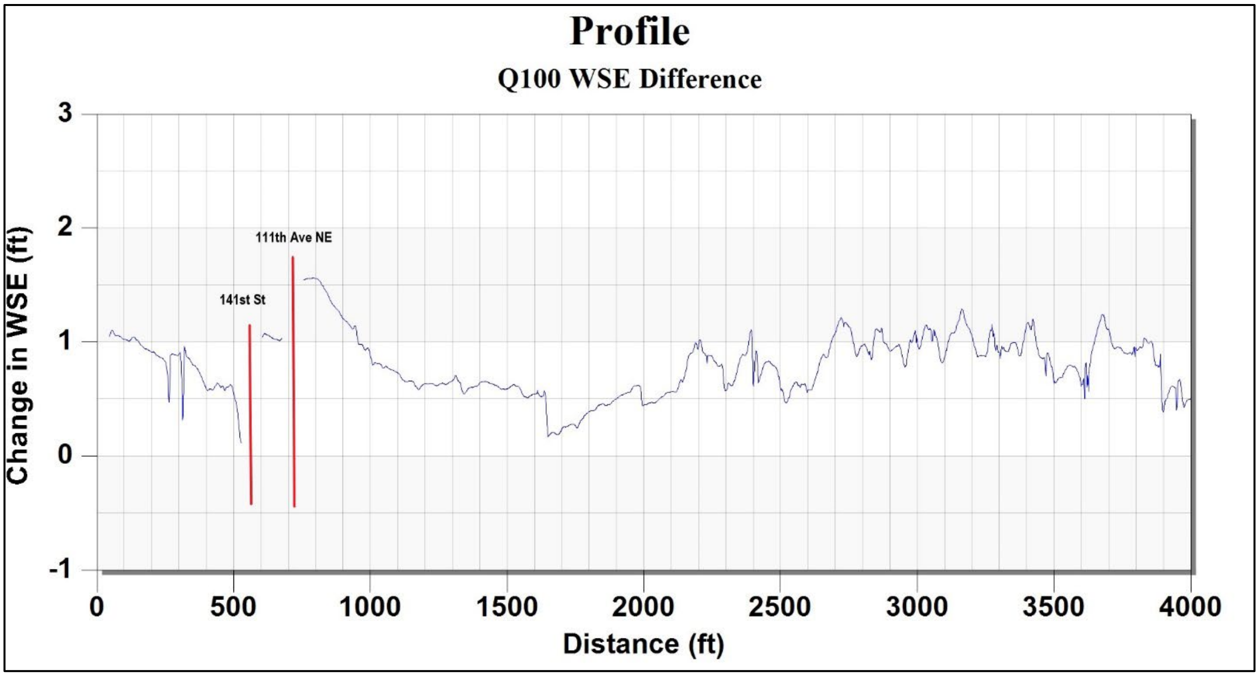


Figure 11: Juanita Creek Change in Water Surface Elevation from NE 140th Street (at Distance 0) to I-405 (at Distance 4000)

Conclusion

The study provides a new calibrated/vetted understanding of hydrology and hydraulics within the upper reaches of Juanita Creek and serves as a reliable resource for continued work in restoring a natural stream and fish habitat through the area.

It analyzed and quantified the effect of removing the City of Kirkland owned High Woodlands Regional Detention Facility. Downstream flows increase the greatest at I-405 and the effect diminish as runoff is added from various tributaries.

The peak 100-year flood elevations increase but the channel continues to stay confined within the existing terrain without impacting private properties along the Winsor Vista Park greenbelt.

This study identified the potential impacts caused by City of Kirkland's undersized culvert at NE 141st Street. Outside of the Design-Build contract, WSDOT and the City of Kirkland will continue to coordinate to further investigate solutions to the area around NE 141st Street.

References

Aquaveo. (2018). SMS Version 13.0.10

King County (2016). *PSLC King County 2016-2017 LiDAR*. Puget Sound LiDAR Consortium.

Washington State Department of Transportation (2019). *Hydraulics Manual*. Olympia, WA. Publication Number M 23-03.06

Washington State Department of Transportation (2021). *I-405 MP 21.94 Juanita Creek (998602) Preliminary Hydraulic Design Report*. Environmental and Engineering Programs, Hydraulics Office.

Attachments

Attachment 1 – Summary R1a

Attachment 2 – 100 Year, 6 Hour Hydrograph for SRH2D Unsteady State Model

Attachment 1

Basin Area Comparison

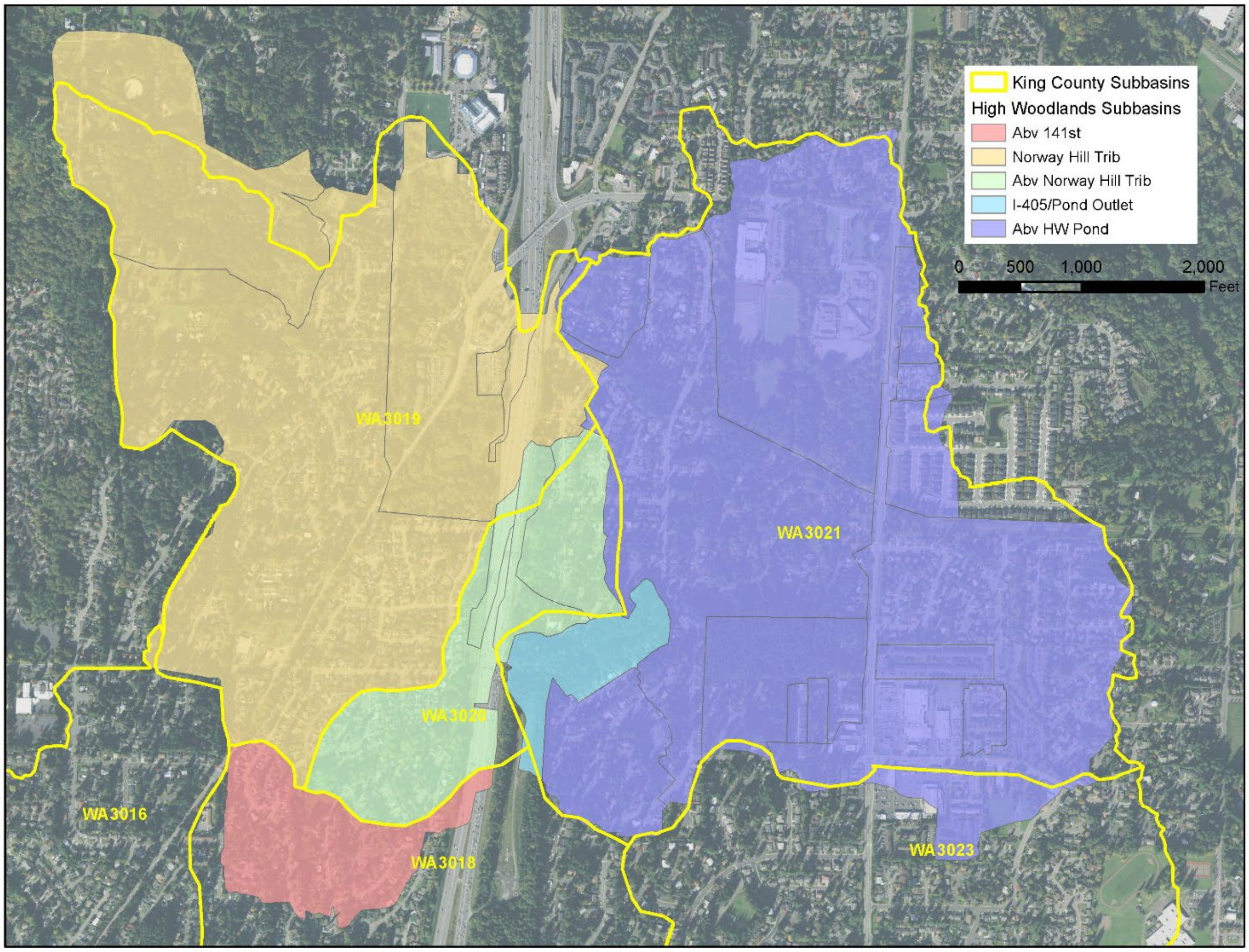
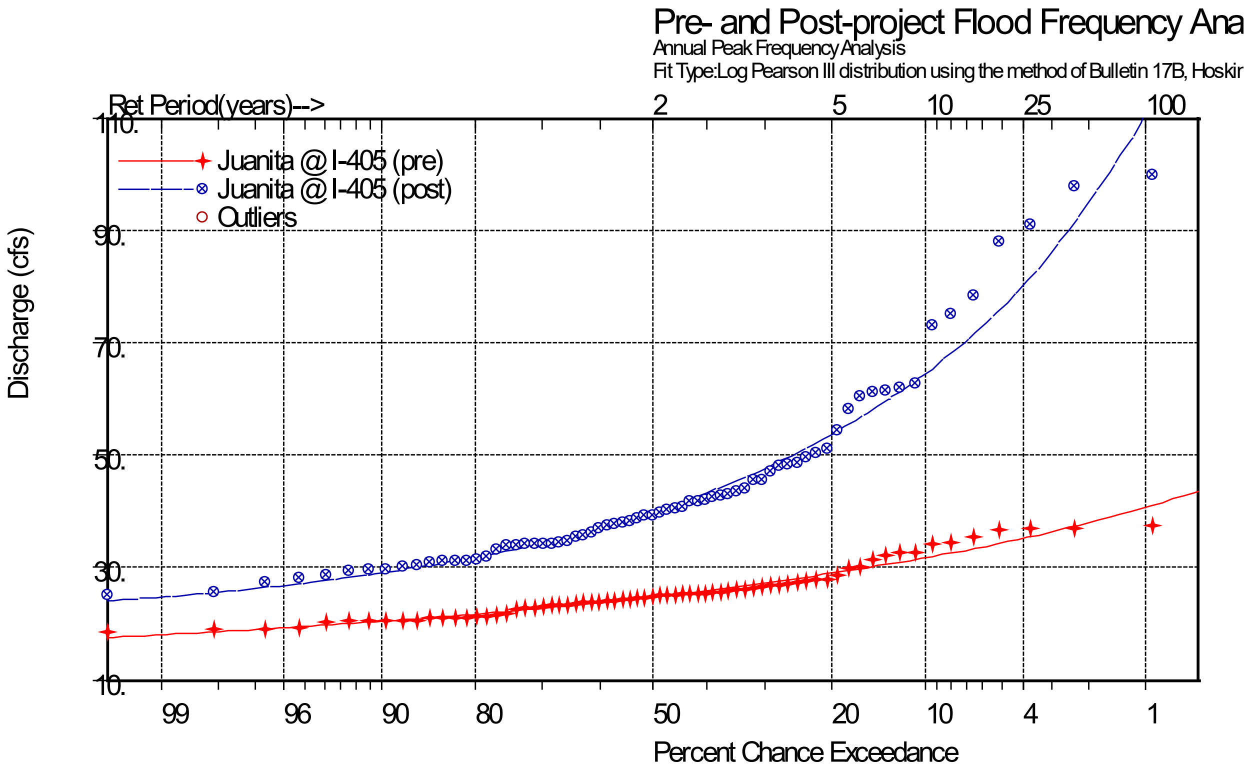


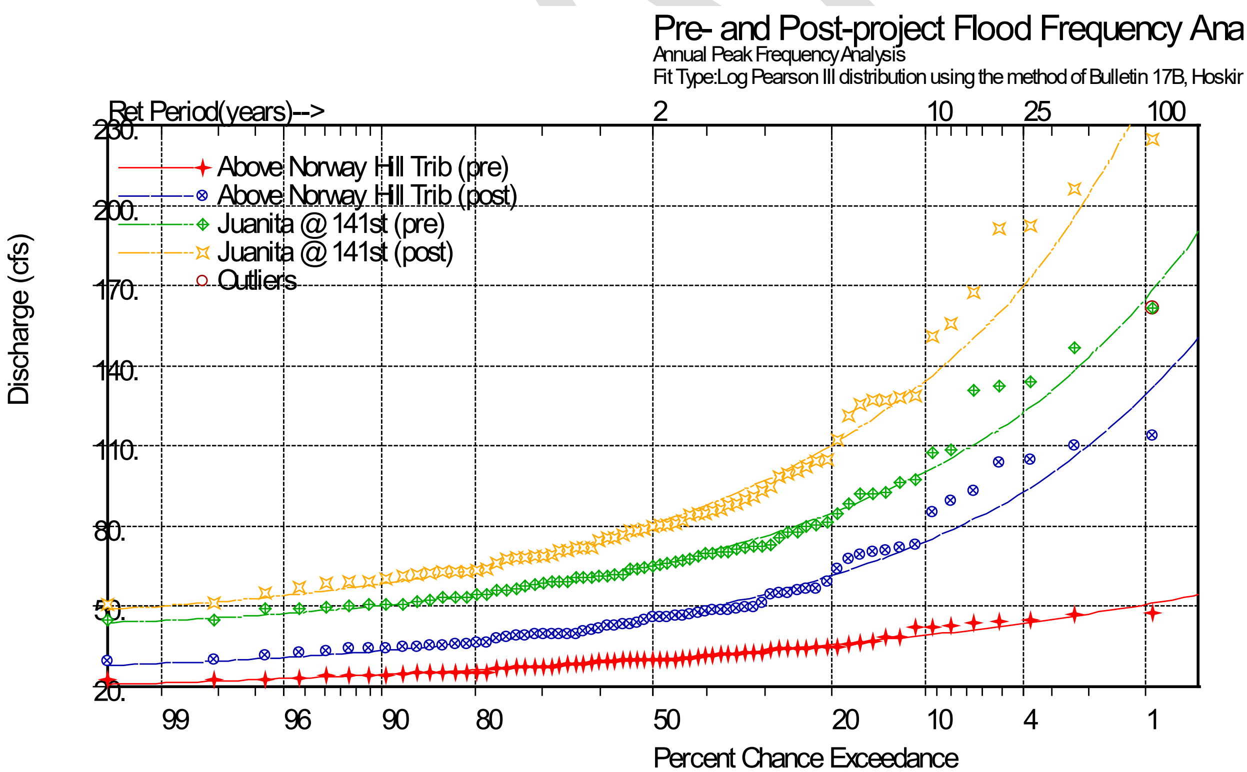
Figure 1. Basin area comparison for King County Juanita model and Kirkland/NHC High Woodlands update. High Woodlands model subbasins are grouped by outlet point, more or less corresponding to King County's larger subbasins. Land use updates in the High Woodlands model extend only to the City's downstream point of interest at NE 141st Street, partway into King County's subbasin 3018. The remainder of the subbasin downstream of NE 141st Street retains King County's routing and land use breakdown (adjusted for the area updated).

Drainage Area	King County Juanita Model		Kirkland/NHC Update	
	Total (ac)	EIA (ac)	Total (ac)	EIA (ac)
Norway Hill Trib (KC 3019)	285.6	84.9	325.8	80.9
Local abv Norway Hill Trib (KC 3020)	68.9	23.3	69.8	22.5
I-405 (KC 3021)	416.1	114.9	415.6	118.6
Total	**770.6**	**223.1**	**811.2**	**222.0**

Flood Frequency Analysis (15-minute peaks)



a) At I-405



b) Downstream of I-405

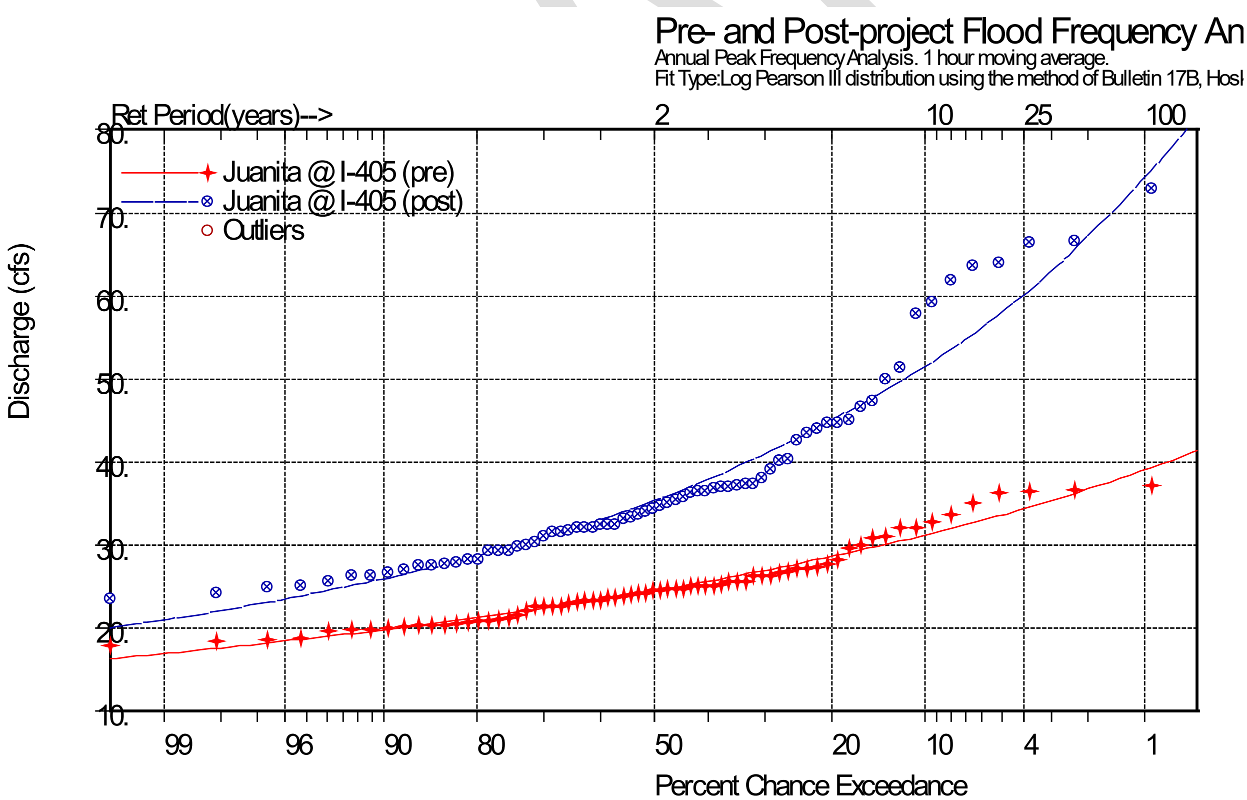
High Woodlands HSPF Flow Frequency Summary – 15-minute

Location	Project Scenario	Recurrence Interval Flow (cfs)			
		2-yr	10-yr	25-yr	100-yr
At I-405	Pre	25	32	35	41
	Post	40	65	81	111
Abv Norway Hill Trib	Pre	30	39	44	51
	Post	45	74	94	129
At 141st	Pre	65	101	124	166
	Post	81	135	172	242

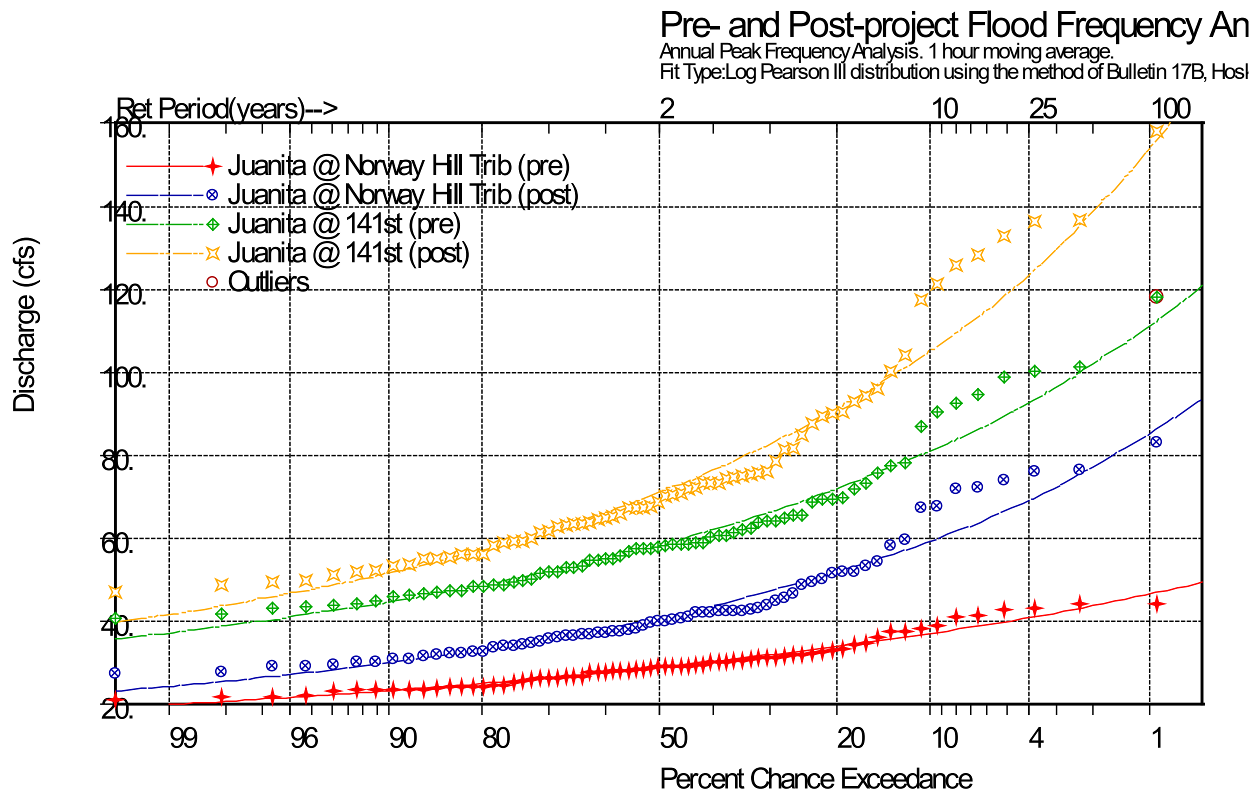
Flow Duration Comparison

Location	Pre-Proj 2-yr (cfs)	Time Exceeding Current 2-yr Flow			
		Percent of Time (%)		Avg Hours per Year	
		Pre	Post	Pre	Post
At I-405	25	2.1	5.7	184	500
Abv Norway Hill Trib	30	1.7	4.8	149	421
At 141st	65	0.7	1.6	61	140

Flood Frequency Analysis (1-hour peaks for comparison with data from King County)



a) At I-405



b) Downstream of I-405

High Woodlands HSPF Flow Frequency Summary – 1-hour

Location	Project Scenario	Recurrence Interval Flow (cfs) 2-yr	10-yr	25-yr	100-yr
At I-405	Pre	25	31	35	39
	Post	35	52	61	74
Abv Norway Hill Trib	Pre	29	37	41	47
	Post	41	60	70	86
At 141st	Pre	58	81	93	112
	Post	71	106	124	154

Comparison to King County Frequency Analysis

Location	POC	Project Scenario	2-yr Flow (cfs) KC	2-yr Flow (cfs) NHC	100-yr Flow (cfs) KC	100-yr Flow (cfs) NHC
At I-405 (Model reach 3021/210)	8	Pre	24	25	42	39
		Post	32	35	61	74
Abv Norway Hill Trib (Model reach 3020/200)	7	Pre	29	29	51	47
		Post	38	41	72	86

Attachment 2

The map below (Figure 1) shows the locations of where the flows are calculated, along with the Table 1 as a legend describing where those flows are located within the basin. 1211 is at the top of the basin (inflow to the upstream of the existing pond) and all the subsequent numbers are lateral flows that should be added in at the appropriate locations. Table 2 is the 100-year flows over a 6 year hydrograph, starting from the top of the basin.

Figure 1

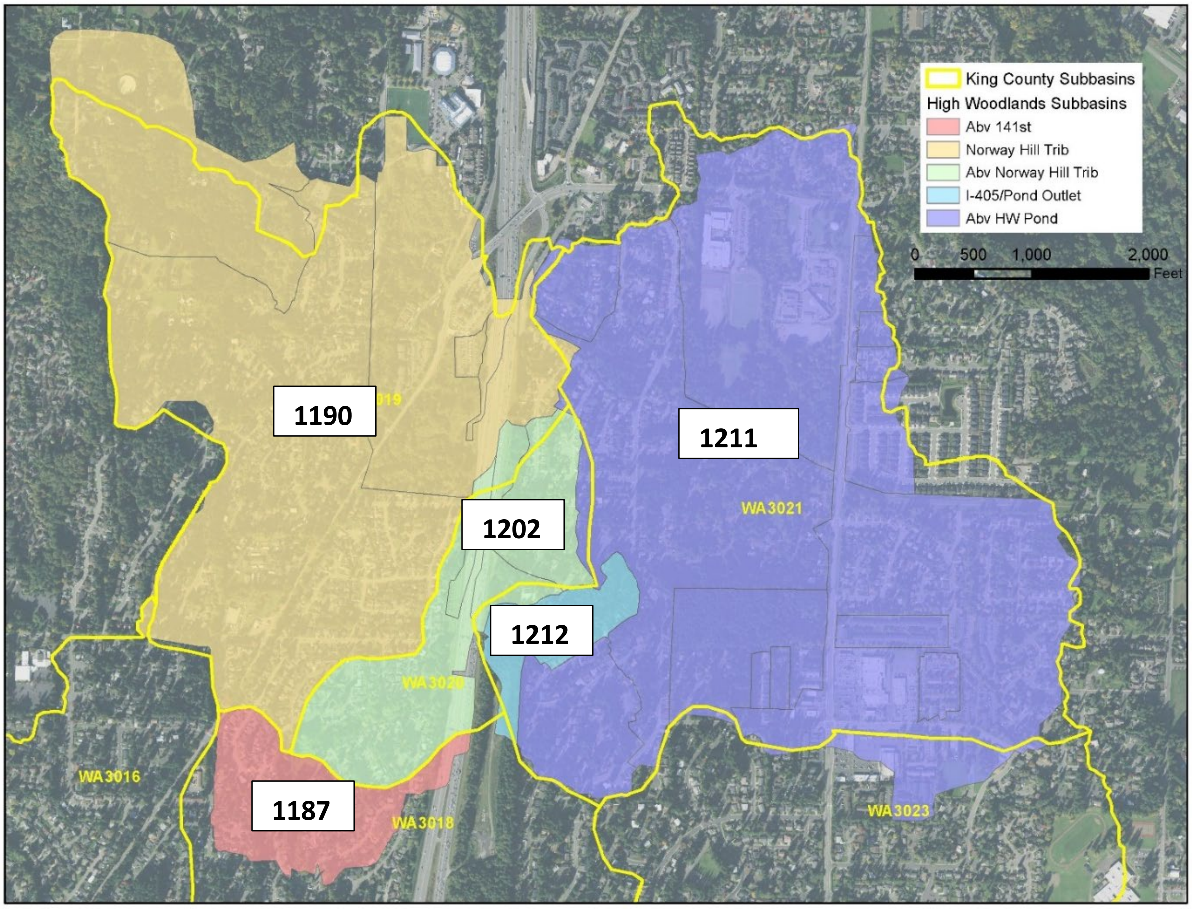


Table 1.

	Represents	Add at
1211	Inflow upstream of existing pond	u/s end of model
1212	Local pond area inflow	u/s of I-405 culvert
1202	Local inflow to reach d/s of I-405	d/s of I-405 culvert
1190	Norway Hill trib	
1187	Local inflow d/s Norway Hill trib	d/s Norway Hill trib

Table 2.

1211	1212	1202	1190	1187
12.79	0.21	2.59	9.69	1.53
15.89	0.22	2.86	11.59	1.66
16.73	0.22	2.88	12.22	1.66
14.94	0.19	2.35	10.45	1.27
12.66	0.17	1.98	8.91	1.04
11.14	0.16	1.76	7.79	0.90
18.16	0.36	3.78	15.17	2.38
25.95	0.42	5.09	21.81	3.19
27.43	0.34	4.55	20.53	2.72
22.93	0.24	3.21	15.00	1.78
17.34	0.20	2.48	11.38	1.33
13.09	0.17	1.82	8.69	0.90
11.00	0.17	1.66	7.44	0.82
12.63	0.21	2.25	9.08	1.25
15.34	0.22	2.72	10.98	1.54
71.27	2.43	15.39	77.78	10.68
112.49	2.94	19.58	108.79	13.07
75.39	1.30	11.56	60.68	7.09
51.24	0.76	7.64	38.14	4.73
34.55	0.32	4.14	20.85	2.23
26.62	0.29	3.59	16.52	2.02
21.36	0.22	2.54	11.84	1.28
15.59	0.20	1.91	9.03	0.89
12.99	0.19	1.78	7.89	0.83