US Army Corps of Engineers®
Walla Walla District

LONG-TERM SEDIMENT MANAGEMENT FOR THE FEDERAL NAVIGATION CHANNEL AT OR NEAR THE CONFLUENCE OF THE LOWER SNAKE AND CLEARWATER RIVERS

ASOTIN, WHITMAN, AND GARFIELD COUNTIES, WASHINGTON, AND NEZ PERCE COUNTY, IDAHO

TIERED ANALYSIS FROM THE LOWER SNAKE RIVER PROGRAMMATIC SEDIMENT MANAGEMENT PLAN FINAL ENVIRONMENTAL IMPACT STATEMENT

TIERED ENVIRONMENTAL IMPACT STATEMENT

APPENDIX A, HYDRAULICS AND SEDIMENT

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APPENDIX A, HYDRAULICS AND SEDIMENT

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EXECUTIVE SUMMARY

Recurring sediment deposition in the Lower Snake River at the confluence of the Snake and Clearwater Rivers impedes commercial navigation to the Ports of Lewiston and Clarkston. This necessitates periodic dredging to maintain the federally authorized 14-foot channel depth. Hydraulic and sediment analysis was conducted to evaluate long-term sediment management measures outlined in the Programmatic Sediment Management Plan (PSMP), with the objective of reducing the volume and frequency of dredging. The effectiveness of various measures was assessed using one-dimensional mobile bed modeling and two-dimensional hydraulic modeling, supplemented by data from a historical drawdown in 1992 at Lower Granite Dam. The most effective measures were a navigation channel reconfiguration, and a combination of measures that included an in-water structure and reservoir drawdown. This appendix does not attempt to assess the alternatives carried forward in the main EIS document but instead assesses the effectiveness of individual measures or combinations of measures.

Navigation Channel Reconfiguration: A non-structural evaluation of the existing navigation channel alignment showed an opportunity for dredging reduction. By reconfiguring the channel to better align with the river's natural thalweg and observed commercial vessel traffic patterns, it is estimated that dredged volumes could be reduced by more than 40%. The dredged volumes were computed from bathymetric surveys. Depositional rates were computed from successive surveys and from dredging records.

In-Water Structures and Reservoir Drawdown: Sediment accumulation is caused by the river slowing as it enters the reservoir pool created by the Lower Granite Dam. Several in-water structure measures were modeled, such as dikes and weirs, that increase river velocity to reduce sediment deposition. The models' effectiveness was validated by comparing results to a reference reach of the river that does not experience problematic sediment deposition.

Modeling determined that structures on the Snake River were likely to be cost-prohibitive. The analysis, therefore, focused on the Clearwater River near the Port of Lewiston.

A full-height longitudinal dike was found to be effective at increasing channel velocity but would increase flood risk during high-flow events by raising the upstream water surface elevation. A levee raise may be required with this measure to maintain flood risk levels.

The most effective structural solution identified is a combination of a submerged longitudinal dike and periodic reservoir drawdowns. The submerged dike does not raise the water surface during high flow events. During drawdown the water velocity and bed shear stress increase notably. Drawdown not only prevents new sediment from settling but also erodes and flushes previously deposited material from the navigation channel. The in-water structure and reservoir drawdown measure could be combined with the navigation channel reconfiguration to achieve the maximum benefit.

1.0 INTRODUCTION

This hydraulic analysis accompanies the Environmental Impact Statement (TEIS) that was tiered from the Programmatic Sediment Management Plan and Final Environmental Impact Statement (PSMP/FEIS). The TEIS evaluates alternatives for a long-term plan to manage, and reduce, if possible, the accumulation of sediment that interferes with the authorized project purpose of commercial navigation over the next 50 years.

1.1 SYSTEM DESCRIPTION

The lower Snake River navigation channel begins at the confluence with the Columbia River near Pasco, Washington, passes four dams, and ends near Lewiston, Idaho. The U.S. Army Corps of Engineers, Walla Walla District (USACE) operates and maintains the Federal navigation channel on the lower Snake and Clearwater Rivers up to the Port of Lewiston. While there are periodic sediment deposition problems at several locations within the system, the main problem area is at the confluence of the Snake and Clearwater Rivers (Confluence), which primarily affects commercial navigation at the ports in Lewiston and Clarkston Figure 1. Downstream from the Lewiston/Clarkston area are four dams on the Snake River that include navigation locks and other aids to navigation. The uppermost dam, Lower Granite Dam (LWG), has the most influence on sediment deposition in the problem area since it is the most upstream dam and creates a lake extending upstream through the Confluence into the Clearwater River.

Lower Granite Dam is located approximately 27 miles northeast of Pomeroy, Washington, and southwest of Pullman, Washington, at river mile (RM) 107.5 on the Snake River. This dam is about 32 miles downstream from the Confluence. The dam straddles both Garfield and Whitman Counties, while Lower Granite Lake extends up the Snake River into Asotin County, Washington, and up the Clearwater River into Nez Perce County, Idaho. The Confluence, within the Lower Granite Reservoir, is located at RM 138, measured from the Columbia River near Pasco, Washington.

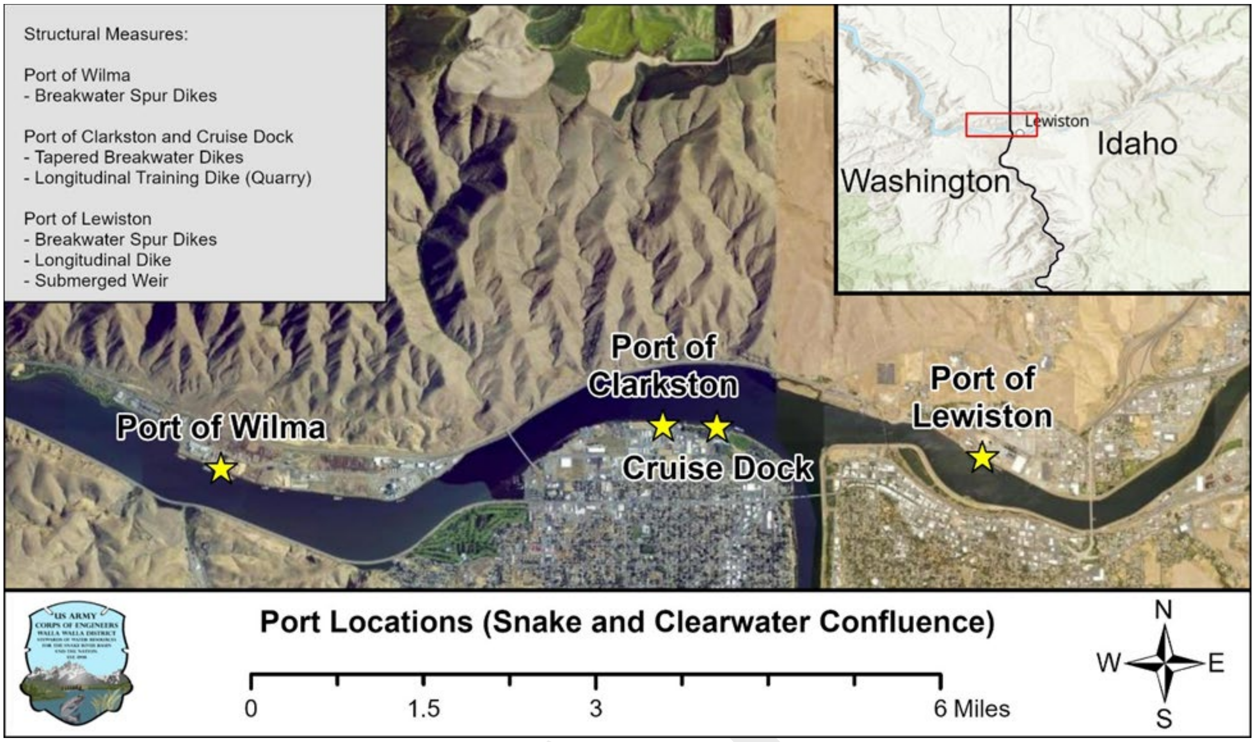


Figure 1. Location of the Ports in the Lewiston/Clarkston Area

All forebay elevations obtained from PSMP Appendix F (Corps 2014)were adjusted by +3.40 feet to shift elevations from NGVD29 to NAVD88. Note that this datum shift is slightly different than the +3.44 feet shift reported by NGS NCAT (National Geodetic Survey 2025) at the dam forebay. Additionally, the river stations used in the modeling for this TEIS are based on the CRSO Reach 9 HEC-RAS 1D model schematic, and the stationing is slightly different from the historical Snake River miles used for the 2014 PSMP.

1.2 HYDRAULICS AND SEDIMENT APPENDIX PURPOSE

The Lower Snake River Federal navigation channel experiences recurring sediment deposition that impacts commercial navigation to the Port of Clarkston and the Port of Lewiston. This analysis aims to identify the most cost-effective technically and environmentally acceptable long-term solution for managing sediment in the navigation channel near the Confluence. Only sediment that interferes with navigation is considered.

Congress has authorized maintaining the Federal navigation channel to a depth of 14 feet at the minimum operating pool (MOP). The current management approach when sediment deposition reduces the channel depth to below 14 feet is to temporarily raise the pool until the channel can be dredged. This interim pool raise can last as long as 3 years while funding is secured, and the required environmental and contract documents are prepared. The objective of this analysis is to reduce the volume, and if possible, the frequency, of sediment dredged. The Programmatic Sediment Management Plan (PSMP) in Appendix A of the Lower Snake River Programmatic Sediment Management Plan Final EIS requires USACE to evaluate the eight navigation-specific measures below when developing a long-term sediment management solution:

i. Dredging
ii. Disposal
iii. Bendway weirs
iv. Dikes/dike fields
v. Sediment traps
vi. Reservoir drawdown to flush sediment
vii. Reconfigure affected (Federal) facilities
viii. Relocate affected (Federal) facilities

To simplify the discussion, this TEIS has grouped them into five measures:

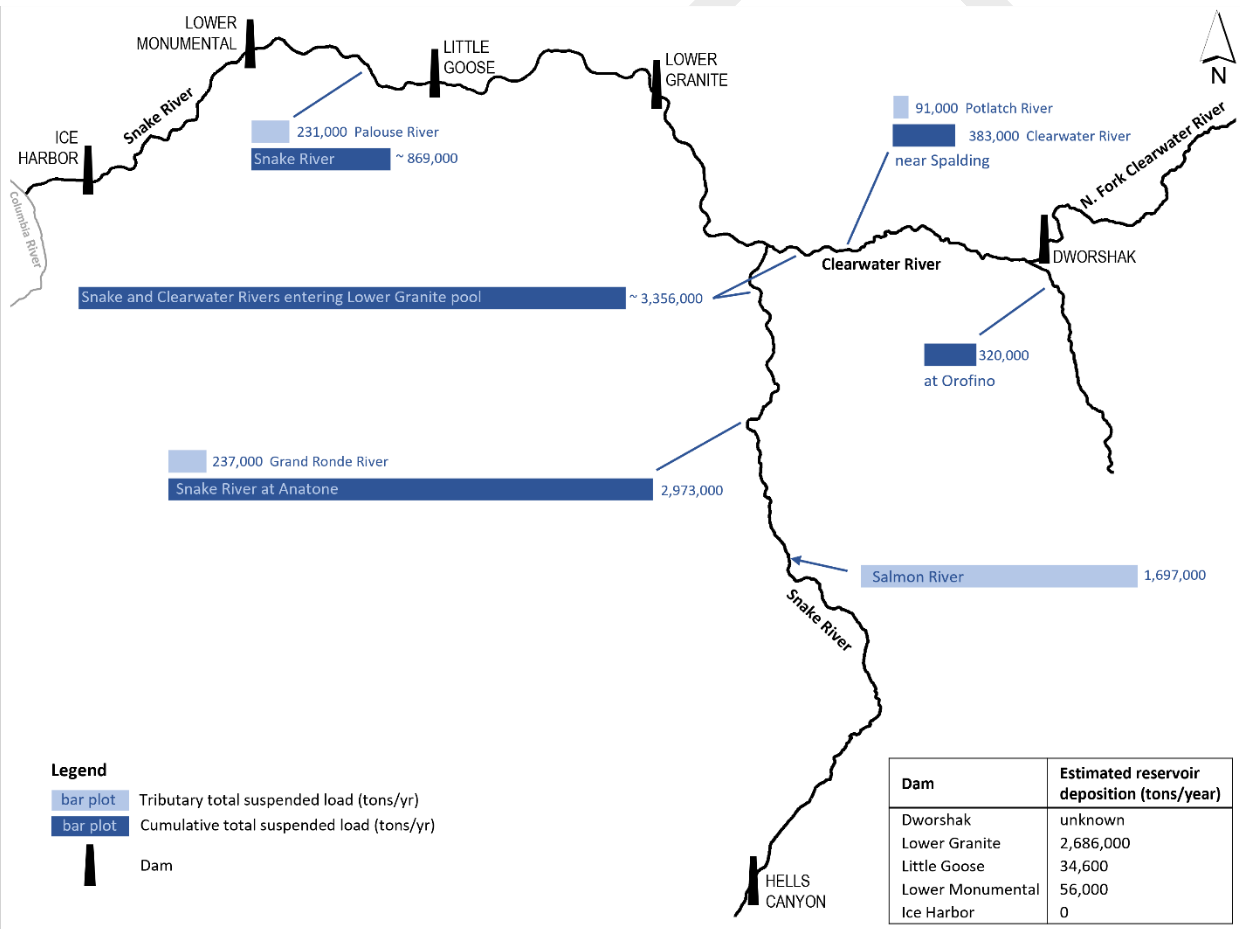
a) Dredging and Disposal (i and ii)
b) In-Water Training Structures (iii and iv)
c) Sediment Traps (v)
d) Reservoir Drawdown to Flush Sediment (vi)
e) Relocate or Reconfigure Affected Federal Facilities (vii and viii)

The final alternatives presented in the TEIS are combinations of the five measures. This appendix focuses on the hydraulic and sediment analysis of the measures, rather than the alternatives. Combinations of measures that when paired result in greater benefit are also discussed. This TEIS only focuses on managing sediment deposition in the federal navigation channel that interferes with navigation, although sediment deposition also reduces flood conveyance capacity.

2.0 SEDIMENTATION

Lower Granite Dam is a run-of-river dam, meaning the volume of water passing through the dam over a short period of time, such as a day, is roughly the same as it would be if the dam were not present. However, due to increased depth, the river velocity is much slower than it would be without the dam. One of the primary purposes of the dam is to force deeper water to provide more clearance for navigation vessels. While the deeper, slower water facilitates navigation, it also promotes sediment deposition. As sediment accumulates on the channel bed, vessel clearance is reduced, which can be mitigated by raising the water surface elevation or by removing the sediment. Raising the water surface elevation is a temporary solution, as it promotes more sediment deposition.

As the faster upstream river approaches Lower Granite Dam, the water gradually gets deeper and moves more slowly, and sediment begins to deposit. Course sediment settles out first near the upstream end of the reservoir, followed by finer sediment moving downstream.



Dam	Estimated reservoir deposition (tons/year)
Dworshak	unknown
Lower Granite	2,686,000
Little Goose	34,600
Lower Monumental	56,000
Ice Harbor	0

Notes: Loads were taken from USGS Scientific Investigations Report 2013–5083 (Clark, Fosness, and Wood 2013, Table 4). Sediment loads marked with a ~ were estimated for the figure by summing loads. Tabular reservoir deposition was estimated from trapping efficiencies and are not from the USGS study.

Figure 2. Mean Annual Total Suspended Load for 2009 to 2011 Estimated by USGS, except as noted

Motivated by the PSMP, the U.S. Geological Survey (USGS) conducted a detailed sediment-load study spanning 2008 to 2011 (Clark and Wood 2013). Measurements from that study are summarized in Figure 2. The study spans a short timeframe and only measures suspended sediment load but illustrates well the sediment challenges. Sediment load and deposition vary notably from year to year, so the magnitudes in that figure are not representative of every year but the system is conceptually similar each year. More deposition tends to occur during years with higher discharge, due to increased sediment load.

Sediment dredged from the Clearwater and Snake River is primarily sand (PSMP 2014). This sand is transported downstream either suspended in the water or rolling, sliding, or bouncing along the channel bottom. A significant portion of the sand entering the confluence of the Snake and Clearwater River is carried in suspension. As the river velocity slows near the confluence, this sand drops out of suspension. The reduction in velocity is due to channel widening, backwater effects from the downstream dam, and the channel becoming less steep. Turbulent motions that keep the sand in suspension are no longer sufficient to support the sand, so it deposits in the channel. However, some of the deposited sand continues to be transported downstream as bed load.

Figure 3 shows an estimate of the maximum grain size that can be held with 100 percent of its transporting mass in suspension. The grain size was estimated from daily average hydraulic conditions extracted from a 5,000-year stochastic model, with one-dimensional hydraulics (CRSO Appendix C 2020). The threshold between 100 percent suspension and partial deposition was estimated using the Rouse number, which is a ratio between a characteristic particle fall velocity and the boundary layer shear velocity. Near the confluence, the suspended sediment becomes finer, implying that the coarser portion of the sediment load drops out of suspension. A significant proportion of the simulated flow range shown in Figure 3 has sufficient capacity to transport sand in suspension upstream of the confluence. Downstream of the confluence the capacity to transport sand in suspension is limited, resulting in finer material being transported. Figure 3 and Table 2-1 show sediment size using the Ψ scale (Psi scale), where the sediment diameter, *D*, in millimeters is computed as $D = 2^{\Psi}$. For example, -2Ψ:MS in Figure 2-2 indicates that medium sand (MS) has a size range of 2^{-2} to 2^{-1} (i.e. 0.25 to 0.5) mm.

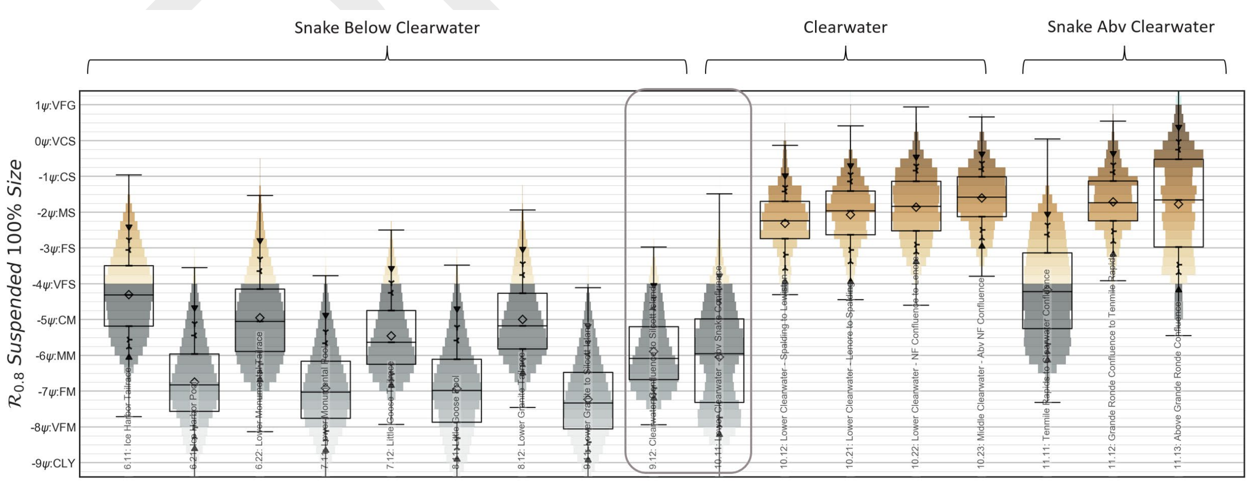


Figure 3. Rouse 100 Percent Suspended Grain-Size Threshold Over a Range of Simulated Daily Flows

Table 2-1. Grain Size Classes

Grain Class	Abbreviation	Ψ	Lower Bound (mm)
Very Fine Gravel	VFG	1	2
Very Coarse Sand	VCS	0	1
Coarse Sand	CS	-1	0.5
Medium Sand	MS	-2	0.25
Fine Sand	FS	-3	0.125
Very Fine Sand	VFS	-4	0.0625
Coarse Silt	CM	-5	0.032
Medium Silt	MM	-6	0.016
Fine Silt	FM	-7	0.008
Very Fine Silt	VFM	-8	0.004
Clay	Clay	-9	0.002

While the Rouse number provides an estimate of the expected maximum particle size capable of passing through a cross section as suspended load, it does not account for localized sediment carrying capacity due to two or three-dimensional flow, and it does not quantify bed load capacity. Figure 4 gives some indication of the bedload carrying capacity. The critical grain size is the largest grain size that is expected to move from the forces of the flow on the bed. This threshold is based on the critical Shields shear stress at the point of initiation of motion, rather than wide-spread movement of sediment, so sediment larger than the critical grain size is not expected to move in notable quantities. This figure does not show what sediment is moving on the bed but instead shows what sediment the flow is capable of moving at the bed. The actual sediment being moved depends on what is being supplied upstream. However, the spatial distribution of sediment can be seen in bathymetric surveys – the bedforms in Figure 5 show where sand has deposited and is being actively transported along the bed at the Confluence. As previously discussed, measurements have shown that sand is present in upstream suspended load. Suspended sand drops out of suspension at the confluence, as demonstrated by the suspended load capacity in Figure 3, and becomes bed load as shown in Figure 4. Modeled velocity and bed shear stress are used in this report to explain localized sediment depositional patterns in proposed measures.

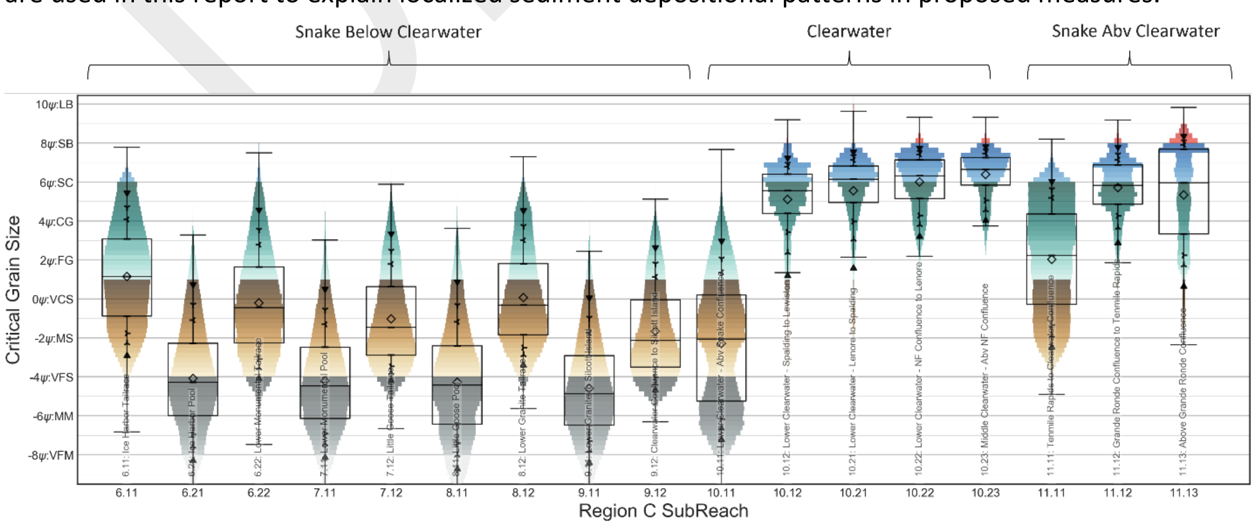


Figure 4. Critical Grain Size for Initiation of Motion at the Bed Over a Range of Simulated Daily Flows

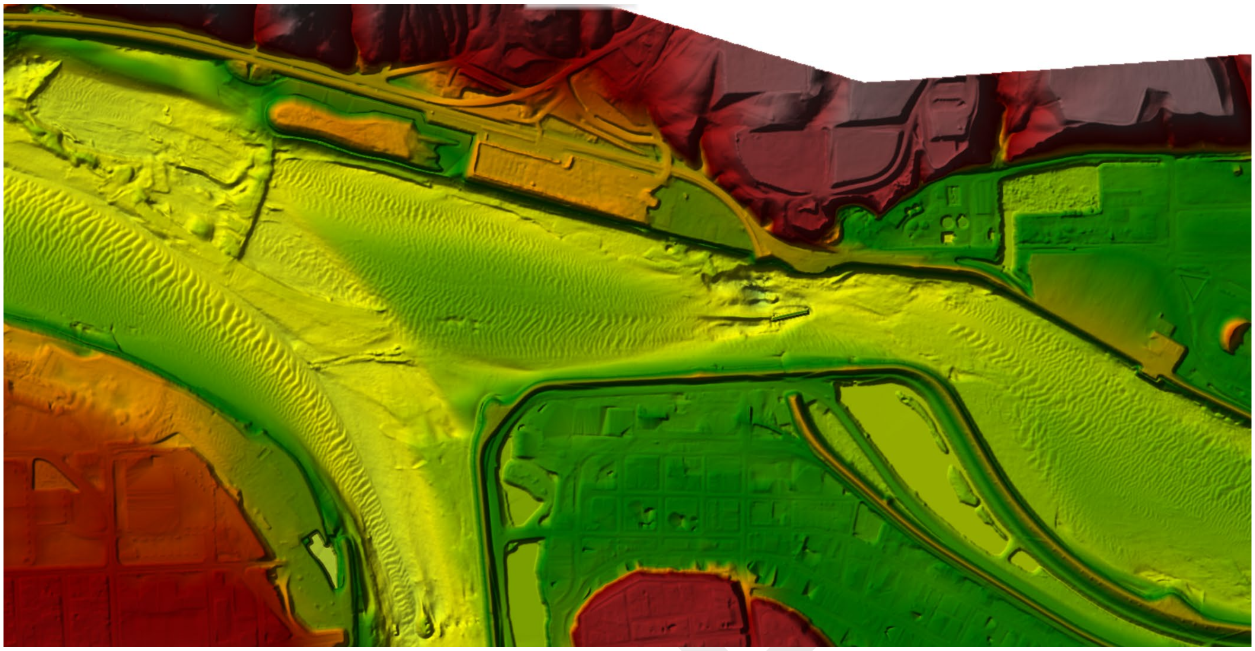


Figure 5. Sand Bedforms in Bathymetric Survey of the Confluence.

2.1 DREDGING

Sediment deposition can be both continuous and episodic in nature and varies considerably with runoff magnitude and intensity. Since streamflow is a function of weather, predicting next year's sediment volume would require a detailed forecast of next year's weather and knowing how and where erosion would initiate, which is not possible with reasonable accuracy. Despite this high level of variability, long-term historical sediment dredging can be used as an indicator of future long-term sediment deposition within the navigation channel and at the ports. Since long-term (50-year), rather than short-term, sediment deposition is being evaluated, future cumulative dredge volume estimates are less variable. In some cases, dredging will be reported as average annual dredged volumes, but this is for convenience in describing magnitudes and generating costs – uniform deposition over time is not expected. Figure 6 shows historical sediment dredging along with estimated sediment load. Cumulative sediment volumes are shown instead of event volumes to help dampen variability and establish a trend. The streamflow sediment loads were computed from relationships developed in the PSMP Appendix F Tables 49, 51, 53, 55, 57, and 59 (2014).

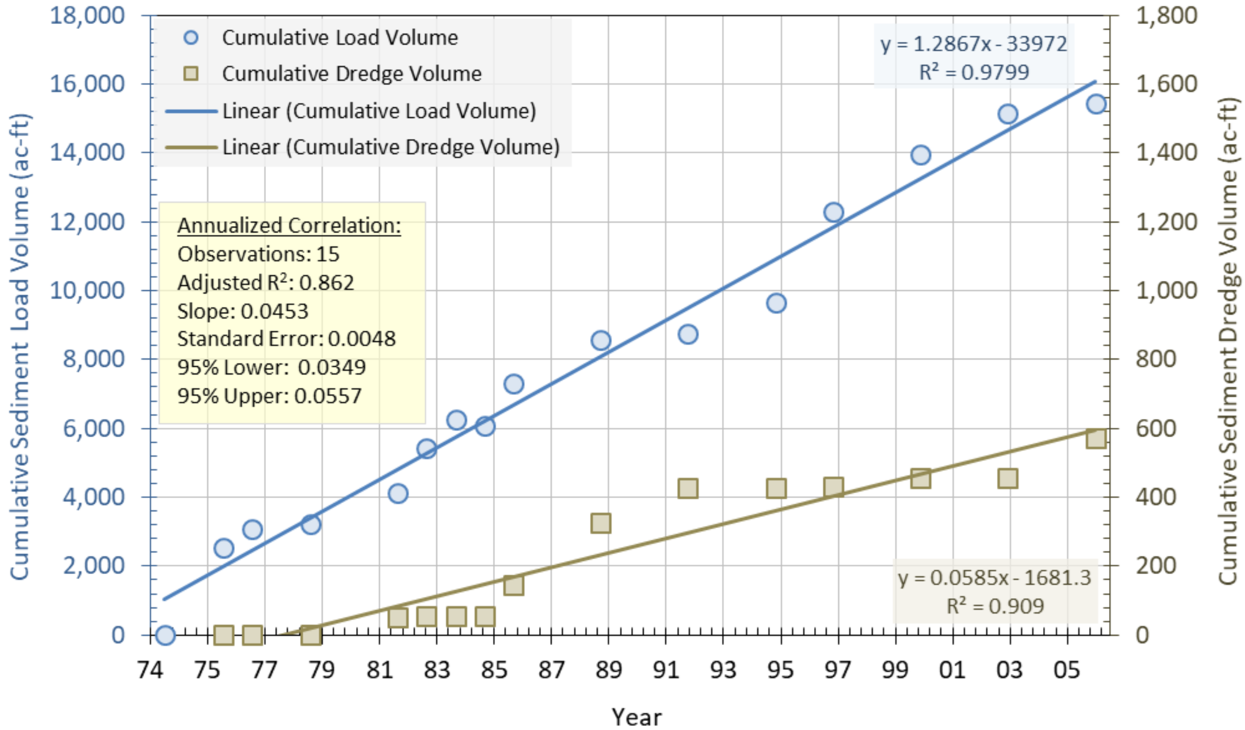


Figure 6. Lower Granite Reservoir Sediment Accumulative Trends 1974-2006

Sediment deposition varies spatially both across the channel and upstream to downstream. Figure 7 shows the difference between two bathymetric surveys (2021 and 2011) at the confluence; positive numbers indicate deposition and negative numbers indicate erosion. Pre- and post-dredging bathymetric surveys do not always span the full channel, as they do in Figure 7, and sediment deposition is typically only computed at locations where dredging occurs. Since dredging extents have historically varied from year to year and the navigation channel is only dredged to the authorized 14-foot depth, dredged quantities differ from depositional volumes. For example, Figure 8 shows the 2023 dredged areas and depths, which differ from the depositional areas in Figure 7 because only the areas in Figure 8 needed to be dredged to maintain the navigation channel and provide access to the docks. The long-term dredging history was used to estimate long-term averages of expected sediment deposition within the navigation channel. Historically, dredging has been performed for navigation, recreation, construction, and to increase channel conveyance capacity for flood risk reduction; conveyance dredging has not been performed since 1992. Table 2-2 shows historical dredged volumes at the confluence; some dredging actions began in December of one calendar year and continued into the following calendar year, but only one year is labeled in the table. Although total dredged volumes are shown in Table 2-2, this study only considers navigation dredging. Tabel A-2 in PSMP Appendix A breaks down historical dredging by dredging purpose, and an updated table with just navigation channel dredging at the confluence is included in the introduction of the report accompanying this appendix. Estimated future average annualized dredging volumes for navigation are shown in Table 2-3. These volumes were estimated from historical dredged volumes. While the volumes in Table 2-3 are annualized, actual dredging will occur on an as-needed basis as previously discussed in section 1.2.

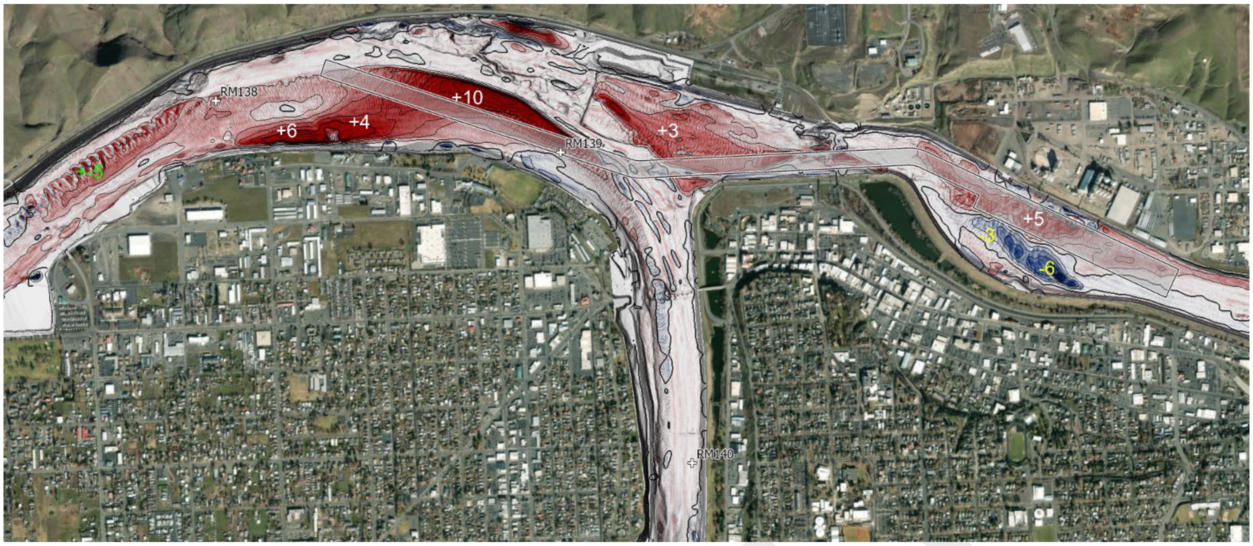


Figure 7. Sediment Deposition Difference Between 2021 and 2011

Figure 8. Dredged Depths in 2023

Table 2-2. Historical Dredging (Payline Cubic Yards).

	1982	1985	1986	1988	1989	1991	1992	1995	1996	1997	1998	2005	2015	2022
CONFLUENCE		771,002		915,970	993,445		520,695		68,701	215,205		311,396	352,622	136,084
Port of Lewiston	256,175		378,000			90,741				3,687		4,583	3,365	2,162
Port of Clarkston	5,000									5,601	12,154	19,896	14,812	79,466

Table 2-3. Estimated Future Annualized Navigation Dredging Based on Current Practices

	Average Annualized Dredging (CY/YR)
Navigation Channel – 2022 Footprint	27,500
Port Access Channels – 2022 Footprint	10,000

2.2 NAVIGATION CHANNEL ALIGNMENT

Sediment deposition within the congressionally authorized 250-foot-wide navigation channel periodically reduces the channel depth to less than the authorized depth of 14 feet (at MOP). The authorized depth is restored by dredging. One approach for reducing the amount of material dredged is to reconfigure the Federal navigation channel. The proposed navigation channel layout was designed by examining existing navigation vessel traffic (Figure 9) and sediment depositional patterns, and consulting navigation alignment standards (EM-1110-2-1611, EM-1110-2-1613). Figure 10 shows the proposed navigation channel along with the existing 2022 (associated with the 2022/2023 dredging activity) navigation channel. The proposed channel aligns better with both the historic pre-dam channel (bottom image in Figure 10) and the current thalweg. Proposed access channels are also shown in that figure.

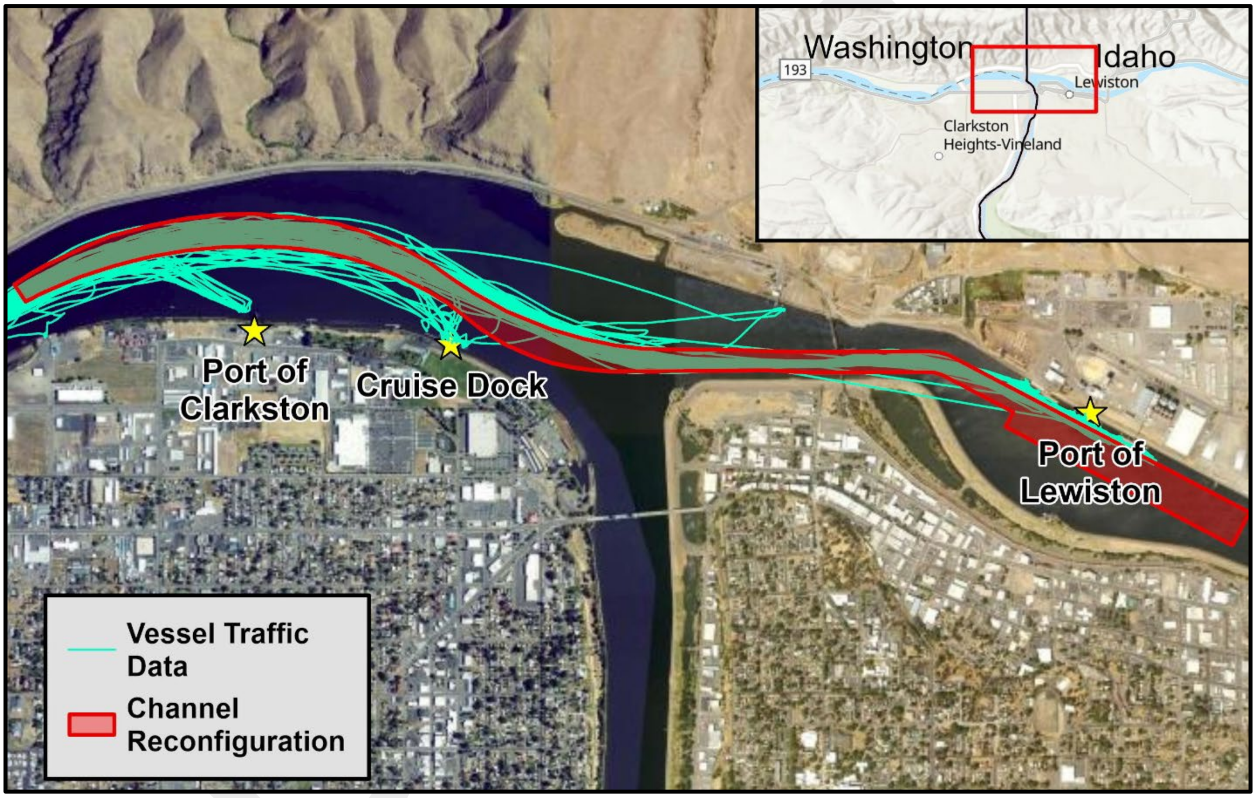


Figure 9. Commercial Vessel Traffic with Navigation Channel Reconfiguration Overlayed

Adopting the proposed alignment would reduce dredging by more than 40%. All reductions in dredging are on the Snake River, as the Clearwater River navigation channel and associated turning area would remain unchanged.

The 250-foot channel width is understood to be measured at the channel bottom along straight channel segments (ER-1130-2-520, EM-1110-2-1611, EM-1110-2-1613). Submerged sand cannot support vertical walls and will naturally settle at a slope, resulting in some sand being dredged beyond the 250-foot width to maintain the channel bottom width. The slope at which sand will naturally settle in water is the submerged angle of repose. However, in a dredging environment with moving water, the slope will be milder than the submerged angle of repose. Previous dredging efforts have assumed side slopes are approximately 2:1 (Horizontal:Vertical). Despite the 2:1 side slopes, access channels are expected to fill in at a faster rate than the navigation channel, as they experience more cross flow and have a readily available sediment supply. Per ER-1130-2-520, a maximum of 2 feet of overdepth (i.e. deeper than 14 ft below MOP for this channel) dredging is permitted.

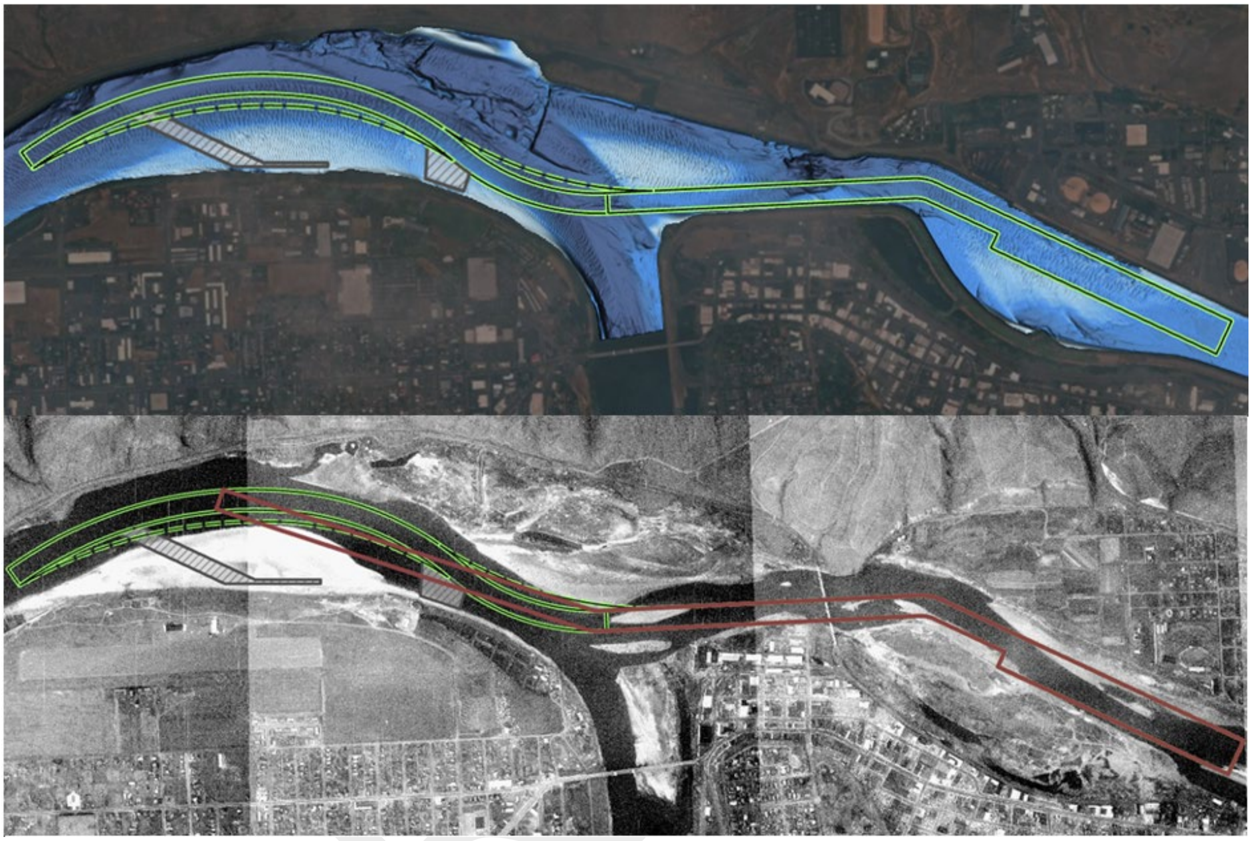


Figure 10. Proposed Navigation Channel. Top: proposed navigation channel with current bathymetry. Bottom: Historical 1956 pre-dam photo with existing (red) and proposed (green) navigation channel.

Additional width at bends (dashed lines in Figure 10) were designed using guidance in EM-1110-2-1613 and EM-1110-2-1611, for deep draft and shallow draft navigation, respectively. Deep draft navigation refers to channel depths greater than 15 feet (per section 1-5 in EM 1110-2-1613). The navigation channel is deeper than 15 feet in many locations but only 14 feet in other locations. The authorized depth is 14 feet, but it can be dredged up to 16 feet deep when including overdepth dredging. Additionally, some vessels navigating the channel have drafts as deep as 13 feet 6 inches, which is likely uncommon in shallow channels, and vessels with that much draft are not addressed in the shallow channel EM. For this reason, both EM's were considered when designing the additional width at bends. The deep draft EM (1110-2-1613) tended to be more conservative for the radius lengths considered.

The design vessel for the 250-foot authorized channel depth is unknown. As a result, the largest vessels currently navigating on the channel were used to design the curves. A vessel length of 650 feet was assumed, with a beam width of 86 feet.

Both EM's recommend straight reaches between alternate bends. However, the deep draft channel allows for s-bends but recommends ship simulation testing to develop appropriate channel alignments and dimensions for these types of bends. The deep draft EM requires straight segments preceding the bends to be at least five times the length of the design ship, whereas the shallow draft EM uses a diagram to determine the minimum length, which resulted in 4.5 times the ship length. The straight segments are desirable because a vessel leaving one bend crosses toward the opposite bank of the next alternate bend.

According to EM 1110-2-1613, alignments consisting of straight reaches with small turns between channel segments are preferred over circular bends because this permits pairs of range markers to be located on the channel centerline. This provides better channel location and ship pilot control than other possible channel aids to navigation. However, the EM also indicates that alignments that cut across sandbars and mud banks should be avoided. The proposed alignment follows the curvature of the sand point bar, satisfying this criterion. The upstream portion of the proposed alignment uses straight channel segments rather than curves.

The ratio of the ship length, L, to the ship beam width, B, for the design vessel is L/B = 650/86 = 7.6. The proposed radius, R, of the curve depicted in Figure 10 is 5,000 feet, so R/L = 5000/650 = 7.7. Applying these values in to Figure 8-3 in EM 1110-2-1613 results in an increase in channel width of one beam width (86 feet), which is the basis for the curve widening (dashed line) in Figure 10.

3.0 ONE-DIMENSIONAL HYDRAULIC MODELING

The purpose of the one-dimensional (1D) hydraulic models is to estimate shear stress, sediment transport, and provide boundary conditions for the two-dimensional (2D) models. The downstream boundary condition of the 2D models was established by routing backwater stage from Lower Granite to the downstream boundary of the 2D models. A range of spillway gate operations at Lower Granite Dam were modeled to simulate drawing down the lake elevation to increase velocity upstream to flush sediment out of the channel. These drawdown simulations provided boundary conditions for 2D models and were used to infer sediment transport volumes. Both mobile bed and fixed bed models were developed.

3.1 FIXED-BED MODEL DEVELOPMENT

An existing 1D unsteady flow, fixed bed, Hydrologic Engineering Center River Analysis System software (HEC-RAS) model previously used for the Columbia River System Operations (CRSO) Environmental Impact Statement (EIS) (2020) was adapted for fixed-bed modeling. Multiple models were developed from this model, some of which were used for routing backwater stage from Lower Granite to the downstream boundary of the 2D models. Another purpose of these models was to analyze potential drawdown operations and impacts.

3.1.1 Geometry

On the Snake River, the fixed-bed models begin upstream at the Anatone USGS gage and extend downstream to Lower Granite Dam; the models with Lower Granite Dam operations extend downstream to Little Goose Dam. On the Clearwater River, the models begin upstream at the Spalding USGS gage and extend downstream to the confluence with the Snake River. The initial geometry file was based on the Iteration 4 reaches 9, 10, and 11 model dated October 2017. The Manning's roughness coefficients upstream of river mile 1.97 on the Clearwater River were modified based on a Corps Water Management System model to achieve a better calibration. The PSMP Manning's roughness coefficients were adopted to improve the calibration on the Snake River within the main channel from the confluence upstream to river mile 147.85, based on Table 26 in Appendix F (2014). Note that the PSMP river miles differ from the river miles in the CRSO model.

The NAVD88 vertical datum of the CRSO models was retained and the model was upgraded to RAS 6.6. The inline structure geometry for Lower Granite Dam was obtained from a 2009 Walla Walla District Corps of Engineers model and the datum was converted from NGVD29 to NAVD88 by adding 3.435, as queried from NCAT (National Geodetic Survey 2025).

Operations at Lower Granite were added to one of the 1D fixed-bed models. The gates are controlled by operational rules based on the current regulations and parameters defined within the Fish Operations Plan (FOP) (Corps 2024). The operational rule set first ensures that the forebay elevation of Lower Granite and the confluence stay between the operating range of the project, 733-738 Feet NGVD29 (736.435 to 741.435 feet NAVD88), as specified in table 2 of the FOP. Then the rules divide the gate operations up by seasonal requirements with three spill seasons: Fall/Winter, Spring, and Summer.

Within each spill season, the gate flows and operational timings are dictated by the guidelines in tables 3, 4, and 5 of the FOP.

To ensure the inline structure and model accurately represented the project's spill patterns and power generation targets, the gate openings and/or flow values were coded from tables within the 2024 Fish Passage Plan (FPP) (Corps 2024). For the Removable Spillway Weir gate, Chapter 9 – Lower Granite Dam section 2.3.2.6 paragraph ii. was used. The other Spillway gate openings came from table LWG-7 in the FPP. For the turbines, tables LWG-6 and LWG-6-A were used, with the flow value being the average between the 1% lower and upper limit flow values. The remaining gate operations are to assist in model stability and to ensure the model inflows reflect the physical project capacity.

3.1.2 Boundary Conditions

Historical gage data or scenario target flows (see 4.4 Boundary Conditions) were used as model inflows at Spalding and Anatone. Measured stages or operational scenario stages stipulated the downstream water surface elevation at either Lower Granite Dam forebay or Little Goose Dam forebay. The NAVD88 vertical datum of the CRSO model was maintained.

3.1.3 Calibration

Three different downstream conditions were modeled: 1) specified downstream stage 2) Lower Granite FPP-based operations, and 3) pulse operations. The original Iteration 4 model had been calibrated but was refined with the Manning's roughness coefficients discussed in 3.1.1. The refinements were verified using gage data at the confluence (LWSI) and the gage 2.83 miles upstream of the confluence (CRLI), combined with flow data from Anatone and Spalding. This is the base calibration. Downstream condition 1) used the base calibration. Conditions 2) and 3) also used this base calibration but the Lower Granite operations also had to be calibrated.

The FPP operations were calibrated by comparing gage data at Lower Granite with model output. The years in Figure 11 where the model deviates from the observed data (2018-2022) result from a change in operations at Lower Granite. The forebay was operated at a higher stage during these years to compensate for sediment deposition at the Confluence. The model results match the historical data again in 2023 after the channel was dredged.

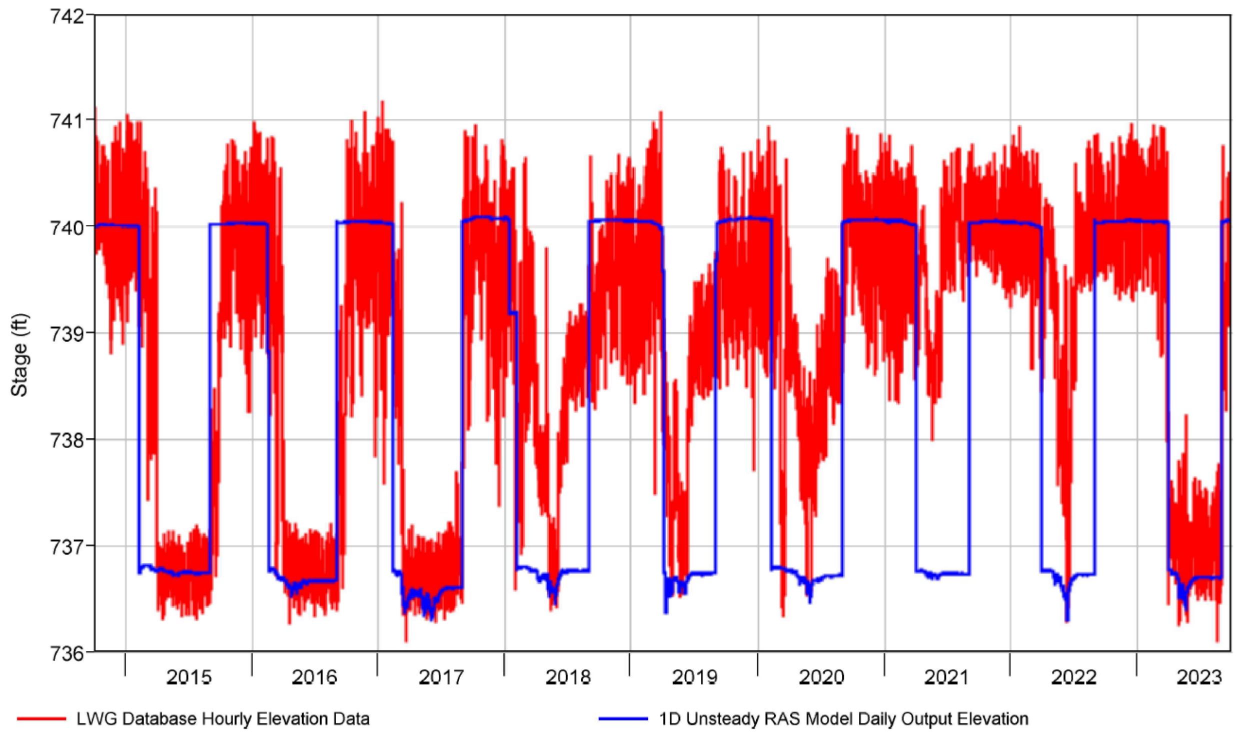


Figure 11. Measured Hourly Forebay Elevations at LWG Compared to Simulated Daily Elevations.

The pulse operations (condition 3) consisted of raising and lowering the pool cyclically. Additional calibration was not performed for this condition, but creating this operation was an iterative process. The target criteria for this operation were: a maximum stage of 741.435 feet NAVD88 at the Confluence to prevent levee overtopping, a specified minimum pool elevation such as 10 ft below minimum operating pool (-10 MOP), and a pool elevation rate of change of 2 feet per day. The drop rate of 2 feet per day was needed for riverbank stability and the fill rate of about 2 feet per day was to prevent waves from surging upstream. To achieve this, spillway gate patterns were extrapolated from the FPP and the 1987 Lower Granite Dam Water Control Manual.

3.2 MOBILE-BED MODEL DEVELOPMENT

A 1D unsteady flow HEC-RAS mobile bed model was developed by the Corps as part of the CRSO EIS dam embankment removal modeling (2020) . That model was adopted and modified to achieve study goals. The original model was an HEC-RAS 5.0.6 quasi-unsteady flow model, which was converted to an unsteady flow HEC-RAS 6.6 model for this study.

3.2.1 Geometry

The extents of this model were modified from the original CRSO model by truncating the downstream limits on the Snake River to the forebay of Lower Granite Dam. The model's spatial domain encompasses the lower ten miles of the Clearwater River upstream of its confluence with the Snake River. The Snake River component of the model is segmented into two distinct reaches: the upper Snake River reach, which extends approximately 9.5 miles upstream of the confluence, and the lower Snake River reach, which continues downstream to the Lower Granite Dam forebay.

The CRSO geometry was based on 1997 data, with bathymetry derived from range line surveys conducted between 1997 and 1999. The surveys were supplemented with light detection and ranging (lidar) data from 2010 to extend the geometry to the valley walls. The final CRSO model includes a combination of surveyed and interpolated cross-sections, with interpolated sections added to areas with sparse survey data. The cross-section spacing averages 2-3 times the bank width.

Although bridges are included in the fixed bed models, they were excluded from the mobile bed HEC-RAS model (and the original CRSO model) to avoid unnecessary complexity and potential compromise of the mobile bed simulation. The bridge low-chord elevations do not significantly impact non-extreme flood flows, and their piers occupy a small portion of the channel's conveyance area.

3.2.2 Boundary Conditions

The boundary conditions applied to the modeling domain were based on data from WY2016-2020, comprised of reported gage data for upstream inflows and observed water surface elevations at the Lower Granite forebay, which served as the downstream boundary under normal operational conditions. Water surface elevation outputs from the fixed-bed model runs at Lower Granite were used as downstream boundary conditions in operational scenarios. The NAVD88 vertical datum of the CRSO model was maintained.

3.2.3 Sediment Data

Sediment data was retained from the CRSO model. The CRSO bed material gradation data was based on sediment samples taken approximately every 2.0 river miles upstream of Lower Granite (2020).

The inflow sediment used in the CRSO model for the Snake River was calculated using the equation below, which is based on a 2013 USGS report (Clark and Wood 2013) that was conducted for the PSMP (2014):

$$\boldsymbol{Q_s = 1.06 * 2.17 \times 10^{-10} {Q_w}^{2.85}} \qquad (1)$$

The inflow sediment for the Clearwater River was based on the same report:

$$\boldsymbol{Q_s = 1.23 * 8.17 \times 10^{-7} {Q_w}^{2.08}} \qquad (2)$$

The final calculated inflow Sediment Load Rating Curves used in the model for upstream boundaries of the two reaches are shown in Table 4 and Table 5.

Table 4. Snake River Sediment Load Rating Curve (RS 147.8569)

Flow (cfs)	10000	50000	100000	200000
Total Load (tons/day)	75	7318	52766	380444
Clay	20	5	2	2
VFM	10	5	2	2
FM	26	5	2	2
MM	27	45	35	35
CM	11	22	26	26
VFS	4	8	15	15
FS	2	6	10	10
MS	1	3	5	5
CS		0.3	1	1
VCS		0.7	1	1
VFG			0.5	0.5
FG			0.5	0.5

Table 5. Clearwater River Sediment Load Rating Curve (RS 7.8160)

Flow (cfs)	5000	25000	50000	130000
Total Load (tons/day)	64	1822	7701	56198
Clay	30	5	2	2
VFM	10	5	2	2
FM	35	5	2	2
MM	13	45	2	2
CM	7	17	35	35
VFS	3	9	18	18
FS	2	6	18	18
MS	1	5	13	13
CS		2	5	5
VCS		1	2	2
VFG			1	1
FG			0.5	0.5

3.2.4 Calibration

Both the hydraulic and sediment calibrations were retained from the CRSO model. The hydraulics were calibrated using historical gage data from 1985 through 2008. The sediment model was calibrated for both erosion and deposition. The depositional calibration was based on long-term historical deposition, and the erosion calibration was based on the 1992 drawdown. Details of the calibration are available in the CRSO report (2020). To avoid introducing errors in the calibration, only model parameters with both high uncertainty and high sensitivity were calibrated. Table 6, taken from the CRSO report, shows the data, algorithms, and parameters in the sediment transport model. Only parameters under the "Deposition Calibration" and "Erosion Calibration" subheadings were modified during the calibration process; all other parameters were fixed to best estimates.

Table 6. Relative Uncertainty and Sensitivity Ranking of the Data, Algorithms, and Parameters in the CRSO Sediment Transport Model (2020)

Uncertainty	High Sensitivity	Lower Sensitivity
High Uncertainty	<u>Deposition Calibration:</u> Load Gradation: Fine Distribution <u>Erosion Calibration:</u> Erodible Depth: Clearwater Krone Partheniades Parameters	Tributary Flow Operations Non-Cohesive Transport Equation: Larsen Copeland
Lower Uncertainty	Dredging Roughness/Ineffective Flow Bed gradation Flow: Snake and Clearwater Erodible Depth: Snake Mixing Method: Copeland	Temperature Fall Velocity: Report 12

4.0 TWO-DIMENSIONAL HYDRAULIC MODEL DEVELOPMENT

2D hydraulic models were created using HEC-RAS versions 6.5 and 6.6 to evaluate different combinations of measures to manage sediment deposition near the Snake River and Clearwater River confluence. Measures include dredging and disposal, various in-water structures, sediment traps, Lower Granite reservoir drawdown to flush sediment, and reconfiguration of the Federal navigation channel. Measures were combined and categorized into these groups and the most feasible option from each group was identified to be carried forward in the process. Two-dimensional modeling was primarily used to evaluate hydraulic structures and measures that can be combined with hydraulic structures (e.g. drawdown).

The 2D models provide velocity patterns and magnitudes that aid in determining whether a structure might be effective. Detailed 2D sediment modeling was unnecessary because the structures effectiveness could be deduced from the simulated velocity fields and shear stress. Less effective structures were eliminated, and sediment deposition was only estimated for the most effective structures.

4.1 STRUCTURAL MEASURES

Various in-water structure types and configurations were evaluated to determine if sediment depositional patterns in the vicinity of the confluence could be altered. These structures were designed to either promote or reduce sediment deposition. Structures that promote deposition were either designed to deposit sediment in more convenient locations for dredging or to promote narrowing or reshaping of the channel. Forcing sediment deposition in one region of the channel can, over time, constrict the flow and reduce deposition in another region. Structures designed to reduce deposition manage sediment by sending it downstream where it does not interfere with navigation. When sediment deposition is reduced, less dredging is required in the targeted area, but more sediment deposits downstream.

The river's ability to transport sediment in suspension is a function of seemingly random chaotic motions (turbulence) in the flow that are continually moving fluid and sediment across velocity gradients. This movement creates internal fluid resistance, which is usually thought of as shear stress. Similarly, sediment transported along the channel bed can be characterized in terms of bed shear stress. Because the velocity in the river is related to the shear stress, looking at the velocity can provide some indication of transport capacity. If two river segments have similar channel dimensions, transport similar sand sizes, and have similar velocity magnitudes, the sediment transport capacity is likely similar. If a river reach can be identified where sediment is not depositing, then that reach can be used to draw conclusions about what velocity might be required to prevent deposition in a similar reach. In-water structures can be designed to create these conditions in the reach that had previously been depositional. This rudimentary approach was used to evaluate numerous conceptual designs of in-water structures.

Using a 1D model was not possible because they do not simulate horizontal variations in velocity but instead compute a single average velocity per cross section. As a result, 2D HEC-RAS models were created for each structure considered. The velocity patterns and magnitudes were used to determine if a particular structure might be effective at preventing sediment deposition. The structure was then modified to improve its effectiveness or to reduce costs. Velocity and shear stress magnitudes were compared to upstream reference reaches where sediment deposition is not problematic to determine if the structure would be successful.

One way to increase the velocity in the channel is to constrict the flow. Unfortunately, constricting the flow often forces a higher water surface elevation upstream. When the water surface rises, the risk of overtopping the banks or levees increases. To avoid increased flood risk, some of the structures were designed to minimize water surface rise. These structures are generally submerged and therefore unable to increase the velocity enough during high flows to reduce deposition substantially, because water flows unconstricted over the structures, gaining access to the full channel width. Some of these structures were not designed to prevent sand from depositing but instead were designed to be self-cleaning when coupled with a water surface elevation drawdown. To achieve this, the water level at the dam needs to be drawn down periodically to increase the velocity in the channel. As the velocity increases, the sand mobilizes and is washed downstream.

To estimate the reservoir drawdown needed to mobilize the sediment without the aid of structures, the 1D HEC-RAS sediment transport models were used. By comparing results from the 1D model with the 2D model, it was possible to infer approximately how much the water surface might need to be drawn down with in-water structures present. The velocity required to remobilize sediment and wash it downstream is greater than the velocity needed to maintain it in suspension.

Two main types of in-water training structures were evaluated: bendway weirs and dikes. Sediment traps located in the Snake River channel were also evaluated.

Bendway weirs are rock sills located on the outside of a river bend that are angled upstream into the flow. With the weirs angled, flow is directed away from the outer bank of the bend and toward the point bar or inner part of the bend. Since this redirection of flow occurs at all river stages higher than the weir crest, bendway weirs are generally designed to be submerged. Where there is sufficient velocity and volume, the redirection of flow generally results in a widening of the channel through scour of the point bar. Examples of weir measures that were evaluated are shown in Figure 12. The terrain shown does not reflect future scoured conditions but reflects current post-dredge conditions.

Dikes are structures used to maintain navigation channels through variations of channel depth and dike alignment. Dikes constrict low and intermediate flows, causing the channel velocity to increase within the reach, thereby reducing deposition or scouring a deeper channel. Dikes are typically built of rock but could also be constructed using other materials. Examples of longitudinal dike measures evaluated are shown in Figure 13. Dikes can also project from the banks into the flow like bendway weirs. However, dikes are taller than bendway weirs and oriented in a different direction because they constrict or deflect flow. Examples of dike measures protruding into the flow that were evaluated are shown in Figure 14.

Additional structures evaluated, including localized structures at ports, are conceptually depicted in the alternative development section of the main report.

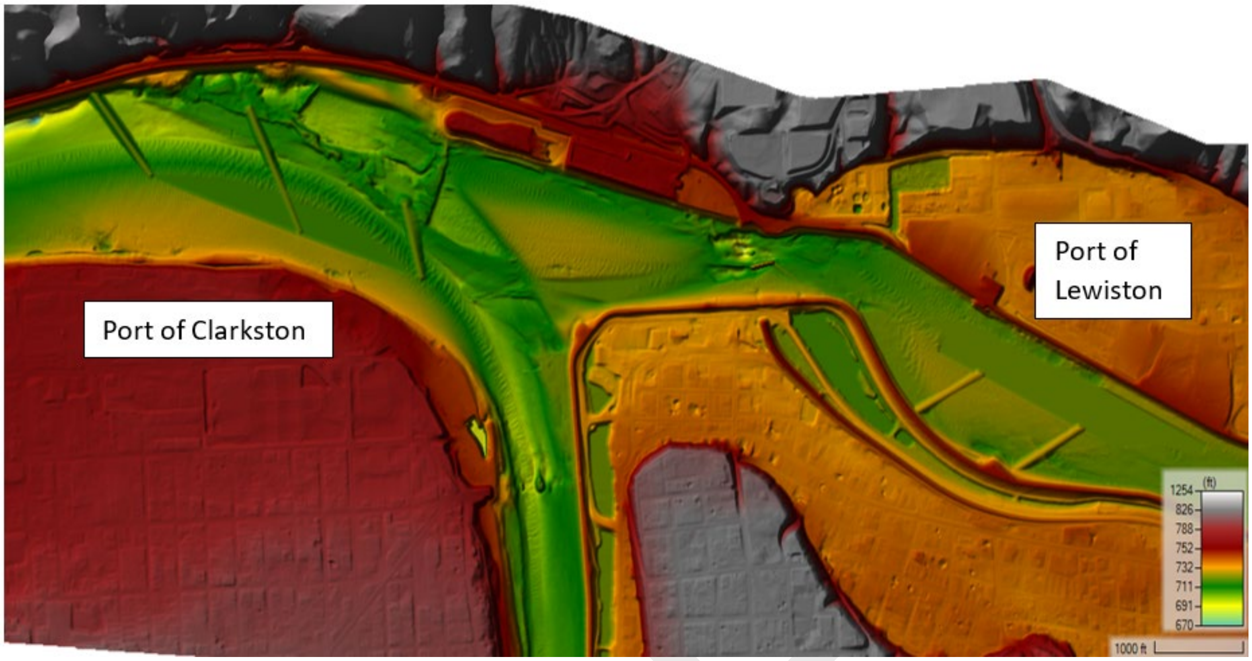


Figure 12. Example of Bendway Weirs Modeled at The Port of Clarkston and the Port of Lewiston

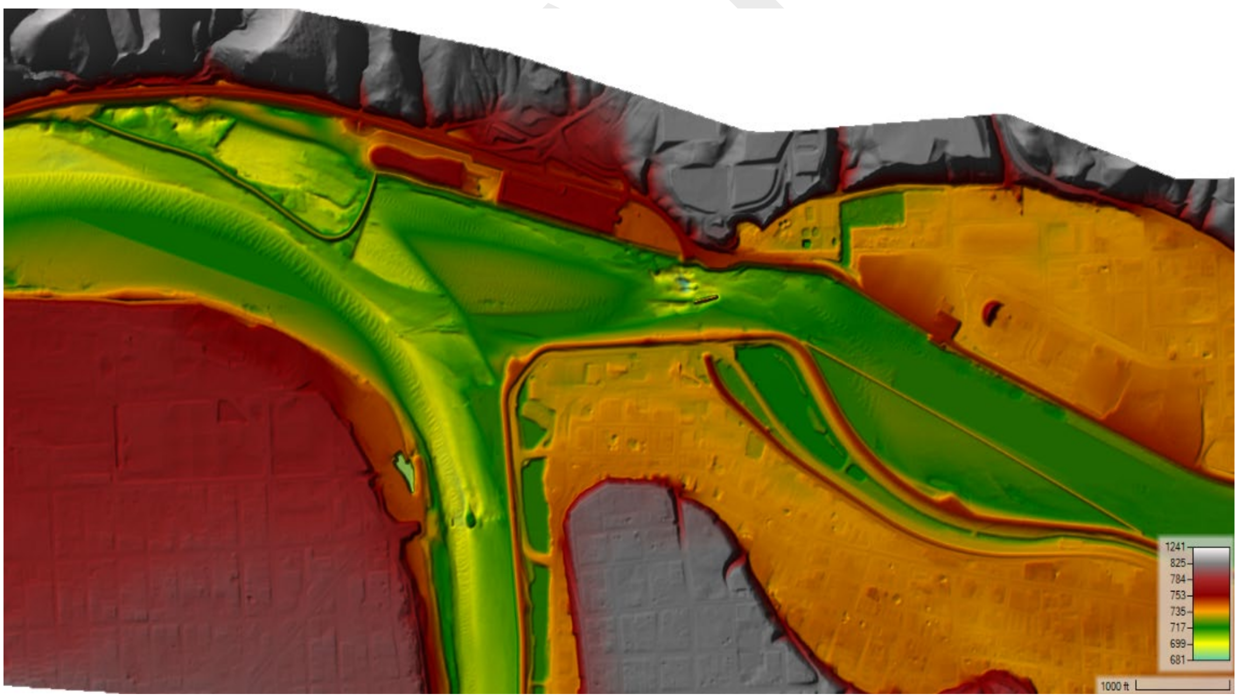


Figure 13. Example of Longitudinal Dikes Modeled at The Port of Clarkston and the Port of Lewiston

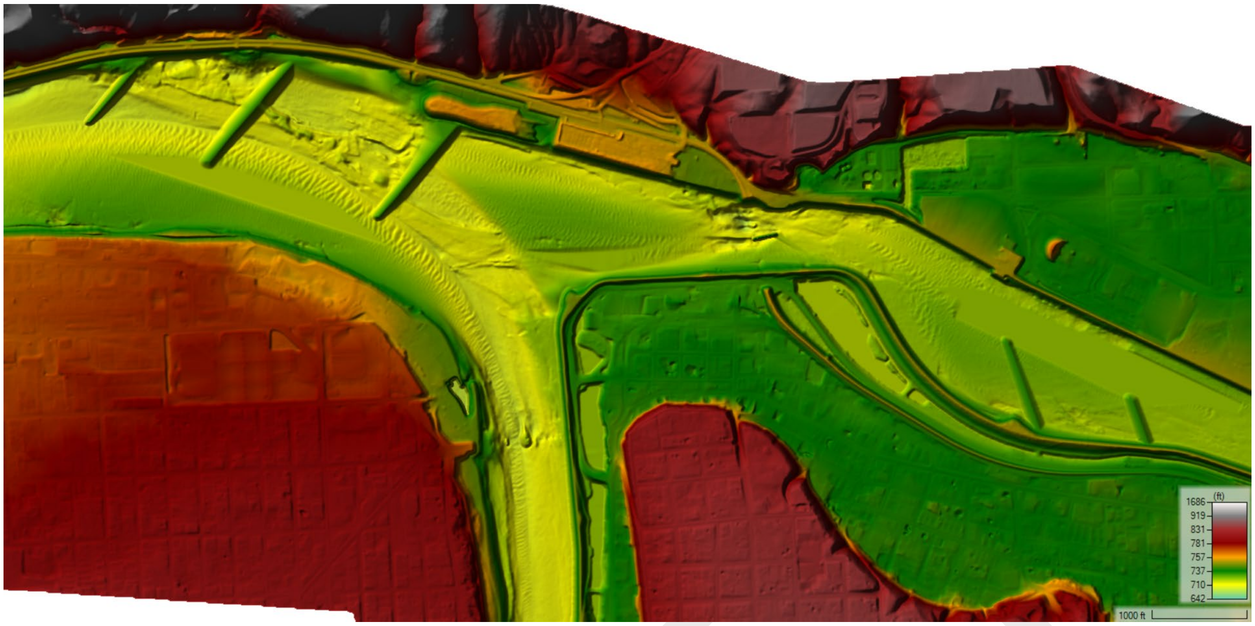


Figure 14. Example of Dikes Modeled at the Port of Clarkston and the Port of Lewiston

4.2 MODEL DOMAIN

The 2D models extend from Station 127.544 on the Snake River upstream to Station 142.56 on the Snake River and Station 3.77 on the Clearwater River (Figure 15). Initial modeling of measures was done using HEC-RAS 6.5. In order to reduce model runtime while still providing a sufficient level of detail, remaining measures were meshed in RAS 2025 Alpha v0.1.0.1000 and run in HEC-RAS 6.6. The mesh development is discussed below. The majority of the measures were modeled with HEC-RAS 6.6. The projection used for the 2D models is NAD83(2011) Washington South (ftUS), EPSG 6599.

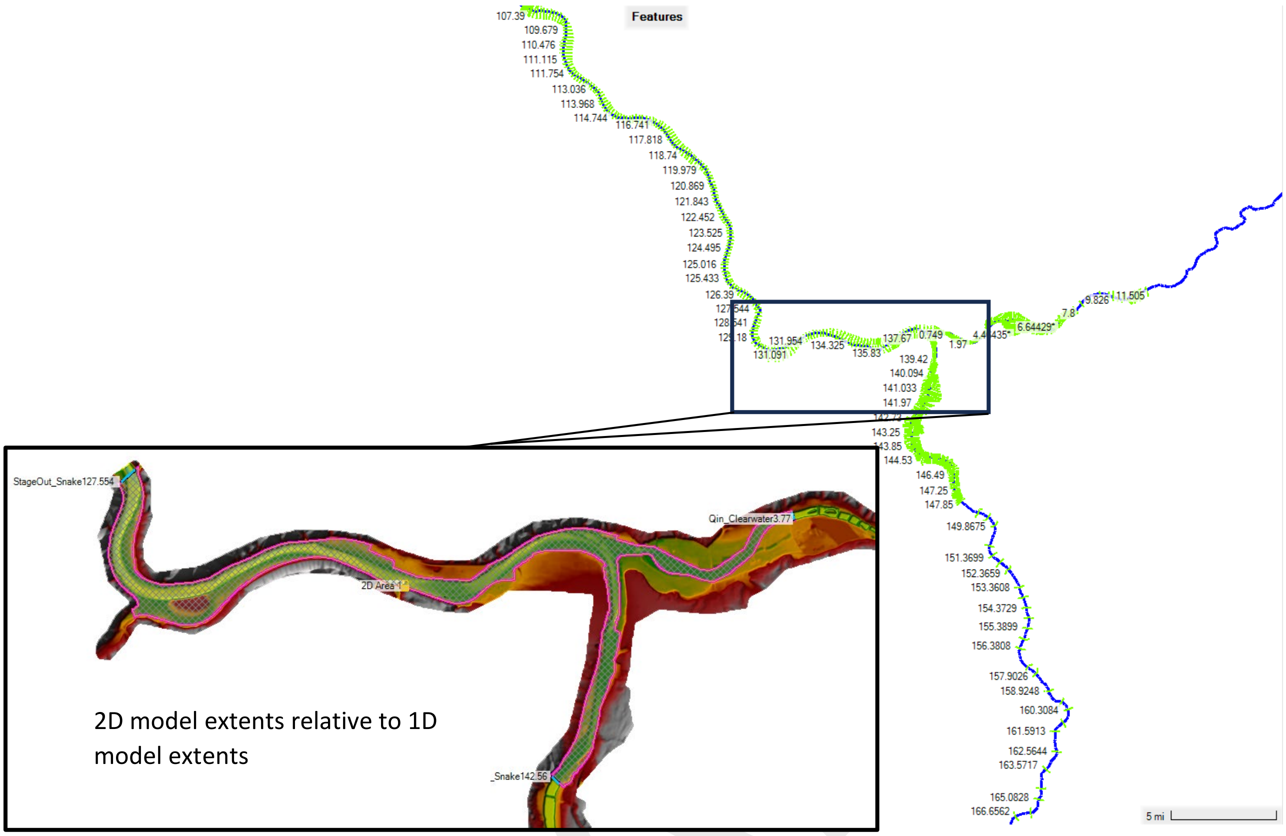


Figure 15. 2D HEC-RAS model extents shown in comparison with the 1-D model extents.

4.2.1 <u>Terrain Data</u>

The baseline conditions terrain was developed as a composite of overbank lidar, main stem channel bathymetry and select local surveys of boat basins and other shallow areas as available.

The in-water structures were represented in the models by either grading the structures into the DEM raster using Autodesk Civil 3D software, or by adding a terrain modification layer with a shapefile that contains elevation data created in Autodesk Civil 3D.

4.2.2 <u>Mesh Development</u>

The meshes used for the models created in HEC-RAS 6.5 and earlier have a nominal cell size of 25 feet. Refinement regions around the in-water structures were created using break lines with smaller cell sizes. The meshes used for the models created in HEC-RAS 6.6 were developed using RAS2025 and are composed of larger quadrilateral cells in the main channel and triangular cells around areas that have more lateral flow. Cell sizes for the meshes created using RAS2025 vary, with sizes ranging from 300 feet in some portions of the main channel to 10 feet around some structures. A base condition mesh was created with subregions around the ports to maintain continuity across measures, allowing for a more direct comparison between existing conditions and measures. The base mesh was duplicated for each measure and individual subregions were created and modified as needed.

4.3 MODEL PARAMETERS

4.3.1 2D Flow Options

The Shallow Water Equations, Eulerian Method (SWE-EM), were selected as the equation set for the models. The Diffusion Wave equation (DWE) set would not have been appropriate for determining velocities and water surface elevations around structures where the flow is not expected to follow the boundary of the structure. The DWE can erroneously result in flow that follows the boundary of the structure because the inertial terms included in the SWE are neglected in the DWE. Many of the structures were designed to force the flow to separate from the structure, resulting in two distinct flow regions that are separated by a shearing region with a high velocity gradient (i.e. local acceleration) associated with inertial exchange and high turbulence intensity. In many cases, one of these flow regions is a large recirculating flow area commonly referred to as an eddy. In early modeling efforts a turbulence mixing method was needed to achieve realistic flow separation and recirculation regions. This was an indication that the model was sensitive to mesh structure and/or time step since large-scale eddies would not be expected to be highly sensitive to small-scale turbulence mixing. But mesh structure and time step can influence computations of inertial forces. A range of timesteps, mesh sizes, and mesh configurations were tested to ensure the final meshes were not sensitive to mesh structure or time step. After a stable configuration and set of parameters were achieved, a turbulence mixing model was not needed as numerical diffusion was sufficient to achieve the desired mixing and inertial forces were now modeled more correctly. It should be noted that the SWE equations result in depth-average 2D flows, so any three-dimensional (3D) motions induced by structures had to be inferred conceptually.

4.3.2 Manning's Roughness

The Manning's *n* values assigned in most of the HEC-RAS models were varied by water depth. Depth-varying Manning's *n* is not a function built into HEC-RAS 6.6, but there are manual methods to achieve Manning's *n* based on depth. The 50% AEP with nominal stage boundary conditions were run using a constant Manning's n from the baseline calibration. The resulting depth raster was processed in ArcGIS Pro 3.3 to obtain a shapefile with polygons classified by depth. The shapefile was imported to RAS Mapper as a pseudo land cover and Manning's *n* values were assigned according to depth intervals shown in Table 7. Although, several models during initial modeling of baseline conditions, without any measures in place, used a constant Manning's n of 0.03 throughout the channel to simplify the model.

Varying n values by flow could not be used to approximate a variation in n values by depth as the stage is controlled by the downstream dam, independent of flow. For the range of flows considered in this study, during normal operations the stage remains constant or decreases as flow increases. When the stage is constant across flow rates, the n values can be thought of as depth-varying. But when the stage or flow changes, the n variation with depth is more appropriately thought of as a spatial variation rather than a flow or depth variation. The n values are fixed before a simulation and cannot change dynamically during the simulation using this method.

Table 7. Assigned Manning's n Values Relative to 50% AEP Depth

50% AEP Depth (ft)	Manning's *n*
0-2	0.100
2-4	0.080
4-6	0.060
6-8	0.050
8-10	0.040
10-15	0.035
15-20	0.030
20-25	0.028
25-30	0.026
30-40	0.024
40-50	0.023
>50	0.022

4.4 BOUNDARY CONDITIONS

There are three boundary conditions in each of the 2D models: a downstream stage boundary condition and two upstream flow boundary conditions. At the downstream boundary condition (Station 127.544), two different stage scenarios were modeled depending on the measure. The first was the nominal stage scenario, and the second stage scenario modeled was a reservoir drawdown measure. When historical water years were modeled, the historical stage was used instead.

For the nominal stage scenario, hydrologic loading for the Snake and Clearwater rivers utilized the estimated annual peak steady flow discharges developed in the PSMP Appendix F Table 29 (Corps 2014) and the corresponding Lower Granite (LWG) stage. The backwater was routed upstream from the Lower Granite Dam forebay to RS 127.544 using the regional 1D model (discussed in section 3.1) to obtain the nominal stage at the 2D model downstream boundary. Table 8 shows the peak inflows and corresponding staging.

To simulate a reservoir drawdown at Lower Granite, the forebay stage was dropped in the regional 1D model and routed upstream to RS 127.544. The output of this routing was used as the input for the 2D model downstream boundary condition for measures that included stage-dropping.

The Minimum Operating Pool (MOP) at the Lower Granite Dam forebay is 736.435 feet (733 feet NGVD29). From April 3 to August 31 the Corps operates the dam within the "MOP range" to satisfy fish passage criteria, unless greater depths are required to maintain navigation safety (FPP 2024). The "MOP range" is defined as MOP to MOP + 1.5 feet. For flood risk reduction operations, Lower Granite Dam uses a hinge pool operation, meaning that the target pool elevation is at an upstream location (the Confluence) rather than at the forebay. To maintain levee freeboard at the confluence, the maximum water surface elevation allowed at the Confluence gage is 741.435 feet (738 feet NGVD29). At low flow both the forebay and the Confluence can be operated near the MOP elevation, i.e. the pool water

surface elevation is nearly level. As flow increases the water surface slope steepens and it is no longer possible to satisfy both the forebay MOP range and the maximum pool elevation at the Confluence. At flows greater than this the pool is operated to maintain levee freeboard at the Confluence.
The reservoir can be operated above MOP to maintain an authorized depth of 14 feet in the navigation channel. However, when flows exceed about 200,000 cfs large vessels no longer navigate the Snake River and the 14 feet of depth no longer needs to be maintained.

Table 8. Estimated Annual Peak Steady Flow Discharges

Exceedance Probability (%)	Snake Peak Inflow (cfs)	Clearwater Peak Inflow (cfs)	Snake Outflow (cfs)	LWG Forebay Stage (ft NAVD 88)	LWG Forebay Δ MOP	RS 127.544 Stage (ft NAVD88)
50	110,991	49,433	160,423	733.78	-2.65	734.13
20	148,115	65,967	214,082	732.00	-4.43	732.68
10	170,189	75,799	245,988	730.66	-5.77	731.63
4	195,65	87,138	282,788	729.32	-7.11	730.69
2	213,089	94,905	307,995	728.40	-8.03	730.11
1	229,394	102,167	331,562	728.21	-8.22	730.20
0.4	249,505	111,124	360,629	727.98	-8.45	730.34
0.2	264,016	117,587	381,603	727.81	-8.63	730.46
0.1	277,83	123,739	401,569	727.60	-8.83	730.55
SPF	295,000	125,000	420,000	727.40	-9.03	730.65

Notes:
Source: Flow and forebay elevations are from PSMP Appendix F Table 29
Full Pool = 741.4 (738 feet NGVD29)
Min Operating Pool (MOP) = 736.4 (733 feet NGVD29)
Δ MOP = Forebay Stage - MOP

5.0 MODELING RESULTS DISCUSSION

Various combinations of in-water training structures were modeled to determine their ability to reduce problematic sediment deposition. In-water structural measures alone are unlikely to prevent all problematic sediment from depositing in the Federal navigation channel – some dredging will still be required.

Reducing deposition just downstream of the Confluence on the Snake River tended to be less effective and require more costly structures than were required on the Clearwater River near the Port of Lewiston. This was due to more complex flow patterns at the Confluence on the Snake River, and the ports being located on an inside bend of a wider river. As structures on the Snake River were too expensive to be feasible, this discussion focuses on the Clearwater River near the Port of Lewiston where structures could be more efficient and more cost effective.

The Clearwater River widens at the Port of Lewiston, which reduces channel velocity and promotes sediment deposition. Narrowing that portion of the channel would increase the velocity and reduce deposition. This would require a ¾ mile long dike. Some simulated dike configurations changed flow patterns downstream near the railroad bridge, potentially increasing local pier scour. 2D hydraulic modeling was conducted to optimize this structure's overall linear length, angle, and height to reduce cost and the likelihood of increased scour at the railroad bridge. The structure, depicted in Figure 16, starts above the water surface upstream and slopes downward in the downstream direction such that the downstream end is submerged during very high-flow events. This configuration has the potential to prevent most of the deposition in the navigation channel in front of the Port of Lewiston. However, sediment that would have deposited in the navigation channel near the Port of Lewiston would be transported downstream. Some of that sediment might deposit within the navigation channel and would need to be managed to prevent impacts to navigation and increases in flood risk. Sediment would also accumulate over time in the low-velocity region south of the structure, which would impact the boat dock but likely would not reduce conveyance.

During very high flows, near the capacity of the levee system, the water surface upstream of the structure would rise almost half a foot compared to the without-structure modeling, resulting in an increase in flood risk. The increased risk, due to loss of channel capacity, may need to be mitigated by raising the Lewiston Levee system.

The sand bedforms in Figure 16 coincide with where sediment has deposited in the channel (see also Figure 7). Note the absence of bedforms in the narrower channel just upstream. This upstream channel segment was used as a reference reach. The assumption is that if velocity and shear stress could be increased to match the upstream reach characteristics, sediment would not deposit in front of the Port of Lewiston.

Velocity simulations shown in Figure 17 show notable increases when compared to the existing conditions in Figure 18. The dike increases velocity in front of the Port of Lewiston to magnitudes similar to those upstream of the river widening, implying similar sediment transport capacity. Additionally, turbulence levels will be higher due to flow separation downstream of the dike which could further increase sediment suspension capacity. However, the upstream bend and bridge piers also elevate turbulence intensity.

Figure 19 shows channel bed shear stress which can be compared to the upstream shear stress and the shear stress in the existing conditions (Figure 20) simulation. With both shear stress and velocity similar to the upstream reference reach, similar sediment deposition is expected. Most of the sediment will likely deposit further downstream, reducing dredging needs at the Port of Lewiston. The simulations use a post-dredge terrain, rather than the one shown in Figure 16, as dredging would occur prior to construction. Overtime, as sediment accumulates in the dredged area, shear stress would increase due to reduced depth, which has the potential to keep bed sediment mobilized at some flow rates.

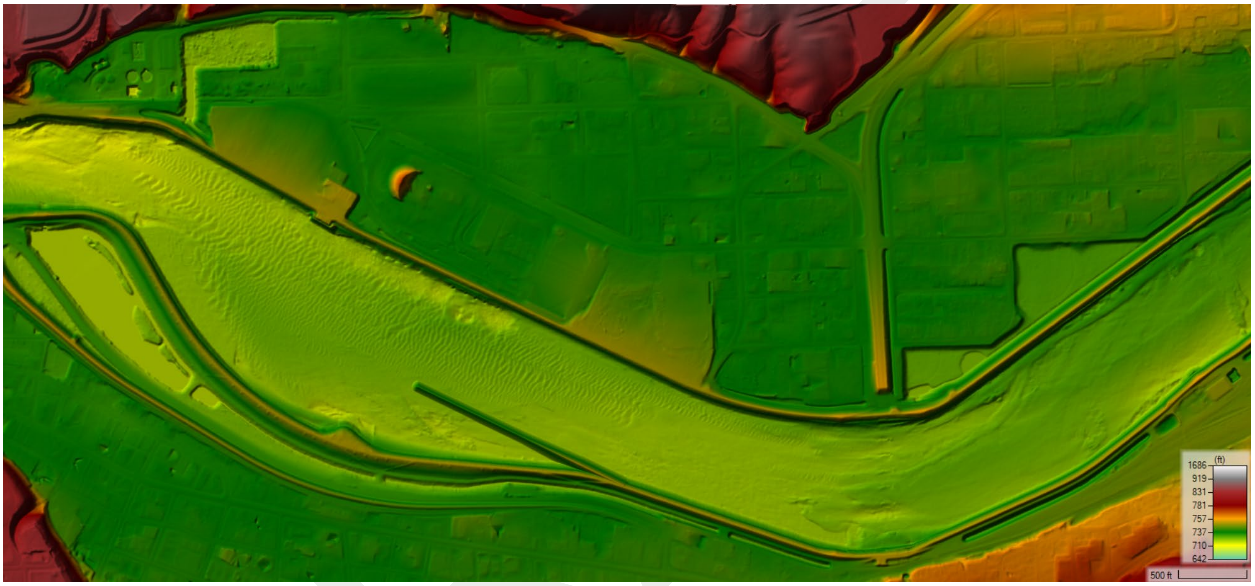


Figure 16. Longitudinal Dike Measure at the Port of Lewiston

To prevent a rise in upstream water surface during high-flow events, a submerged longitudinal dike was considered (Figure 21). This structure was optimized to minimize impacts to the upstream water surface and downstream railroad bridge scour. The shorter structure would be fairly inefficient at reducing sediment deposition during low-flow events when navigation depths are maintained and the dam is operated above MOP. However, if the dam forebay were drawn down below MOP the structure has the potential to be very effective. For example, Figure 22 shows the submerged dike with the dam operating at 12.3 feet below MOP during a 20% annual chance exceedance event. The forebay stage is 9.7 ft below the stage depicted in Table 8. At this flow rate and forebay stage, the post-dredge navigation channel depth at the Port of Lewiston would be slightly shallower than 14 feet. Since large vessels are unlikely to be navigating the river at this fairly high flow rate, the channel depth is not a concern. When comparing velocity magnitudes in Figure 22 to Figure 18 it’s clear that the sediment deposition would be reduced significantly. Drawing down the reservoir also increases downstream velocity which would push deposition farther downstream, which is an added benefit over the unsubmerged dike since with that dike the sediment is more likely to deposit in problematic locations downstream due to lower downstream velocity.

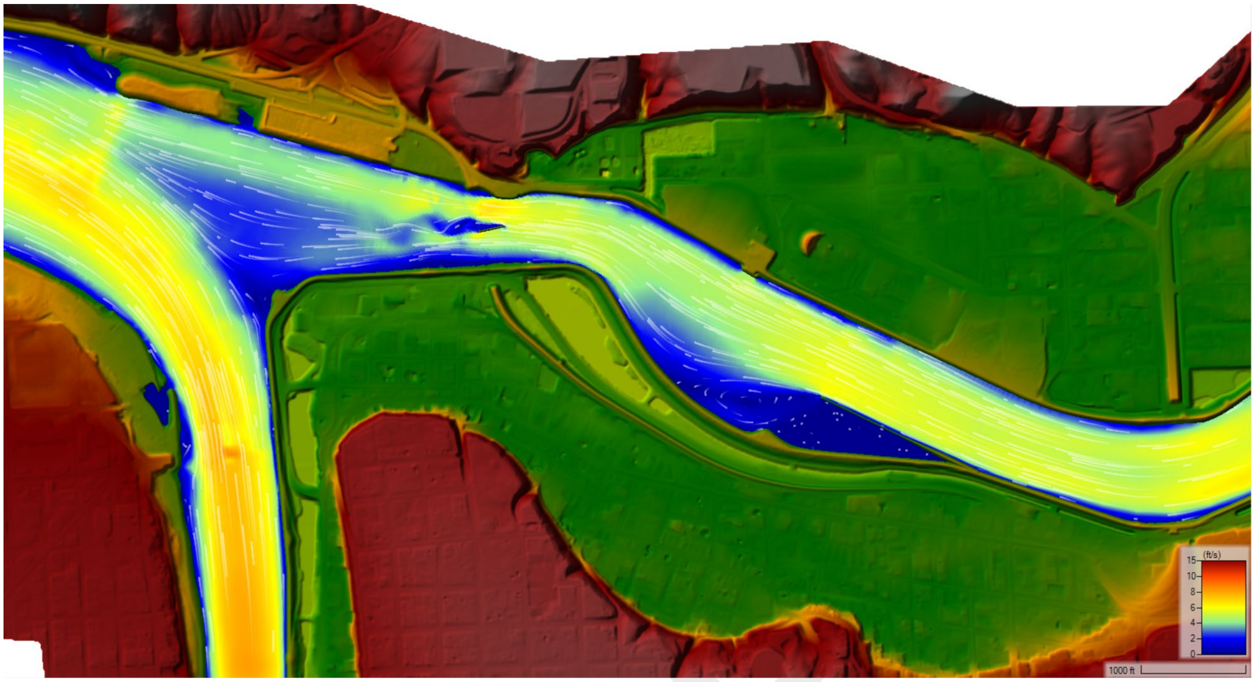


Figure 17. Simulated Velocity Patterns for the 20% Annual Chance Exceedance Flow Rate – Longitudinal Dike Measure at the Port of Lewiston

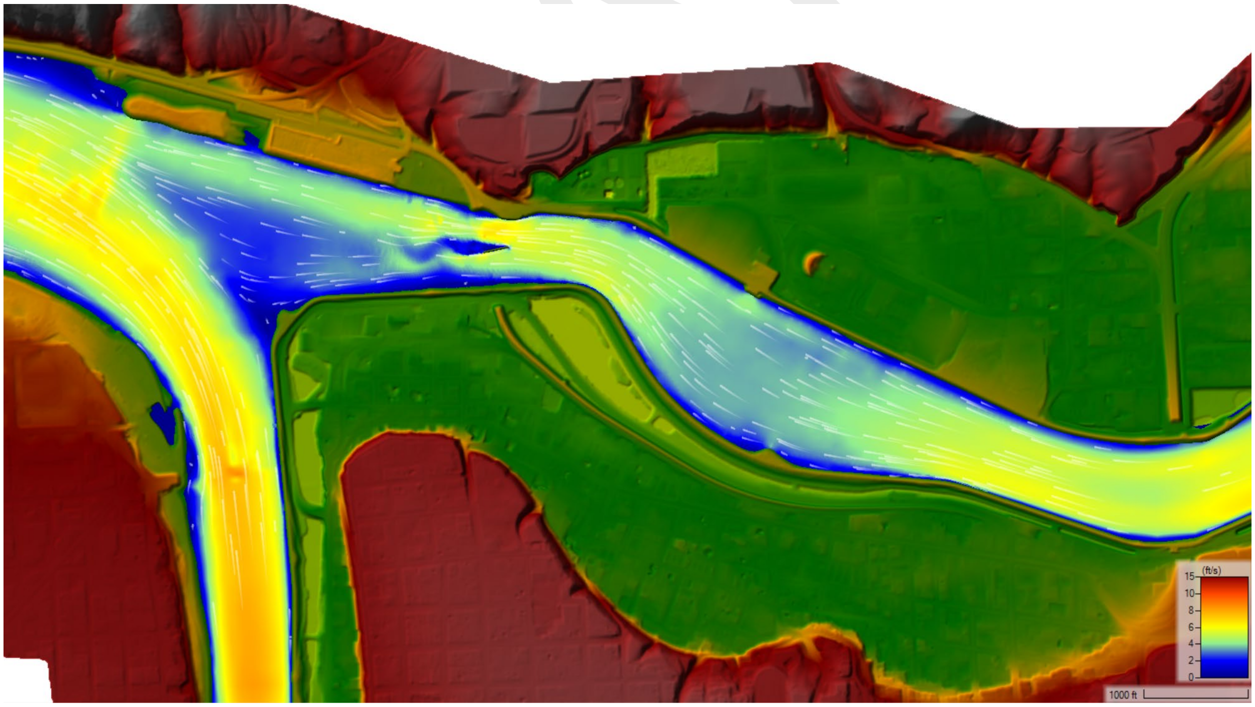


Figure 18. Simulated Velocity Patterns for the 20% Annual Chance Exceedance Flow Rate – Existing Conditions at the Port of Lewiston

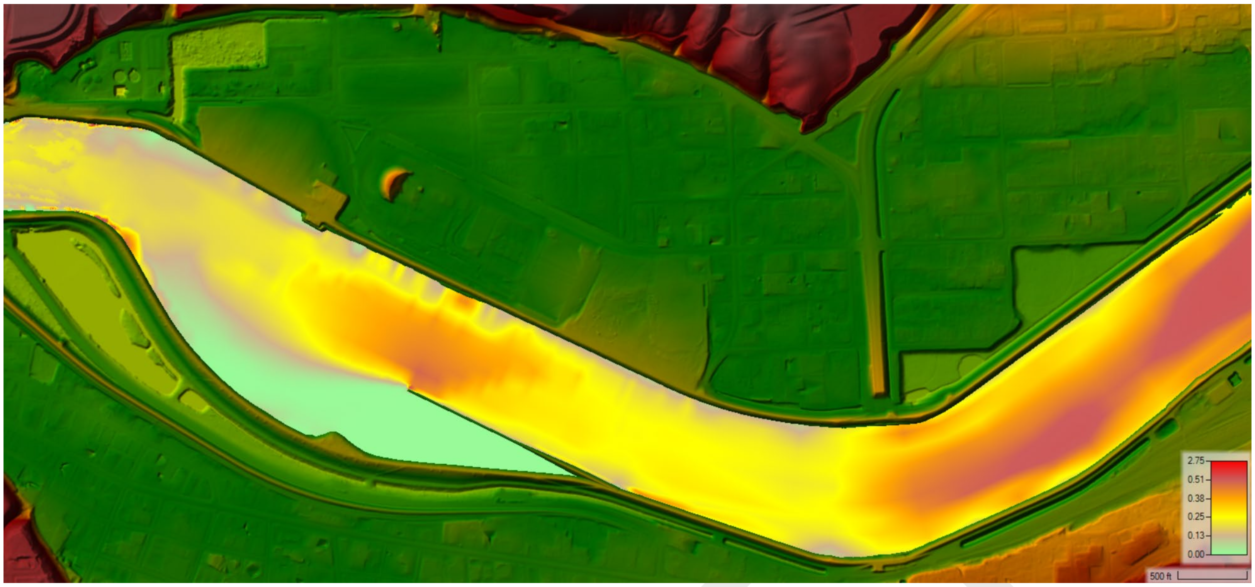


Figure 19. Simulated Bed Shear Stress (lbs/ft^2) for the 20% Annual Chance Exceedance Flow Rate – Longitudinal Dike Measure at the Port of Lewiston

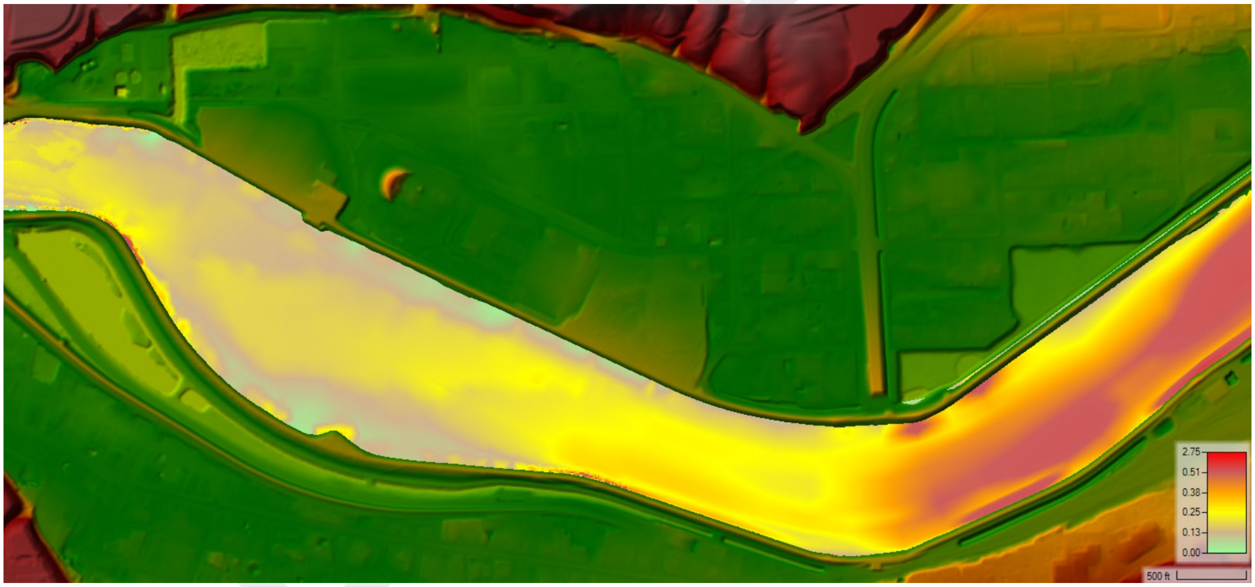


Figure 20. Simulated Bed Shear Stress (lbs/ft^2) for the 20% Annual Chance Exceedance Flow Rate – Existing Conditions at the Port of Lewiston

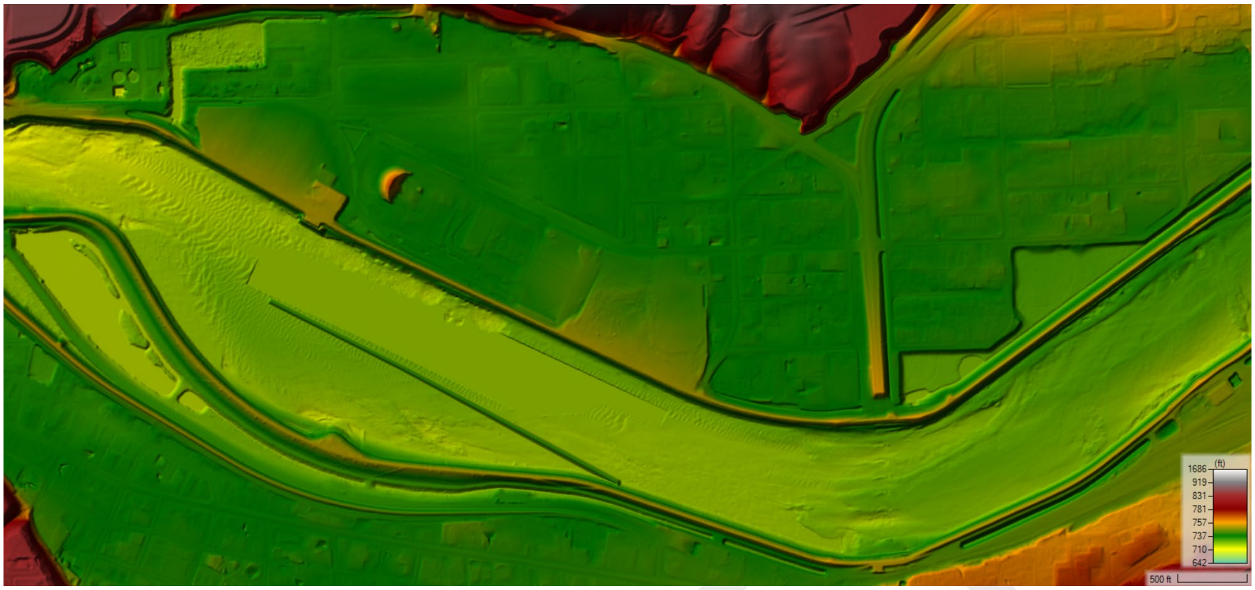


Figure 21. Submerged Longitudinal Dike Measure at the Port of Lewiston

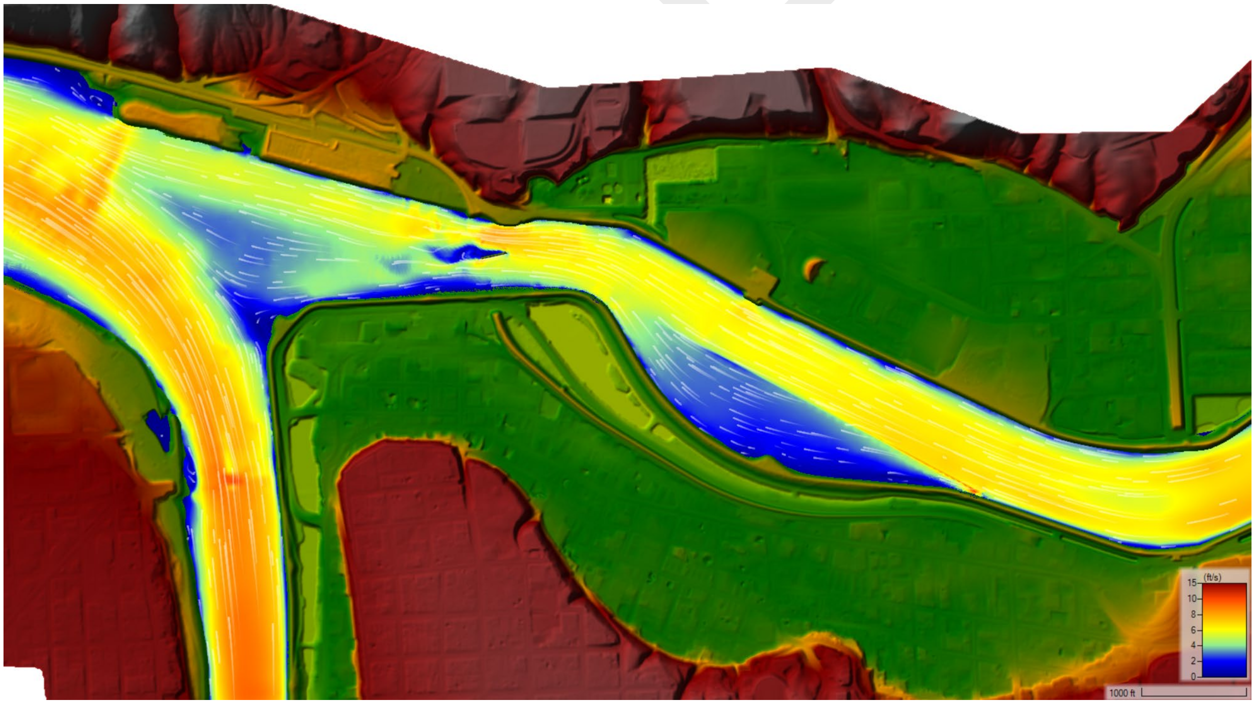


Figure 22. Simulated Velocity Patterns for the 20% Annual Chance Exceedance Flow Rate – Submerged Longitudinal Dike Measure at the Port of Lewiston

Figure 23 shows simulated bed shear stress for the submerged longitudinal dike. Because of the drawdown, shear stress is notably higher than both the existing conditions simulation (Figure 20) and the unsubmerged dike (Figure 19). Because of elevated bed shear stress when combining a reservoir drawdown with a longitudinal dike, it may be possible to erode bed sediment and clean out the channel via drawdown instead of dredging. Some dredging would occasionally still be required at the Port of Lewiston, but both the volume and the frequency could be reduced significantly. Notable sediment deposition tends to occur during high-flow events. The reservoir water surface could be drawn down during high-flow events to reduce deposition. When flows are low enough to permit navigation, the 14-foot-deep navigation channel could be maintained during drawdown. When flows are too high for navigation, the reservoir could be drawn down more. Reducing deposition in this way could reduce the frequency of drawdown events needed to erode sediment that has already deposited. Experimental drawdowns could be used to prove reliability and to evaluate possible adverse impacts.

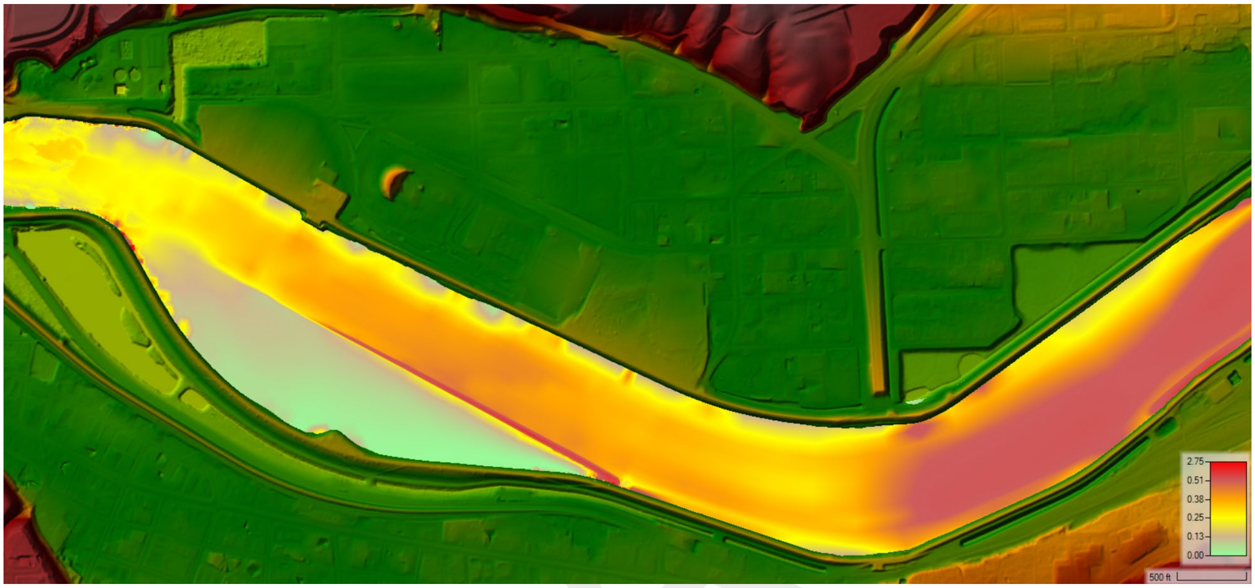


Figure 23. Simulated Bed Shear Stress (lbs/ft^2) for the 20% Annual Chance Exceedance Flow Rate – Submerged Longitudinal Dike Measure at the Port of Lewiston

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