PUGET SOUND NEARSHORE ECOSYSTEM RESTORATION

APPENDIX B

ENGINEERING APPENDIX

Integrated Feasibility Report and Environmental Impact Statement

US Army Corps of Engineers
Seattle District

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Contents

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Section 1 – Duckabush River Estuary

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1-1 GENERAL – DUCKABUSH RIVER ESTUARY

1-1.1 Overview of Restoration Site

The Duckabush River is one of several major river systems that drain the east slope of the Olympic Mountains to Hood Canal. The broad river delta fans out into Hood Canal on the south side of the Black Point Peninsula at approximately Mile 310 of Highway 101 (Figure 1- 1-1). The estuary contains salt marshes, eelgrass beds, and extensive mud and gravel flats that are productive shellfish beds. The Duckabush Estuary is also home to harbor seals, bald eagles, and regionally significant winter waterfowl.

Figure 1- 1-1. Duckabush River Estuary and vicinity.

The Duckabush River is contained within a single channel through the site before emptying into the marsh and submerged marsh outboard of the site. The historic northern arm of the river has been blocked, is aggraded, and is a dead-end channel in the middle portion of the site. The Duckabush River Estuary was bisected by an early roadway and bridge that spanned the two distributary channels. A portion of the roadway, dikes, and abutments remain in place today. The majority of this infrastructure was removed and replaced in 1934 with two separate bridges as part of the construction of Highway 101. This highway cuts across the intertidal river delta and estuary wetland complex, spanning the main channel and a former distributary channel via two bridges. The Highway 101 bridges impact the Duckabush estuary, disrupting tidal circulation and impeding fish access to productive salt marsh and slough habitats. These hydraulic constrictions along with fill within the estuary have led to decline in mudflats and salt marsh. In addition, training berms are in place at the southern arm of the Duckabush distributary channel, just upstream of the Highway 101 crossing, to control lateral movement of the channel and prevent river flows into historical distributary channels. These berms severely restrict lateral connectivity with tidal channels and salt marsh habitat.

From conversations between the Washington Department of Transportation (WSDOT), the Corps and the Washington Department of Fish and Wildlife (WDFW) about the Duckabush bridges, they are both listed by WSDOT as functionally obsolete, but neither are on any list for replacement or repair neither due to their obsolescence nor for chronic environmental deficiencies. Since SR 101 is not on the Interstate System and there is no anticipated involvement from the Federal Highway Administration (FHWA).

The Duckabush Estuary is home to trumpeter swans, bald eagles, and regionally significant winter waterfowl. Harbor seals haul out in this location throughout the year and pupping occurs in the winter. The extensive mud and gravel flats are productive shellfish beds. Salt marshes and eelgrass beds characterize the upper and lower intertidal and subtidal areas, respectively. Herring use this eelgrass for spawning. The Duckabush River hosts four Endangered Species Act (ESA) listed species of salmon: Hood Canal summer chum, Puget Sound steelhead, Coastal/Puget Sound bull trout, and Puget Sound Chinook salmon. The wild Chinook run is nearly extirpated from this river.

The proposed restoration would restore tidal and riverine hydrology to 38 acres of the Duckabush River Delta. This action would allow for natural habitat forming processes including sediment and detritus exchange, tidal channel formation, freshwater input, and tidal flushing within the delta.

1-2 HYDROLOGY AND HYDRAULICS

The Duckabush River Estuary site lies along Hood Canal on the east side of the Olympic Peninsula. The headwaters of the 20-mile Duckabush River are located on the eastern slope of the Olympic Mountains on the Olympic Peninsula. The 67 square-mile basin is comprised of 95% forested land, most of which is located in the National Park or National Forest boundary. The river is characterized by a steep gradient (150 feet per mile, on average) with average precipitation in the basin is approximately 113 inches per year, with a 2-year event equaling 3.5 inches per hour. For the last mile, the river flattens quickly and flows through a 1960's development called the Olympic View Tracts before it terminates in Hood Canal just south of the town of Brinnon. The gradient through this section is approximately 40 feet per mile and much of the river in this reach is tidally influenced. In a typical water year, the Duckabush River has a dual peak, with the largest flows in the fall and a lesser peak during the spring snow melt. Upstream of the estuary, residents in Olympic View Tracts have constructed private revetments along the banks of the river (USACE 2003).

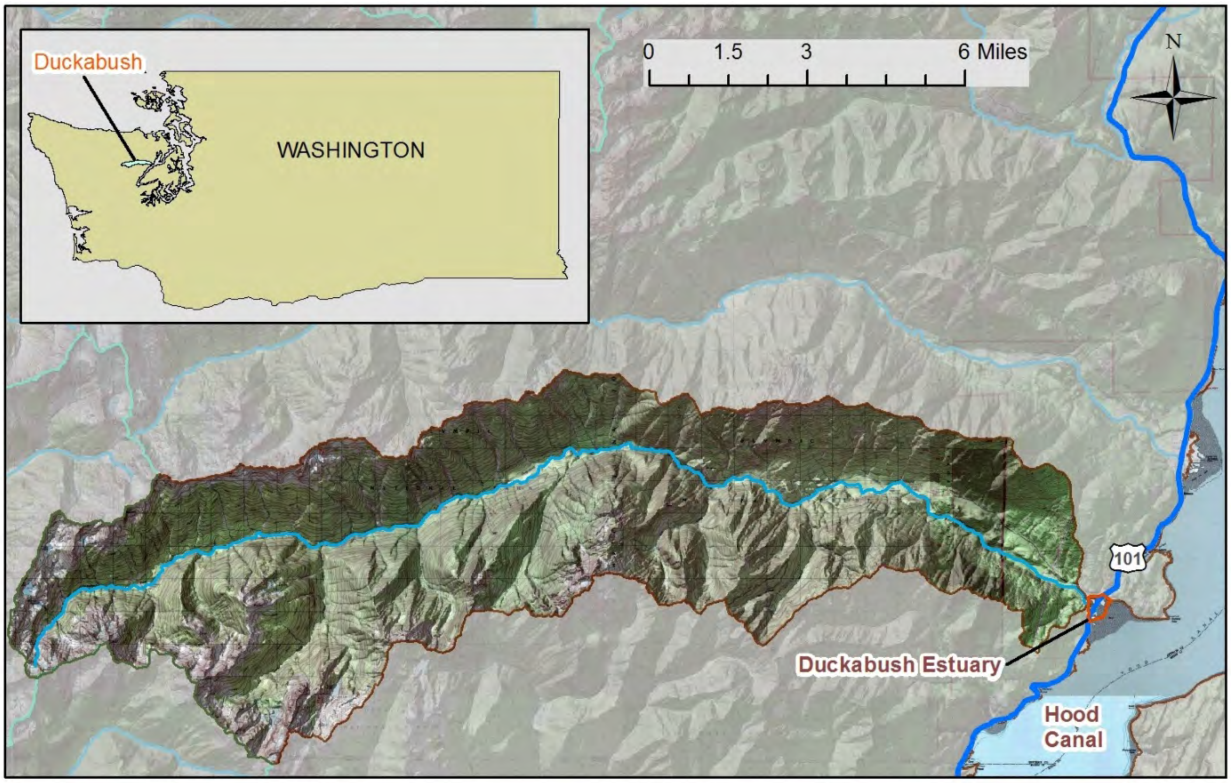

Figure 1-2-1. The Duckabush River Estuary site – Duckabush River watershed

The site is located in the estuary of the Duckabush River. Figure 1-2-1 shows a map of the watershed, including the Duckabush River, Hood Canal and the local drainages that affect the Duckabush site. On the north side of the estuary there is a small catchment of 0.5 square miles that terminates in a fish-bearing culvert at Highway 101. The construction plan calls for removal of this culvert and rerouting of the catchment drainage to the restored estuary.

The hydraulics and hydrology for all restoration sites in the Puget Sound Nearshore Ecosystem Restoration Project were evaluated using an area of potential hydraulic effects specific to the construction requirements for each particular site. The upstream and lateral limits of the area for this site were established using 100-year Base Flood Elevations (BFEs) derived from a combination of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRMs) and Flood Insurance Studies (FISs) (FEMA 1982).

According to the 1% Annual Exceedance Probability (100-year) BFE as determined by the effective FEMA flood insurance mapping for unincorporated areas of Jefferson County, community 530069 (revised 1982), the entire site lies within the 100-year floodplain. Figure 1-2-2 shows the area of potential hydraulic effects for the Duckabush site. Downstream and seaward limits are based on changes in shoreform type and best professional judgment.

The BFE at the site depends primarily on coastal flooding from storm surge that originates on the West side of the Olympic Peninsula. According to the FEMA FIS, most of the very high stillwater levels were not associated with high wind velocities and were instead associated with the dynamics of a Pacific Ocean surge tracking into Puget Sound. The coastal BFE is 15.3 feet (NAVD88) which extends the stillwater surface upstream of the current Highway 101 location. The Duckabush River will have the same tailwater elevation for the base flood both with and without the project. The upstream end of hydraulic effects may extend above the BFE since the increased conveyance from bridge removal will likely reduce upstream water levels for high rainfall flow events that occur without the coastal flooding.

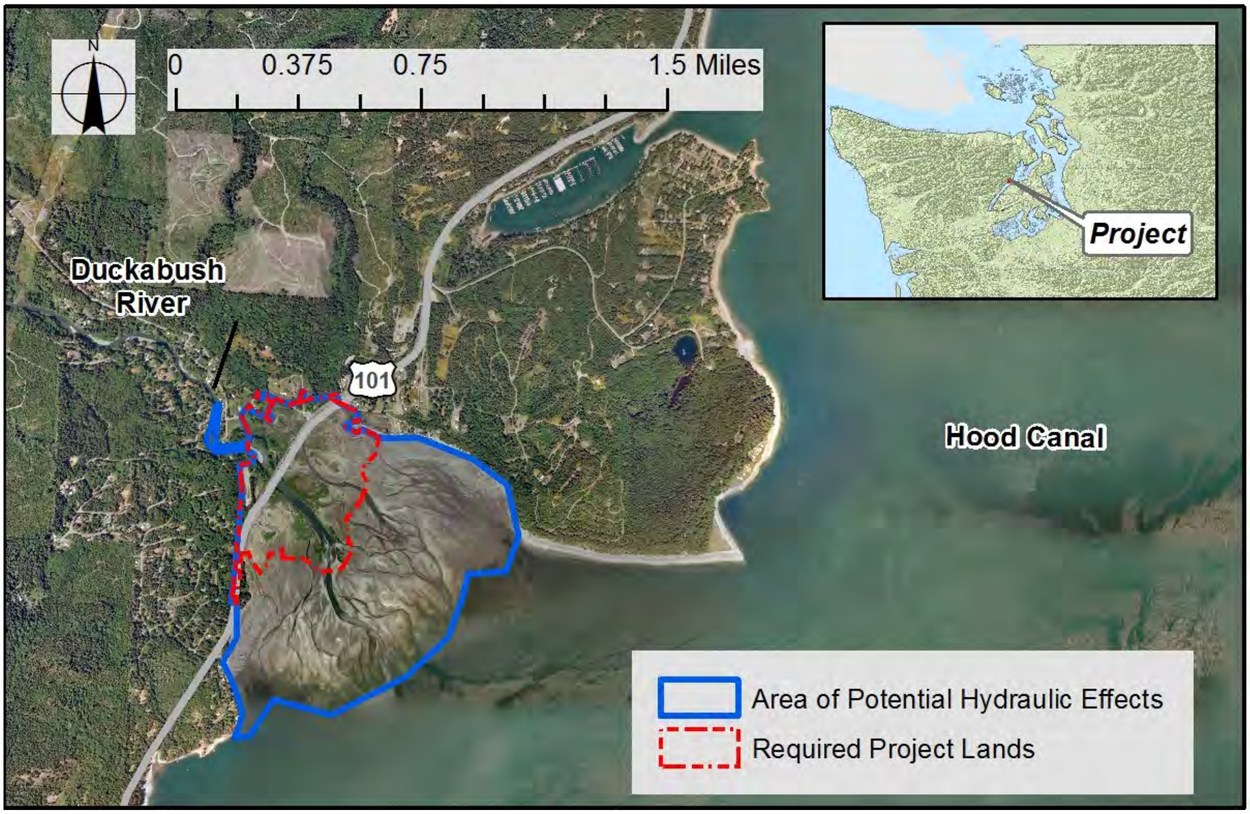

Figure 1-2-2. The Duckabush River Estuary: Area of potential hydraulic effects.

The Ecosystem Output Model (EOM), described in Appendix G utilized an area of restored process determined as follows:

The upland portion of each analysis area was delineated to ensure that the area included all stressor distributions within defined buffer distances from the shoreline. In the aquatic areas, the shape of the analysis area was determined by a combination of:

- The area provided initially by the design team and the associated parcel map for the proposed action
- Ensuring an area encompassed all delineated tidal wetlands
- For any analysis area that extended through an aquatic area, boundaries were established approximately perpendicular to the shoreline orientation where the upland meets the shoreline.

The area of restored process at Duckabush is shown in Figure 1-2-3 as 38.1 acres. For more information, please refer to Puget Sound Nearshore Ecosystem Restoration Project Fish and Wildlife Coordination Act Section 2(b) report in Appendix J, Environmental Compliance Documentation.

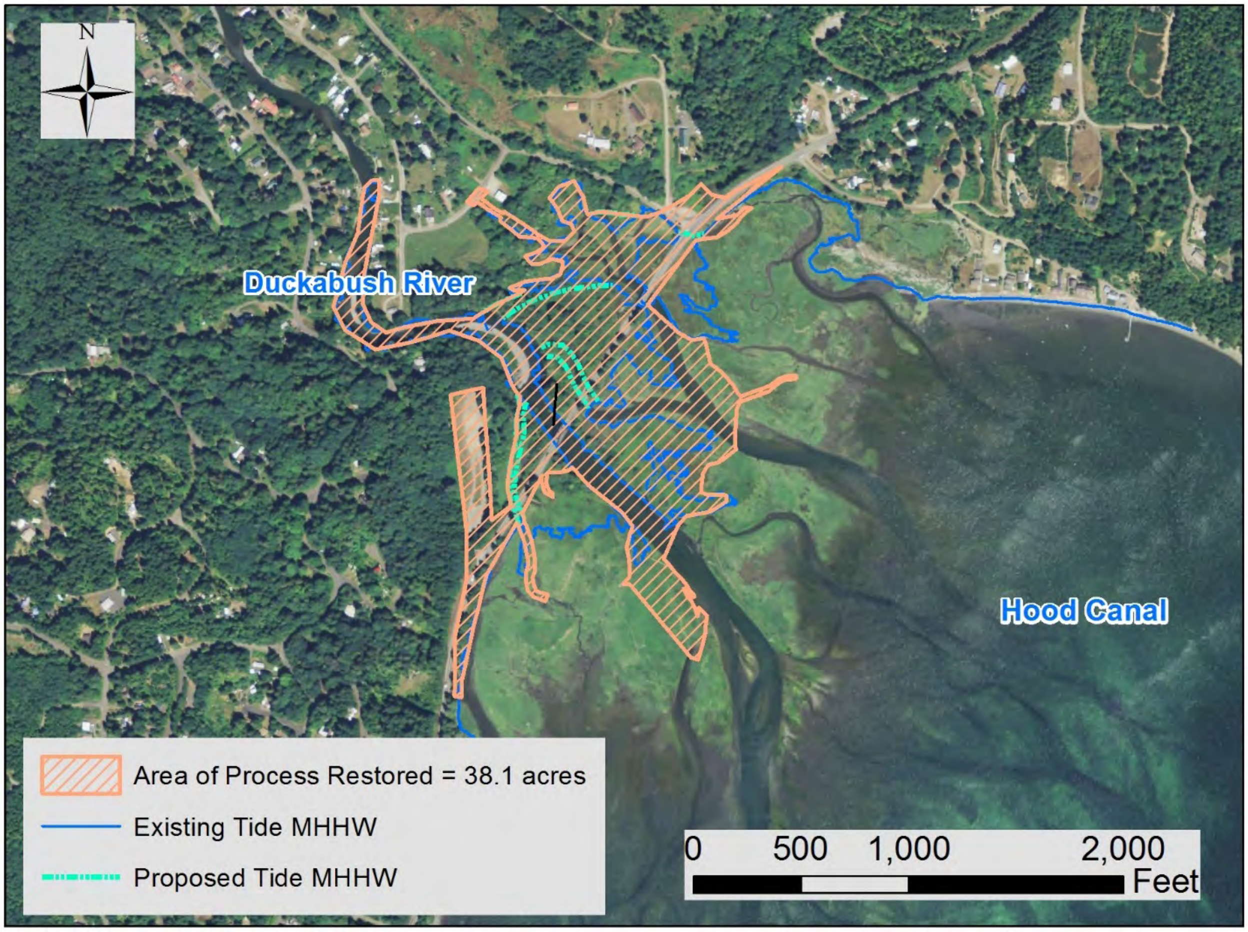

Figure 1-2-3. Area of process restored used in ecosystem output model at the Duckabush River Estuary.

1-2.1 Functional Design Requirements

This section describes the hydrologic and hydraulic setting for the site and the intended hydraulic consequences of the design features.

1-2.1.1 Consequences of flows exceeding discharge capacity of the project

The purpose of the work at this site is to remove Highway 101 roadway embankment and bridges that impact the Duckabush estuary, disrupting tidal circulation and impeding fish access to productive salt marsh and slough habitats. This site does not include water control facilities other than roadway drainage culverts. Should discharges exceed the design capacity of the culverts there would be a potential for roadway overtopping.

1-2.1.2 Project-induced changes obligating mitigation

No compensatory mitigation is included for this site as none is required. Temporary fill for construction in the estuary will be designed according to conservation measures provided in the Biological Opinion from the National Marine Fisheries Service to minimize impacts to aquatic species. The restoration actions would have negligible, short-term construction related effects. All of these minor and temporary effects can be avoided and minimized through construction designs and standard best management practices (BMPs). Specific measurable and enforceable measures would be developed based on the specific effects of the project.

Mitigation for any change to or removal of the Duckabush Bridge may be required due to its listing on the National Register of Historic Places. The Corps is coordinating a Programmatic Agreement for compliance with Section 106 of the National Historic Preservation Act. If it is determined that the project will have an adverse effect on any significant structures, including the Duckabush Bridge, the Corps will avoid, minimize, or mitigate following Section 106 procedures and stipulations in the Programmatic Agreement.

1-2.1.3 Discharge-frequency relationships

The site is located on the Duckabush River near Brinnon, Washington. Discharges at this location are taken from the effective FEMA FIS (1982) and are based on USGS gage data from Gage 12054000 with records from 1911 and 1938 through 1982. USGS gage has data available from 1938 through the current year. Annual Exceedance Probability (AEP) along with discharge is shown in Table 1-2-1. The hydrology for the Duckabush River will be reviewed and updated in PED.

Table 1-2-1. Peak Discharge – Frequency predictions for Duckabush River at Brinnon (FEMA 1982).

Location	10% AEP (cfs)	2% AEP (cfs)	1% AEP (cfs)	0.2% AEP (cfs)
Duckabush River Near Brinnon	6,760	8,870	9,770	11,900

1-2.1.4 0.2% Annual Exceedance Probability (500-year return interval) flood

The area of potential hydraulic effect for the Duckabush Estuary is dominated in the lower reaches by storm surge from Hood Canal. The 0.2% AEP coastal base flood elevation of 15.6 feet as reported by FEMA (1982) is very close to the 1% AEP (15.3 feet). Since work at the site involves the construction of a bridge, the 0.2% AEP will have to be re-evaluated during PED.

1-2.1.5 Stage-discharge relationships

Stage discharge relations are reported in the FEMA FIS (1982) are shown in Table 1-2-2. In order to update the forecast for stage discharge relations a coastal flooding analysis will be conducted in PED. This can be combined with a 1D - 2D HECRAS model of the upstream reach of the Duckabush River to evaluate any effects that may occur under various flow scenarios. While a hydraulic model is not needed for purposes of flood reduction, the project design must take in to account possible impacts that could occur during high flow events (bankfull or greater) that don't include coastal flooding. The FEMA FIS assumes the coastal BFE as a tailwater condition. More frequent flood events on the Duckabush River can result from storm events without coastal flooding. For a riverine flood with tailwater at Mean Lower Low Water, the increased conveyance at the Duckabush River crossing may result in substantially less backwater and higher flow velocities upstream. The riverine model will be used to ensure that the project design will not cause headcutting or scour at revetments upstream or unwanted sedimentation in the estuary. It will also be used to ensure that piers and abutments are sized and aligned so as to minimize scour in bankfull flood events.

Table 1-2-2. Stage-discharge relations as shown in the effective FEMA Flood Insurance Study for the Duckabush site (FEMA 1982)

Location	10% AEP Stage (feet NAVD88)	2% AEP Stage (feet NAVD88)	1% AEP Stage (feet NAVD88)	0.2% AEP Stage (feet NAVD88)
Duckabush River near Brinnon	14.9	15.2	15.3	15.6

1-2.1.6 Flow duration

Flow duration data are available from daily discharge readings near Brinnon (USGS 12054000) for the period between July, 1938 and the present. An unsteady flow analysis or flood flow routing will not likely be required for this site since flooding is governed by the static coastal flood elevation.

1-2.1.7 Flood inundation boundaries and flood stage hydrographs

The current flood inundation boundaries as reported for the 1% AEP (100-year) flood event in the Jefferson County FEMA Flood Insurance Study are shown in Figure 1-2-4. For the 1% AEP event, the Duckabush River floods from valley wall to valley wall at the site.

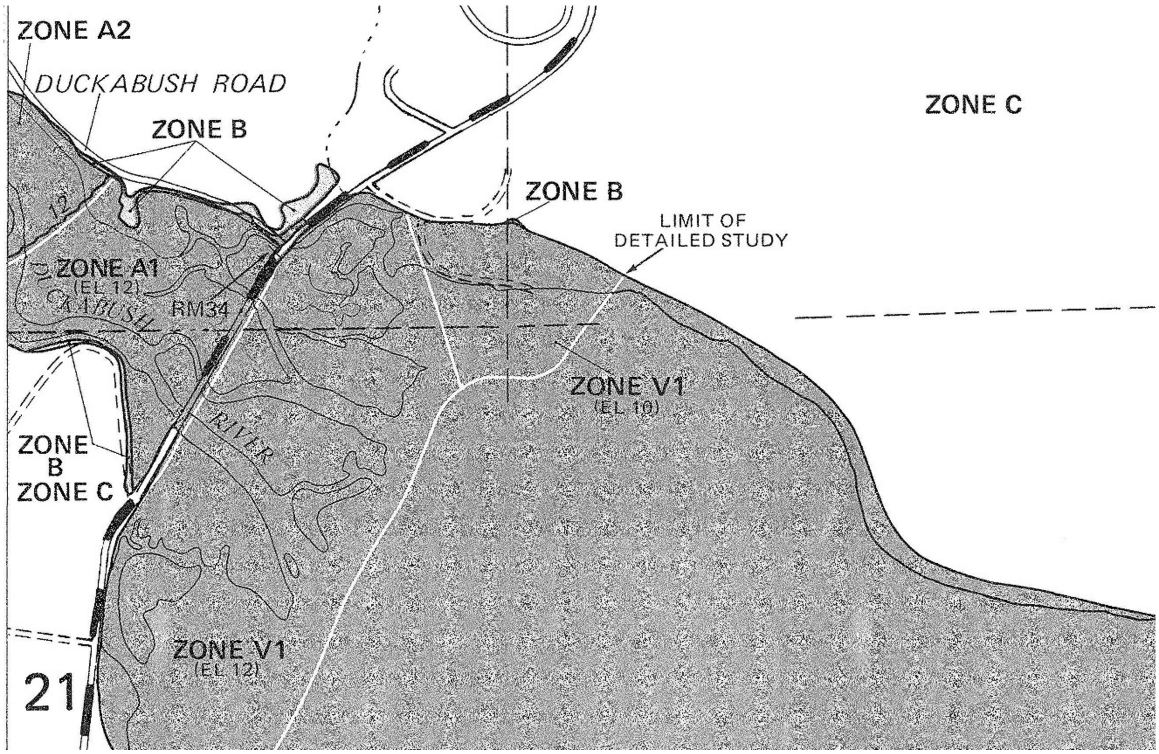

Figure 1-2-4. Effective FEMA 100-year flood zone as adapted from Flood Insurance Rate Map (FIRM 5300691245B) (FEMA 1982) (Elevations in NGVD29; NGVD29 + 3.57 feet= NAVD88). Note that inundations have been rounded to the nearest foot on this map.

1-2.1.8 Reservoir yields

No reservoirs are planned as part of this site.

1-2.1.9 Risk and uncertainty analysis for sizing of the project under study

Channel sizing:

Channels will be excavated to equilibrium dimensions as described in the *Applied Geomorphology Guidelines* (Attachment B). Equilibrium channel top widths for an estuary of this size are around 50feet, with depths between 5 and 10 feet below existing grade. A channel depth of 10 feet would allow for some infilling of the channels without impacting the performance of the restoration. The way in which the new distributary channels will evolve has not been analyzed in the Feasibility phase, but the concept of natural evolution, coupled with equilibrium channel design in PED, minimizes risk because there are no specific discharge requirements for the new channels beyond providing an initial equilibrium design..

Sea Level Change

The Duckabush River Estuary is located in the Hood Canal Sub-basin of Puget Sound. Sea Level Change (SLC) calculations for the Hood Canal Sub-basin are based on the Port Townsend tide gauge and are calculated using the guidance in ER 1100-2-8162, Incorporating Sea Level Change in Civil Works Programs, and ETL-1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses and Adaptation (USACE 2013, 2014). Table 1-2-3 shows the range of sea level change projections for the 50-year project life as well as the 100-year horizon assuming a project start date in 2020. Changes are referenced to 1992, which is the midpoint of the most recent National Tidal Datum Epoch as established by NOAA. The high rate calculations indicate a sea level rise of 2.3 feet in 50 years after project start and a rise of 6.4 feet after 100 years .

The largest risk associated with sea level change at this site is the displacement of habitat upstream, with vegetated marshes becoming intertidal habitat and intertidal habitat becoming sub-tidal habitat. Tidal marshes can adapt to sea level change by building elevation to keep pace with the rising water levels, but this requires an adequate supply of sediment and/or organic matter accumulation. Future studies should include a sedimentation analysis to determine what impact the restoration will have on sedimentation rates and if there is sufficient sediment accumulation to keep pace with the projected sea level change. The sedimentation analysis will be used to fine tune initial channel design and to assess the anticipated environmental benefits from the project over the design life.

Table 1-2-3. Projected Sea Level Change (feet) Port Townsend (Gauge 9444900). Source: USACE Sea-Level Change Curve Calculator (2015.46).

Year	Low (feet)	Intermediate (feet)	High (feet)
1992	0	0	0
1995	0.02	0.02	0.02
2000	0.05	0.06	0.08
2010	0.12	0.15	0.24
2020	0.18	0.25	0.47
2030	0.25	0.38	0.78
2040	0.31	0.52	1.17
2050	0.38	0.68	1.62
2060	0.44	0.85	2.16
2070	0.51	1.05	2.76
2080	0.57	1.26	3.44
2090	0.64	1.49	4.2
2100	0.7	1.74	5.03
2110	0.77	2.01	5.93
2120	0.83	2.29	6.91

Figure 1- 2-5 shows the land elevations that fall within the three sea level changes assumptions for coastal BFE. Each color indicates the additional area inundated from Hood Canal for each successively higher SLC assumption. The elevations shown in the figure are based on 2011 Lidar from the Puget Sound Lidar Consortium (PSLC 2011) and indicate land elevation only. Figure 1- 2-5 is not an inundation map. It does not show the influence of riverine inundation or the effects of roadways or flow control structures. USACE hydraulic analysis has not been carried out for this site and the riverine BFE has not been established for the three rate of rise assumptions. Most of the land in the Duckabush Estuary lies below the coastal BFE elevation, including some of the residences upstream of the site. Under conditions of sea level changes,

the inundation at the site will become deeper and the coastal flooding will affect more of the river valley upstream of the estuary including more of the residences. This flooding will occur regardless of whether the project is constructed or not.

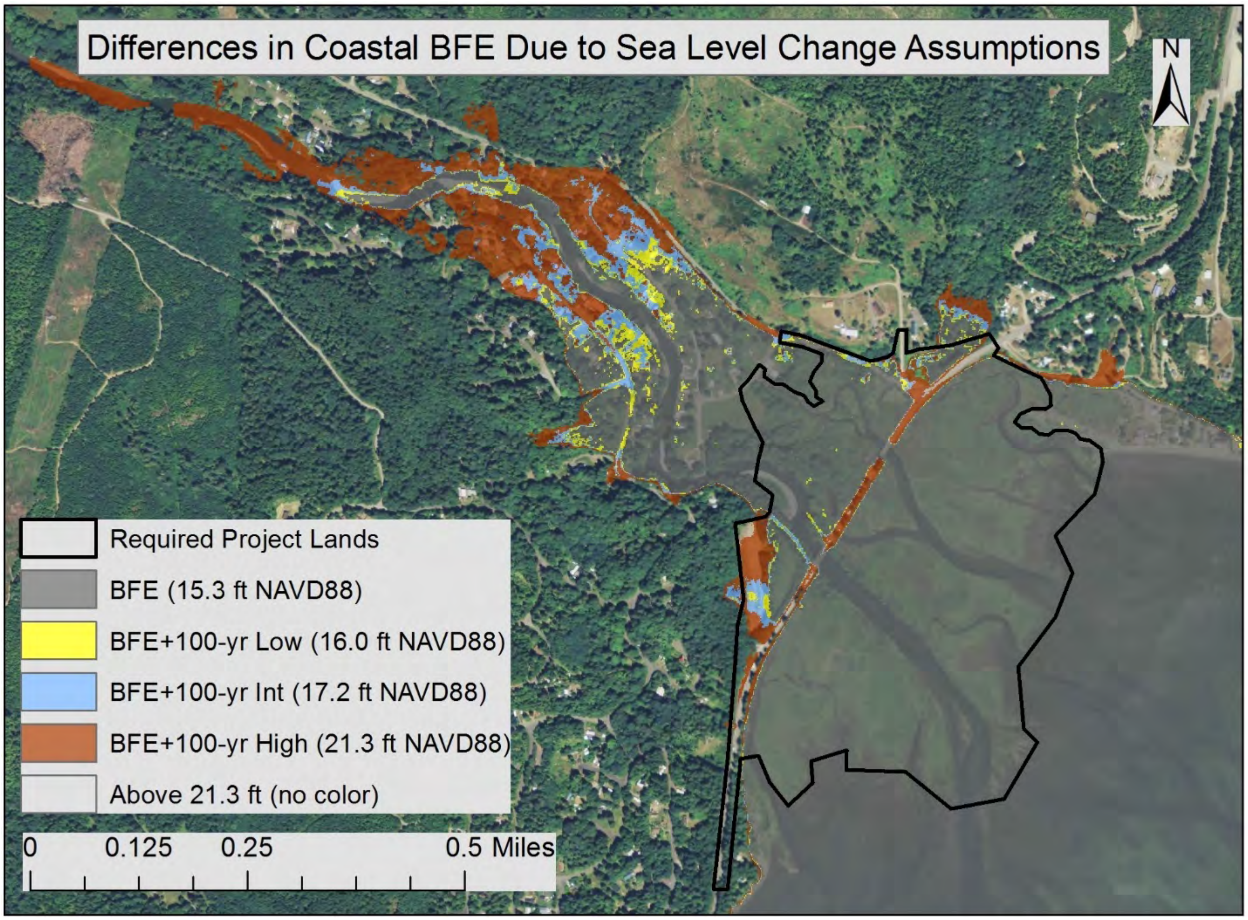

Figure 1- 2-5. Land at or below coastal BFE elevation for present day and projected 100 year low, intermediate and high rates of sea level change.

Figure 1- 2-6. Land at or below MHHW for present day and shows the land elevations that fall within the three sea level changes assumptions for Mean Higher High Water (MHHW). Each color indicates the additional area inundated from Hood Canal for each successively higher SLC assumption. Figure 1- 2-6 is not an inundation map, since USACE hydraulic analysis has not been carried out for this site. The elevations shown in the figure are based on 2011 Lidar from the Puget Sound Lidar Consortium (PSLC 2011) and indicate land elevation only – not the effects of roadways or flow control structures. The present day MHHW elevation covers some areas upstream of the Highway 101 roadway embankment. For successively higher rates of sea level change, more of the river valley is affected by SLC until, for the 100-year high rate of change assumption, some of the residences in the upper part of the estuary will lie below the MHHW elevation. This flooding will occur regardless of whether the project is constructed or not.

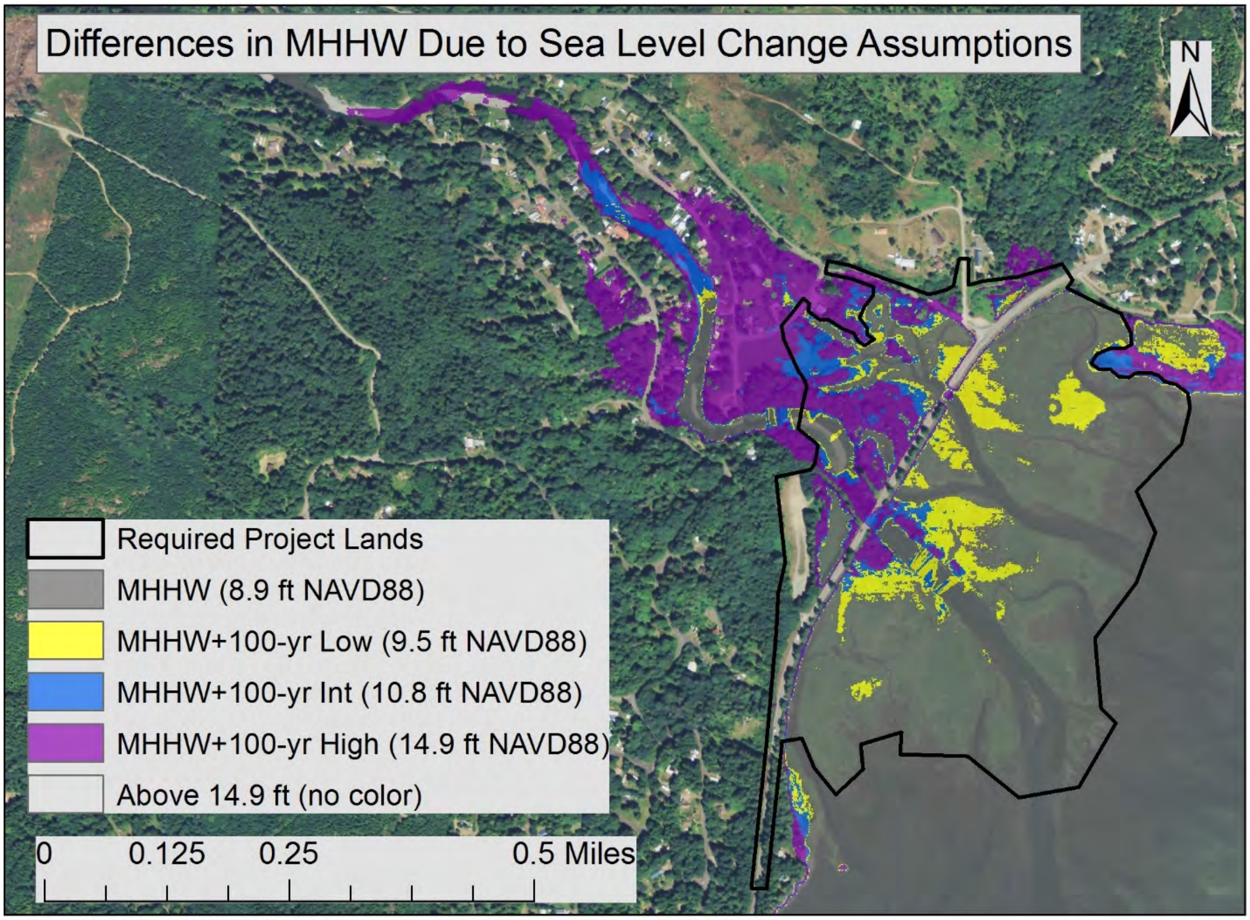

Figure 1- 2-6. Land at or below MHHW for present day and projected 100 year low, intermediate and high rates of sea level change

Figure 1- 2-7 shows the effects of potential SLC on the bridge at the Duckabush River Estuary. The design elevation chosen for the bridge is the 100 year intermediate level of change (roughly equivalent to the 50-year high level of change). While the PSNERP project planning timeline is 50 years, bridges are typically in place for much longer than 50 years. WSDOT bridges have a design life of 75 years and a service life which can extend well beyond that. The 100-year intermediate level of SLC was chosen as a scenario because the bridge is anticipated to be in place at least that long and likely longer. This design elevation was chosen using the following rationale:

- The bridge will still be usable for the 100 year high scenario. For the 100-year high scenario at Duckabush Bridge, the SLC is about 6 feet – if the bridge is designed for the 100 year intermediate of about 2 ft and the 100 year high SLC occurs, then the additional 4 feet of rise will eliminate the 3 foot debris clearance and impact the lower 1 foot of the bridge superstructure. The bridge will be designed to withstand this level of inundation on the bridge girders as well as possible debris impacts. A 100-yr high SLC scenario would mean higher OMRR&R costs for the bridge but not risk the integrity of the bridge or limit access.
- Highway 101 is the only North-South continuous roadway on the east side of the Olympic Peninsula. For a distance of 60 miles there are numerous low-lying waterway crossings. Most of these lie to the South of the Duckabush River. There is a single river crossing to the North at the Dosewallips River. For the 100 year intermediate scenario, the Duckabush Bridge may remain connected for emergency access to and from the North. For the 100 year high scenario, the Duckabush Bridge would not have connection to emergency services either northward or southward even if the bridge itself was unaffected.

- For the 100-year intermediate scenario, the road to the north may be passable since the Dosewallips River Bridge may only be partially inundated. Fewer locations on the road to the south are inundated compared to the 100-year high scenario.
- For the 100-year high scenario, much of the current road southward will not be passable in the BFE event as there are over ten inlet crossings that would lie below the BFE. The road to the North will be inundated at the Dosewallips River crossing preventing access to high ground inland.

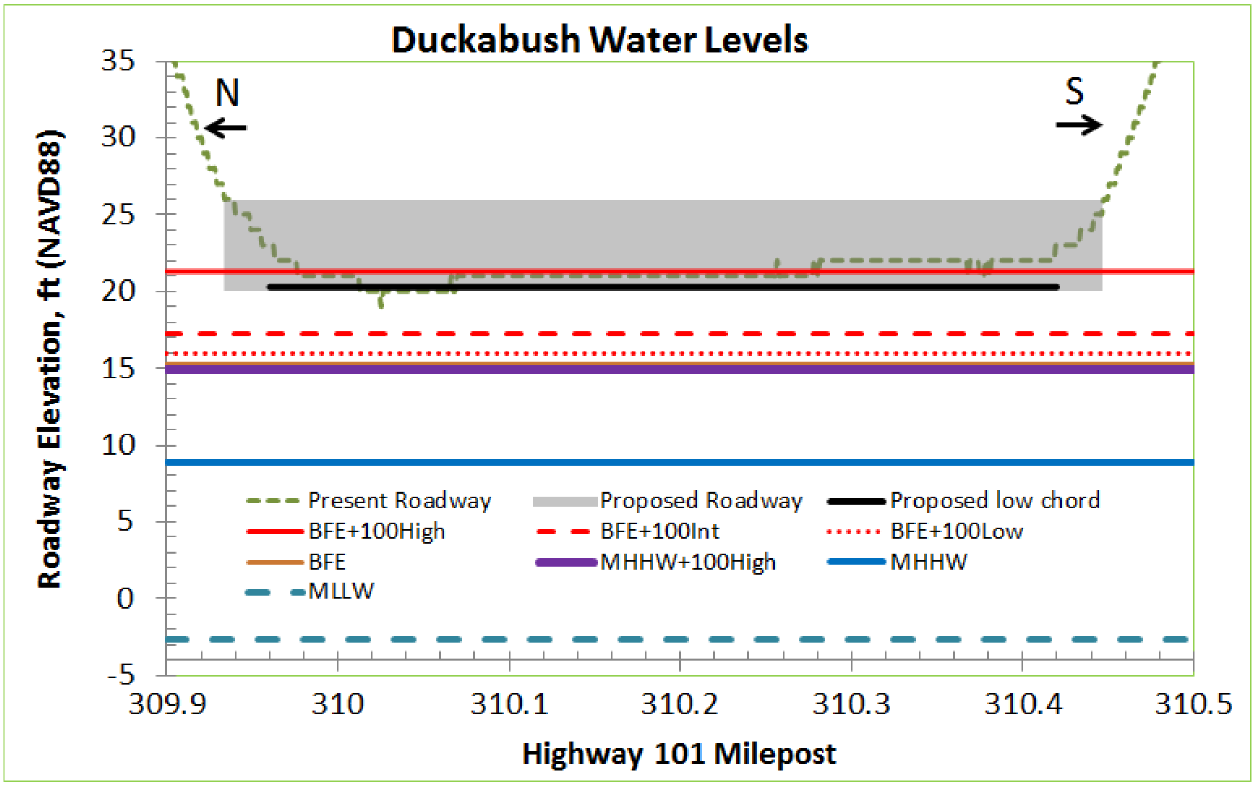

Figure 1- 2-7. Effects of potential sea level change on the base flood elevation at the Duckabush Estuary.

<u>Change in Inland Hydrology due to Climate Change</u>

ECB No. 2014-10 (USACE 2014) provides initial guidance for incorporating climate change information in hydrologic analyses in accordance with the USACE overarching climate change adaptation policy. There is a strong consensus among recent studies that future storm events in the Pacific Northwest region will be more intense and more frequent compared to the recent past (USACE 2015). The overall projected trends for the Pacific Northwest are summarized in the FR/EIS section 3.6.5.1.

Halofsky et al (2011) and Mauger et al (2015) present findings based on Global Climate Models (GCMs) for the Olympic National Forest and Puget Sound respectively. The modeling is based on greenhouse gas emissions scenarios proposed by the International Panel on Climate Change (IPCC) and range from an extremely low scenario involving aggressive emissions reductions to a high "business as usual" scenario with substantial continued growth in greenhouse gases.

The Duckabush watershed is a "two peak" water shed with a high peak flows from winter storms and a secondary snowmelt runoff peak. Reductions in snowpack and shifts in timing of snowmelt are expected with increasing temperatures in the 21st century. Halofsky et al report that April 1 snow water equivalent (a measure of water in snowpack) is projected to decrease by an average of 27 to 29 % across Washington State by the 2020s, 37 to 44 % by the 2040s, and 53 to 65 % by 2080. In the eastern Olympics they project an 8% increase in 5% AEP winter high flows and a 9% decrease 5% annual chance of occurrence summer low flows.

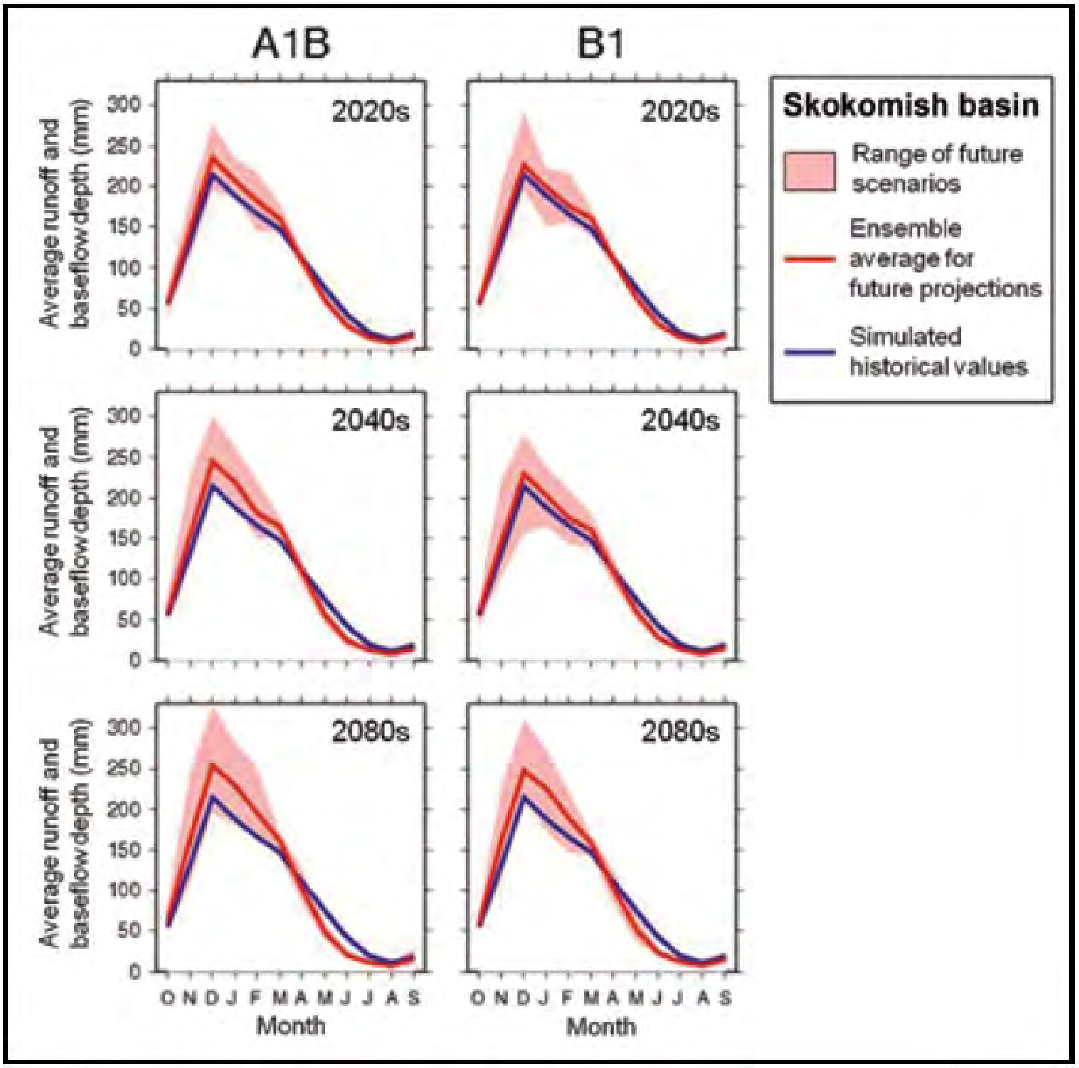

Figure 1- 2-8. Future projections for runoff for Skokomish basin using 10 global climate models (Source: Halofsky et al 2011).

Figure 1- 2-8 shows the simulated combined monthly average total runoff and baseflow over the entire Skokomish basin expressed as an average depth (millimeters). This variable is a primary component of the simulated water balance, and is one of the primary determinants of streamflow. The blue line shows the simulated historical values. Light red bands show the range of all future scenarios from 10 global climate models for the A1B (left column) and B1 (right column) emissions scenarios, and the red lines show the ensemble average for the future projections.

The current set of projections for the Eastern Olympic Mountains, reported in Mauger (2105) which stem from the most recent 2013 IPCC report and the results of 10 GCMs, predict a 20 to 60% increase in total winter runoff (December - February) by 2080, an increase in 24 hour runoff of 10 to 25% by 2080 and a decrease of 8 to 12% in summer precipitation. The results are less clear for peak flow values since less than 80% of the models had agreement on magnitude of change for the Eastern Olympic region.

1-2.1.10 Water quality conditions

Water quality information has not been reviewed in detail for this site. The restoration is not anticipated to generate any long-term effects on surface water quality. Anticipated water quality effects are as follows:

- Construction-related turbidity and suspension of sediments may occur due to removal of bank erosion protection and excavation of tidal channels. Sediment control will have to be carefully considered in the construction planning.
- Temporary increases in sedimentation may occur downstream of the site because of the release of sediment during the formation of any new distributary channels. These effects, together with other sedimentation issues, will be evaluated during PED.
- Levee lowering and channel construction may increase salinity within the site due to the increased tidal prism. Since the goal is to restore historic conditions, restoration of historic salinity patterns is presumed to be a desirable outcome. If needed, water quality sampling and analysis of water quality effects can take place during PED.

1-2.1.11 Groundwater conditions

No groundwater information has been reviewed for this site. The excavation of distributary channels will allow an increased tidal prism within the site which may be accompanied by saltwater intrusion. Since the goal is to restore historical conditions, restoration of historical salinity patterns is presumed to be a desirable outcome.

1-2.1.12 Preliminary project regulation plan

Not Applicable.

1-2.1.13 Preliminary real estate taking line elevations

The current real estate limits are delineated by the construction area, staging areas, and access roads and include the entire potential area of hydraulic effects. Real estate assumptions, valuations, and planning documents have been appropriately scaled for the current level of design. As additional surveys, modeling, and design are completed during the PED phase, the real estate documentation will be modified accordingly. For the current real estate status, refer to the Feasibility Report and Environmental Impact Statement (FR/EIS), Appendix C, *Real Estate Plan*.

1-2.1.14 Criteria for facility/utility relocations

Local utilities along Highway 101 will be relocated to follow the new bridge alignment. . For further details on the utility relocations see Section 1-6.3.

1-2.1.15 Criteria for identification of flowage easements required for project function

No flowage easements are anticipated for this site. This will be reviewed and confirmed during PED.

1-2.1.16 Criteria in support of project OMRR&R requirements

While channels are expected to evolve naturally once the stressors are removed, the larger channels and levee breaches have the potential to cause local aggradation, scour and transient sediment loading in and around the project and should be designed close to equilibrium conditions to avoid high initial maintenance costs. High initial sedimentation changes could cause a disruption to wildlife, fisheries and eelgrass downstream as well as impact the usual and accustomed fishing areas of local tribes. Rapid scour and aggradation of a channel as it adjusts to stressor removal could also require more maintenance and result in the loss of plantings. Operations, maintenance, repair, replacement and rehabilitation needs associated with the hydraulic function of the site are as follows:

- This restoration concept relies on the natural evolution of the floodplain and channels and initial channel shape close to the equilibrium condition, therefore no hydraulic performance maintenance is anticipated. If site specific objectives aren't being met with the process based restoration features, there may be some adaptive management required. Adaptive management costs are separate from OMRR&R.
- The roadways, bridges and culverts will require periodic maintenance. OMRR&R tasks are described in Section 1-15.

1-2.1.17 Environmental engineering considerations

In the context of hydrology and hydraulics, environmental engineering is taken to mean water supply and sanitation. Any existing water supply or sanitation systems within site boundaries will be decommissioned.

1-2.2 Residual Flooding Consequences – With Project Flooding

This section discusses the predicted hydraulic conditions after construction of the proposed restoration.

1-2.2.1 Warning time of impending inundation

There will be no residences or infrastructure within the site, except for the Highway 101 roadway and bridge. Aside from regional warnings for possible flooding, no warning system is planned.

1-2.2.2 Rate of rise, duration, depth, and velocity of inundation

No analysis of rate of rise and flow duration is planned for flood flows. The depths and velocities in the new tidal channels due to the combined effects of river flow and tidal prism will be evaluated during PED. This evaluation is required to confirm final channel dimensions and alignments.

1-2.2.3 Historic, 1% and 0.2% exceedance (100-year and 500-year) flood extents

The project is not expected to affect the 1% and 0.2% AEP flood extents.

1-2.2.4 Access and egress problems created by flooding

Highway 101 and Duckabush Road cross the site. The bridge and approaches are above the 1% AEP (100-year) flood elevation and are designed with 3 feet of debris clearance as well as an allowance for 100 year intermediate level of Sea Level Change (50-year high level). There will be no loss of access or egress during flood events due to the project. Area flooding may limit access to the bridge during floods due to local flooding upstream and outside the project area.

1-2.2.5 Potential for loss of life as a result of 1-2.2.1 through 1-2.2.3

The potential for loss of life as a result of the restoration is low. Areas within the site will be inundated by high tides. The entire site lies within the 100-year floodplain and is not likely to be occupied by people during floods.

1-2.2.6 Identification of any potential loss of public services

No public services are identified within the site. The fire station adjacent to the site should not be affected by the project. This will be confirmed during PED.

1-2.2.7 Potential physical damages

Potential physical damages that can occur during flooding will be addressed by the hydraulic analyses conducted during PED. This will include an evaluation of erosion and sedimentation as well as scour analysis of the bridge piers.

1-2.3 Project Induced Flooding – Change from Pre-Project Conditions

This section describes the effects of the site on flood elevations, flood patterns, and flood frequency.

1-2.3.1 Information categories required by 1-2.2

The project is not anticipated to cause any induced flooding.

1-2.3.2 Anticipated frequency of induced flooding

The project is not anticipated to cause any induced flooding.

1-2.4 Inundation Risk 0.2% Exceedance (500-year Return Interval) Flood

The site is dominated by the coastal base flood elevation which will not be affected by the project. Inundation risk for the 0.2% AEP event will remain the same.

1-2.5 Hydraulic Studies

This section discusses the hydraulic studies, construction considerations, and instrumentation and monitoring needs for the site. Hydraulic modeling during PED is being used both to fine tune the design of the bridge structure (piers and abutments) and to insure that channels are designed to achieve and to continue to achieve the desired ecosystem benefits with minimal maintenance. The anticipated hydraulic studies at this site are summarized in Section 1-21.

1-2.5.1 Hydraulic roughness determinations

If a hydraulic roughness determination is required to complete hydraulic analyses, then roughnesses will be determined using a combination of aerial photographs and field surveys during PED. Roughnesses will be calibrated using high water marks if available.

1-2.5.2 Water surface profiles

Water surface elevations as reported in the FEMA Flood Insurance Study (1982) require updating with recent hydrology, tidal information and coastal modeling. In order to predict the with-project water surface profiles, a 2-D coastal hydrodynamic model will be implemented which reflects the proposed geometry within the site and predicts an accurate Base Flood Elevation. This elevation is needed for design of the bridge and roadway, for detailed design of channels and to confirm the extent of ecological benefits. The model will include local inflows from surface water. In addition, an updated model of the flow in the Duckabush River is also recommended to evaluate the water surfaces within and upstream of the project boundaries and the effects of increasing the conveyance in the estuary. This modeling will be addressed during PED.

1-2.5.3 Stage-discharge relationships

Stage discharge relationships as reported in the FEMA Flood Insurance Study (1982) require updating with recent hydrology, tidal information and coastal modeling. In addition, an updated model of the flow in the Duckabush River is recommended to evaluate the frequency of occurrence of different stages within and upstream of the project boundaries. This information is needed to compete detailed design and to evaluate ecological benefits. This modeling will be addressed during PED.

1-2.5.4 Head loss

Other than the head losses that will be incorporated into the revised hydraulic model, no additional head loss studies are planned.

1-2.5.5 Flow and velocity

Flow and velocity information from the revised hydraulic analysis will be used to assess the possibility for sediment transport, scour, and bank erosion in the site area.

1-2.5.6 Structural sizing needed to meet design capacity including slope protection

No slope protection is planned for the site. Pier depths for the bridge will be designed to extend below scour depth or to terminate at bedrock. It is possible that some armoring will be needed for the bridge abutments or for the roadway along the shoreline. This has been incorporated into the cost risk register.

1-2.5.7 Water control facilities

Water control facilities at this site are limited to drainage culverts. Although they are included in the cost estimate, specific designs have not been developed for these features. These features will be designed during PED.

1-2.5.8 Energy dissipating facilities

No energy dissipation facilities are proposed. (Not applicable.)

1-2.5.9 Erosion control requirements

Construction

The currently planned earthwork for this site does not require dredging or water-based equipment. Overwater work will be completed with excavators from channel banks or access pads. Since the restoration includes earthmoving in and around water channels, appropriate in-water sediment control measures will need to be used during construction. Any in-water or overwater construction should follow accepted best management practices for erosion control. Planning during PED should evaluate the best and most cost-effective methods for excavation of the channels and for bridge pier installation. These may include excavation at extreme low tides, installing silt curtains, or possibly using a containment structure for work in the dry.

With Project

No erosion control is anticipated outside of the construction boundaries since the goal is to reestablish natural erosion and sedimentation processes. Roadway embankments and any existing beach face protection should be monitored for signs of scour at an interval to be determined during PED. The shoreline area planned for restoration should be monitored as well.

1-2.5.10 Existing and post-project sedimentation

The blockage of the Duckabush Estuary with the existing causeway limits the distribution of sediments across the estuary. Geomorphic evidence of abandoned channels, mostly to the north of the current channel shows that the entire delta was part of the active channel migration zone (USBR 2004). Review of Lidar does not seem to indicate that much sediment is impounded upstream of the current roadway embankment aside from in the aggraded blocked distributary channels, implying that most of the sediment is carried out of the estuary in the single thread channel and distributed offshore.

After restoration, distributary channels in the estuary may shift or avulse as part of natural sedimentation patterns. More sediments may be retained in the intertidal zone. The amount and potential areas of flow changes and sedimentation will be addressed during PED.

1-2.5.11 Water control and order of work during construction

Construction should be sequenced so as to minimize traffic interruptions and to maximize excavation under dry conditions. For further considerations refer to Section 1-2.5.9.

1-2.5.12 Criteria for facility/utility relocations

For details on the utility relocations see Section 1-6.3.

1-2.5.13 Other facilities to meet project goals

Stormwater detention is discussed in Section 1-6. No other facilities are planned in order to meet project goals.

1-2.5.14 Instrumentation and monitoring

A combination of field surveys and aerial photographs will be used to document biological and physical changes to the landscape. Monitoring data can be used to refine adaptive management and corrective measures, as needed. Some of the key monitoring needs and opportunities are summarized in the FR/EIS in Section 6.6.

1-2.6 Coastal Studies

Coastal base flood elevations were calculated by FEMA as a part of the Jefferson County Flood Insurance Study. The base flood elevations are calculated by combining the effects astronomical tide (caused by gravitational effects of sun and moon), storm surge (rise in water levels as a result of wind stress and low atmospheric pressure), waves breaking onto the shoreline, produce an additional water level rise at the beach (wave setup), and waves running up the beach (wave runup). The 1% annual exceedance coastal base flood elevation is shown on Figure 1-2-4.

It is assumed that the Duckabush Estuary is only subjected to wind waves caused by local winds. Measurements at the nearby Bremerton airport (Figure 1-2-9) show that the maximum wind speeds come from the southerly direction and rarely exceed 30 miles per hour. The orientation of Hood Canal results in a very narrow band of fetch in the southwesterly direction of up to 20 miles, which could result in wave heights up to 5.0 feet with a period of 5 seconds. The impact of wind waves is generally limited to the outer portion of the estuary; however, this area should be designed to withstand this type of wind wave action. These issues will be addressed during PED. The influence of wind wave activity, storm surge and wave setup will be evaluated during PED.

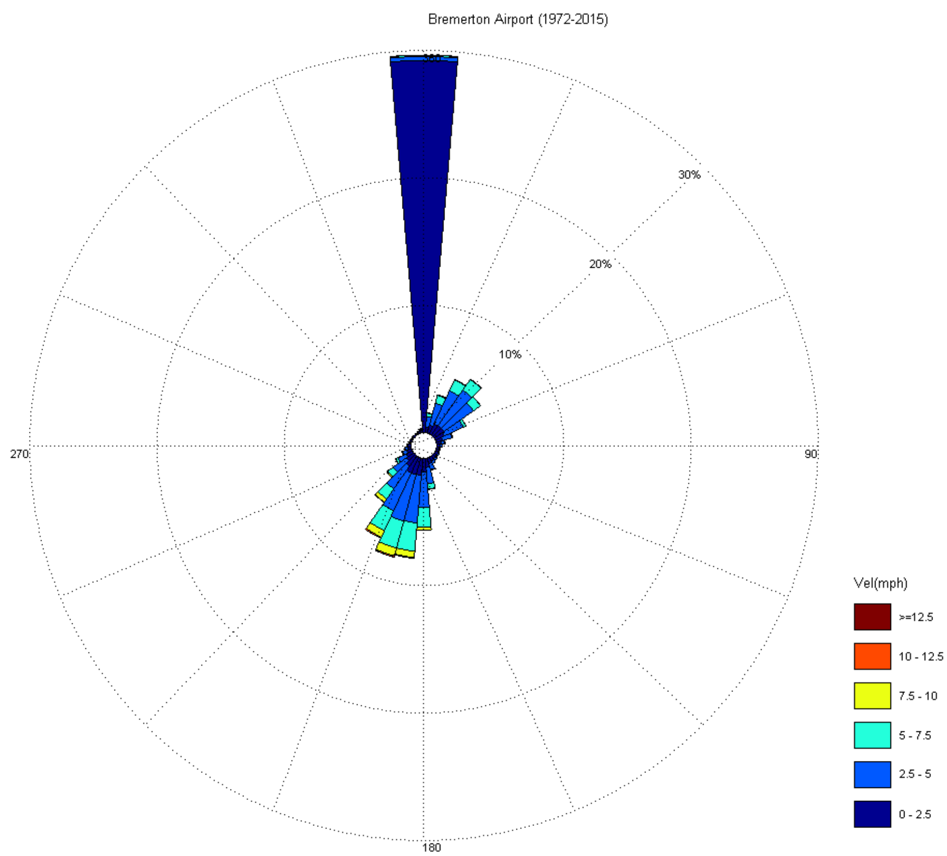

Figure 1-2-9. Wind Rose for Bremerton Airport.

Project plans formulated during the conceptual design phase for the Duckabush Estuary are based on a Mean Higher High Water tidal datum of 8.87 feet (NAVD88). This datum is from the tide gage at Seabeck (NOAA Gage 9445296). Major tidal datums are summarized in Table 1-2-4. The final design tidal datums will be reviewed and established during PED.

Table 1-2-4. Major tidal datums for The Duckabush site, Seabeck (Station 9445296)

Datum Description	Water Level (ft, NAVD88)
FEMA BFE (Coastal)	15.3
Mean Higher-High Water (MHHW)	8.87
Mean High Water (MHW)	7.92
Mean Tide level (MTL)	4.14
Mean Sea Level (MSL)	4.13
National Geodetic Vertical Datum of 1929 (NGVD29)	3.57
Mean Diurnal Tide Level (DTL)	3.12
Mean Low Water (MLW)	0.37
North American Vertical Datum of 1988 (NAVD88)	0
Mean Lower Low Water (MLLW)	-2.62

A summary table for the anticipated coastal studies at this site is presented in Section 1-21.

1-2.6.1 Design of coastal shore protection projects (ER 1110-2-1407)

This site does not include coastal shore protection. (Not applicable.)

1-2.6.2 Effects on adjacent shores

Downstream of the site, the shoreline transitions from a river delta to a bluffed-backed beach. The primary risk is an increase in sediment loading which could affect downstream intertidal and subtidal habitats in the river delta portion. At the bluff-backed beach, the primary forcing processes are coastal wind waves and longshore sediment transport which are expected to be minimally if at all affected by the restoration. The effects on downstream and intertidal habitat should be evaluated during PED, using results from similar inlets in Puget Sound.

1-2.7 Navigation Projects

There are no Federal navigation projects in the vicinity of the site. The Duckabush Estuary lies within a Regulated Navigation Area (RNA) subject to regulation 165.1328. This regulation applies to waters in the Hood Canal whenever any U.S. Navy submarine is operating in the Hood Canal and is being escorted by the Coast Guard. All persons and vessels located within the RNA shall follow all lawful orders and/or directions given to them by Coast Guard security escort personnel. Work at the site is not expected to affect operations in the RNA or other navigation in this area. The Bangor Submarine facility is located across Hood Canal over 8 miles away from the site and will not be affected by the project.

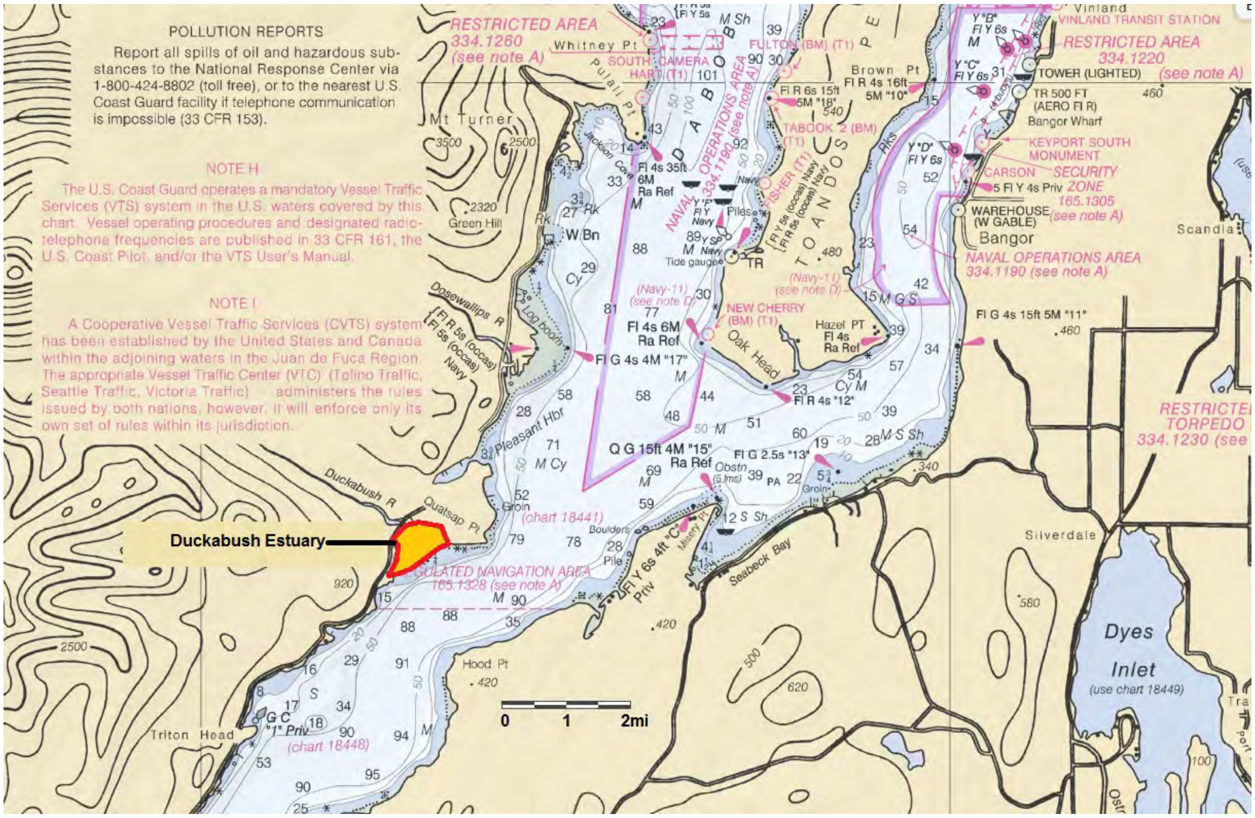

Figure 1- 2-10. Navigation chart for the vicinity of the Duckabush River Estuary (Source: NOAA RNC Online).

1-3 SURVEYING, MAPPING, AND OTHER GEOSPATIAL DATA REQUIREMENTS

This section describes surveying, mapping, and other geospatial data information to support preparation of the FR/EIS and the *Real Estate Plan* (Appendix C of FR/EIS). A brief outline of additional surveying and mapping required for subsequent design, plans and specifications, construction, and operations is also included.

1-3.1 Surveying, Mapping, and Other Geospatial Data Information Used

Geospatial data for the Duckabush River Estuary site were obtained primarily from remote sensing applications. No site-specific topographic, bathymetric, property, or utility surveys were conducted during the conceptual design phase. LiDAR, aerial imagery, and other geospatial data were used to delineate topographic features, determine surface elevations, and to estimate areas, volumes, lengths, and other dimensions of key features using CAD and/or ArcGIS. High-resolution LiDAR was obtained from the Puget Sound LiDAR Consortium (2005 LiDAR; 3m grid; State Plane projection in NAD83 [horizontal datum] and NAVD88 [vertical datum]; available at http://pugetsoundlidar.ess.washington.edu/lidardata/index.html). The Puget Sound Digital Elevation Model was used for combined bathymetry and topography of the Puget Sound lowland (Finlayson D.P., 2005; University of Washington; State Plane projection in NAD83 [horizontal datum] and NAVD88 [vertical datum]; available at http://www.ocean.washington.edu/data/pugetsound). Recent aerial photography (Aerials Express, 5/15/2009, 0.3m resolution, 2.45 m accuracy) was evaluated to determine recent site conditions. The conversion from Mean Lower Low Water (MLLW) to North American Vertical Datum (NAVD88) and to the NGVD29 datum was derived from the Seabeck tide gage (Station #9445296).

Information on land ownership was derived from the Washington Public Lands Database. Additional parcel data, including parcel boundaries, were obtained from the Jefferson County assessors' office (2015). Information on utilities, existing roadway geometry, and other site features was generally scaled off of aerial photographs because as-built drawings were not available. A site reconnaissance was performed in September 2010 in a prior study phase.

Designers consulted the Nearshore Geodatabase for additional site context. The Nearshore Geodatabase is available from the Washington State Geospatial Data Archive at: http://wagda.lib.washington.edu/data/geography/wa_state/#PSNERP. Metadata are provided in the *Geospatial Methodology Used in the PSNERP Comprehensive Change Analysis of Puget Sound* (Anchor QEA et al., 2009) (see Annex B). The geodatabase includes numerous datasets listed below:

- Shoreline
- Bathymetry
- Digital Elevation Model (DEM)
- LiDAR (terrestrial)
- Oblique aerial imagery (from the Washington Coastal Atlas)
- Hydrographic sheets
- Geology
- Slope stability
- Drift cells (net shore-drift)
- Streams
- Impervious surfaces
- Overwater structures
- Marinas
- Armoring
- Breakwaters/jetties
- Groins
- Levees
- Dams
- Nearshore fill
- Roads
- Railroads
- Land cover

Designers also consulted the University of Washington Puget Sound River History Project 19th Century Coast Survey Topographic Sheets (2009) for information on historical geomorphic conditions. Conceptual designs were intended to replicate historical conditions and remove stressors to nearshore processes to the extent practicable and feasible. As a result, these datasets informed the selection of restoration strategies and features. Designers created additional GIS data layers (point files, line files, and polygon files) to represent civil design features, such as areas of lowland excavation, to be depicted on the plan view drawings. Designers also created simple line drawings in CAD to represent typical sections and estimate quantity take-offs. Limited surface modeling was used to aid new roadway alignments and earthwork quantity take-offs.

1-3.1.1 Additional survey and mapping required

Substantial additional information will be required in PED to refine the design assumptions, confirm real estate requirements, and develop plans and specifications. Additional survey, mapping, and other geospatial data needs include the following:

- Property/Utility Survey – More detailed information on property boundaries and utilities will be needed to finalize the design and support real estate negotiations. The survey would also be useful in providing more accurate preliminary designs and quantities for excavations, utilities, and removal of existing features.
- Topographic/Planimetric/Bathymetric Survey – The conceptual design was based on LiDAR and aerial photos, which have inherent inaccuracies. Site-specific topographic, planimetric and bathymetric survey data will be needed to refine design of key elements, confirm that target elevations are appropriate for the desired ecosystem components (low marsh, etc.), and develop detailed construction and demolition plans. Survey data could also be used as a baseline for pre- and post-construction modeling, including hydrodynamic modeling. A temporary tide gauge may be required in the early design stages to obtain site-specific tidal statistics.

1-3.1.2 Timeline for incorporation of new mapping or other geospatial data

Planning, design, and implementation are expected to take several years. The site-specific surveys identified above are standard components of the design process and should be completed in the early stages of PED to ensure that the design work proceeds efficiently. Incorporating these data into the design process is not expected to delay the restoration.

1-4 GEOTECHNICAL

This section describes the geologic setting of the site, previous and recommended studies, and proposed geotechnical explorations relevant to design features.

1-4.1 Geotechnical Information

1-4.1.1 Regional and Site Geology

Regional geologic mapping from the State department of Natural Resources 2012, Geologic Map of the Brinnon 7.5-minute Quadrangle, Jefferson and Kitsap Counties, Washington, indicates site specific geologic features include sand, mud, pebbles, cobbles, and organic salt marsh deposits. This is part of the Marine deltaic alluvium (Qam). Bridge abutments are likely to be located on this deltaic alluvium, which has the potential for liquefaction. A section of the geologic map is shown below in Figure 1-4-1 and Figure 1-4-2.

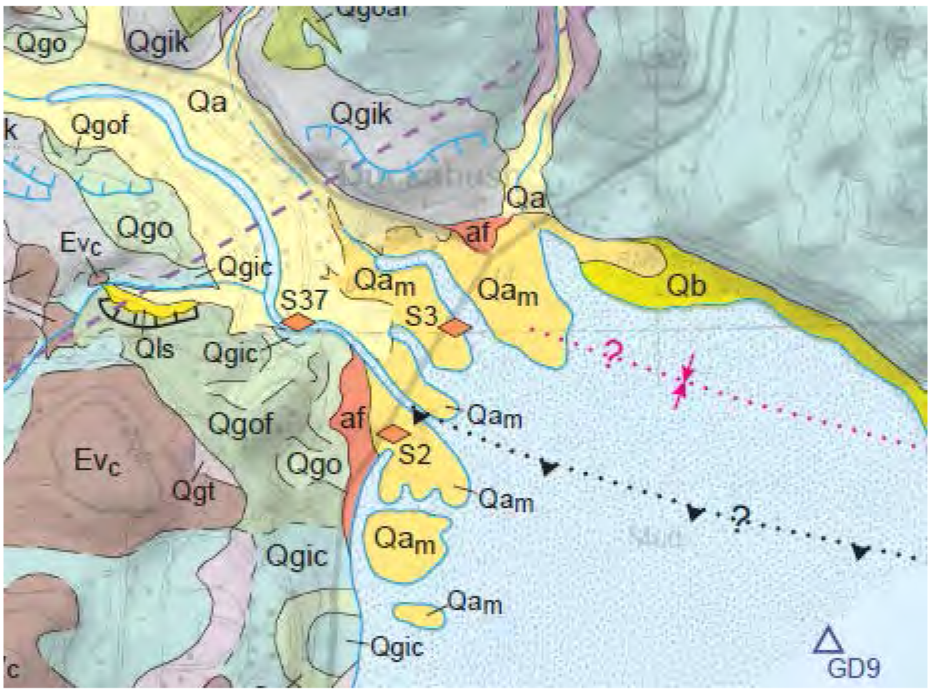

Figure 1-4-1. Geologic Map of Duckabush River Estuary.

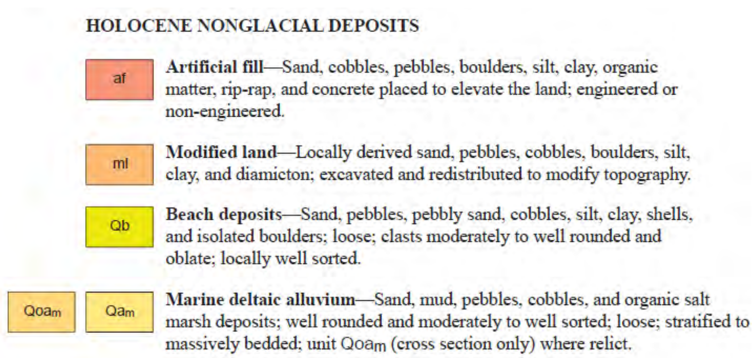

Figure 1-4-2. Soil type legend.

Near surface soils mapped in the project area by the United States Department of Agriculture (USDA) in the Soil Survey of Jefferson and Kitsap Counties, Washington is characterized by these soil types detailed in the figure above: Artificial Fill, Modified Land, and Marine Deltaic Alluvium.

According to the Washington Department of Ecology website there are three residential wells installed within 1300 feet of the existing bridges. One well is 350 feet northeast of the existing bridges, one well is 900 feet northeast, and one well is 1300 feet northeast. The three wells vary from 39-232 feet in depth and were drilled in 1994, 1999, and 1999. Typical profile has topsoil from 0-2 feet, sand to sandy gravel with clay from 2-20 feet, and clay with varying amounts of gravels and cobbles from 20 feet to bottom of hole, with basalt bedrock being encountered in one hole at 230 feet.

1-4.1.2 Completed explorations

At this time no subsurface explorations have been completed for this project. All subsurface information is based on research of soil surveys, geologic mapping, and wells logs available from the Department of Ecology. See Section 1-4.3 for the proposed subsurface exploration plan.

1-4.1.3 Selection of preliminary design parameters

Based upon research of the soils and geology in the project vicinity it is anticipated that subsurface soils will consist mostly of clay with varying amounts of sands, gravels, and cobbles. Preliminary design parameters have been selected for various soil descriptions which are likely to be observed at the proposed bridge foundation locations. Table 1-4-1. Preliminary design parameters.below provides a range of preliminary design values for the anticipated soils in the foundation.

Table 1-4-1. Preliminary design parameters.

Soil Description	Depth Range	Unit Weight, γ (pcf)	Friction angle, ϕ’	Undrained Shear Strength, S (ksf)
Clay to sandy clay with gravel and cobbles	0’ – 200’+	90-115	-	2.0

Groundwater table was assumed at the ground surface. Bedrock is not likely to be encountered within upper 100 feet.

1-4.1.4 Geophysical investigations

No geophysical investigations have been conducted. It is recommended that there be shear wave velocity measurements, such as a seismic refraction survey to define the site class since the geologic map shows loose materials and organic salt marsh deposits. There is also the potential for liquefiable soils around the site.

1-4.1.5 Groundwater studies

No groundwater studies have been conducted for geotechnical design. Groundwater elevation is dependent on flows from the Duckabush River and the water surface elevation of Puget Sound. For geotechnical design purposes the groundwater will be assumed at the ground surface when considering the bridge foundations.

1-4.1.6 Recommended instrumentation

No instrumentation is recommended for this site.

1-4.1.7 Earthquake studies

In accordance with Table 20.3-1 of the 2010 ASCE 7, a Site Class C or D (NEHRP BC) is preliminarily recommended for this site when considering the average of the upper 100 feet. According to the 2008 United States Geological Survey (USGS) Earthquake Hazards website https://geohazards.usgs.gov/deaggint/2008/, the Peak Ground Acceleration (PGA) predicted for the site is 0.428 g, and the maximum considered earthquake (MCE) ground motions for the site are S_s=0.946 g and S_1=0.335 g. In accordance with Tables 11.4-1 and 11.4-2, Site Coefficients F_a and F_v are 1.1 and 1.7, respectively for a Site Class D. Therefore the adjusted MCE ground motions are S_{MS}=1.041 g and S_{M1}=0.570 g. Therefore, design spectral acceleration parameters are S_{DS} = 0.694 g and S_{D1} = 0.380 g. The return interval for these ground motions is 5 percent probability of exceedance in 50 years (975 years). See Figure 1-4-3 below for earthquake deaggregation output.

Seismic design for deep foundations and bridge abutments shall be performed in accordance with the WSDOT requirements and the AASHTO LRFD Seismic Design Specifications. (AASTHO specifies 7% in 75 years, which is comparable to USGS 5% in 50 years.

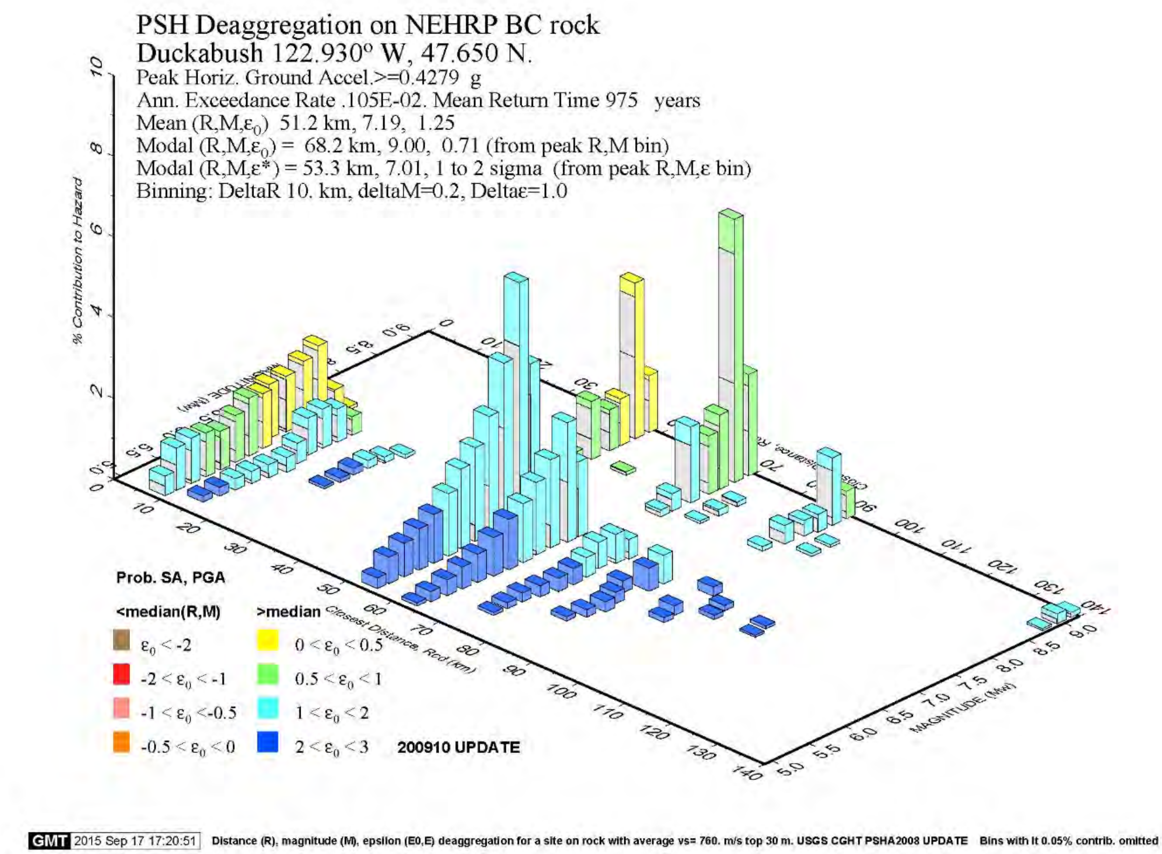

Figure 1-4-3. Deaggregation plot for Duckabush

1-4.1.8 Preliminary engineering analysis

There are two existing bridges on the site that will be replaced and removed. The existing bridges were built in the 1930's. The first is a concrete tied arch bridge with a span of 110 feet supported by concrete piers. The second bridge has 4 spans at 30 feet apart and is supported by concrete piles at the end of each span. At both bridges, the piles are at an embedment depth sufficient to develop the minimum HS-15 (the highway rating at the time) load capacity.

The proposed 2100-foot long multi-span concrete I-girder bridge that will replace these two bridges will be supported by deep foundations. The foundation design assumes two, 7-foot diameter drilled shafts at 15-foot spacing (inside edge to inside edge) with a 135-foot embedment depth at the end of each span.

Drilled shafts or driven piles are acceptable foundation alternatives for the proposed bridge. To minimize negative effects to protected species, pile driving is not a preferred option due to the loudness and vibration of pile hammer impacts. In addition, if rock or till is encountered at a shallow depth under the bridge abutments, drilled shafts socketed in rock will be the preferred alternative as driving piles at such subsurface conditions would be difficult. Shallow foundations are not a preferred option at this time due to potential seismic loading and scour.

A preliminary estimate of foundation capacity using the lower range of the parameters in Table 1-4-1 was used as a verification of the conceptual foundation design from the 10% design. See Table 1-4-2 for results of the estimate.

Table 1-4-2. Preliminary foundation axial capacity estimate.

Feature	Description	
Bridge	Total length (feet)	2,100
	# of spans x Approx. span length (feet)	18 x 120
	Approximate width (feet)	34
	Dead load x 1.25 [LRFD strength I] (kips) / pier[1]	1,525
	Live load x 1.75 [LRFD strength I] (kips) / pier[2]	395
Foundation	Type	Drilled shaft
	Diameter (inch)	84
	# piles / pier	2
	Depth (feet)[3]	135
Load	Estimated static loading demand (kips)	960
Capacity	Factored pile resistance (kips)	1,125
	Sufficient capacity	OK

[1] Dead load estimate is based on bridge dimensions in the 10% design report
[2] Live load estimate is based on HL-93 (HS-20 Truck + 0.64k/ft lane).
[3] A 20% contingency was added onto the depth calculated.

Each end of the bridge will require an earthen abutment to tie into. If there is liquefaction potential found in the site soils, the abutments will need to be supported by 4 or more shafts that extend down to a stable depth, preliminarily set at 135 feet below ground surface.The foundation capacity estimate is preliminary without any site specific subsurface information. Upon completion of subsurface explorations, foundations should be designed using encountered subsurface conditions. Downdrag on the drilled shafts shall also be analyzed. At this time the foundation design includes the use of drilled shafts at a depth of 135 feet. It should be noted that if liquefaction potential soils in the foundation are present, the depth of the drilled shafts may increase. Likewise, if bedrock or dense or hard soil is encountered the depth of the drilled shafts will significantly decrease. Seismic loading, liquefaction potential, and scour are not included in the current conceptual level design.

Slope stability analysis has not been evaluated at this time. Slope stability and settlement analysis for the entire length of the approach embankments shall be performed upon completion of the design and geometrical configuration of the bridge. Ground improvements may be required at the bridge abutments/roadway approaches if liquefiable soils are encountered.

1-4.1.9 Excavatability analysis

Excavation of the existing channel, bridge abutments and approach embankments will be required to meet the proposed ecosystem restoration. The amount of riprap to be used to protect the piers of the existing bridges is not known. It is estimated that approximately 500 CY of rock armor (12-24 inch riprap) has been placed to protect the piers and abutments from scour. No explorations or construction records were located for the new embankment site therefore the embankment material is unknown. Based on soil and geology maps in the area and historic project as-builts, as well as explorations from the Department of Ecology, it may be assumed at this time that the existing embankments consist of a combination of clay with gravel, sand, and cobbles throughout. Excavation of riprap and fill may be accomplished using an excavator.

The anticipated amount of rock could be small and rock excavating equipment may be necessary with the unlikely need for blasting. According to construction records for the existing bridges, bedrock was not encountered during construction and is not likely to be encountered during construction of the new bridge.

1-4.1.10 Anticipated construction techniques and limitations

The existing bridges can remain open to traffic during construction of the new bridge. Some rerouting and traffic control may be necessary when connecting the new roadway to the existing roadways.

Demolition of the existing bridge and embankments will likely require a crane and excavator. The existing bridge superstructure and piles will be removed by crane. The concrete piles will likely be demolished to the ground surface if deemed acceptable. Removal of timber piles is typically accomplished by cutting or breaking them at the ground line. In a sensitive marine environment, more careful excavation around each pile and cutting a certain distance, a foot or more below ground line, may be required. One of the existing bridges is considered a historical structure and may need to have preservation efforts in place before removal. The existing fill embankments and armoring will be removed by excavator.

Land-based drilling augers will be used to install the deep foundation at the bridge abutments and at each pier. At this time, work is anticipated to require land-based large augers, excavators, cranes, concrete trucks, and dump trucks.

The type of deep foundation will be confirmed during PED once subsurface explorations have been completed. At this time it is assumed drilled shafts will be used to support the proposed vehicle bridge. Due to the presence of soft and caving soils and anticipated high groundwater, either casing or wet method is recommended for construction of drilled shafts. Upon completion of the shaft excavation, the hole is cleaned and the reinforcing steel cage is placed to the bottom of the hole. The casing is then carefully extracted fully or partially leaving a top segment to facilitate column installation and concrete is cast. Once the shafts are installed, the columns are cast, pilecaps and bridge superstructure are constructed.

There are potentially several utilities at this site which will need to be relocated. Utility configuration and design will be determined through coordination with service providers.

See Civil Section 1-21 for additional construction notes.

1-4.1.11 Potential borrow sources and disposal sites

No borrow sites or disposal sites have been identified within the project extents. Approximately 21,300 CY of borrow will be required for the project. Borrow/fill for the roadway transitions will likely come from a local quarry. Over 41,900 CY of material will require disposal. Offsite disposal and borrow sites are available within a 60 mile distance from the site, either to the North at Port Angeles, WA, or to the South at Tumwater, WA. Borrow and disposal sites shall be confirm during PED. The uncertainties associated with confirming suitable borrow and disposal sites have been captured in the cost risk register.

1-4.1.12 Potential sources of concrete and materials

Preliminary investigations indicate that there are several options for receiving concrete materials. This is a major highway and delivery to the site will be relatively easy.

Suitability of concrete and materials will be evaluated at later stages of design or during construction.

1-4.1.13 Suitability of concrete and materials

Suitability of concrete and materials will be evaluated during PED.

1-4.2 Additional Studies and Analysis

Additional studies and analysis to be completed during preliminary engineering design (PED) or subsequent phases of design at a minimum include the following:

- Geotechnical Investigation: subsurface explorations, testing, and field reconnaissance
- Foundation Design: static and seismic analysis according to AASHTO LRFD
- Abutment Stability (include potential for liquefaction and ground improvement)
- Pavement Design: new roadways and approaches (include traffic analysis for ESALs)
- Scour Study: at roadway embankments, abutments, and bridge piers

1-4.3 Additional Explorations and Testing

The proposed subsurface exploration plan consists of drilling borings at the proposed bridge abutments, piers and approach embankments. In addition, test pits should be conducted along the roadway transitions for at grade construction and pavement design. For the roadways test pits shall be spaced approximately every 250-500 feet. Based on research of the site and preliminary foundation design the bridge borings should be a minimum of 135 feet below the ground surface and test pits a minimum of 10 feet. The preferred exploration method for the borings is mud rotary. Test pits shall be accomplished with a backhoe or small excavator.

Sampling in the soil borings shall be accomplished using standard penetration test (SPT) with samples taken typically every 2.5' for the top 10 feet and every 5' for the remained of the boring depth. Proposed soil lab testing shall comprise shear strength testing of undisturbed cohesive soil samples, moisture content, grain size analysis, and percent finer than #200 sieve. Atterberg limits and consolidation tests are recommended for cohesive soils, and unconfined compressive strength test for rock cores if they are encountered.

1-4.4 Laboratory-testing Program and Evaluations

No laboratory testing or evaluation of materials has been completed at this time. Testing to be completed during PED is outlined in Section 1-4.2.

1-5 ENVIRONMENTAL ENGINEERING

This section describes environmental engineering factors relevant to the proposed design features.

1-5.1 Use of Environmentally Renewable Materials

At this design stage, use of environmentally renewable materials is not planned. However, if renewable materials are available they could be incorporated into the design. Specific details will be developed during subsequent design stages.

1-5.2 Design of Positive Environmental Attributes into the Project

The Duckabush River Estuary site was selected to address River Delta restoration objectives to restore freshwater input and tidal processes where major river floodplains meet marine waters. The proposed action involves removing the substantial fill for Highway 101 and the two bridges from the intertidal zone to allow restoration of the natural processes of freshwater input, sediment accretion and erosion, and distributary channel formation. This action will restore habitat for threatened salmonid species, estuarine and saltwater marsh, and estuarine biodiversity.

1-5.3 Inclusion of Environmentally Beneficial Operations and Management for the Project

Design and construction will incorporate sustainable and ISO 14000 compliant practices. The U.S. Army Corps of Engineers (USACE) Environmental Operating Principles (EOPs) are designed to provide direction on achieving better stewardship of air, water, and land resources while showing the connection between managing those resources and protecting environmental health. The EOPs are to ensure that USACE actions consider the environment and are sustainable now and in the future.

1-5.4 Beneficial Uses of Spoil or Other Project Refuse during Construction and Operation

Beneficial uses of spoil or other refuse are possible. If spoils or other refuse materials are available for reuse, they could be incorporated into the design. Specific details will be developed during PED. The FR/EIS describes measures to minimize energy consumption for the purpose of reducing greenhouse gas emissions. These measures would also serve to minimize energy consumption.

1-5.5 Energy Savings Features of the Design

At this design stage, energy savings features have not been incorporated. In accordance with the EOPs, energy savings features will be a component of the design to the maximum extent practicable.

1-5.6 Maintenance of the Ecological Continuity in the Project with the Surrounding Area and Within the Region

The restoration will increase ecological continuity within the site and with the surrounding area. This is one of several sites designed to restore the productivity and increase interconnectivity of the Puget Sound ecosystem.

1-5.7 Consideration of Indirect Environmental Costs and Benefits

All direct, indirect and cumulative environmental costs and benefits were evaluated during the environmental impact assessment and alternatives analysis recorded in the Final FR/EIS

1-5.8 Integration of Environmental Sensitivity into All Aspects of the Project

Construction will be conducted to ensure no long term deleterious impacts to the ecosystem will occur. Best management practices will be incorporated into the contract documents. Most management practices will cover erosion and sediment control, stormwater management, spill response and hazardous material management, trash and debris management, air emissions from construction vehicles, and noise standards.

1-5.9 Use of Environmental Review Guide for Operations (ERGO) with Respect to Potential Future Environmental Problems

This is not a USACE operating facility. (Not applicable.)

1-5.10 Incorporation of Environmental Compliance Measures into the Project Design

All applicable laws and regulations will be followed during design and construction in accordance with the USACE contract documentation.

1-6 CIVIL DESIGN

This section discusses the key elements of the civil design including the selection of the site and evaluation of alternative layouts, alignments, and components.

1-6.1 Site Selection and Project Development

The Duckabush River opens to a moderately wide steep-walled valley within the action area. The river is contained within a single channel through the site before emptying into the marsh and submerged marsh outboard of the site. The historic northern arm of the river has been blocked, is aggraded, and is a dead-end channel in the middle portion of the site. Both channels are tidally influenced and pass under bridge crossings. Training berms are in place at the southern arm, just upstream of the Highway 101 crossing, to control lateral movement of the channel. The northern channel branches to form smaller dead-end channels upstream of Highway 101, and receives freshwater flow from a connection to the small tributary that crosses Shorewood Road.

Pierce Slough, located at the northwest corner of the site, is partially disconnected from tidal flows by the culverted Highway 101 crossing. A remnant tidal channel network exists outboard of the highway between the north and south channels. The northern tidal channel network appears to have aggraded over time, though it is partially present today.

The proposed action would restore the natural geomorphology to the Duckabush River delta wetlands by removing major roadway obstructions, excavating channels, and removing fill. The action would realign Highway 101 across the estuarine delta to restore tidal connection to the estuary. A surface street crossing (Shorewood Road) would be modified and adjacent fill at a distributary channel (Pierce Slough) would be removed. Multiple tidally influenced distributary river channels would be reestablished, and blind tidal channels would be excavated within the marsh areas.

Table 1- 6-1-summarizes the key design elements associated with the proposed restoration. Annex 1-1 contains exhibits that depict the proposed restoration design elements.

Table 1- 6-1. Key Design Elements

Item	Description of Item	Approximate Quantity
Roadway Removal	Remove 3,800 LF of Highway 101 embankment including removing several culverts, approximately 900 feet of Duckabush Road, and 150 feet of Shorewood Road and culvert. The current highway roadway extends down to about the MHHW line (MHHW is 11.5 feet above MLLW).	4,850 LF
New Roadway	Build 1,900 feet of new highway including 1 drainage culvert replacement, 800 feet of Duckabush Road, and 80 feet of new Shorewood Road. New highway roadway elevation will be at about 28.5 feet above MLLW. About 1000 feet of the revised highway embankment may extend below the MHHW. Duckabush road and Shorewood road will remain at their current elevations.	2,780 LF
Bridge Removal	Remove two existing Highway 101 bridges. Bridge decks at 22.5 feet above MLLW.	970 LF
New Highway 101 Bridge	Build one 2,100-foot bridge at 28.5 feet above MLLW (18 spans at ~E120 feet). In the next phase of design, the span length and number of piers in the bridge design will be refined to maximize environmental benefits by holding the total number of piers to the minimum required for structural safety, adherence to AASHTO specifications and optimization of costs. Build 60-foot bridge approach at Duckabush Road.	2,100 LF
New Shorewood Road Bridge	70-foot bridge at Shorewood Road. Shorewood Road elevation 13.8 feet above MLLW.	70 LF
Overhead Power	Relocate to new alignment	
Distributary Channels (large)	675 feet of north channel connection to the mainstem of the Duckabush River (12.5 to 2.5 feet above MLLW) and 480 feet of south channel connection to mainstem (12.5 to 6.5 feet above MLLW). Maximum channel depth: 9 to 10 feet	1,155 LF
Distributary Channels (small)	1,900 feet of Pierce Slough Reconstruction (12.5 to 6.5 feet above MLLW); 2,300 feet of other tidal channels 12-6 feet above MLLW). Maximum channel depth: 5.5 feet.	4,200 LF
Fill Removal	Remove training berms along river (0.7 acre) (0-18 feet above MLLW), road embankment and roads (3.3 acres), and developed areas (2.5 acres)(all > 11.5 feet above MLLW)	6.5 acres

1-6.1.1 Basis of Design

The Highway 101 roadway cuts across the intertidal river delta and estuary wetland complex, severely affecting water flow, sediment transport, and morphology. Removal of this stressor will restore and improve physical and ecologic processes. Reconnection of the north distributary channel would improve estuary processes by restoring delivery of fresh water and fluvial sediment. Bridging of existing surface streets (Duckabush River Road, Shorewood Road) would reconnect freshwater and tidal flows to remnant distributary, tidally influenced channels and tributary wetlands. Removal of training berms along the active river channel would reconnect the river to its intertidal floodplain and wetlands, restoring floodplain and estuary wetland processes, and increasing channel density.

Removing these multiple stressors would restore dynamics and promote greater diversity of delta wetland habitats. Together the Highway 101 bridges and roadway embankment occupy a span of over 1700 lineal feet across the mouth of the estuary. The bridge span of 2,100 feet is the result of feasibility level engineering of the conceptual design to meet the required ecosystem outputs. There are several reasons why this bridge length was chosen.

- The design of a bridge built to modern standards requires raising the low chord of the bridge about 5 feet to allow for the base flood, clearance for debris and an assumption of about 2 feet for an intermediate rate of sea level rise in the next 100 years (see Section 1-2.1.9). The conceptual design did not take into account the need to raise the bridge.
- Raising the bridge and roadway by 5 feet would require significant additional fill in the estuary for all but the estuary spanning bridge length. That is, benefits would actually be negative until the bridge reaches about 1700 feet.
- Since the Duckabush estuary is in a steep-walled valley, connecting the new bridge with the existing roadway within DOT guidelines for vertical and sag curves requires either large amounts of roadway fill or an extension of the bridge to 2100 feet. While it may be possible to use a somewhat shorter bridge length, it is not clear that there would be a significant cost savings so this optimization is deferred to PED.

During PED phase the PDT will examine trade-offs and value engineer the recommended alternative to minimize costs while optimizing desired benefits. This will be accomplished by detailed engineering once the site investigations and surveys recommended for PED have been completed and base flood elevation has been confirmed by modeling. The proposed action includes removal of approximately 3,800 LF of Highway 101 including two existing bridges. The highway will be replaced with an elevated structure, with new roadway approaches at either end of the bridge. Duckabush Road will be realigned both horizontally and vertically to connect to the new highway bridge. An interchange will be constructed at Duckabush Road to allow tidal exchange with the marsh to the northwest. The overhead power lines that currently run parallel to the existing roadway will be relocated to the new structure. New roadway embankments will be required at the north and south approaches to the new Highway 101 Bridge. A new bridge will be constructed at Shorewood Road and a culvert will be removed to make room for a restored (widened) distributary channel (Pierce Slough). Highway 101 modifications are based on WSDOT 2010, Design Manual M, Modified Design Level. Table 1- 6-2. Purpose and usage of transportation features.summarizes the rationale for modification of the roads and bridges at the Duckabush site.

Table 1- 6-2. Purpose and usage of transportation features.

Element	Element Purpose	Road/Bridge Use	Emergency use (Y/N)
• Remove 3800 ft of Highway 101 • Remove 2 Hwy 101 bridges • Build one 2100-foot bridge (18 spans at 120 feet) • Build 1900 feet of new roadway	Remove principal stressor of Duckabush Estuary Allow free estuary development and migration by use of piers and bridge location further upstream Minimize maintenance by use of piers	Interstate Highway [2200 vehicles per day (2014)] only means of travel N-S on Eastern side of Olympic Peninsula	Yes
• Remove 900 ft of Duckabush Road • Build 60-foot bridge approach and entrance/exit for Highway 101 • Build 800 feet of Duckabush Road	Allow activation/passage of Pierce Slough Allow connection from Duckabush Valley to Highway 101	Secondary road Only route of ingress-egress to/from Duckabush river valley, (Population ~ 350) Junction with Highway 101.	Yes
• Remove 150 of Shorewood Road • Build 70-foot bridge at Shorewood Road • Build 80 feet of Shorewood Road	Remove culvert, allow reactivation and passage of Pierce Slough	Secondary road Access road for Duckabush Fire station	Yes

Stormwater runoff from the project is required to meet Federal Energy Independence Security Act (EISA), State, Washington State Department of Ecology and local requirements. EISA requires sites in excess of 5000 square feet to retain the total volume of rainfall from the 95th percentile of the 24-hour storm on site. The alternative is to require the post development hydrology to not exceed the pre-development hydrology (prior to man) by using site specific stormwater BMP's such as infiltration, evapotranspiration and detention. WSDOE requires that the duration of the developed storm flow be less than 50% of the 2 year through the 50 year events. The WDOE water quality provisions require that treatment facilities be designed for the 24- hour storm with a 6-month return frequency or a simulated daily volume that accounts for 91% of the entire runoff volume over a multi-decade period of record.

For feasibility design, a provision for stormwater is included in the estimate. Once the site hydrology is confirmed in PED, stormwater detention will be designed where required. The risk of the assumption for stormwater treatment size has been captured in the Cost and Schedule Risk Analysis.

Distributary channels will be excavated at or near their historic configurations. Large and small distributary channels will be excavated to thalweg elevations of 2.6 feet and 6.6 feet MLLW (0 feet and 4 feet NAVD88), respectively, and top widths of 50 feet. These channels are sized based on historic data, primarily the drawings for the existing Highway 101 (Washington State 1933), and interpretation that the datum was close to MSL, which is about 6.8 feet MLLW (4.1 feet NAVD88). Historic maps, LiDAR, and aerial photographs were used to locate the excavations.

Two large distributary channels will be created, connecting to the existing Duckabush River mainstem on the north side, and ending at the remnant channel in the middle of the delta. Four small distributary channels would be excavated, and two existing channel connections expanded farther toward Hood Canal. One of the new small distributary channels would reestablish Pierce Slough at or near its historic alignment.

The road embankment will be excavated and removed to match the adjacent grade on the downstream side, which is at elevation 11.6 to 12.6 feet MLLW (9 to 10 feet NAVD88). Six areas of about 2 to 6 acres each will have channels excavated to establish natural intertidal marsh morphology (Exhibit A). The channel cross sections have been sized using the Applied Geomorphology Guidelines (Attachment B). The channels include-third-order cross sections, connected to the distributary channels.

The 1883 topographic sheet (T-sheet) shows two distributary channels at the outlet of the Duckabush River (Figure 1-6-1). The banks of the river transition from mixed forest and grassland upstream, to salt marsh and submerged marsh at the mouth of the estuary. Some settlement had occurred at the time of the survey, with an orchard visible on the north side of the northern channel and a crossing on the mainstem of the river upstream of the distributary channels. A developed tidal channel network was present in the outboard marsh between the two main channels and north of the northern channel. A single road is shown to access the settlement from the north at the northern edge of the marsh, cross a tidal slough (Pierce Slough), and then cross the mainstem of the river upriver from the more recent alignments (existing and previous crossings are farther east).

Figure 1-6-1. Historic Map (T-Sheet) and River History Project Data

1-6.1.2 Constructability

A 2,100 foot long x 30 foot wide x 4 feet high temporary construction platform adjacent to the bridge alignment, composed of granular fill over a geotextile fabric is planned. This assumes the existing ground is at elevation 8 feet NAVD88 and the platform top is at elevation 12 feet (MHHW is 8.9 feet). The embankment would not span any waterway so the estimate is conservative in length. This extra material will be used for access and for a 40 foot x 60 foot x 4 feet high work pad at the pier locations.

The 30 foot wide top width should provide enough room for a crane and room to work around. This would look similar to Section C on Exhibit B.

The construction and removal of the platform will cause temporary disturbance to fish and wildlife though increased noise and turbidity. These impacts will be minimized by work during designated in-water work windows when sensitive species are least likely to be present, implementing BMPs, and monitoring water quality during in-water work. The entire temporary platform would be removed and the site restored on completion. This would include uncompacting 3 feet of soil that lies below the embankment, and supplementing as needed to raise the elevation to the adjacent ground on all sides. The stripped material will be stockpiled and blended with the final topsoil layer, utilizing the native seeds in the native material. Any vegetation that is disturbed would be replanted post construction.

No significant water diversions are anticipated with this work.

The distributary channel excavation cross sections have 2:1 slopes (Exhibit B). This is a steep slope, especially for the two large distributary channels that require excavation to approximately 10 feet below existing grade. It is assumed that a track-mounted crane with a clamshell (or dragline) bucket could accomplish the excavation in the wet, and that slopes would be stable during construction. Further consideration of constructability is recommended, including subsurface exploration of soil properties and a geotechnical report. Also, sloughing of the banks should be expected, and the limits of excavation should be evaluated to prevent impacts on adjacent areas. Excavation would be required at a few locations that are well within the delta and away from established access roads. Construction of these areas would need to occur before distributary channels are excavated through the delta cone deposits. While marine equipment could be used, the complexity of offloading the excavated sediment and taking it upland would add costs; therefore, the use of marine equipment is not considered practical. Access to construct the distributary channels would attempt to minimize disturbance of the area and the access routes would be restored on completion. The access routes will be evaluated during PED.

Construction sequencing should keep the fire station and Duckabush road open for 24/7 access. This sequencing will be further evaluated during PED. During PED phase the PDT will examine trade-offs and engineer the recommended alternative to minimize costs while optimizing desired benefits. This will be accomplished once the site investigations and surveys recommended for PED have been completed.

1-6.2 Real Estate

Real estate assumptions, valuations, and planning documents have been appropriately scaled for the current level of design. As additional surveys, modeling, and design are completed during PED, the real estate documentation will be modified accordingly. For the current real estate status, refer to the Final FR/EIS, Appendix C, *Real Estate Plan*.

1-6.3 Relocations

The overhead power, telephone, and telecommunications lines (approximately 3,200 feet) that currently pass along the existing causeway would be relocated to the new bridge and roadway alignment. Additionally, an overhead electric/phone line approximately 550 feet long runs from Highway 101 near the northern channel to the fire station.

Table 1-6-1. Facility / Utility Relocations

Facility / Utility	Activity	Subsequent Design
Overhead power distribution and transmission lines	Relocate from existing causeway to new bridge and roadway alignment	Coordinate with utility owner on phasing of work.
Telephone and telecommunications lines	Determine locations and relocate if applicable	Coordinate with utility owner on phasing of work.
Gas lines	Not Applicable	Not Applicable
Sanitary sewer septic systems	Determine locations and assess removal or relocation if applicable	Need for decommissioning analyzed during PED.
Water wells	Determine locations and assess protection or removal if applicable.	Need for decommissioning analyzed during PED.

1-7 STRUCTURAL REQUIREMENTS

This section discusses the structural elements of the proposed restoration including preliminary design requirements and criteria for bridges or roads, a description of major structures and construction considerations, and recommended analyses.

1-7.1 Functional Design Requirements and Technical Design Criteria

Functional Requirements: The new Highway 101 Bridge is designed to support two lanes of traffic in addition to shoulder spaces. The design accommodates horizontal and vertical hydraulic clearances needed to achieve the restoration goals at this site. The design is intended to be low maintenance and should meet the AASHTO Bridge design specifications as well as the WSDOT Bridge Design Manual requirements. The bridge is designed for a minimum service life of 75 years. In addition to the main stretch of the Highway 101 Bridge, a short bridge that connects the Duckabush Road to the main alignment is needed. The Duckabush Approach Ramp bridge structure will be constructed of the same design as the main alignment and it will be physically isolated from the main alignment by a separation joint.

Key design elements are summarized in Table 1-7-1 below.

Table 1-7-1. Summary of Bridge Information

Bridge Location	Description
New Highway 101 Bridge alignment	• New 2,100-foot long curved bridge, 32-feet wide; • (18) spans no greater than 120-feet each span; • (5) lines of standard WASDOT WF50G girders at each span; • (17) cast-in-place concrete pile caps, 5-foot deep by 6-foot wide by 32 feet long; • (2) concrete columns, 4-ft in diameter, at each pilecap; • (1) 7-foot drilled in concrete shaft at each column; • (1) Abutment at each end with four drilled-in shafts, 7-foot diameter. (see Geotechnical for shaft depths)
Approach to the new bridge alignment at Duckabush Road	• New 34-foot approach section similar in construction with the main bridge; • (1) span, 35-ft long; • (5) lines of standard WASDOT WF50G girders at the span; • (1) cast-in-place concrete pile caps, 5-foot deep by 6-foot wide by 50 feet long; • (5) concrete columns, 4-ft in diameter, at the pilecap; • (1) 7-foot drilled in concrete shaft at each column (see Geotechnical for shaft depth); • (1) Abutment with four drilled-in shafts, 7-foot diameter.

Design Criteria, Seismic: The bridge design shall include earthquake resisting systems (ERS) corresponding to the requirements of a Seismic Design Category (SDC) of C or D which is typical for the Puget Sound region. Determination of SDC is based on the parameters identified in the geotechnical content of this document (Section 1-4). Category D is more stringent of the two categories and a more complex analysis and detailing for structures subjected to this category will be required during PED. Category D requirements can lead to an increase in both the design and construction of the bridge.

On October 26, 2015 a State of Washington DoT State bridge and structures engineer indicated that "…the SR 101 corridor at Duckabush has the same life-safety requirements as bridges classified as "ordinary bridges' by AASHTO specifications." The required seismic performance objective for this bridge is therefore Life Safety (WSDOT Bridge design Manual Section 4.1). AASHTO Seismic Design guide specifications are intended for conventional bridges designed for the life safety performance objective considering a seismic hazard corresponding to a 7% probability of being exceeded in 75 years. This performance objective corresponds with a low probability of bridge collapse in a 1000-year event but the bridge may suffer significant damage, and significant disruption is possible. Partial or complete replacement of the bridge may be required.

For SD1 of 0.38g (See geotechnical), and based on AASHTO Table 3.10.6-1, the seismic zone for this bridge is zone 3. And according to AASHTO tables 4.7.4.3.1-2 this bridge does not qualify as a "Regular Bridge" due to its total length and localized spans and bridge curvature. For multi-span bridges that are not regular and fall in AASHTO seismic zone 3, a Multi Mode (MM) elastic seismic analysis is required (AASHTO Table 4.7.4.3.1-1). The MM elastic analysis will be more sophisticated and it may noticeably contribute to the overall cost of bridge design and construction.

Design Criteria, Gravity: The Bridge will be designed for loads from HL-93 (HS-20 Truck + 0.64k/ft lane) truck and its own self weight in accordance with AASHTO loading criteria.

Design Criteria, Hydraulic: A minimum distance of 3 feet from the Base Flood Elevation (BFE) to the bottom of the lowest point of bridge girders is selected in order to provide adequate clearance for debris in a BFE event. With the concept design bridge girder depth and deck thickness, the bridge deck should be at an elevation of about 26 feet with respect to the NAVD 88 datum. Subsequent design will evaluate the minimum clearance requirements for this site for large debris taking into account future sea level change projections identified in Section 1-2.1.9.

Summary of technical criteria / requirement is tabulated in Table 1- 7-2. The latest and most current edition of the specified criteria will apply at the time of final design.

Table 1- 7-2. Technical Requirements.

Item	Description
Design Specifications	• WSDOT Bridge Design Manual • AASHTO LRFD Seismic Bridge Design • AASHTO LRFD Bridge Design Specifications
Load Criteria	• Live Load: HL-93 (HS-20 Truck + 0.64k/ft lane), 1.3 Impact Factor • Pedestrian (if required): 75 psf • Dead: Concrete = 0.16 K/cu ft, Steel 0.49 k/cu ft. • Load Combinations: Per Table 3.4.1-1 LRFD (Load Combinations and Load Factors)

1-7.2 Survey, Hydrologic, Hydraulic, and Geotechnical Data Used

LiDAR survey and probable water surface elevations were used to develop the conceptual plan. For information about data used for the conceptual design, see Section 1-3.1.

No site specific geotechnical data were available at the time of this conceptual design. Numerous borings will be required to facilitate development of an accurate cross section of the geology below the bridge. For this conceptual design phase, typical near shore soil characteristics of Puget Sound are used to select the bridge foundation type. Geotechnical investigations will be required for completion of PED; see Section 1-4.3.

1-7.3 Site Selection Studies

The site selection is discussed in Section 1-6.

1-7.4 Major Structures

General: The proposed State Highway 101 Bridge is the only major structure included in the proposed restoration of Duckabush River Estuary. The bridge superstructure consists of a reinforced continuous concrete deck supported by a series of standard AASHTO pre-cast, pre-stressed concrete girders. A concrete cap beam and two concrete columns are provided at each bent to support the girders. The concrete columns are supported by drilled-in concrete piers.

Superstructure: Pre-cast, pre-stressed concrete girders are fabricated of high-quality concrete and they are the most common type of girders used in state of Washington. Concrete girders require lower-maintenance than their steel counterparts and are competitively priced with steel girders. The economy in structural design can be achieved by designing around the standard girders from the Bridge Design Manual Span Capability Sheet.

Substructure: Deep concrete shafts are used in liquefiable soils, which are commonly found in the flat tidal zones of Puget Sound. The design objective is to extend the foundation shafts to a suitable bearing depth to provide the necessary lateral support for the structure during a seismic event. See Section 1-4 for additional information.

Technical Considerations: Several technical goals were taken into consideration in selecting the type of bridge recommended for the project, including: cost-effectiveness, functionality (traffic and hydraulic needs), and structural strength/safety. The proposed design incorporates a repetitive concept that incorporates standard structural components, leading to maximum economy for fabrication and installation. The geometric design facilitates the two-lane traffic requirements and the selected spacing between the support bents and the deck height provide the opening that is required by hydrology.

Bridge Type recommended for this site is a pre-cast, pre-stressed concrete girder bridge that supports a continuous concrete deck. The girders are supported by cap beam which comprise the transverse beam of the pier system. The pre-cast girders will be fabricated offsite and shipped by truck to the site for installation. Standard WSDOT pre-cast concrete girders are an efficient and economical bridge type for continuous span construction.

Span Length affects many aspects of the design. The overall thickness of the superstructure is a function of the span length, and the thickness of the superstructure relates to a vertical clearance need for flood conditions. The quantity of bridge piers/bents is another byproduct of the selected span. More bends require more construction as they create more constriction to the hydraulic flow. The selected approximate 120 foot span is considered to be a reasonable balance between economy, repetitiveness, constructability and functionality.

Depth of Structure: Total thickness of the superstructure is the sum of the “A” dimension and the girder depth. The “A” dimension is the thickness of concrete deck directly above the girder. Conceptual level design indicates that a WF50G standard AASHTO girder with a deck of 11.25 inches satisfies structural requirements. See Figure 2-7-1 below.

Alignment: (see Section 1-6, Civil Design)

Appendix 5.6-A1-2 Span Capability of WF Girders

Girder Type	Girder Spacing (ft)	CL Bearing to CL Bearing (ft)	"A" Dim. (in)	Deck Thickness (in)	Shipping Weight (kips)
WF50G	5	140	11.00	7.50	124
	6	135	11.25	7.50	119
	7	125	11.25	7.50	111
	8	120	11.50	7.50	107
	9	115	11.25	7.50	102
	10	110	11.25	7.50	98
	11	110	11.25	7.50	98
	12	105	11.50	8.00	94

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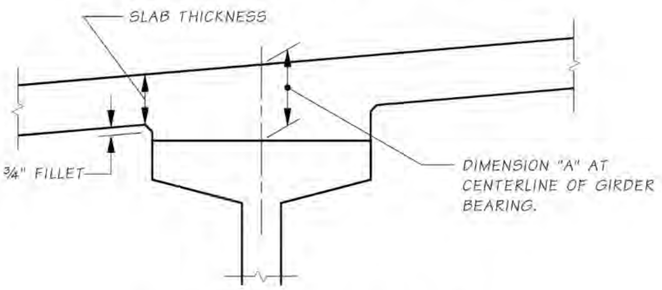

Depths for Deck Slab Design at Interior Girder
Figure 5.7.1-1

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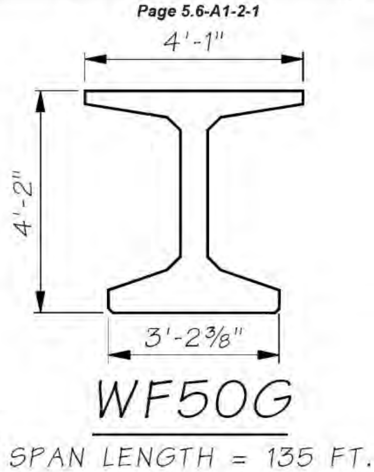

Figure 2-7-1. Detail of WF50G standard AASHTO girder dimensions.

1-7.5 Describe Evaluation and Selection of Substructure Alternatives Based On Economy and Performance

These bridges are located in an estuarine environment, likely requiring deep foundation as recommended by the geotechnical engineer.

The soils at this site are likely to experience liquefaction during an earthquake. As such, the shafts will have to extend to a suitable bearing depth for a solid fixed embedment.

The cost comparison between types of deep foundations (piles versus shafts) does not always result in a clear cost advantage for either foundation type. Many factors come into play such as availability of equipment to a contractor, a contractor's preferred method, the depth of the footing and the ease of access, construction schedule, and depth of foundation. In general, cost is not a determining factor for deep foundation type. Forces, displacement, and geological conditions will determine which system is best to use.

General and local scour is always a consideration with deep foundations. Subsequent design will include a hydraulic scour analysis. Protection of the structure from hydraulic scour may compete with the goals of the restoration. Preliminary design will evaluate these considerations and mitigate accordingly.

For additional information, see Section 1-2.

1-7.6 Construction Considerations

(See Section 1-6, Civil Design).

1-7.7 Stability Analyses

A precise numerical analysis of the bridge superstructure will be performed at the time the bridge design gets underway. At this conceptual stage of the design sufficient qualitative stability elements have been incorporated to ensure the bridge is stable in all three spatial directions. The recommended continuous bridge deck provides the superstructure with a continuous diaphragm. The deeply drilled-in reinforced concrete shafts provide the vertical and lateral stability of the bridge, and the concrete columns that connect the bridge superstructure to the substructure form a stable system for vertical and lateral load transfer to the substructure.

1-7.8 Stress Analyses

Stress analyses are a fundamental component in the design process and serve as the basis of how all structural elements are assembled to work together. Design shall be in accordance with governing standards of the WSDOT Bridge Design Manual and the AASHTO LRFD Manual.

1-7.9 Thermal Stress Analyses

Thermal analysis is a fundamental component of the design process and will be considered per the AASHTO LRFD design specifications. In general thermal stresses are handled by providing expansion joints in strategic locations to permit a bridge to expand and contract without a large buildup of stresses or movement.

1-7.10 Other Analyses

The conceptual design has been based on traffic requirements, hydraulic analyses, loading requirements of structures, and constructability considerations.

1-7.11 Additional Studies, Tests, Analyses

The information needed to design a bridge is generally captured in the following studies, tests, and analyses:

- Boundary and Topographic Survey
- Geotechnical Investigation and Report
- Hydraulic and Scour Analysis

Additional investigation and studies may be needed for permitting or other site requirements unrelated to the infrastructure. See Section 1-21 for a complete list of recommended additional studies and investigations.

1-8 ELECTRICAL AND MECHANICAL REQUIREMENTS

Electrical and mechanical structure requirements are not applicable to this site. Utility line relocations are discussed in the Section 1-6.3

1-9 HAZARDOUS AND TOXIC MATERIALS

A Phase 1 Environmental Site Assessment was conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments,* and ER 1165-2-132: *HTRW Guidance for Civil Works Projects*.

The assessment revealed no evidence of recognized environmental conditions in connection with the proposed project footprint, nor any conditions at neighboring sites which have the potential to affect work at the Duckabush site.

1-10 CONSTRUCTION PROCEDURES AND WATER CONTROL PLAN

The proposed restoration will involve earthwork and exposure of bare ground excavation of channels. At this stage of design, it is assumed that standard best management practices will be implemented to control erosion and sedimentation and ensure construction areas are stabilized as needed to prevent adverse impacts. A standard temporary erosion and sediment control plan will be developed during PED.

Some in-water work may be required during channel creation. For channel creation, work will be sequenced to avoid in-water work as much as possible and most of the channel creation work will occur in the summer months prior to removal of the roadway embankment. For required in-water work, channel excavation will take place at low water to reduce the likelihood of releasing sediments into downstream waters and will make use of the temporary erosion and sediment control plan. Standard soil cover and stabilization practices will be implemented to stabilize the channels prior to introduction of water.

Specific measures for construction procedures and water control will vary depending on the location and nature of the work associated with the site. State and Federal resource agencies will impose specific timing restrictions on in-water work to protect fish and wildlife, and the Corps will adhere to conservation measures detailed in environmental compliance documents. In addition, specific measures may be required to protect downstream infrastructure or built environments. A complete description of best management practices will be determined during PED.

1-11 INITIAL RESERVOIR FILLING AND SURVEILLANCE PLAN

The proposal is for ecosystem restoration. (Not applicable.)

1-12 FLOOD EMERGENCY PLANS FOR AREAS DOWNSTREAM OF CORPS DAMS

The proposal is for ecosystem restoration. (Not applicable.)

1-13 ENVIRONMENTAL OBJECTIVE AND REQUIREMENTS

Feasibility-level information to develop designs, plans, and specifications, and to execute construction and operations is included in the Project's supporting documents including the U.S. Fish and Wildlife Service report titled "Strategic Restoration Conceptual Design - Preliminary Environmental Contaminant, Cultural Resource, and Endangered Species Site Evaluations." The environmental information developed for the analysis in the FR/EIS provides additional environmental objectives and requirements for final site design development. As summarized in Section 1-6, Civil Design, substantial environmental information was developed for the FR/EIS regarding environmental problems, opportunities, and constraints such that the Corps could estimate costs of the restoration sites and prepare the Real Estate

Plan. The Corps will adhere to requirements stated in the Endangered Species Act consultation documents, Clean Water Act Section 401 certification, and other site-specific environmental compliance documents. The Corps has prepared a Programmatic Agreement (PA) for Section 106 of the National Historic Preservation Act compliance. As outlined in the PA, cultural resource investigations are necessary in the PED phase to determine if National Register eligible historic properties are located in the restoration project area prior to construction. The Monitoring and Adaptive Management Plan will be used to determine whether the site is meeting environmental objectives after construction.

1-14 RESERVOIR CLEARING

The proposal is for ecosystem restoration. (Not applicable.)

1-15 OPERATION, MAINTENANCE, REPAIR, REPLACEMENT AND REHABILITATION (OMRR&R)

Operations and maintenance costs for the Duckabush estuary restoration are related to maintaining flow in the North channel, invasive species removal, and maintenance of the infrastructure components of culverts, the roadways, and the new Highway 101 bridge.

- Maintaining flow to the North Distributary Channel will involve debris removal, which is likely to accumulate after large storms when high river flows wash debris from upstream, and sediment removal if the crossing connection channel fills in to the point that flow is cut off from the channel on the north side of the estuary. This channel provides a significant amount of sediment distribution for maintenance of substrate for shellfish beds and provides a rearing and migration corridor for salmonids.
- Vegetation maintenance costs are primarily to prevent and eradicate invasive plant species. This will require annual monitoring and semi-annual treatments if invasive species are found on site. This project has no planting plan other than hydroseeding at the end of construction; therefore standard O&M of watering and replacement will not be needed on plants at this site. The purpose of the project is primarily to restore freshwater input and sediment transport processes to the intertidal zone where irrigation is not necessary. Native saltmarsh plants often recruit on their own where substrate conditions are provided.
- Yearly culvert inspection and maintenance includes removal of debris and sediment.
- Roadway & embankment inspection, maintenance and repair - Maintenance costs for roadways and road bridges were developed based upon the WSDOT Pavement Policy. It is assumed that roadways will be constructed with hot-mix asphalt, and that the maintenance of a particular road will occur as part of a larger effort that includes adjacent road sections. Repair and maintenance includes:
    - Roadway asphalt overlay twice during the 50-year period of analysis
    - Roadway grind and inlay once during the 50-year period of analysis
    - Roadway guardrails, signs and striping.

- Bridge maintenance - Bridges will be constructed using pre-stressed concrete girders which are commonly used due to their low maintenance costs. WSDOT staff indicated that the maintenance costs do not vary greatly by bridge length (Wilson, 2011 and Baroga, 2011). Maintenance activities will include:
    - Bridge inspection & cleaning every year 2 or 3 man crew for 1 week
    - Replacement of guardrails, retrofit and structural repairs.

Annual OMRR&R is estimated at $122,000 for the 50-year project period. Additional assessment of O&MRR&R activities will be conducted during PED.

1-15.1 33cfr Part 208 Projects

The site is not a flood control project to be maintained and operated according to regulations in 33 CFR 208. (Not applicable.)

1-15.2 Channel or Basin Clean Out Projects

The restoration does not include channel or basin cleanout activities. (Not applicable.)

1-15.3 Multiple-Purpose, Complex Projects with Power Production

No power production is proposed. (Not applicable.)

1-15.4 Frequency and Cost of Maintenance Dredging

No maintenance dredging is proposed. (Not applicable.)

1-16 ACCESS ROADS

Construction activities will require heavy equipment to be mobilized to the site. It is assumed access to the site during construction will utilize highway 101 and Duckabush road. Temporary traffic control is necessary during mobilization and site access activities. Construction sequencing should keep the fire station and Duckabush road open for 24/7 access. The area south of the fire station and the parking area to the south of the highway 101 bridge of the project are possible staging areas. Construction staging areas, site access and construction sequencing will be further analyzed during PED, after a cultural resource survey has been completed. Staging areas will be placed in locations that avoid impacts to cultural resources that may be identified during a cultural resources survey.

1-17 CORROSION MITIGATION

No new corrodible construction is proposed. (Not applicable.)

1-18 PROJECT SECURITY

The proposal is for ecosystem restoration. (Not applicable.)

1-19 COST ESTIMATES

The Duckabush River Estuary construction cost estimate of $90,523,000 (Mar 2016 dollars) consists of costs for removal of roadway embankment and bridges, channel excavation and construction of new bridges and roadway embankments. Other work includes culvert replacement and hydroseeding.

The largest cost driver is relocations including demolition of existing bridges and roadways and construction of new roads and bridges ($56,506,000 construction cost, Mar 2016 dollars). This work consists of all temporary construction facilities and platforms needed to construct the bridges as well as the new intersection with Duckabush Road, a private road and US Highway 101.

Following a formal cost and schedule risks analysis, a project contingency of 46% was developed. Primary engineering related cost risks came from the high uncertainty surrounding bridge and bridge foundation design. Minimal detail was provided in the design report and additional geotechnical and survey information will be required to constrain the uncertainty in these costs. The current estimate includes $500,000 for mitigation for removal of the Duckabush Bridge 101-266 which is on the National Register of Historic Places. Should additional National Register eligible cultural resources be identified during the cultural resource survey and need to be mitigated then cost would rise to account for those mitigation costs. The cost risk register accounts for cultural and historical properties that may be impacted by the project. Also, there is a likelihood that newly built roadway embankments will settle and will require additional material to maintain the required roadway elevation. Schedule risks are controlled by the potential for work to need to stop during the rainy seasons and when in-water work is prohibited. Additional mobilizations and lost time would be incurred because of this.

There are non-cost related risks as well. There could be either erosion or sedimentation in excavated channels during river flooding. Channels will need to be watched following construction as part of the overall monitoring plan. Risks that do not directly affect cost include sedimentation of the distributary channels that lead to flow reduction or obstruction. Further analysis is needed in this area, and requirements could range from a monitoring plan to omission from the project scope.

Opportunities to reduce the project cost include a potential for reductions in material prices. Opportunities for this site for cost come from the possibility that bridge work may be substantially less expensive than predicted. This chiefly comes from the standardized designs used throughout PSNERP for bridges. Foundation piers were all designed very conservatively, and may be reduced with additional analysis. Schedule opportunities come from the same issue. The PDT considered it likely that there could be schedule reductions from lowered requirements.

1-20 SCHEDULE FOR DESIGN AND CONSTRUCTION

The proposed restoration at the Duckabush River Estuary is considered to be relatively straightforward. Based on the low level of complexity, the anticipated design period for the site is approximately two years. This includes preparation of final design, plans and specifications, and the construction contract.

The anticipated construction period for construction of the new bridges and roadway embankments, channel excavation, and removal of existing bridges, roadways and embankments is approximately three years. Any in-water construction activities will take place during established work windows. Road and bridge elements would need to be phased in a manner that allows for continuous access across the estuary on Highway 101.

Property acquisition and environmental compliance timelines are not included in this duration. The time required to complete these upfront activities is unknown, but is assumed to be relative to the length of the anticipated design period for the site as described above.

1-21 STUDIES TO BE COMPLETED IN PED

Table 1-21-1summarizes recommended studies and additional investigations to be conducted at the site to support subsequent stages of design and implementation. Unless otherwise noted, these studies are recommended to take place during PED. In the table, studies are classified according to the following purposes:

- Data required for design, cost estimation or project compliance,
- design analysis to minimize project construction costs,
- design analysis to optimize environmental benefits,
- identification of induced flooding,
- and identification of actions needed for O&M.

Table 1-21-1. Studies Recommended for the Duckabush River Estuary Site

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Property Investigation/Survey	• Compile more detailed information on parcel ownership and property boundaries to finalize the design, confirm acquisition requirements, and support negotiations with property owners.	X	X			
Topographic/ Planimetric/ Bathymetric Survey	• Complete site-specific topographic and bathymetric surveys to refine design of key elements, confirm that target elevations are appropriate for the desired ecosystem components (low marsh, etc.), develop detailed construction and demolition plans, and provide a baseline for pre- and post-construction modeling, including hydrodynamic modeling.	X				
	• If needed, install a temporary tide gauge in the early design stages to obtain site-specific tidal statistics.	X				
Hydraulic Analysis/Modeling	• Implement a hydraulic model for the Duckabush River and Estuary reflecting the proposed geometry to predict the with-project water surface profiles and confirm the extent and nature of hydraulic effects from the project.		X	X	X	X
	• Combine review of aerial photographs with field surveys to quantify channel topology and hydraulic roughness and inform geomorphic evaluation under restored conditions.	X				
	• Assess hydraulics and effects of increased tidal prism to quantify effects on adjacent shores.		X	X		X

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Hydraulic Analysis/Modeling (cont.)	• Evaluate changes in salinity and flow patterns within, adjacent to and downstream of the site, if required.			X		
	• Formulate the detailed monitoring plan, including any required field surveys or instrumentation that will be used to evaluate the project's hydraulic performance.			X		X
Sedimentation Analysis/Modeling	• Analyze potential channel infilling and evolution of channels to determine long-term stability of the site.			X	X	X
	• Evaluate temporary increases in sedimentation downstream of the site during the establishment of new distributary channels to evaluate effects and fine tune channel design.		X	X		X
	• Perform a scour analysis to predict the range of scour at the new bridge piers and abutments.		X			
Coastal Engineering Studies	• Refine sea level projections using localized tide gauge data.	X				
	• Review and establish the final design tidal datums		X			
	• Conduct wind direction and run-up analyses to confirm coastal BFE.		X	X	X	
Geotechnical Investigation	• Complete a standard investigation to include subsurface explorations, testing, and field reconnaissance.	X	X			
	• Confirm borrow and disposal sites.	X				
	• Complete additional geotechnical study and recommendations to finalize design of levees, roads, and bridges.		X			
	• Perform a settlement analysis for roadway embankments.		X			
Foundation Design Study	• Perform static and seismic analysis according to AASHTO LRFD for vehicle bridges.		X			
Abutment Stability Study	• Evaluate the potential for liquefaction and ground improvement.		X			
Excavated Materials Study	• Evaluate the suitability of excavated materials for reuse.	X	X			

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Pavement Design Study	• Complete a pavement design study for new roadways and approaches (include traffic analysis for ESALs).		X			
Structural Engineering	• Structural analysis of the bridge, bridge piers, and foundation.		X			
	• Analysis for gravity, wind and seismic effects.		X			
	• Design of bridge deck and supporting structure for gravity, wind and seismic effects in accordance with criteria established in this report.		X			
Utility Survey	• Compile more detailed information on utilities to finalize the design and confirm acquisition requirements.	X	X			
Cultural Resources Investigation	• Complete surveys for archaeological and historic resources, particularly in areas proposed for excavation. If required, consider various mitigation measures for the removal of the National Historic Register listed Duckabush Bridge.	X	X	X		
Wetlands Investigation	• Document the location, extent, and character of wetlands.	X		X		
Cost Study	• Assess potential for cost and schedule reductions during refinement of restoration design.		X			
Environmental Compliance	• The Corps will coordinate with all relevant natural resource agencies during PED. Results of PED-phase studies will be provided to agencies and tribes as appropriate.			X		

1-22 DATA MANAGEMENT

Project documents, background materials, and digital files from the local sponsors were provided to the project team directly, through the State's Habitat Work Schedule, or via the Nearshore Portal. The project team also used databases previously developed by and for the Puget Sound Nearshore Ecosystem Restoration Project including the Change Analysis and backing geospatial data (see Section 1-3.1.1 for additional detail).

Work products for the conceptual restoration designs were developed primarily in GIS and typical word processor and spreadsheet applications. GIS products for all action areas were collected in a single geodatabase that captured spatially referenced locations and sizes of major design elements.

1-23 USE OF METRIC SYSTEM MEASUREMENTS

This report uses United States customary units for design and construction measurements. To remain consistent with work conducted to date, the metric system of measurement was not used.

1-24 REFERENCES

Earhart, C. and J. Shedd, 1996. West Shore Hood Canal Outfall windshield survey, compiled for Washington State Department of Transportation.

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USACE, 2003. Lower Duckabush River, Channel Migration Report, Floodplain Management Services program, prepared for Jefferson County, 25 pps.

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ANNEX 1-1: EXHIBITS

This annex contains a set of site-specific exhibits prepared for the proposed restoration. The exhibits include:

Exhibit A – Conceptual Design Plan

Exhibit B – Conceptual Design Sections

Exhibit C – Phase 1 Environmental Site Assessment

Exhibit D - Preliminary Foundation Axial Capacity Estimate Worksheet

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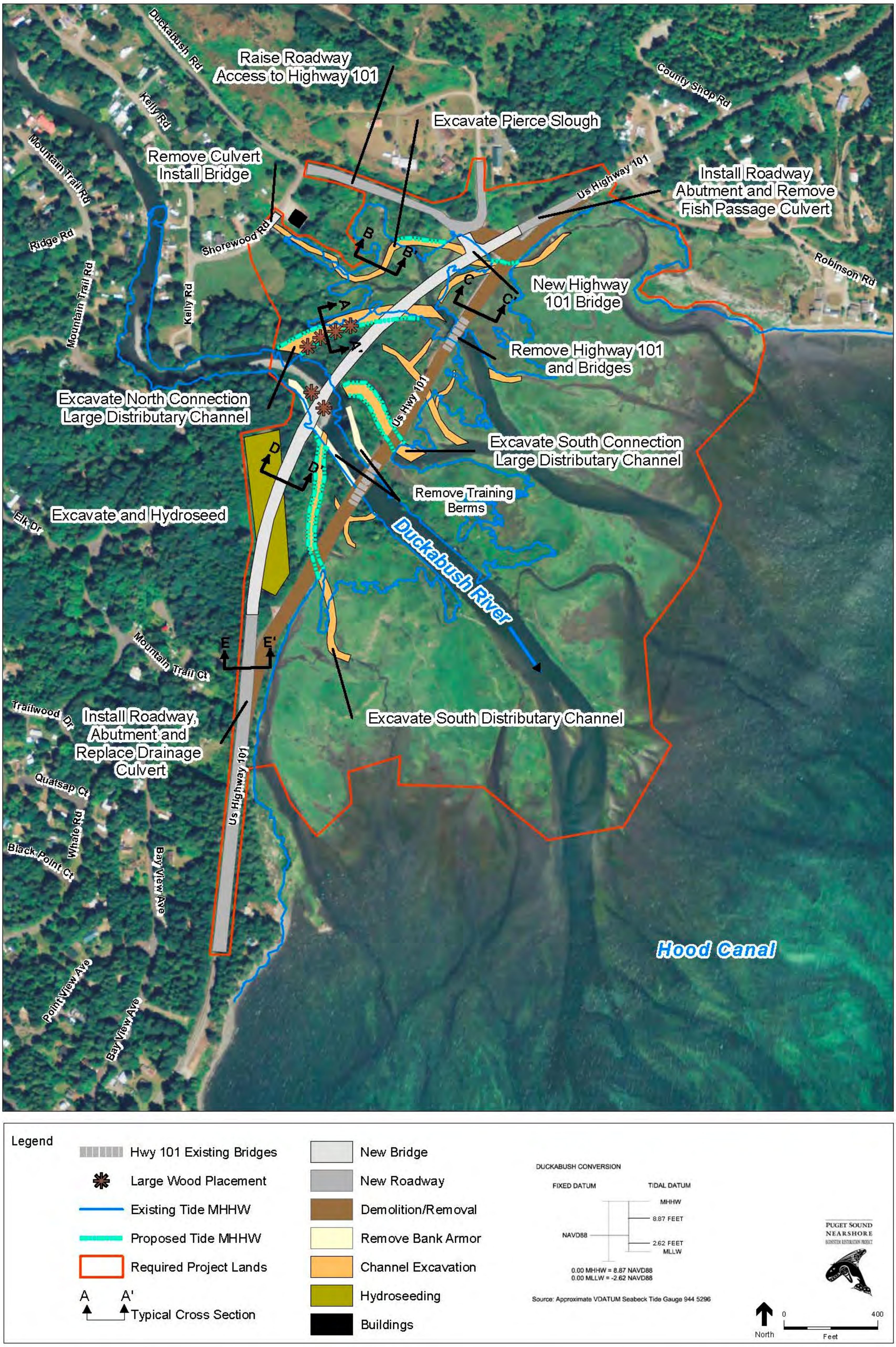
Exhibit A



ORTHO_2013_NAIP_WASHINGTON

Site Name: Duckabush River Estuary

Lead Contractor: ESA
Design Lead: ESA PWA with KPFF
Revised: USACE Petroff/Campbell February 2016

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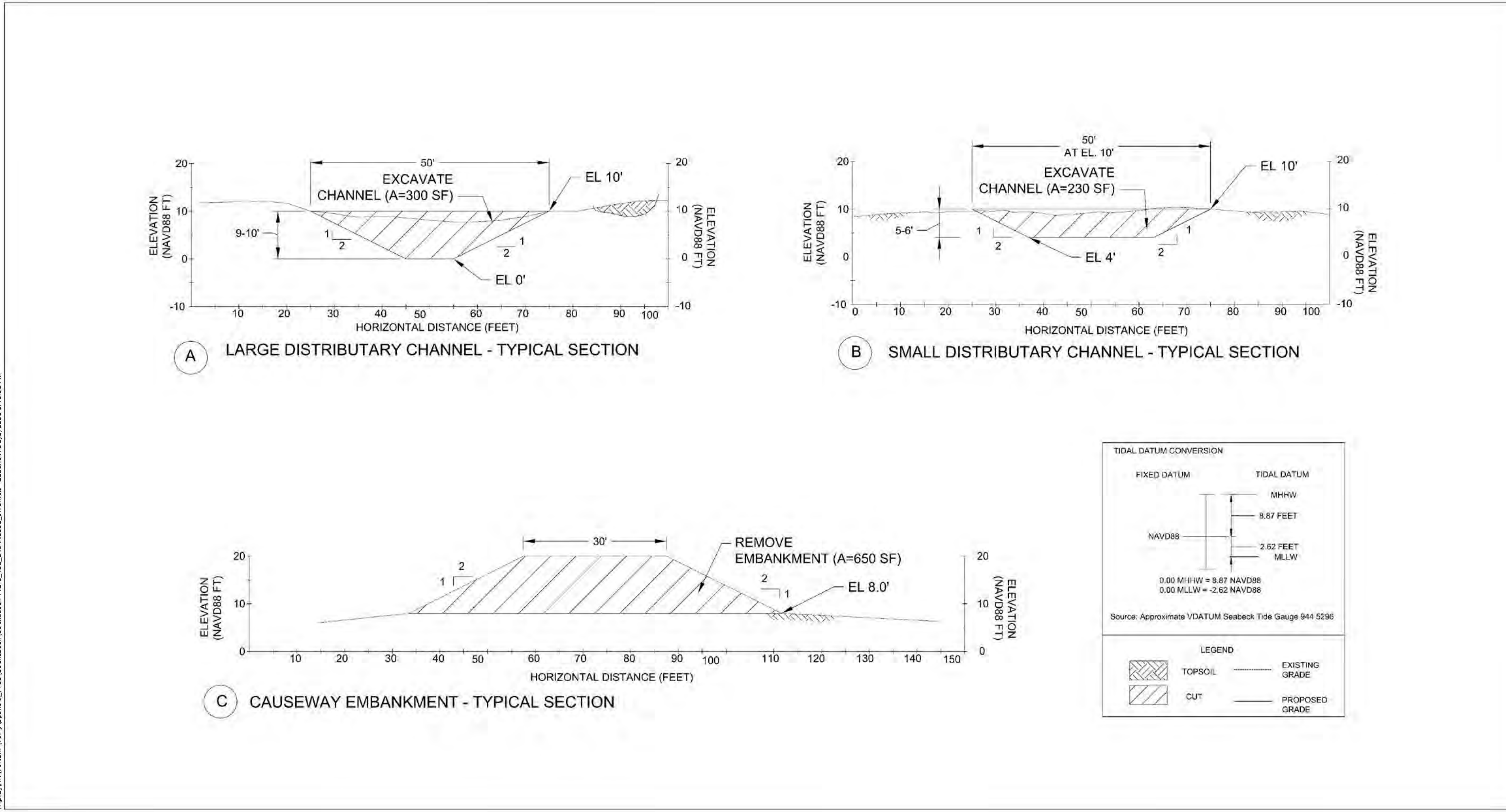
Exhibit B

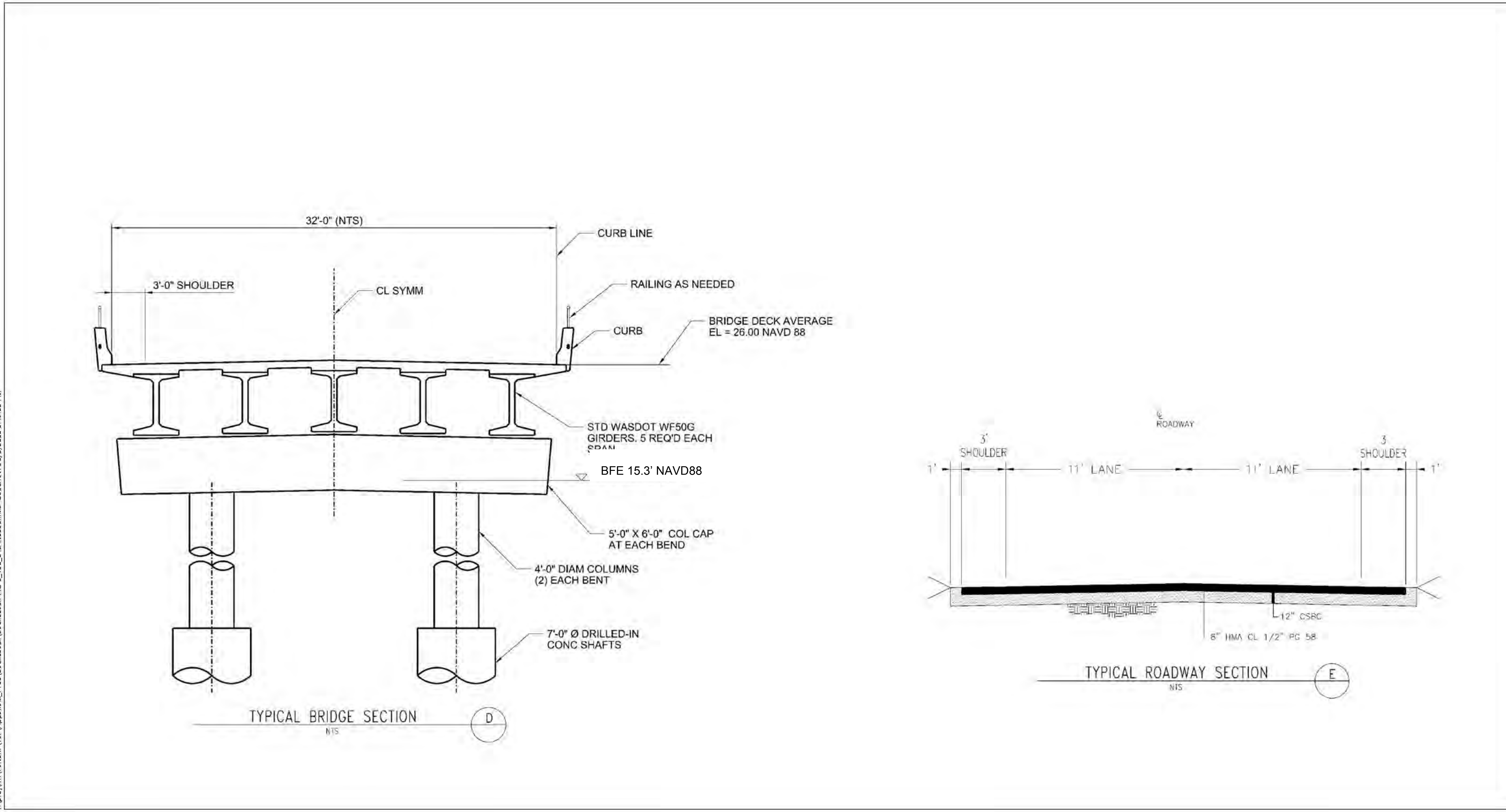


ORTHO_2013_NAIP_WASHINGTON

Site Name: Duckabush River Estuary

Lead Contractor: ESA
Design Lead: Anchor QEA, G. Sassen, ASLA
Revised: USACE Petroff/Campbell December 2015

Exhibit B



ORTHO_2013_NAIP_WASHINGTON

Site Name: Duckabush River Estuary

Lead Contractor: ESA
Design Lead: Anchor QEA, G. Sassen, ASLA
Revised: USACE Petroff/Campbell December 2015

Exhibit C

PUGET SOUND NEARSHORE ECOSYSTEM RESTORATION PROJECT FEASIBILITY STUDY HAZARDOUS, TOXIC, AND RADIOACTIVE WASTE PHASE 1 ENVIRONMENTAL SITE ASSESSMENT

EXECUTIVE SUMMARY

The Seattle District Corps of Engineers (Corps), working collaboratively with the Washington Department of Fish and Wildlife (WDFW) as local sponsor, along with many other regional partners, has conducted a General Investigation (GI) to evaluate problems and potential solutions of ecosystem degradation and habitat loss in Puget Sound, Washington. The Puget Sound Nearshore Study (Nearshore Study) is authorized under Section 209 of the River and Harbor Act of 1962 (Pub. L. 87–874). The Corps and local sponsor are recommending implementation of restoration actions at three sites throughout the study area as the outcome of the Nearshore Study. Pursuant to Section 102(2)(C) of the National Environmental Policy Act (NEPA) of 1969, as amended, the U.S. Army Corps of Engineers is preparing an Integrated Feasibility Report/Environmental Impact Statement (FR/EIS) for the three restoration actions. The Phase 1 Environmental Site Assessment for the Duckabush Estuary site is being conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments,* and ER 1165-2-132: *HTRW Guidance for Civil Works Projects,* except where noted below.

The assessment revealed no evidence of recognized environmental conditions in connection with the proposed project footprint, nor any conditions at neighboring sites which have the potential to affect work at the project site.

Exhibit C

PUGET SOUND NEARSHORE ECOSYSTEM RESTORATION PROJECT FEASIBILITY STUDY HAZARDOUS, TOXIC, AND RADIOACTIVE WASTE PHASE 1 ENVIRONMENTAL SITE ASSESSMENT

Exhibit C

1.0 INTRODUCTION

1.1 Involved Parties

The Corps is the lead Federal agency for the Puget Sound Nearshore Ecosystem Restoration Project (PSNERP) Report. The non-Federal, cost-sharing sponsor is the Washington Department of Fish and Wildlife (WDFW). As the non-Federal sponsor, WDFW contributes 50 percent of the total feasibility study costs in the form of cash or in-kind contributions; a feasibility cost sharing agreement was executed in 2001, with amendments.

1.2 Authority

The Puget Sound Nearshore Study (Nearshore Study) is authorized under Section 209 of the River and Harbor Act of 1962 (Pub. L. 87–874).

1.3 Guidance and Policy

Corps policy providing guidance for consideration of issues and problems associated with hazardous, toxic, and radioactive wastes (HTRW), as defined in this regulation, which may be located within project boundaries or may affect or be affected by Corps Civil Works projects is contained in ER 1165-2-132, *Hazardous, Toxic, and Radioactive Waste Guidance for Civil Works Projects*, which defines HTRW as "…any material listed as a 'hazardous substance' under the Comprehensive Environmental Response, Compensation, Liability Act (CERCLA)". ASTM International (ASTM) Standard E 1527-13 *Standard Practice for Environmental Site Assessments: Phase I Environmental Site Assessment Process* provides a comprehensive guide for conducting an HTRW Assessment. An assessment identifies known or suspected releases of hazardous substances (recognized environmental conditions) based on records review, site visit, and interviews.

1.4 Scope of Work

The complete investigation serves to identify any recognized environmental condition, as defined in ASTM Standard E 1527-13. This site assessment documents known and suspected HTRW sites discovered through a search and review of all reasonably attainable federal, state, and local government information and records. A site visit, interviews with relevant stakeholders, and review of aerial photographs are also mandated under the above standard.

1.5 Significant Assumptions

This report identifies known and suspected environmental concerns, both past and present based on availability of information at the time of the assessment. It is possible that unreported disposal of waste or illegal activities impairing the environmental status of the properties may have occurred which could not be identified.

1.6 Limitations and Exceptions

This document deviates slightly from the exact procedures outlined in ASTM E1527-13. Specifically, no "User Provided Information" nor "Non-Scope Services" were provided, and those sections of the report were omitted. Also, due to the layout of the overall document to which this report will be incorporated, it was decided that no appendices were to be generated for this report. Additionally, it should be noted that portions of this report were conducted by separate entities that did not have the ability to coordinate their efforts. However, this does not change the results or outcome of the report.

1.7 Special Terms and Conditions

No special terms or conditions with respect to ER 1165-2-132 and ASTM E 1527-13 standards were made.

Exhibit C

1.8 User Reliance

In accordance with ASTM E 1527-13 Section 7.5.2.1 "Reliance," the environmental professional is not required to independently verify the information provided by various sources but may rely on the information unless there is actual knowledge that certain information is incorrect or unless it is obvious that certain information is incorrect based on other information obtained during the course of the investigation or otherwise actually known to the investigators conducting the assessment. At the present time there is no indication that the information provided by the database search is incorrect.

2.0 SITE DESCRIPTION

2.1 Location and Legal Description

The "property", as defined by the referenced ASTM standard, in this case includes several different properties in the Duckabush River Estuary. For the purposes of this assessment, the proposed Duckabush project footprint will serve as the "property" under review (See Figure 6-2 in the main body text).

2.2 Site and Vicinity General Characteristics

The physical setting of the subject property and vicinity is detailed in Section 6.1.1 of the main feasibility report. According to Department of Ecology (Ecology) well logs, depth to groundwater immediately northwest of Shorewood Rd. is anywhere from 4-10 ft. below ground surface. Given this information, it can be inferred that groundwater elevations farther south in the estuary are even lower.

The project footprint consists of a large portion of the Duckabush River delta, covering the main stem, several distributary channels, intertidal marshes, and two bridges of Highway 101. The footprint is predominantly used as a wildlife protection area, and there has been little to no development or construction in the footprint due to the estuary's transitory nature and frequent flooding. The beaches in the estuary may have been used for commercial oyster and Manila clam aquaculture in the past, and currently WDFW allows visitors to harvest shellfish on a small scale.

3.0 RECORDS REVIEW

3.1 Standard Environmental Records

A records search was conducted on September 25, 2015, using a variety of sources. The primary sources included EPA's National Priority List Mapper, EPA's EnviroFacts database, Ecology's Toxics Cleanup Program (TCP) database, and Ecology's Facility/Site database. Below are the parameters and results of the records search.

Parameter	Source	Minimum Search Distance (mi.)	Results
Federal NPL	EPA NPL Mapper	1	None
Federal Delisted NPL	EPA NPL Mapper	0.5	None
Federal CERCLIS	EnviroFacts	0.5	None
Federal RCRA Generators	EnviroFacts	Property and Adjoining Properties Only	1 finding (Pereles Herrera)

Exhibit C

Federal RCRA TSDs	EnviroFacts	0.5	See above
Federal RCRA Corrective Action Sites	EnviroFacts	1	See above
Federal and State ICs Registry	Ecology TCP	Property Only	None
Federal Toxic Release Inventory	EnviroFacts	0.5	None
State and Tribal Cleanup Sites	Ecology TCP	1	None
State and Tribal Landfills and/or Solid Waste Disposal Facilities	Ecology Facility Search	0.5	None
State and Tribal UST Registry	Ecology TCP	Property and Adjoining Properties Only	None
State and Tribal LUST	Ecology TCP	0.5	None
State and Tribal Brownfields	Ecology TCP	0.5	None

The initial record search shows one permitted hazardous waste generator (the Pereles and Herrera site) approximately a tenth of a mile north of the footprint, along the mainstem of the Duckabush River. However, further investigation indicated that the permit is inactive, the property has since been sold to another owner, and an aerial investigation spanning the past 75 years shows no indication of current or past commercial or industrial activity.

3.2 Historical Records

Historical aerial photographs on Google Earth were reviews, along with historical maps from the U.S. Forest Service (USFS), and various documents provided by the Washington State Department of Fish and Wildlife (WDFW).

An examination of a 1923 USFS map of the Olympic Peninsula shows that a narrow gauge logging railroad was built from Brinnon up the watershed into what is now Olympic National Park. Further investigation attributed the railroad to the Webb Logging Company, which built the line in or around 1910. The railroad appears to turn northeast as it approaches the Duckabush estuary coming down from the hills, and probably passed a quarter of a mile north of the project footprint. The Webb Logging Company stopped logging in the Duckabush in 1929, and the railroad was not shown on subsequent USFS maps.

A 2001 document entitled *Brinnon Subarea Plan: A Chapter of the Jefferson County Comprehensive Plan* mentions that the Town of Brinnon used to have a solid waste transfer station on Duckabush River Road that was subsequently closed by Jefferson County. It is unclear whether the document is referring to Duckabush Rd. or River Rd. An aerial photograph search shows no obvious locations for such a facility, and further document searches did not find any subsequent references.

Exhibit C

3.3 Additional Environmental Record Sources

There are no additional environmental record sources included in this assessment.

4.0 SITE RECONNAISSANCE

A site visit was conducted on September 29, 2010 by Rich Carlson, a biologist at the USFWS. The reconnaissance found that development in the areas adjacent and near the project footprint is almost exclusively residential, except for a fire station, the two bridges, and fill associated with the Highway 101 grade. The two bridges are both made of steel and concrete, and are not painted. All other observations were confirmed by the record search.

An interview was conducted by Rich Carlson of the USFWS on September 29, 2010 with Richard Brocksmith of the Hood Canal Coordinating Council and Neil Werner with the Hood Canal Salmon Enhancement Group. No substantive record of this interview exists.

In order to query the current land owner, a further interview occurred on October 8, 2015 with Shane Belson, WDFW Site Manager of the Duckabush Wildlife Unit. Below is a record of that interview.

<table>
<tr><th colspan="5">Interview Record</th></tr>
<tr><td>Site:</td><td colspan="2">Duckabush River Estuary, PSNERP</td><td></td><td></td></tr>
<tr><td colspan="5">Interview Type: Phone
Location of Visit: N/A
Date: October 8, 2015
Time: 230pm</td></tr>
<tr><th colspan="5">Interviewers</th></tr>
<tr><td colspan="3">Name</td><td>Title</td><td>Organization</td></tr>
<tr><td colspan="3">David Clark</td><td>Remediation Biologist</td><td>CENWS-EN-TS-ET</td></tr>
<tr><th colspan="5">Interviewees</th></tr>
<tr><td>Name</td><td>Organization</td><td>Title</td><td>Telephone</td><td>Email</td></tr>
<tr><td>Shane Belson</td><td>WDFW</td><td>Site Manager</td><td>360-480-9105</td><td>N/A</td></tr>
</table>

Exhibit C

Summary of Conversation

OBJECTIVE: The objective of the interview is to obtain information indicating presence or likely presence of any hazardous substances or petroleum products in, on, or at a property: (1) due to any release to the environment; (2) under conditions indicative of a release to the environment; or (3) under conditions that pose a material threat of a future release to the environment

1. The current use of the property is a wildlife protection area. What is your knowledge of prior uses of the project area?
   - Property was acquired in 1997 by WDFW. Respondent had heard talk of past shellfish harvesting in estuary, although most likely not at a commercial level. Respondent had removed a sign from the parking area that said "Twin Eagle RV Park", implying that an RV park had been located there before acquisition. Recommended following up by looking at the WDFW management plan for the Duckabush Unit.

2. What is your knowledge of prior and current uses of adjacent properties to the project area? Any industrial activity? Any cleanups?
   - No current or historical industrial activity on or adjacent to WDFW property. Current land use is residential in some areas, and conservation in others, mostly the estuary.

3. Tell me about the parking area to the south of the 101 bridge; does WDFW own that property? Have any spills or releases occurred in that area?
   - See the answer to question 1. WDFW currently owns this area.

4. Are you aware of any releases to the project area or surrounding areas? (Spills, etc.) Do you know of any chemicals that were used or stored on the property?
   - No.

5. Are you aware of any environmental reports written for the project area or surrounding areas? Such as, Environmental site assessment reports, environmental compliance audit reports, environmental permits (HW disposal permits, NPDES permits, etc), community right-to-know plans, risk assessments, etc.
   - No.

6. Any other information that might be pertinent to the site assessment?
   - No.

Additional Site-Specific Information

- The Corps will follow up on the possibility of the Twin Eagle RV Park being located in the now-WDFW parking area, and by reviewing the WDFW Duckabush Management Plan.
- Management Plan – No additional information
- Joe Leonard of Waketikeh Creek (later Joe Leonard Oyster Co.?) harvested oysters in Duckabush. Currently, Olympic Canal Tracts Owners Association has explored placement of geoduck seed.

Upon completion of the interview, further research was conducted to gather more information about the potential past location of the Twin Eagle RV Park on the current WDFW parking area, which is in the proposed project footprint. The WDFW Duckabush Unit Management Plan contained no further information concerning a past RV park, and further records of the RV Park's existence cannot be located.

Another key piece of information gleaned from the 2015 interview is the statement that the adjacent areas to the footprint do not, and most likely have not, contained any sort of industrial activity.

Exhibit C

5.0 FINDINGS AND CONCLUSION

This Phase 1 Environmental Site Assessment was conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments,* and ER 1165-2-132: *HTRW Guidance for Civil Works Projects*. The assessment was initially conducted by Rich Carlson of the USFWS in 2010, and completed by David Clark, remediation biologist at the U.S. Army Corps of Engineers (Corps) in 2015.

This assessment has revealed no evidence of recognized environmental conditions in connection with the proposed project footprint, nor any conditions at neighboring sites which have the potential to affect work at the project site.

6.0 SOURCES

ASTM E1527-13: *Standard Practice for Environmental Site Assessments.*

Engineer Regulation 1165-2-132: *HTRW Guidance for Civil Works Projects.*

Environmental Protection Agency. 2015. "Cleanups in my Community." http://www2.epa.gov/cleanups/cleanups-my-community

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Exhibit D

Preliminary Foundation Axial Capacity Estimate					
Project	Duckabush Bridge				
Date	4 November 2015				
Parameter	***Value***	***Unit***	***Definition***	***Equation***	***Reference***
Dead Load Estimate					
L_{bridge}	2100	ft	total length of bridge	user input	-
N_{spans}	18	-	number of spans	user input	-
L_{span}	120	ft	length of largest span (length in calc)	user input	-
N_{lanes}	2	-	number of lanes	user input	-
W_{bridge}	34	ft	width of bridge	user input	-
D_{deck}	0.700	ft	thickness of deck	user input	-
$W_{pile\ cap}$	6.0	ft	width of pile cap	user input	-
$D_{pile\ cap}$	5.0	ft	depth of pile cap	user input	-
M_{girder}	1.0	k/ft	approx. weight/LF of girder	user input	-
N_{girder}	5	-	number of girders	user input	-
$\gamma_{concrete}$	0.160	kcf	unit weight of concrete	-	-
P_{dead}	1,220	kips	unfactored dead load	sum of bridge weight	-
Live Load Estiamte					
M_{truck}	72	kips	truck load estimate	user input	-
LL_{lane}	0.64	k/ft lane	live load per ft per lane	user input	-
P_{live}	226	lb	unfactored live load	sum of live loads	-
Load Combination					
DL	1,525	kips	dead load, strength I	$DL = 1.25P_{dead}$	(2) table 10-4
LL	395	kips	live load, strength I	$LL = 1.75P_{live}$	(2) table 10-3
P_U	1,920	kips	combined load, AASHTO LRFD 2010	$P_U = DL + LL$	-
Shaft Geometry and Various Parameters					
B_b	84	in	base diameter of drilled shaft	user input	-
D	135	ft	depth to bottom of shaft	user input	-
# of Piles	2	-	Number of piles per pier	user input	-
A_t	38.48	sf	toe-bearing contact area	$A_t = (B_b/12/2)$^2 x pi	-
A_s	2969	sf	side-friction contact area	$A_s = (B_b/12)$ x pi x D	-
γ_{soil}	0.1	kcf	unit weight of soil considered (average	user input	
γ_w	0.0624	kcf	unit weight of water	-	-
Base Resistance Estiamte (O'Neill and Reese, 1999) - N-value method					
N_{60}	15	blows/6"	SPT N-value $2B_b$ below the toe	user input	-
q'_t	18	ksf	net unit toe-bearing resistance	$q'_t = 1.2\ N_{60} \leq 60$	(1) equation 14.6
φ_s	0.4	-	resitance factor	user input	(2) table 10-5
P_t	277	kips	toe-bearing resistance	$P_t = \varphi \times q'_t \times A_t$	-
Side Resistance Estiamte (O'Neill and Reese, 1999) - Beta method					
z	67.5	ft	depth to midpoint of soil layer	z = D/2	-
σ'_z	2.54	ksf	average vertical effective stress	$\sigma'_z = z \times (\gamma_{soil}-\gamma_w)$	-
β	0.25	-	beta (0.35 > β > 0.25) for clays	user input	(1) section 14.3
f_s	0.63	ksf	unit side-friction resistance	$f_s = \beta \times \sigma'_z$	(1) equation 14.22
φ_s	0.45	-	resitance factor	user input	(2) table 10-5
P_s	848	kips	side-friction resistance	$P_s = f_s \times A_s$	-
Load Demand and Capacity					
Q_{des}	960	kips	Estimated static load demand / pile	-	-
Q_{all}	1,125	kips	Estimated pile load capacity	$Q_{all} = (P_t + P_s)$	(1) equation 13.1
$Q_{des} \leq Q_{all}$	**OK**	-	Design Load ≤ Load Capacity	-	(1) equation 13.5

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Section 2 – Nooksack River Delta

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Section 2: Nooksack River Delta

Table of Contents

2-1 GENERAL – NOOKSACK RIVER DELTA

2-1.1 Overview of Restoration Site

The Nooksack River originates on and around Mt. Baker, a 10,700-foot-high peak in the Cascade Mountains. In the upper watershed, three main forks converge before the river enters the flatter, agricultural lowlands. The Nooksack River Delta is centered on the Lummi Nation lands north of Bellingham in the San Juan/Georgia Strait Sub-basin. The restoration site encompasses nearly all of the Nooksack and Lummi River Estuaries below Ferndale.

The flow path of the Nooksack River has been modified since the mid-19th century beginning with active removal of large wood, draining, diking, and levee construction, which forced almost all flow to the east side of the delta. Prior to 1860, the Nooksack River emptied into both Lummi and Bellingham Bays with flows shifting between the two outlets over time, depending on logjams. In the late 1800s, the Nooksack River was diverted to drain into Bellingham Bay (Collins and Sheikh 2003).

This shift of the lower Nooksack River virtually eliminated migration of stream channels over the Lummi River delta (Bortleson et al. 1980). Early General Land Office mapping (circa 1887-1888) shows that significant meandering channels and intertidal habitats existed on both sides of the Lummi Peninsula. Today, substantial surface water diversions, groundwater withdrawals, and drainage activities within the Nooksack River watershed also impact the magnitude, timing, and duration of surface water flows in the Nooksack River.

The Nooksack River Delta site was selected to address river delta restoration objectives to protect and restore freshwater input and tidal processes where major river floodplains meet marine waters. Target ecosystem processes include:

- Tidal flow
- Freshwater input (including alluvial sediment delivery)
- Erosion and accretion of sediments
- Distributary channel migration
- Tidal channel formation and maintenance
- Detritus recruitment and retention
- Exchange of aquatic organisms

The proposed restoration would modify levees, roads, and other hydrological barriers to restore riverine and tidal flow as well as sediment transport and delivery processes throughout a substantial portion of the historical Nooksack River delta. Construction of new setback levees would provide flood risk management for active businesses, residences, farms, transportation infrastructure, and Lummi Nation lands in the project area. The conceptual restoration plan for the Nooksack River Delta has the following main elements:

- Armor removal for streambank restoration and reconnecting floodplain habitat
- Dike removal or modification for floodplain freshwater marsh restoration
- Setback levees for floodplain reconnection and side channel development
- Riparian revegetation for shading, nutrient inputs, and complexity of bank habitat
- Large wood placement for increased habitat complexity
- Hydraulic modification: partial restoration of river flow to Lummi River through installation of water control structure at confluence of Lummi and Nooksack Rivers; structure intended to facilitate transfer of freshwater and sediment to the Lummi River
- Topography restoration: regrading of the Lummi River to allow for more frequent engagement by fluvial flows from the upper watershed
- Non-structural measure: residential relocations

Details of the restoration design are provided in Section 2-6 and shown on the exhibits provided in Annex 2-1. Figure 2-1-1 shows the Nooksack River Delta site and vicinity.

For the discussion in this Engineering Appendix chapter, the term Nooksack River Delta is taken to mean both the Nooksack River, the Lummi River (also known as the Red River) and their estuaries.

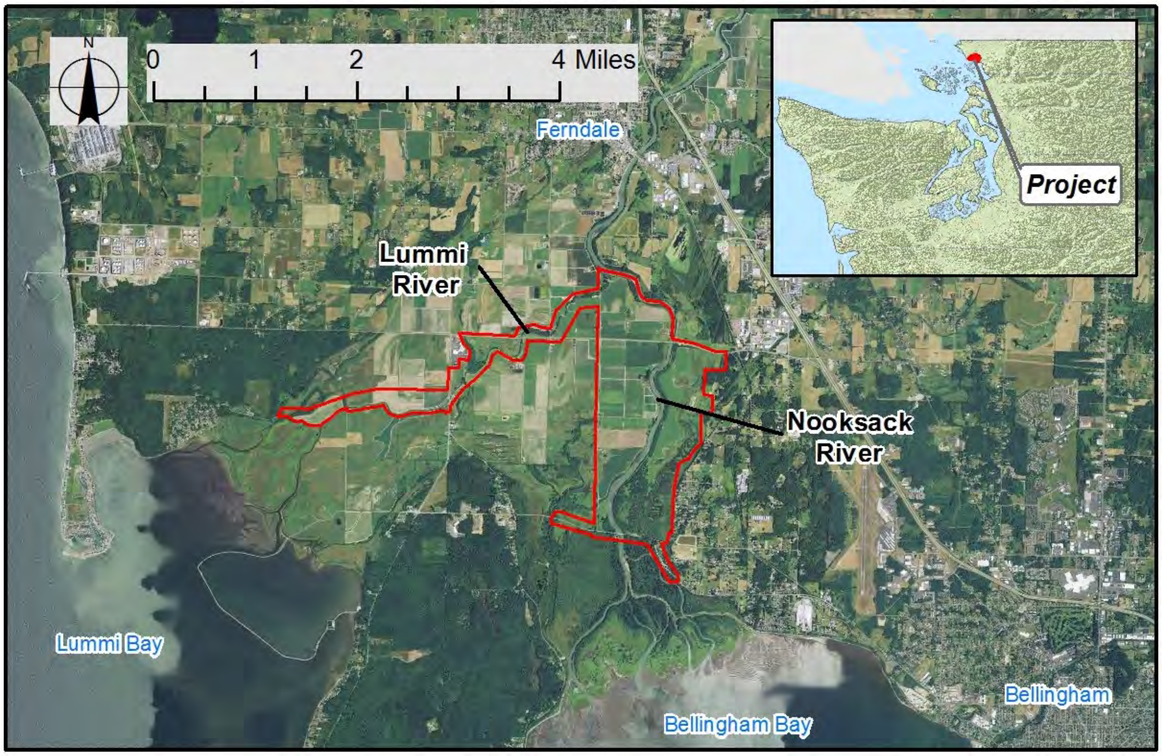

Figure 2-1-1. Nooksack River Delta and Vicinity

2-2 HYDROLOGY AND HYDRAULICS

The Nooksack River watershed (Figure 2-2-1) covers 950 square miles ranging from sea level up to the glaciers of Mt. Baker, a Cascades mountain range peak of 10,781 feet. In the upper watershed, three main forks converge before the river enters low gradient, agricultural lowlands. The Lummi River, located in the lowland estuary, was the main discharge route for the Nooksack Watershed until the mid 1800s (Collins and Sheikh 2003). The Lummi River is now mostly disconnected from the Nooksack River and only receives occasional limited flows from the Nooksack through a partially collapsed culvert. In the area of the junction between the two rivers, the active sedimentary environment has resulted in a grade difference between the Nooksack and the Lummi Rivers, making the reestablishment of a hydraulic connection difficult. The Nooksack River currently discharges to Bellingham Bay while the Lummi River discharges to Lummi Bay. Estimated annual rainfall averaged over the watershed is 78.5 inches.

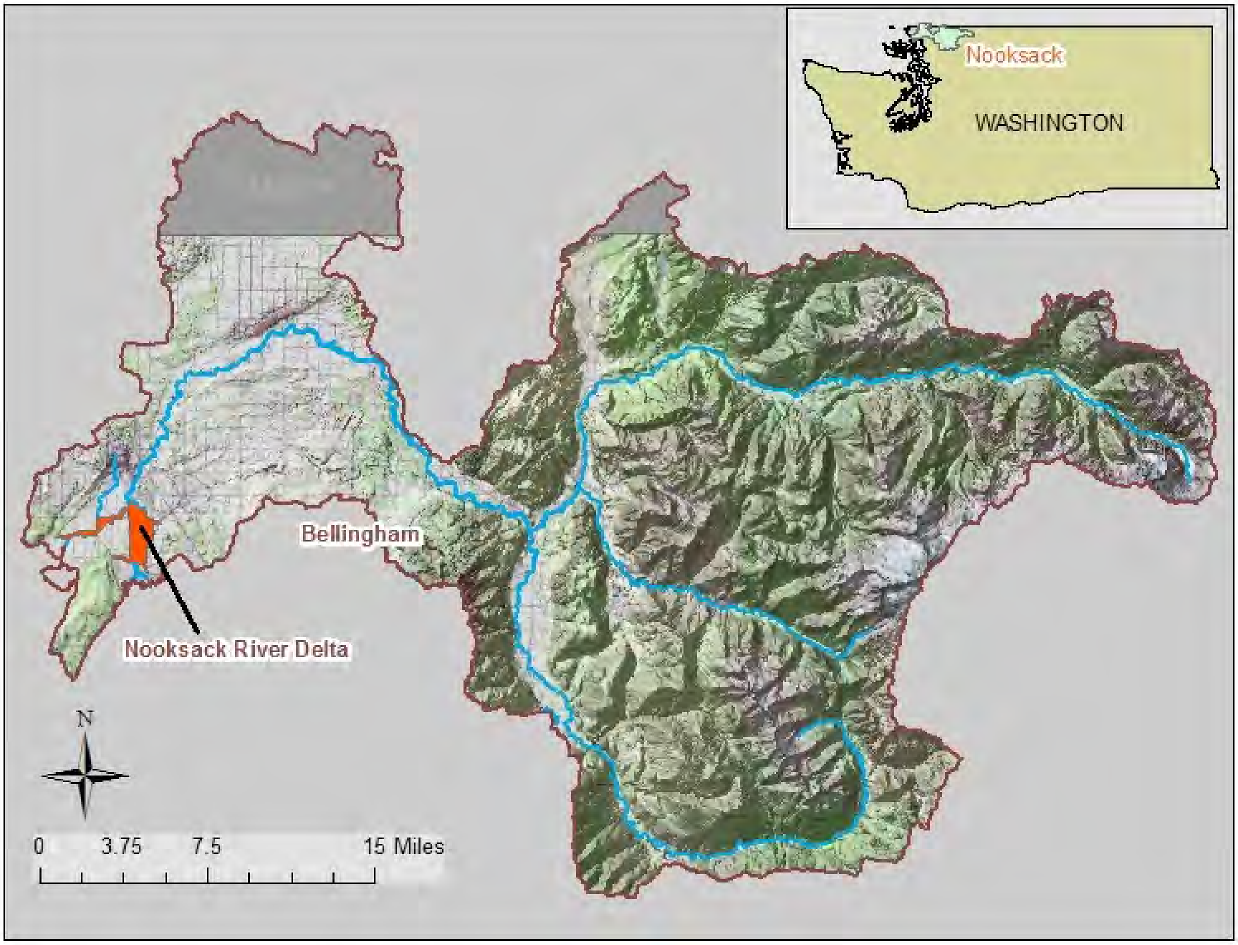

Figure 2-2-1. Nooksack River Watershed

The project site encompasses portions of the Nooksack and Lummi River Estuaries downstream of the city of Ferndale, Washington. It covers parts of the Lummi Nation lowlands as well as agricultural land south of Ferndale. Almost the entire project area lies below the 100-year flood elevation. The hydraulic intent at this site is to restore aspects of natural river and tidal flow to the Nooksack and Lummi Estuaries. The proposed work at this site is intended to increase the frequency of flooding in uninhabited riparian areas. Flood impacts will be mitigated by use of levee setbacks, raised roadways and the installation of flow control structures. The restoration is not anticipated to affect the 100-year or 500-year return interval flooding. No new riverside levees are planned for this site; however, new setback levees are planned. Since the planned setback levees and the setback of the North Red River Road may alter the flooding pattern from riverine and coastal flooding in the estuary, there may be some net changes in flowage easements.

The hydraulics and hydrology for all restoration sites in the Puget Sound Nearshore Ecosystem Restoration Project were evaluated using an area of potential hydraulic effects specific to the construction requirements for each particular site. The upstream and lateral limits for this area represent the 100-year base flood elevation derived from a combination of Federal Emergency Management Agency (FEMA) flood maps and Flood Insurance Studies as well as U.S. Army Corps of Engineers (USACE) base flood elevation determinations. Downstream and seaward limits are based on changes in shoreform type and best professional judgment.

Figure 2-2-2 shows the area of potential hydraulic effects for the Nooksack River Delta. The upstream and lateral limits were set according to the 100-year base flood elevation as determined by the FEMA Flood Insurance Rate Maps for unincorporated Whatcom County, community 53073C (revised 2004). The seaward limit was taken as the downstream extent of most estuarine sediments visible on aerial photographs. The base flood elevation as determined by FEMA ranges from approximately 12 feet (NAVD88) near Bellingham Bay and Lummi Bay to approximately 25 feet (NAVD88) at the junction of the Lummi and Nooksack Rivers, a distance of about 4.6 miles.

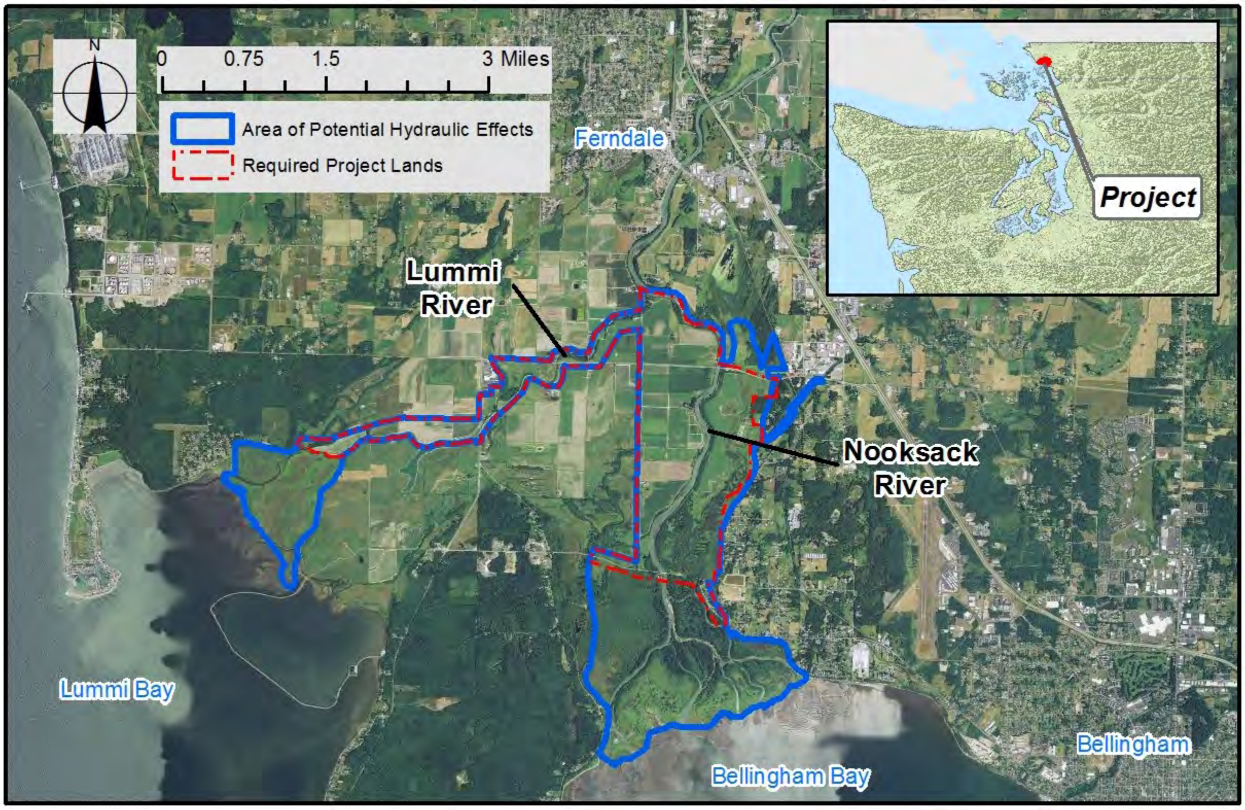

Figure 2-2-2. Nooksack River Delta: Area of potential hydraulic effects

The Ecosystem Output Model (EOM), described in Appendix G utilized an area of restored process determined as follows:

The upland portion of each analysis area was delineated to ensure that the area included all stressor distributions within defined buffer distances from the shoreline. In the aquatic areas, the shape of the Analysis Area was determined by a combination of:

- The GIS area provided initially by the design team and the associated parcel map for the proposed action
- Ensuring an area encompassed all delineated tidal wetlands
- For any Analysis Area that extended through an aquatic area, boundaries were established approximately perpendicular to the shoreline orientation where the upland meets the shoreline.

The area of restored process at Duckabush is shown in Figure 2-2-3 as 1807 acres. For more information, please refer to Puget Sound Nearshore Ecosystem Restoration Project Fish and Wildlife Coordination Act Section 2(b) report in Appendix J, Environmental Compliance Documentation.

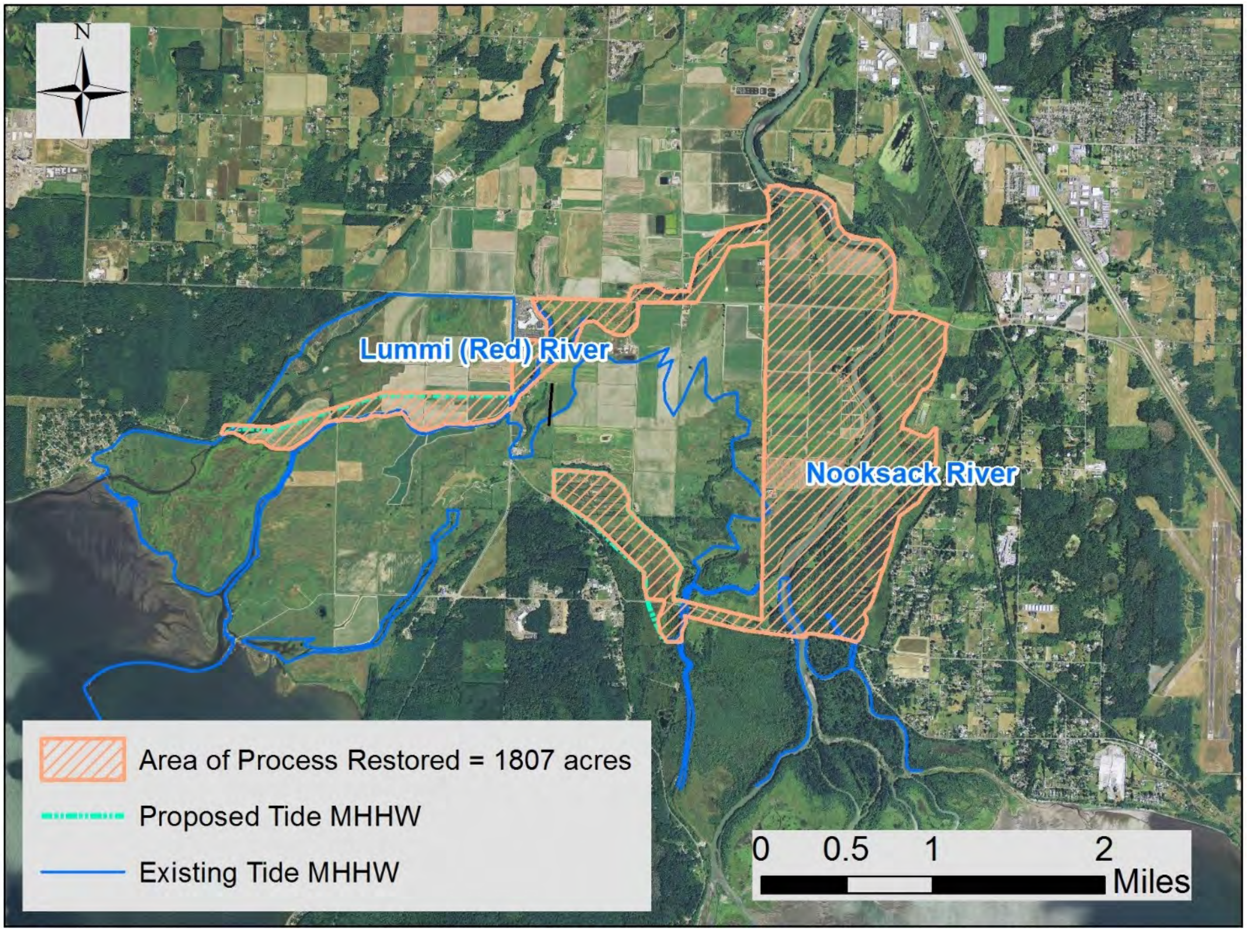

Figure 2-2-3. Area of process restored used in ecosystem output model at the Nooksack River Delta.

The Nooksack River Delta has three leveed areas listed in the National Levee Database (shown in Figure 2-2-4):

- The Ferndale/Nooksack levee system consisting of Rainbow Slough, Rayhorst, Sigardson, Ferndale Water Treatment Plant and Ferndale levees along with high ground, Haxton Way and the North Red River Road provide flood risk reduction to the largest area.
- The Red River Levee provides flood risk management to the Lummi Delta and is entirely on Lummi Tribal Lands.
- The Hovander Park and Dean Foods levee areas are located on the left bank of the Nooksack River. The Dean Foods levee is no longer maintained and has been abandoned as a levee. The land behind and including the Hovander-Dean Foods levees up to high ground has been purchased as a conservation easement by the Washington Department of Fish and Wildlife.

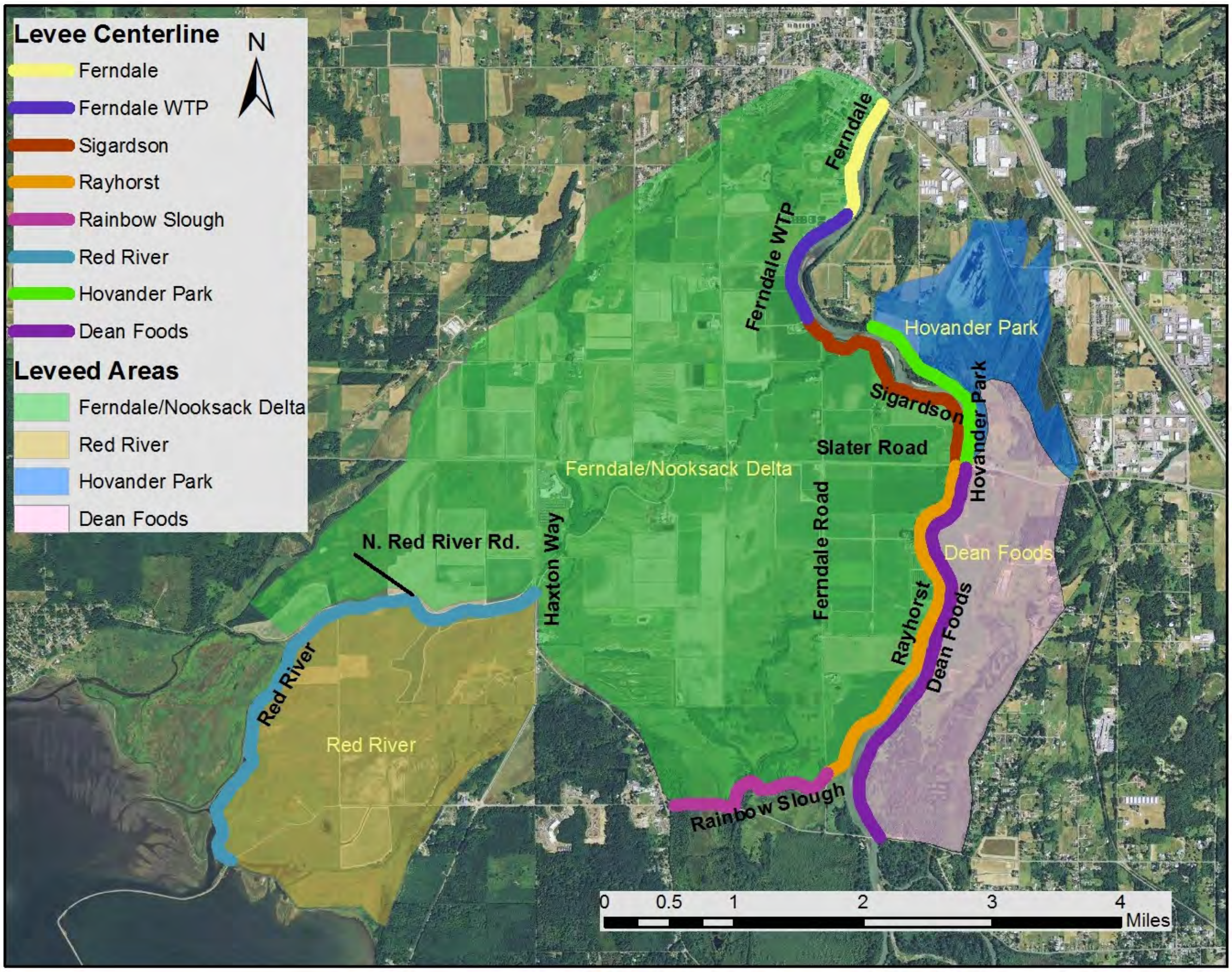

Figure 2-2-4. Leveed Areas in the Nooksack River Delta (Source: National Levee Database)

Table 2- 2-1 summarizes the levees in the Nooksack Delta. The level of protection is given, where available, as the Annual Exceedance Probability (AEP) for the overtopping flow. The Levee Screening Action Classification (LSAC) for all right bank levees in the Nooksack Delta is “4” or “low risk warranting priority actions to reduce risk.”

Table 2- 2-1. Levee details for Nooksack Delta. (Sources: Corps Levee Screening, Whatcom County)

Levee System	Levee Name	LSAC	PL 84-99	Residual Risk AEP (USACE)	Residual Risk AEP (Whatcom Co.)	% Area Inundated > 2 ft
Ferndale/Nooksack Delta						
	Rainbow Slough	4	Y	10%	<1%	82%
	Rayhorst	4	Y	10%	10% - 20%	83%
	Sigardson	4	Y	10%	2% - 4%	94%
	Ferndale WTP	4	Y	20%	<1%	99%
	Ferndale	4	Y	20%	4% - 10%	98%
	Red River Road	NA	N	≥ 20%††	NA	NA
Red River						
	Red River	4	Y	20%	NA	100%
Hovander/Dean Foods						
	Hovander Park	NA	Y	10%	> 20%	100%
	Dean Foods	NA	N	NA	> 20%	NA

†† Assumption is the same or less than Red River Levee which was built at the same time.

NA = Not Applicable.

2-2.1 Functional Design Requirements

This section describes the hydrologic and hydraulic setting for the site and the intended hydraulic consequences of the design features.

2-2.1.1 Consequences of flows exceeding discharge capacity of the project

The purpose of this site is to restore aspects of natural tidal flow and sediment transport to the Nooksack and Lummi Estuaries, allowing the reestablishment of portions of a distributary channel system. Water control structures include setback levees and an engineered diversion structure controlling flow at the junction of the Nooksack and Lummi Rivers. These structures will be designed to convey discharges equivalent to the current capacities of the existing levees. Flows in excess of these discharges may result in local flooding of areas adjacent to and downstream of these structures. These consequences will be assessed during Project Engineering and Design (PED).

2-2.1.2 Project-induced changes obligating mitigation

No compensatory mitigation is included for this site as none is required. Implementation of restoration at this site would involve only minor construction activities in the aquatic environment. The restoration actions would have negligible, short-term construction related effects. All of these minor and temporary effects can be avoided and minimized through construction designs and standard best management practices (BMPs). Specific measurable and enforceable measures would be developed based on the specific effects of the project.

2-2.1.3 Discharge-frequency relationships

Predictions for river discharge-frequency relationships are available from multiple sources. A Flood Frequency Analysis (FFA) prepared by WEST Consultants Inc. for USACE in 2011 gives the most conservative estimates. This source has the longest period of record. This study used U.S. Geological Survey (USGS) gage readings taken from the Nooksack River at Ferndale (USGS 12213100) between 1945 and 2010. The Ferndale gage is about 1.5 miles upstream of the head of the Lummi River. Estimates are shown in Table 2-2-2. Also included are discharge-frequency estimates from a 2004 FEMA Flood Insurance Study for Whatcom County (53073CV000A). Discharges from this study are adjusted for overflow losses at Everson. Overbank losses to the Sumas River, to the north, are significant for events greater than 10 years (FEMA 2007). Another study by Franz (2005) looked more closely at the overbank losses. The two studies that consider losses to the Sumas River predicted lower discharges than the traditional Flood Frequency Analysis. For the purposes of analysis, the higher estimates from the USACE Flood Frequency Analysis study are included as a conservative assumption. The hydrology for the Nooksack River Delta will be reviewed in PED.

Table 2-2-2. Peak Discharge-Frequency predictions for the Nooksack River near Ferndale

Method	10% AEP (cfs)	2% AEP (cfs)	1% AEP (cfs)	0.2% AEP (cfs)
USACE FFA 2011	41,100	58,600	67,200	90,600
FEMA FIS, Whatcom Co. 2007	40,000	48,500	51,000	-
Franz 2005 Study	39,600	56,700	60,500	70,000

2-2.1.4 0.2% Annual Exceedance Probability (500-year return interval) flood

The area of potential hydraulic effect for the Nooksack Estuary is dominated in the lower reaches by storm surge from Lummi Bay and Bellingham Bay, and in the upper reaches by fluvial flows. Table 2-2-3 summarizes the 0.2% AEP hydraulic conditions for the site area. Since work at this site involves the construction or modification of several bridges on major roadways (Ferndale Road, Imhoff Road, Slater Road [2 bridges], Hillaire Road, and Haxton Way), the 0.2% AEP coastal base flood elevations from Lummi and Bellingham Bays will need to be evaluated during PED.

Table 2-2-3. 0.2% AEP hydraulic conditions for the Nooksack Estuary

Flooding source	Elevation (feet, NAVD88)	Discharge (cfs)
Strait of Georgia (BFE)	TBD	-
Bellingham Bay (BFE)	TBD	-
Nooksack River	-	90,600

2-2.1.5 Stage-discharge relationships

Current stage–discharge relationships as reported in Table 2-2-4 are from the FEMA Flood Insurance Study (FEMA 2007) and are based on modeling conducted in 1991. Stage locations are at the Slater Road Bridge and the head of the Lummi River. In order to forecast the new stage-discharge relationships, a 2-D hydraulic model will have to be implemented which reflects the proposed geometry of the estuary including the effects of possible future sedimentation. In certain locations, such as at the flow diversion, a 3-D model or physical model may be required. This will be addressed during PED.

Whatcom County, as part of their Comprehensive Flood Hazard Mitigation Planning (CFHMP) has developed a 1-D unsteady hydrodynamic model for the Lower Nooksack River using the FEQ (Full Equations) model. This model has also been used to examine several restoration options for the lower Nooksack River floodplain using the hydrology developed by Franz (2005). It is difficult to compare the Whatcom County FEQ and FEMA models as they are based on different assumptions for watershed hydrology.

Updated inundation modeling will be conducted in PED using a 2-D hydrodynamic model such as HECRAS to compare with and without project conditions including the effects of possible future sedimentation. hydraulic modeling during PED is being used both to fine tune the design of the levee setbacks, to provide a level of flood risk management equal to the existing level and to ensure that project features are designed to achieve and to continue to achieve the desired ecosystem benefits with minimal maintenance.

Table 2-2-4 Stage discharge relations as shown in 2007 FEMA Flood Insurance Study for the Nooksack River. Elevations have been converted from NGVD29 to NAVD88 using an offset of 3.9 feet.

Location	10% AEP Stage (feet NAVD88)	1% AEP Stage (feet NAVD88)	0.2% AEP Stage (feet NAVD88)
Slater Road Bridge near river mile 3.3	20.8	21.1	21.4
Head of Lummi River near river mile 4.6	25.2	25.8	26.4

2-2.1.6 Flow duration

Flow duration data are available from daily discharge readings near Ferndale (USGS 12213100) for the period between October 1966 and the present. An unsteady flow analysis or flood flow routing will likely be required for this site and will be part of the 2-D hydrodynamic modeling conducted in PED.

2-2.1.7 Flood inundation boundaries and flood stage hydrographs

The current flood inundation boundaries as reported for the 100-year flood event in the Whatcom County FEMA Flood Insurance Study are shown in Figure 2-2-5. For clarity, forecast 1% AEP flood elevations have been noted on the FEMA map at the flow split of the Lummi and Nooksack River and at the downstream project limits. In order to forecast the new flood inundation boundaries, a 2-D hydraulic model will have to be implemented which reflects the proposed geometry of the estuary including the effects of possible future sedimentation. The 2-D modeling is in support of design of the levee setbacks and to ensure that project features are designed to achieve and to continue to achieve the desired ecosystem benefits with minimal maintenance. A 3-D model or possibly a physical model may be required to assess the hydraulics of the diversion structure for design purposes. It will be used only if this structure is retained as a site feature and site conditions and operational considerations require it. This will be addressed during PED.

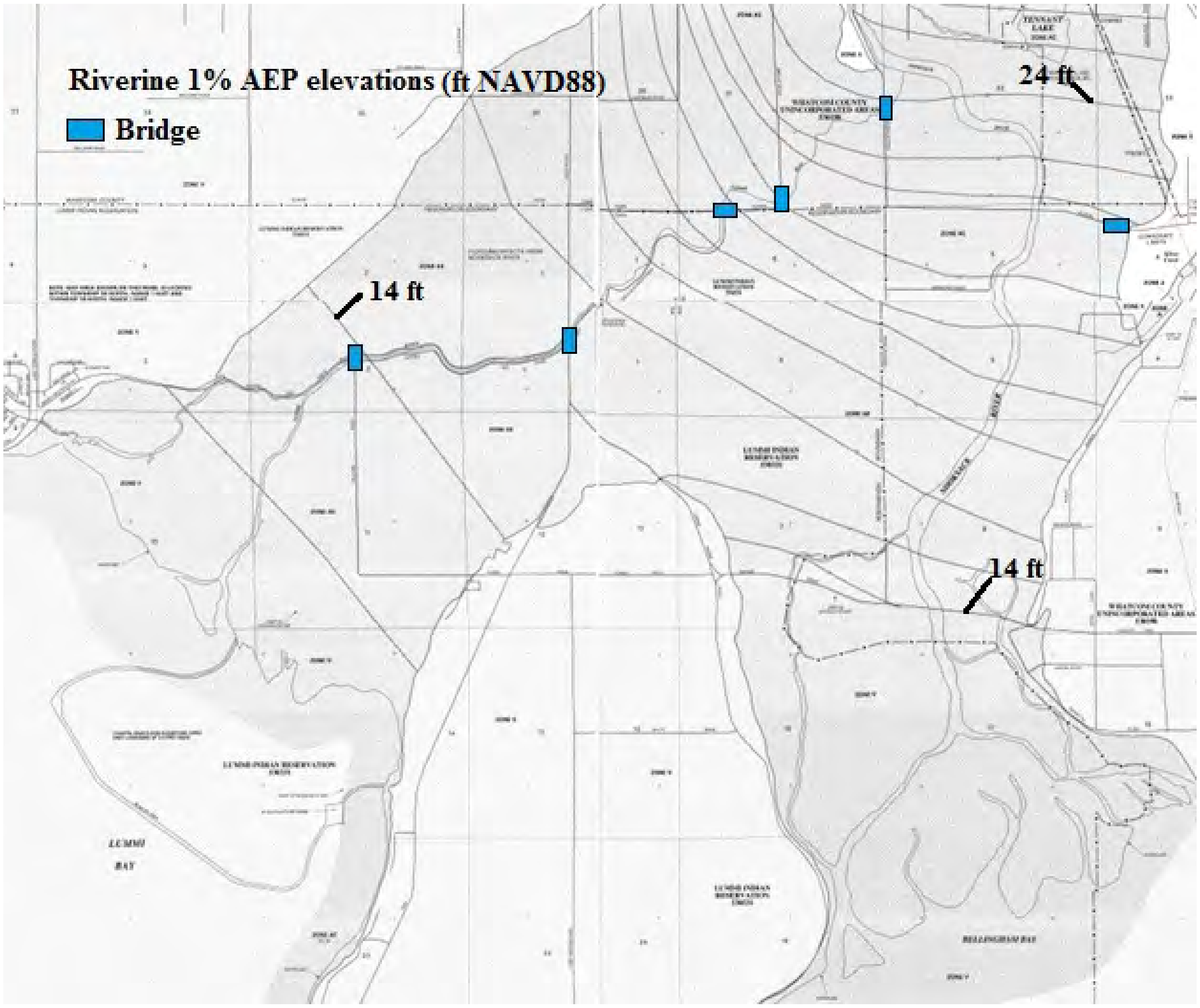

Figure 2-2-5. FEMA 1% AEP flood zone from Flood Insurance Rate Map. Aggregated from map numbers 53073C-1160D, -1170D, -1180D, and -1190D (FEMA 2004). Note: Maps are based on 1991 modeling.

2-2.1.8 Reservoir yields

No reservoirs are planned as part of this site. (Not applicable.)

2-2.1.9 Risk and uncertainty analysis for sizing of the project under study

Channel sizing

Given the complexity of this restoration, none of the empirical channel sizing equations are appropriate for the channel sizing design. A 2-D hydrodynamic model should be used to determine the appropriate channel sizing and configuration. In certain locations, a 3-D model may be appropriate. This will be addressed during PED.

Sea Level Change

The Nooksack River Delta is located in the San Juan Islands – Georgia Strait Sub-basin of Puget Sound. Sea level change calculations for the San Juan Islands – Georgia Strait Sub-basin are based on the Friday Harbor tide gage and are calculated using the guidance in ER 1100-2-8162, Incorporating Sea Level Change in Civil Works Programs, and ETL-1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses and Adaptation (USACE 2013, 2014). Table 2-2-5 shows the range of sea level change projections for the 50-year project life as well as the 100-year horizon assuming a project start date in 2020. Changes are referenced to 1992, which is the midpoint of the most recent National Tidal Datum Epoch as established by NOAA. The high rate calculations indicate a sea level rise of 2.2 feet in 50 years after project start and a rise of 6.2 feet after 100 years.

The largest risk associated with sea level change at this site is the displacement of habitat upstream, with vegetated marshes becoming intertidal habitat and intertidal habitat becoming sub-tidal habitat. Tidal marshes can adapt to sea level change by building elevation to keep pace with the rising water levels, but this requires an adequate supply of sediment and/or organic matter accumulation. Future studies should include a sedimentation analysis to determine what impact the restoration will have on sedimentation rates and if there is sufficient sediment accumulation to keep pace with the projected sea level change.

Table 2-2-5. Projected Sea Level Change (feet) Friday Harbor (Gage 9449880). Source: USACE Sea-Level Change Curve Calculator (2015.46).

Year	Low (feet)	Intermediate (feet)	High (feet)
1992	0	0	0
1995	0.01	0.01	0.01
2000	0.03	0.04	0.05
2010	0.07	0.1	0.19
2020	0.1	0.17	0.39
2030	0.14	0.27	0.68
2040	0.18	0.38	1.03
2050	0.22	0.51	1.46
2060	0.25	0.66	1.97
2070	0.29	0.83	2.55
2080	0.33	1.01	3.2
2090	0.36	1.22	3.92
2100	0.4	1.44	4.72
2110	0.44	1.68	5.6
2120	0.48	1.93	6.55

For levees, the project is designed to meet the current level of residual risk which, at Nooksack, varies from 20% AEP to 2% AEP depending on the levee. Since the project is not for flood control, future adaptation of levees for sea level change is not within the project authority. The levee system at the Lummi River Delta includes long segments of coastal dikes which would be affected by sea level change well before the majority of the setback levees in the project. It is assumed that adaptation of the levee system to sea level change would be undertaken either by the Lummi Nation, individual diking districts or under a separate authority. Elements of the levee design that will be finalized in PED will include robustness considerations such as shallow side slopes and wide crest-widths. Some of these features may be usable by others to adapt to future sea level change. The Lummi River setback levees, as currently estimated, range in crest elevation from 15 to 20 feet NAVD88. This implies that the most seaward extents of the levees will begin to be overtopped by the base coastal flood by 2078 (about 50 years after construction) for the high rate of rise assumption and by 2155 (about 130 years after construction) for the intermediate level of rise. The Nooksack River setback levee, as currently estimated, ranges in crest elevation from 20 to 24 feet NAVD88. The base coastal flood elevation will begin to exceed the most seaward extents of the levee by 2135 (about 105 years after construction) for the high rate of rise assumption and by 2270 (about 240 years after construction) for the intermediate level of rise. However, since the levee is primarily governed by river flooding, overtopping events will likely occur sooner than those times, as river flows encounter a higher coastal backwater condition. Levee performance and expected effects of sea level change will be reassessed in PED once hydraulic modeling is completed and levee crest elevations have been refined.

For bridges, one bridge replacement in the Nooksack Site (Hillaire Road) would be affected by the 100-year intermediate level of rise of 1.7 feet. This level of rise puts the coastal base flood elevation (BFE) at the Hillaire Road Bridge at approximately the same elevation as the current FEMA forecast 1% AEP riverine flood level. As a result, an intermediate rate of sea level change assumption will not significantly affect the bridge design. For 100-year high assumption, Haxton Way and Hillaire Road Bridges would be affected. Bridges designed with the 100-year intermediate SLC assumption would incur higher maintenance costs as the 3 foot debris clearance below the bridge would no longer be available and water surface could inundate the lower foot of the bridge superstructure. Bridge design during PED should confirm that the bridges will be able to withstand debris impacts and up to one foot of girder inundation. These conditions are summarized in Table 2-2-6.

Table 2-2-6. Controlling water surface in design of bridges at the Nooksack site.

SLC Condition	Elevation of coastal BFE (NAVD88)	Bridge clearance criterion
Current BFE	12.2 ft (Nooksack R.) 12.5 ft (Lummi R.)	All 6 bridges governed by riverine flooding
BFE + 100-year Low SLC	12.9 ft	All 6 bridges governed by riverine flooding
BFE + 100 year Intermediate SLC	14.2 ft	Hillaire Road Bridge marginally governed by coastal BFE (14.2 ft vs 14 ft)
BFE + 100 year High SLC	18.3 ft	Haxton way and Hillaire Road Bridges governed by coastal BFE

Figure 2-2-6 shows the land elevations that fall within the three sea level changes assumptions for coastal BFE. Each color indicates the additional area inundated from Lummi or Bellingham Bay for each successively higher SLC assumption. The elevations shown in the figure are based on 2011 Lidar from the Puget Sound Lidar Consortium (PSLC 2011) and indicate land elevation only. Figure 2-2-6 is not an inundation map. It does not show the influence of riverine inundation or the effects of roadways, flow control structures or levees.

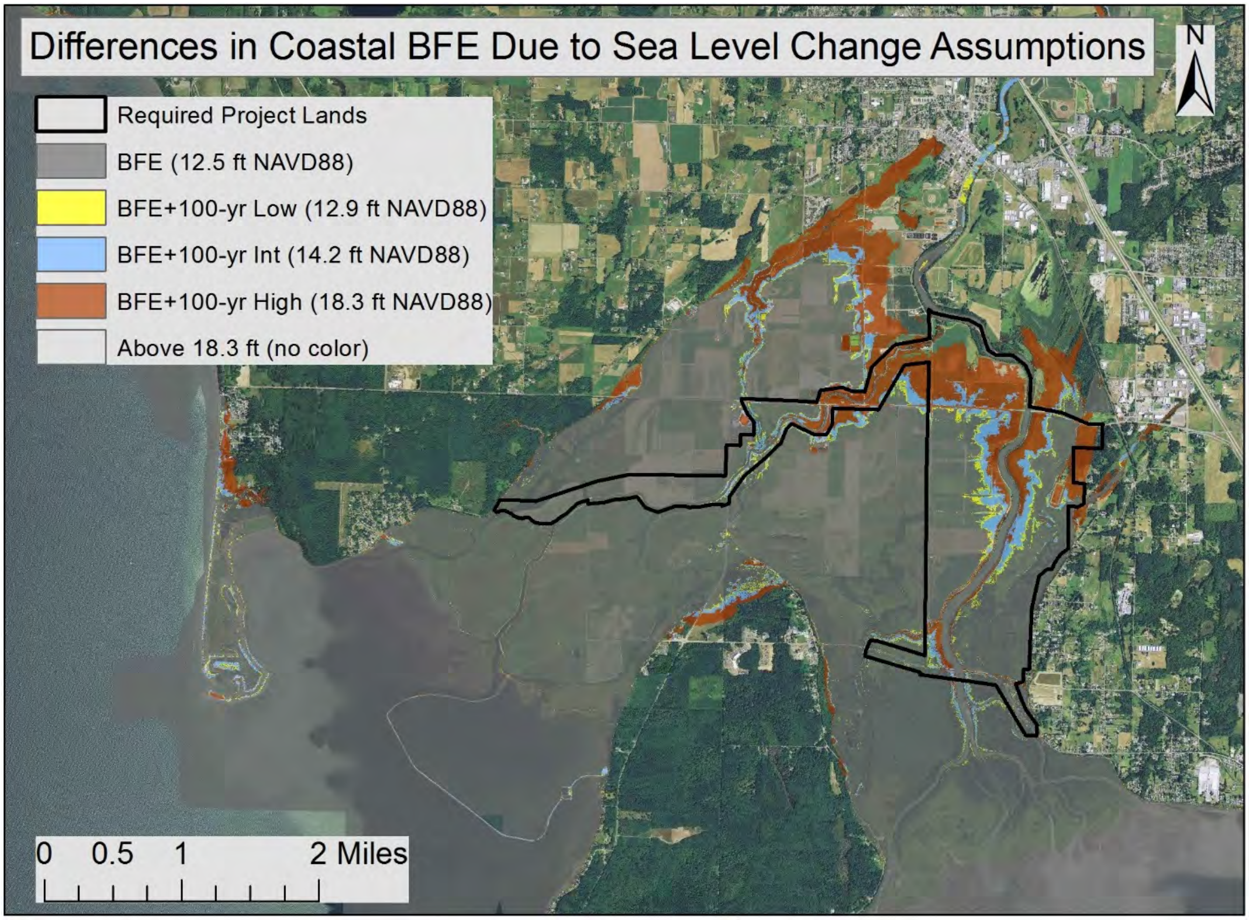

Figure 2-2-6. Land at or below coastal BFE elevation for present day and projected 100-year low, intermediate and high rates of sea level change.

USACE hydraulic analysis has not been carried out for this site and the riverine BFE has not been established for the three rate of rise assumptions. Most of the land in the Nooksack River delta lies below the Coastal BFE elevation with the exception of the Lummi peninsula and areas adjacent to existing levees.

Figure 2-2-7 shows the land elevations that fall within the three sea level changes assumptions for Mean Higher High Water (MHHW). Figure 2-2-7 is not an inundation map, since USACE hydraulic analysis has not been carried out for this site. The elevations shown in the figure are based on 2011 Lidar from the Puget Sound Lidar Consortium (PSLC 2011) and indicate land elevation only – not the effects of levees. The land on the east side of the Nooksack River delta lies above current MHHW level, while the land on the west side of the delta is mostly below the current MHHW elevation. This map supports the assumption from Section 2-4.1.11 that land subsidence has likely occurred in the diked agricultural lands, as the highest elevations are immediately adjacent to the levees. Properties in the delta do not flood under high tide conditions because of the system of coastal dikes and riverine levees that line the delta. Under the largest sea level change assumption (100-year high), MHHW would extend inland along the banks of the Lummi River channel as well as inland along the banks of the Nooksack River Channel.

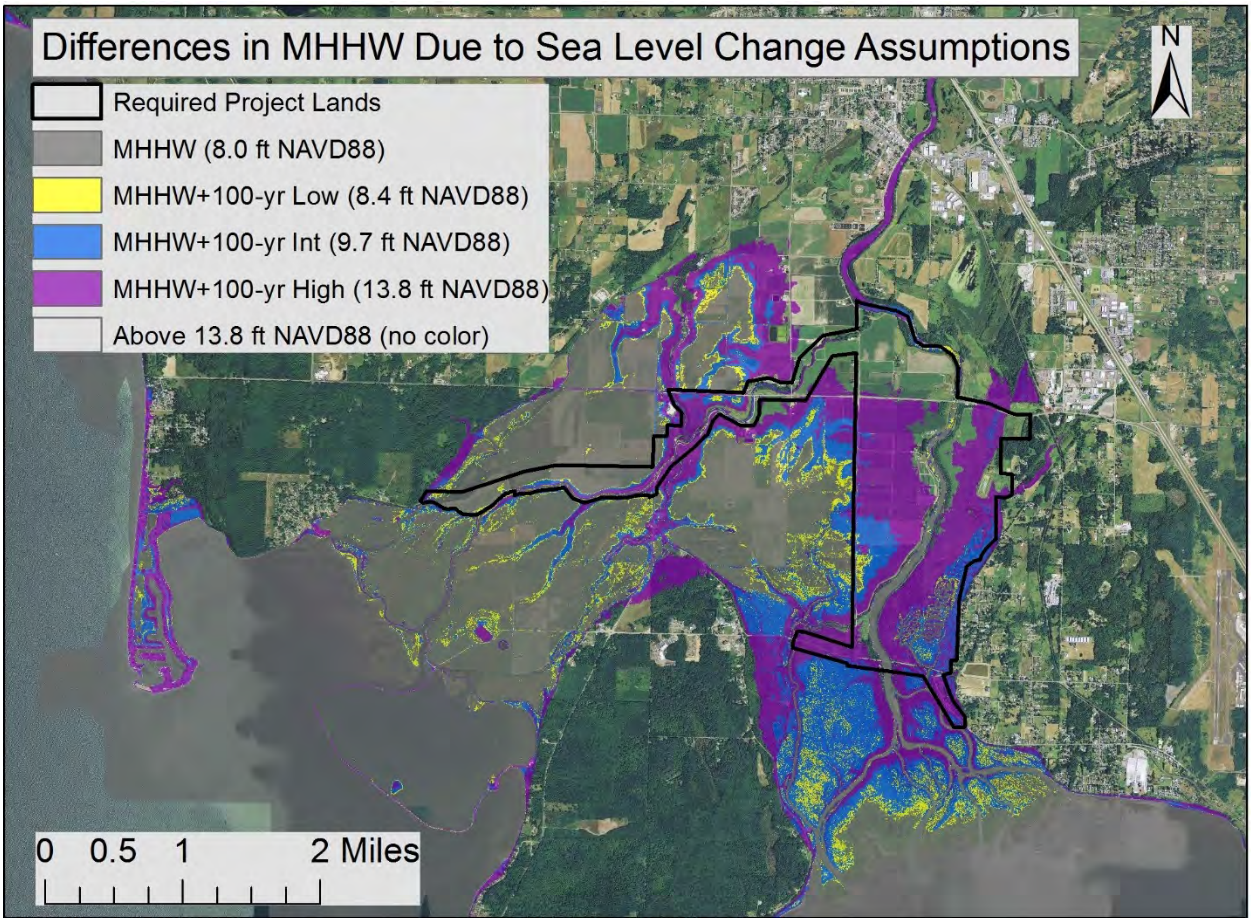

Figure 2-2-7. Land at or below MHHW for present day and projected 100 year low, intermediate and high rates of sea level change.

Climate Change

ECB No. 2014-10 (USACE 2014) provides initial guidance for incorporating climate change information in hydrologic analyses in accordance with the USACE overarching climate change adaptation policy. There is a strong consensus among recent studies that future storm events in the Pacific Northwest region will be more intense and more frequent compared to the recent past (USACE 2015). The overall projected trends for the Pacific Northwest are summarized in the FR/EIS section 3.6.5.1.

Streamflow in the Nooksack River responds to local climate variations and typically has higher precipitation-driven flows in the fall and early winter with a secondary peak from spring snowmelt. Using 3 climate scenarios downscaled from Global Climate Models (GCMs) and the Distributed-Hydrology-Soil-Vegetation model Dickerson-Lange and Mitchell (2014) modeled the potential impacts of climate change on Nooksack River hydrology. Simulations of future streamflow in the Nooksack River project increases in median winter discharge ranging from34% to 60% by the year 2050. Projected decreases in summer flows are on the order of 20% to 30%. Additionally the spring melt peak is forecast to shift from June to May by 2050 eventually leading to a one-peak annual hydrograph for the Nooksack more typical of rain dominated basins.

Dickerson-Lange and Mitchell conclude:

> "With a growing area contributing to runoff and less snow to attenuate winter rain as temperatures warm, the frequency and magnitude of Nooksack River floods will likely increase. Currently, the flood risk declines later in the winter season because the area contributing to runoff shrinks and the snowpack reaches a threshold thickness for attenuating rainfall. Due to future warming, the basin will have a reduced snowpack for a longer period of time; our results support

a shift from November into December or January as the dominant time period for flood risk. Our results also show an increase in annual flood peak magnitudes due to larger precipitation extremes projected by the GCMs in the winter months".

As an example, one of the conclusions from the study is that a 10% AEP flood discharge may have an AEP of 33% by 2050. While the Nooksack River is projected to exhibit similar trends to other Western Washington Rivers, the Nooksack is anticipated to change more quickly and to a greater extent than rivers in the southern part of Western Washington. Of particular concern is the loss of glacier melt contribution to streamflow during the low flow season, with projected decreases in snowpack from 36% -71% below elevations of 1000 m, 25-63% at 1000 – 2000 m and 15%-54% above 2,000 m (Grah and Beaulieu 2013).

Higher winter flows, especially flood discharges, could result in an increase in sediment transport. The Nooksack River carries an annual sediment discharge of about 1,400,000 tons per year at a mean annual discharge of 3,200 cfs (Czuba 2011). Compared with the other main contributor of sediment in Puget Sound, this is half the sediment discharge but 1/6 of the flow. Should climate change result in higher winter discharges becoming more common, the sediment yield may increase accordingly. Conversely lower summer flows may slow the movement of sediment out of the river system. The potential impacts to river deposition are difficult to estimate. That will depend on the future balance between sediment transport potential and the available sediment supply. The sedimentation analysis conducted during PED will provide an indication of whether more frequent high flow events will trend towards higher rates of sedimentation at the Nooksack site. The results of the analysis will determine final setback levee elevations, predict future stage-discharge relations, channel slope and inundation limits as well as assess the environmental effects of changes in sedimentation and the requirements for operations and maintenance.

2-2.1.10 Water quality conditions

No water quality information has been reviewed for this site. The anticipated water quality effects are as follows:

- Construction-related turbidity and suspension of sediments may occur due to fill removal, construction of new embankments, installation of water control structures, filling of ditches, and excavation of new channels.
- Temporary increases in sedimentation may occur in Lummi Bay because of the release of sediment currently impounded upstream and because of the evolution of the distributary channel system. The work at this site proposes to increase sedimentation in the Nooksack Estuary downstream of the junction with the Lummi River, which may also affect water quality. These effects, together with other sedimentation issues, should be evaluated during PED.
- The quality of water from the Nooksack River watershed presents a significant design consideration. Fecal coliform bacteria loading from the Nooksack River adversely impacted Portage Bay to the point that shellfish harvesting was halted over the 1996 to 2006 period. Recent trends in fecal coliform densities may argue against sending additional water into the Lummi River and Lummi Bay, due to the resultant potential closure of Lummi Bay shellfish beds to ceremonial, subsistence, and commercial harvest.

2-2.1.11 Groundwater conditions

No groundwater information has been reviewed for this site. The restoration proposes to alter both flood-related and non-flood-related hydraulic grade lines of flows in both the Lummi and Nooksack Rivers, which may have consequences for groundwater. The extent of freshwater seepage into the estuary and the character of aquifers in the area have not been assessed. Many of the properties in the estuary area are most likely on septic systems. A review of the potential effects on water wells, septic systems, and groundwater seepage will be carried out during PED.

The planned work at this site will allow an increased tidal prism upstream of current limits, which can be accompanied by saltwater intrusion into the hyporheic zone. Since the goal is to restore the historic function of the estuary, restoration of historic salinity patterns is presumed to be a desirable outcome.

2-2.1.12 Preliminary project regulation plan

The primary water control structure at this site is the engineered diversion structure at the junction of the Lummi and Nooksack Rivers.

According to USACE design drawings from 1950, a diversion structure installed at that time consisted of a V-shaped weir/orifice installed within the levee between the Nooksack River and the Lummi River. The weir/orifice was accessed by an open channel leading from the Nooksack River and included baffle blocks for energy dissipation on the downstream (Lummi River) side. Due to excessive sedimentation in the inflow channel, it appears that, some time later, this diversion was retrofitted with an 80 foot long culvert pipe leading from the Nooksack River to the Lummi River. The pipe inlet is perched above the Nooksack River water surface until flows reach about 9600 cfs. In addition, the pipe had been damaged and is partially collapsed, severely limiting the amount of water that can pass at flows above 9600 cfs. The flow requirements for the Lummi River for salmonid use are about 200 cfs, with the possibility for somewhat lower flows during the fall months (USACE 2000).

No significant details have been developed for the new flow diversion structure. Since one of the requirements for the engineered diversion structure is that it will limit flows into the Lummi River to 200 cfs, this will be either an underflow structure, a structure with a controlled crest height or possibly a controllable culvert. The structure design, as well as a regulation plan, if necessary, will be addressed during PED. The remaining water control structures at this site are passive (levee setbacks, culverts).

2-2.1.13 Preliminary Real Estate taking line elevations

The current real estate limits are delineated by the construction area, staging areas, and access roads and do not include the entire potential area of hydraulic effects. Real estate assumptions, valuations, and planning documents have been appropriately scaled for the current level of design.

In the case of the removal or modification of flow controls such as levees, roads, bridge openings, and culverts, the restoration will likely cause a reduction in backwater effects during high river flows, thus altering current flood patterns. These changes will also allow the tidal prism to travel further upstream, increasing tidal effects. In order to forecast the hydraulic effects of the site and to refine the real estate taking line elevations, a 2-D hydraulic model will have to be implemented that reflects the proposed geometry of the completed work. In certain locations, such as at the flow diversion, a 3-D model or physical model may be required. This will be addressed during PED.

As additional surveys, modeling, and design are completed during the PED phase, the real estate documentation will be modified accordingly. For the current real estate status, refer to the Final Feasibility Report/Environmental Impact Statement (FR/EIS), Appendix C, *Real Estate Plan*.

2-2.1.14 Criteria for facility/utility relocations

The hydraulic impacts from relocation of utilities have not been evaluated for this site. Bridge replacements, road abandonment and relocation, and channel alterations will likely all require the relocation of utilities. In addition, the construction of the engineered diversion structure at the junction of the Nooksack and Lummi Rivers may also require the routing of utilities to the new facility. Criteria for these activities will be evaluated during PED.

2-2.1.15 Criteria for identification of flowage easements required for project function

As discussed in Section 2-2.1.13, the planned work at the Nooksack River Delta will alter both the flooding pattern from river flows and the tidal elevations in the Nooksack River and Lummi River estuaries. Although these effects are not anticipated to affect the site function, there may be some net changes in flowage easements. In addition, the planned levee setbacks and the removal and setback of the North Red River Road may also require changes in flowage easements. In order to identify the flowage easements, a 2-D hydraulic model will have to be implemented which reflects the proposed geometry of the levees and roadways including potential future sedimentation. In certain locations, a 3-D model or physical model may be required. This will be addressed during PED.

2-2.1.16 Criteria in support of project OMRR&R requirements

Monitoring needs associated with the hydraulic function of the site include the following:

- Water control structures such as the engineered diversion structure at the junction of the Lummi and Nooksack Rivers require monitoring and maintenance to ensure that they are operating as designed. Operation of the engineered diversion structure is discussed in Section 2-2.1.12 and will be addressed during PED.
- Roadway embankments and slope protection on levee setbacks should be monitored for signs of instability or scour at an interval to be determined during PED.
- Bridge abutments and piers will require periodic inspection to ensure that channel migration is not affecting them and that any scour or slope protection is functioning as designed.
- Project areas in the Nooksack estuary will require periodic monitoring to observe whether excessive erosion or sedimentation is occurring that affects either habitat or properties.
- Salinity and pollutant monitoring in the estuary should be carried out to confirm no significant impacts to water quality.

2-2.1.17 Environmental engineering considerations

In the context of hydrology and hydraulics, environmental engineering is taken to mean water supply and sanitation.

Water Supply

Numerous water supply lines are assumed to exist throughout the entire site. Rerouting of water lines will need to be coordinated with local landowners and utilities. Location, depth and possible groundwater impacts to any wells in the area of potential hydraulic effect will be reviewed during PED.

Sanitation

The properties in the site area are assumed to be on septic systems. The extent to which changes in tidal prism will affect leach fields and groundwater flow will be addressed during PED.

2-2.2 Residual Flooding Consequences – With Project Flooding

This section discusses the predicted hydraulic conditions after construction of the proposed restoration.

2-2.2.1 Warning time of impending inundation

The closest USGS gage for this part of the watershed is at Ferndale (USGS 12213100), approximately 1.5 miles upstream from the upstream end of the site at the junction of the Lummi and Nooksack Rivers. Aside from regional warnings for possible flooding, no new warning system is planned.

2-2.2.2 Rate of rise, duration, depth, and velocity of inundation

In order to forecast the with-project depths and velocity of inundation, a 2-D hydraulic model will have to be implemented which reflects the proposed geometry of work at the site. This will be addressed during PED. In certain locations, a 3-D model will likely be required. Since the area is quite large, flood routing may be a factor in the rate of rise and flow duration at various locations in the site. Therefore, an unsteady flow analysis or flood flow routing may be required.

2-2.2.3 Historic, 1% and 0.2% exceedance (100-year and 500-year) flood extents

In the past, the Lummi and Nooksack Rivers have occupied channels spanning the entire estuary, transporting sediment and creating the current floodplain. In order to forecast the with-project 100-year and 500-year flood inundation boundaries, a 2-D hydraulic model will have to be implemented which reflects the proposed geometry of work at the site. This will be addressed during PED. See Section 2-2.1.7 for the current 1% (100-year) predicted flood extents.

2-2.2.4 Access and egress problems created by flooding

The restoration will raise the height of many of the roadways and bridges in the area as well as the height of the roads on setback levees. This will increase the reliability of access and egress from the area during floods. Other bridge approaches may be designed to allow for overtopping, so that access may be not be available to those structures during high water events. The selection of reliable routes for flood access/egress will occur during PED and will determine which bridges and roadways are required for emergency access/egress.

2-2.2.5 Potential for loss of life as a result of 10-2.2.1 through 10-2.2.3

The potential for loss of life as a result of the proposed restoration is low and does not represent a substantial change from the current conditions.

2-2.2.6 Identification of any potential loss of public services

The potential for loss of public services as a result of the proposed restoration is low. Since the restoration will raise bridge elevations and roadways, this will reduce the possibility of access issues and utility disruption during floods. Some reduced access to areas of the Lummi River Estuary may occur.

2-2.2.7 Potential physical damages

Potential physical damages that can occur during flooding will be addressed by the hydraulic analyses conducted during PED. This will include impacts due to flooding for property owners in the site vicinity. It will also include an evaluation of the need for scour protection on bridge abutments and piers as well as roadway embankments and levees and it will address the issues of channel stability and sediment outflow from the estuary. Potential physical damages to the water control structures that are planned for the site will be assessed as well.

2-2.3 Project Induced Flooding – Change from Pre-Project Conditions

This section describes the effects of the proposed restoration on flood elevations, flood patterns, and flood frequency.

2-2.3.1 Information categories required by 2-2.2

Flooding in the Nooksack River Delta is controlled by riverine flows in the upper areas and by tides and storm surge in the lower estuary. The proposed work at this site will change the pattern of flooding in the site vicinity and may also change the frequency of flooding in some areas for some high occurrence (low return interval) flood events. Work at the site is not anticipated to appreciably change the 100-year flood limits. In order to forecast the new flood inundation boundaries, a 2-D hydraulic model will have to be implemented that reflects the proposed geometry of the estuary. In certain locations, a 3-D model may be required. This will be addressed during PED.

2-2.3.2 Anticipated frequency of induced flooding

Due to the planned changes in hydraulic grade lines, the proposed work at this site may change the frequency of flooding in some areas for some high occurrence (low return interval) flood events. The restoration is not anticipated to affect the 100-year or 500-year return interval flooding. In order to forecast the changes in flood frequency for different locations, a 2-D hydraulic model will have to be implemented that reflects the proposed geometry of the estuary. In certain locations, a 3-D model may be required. This will be addressed during PED.

2-2.4 Inundation Risk 0.2% Exceedance (500-year Return Interval) Flood

The proposed work at the site is not anticipated to appreciably change the 500-year flood limits. In order to forecast the 500-year flood inundation boundaries, a 2-D hydraulic model will have to be implemented that reflects the proposed geometry of the estuary. In certain locations, a 3-D model may be required. The principal risk for the 500-year flood in the lower areas of the estuary is due to sea level rise (refer to Section 2-2.1.9).

2-2.5 Hydraulic Studies

This section discusses the hydraulic studies, construction considerations, and instrumentation and monitoring needs for the site. The anticipated hydraulic studies at this site are summarized in Section 2-21.

2-2.5.1 Hydraulic roughness determinations

The FEMA Flood Insurance Study lists typical Manning hydraulic roughness values for Whatcom County hydraulic analyses as 0.040 for natural channels, 0.070 for overbanks with dense brush, and 0.100 for overbank areas with trees. Roughness values will be reviewed during PED using engineering judgment, aerial photographs of the site area and, if necessary, fieldwork.

2-2.5.2 Water surface profiles

Current water surface profiles as reported in the FEMA Flood Insurance Study will need to be revised to reflect the proposed changes in the floodplain. In order to forecast the new water surface profiles, a 2-D hydraulic model will have to be implemented which reflects the proposed geometry of the estuary and the planned design and operations of the engineered diversion structure. The effects of storm surge and wind waves will need to be incorporated into the analysis of water surface levels. In certain locations, a 3-D model may be required. The predicted water surface profiles depend on the completion of design and operation plans for the flow control structure at the junction of the Lummi and Nooksack Rivers, which may require both numerical and physical modeling. This will be addressed during PED.

2-2.5.3 Stage-discharge relationships

In order to forecast the new stage-discharge relationships at flow control structures and bridges, a 2-D hydraulic model will have to be implemented that reflects the proposed geometry of the estuary and the planned design and operations of the engineered diversion structure. In certain locations, a 3-D model may be required. The flow control structure at the junction of the Lummi and Nooksack Rivers may require both numerical and physical modeling. This will be addressed during PED.

2-2.5.4 Head loss

The predicted head losses depend on the completion of design and operation plans for the flow control structure at the junction of the Lummi and Nooksack Rivers. Design of this structure may require both numerical and physical modeling. This will be addressed during PED.

2-2.5.5 Flow and velocity

Flow and velocity information from the hydraulic analyses will be used to assess the possibility for sediment transport, scour, and bank erosion in the site area.

2-2.5.6 Structural sizing needed to meet design capacity including slope protection

Sizing of the flow control structure at the junction of the Lummi and Nooksack Rivers may require both numerical and physical modeling. The hydraulic analysis conducted during PED will include the need for slope protection on levees, roadway embankments, and bridge abutments and will address the issue of scour at bridge pilings. Additionally, the need for scour protection from effects of waves and storm surge levee will also be evaluated. Several large woody debris installations are included in the plans for this site. These will need to be designed for size, composition, and stability as part of PED.

2-2.5.7 Water control facilities

The water control facilities planned at this site include levees, culverts, large wood jams, and an engineered diversion structure. Specific designs are not yet formulated for these structures. Design of all these features will be addressed during PED. The flow control structure at the junction of the Lummi and

Nooksack Rivers will require numerical modeling and physical modeling if needed, to complete design and operation plans. The modeling will evaluate the design of the facility and its potential effects on hydraulics and sedimentation as well as determine the need for an energy dissipation facility in conjunction with conveying flow from the Nooksack River to the Lummi River.

2-2.5.8 Energy dissipating facilities

Depending on the design of the diversion structure, an energy dissipation facility or stilling basin may be needed as part of conveying flow from the Nooksack River to the Lummi River. The need for such a structure, its design, and its potential effects on hydraulics and sedimentation will be evaluated during PED.

2-2.5.9 Erosion control requirements

Construction

The planned earthwork for this site does not specify dredging or water-based equipment. Since bridge supports, slope protection, roadway, bank fill material, culverts, and in-channel sediments will be removed, appropriate in-water sediment control measures will need to be used during construction. Any in-water or overwater construction should follow accepted best management practices for both erosion and contaminant control.

With Project

The hydraulic analysis conducted during PED will include the need for erosion control or scour protection on levees, roadway embankments, bridge foundations, and water control structures. No erosion control is anticipated outside of the construction boundaries since the goal is to reestablish natural erosion and sedimentation processes. New and existing slope protection should be monitored for signs of erosion at an interval to be determined during PED.

2-2.5.10 Existing and post-project sedimentation

The planned levee setbacks and the removal of flow obstructions in both parts of the Nooksack and Lummi River Estuary will allow the mobilization of sediments that have been impounded upstream and at the channel margins. Shoreline properties and habitat in and downdrift of the Lummi River Estuary will likely experience some temporary increases in sedimentation as these sediments are transported offshore. Areas in the lower part of the Nooksack River Estuary may also experience changes in sedimentation patterns as a result of the planned changes in flow. Restoration of sediment erosion, transport, and accumulation processes are objectives and considered benefits of the project. The amount and potential areas of sedimentation will be evaluated during PED. Monitoring of sedimentation after construction is addressed in the Final FR/EIS, Appendix E, *Monitoring and Adaptive Management*.

2-2.5.11 Water control and order of work during construction

Most channel excavation, embankment removal, and fill removal will be accomplished with land-based heavy construction equipment. Temporary trestle structures and/or local filling may be required along portions of the proposed bridge alignments to provide access for heavy equipment during construction. Large-diameter casing shoring may be required to keep out water and allow access to the top of the drilled bridge pier shafts. A crane will be required to set the girders in place. The temporary trestle or earth fill can then be removed.

Material from the excavated portions of roadway and levees can likely be used for the fill required in the new roadway approaches and setback levees. However, much of the earthwork will be excavation of lowland areas, requiring substantial bucket dredging to form channels. Substantial offhaul and offsite disposal may be required unless beneficial reuse onsite is identified.

If vibratory extraction methods are used to remove pilings, measures should be taken to minimize the loosening of soil and suspension of sediments into the surrounding waterway. In a sensitive estuarine environment, careful excavation and removal of structures may be required.

2-2.5.12 Criteria for facility/utility relocations

See Section 2-2.1.14.

2-2.5.13 Other facilities to meet project goals

Stormwater detention is discussed in Section 2-6.1.1. No other facilities are planned in order to meet restoration goals.

2-2.5.14 Instrumentation and monitoring

A combination of field surveys and aerial photographs will be used to document biological and physical changes to the landscape. Monitoring data can be used to refine adaptive management and corrective measures, as needed. Some of the key monitoring needs and opportunities are summarized in the table in the Final FR/EIS.

2-2.6 Coastal Studies

Coastal base flood elevations were calculated by FEMA as a part of the Whatcom County Flood Insurance Study. The base flood elevations are calculated by combining the effects astronomical tide (caused by gravitational effects of sun and moon), storm surge (rise in water levels as a result of wind stress and low atmospheric pressure), waves breaking onto the shoreline, producing an additional water level rise at the beach (wave setup) and waves running up the beach (wave runup). The 1% annual exceedance coastal base flood elevations are shown on Figure 2-2-3.

It is assumed that the Nooksack Estuary in Lummi Bay is only subjected to wind waves caused by local winds. Measurements at the nearby Bellingham airport (Figure 2-2-8) show that the maximum wind speeds come from the southerly direction and rarely exceed 30 miles per hour. The fetch length in the southwesterly direction is approximately 8 miles, which could result in wave heights up to 5.5 feet with a period of 7 seconds. The impact of wind waves is generally limited to the outer portion of the estuary; however, this area should be designed to withstand this type of wind wave action. Additionally, Lummi Bay may be more exposed to the northerly winds off the Fraser River than Bellingham Bay which is shielded by the peninsula and Lummi Island. It may be appropriate to develop a wind rose for Lummi Bay for the purposes of work at this site. These issues will be addressed during PED. The influence of wind wave activity, storm surge and wave setup will be evaluated during PED. Wave height is not anticipated to be a significant issue for this project as the site footprint is upstream from the estuarine area that is most affected by wave action.

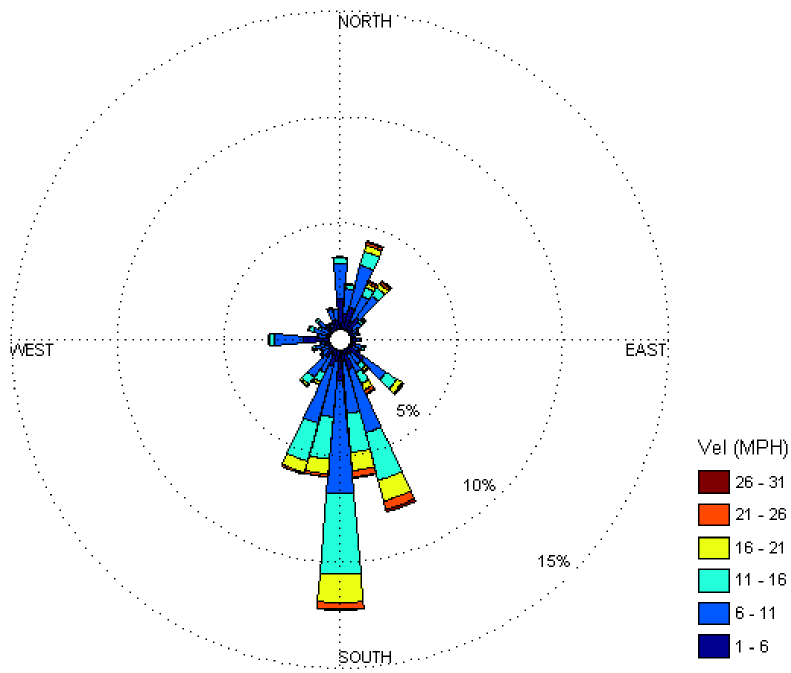

Figure 2-2-8. Wind Rose for Bellingham Airport

Project plans formulated during the conceptual design phase for the Nooksack Delta are based on a Mean Higher High Water tidal datum of 8.03 feet (NAVD88). This datum is from the tide gage at Bellingham (NOAA Gage 9449211). Major tidal datums are summarized in Table 2-2-5. The final design tidal datums will be reviewed and established during PED.

Table 2-2-7. Major tidal datums for Nooksack River Delta, Bellingham (Station 9449211),

Datum Description	Water Level (ft, NAVD88)
FEMA BFE (Coastal)	12.2 – 12.5
FEMA BFE (River)	13-24
Mean Higher-High Water (MHHW)	8.03
Mean High Water (MHW)	7.31
Mean Tide level (MTL)	4.59
Mean Sea Level (MSL)	4.47
National Geodetic Vertical Datum of 1929 (NGVD29)	3.92
Mean Diurnal Tide Level (DTL)	3.77
Mean Low Water (MLW)	1.87
North American Vertical Datum of 1988 (NAVD88)	0
Mean Lower Low Water (MLLW)	-0.48

A summary table for the anticipated hydraulic studies at this site is presented in Section 2-21.

2-2.6.1 Design of coastal shore protection projects (ER 1110-2-1407)

This site does not include coastal shore protection.

2-2.6.2 Effects on adjacent shores

Downstream of the site, the shoreline transitions from a river delta to a bluffed-backed beach. The primary risk is an increase in sediment loading which could affect downstream intertidal and sub-tidal habitats in the river delta portion. At the bluff-backed beach, the primary forcing processes are coastal wind waves and longshore sediment transport which are expected to be minimally if at all affected by the restoration. The effects on downstream and intertidal habitat should be evaluated during PED, using results from similar inlets in Puget Sound.

2-2.7 Navigation Projects

This site does not affect navigation (see Figure 2- 2-9). The nearest shipping lanes are over 4 miles away and separated from the site by Lummi Island.

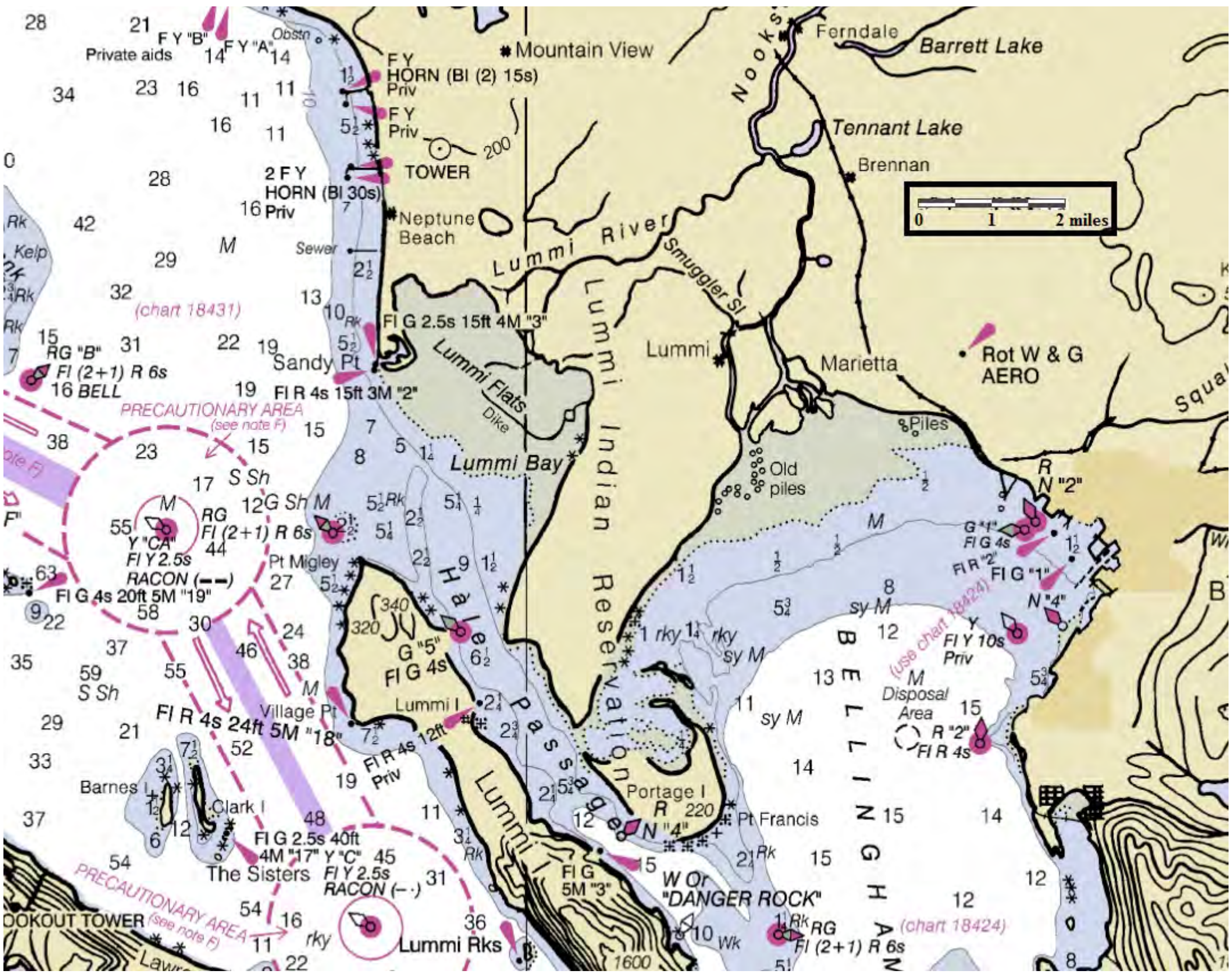

Figure 2- 2-9. Navigation chart for the vicinity of the Nooksack River Delta (Source NOAA RNC Online)

2-3 SURVEYING, MAPPING, AND OTHER GEOSPATIAL DATA REQUIREMENTS

This section describes surveying, mapping, and other geospatial data information to support preparation of the FR/EIS and the *Real Estate Plan* (Appendix C of the FR/EIS). A brief outline of additional surveying and mapping required for subsequent design, plans and specifications, construction, and operations is also included.

2-3.1 Surveying, Mapping, and Other Geospatial Data Information Used

Geospatial data for the Nooksack River site were obtained primarily from remote sensing applications. No site-specific topographic, bathymetric, property, or utility surveys were conducted during the conceptual design phase. LiDAR, aerial imagery, and other geospatial data were used to delineate topographic features, determine surface elevations, and to estimate areas, volumes, lengths, and other dimensions of key features using CAD and/or ArcGIS. High-resolution LiDAR was obtained from the Puget Sound LiDAR Consortium (2005 LiDAR; 3m grid; State Plane projection in NAD83 [horizontal datum] and NAVD88 [vertical datum]; available at http://pugetsoundlidar.ess.washington.edu/lidardata/index.html). The Puget Sound Digital Elevation Model was used for combined bathymetry and topography of the Puget Sound lowland (Finlayson D.P., 2005; University of Washington; State Plane projection in NAD83 [horizontal datum] and NAVD88 [vertical datum]; available at http://www.ocean.washington.edu/data/pugetsound). Recent aerial photography (Whatcom County Planning and Development Services 2004) was evaluated to determine recent site conditions. The conversion from Mean Lower Low Water to North American Vertical Datum (NAVD88) was derived from the Bellingham tide gage (NOS 9449211).

Information on land ownership was derived from the Washington Public Lands Database. Additional parcel data, including parcel boundaries, was obtained from the Whatcom County assessors' office (2010). Information on utilities, existing roadway geometry, and other site features was generally scaled off of aerial photographs when as-built drawings were not available. A site reconnaissance was performed in October 2010.

Designers consulted the Nearshore Geodatabase for additional site context. The Nearshore Geodatabase is available from the Washington State Geospatial Data Archive at: http://wagda.lib.washington.edu/data/geography/wa_state/#PSNERP. Metadata are provided in the *Geospatial Methodology Used in the PSNERP Comprehensive Change Analysis of Puget Sound* (Anchor QEA et al. 2009) (see Annex B). The geodatabase includes numerous datasets listed below:

- Shoreline
- Bathymetry
- Digital Elevation Model (DEM)
- LiDAR (terrestrial)
- Oblique aerial imagery (from the Washington Coastal Atlas)
- Hydrographic sheets
- Geology
- Slope stability
- Drift cells (net shore-drift)
- Streams
- Impervious surfaces
- Overwater structures
- Marinas
- Armoring
- Breakwaters/jetties
- Groins
- Dikes
- Dams
- Nearshore fill
- Roads
- Railroads
- Land cover

Designers also consulted the University of Washington Puget Sound River History Project 19th Century Coast Survey Topographic Sheets (2009) for information on historical geomorphologic conditions. Conceptual designs were intended to replicate historical conditions and remove stressors to nearshore processes to the extent practicable and feasible; as a result these datasets informed the selection of restoration strategies and features. Designers created additional GIS data layers (point files, line files, and polygon files) to represent civil design features such as areas of lowland excavation to be depicted on the plan view drawings. Designers also created simple line drawings in CAD to represent typical sections and estimate quantity take-offs. Limited surface modeling was used to aid new levee and existing levee excavation quantity take-offs.

2-3.1.1 Additional survey and mapping required

Substantial additional information will be required in PED to refine the design assumptions, confirm real estate requirements, and develop plans and specifications. Additional survey, mapping, and other geospatial data needs include the following:

- Property/Utility Survey – More detailed information on property boundaries and utilities will be needed to finalize the design and support real estate negotiations. The survey would also be useful in providing more accurate preliminary designs and quantities for roadways, utilities, bridges, and removal of existing features.
- Topographic/Bathymetric Survey – The conceptual design was based on LiDAR and aerial photos, which have inherent inaccuracies. Site-specific topographic and bathymetric survey data will be needed to refine design of key project elements and develop detailed construction and demolition plans. Survey data could also be used as a baseline for pre- and post-construction modeling, including hydrodynamic modeling. A temporary tide gage may be required in the early design stages to obtain site-specific tidal statistics.

2-3.1.2 Procedure for incorporation of new mapping or other geospatial data

Planning, design, and implementation are expected to take several years. The site-specific surveys identified above are standard components of the design process and should be completed in the early stages of PED to ensure that the design work proceeds efficiently. Incorporating these data into the design process is not expected to delay the restoration.

2-4 GEOTECHNICAL

This section describes the geologic setting of the site, previous and recommended studies, and proposed geotechnical explorations relevant to design features.

2-4.1 Geotechnical Information Used

2-4.1.1 Regional and site geology

Regional geologic mapping indicates the Nooksack River delta is composed of alluvium deposits (Dragovich et al. 2002). Alluvium deposits (Qa) consist of sorted combinations of silt, sand, and gravel deposited in deltas and alluvial fans. A section of the geologic map is shown in Figure 2-4-1.

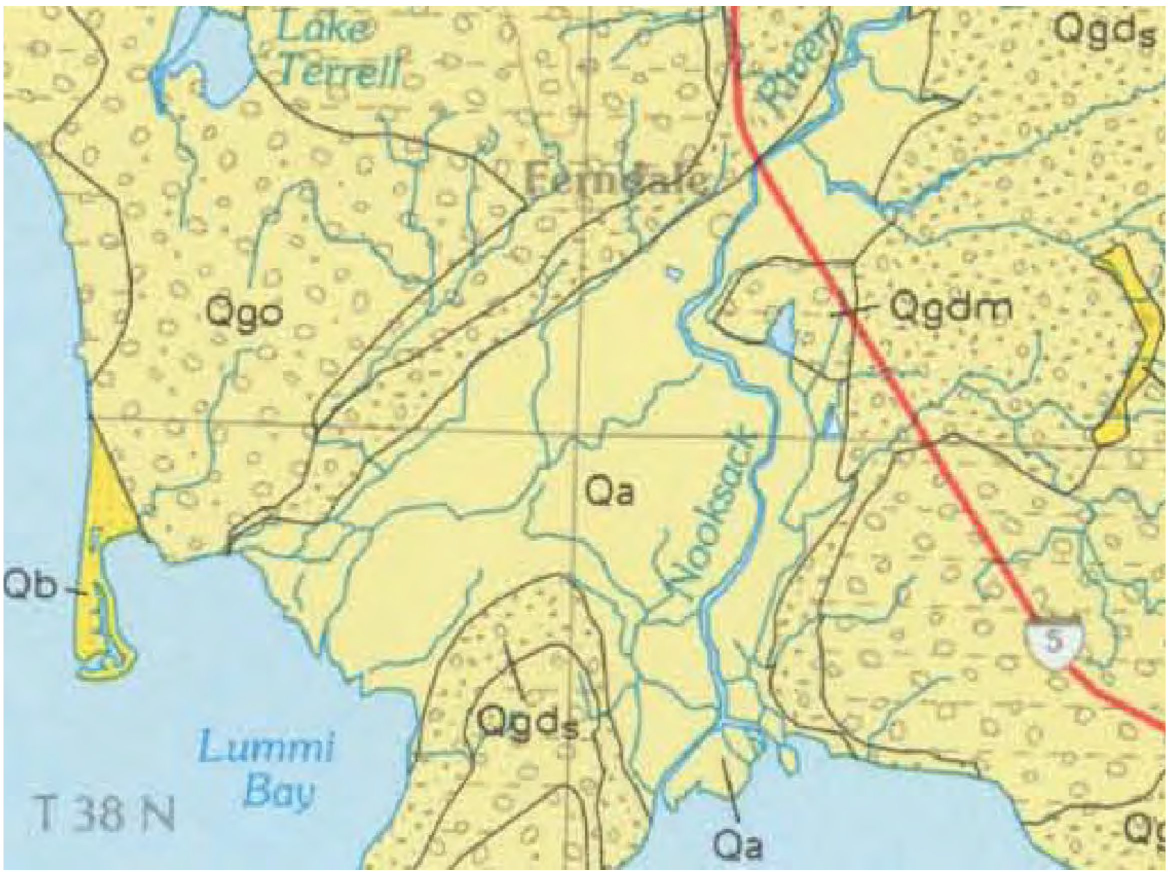

Figure 2-4-1. Geologic Map of Nooksack Delta

The *Soil Survey of Whatcom County Area, Washington* maps six soil types in the site vicinity: Eliza silt loam, Eliza-Tacoma silt loam, Hovde silt loam, Mt. Vernon fine sandy loam, Tacoma silt loam, and Whatcom-Labounty silt loam (Goldin 1992).

According to the Washington State Department of Ecology (Ecology) website, approximately 15 borings were conducted in the Nooksack River delta in October 2011 and April 2012. The wells and borings are located at the Silver Reef Hotel at 4876 Haxton Way and were drilled between depths of 23 feet and 101.5 feet. The driller's log indicates subsurface conditions typically consist of sandy silt and silty sand from the ground surface to the bottom of the hole.

Design drawings from 1967 for the Marine Drive Bridge over the Nooksack River include five borings along the alignment of the bridge. Borings varied in depth from 54 feet to 146 feet. The typical profile consists of loose to medium dense silty sand in the top 10 feet, loose to medium dense sand with shells from 10 to 30 feet, medium stiff silty clay with sand from 30 to 55 feet, medium stiff to stiff sandy silty clay with gravel from 55 to 120 feet, stiff clay from 120 to 135 feet, and dense sand from 135 to 146 feet (bottom of hole).

In addition, design drawings from 1977 for the Marietta Slough Bridge, located 1,500 feet east of the Nooksack River Bridge; include two borings varying in depth from 69 feet to 75 feet. The typical profile consists of dense sand and gravel fill in the top 10 feet, loose to medium dense silty sand from 10 to 20 feet, loose sandy silt with shells from 20 to 30 feet, soft to medium stiff silty sandy clay from 30 to 60 feet, and dense sandy silt from 60 to 75 feet (bottom of hole).

2-4.1.2 Completed explorations

No subsurface explorations have been completed for this site. All subsurface information is based on soil surveys, geologic mapping, and logs from Ecology. See Section 2-4.3 for the proposed subsurface exploration plan.

2-4.1.3 Selection of preliminary design parameters

Based upon research of the soils and geology in the project vicinity, subsurface soils are likely to consist mostly of silt, sand, and clay. Preliminary design parameters have been selected for the types of soils that are likely to be observed at the proposed bridge foundation locations. Table 2-4-1 provides a range of preliminary design values for the anticipated soils in the foundation.

Table 2-4-1. Preliminary design parameters

Soil Description	Depth Range (feet)	Unit Weight, γ (pcf)	Friction angle, ϕ’
Loose to medium dense, silty sand	0 – 30	115-120	28˚-30˚
Medium dense, silty clay w/ sand and gravel	30- 60	105-115	26˚-30˚
Medium dense to dense, sand w/ silt	60 – 100	120-125	30˚-34˚

Groundwater table was assumed at the ground surface.

2-4.1.4 Geophysical investigations

No geophysical investigations have been conducted for this project. It is recommended that there be shear wave velocity measurements, such as a seismic refraction survey to define the site class since the geologic map shows loose materials. There is also the potential for liquefiable soils around the site.

2-4.1.5 Groundwater studies

No groundwater studies have been conducted for geotechnical design. Groundwater elevation is dependent on flows from the Red (Lummi) River, Nooksack River, and the water surface elevation of Puget Sound. The site spans over many square miles and the groundwater table may be variable. For geotechnical design purposes, the groundwater will be assumed at the ground surface when considering the bridge foundations.

2-4.1.6 Recommended instrumentation

No instrumentation is recommended for this site. (Not applicable.)

2-4.1.7 Earthquake studies

In accordance with Table 20.3-1 of the 2010 American Society of Civil Engineers (ASCE) 7, a Site Class C or D is recommended for this site when considering the average of the upper 100 feet. According to the 2008 United States Geological Survey (USGS) Earthquake Hazards website https://geohazards.usgs.gov/deaggint/2008/, the Peak Ground Acceleration (PGA) predicted for the site is 0.410 g, and the maximum considered earthquake (MCE) ground motions for the site are Ss=0.934 g and S1=0.385 g. In accordance with Tables 11.4-1 and 11.4-2 from ASCE 7, Site Coefficients Fa and Fv are 1.1 and 1.6, respectively for a Site Class D. Therefore the adjusted MCE ground motions are SMS=1.028 g and SM1=0.617 g. The return interval for these ground motions is 5 percent probability of exceedance in 50 years (975 years). See Figure 2-4-2 below for earthquake deaggregation output.

Seismic design for deep foundations and bridge abutments will be performed in accordance with Washington State Department of Transportation (WSDOT) requirements and the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Seismic Design Specifications. (AASTHO specifies 7% in 75 years, which is comparable to USGS 5% in 50 years.)

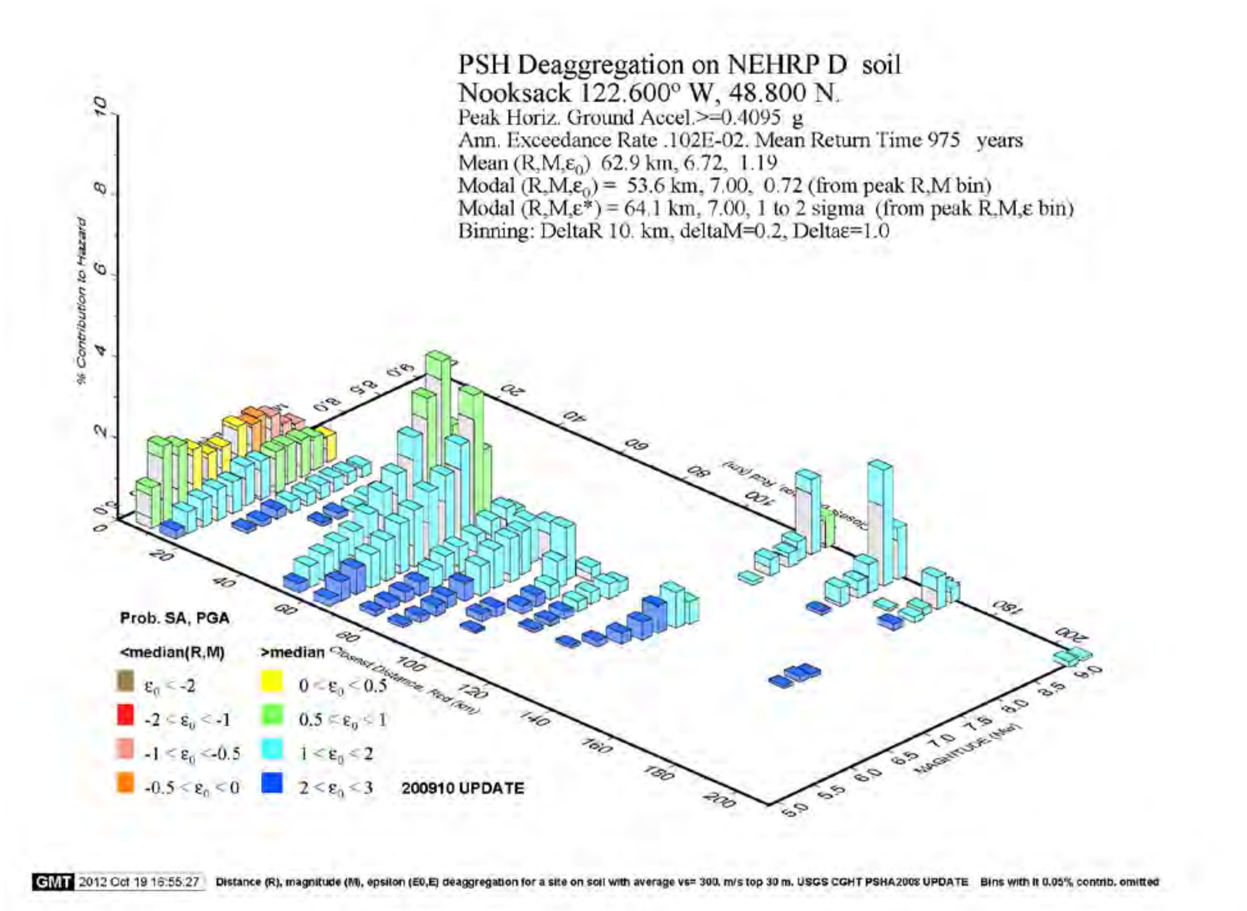

Figure 2-4-2. Deaggregation plot for Nooksack Delta

Earthquake loadings are not normally considered in analyzing the stability of levees because of the low risk associated with an earthquake coinciding with periods of high water. Depending on the severity of the expected earthquake and the importance of the levee and duration of flood event, seismic analyses to determine liquefaction susceptibility and stability may be required. However, this is not anticipated for this site.

2-4.1.8 Preliminary engineering analysis

Several bridges will be replaced to meet design goals. All bridges will be supported by deep foundations. Preliminary foundation estimates were included in the conceptual design for cost estimating purposes. The foundation design assumed two, 7-foot-diameter drilled shafts at each pier with a 100-foot embedment depth. The information for the majority of the existing bridge foundations is currently unavailable.

Drilled shafts or driven piles are acceptable foundation alternatives for the proposed bridges. Shallow foundations are not an option at this time due to potential seismic loading, scour, liquefiable soils, and soft soils.

A preliminary estimate of foundation capacity using the lower range of the parameters in Table 2-4-1was used as a check on the foundation design from the conceptual design. See Table 2-4-2 , Table 2-4-3 and Table 2-4-3 for results of the estimate.

Table 2-4-2. Preliminary Foundation Axial Capacity Estimate for Ferndale, Slater, and Imhoff Roads at Lummi River

Feature	Description	
Bridge	Total length (feet)	250
	# of spans x Approx. span length (feet)	2 x 125
	Approximate width (feet)	44
	Dead load x 1.25 [LRFD strength I] (kips) / pier[1]	1,800
	Live load x 1.75 [LRFD strength I] (kips) / pier[2]	400
Foundation	Type	Drilled Shaft
	Diameter (inch)	84
	# shafts / pier	2
	Depth (feet)	100
Load	Estimated static loading demand (kips)	1,100
Capacity	Factored pile resistance (kips)	1,900
	Sufficient capacity	OK

[1] Dead load estimate is based on conceptual design bridge dimensions.

[2] Live load estimate is based on HL-93 (HS-20 Truck + 0.64k/ft lane).

Table 2-4-3. Preliminary Foundation Axial Capacity Estimate for Haxton Way and Hillaire Road at Lummi River

Feature	Description	
Bridge	Total length (feet)	450
	# of spans x Approx. span length (feet)	3 x 150
	Approximate width (feet)	44
	Dead load x 1.25 [LRFD strength I] (kips) / pier[1]	2,100
	Live load x 1.75 [LRFD strength I] (kips) / pier[2]	500
Foundation	Type	Drilled Shaft
	Diameter (inch)	84
	# shafts / pier	2
	Depth (feet)	100
Load	Estimated static loading demand (kips)	1,300

Feature	Description	
Capacity	Estimated pile capacity (kips)	1,900
	Sufficient capacity	OK

1 Dead load estimate is based on conceptual design bridge dimensions.

2 Live load estimate is based on HL-93 (HS-20 Truck + 0.64k/ft lane).

Table 2-4-4. Preliminary Foundation Axial Capacity Estimate for Slater Road at Tennant Creek

Feature	Description	
Bridge	Total length (feet)	390
	# of spans x Approx. span length (feet)	3 x 130
	Approximate width (feet)	44
	Dead load x 1.25 [LRFD strength I] (kips) / pier[1]	1814
	Live load x 1.75 [LRFD strength I] (kips) / pier[2]	417
Foundation	Type	Drilled Shaft
	Diameter (inch)	84
	# shafts / pier	2
	Depth (feet)	100
Load	Estimated static loading demand (kips)	1,115
Capacity	Estimated pile capacity (kips)	1,937
	Sufficient capacity	OK

1 Dead load estimate is based on conceptual design bridge dimensions.

2 Live load estimate is based on HL-93 (HS-20 Truck + 0.64k/ft lane).

Each end of the bridge will require an earthen abutment to tie into. If there is liquefaction potential found in the site soils, the abutments will need to be supported by 4 or more shafts that extend down to a stable depth, preliminarily set at 100 feet below ground surface.

Foundation capacity estimate is preliminary without any site specific subsurface information. Upon completion of subsurface explorations, foundation should be designed using encountered subsurface conditions. Downdrag on the drilled shafts shall also be analyzed. Foundation design shall include drilled shafts and driven piles as a comparison if deemed as a valid alternative. It should be noted that if liquefaction potential soils in the foundation are present, the depth of the drilled shafts or driven piles may increase. Likewise, if bedrock or dense or hard soil is encountered the depth of the drilled shafts will significantly decrease. Seismic loading, liquefaction potential, and scour are not included in the current conceptual level design.

Slope stability analysis has not been evaluated at this time. Slope stability and settlement analysis for the entire approach embankments shall be performed upon completion of the design and geometrical configuration of the bridge. Ground improvements may be required at the bridge abutments/roadway approaches if liquefiable soils are encountered.

The proposed levee should be designed in accordance with the USACE Engineering Manual 1110-2-1913 Design and Construction of Levees. For levees constructed on soft subsurface conditions, stability and long-term settlement analyses are typically performed.

2-4.1.9 Excavatability analysis

According to the conceptual design, significant excavation will be required. Several thousand linear feet of levee and rock armor will be removed, a portion of the Lummi River will be excavated to increase capacity, and several roadway embankments will be removed for construction of new bridges and raised roads. No explorations or construction records were located, and therefore the levee and embankment material is unknown. Based on soils and geology maps, it may be assumed that the levee and embankment fill consists of compact sandy silt and structural fill. Excavation of riprap and fill may be accomplished using an excavator. Bedrock and boulders are not anticipated; therefore, rock excavation and blasting are unlikely.

2-4.1.10 Anticipated construction techniques and limitations

The type of deep foundation to be used will be confirmed during PED once subsurface explorations have been completed. At this time it is assumed drilled shafts will be used to support the proposed vehicle bridges. Due to the presence of soft and caving soils and anticipated high groundwater, either casing or wet method is recommended for construction of drilled shafts. Upon completion of the shaft excavation, the hole is cleaned and the reinforcing steel cage is placed to the bottom of the hole. The casing is then carefully extracted, fully or partially, leaving a top segment to facilitate column installation and concrete is cast. Once the shafts are installed, the columns are cast, and the pilecaps and bridge superstructure are constructed.

Most of the earthwork will be accomplished with standard excavation equipment. Construction of roadways and setback levees may be accomplished year-round using dozers and excavators due to the existing dikes and drainage ditches. Excavation of the dike breaches and removal should be scheduled to coincide with periods of low water.

Settlement may be observed along portions of the new levee, access levees, and roadway embankments. Depending on geotechnical evaluation, construction of the embankments may need a sure-charge, the work may be staged, or ground improvements may be advised to reduce post-construction settlement. Construction practices and methods outlined in the USACE Engineering Manual 1110-2-1913 Design and Construction of Levees are recommended for levee construction.

Construction activities and proposed restoration features will impact the existing utilities that run across the site. Evaluating the impact and protecting the utilities will be coordinated with service providers during later stages of design. See Section 2-6.2 for utility relocation information.

See Section 2-6.1.1 for additional construction notes.

2-4.1.11 Potential borrow sources and disposal sites

No borrow sources have been identified within the site. Substantial volumes of both borrow and disposal will be required. Due to the large footprint of the proposed restoration, it is likely that borrow and disposal sites can be found within the site boundaries. Suitability of local borrow materials for use in setback levees and elevated roads will be evaluated during later stages of design. Some land subsidence has likely occurred in the diked agricultural lands. Excavated materials may be disposed in subsided areas or in existing borrow ditches. Offsite disposal and borrow sites are available within a reasonable distance from the site. Multiple borrow and disposal sites are located along the Interstate 5 corridor within 30 miles of the site. Project plans include a 30 mile haul distance for levee and armor excavation, provision of fill material, a 20 mile haul distance for pavements and sidecast of channel excavations. Borrow and disposal sites shall be confirmed during PED. The uncertainties associated with confirming suitable borrow and disposal sites have been captured in the cost risk register.

2-4.1.12 Potential sources of concrete and materials

Preliminary investigations indicate that there are four concrete ready-mix batch plants located within 20 miles of the site and nine gravel suppliers within 30 miles.

2-4.1.13 Suitability of concrete and materials

Suitability of concrete and materials will be evaluated during PED.

2-4.2 Additional Studies and Analysis

Additional studies and analysis to be completed during PED or subsequent phases of design include the following at a minimum:

- Geotechnical Investigation: subsurface explorations, testing, and field reconnaissance
- Foundation Design: static and seismic analysis according to AASHTO LRFD for vehicle bridges
- Abutment Stability: include potential for liquefaction and ground improvement
- Pavement Design: new roadways and approaches (include traffic analysis for Equivalent Single Axle Load (ESALs))
- Scour Study: at roadway embankments, abutments, and bridge piers
- Settlement Analysis: for roadway and railway embankments
- Levee Design: stability, settlement, seepage analysis

2-4.3 Additional Explorations and Testing

The proposed subsurface exploration plan consists of drilling borings along the alignment of the proposed roadway and railway bridges. In addition test pits, cone penetrometer testing (CPT), and borings should be conducted along the roadway and railway embankments. Borings along the bridge alignments should occur at the abutments and at least one every pier, approximately every 110 to 150 feet (closer for the railway bridge). For the embankments, borings will be spaced approximately every 250 to 500 feet, with additional CPTs between the borings to provide additional parameters and an adequate soil profile along the proposed embankments. Test pits could be performed if needed for at-grade construction and pavement design.

Explorations for the proposed levees should be conducted in accordance with USACE Engineering Manual 1110-2-1913. This will include a combination of test pits and borings along the levee alignment. Depth of borings and test pits for the levee should be a minimum of 10 feet and spaced approximately every 200 feet. Test pits will be accomplished with a backhoe or small excavator, and the recommended boring method is mud rotary.

Based on research of the site and preliminary foundation design, the bridge borings should be a minimum of 150 feet below the ground surface, embankment borings and CPTs a minimum of 50 feet, and test pits a minimum of 10 feet. The preferred exploration method for the borings is mud rotary. Test pits will be accomplished with a backhoe or small excavator.

The subsurface exploration plan should be reevaluated and coordinated with hazardous and toxic material investigations during PED to include chemical sampling and testing; see Section 2-9.

Sampling in the soil borings should be accomplished using standard penetration test (SPT) with samples taken typically every 2.5 feet for the top 10 feet and every 5 feet for the rest of the boring depth. Proposed soil lab testing will include moisture content, grain size analysis, and percent finer than #200 sieve. Atterberg limits and consolidation tests are recommended for cohesive soils, and unconfined compressive strength test for rock cores.

2-4.4 Laboratory-Testing Program and Evaluations

No laboratory testing or evaluation of materials has been completed at this time. Testing to be completed during PED is outlined in Section 2-4.2.

2-5 ENVIRONMENTAL ENGINEERING

This section describes environmental engineering factors relevant to the proposed design features.

2-5.1 Use of Environmentally Renewable Materials

At this design stage, use of environmentally renewable materials is not specifically planned, but will be considered wherever applicable during PED. If renewable materials are available, they can be incorporated into the design during PED, or sourced by the contractor during construction. Specific details will be developed during PED.

2-5.2 Design of Positive Environmental Attributes into the Project

The Nooksack River Delta is selected to address River Delta restoration objectives to restore freshwater input and tidal processes where major river floodplains meet marine waters. The proposed restoration would remove levees, roads, and other barriers to restore water and sediment processes throughout a substantial portion of the historical Nooksack River delta. This restoration contains multiple components: deconstruct and reconstruct roadways, build new setback levees, breach river banks, remove channel fill, and fill linear ditches. The restoration has been developed to retain agricultural area, reduce the efforts and costs of changing transportation infrastructure, and be consistent with the proposals for the Lummi Nation Wetland and Habitat Mitigation Bank.

2-5.3 Inclusion of Environmentally Beneficial Operations and Management for the Project

Design and construction will incorporate sustainable and ISO 14000 compliant practices for operations and management. The USACE Environmental Operating Principles (EOPs) are designed to provide direction on achieving better stewardship of air, water, and land resources while showing the connection between managing those resources and protecting environmental health. The EOPs are to ensure that USACE actions consider the environment and are sustainable now and in the future.

2-5.4 Beneficial Uses of Spoil or Other Project Refuse during Construction and Operation

At this design stage, beneficial use of spoil or other refuse is not planned. If spoils or other refuse materials are available for reuse, they could be incorporated into the design. Specific details will be developed during PED.

2-5.5 Energy Savings Features of the Design

At this design stage, energy savings features have not been incorporated. In accordance with the EOPs, energy savings features will be a component of the design to the maximum extent practicable.

2-5.6 Maintenance of the Ecological Continuity in the Project with the Surrounding Area and Within the Region

The restoration will increase ecological continuity in the site and with the surrounding area. This is one of several sites designed to restore the productivity and increase interconnectivity of the Puget Sound ecosystem.

2-5.7 Consideration of Indirect Environmental Costs and Benefits

All direct, indirect and cumulative environmental costs and benefits were evaluated during the environmental impact assessment and alternatives analysis recorded in the Final FR/EIS.

2-5.8 Integration of Environmental Sensitivity into All Aspects of the Project

Construction will be conducted to ensure no long-term deleterious impacts to the ecosystem will occur. Best management practices will be incorporated into the contract documents. Best management practices will cover erosion and sediment control, stormwater management, spill response and hazardous material management, trash and debris management, air emissions from construction vehicles, and noise standards.

2-5.9 Use of Environmental Review Guide for Operations (ERGO) with Respect to Potential Future Environmental Problems

This is not a USACE operating facility. (Not applicable.)

2-5.10 Incorporation of Environmental Compliance Measures into the Project Design

All applicable laws and regulations will be followed during design and construction in accordance with the USACE contract documentation.

2-6 CIVIL DESIGN

This section discusses the key elements of the civil design, including the selection of the site, basis of design, and constructability.

2-6.1 Site selection and project development

Restoration in the Nooksack River Delta represents a large-scale opportunity to restore a substantial portion of a large river delta that drains approximately 825 square miles. The proposed Nooksack River restoration combines multiple elements intended to restore the natural hydrologic, sediment, and ecological processes to a substantial portion of the Nooksack and Lummi deltas. The proposed restoration activities include the following:

- Armor removal for streambank restoration and reconnecting floodplain habitat
- Dike removal or modification for floodplain freshwater marsh restoration
- Setback levees for floodplain reconnection and side channel development
- Riparian revegetation for shading, nutrient inputs, and complexity of bank habitat
- Large wood placement for increased habitat complexity
- Hydraulic modification: partial restoration of river flow to Lummi River through installation of water control structure at confluence of Lummi and Nooksack Rivers; structure intended to facilitate transfer of freshwater and sediment to the Lummi River
- Topography restoration: regrading of the Lummi River to allow for more frequent engagement by fluvial flows from the upper watershed
- Non-structural measure: residential relocations

The main restoration elements are shown in Table 2-6-1 and described in detail in the following sections. Annex 2-1 contains exhibits that depict the proposed restoration design elements.

Table 2-6-1. Key Design Elements

Item	Description of Item	Approx. Quantity
Nooksack River		
Install New Setback Levee and Relocate Ferndale Road	Set back right bank levee to Ferndale Road alignment between Slater Road and Marine Drive. New levee will be 12,633 lf with a typical section of 600 sf. Will include new paved road on the crest	346,100 cy
Remove Portions of Existing Levees on Both Banks	Remove approximately 60% of right and left bank dikes from the Slater Road to near Marine Drive. Total length of 12,280 lf.	93,800 cy
Install Log Jams in Mainstem Nooksack	Install large wood structures within Nooksack mainstem to assist geomorphic response of the river in concert with setting back the levees (location to be determined)	3 structures
Lummi River		
Install New Water Control Structure at Confluence	Upstream Lummi River connection to Nooksack River to be regulated via an engineered diversion structure that will be designed during PED. This structure is intended to facilitate transfer of freshwater and sediment to the Lummi River, while preventing avulsion of the mainstem to the west.	1 ea
Regrade Lummi River Channel and Berms. Remove North Red River Road West of Haxton	Regrade existing Lummi River channel to install 0.04% bed slope and larger channel cross section to better match invert to water surface elevation of the Nooksack River, increase conveyance capacity, and create surface to encourage geomorphic processes. Regrade would occur over the upper 9,980 lf of the channel, with a typical section of 285 sf in the upper 5,000 lf and 80 sf in the lower 4,890 lf.	67,300 cy
	Remove existing berm and road along north side of Lummi River west of Haxton Way. Length of berm to be removed and associated volumes are: west to Hillaire=3843 lf 10,659 cy; Hillaire to Haxton=5927 lf, 13,017 cy; Haxton to Slater=1981LF, 7,176 cy.	30,900 cy
Build New Setback Levees	Install setback levee on south side of the Lummi River channel between Haxton and Ferndale. Length is 11,232 lf with typical section of 175 sf.	84,500 cy
	Install setback levee on the north side of the Lummi River channel from the valley margin to the Ferndale Rd and realign North Red (Lummi) River Road away from channel. Length is 23,025 lf with typical sections varying from 135 sf/lf to 432 sf/lf based on levee heights 5 to 8 feet, crown elevations varying from approximately Elevation 15 to 20 NAVD88.	313,000 cy

Item	Description of Item	Approx. Quantity
Transportation Improvements		
Modify Slater Road at Lummi River	Remove bridge and a portion of the existing roadway.	450 lf
	Raise Slater Road (build new roadway)	200 lf
	Build new bridge (two 125-foot spans) over Lummi River to span new set back levees	250 lf
Modify Slater Road at Tennant Creek	Remove a portion of the existing roadway (and culvert)	5600 lf
	Add bridge on Tennant Creek to allow 100-year flow to pass below the bridge after culvert removal.	390 lf
	Install causeway along Slater from eastern upland to Ferndale Road to maintain emergency access/egress during floods.	5,600 lf
Modify Haxton Way	Remove bridge and a portion of the existing roadway	1,300 lf
	Build new bridge (three 150-foot spans) over Lummi River to span new setback levees	450 lf
	Install new road approaches	200 lf
Re-align North Red River Road and Haxton Way	Remove existing roadway	11,751 lf
	New road on top of setback levees (30-foot width)	9,216 lf
Modify Hillaire Road at Lummi River	Remove Bridge and a portion of the existing roadway	575 lf
	Build new bridge (three 150-foot spans) over the Lummi river to span new setback levees	450 lf
	Build new roadway	200 lf
Modify Imhoff Road at Lummi River	Remove a portion of the existing roadway (and culvert)	400 lf
	Build new bridge (two 125-foot spans) over the Lummi River	250 lf
	Build new roadway	150 lf
Modify Ferndale Road at Lummi River	Remove a portion of the existing roadway (and culvert)	650 lf
	Build new bridge (two 125-foot spans) over the Lummi River.	250 lf
	Build new roadway	12,200 lf
Modify Marine drive between Marine Drive Bridge and Kwina Slough	Raise a portion of existing roadway to maintain emergency access/egress during floods.	3,500 lf

2-6.1.1 Basis of design

The proposed restoration would increase the area of aquatic and floodplain habitats along both the Nooksack and Lummi Rivers, but would retain a level of engineering control that would minimize changes to land use on much of the delta. The restoration of this site is designed to restore ecosystem processes and reestablish natural geomorphic conditions. The civil design is based in part on historical conditions as evidenced by 19th Century Coast Survey Topographic Sheets (Figure 2-6-1). In other words, post-restoration site conditions are intended to resemble or replicate the historical morphology with adjustments for altered surrounding conditions. For the Nooksack/Lummi delta, the restoration would not match historical conditions because it would leave the Nooksack (east) side of the delta as the primary conduit for water and sediment.

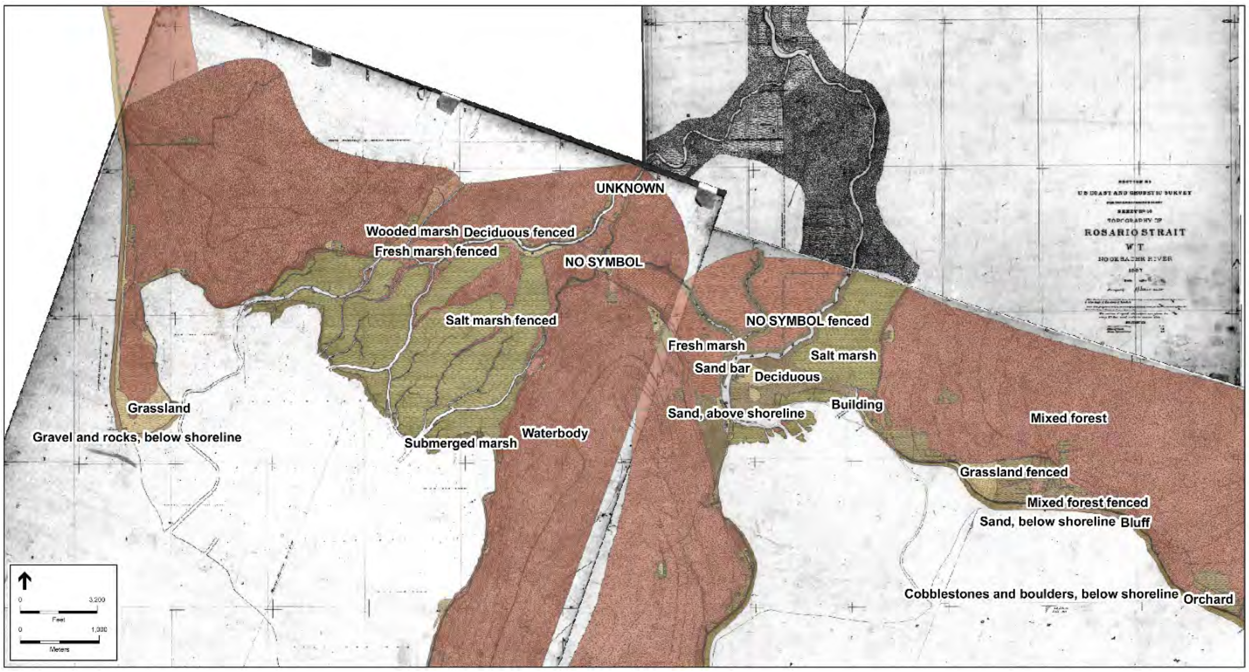

Figure 2- 6-1. Historic Map (T-Sheet) and River History Project Data

A network of transportation corridors has been developed over the delta, and much of the area supports active agricultural operations. The proposed restoration would retain agricultural area, reduce the efforts and costs of changing transportation infrastructure, and be consistent with the existing proposals for the Lummi Nation Wetland and Habitat Mitigation Bank. Changes to the existing infrastructure system are described below.

<u>Levee Removal and Levee Breaches</u>

The levees/berms along the Nooksack River the levee along the Lummi delta, and the plugging of the Lummi River channel have eliminated important floodplain slough and distributary channel habitats, and eliminated or greatly reduced key geomorphic processes. The partial removal and setback of levees along the Nooksack River would restore these geomorphic processes. Near-term effects include the connection of the river and tidal areas to the floodplain/marsh. In a long-term analysis, the removal/setback of the levees would decrease the relative amount of stream power constricted within the presently leveed reach of the river, with a resultant increase in sediment deposition in the channel and floodplain. Sediment deposition—in conjunction with engineered log jams that would cause hydraulic constrictions—would increase stage relative to discharge. This is desirable in increasing the connection of the Lummi River to the Nooksack River at progressively decreasing discharge levels through time. Optimally, the Lummi River would become connected to the Nooksack River at all discharge levels.

There is some risk that the levee breaching may not provide the hydraulic response needed to restore geomorphic processes. Thus, the volume estimates conservatively assume that nearly 60% of the levee length must be removed, even though some hydraulic studies (Whatcom County Department of Public Works 1999) suggest that floodwaters access the left-bank floodplain upstream of the site. Subsequent design may determine that removing and/or breaching the levees in phases may be beneficial in balancing the need to connect the water surface elevation of the Nooksack River to the Lummi River channel with the need to aggrade the Nooksack River channel to engage the Lummi distributary channel at progressively lower discharge levels. Such balancing would be assessed during later modeling in PED.

Setback and strategic breaches in the levees that border the Lummi River delta will restore tidal flux to a substantial portion of the western delta. As part of the proposed Lummi Nation Wetland and Habitat Bank, the seapond will remain and access would be maintained from Hillaire and Kwina Roads. Portions of the levee system west of the Lummi River would be breached, and the tide gates on the levee east of the Lummi River would be replaced with self-regulating tide gates or similar. The self-regulating tide gates would be designed to allow substantial tidal flux during normal flow conditions, but would remain closed during storm events. A setback levee would be installed to protect the golf course at the west end of the site.

<u>Channel Creation and Rehabilitation</u>

The upper 9,500 feet of the Lummi River channel would be regraded to allow for more frequent engagement by fluvial flows from the upper watershed. The Lummi River appears to be perched on deltaic sediments remaining from when most of the flow was directed west to Lummi Bay. Further, the mainstem Nooksack River has been leveed, and flow into the Lummi River is via a relatively small culvert.

Connecting the Nooksack River to the Lummi River would enable distributary flow into the Lummi River at essentially all discharge levels; this would provide a sustained freshwater connection, enhance water quality, and increase habitat. Because the head of the Lummi River channel is substantially disconnected from the Nooksack River channel, the connection to the river includes rehabilitation of the Lummi River channel to more closely approximate water stages that occur within the Nooksack River. If actions are not taken to include flow into the Lummi River distributary channel at lower discharge levels, the restoration benefits would be more transitory and limited to periods when the Nooksack River is flooding. This may or may not coincide with times when water quality in the Lummi River would be substantially improved by upstream inputs of fresh water.

The proposed restoration includes an engineered diversion structure in the right-bank levee of the Nooksack River to allow some flow into the Lummi River. To account for potential future changes in the stage-discharge relationship of the Nooksack River at this location, the structure would likely include some level of adjustment so that only the allowable discharge levels for which the downstream channel is designed are conveyed into the Lummi River. As currently designed, the setback levees along the Lummi River would not provide flood risk management in the event the mainstem were to avulse to the west.

The Lummi River has been highly modified in the last 150 years, including straightening and narrowing the channel. In its upstream extent, the Lummi River is very small (the field estimates suggest it may only convey approximately 150 cfs or less) and would be rehabilitated (enlarged) as part of the restoration. This would include lowering the Lummi River channel in its upper reaches, as well as grading and channel expansion to increase geomorphic processes and create habitat. Setback levees and floodplain grading are designed to confine Nooksack River overflow to within the Lummi River floodway without flooding across the delta.

<u>Modifications to the Transportation System</u>

Roads through the Nooksack River delta provide key transportation connections to portions of the Lummi Indian Reservation and the only access to the Lummi Island ferry. As noted in the 2015 FEMA Flood Insurance study:

> "Flooding along Nooksack River frequently causes closures of Slater Road and Marine Drive (the two primary access roads to the Lummi Indian Reservation and Lummi Island and is the main transportation corridor to two of the industries in the Cherry Point Heavy Impact Industrial Zone). Marine Drive is closed even more frequently than Slater Road by Nooksack River floods and is always closed due to flooding whenever Slater Road is flooded. Marine Drive was closed at least 17 times over a three-year period (2007-2010) with the longest continuous closure of 13 days

in November 2009 and a total closure time of 54 days (Lummi Nation Multi-Hazard Mitigation Plan 2010 Update). When both roads are closed, access to the Reservation, the Lummi Island ferry, and the Cherry Point industries is through, or to the north of, the City of Ferndale, approximately 2 miles north of Slater Road. This detour can more than double travel times to and from Bellingham and result in severe congestion in the City of Ferndale. These road closures have substantial impacts on the economic, public health, and safety of the affected areas."

Much of the existing system of bridges and roads through the delta will have to be modified to allow for successful restoration of water and sediment processes. Modifications to roads and bridges on the delta at and east of Ferndale Road include Ferndale Road at the Lummi River; Slater Road at Tennant Creek; and Marine Drive. Modifications on the west side of the delta include Hillaire Road at the Lummi River, Imhoff Road at the Lummi River, Slater Road at the Lummi River and and Haxton Way at the Lummi River/Smuggler's Slough.

In general, bridges with larger spans would be added to allow for channel migration and greater flood flow conveyance. Bridges are described in detail below in Section 2-7. Bridge spans over the Lummi River were sized to match the proposed levee setback and channel geometry to be controlled by the engineered diversion structure on the mainstem Nooksack River. Bridge spans on Tennant Creek were developed based on previous plans, which are assumed to be based on site-specific hydraulic investigations (DEA 2007). All bridge designs will need to be re-evaluated during PED to conform to design decisions regarding the Lummi diversion structure, setback levee locations, and hydraulic criteria.

Stormwater runoff from the project is required to meet Federal Energy Independence Security Act (EISA), State, Washington State Department of Ecology and local requirements. EISA requires sites in excess of 5000 square feet to retain the total volume of rainfall from the 95th percentile of the 24-hour storm on site. The alternative is to require the post development hydrology to not exceed the pre-development hydrology (prior to man) by using site specific stormwater BMP's such as infiltration, evapotranspiration and detention. WSDOE requires that the duration of the developed storm flow be less than 50% of the 2 year through the 50 year events. The WDOE water quality provisions require that treatment facilities be designed for the 24- hour storm with a 6-month return frequency or a simulated daily volume that accounts for 91% of the entire runoff volume over a multi-decade period of record.

For feasibility design, a provision for stormwater is included in the estimate. Once the site hydrology is confirmed in PED, stormwater detention will be designed where required. The risk of the assumption for stormwater treatment size has been captured in the Cost and Schedule Risk Analysis.

Refer to the exhibits for a depiction of restoration elements and the quantities and dimensions used in cost estimation. During PED phase the PDT will examine trade-offs and engineer the recommended alternative to minimize costs while optimizing desired benefits. This will be accomplished once the site investigations and surveys recommended for PED have been completed.

2-6.1.2 Constructability

Earthwork would require mobilizing heavy equipment to the site. Access to the site can occur via the existing system of county and farm access roads. Temporary traffic control would be necessary during construction removal.

Potential borrow sources and disposal sites are discussed in Section 2-4.1.11.

See Section 2-10 for additional information on construction procedures and Section 2-20 for the anticipated schedule for construction.

2-6.1.3 Real estate

Real estate assumptions, valuations, and planning documents have been appropriately scaled for the current level of design. As additional surveys, modeling, and design are completed during PED, the real estate documentation will be modified accordingly. For the current real estate status, refer to the Final FR/EIS, Appendix C, *Real Estate Plan*.

2-6.2 Relocations

While information about utilities on the site is currently unknown, several utilities will probably need to be relocated to follow the new bridge and roadway alignments. These relocations will be coordinated and permitted with the utility owner. A utility survey will be completed during PED. In addition, land requirements, easement, and/or franchise agreements will be identified and coordinated.

2-7 STRUCTURAL REQUIREMENTS

This section discusses the structural elements of the proposed restoration including preliminary design requirements and criteria for bridges or roads, a description of major structures and construction considerations, and recommended analyses.

2-7.1 Functional Design Requirements and Technical Design Criteria

Roads through the Nooksack River delta provide key transportation connections to portions of the Lummi Indian Reservation and the only access to the Lummi Island ferry. Much of the system of bridges and roads through the delta will need to be modified to allow for successful restoration of water and sediment processes.

Modifications to roads and bridges on the delta at and east of Ferndale Road include Ferndale Road at the Lummi River, Slater Road at Tennant Creek and Marine Drive. Modifications on the west side of the delta include Hillaire Road at the Lummi River, Imhoff Road at the Lummi River, Slater Road at the Lummi River and Haxton Way at Lummi River/Smuggler's Slough.

The AASHTO Seismic Design guide specifications are intended for conventional bridges designed for the life safety performance objective considering a seismic hazard corresponding to a seven percent probability of exceedance in 75 years. This implies that a bridge, when following these specifications, has a low probability of collapse in a 1000-year event but may suffer significant damage and that significant disruption to service is possible. Partial or complete replacement of the bridge may be required. A higher level of seismic performance may be selected by a bridge owner who wishes to have immediate service and minimal damage following a rare earthquake. Seismic engineering analysis and design costs as well as construction costs should be expected to increase as the post-earthquake performance objectives are increased.

Whether a bridge is considered "regular" or "not-regular" is a function of its physical characteristics. A regular bridge is a bridge that has fewer than seven spans, no abrupt changes in weight, stiffness, or geometry. Regardless of its regularity, a bridge shall be designed with earthquake resisting systems (ERS) corresponding to the requirements of a Seismic Design Category (SDC) of C or D (typical for the Puget Sound region). As such, the regularity was not assumed to impact construction costs directly for this level of design. Determination of the Seismic Design Category, SDC, is based on the parameters identified in Section 2-7.9. A category of D would result in more complex analysis and detailing requirements. This suggests an increase in both the design and construction costs associated with the foundations, columns, and connectivity between these structural components.

An important criterion for selecting road bridges for this site is to provide a simple and repetitive structural concept facilitating a healthy bidding climate; one that meets the goals stated below and is considered, within industry standards, to be a cost-effective solution.

The key design requirements for the road bridges are to identify a cost-effective, constructible bridge structure that will support traffic, provide for prescribed horizontal and vertical hydraulic openings, require minimal capital to maintain, meet the AASHTO Bridge and Geometric design specifications and WSDOT Bridge Design Manual specifications, and have a design life of no less than 75 years.

The bridges will be constructed of concrete shaft foundations, concrete piers, pre-stressed concrete girders, and a concrete deck. Concrete is a high-quality, dense mix design that should remain functional throughout the life of the bridge with minimal maintenance.

Pre-stressed concrete girders consist of a very high-quality concrete; they are the most common type of girder used in Washington because of superior local fabrication skills and the availability of high-quality local aggregate. Pre-stressed concrete girders are lower-maintenance structures than their steel counterparts and are competitively priced with steel girders. The economy in structural design can be achieved by designing around the standard girders from the Bridge Design Manual Span Capability Sheets as long as the selected standard design meets the geometrical requirements of the particular bridge. Deep shafts are used in liquefiable soils which are commonly found in the flat tidal zones of the Puget Sound region. The design objective is to extend the foundation shafts through the liquefiable soils and embed them deep into the underlying glacial soils to provide the necessary lateral support for the structure during a seismic event. Additional design details are noted as follows:

- In general, new bridges with longer spans will be added to allow for channel migration and greater flood flow conveyance and transfer. New bridges will be 44 feet wide, with span lengths ranging from 110 to 150 feet. Girders will be 6.5 feet deep, pre-cast concrete. New roadways will have two 12-foot lanes with two 8-foot shoulders.

- Bridge spans over the Lummi River are sized to match the proposed levee setback and channel geometry to be controlled by the engineered diversion structure on the mainstem Nooksack River. The bridge spans on Tennant Creek were developed based on previous plans, which are assumed to be based on site-specific hydraulic investigations (DEA 2007).

Key design elements are identified in Table 2-7-1. These proposed new bridges will achieve significant restoration benefits without high social or economic costs.

Table 2-7-1. Summary of Bridge Information

Bridge Location	Description
Ferndale Road at Lummi River	• New 250-foot-long bridge (two 125-foot spans), 44-feet wide, estimated low chord at El. 24 ft. NAVD88 • 6.5-foot pre-cast concrete girders with 1-foot concrete slab • (1) 44-foot CIP concrete pilecap with (2) 4-Foot diam. columns atop (2) 7-foot-diameter drilled shafts; 100 foot embed at each pilecap • Remove 650 feet of roadway. Add 400 feet of new roadway • Construction Duration: 10 months
Slater Road at Lummi River	• New 250-foot-long bridge (two 125-foot spans), 44 feet wide, estimated low chord at El. 19 ft. NAVD88 • 6.5-foot pre-cast concrete girders with 1-foot concrete slab • (1) 44-foot CIP concrete pilecap with (2) 4-Foot diam. columns atop (2) 7-foot-diameter drilled shafts; 100-foot embed at each pilecap • Remove 450 feet of existing road. Add 200 feet of new roadway • Construction Duration: 10 months
Slater Road at Tennant Creek	• Raise Slater Road per DEA (2007) plans and add a new 44-foot-wide, 390-foot-long bridge (three 130-foot spans) on Tennant Creek to allow 100-year flow to pass below the bridge, estimated low chord at El. 21 ft. NAVD88 • (2) 44-foot CIP concrete pilecaps with (2) 4-Foot diam. columns atop (2) 7-foot drilled shafts; 100-foot embed at each pilecap. • Include temporary detour route • Construction Duration: 18 months

Bridge Location	Description
Hillaire Road at Lummi River	• New 450-foot-long bridge (three 150-foot spans), 44 feet wide, estimated low chord at El. 14 ft. NAVD88 • 6.5-foot–deep, pre-cast concrete girders with 1-foot concrete slab • (2) 44-foot CIP concrete pilecaps with (2) 4-Foot diam. columns atop (2) 7-foot drilled shafts; 100-foot embed at each pilecap • Remove 575 feet of existing road. Add 200 feet of new roadway • Construction Duration: 12 months
Imhoff Road at Lummi River	• New 250-foot-long bridge (two 125-foot spans), 44 feet wide, estimated low chord at El. 20 ft. NAVD88 • 6.5-foot-deep pre-cast concrete girders with 1-foot concrete slab • 44-foot CIP concrete pilecaps with (2) 4-Foot diam. columns atop (2) 7-foot drilled shafts; 100 feet embed at each pile cap • Remove 400 feet of existing road. Add 400 feet of new roadway • Construction Duration: 10 months
Haxton Way at Lummi River	• New 450-foot-long bridge (three 150-foot spans) over Lummi River, 44 feet wide, estimated low chord at El. 15.5 ft. NAVD88 • 6.5-foot-deep pre-cast concrete girders with 1-foot concrete slab • (2) 44-foot CIP concrete pilecaps with (2) 4-Foot diam. columns atop (2) 7-foot drilled shafts; 100 feet embed at each pilecap • Remove 1,300 feet of existing road. Add 200 feet of new road • No change at Smuggler's Slough • Construction Duration: 10 months

The bridge design will be reviewed and approved by Whatcom County Public Works. The design will conform to the most current edition of the standards listed in Table 2-7-2.

Table 2-7-2. Structural Requirements

Item	Description
Design Specifications	• WSDOT Bridge Design Manual, current edition • AASHTO LRFD Seismic Bridge Design, current edition • AASHTO LRFD Bridge Design Specifications, current edition
Load Criteria	• Live Load: HL-93 (HS-20 Truck + 0.64k/ft lane), 1.3 Impact Factor • Load Combinations: Per Table 3.4.1-1 LRFD (Load Combinations and Load Factors) • Pedestrian (if required): 75 psf • Dead: Concrete = 0.16 K/cu ft, Steel 0.49 k/cu ft.

2-7.2 Survey, Hydrologic, Hydraulic, and Geotechnical Data Used

LiDAR survey and probable water surface elevations were used to develop the conceptual plan. For information about data used for the conceptual design, see Section 2-3.

No geotechnical data were available at the time of the conceptual design. Numerous borings will be required at each bridge location to facilitate development of an accurate cross section of the geology below the bridge. Typically, the borings should extend to about 150 feet below ground. During the conceptual design phase, the typical nearshore soil characteristics of Puget Sound were considered in selecting the bridge foundation type. Geotechnical investigations will be required for completion of PED; see Section 2-4.3.

2-7.3 Site Selection Studies

The site selection is summarized in Section 2-6.

2-7.4 Major Structures

Several technical considerations were used to decide on the type of bridge for the site, including: a cost effective structure that provides a hydraulic opening meeting the restoration goals, sufficient geometrical and structural capacity to safely meet the traffic demands, and sufficient capacity to meet seismic demands. Hydraulic openings are affected by bridge length and distance between piers. Bridge superstructure depth is affected by span length. Subsequent design may refine the conceptual plans in terms of bridge type, size, and location. The basis of design at the conceptual phase established the following parameters:

<u>Span Length</u>: In general, span length is highly influenced by the minimum or desirable hydraulic goals. Other factors that can affect the span length are good soils for foundations, minimizing piers in the waterway, achieving sufficient space at the banks to gain inspection access below the bridge, and the elevation of the water in the 100-year flood. Bridges range from 250 to 450 feet long. Longer bridge spans will allow for channel migration and greater flood flow conveyance and transfer. New bridges will be 44 feet wide, with span lengths ranging from 110 to 150 feet.

<u>Bridge Type</u>: The recommended bridge type is a pre-cast, pre-stressed concrete girder bridge. This means the deck is supported by girders below the roadway. The girders are supported by cap beams which comprise the transverse beam of the pier system. The bridges proposed for this site are continuous bridges. This means that they will have no intermediate joints between abutments and provide for a continuous deck over the piers. This allows for a structurally efficient system, reducing the girder depth, but also restricts leakage of water from the deck onto the piers by eliminating expansion joints. The bridge deck and girders will have expansion capabilities at their abutments.

The girders will be constructed of pre-cast, pre-stressed concrete, fabricated offsite and shipped by truck to the site for installation. Standard WSDOT pre-cast concrete girders are an efficient and economical bridge type for continuous span construction.

Depth of Structure: It is assumed at the conceptual design that the bridges will have a depth of 6.5 feet. The bottom of the bridge soffit (low point) is set at 3 feet above extreme high water (EHW) to provide adequate clearance for debris. This places the bridge deck at a minimum elevation of 17.7 feet in tide-dominated areas and a minimum elevation of 27.0 feet at Ferndale Road where the fluvial Nooksack flood regime will determine high water levels.

Alignment Considerations**:** See Section 2-6 for alignment considerations.

2-7.5 Evaluation and Selection of Substructure Alternatives Based on Economy and Performance

These bridges are located in a nearshore riverine environment, likely requiring deep foundation types. The geotechnical engineer will make final recommendations based on data obtained from the onsite boring logs and the structural engineer. See Section 2-4 for additional information.

The soils are likely to experience liquefaction during an earthquake. As such, the shafts will have to extend downward through the soft materials to stiff glacial soils for a solid fixed embedment.

The cost comparison between types of deep foundations (piles versus shafts) does not always result in a clear cost advantage for either foundation type. Many factors come into play such as availability of equipment to a contractor, a contractor's preferred method, the depth of the footing and the ease of access, construction schedule, and depth of foundation. In general, cost is not a determining factor for deep foundation type. Forces, displacement, and geological conditions will determine which system is best to use.

General and local scour are always a consideration with deep foundations. Subsequent design will include a hydraulic scour analysis. Protection of the structure from hydraulic scour may compete with the goals of the restoration. Preliminary design will evaluate these considerations and mitigate accordingly.

2-7.6 Construction Considerations

For bridge construction, a crane positioned on one end of the bridge will be required to set the girders in place. Work is anticipated to require land-based driving rigs or large augers to dig the shaft holes. Other equipment may include excavators, cranes, concrete trucks, and dump trucks. Placing foundations can be a challenge and may require temporary fill areas to facilitate the heavy cranes. It is assumed that the contractor will be able to install one shaft per week. In areas near the slough, large-diameter shoring will be required to keep out water and allow access to the top of the shaft for column form placement and removal. Once the shafts are installed, the columns are cast inside a shoring casing. After the casing is removed, the cast-in-place pile caps and bridge superstructure are constructed.

The construction duration for each bridge is noted in Table 2-7-1.

2-7.7 Stability Analyses

Bridge stability is a fundamental component in the design process and depends on boundary conditions. In general, the bridges are made stable by fixity in the soil/ structure relationship, fixity between the cap beams and the foundation elements, and designing/detailing for unbalanced loads. Longitudinally the bridge superstructure is held in position and restrained during earthquakes by positive connectivity to each intermediate pier, either "pinned" (combined with shear keys) or "fixed." The bridge superstructure, however, is allowed to expand at each abutment. Transversely the bridge is tied together along its length, fixed or pinned to each pier, and designed to transfer all transverse loads directly to the foundation.

2-7.8 Stress Analyses

Stress analyses are a fundamental component in the design process and serve as the basis of how all structural elements are selected. Design will be in accordance with governing standards of the WSDOT Bridge Design Manual and the AASHTO LRFD Manual.

2-7.9 Seismic Analyses

All seismic analyses are performed in compliance with the WSDOT requirements and the AASHTO LRFD Seismic Design Specifications. This site is located in an active seismic zone. Bridges will be designed for a seismic event with a 7% probability of exceedance in 75 years (approximately a 1,000-year return period).

The essential seismic parameters to develop the Design Response Spectrum are arrived at by the geotechnical engineer, if site specific; see Section 2-4.1.7 for details of the seismic analysis.

2-7.10 Thermal Stress Analysis

Thermal analysis is a fundamental component of the design process and will be considered per the AASHTO LRFD design specifications. In general, thermal stresses are handled by providing expansion joints in strategic locations to permit a bridge to expand and contract without a large buildup of stresses or movement.

2-7.11 Other Analyses

The present level of design has been based on local standards for roadway design requirements, hydraulic analyses, loading requirements of structures, and constructability considerations.

2-7.12 Additional Studies, Tests, Analyses

The information needed to design a bridge is generally captured in the following studies, tests, and analyses:

- **Boundary and Topographic Survey**
- **Geotechnical Investigation and Report**
- **Hydrodynamic, Hydraulic, and Scour Analyses**

Additional investigation and studies may be needed for permitting or other site requirements unrelated to the infrastructure. See Section 2-21 for a complete list of recommended additional studies and investigations.

2-8 ELECTRICAL AND MECHANICAL REQUIREMENTS

Electrical and mechanical structure requirements are not applicable to this site.

2-9 HAZARDOUS AND TOXIC MATERIALS

A Phase 1 Environmental Site Assessment was conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments*, and ER 1165-2-132: *HTRW Guidance for Civil Works Projects*. The Phase 1 Environmental Site Assessment report is attached in Annex 2-1.

One recognized environmental condition exists at the Nooksack site. The Wilder hazardous landfill contains uncontrolled but currently non-migrating contamination. EPA identified that no action was necessary based on the fact that the contamination did not have a migration pathway. However, the proposed Corps project will extend the river adjacent to the site in areas that were not previously saturated.

One of these areas is Claypit Pond, located immediately west and down-gradient from the Wilder site. Claypit Pond has historically had elevated concentrations of contaminants in sediment, although no human health risk was identified. These contaminants were suspected to come from three sites to the east, including the Wilder location. It was suspected that surface water drainage through culverts was the mechanism by which contaminants moved into the pond.

As of today, there is no reason to believe that contaminants from these sites are flowing into the pond, due to low permeability of the surrounding soils. However, inundation of Claypit Pond from the Corps project has a limited probability of changing the groundwater gradient, or more generally the overall hydrology, such that contaminants originating from the hazardous waste landfill could more easily flow into the pond (See Annex 2-1, Exhibit C). Therefore, the Corps project has the potential to be impacted by contamination. This potential risk is addressed in the PSNERP Nooksack Risk Register and in the Memorandum for Record dated 8 December 2015, located in the project file.

2-10 CONSTRUCTION PROCEDURES AND WATER CONTROL PLAN

The proposed restoration will involve earthwork and exposure of bare ground during modification of multiple dikes, removal and replacement of multiple bridges, and river channel rehabilitation. At this stage of design, it is assumed that standard best management practices will be implemented to control erosion and sedimentation and ensure construction areas are stabilized as needed to prevent adverse impacts. A standard temporary erosion and sediment control plan will be developed during PED.

The proposed restoration will also involve in-water work during removal and replacement of multiple bridges, conversion of tide gates, construction of an engineered diversion structure, river channel rehabilitation, and dike breaches. Most of the bridge work will be constructed above the ordinary high water mark. However, where bridge piers would be located in water, installation of caissons or coffer dams will be required prior to drilling of shafts to isolate the work zone. Removal of piles from existing bridges will require measures to contain sediment. The most appropriate methods will be selected during PED and will be coordinated with the natural resource agencies for environmental protection.

Other in-water work associated with dike breaches and rehabilitation of channels will be sequenced and timed to minimize exposure, and industry standard best management practices would be used. Specific measures of the in-water workplan will be determined during PED.

Specific measures for construction procedures and water control will vary depending on the location and nature of the work. State and Federal resource agencies will impose specific timing restrictions on in-water work to protect fish and wildlife. In addition, specific measures may be required under site-specific environmental compliance requirements and to protect downstream infrastructure or built environments. The erosion and water quality control plan will also need to consider and incorporate the findings of future analyses for hazardous and toxic materials at the site (as described in Section 2-9). A complete description of best management practices will be determined during PED.

2-11 INITIAL RESERVOIR FILLING AND SURVEILLANCE PLAN

The proposal is for ecosystem restoration. (Not applicable.)

2-12 FLOOD EMERGENCY PLANS FOR AREAS DOWNSTREAM OF CORPS DAMS

The proposal is for ecosystem restoration. (Not applicable.)

2-13 ENVIRONMENTAL OBJECTIVE AND REQUIREMENTS

Feasibility level information to develop designs, plans, and specifications, and to execute construction and operations is included in the Project's supporting documents including the U.S. Fish and Wildlife Service report titled "Strategic Restoration Conceptual Design - Preliminary Environmental Contaminant, Cultural Resource, and Endangered Species Site Evaluations." The environmental information developed for the analysis in the FR/EIS provides additional environmental objectives and requirements for final site design development. As summarized in Section 2-6, Civil Design, substantial environmental information was developed for the Final FR/EIS regarding environmental problems, opportunities, and constraints such that the Corps could estimate costs of the restoration sites and prepare the Real Estate Plan. The Corps will adhere to requirements stated in the Endangered Species Act consultation documents, Clean Water Act Section 401 certification, and other site-specific environmental compliance documents. The Corps has prepared a Programmatic Agreement (PA) for Section 106 of the National Historic Preservation Act compliance. As outlined in the PA, cultural resource investigations are necessary in the PED phase to determine if National Register eligible historic properties are located in the restoration project area prior to construction. The Monitoring and Adaptive Management Plan will be used to determine whether the site is meeting environmental objectives after construction.

2-14 RESERVOIR CLEARING

The proposal is for ecosystem restoration. (Not applicable.)

2-15 OPERATIONS, MAINTENANCE, REPAIR, REPLACEMENT AND REHABILITATION (OMRR&R)

OMRR&R costs for the Nooksack Delta site are related to:

- Levee maintenance and levee repair equal to 900 lf sections spread over 1800 lf every 15 years.
- Vegetation costs such as site watering for plant establishment over the first 2-3 years prior to the start of adaptive management as well as costs to fight invasive vegetation assuming 53 acres of invasive species control per year for 10 years.
- Yearly culvert inspection and maintenance such as removal of debris and sediment.
- Roadway & embankment inspection, maintenance and repair - Maintenance costs for roadways and road bridges were developed based upon the WSDOT Pavement Policy. It is assumed that roadways will be constructed with hot-mix asphalt, and that the maintenance of a particular road will occur as part of a larger effort that includes adjacent road sections. Repair and maintenance includes:
  - Roadway asphalt overlay twice during the 50-year period of analysis
  - Roadway grind and inlay once during the 50-year period of analysis
  - Roadway guardrails, signs and striping
- Bridge maintenance - Bridges will be constructed using pre-stressed concrete girders which are commonly used due to their low maintenance costs. WSDOT staff indicated that the maintenance costs do not vary greatly by bridge length (Wilson, 2011 and Baroga, 2011). Maintenance activities will include:
  - Bridge inspection & cleaning every year 2 or 3 man crew for 1 week
  - Replacement of guardrails, retrofit and structural repairs
- Diversion structure maintenance, assuming excavation of 100 ft x 100 ft x 10 ft every 5 years.

Annual OMRR&R is estimated at $705,000 for the 50-year project period. Additional assessment of OMRR&R activities will be conducted during PED.

The proposed setback levees will be designed in accordance with applicable USACE engineering manuals. Operation and maintenance of these structures will be necessary to ensure proper functioning of the structures and will be the responsibility of the local sponsor as detailed in the applicable O&M manual. Maintenance zones should extend 15 feet from both the riverward and landward setback levee toe. This maintenance zone should remain free of unwanted vegetation and unauthorized encroachments. Sod cover and riprap maintenance will be necessary to ensure proper functioning of erosion protection.

At the completion of the restoration project, an operation manual detailing proper maintenance practices for the setback levees will be provided to the local sponsor. Since the levees would be congressionally authorized and federally constructed, they will be ICW (Inspection of Completed Works) projects and automatically enrolled (eligible) in the PL 84-99 rehabilitation program.

2-15.1 33CFR Part 208 Projects

The proposed site is not a flood control project to be maintained and operated according to regulations in 33 CFR 208. (Not applicable.)

2-15.2 Channel or Basin Clean Out Projects

No channel or basin cleanout activities are proposed. (Not applicable.)

2-15.3 Multiple-Purpose, Complex Projects with Power Production

No power production is proposed. (Not applicable.)

2-15.1 Frequency and Cost of Maintenance Dredging

No maintenance dredging is proposed. (Not applicable.)

2-16 ACCESS ROADS

Access to the sites during construction can occur via the existing system of county and farm access roads. Earthwork activities will require heavy equipment to be mobilized to the sites. Temporary traffic control would be necessary during mobilization and fill removal activities. At least one route to the Lummi Peninsula will be maintained for traffic. Traffic can be staged to minimize road closures. Construction staging areas will be further analyzed during PED.

Permanent access will be required for maintenance of dikes.

2-17 CORROSION MITIGATION

Typical design standards use materials that are suitable for a marine environment such as concrete and galvanized steel pipe. Concrete was selected for the bridge superstructure and for the drilled shafts. If an alternate foundation system is selected, such as CIP steel piles, then galvanized steel should be used. Corrosion is generally not an issue for buried utilities or overhead power lines.

2-18 PROJECT SECURITY

The proposal is for ecosystem restoration. (Not applicable.)

2-19 COST ESTIMATES

The Nooksack River Delta construction cost estimate of $261,805,000 (March 2016 dollars) consists of costs to restore riverine and estuary areas and allow a return to more natural hydrology. This is the action with the highest total cost in the PSNERP project. Major features of work include work related to levee setbacks, and road and bridge demolition and installation. Other minor work includes plantings, large woody debris installation, and channel excavation.

The largest cost driver is relocations including demolition of existing levees, bridges and roadways and construction of new roads, bridges and levees ($149,818,000 construction cost, March 2016 dollars). This work consists of demolishing an existing dike road, setting back the dike, temporary construction facilities and platforms used to construct the bridges.

Following a formal cost and schedule risks analysis, a project contingency of 40% was developed. Primary engineering related cost risks came from the high uncertainty surrounding bridge and bridge foundation design. Minimal detail was provided in the design report and additional geotechnical and survey information will be required to constrain the uncertainty in these costs. The primary schedule risk is from delays that could be experienced during removal of structures at the site.

Risks that do not directly affect cost include excessive sedimentation either at the flow diversion or in the Lummi River channel may lead to flow reduction or obstruction of the river. Further analysis is needed in this area, and requirements could range from a monitoring plan to omission from the project scope.

Opportunities for this site for cost come from the possibility that bridge work may be substantially less expensive than predicted. This chiefly comes from the standardized designs used throughout PSNERP for bridges. Foundation piers were all designed very conservatively, and may be reduced with additional analysis. Schedule opportunities come from the same issue. The PDT considered it likely that there could be schedule reductions from lowered requirements.

2-20 SCHEDULE FOR DESIGN AND CONSTRUCTION

The proposed restoration at the Nooksack River Delta is considered highly complex. Based on the level of complexity, the anticipated design period for the site is approximately 5 years. This includes preparation of final design, plans and specifications, and the construction contract.

This project involves many elements, including removal of levees, reconnection of channels, linear ditches, and modification of transportation infrastructure. The anticipated construction period for the project is approximately 4 to 6 years. Any in-water construction activities will take place during established work windows.

All aspects of construction would need to be carefully phased to avoid elevated flood risks and comply with in-water work windows. Notably, the setback levee system would need to be in place on both sides of the delta before: (1) levee removal on the mainstem, and (2) allowing additional flow to enter the Lummi River channel. Road and bridge elements would need to be phased in a manner that allows for continuous access to and from the Lummi Peninsula.

Property acquisition and permitting timelines are not included in this duration, but the time required to complete these upfront activities is expected to be substantial.

2-21 STUDIES TO BE COMPLETED IN PED

Table 2-21-1 summarizes recommended studies and additional investigations to be conducted to support subsequent stages of design and implementation. Unless otherwise noted, these studies are recommended to take place during PED. In the table, studies are classified according to the following purposes:

- Data required for design, cost estimation or project compliance,
- design analysis to minimize project construction costs,
- design analysis to optimize environmental benefits,
- identification of induced flooding,
- and identification of actions needed for O&M.

Table 2-21-1. Studies Recommended for the Nooksack River Delta Site

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Property Investigation/Survey	• Compile more detailed information on parcel ownership and property boundaries to finalize the design, confirm acquisition requirements, and support negotiations with property owners.	X	X			
Topographic/ Planimetric/ Bathymetric Survey	• Acquire site-specific topographic and bathymetric survey data to refine design of key project elements, develop detailed construction and demolition plans, and serve as a baseline for pre- and post-construction modeling, including hydrodynamic modeling.	X				
	• Install a temporary tide gage in the early design stages to obtain site-specific tidal statistics.	X				
Geomorphic Analysis	• Complete a geomorphic analysis of channel migration potential to consider channel response to dike setback and increased tidal influence and fine tune levee removal locations.		X	X		X
Hydraulic Analysis/Modeling	• Gather hydrologic data on freshwater flows and tidal levels at this site to support hydraulic and sediment studies. Establish the design hydrology of the Nooksack Delta.	X				
	• Develop criteria for minimum bridge clearances over water, Lummi River diversion design, Lummi River channel sizing, and large wood designs.		X	X		
	• Review the existing FEQ hydraulic model developed by Whatcom County and Lummi Nation (or other 2-D models). Conduct hydrodynamic modeling to investigate water surface elevations under pre and post project conditions. This investigation would ensure that significant high flow conveyances are accounted for, optimize levee setbacks and removals, and investigate upstream and downstream impacts to peak flood stages (e.g., Hovander Road, BNSF rail line). Evaluate the design of the facility and its potential effects on hydraulics and sedimentation.		X	X	X	X

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Hydraulic Analysis/Modeling (cont.)	• Combine review of aerial photographs with field surveys to quantify channel topology and hydraulic roughness and inform geomorphic evaluation under restored conditions.	X				
	• Use numerical modeling and physical modeling if needed, to complete design and operation plans for the flow control structure at the junction of the Lummi and Nooksack Rivers. Evaluate the need for an energy dissipation facility in conjunction with conveying flow from the Nooksack River to the Lummi River.		X			
	• Complete water quality sampling and analysis of water quality effects as needed.			X		
	• Review the potential effects on water wells, septic systems, and groundwater seepage in the area of potential hydraulic effect to inform project design.		X			
	• Formulate the detailed monitoring plan, including any required field surveys or instrumentation that will be used to evaluate the project's hydraulic performance.			X		X
Sedimentation Analysis/Modeling	• Conduct sediment transport evaluations to decrease uncertainty about floodplain sedimentation processes and sustainability. Address the amount and potential areas of sedimentation in channels and at levees. This will require a sediment transport model.		X	X	X	X
	• Evaluate the effects of changes in sedimentation patterns in the Nooksack River downstream of the junction with the Lummi River.		X	X		X
	• Evaluate the need for slope protection on levees, roadway embankments, and bridge abutments and address the issue of bridge scour at piers and abutments.		X			
Coastal Engineering Studies	• Refine sea level projections using localized tide gauge data.	X				
	• Review and establish the final design tidal datums		X			
	• Conduct wind direction and wave run-up analysis to inform levee and bridge design.		X	X	X	

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Geotechnical Investigation	• Complete a standard investigation to include subsurface explorations, testing, and field reconnaissance.	X	X			
	• Confirm borrow and disposal sites.	X	X			
	• Complete additional geotechnical study and recommendations to finalize design of levees, roads, and bridges.		X			
	• Perform a settlement analysis for roadway embankments.		X			
	• Complete stability, settlement, and seepage analysis for levee design.		X		X	X
Foundation Design Study	• Perform static and seismic analysis according to WSDOT/AASHTO LRFD for vehicle bridges.		X			
Abutment Stability Study	• Evaluate the potential for liquefaction and ground improvement.		X			
Excavated Materials Study	• Evaluate the suitability of excavated materials for reuse.	X	X			
Pavement Design Study	• Complete a pavement design study for new roadways and approaches (include traffic analysis for ESALs).		X			
Structural Engineering	• Structural analysis of the bridge, bridge piers, and foundation.		X			
	• Analysis for gravity, wind and seismic effects.		X			
	• Design of bridge deck and supporting structure for gravity, wind and seismic effects in accordance with criteria established in this report.		X			
Utility Survey	• Compile more detailed information on utilities to finalize the design and confirm acquisition requirements.	X	X			
Cultural Resources Investigation	• Complete surveys for archaeological and historic resources, particularly in areas proposed for excavation.	X	X	X		
Wetlands Investigation	• Document the location, extent, and character of wetlands.	X		X		

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Cost Study	• Assess potential for cost and schedule reductions during refinement of restoration design.		X			
Environmental Compliance	• The Corps will coordinate with all relevant natural resource agencies during PED. Results of PED-phase studies will be provided to agencies and tribes as appropriate.			X		

2-22 DATA MANAGEMENT

Project documents, background materials, and digital files from the local sponsors were provided to the project team directly, through the State's Habitat Work Schedule, or via the Nearshore Sharepoint. The project team also used databases previously developed by and for the Nearshore Ecosystem Restoration Project including the Change Analysis and backing geospatial data (see Section 2-3.1.1 for additional detail).

Work products for the conceptual restoration designs were developed primarily in GIS and typical word processor and spreadsheet applications. GIS products for all sites were collected in a single geodatabase that captured spatially referenced locations and sizes of major design elements. GIS data resides in two locations: the Corps' Seattle District office, and at the University of Washington where staff used the data to assist with development of the Ecosystem Benefits Model.

2-23 USE OF METRIC SYSTEM MEASUREMENTS

This report uses United States customary units for design and construction measurements. To remain consistent with work conducted to date, the metric system of measurement was not used.

2-24 REFERENCES

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ANNEX 2-1: EXHIBITS

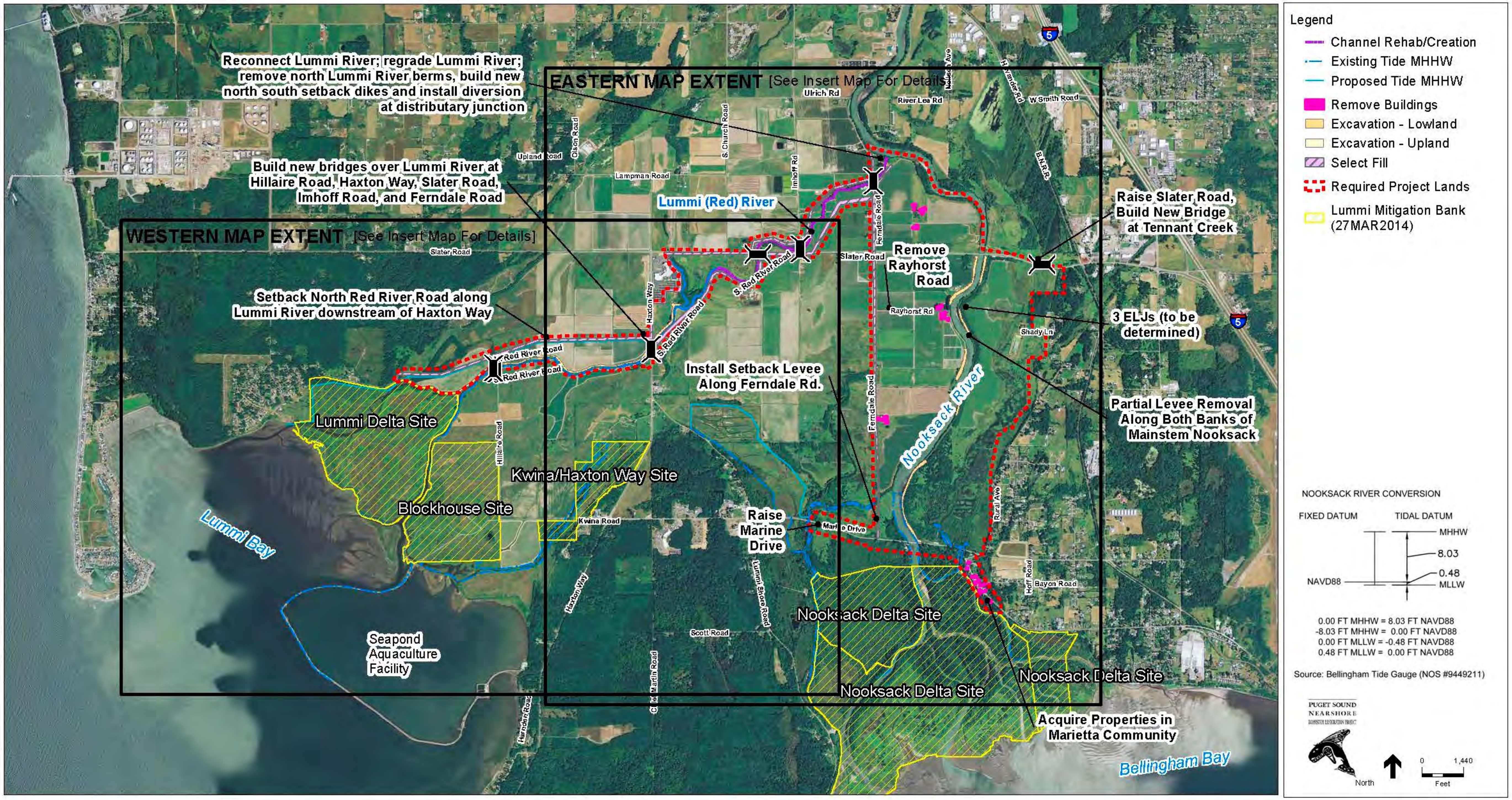
This annex contains a set of site-specific exhibits prepared for the proposed restoration. The exhibits include:

Exhibit A – Design Plan

Exhibit B – Design Sections

Exhibit C – Phase 1 Environmental Site Assessment

Exhibit A



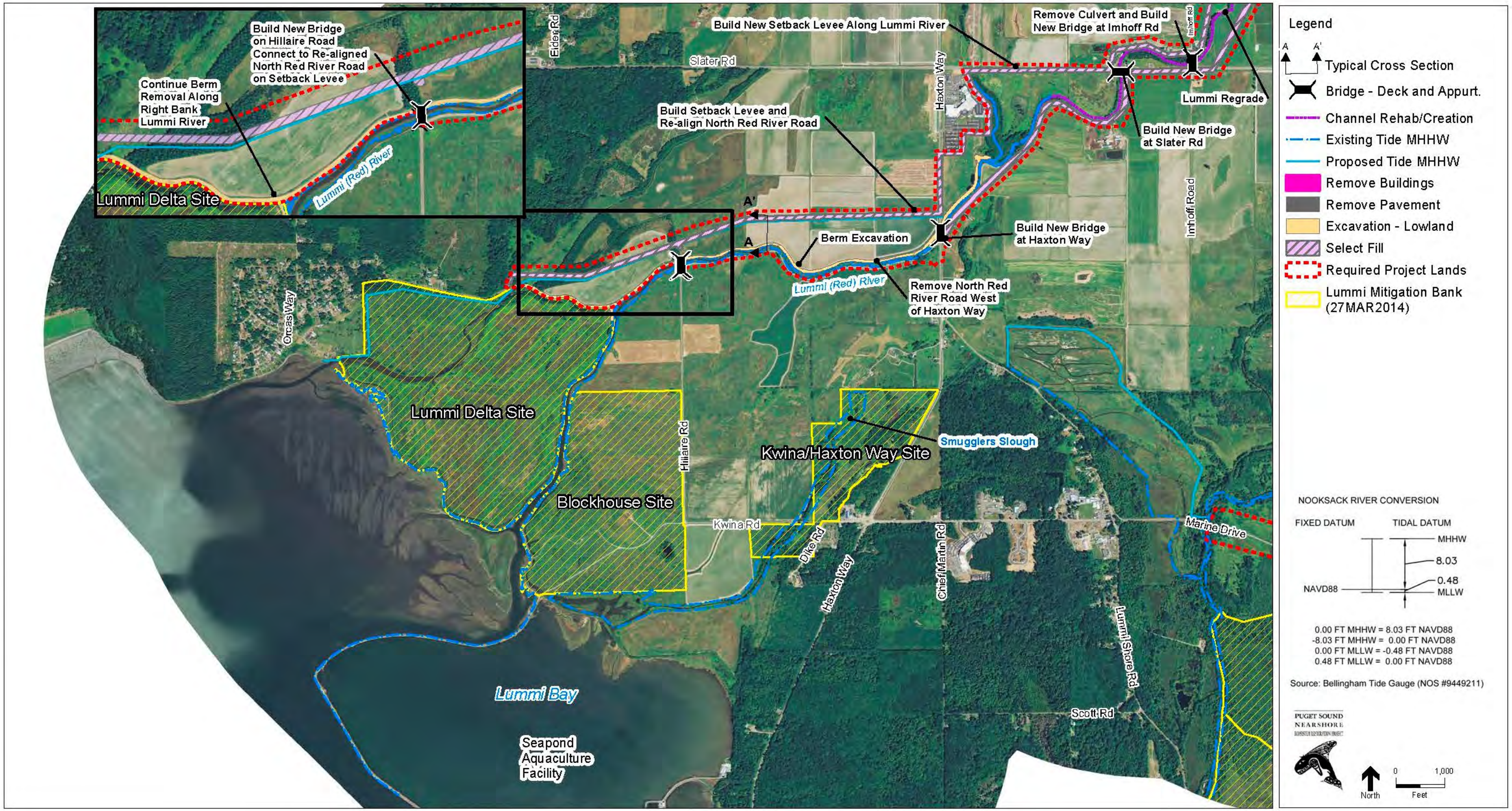
ORTHO_2013_NAIP_WASHINGTON

Site Name: Nooksack River Delta

Index Map

Lead Contractor: ESA
Design Lead: ESA, S. Winter, PH
Revised: USACE Petroff/Campbell December 2015

Exhibit A



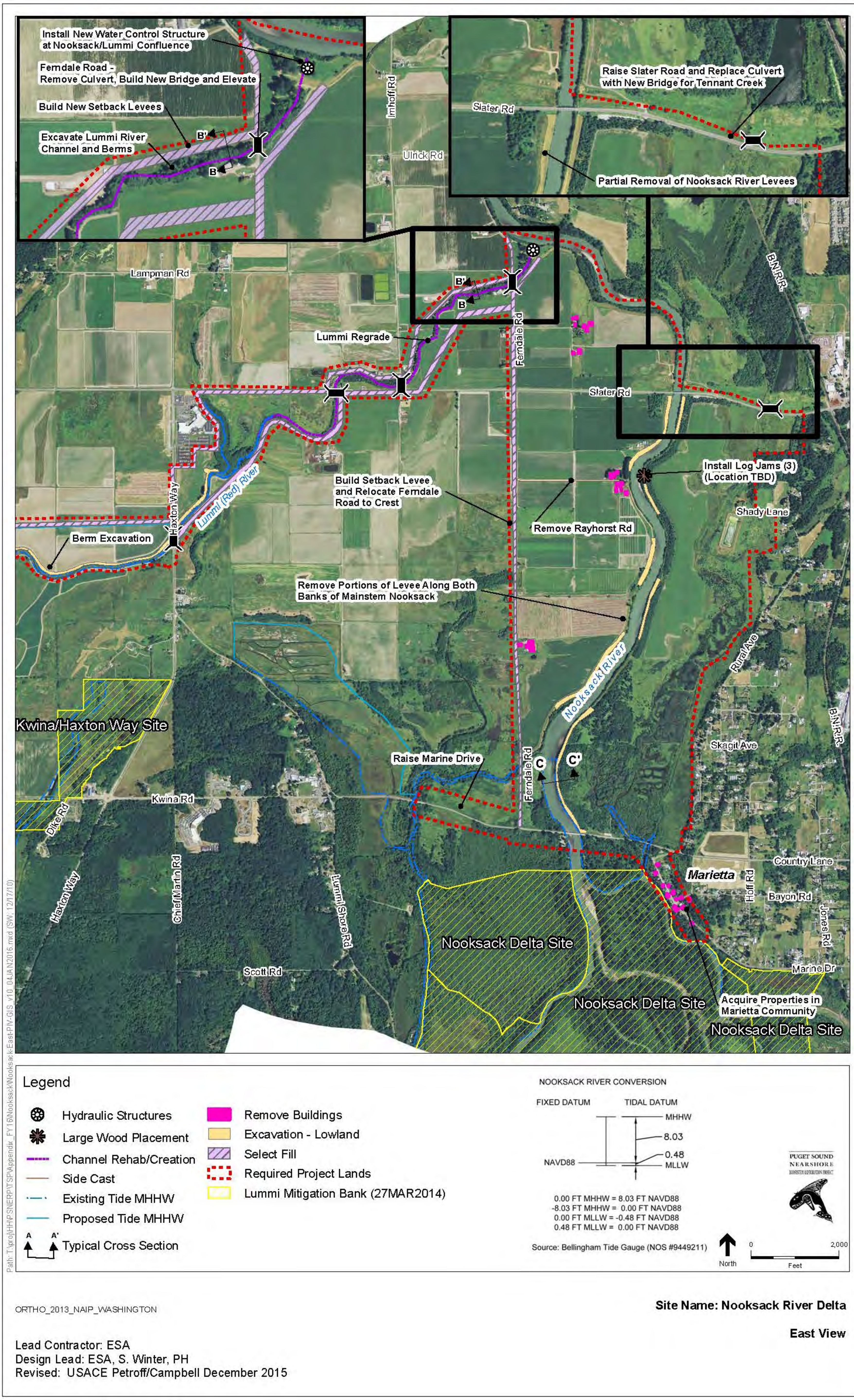
ORTHO_2013_NAIP_WASHINGTON

Site Name: Nooksack River Delta

West View

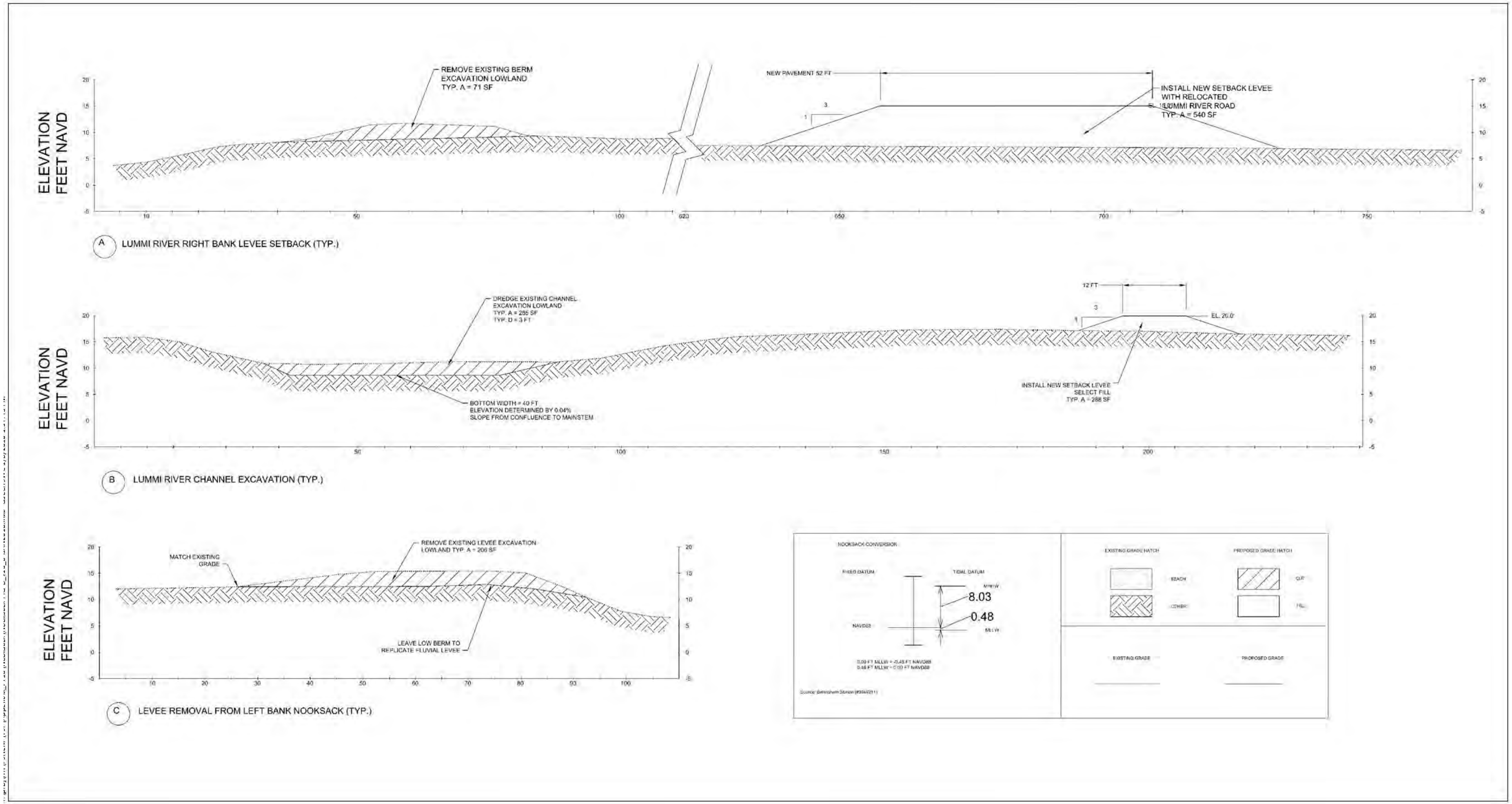
Lead Contractor: ESA
Design Lead: ESA, S. Winter, PH
Revised: USACE Petroff/Campbell December 2015

Exhibit A



Install New Water Control Structure at Nooksack/Lummi Confluence
Ferndale Road - Remove Culvert, Build New Bridge and Elevate
Build New Setback Levees
Excavate Lummi River Channel and Berms
Raise Slater Road and Replace Culvert with New Bridge for Tennant Creek
Partial Removal of Nooksack River Levees
Slater Rd
Imhoff Rd
Ulrick Rd
Lampman Rd
Lummi Regrade
Ferndale Rd
B.N.R.R.
Install Log Jams (3) (Location TBD)
Shady Lane
Build Setback Levee and Relocate Ferndale Road to Crest
Remove Rayhorst Rd
Haxton Way
Lummi (Red) River
Berm Excavation
Remove Portions of Levee Along Both Banks of Mainstem Nooksack
Rural Ave
Nooksack River
Kwina/Haxton Way Site
Skagit Ave
Raise Marine Drive
Kwina Rd
Dike Rd
Marietta
Country Lane
Hoff Rd
Bayon Rd
Chief Martin Rd
Lummi Shore Rd
Jones Rd
Nooksack Delta Site
Scott Rd
Marine Dr
Acquire Properties in Marietta Community
Legend
Hydraulic Structures
Large Wood Placement
Channel Rehab/Creation
Side Cast
Existing Tide MHHW
Proposed Tide MHHW
Typical Cross Section
Remove Buildings
Excavation - Lowland
Select Fill
Required Project Lands
Lummi Mitigation Bank (27MAR2014)
NOOKSACK RIVER CONVERSION
FIXED DATUM
TIDAL DATUM
MHHW
8.03
0.48
MLLW
NAVD88
0.00 FT MHHW = 8.03 FT NAVD88
-8.03 FT MHHW = 0.00 FT NAVD88
0.00 FT MLLW = -0.48 FT NAVD88
0.48 FT MLLW = 0.00 FT NAVD88
Source: Bellingham Tide Gauge (NOS #9449211)
North
0
2,000
Feet
PUGET SOUND NEARSHORE
ORTHO_2013_NAIP_WASHINGTON
Site Name: Nooksack River Delta
East View
Lead Contractor: ESA
Design Lead: ESA, S. Winter, PH
Revised: USACE Petroff/Campbell December 2015

Exhibit B

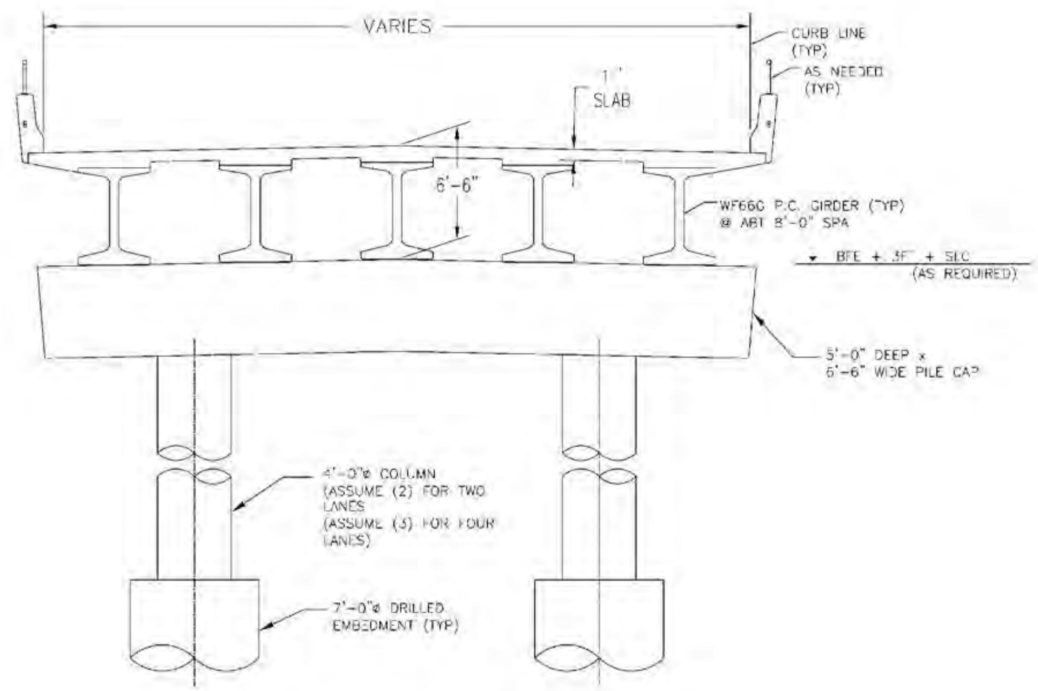


ORTHO_2013_NAIP_WASHINGTON

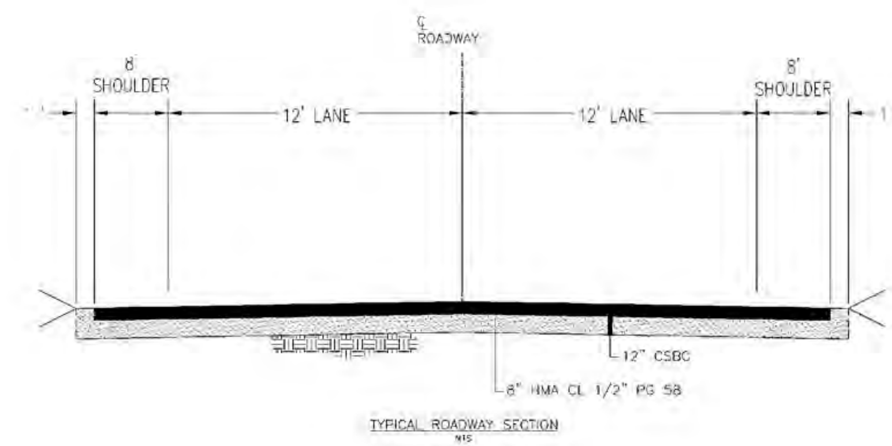
Site Name: Nooksack River Delta

Lead Contractor: ESA
Design Lead: Anchor QEA, G. Sassen, ASLA
Revised: USACE Petroff/Campbell December 2015

Exhibit B



TYPICAL BRIDGE SECTION (D)
NTS



TYPICAL ROADWAY SECTION (E)
NTS

ORTHO_2013_NAIP_WASHINGTON

Site Name: Nooksack River Delta

Lead Contractor: ESA
Design Lead: Anchor QEA, G. Sassen, ASLA
Revised: USACE Petroff/Campbell December 2015

Exhibit C

Puget Sound Nearshore Ecosystem Restoration Project
Feasibility Study
Hazardous, Toxic, and Radioactive Waste
Phase 1 Environmental Site Assessment

EXECUTIVE SUMMARY

The Seattle District Corps of Engineers (Corps), working collaboratively with the Washington Department of Fish and Wildlife (WDFW) as local sponsor, along with many other regional partners, has conducted a General Investigation (GI) to evaluate problems and potential solutions of ecosystem degradation and habitat loss in Puget Sound, Washington. The Puget Sound Nearshore Study (Nearshore Study) is authorized under Section 209 of the River and Harbor Act of 1962 (Pub. L. 87–874). The Corps and local sponsor are recommending implementation of restoration actions at three sites throughout the study area as the outcome of the Nearshore Study. Pursuant to Section 102(2)(C) of the National Environmental Policy Act (NEPA) of 1969, as amended, the U.S. Army Corps of Engineers is preparing an Integrated Feasibility Report/Environmental Impact Statement (FR/EIS) for the three restoration actions. The Phase 1 Environmental Site Assessment for the Nooksack River site is being conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments*, and ER 1165-2-132: *HTRW Guidance for Civil Works Projects*, except where noted below.

The assessment revealed several recognized environmental conditions in connection with areas adjacent to the project footprint, although all but one of these sites have no potential to impact the proposed project. The one exception is the Wilder Hazardous Waste Landfill site, located approximately a half mile east of the project footprint. There was determined to be a small risk of contaminants in the landfill being mobilized by flooding as a result of the proposed project. This risk is fully detailed in the Nooksack Risk Register, and in the project file Memorandum For Record dated 8 December 2015. This assessment is intended to reduce, but not eliminate, uncertainty regarding the existence of current and potential HTRW sites in connection with a property within reasonable limits of time and cost.

Exhibit C

Puget Sound Nearshore Ecosystem Restoration Project
Feasibility Study
Hazardous, Toxic, and Radioactive Waste
Phase 1 Environmental Site Assessment

Exhibit C

1.0 INTRODUCTION

1.1 Involved Parties

The Corps is the lead Federal agency for the Puget Sound Nearshore Ecosystem Restoration Project (PSNERP) Report. The non-Federal, cost-sharing sponsor is the Washington Department of Fish and Wildlife (WDFW). As the non-Federal sponsor, WDFW contributes 50 percent of the total feasibility study costs in the form of cash or in-kind contributions; a feasibility cost sharing agreement was executed in 2001, with amendments.

1.2 Authority

The Puget Sound Nearshore Study (Nearshore Study) is authorized under Section 209 of the River and Harbor Act of 1962 (Pub. L. 87–874).

1.3 Guidance and Policy

Corps policy providing guidance for consideration of issues and problems associated with hazardous, toxic, and radioactive wastes (HTRW), as defined in this regulation, which may be located within project boundaries or may affect or be affected by Corps Civil Works projects is contained in ER 1165-2-132, *Hazardous, Toxic, and Radioactive Waste Guidance for Civil Works Projects*, which defines HTRW as "…any material listed as a 'hazardous substance' under the Comprehensive Environmental Response, Compensation, Liability Act (CERCLA)". ASTM International (ASTM) Standard E 1527-13 *Standard Practice for Environmental Site Assessments: Phase I Environmental Site Assessment Process* provides a comprehensive guide for conducting an HTRW Assessment. An assessment identifies known or suspected releases of hazardous substances (recognized environmental conditions) based on records review, site visit, and interviews.

1.4 Scope of Work

The complete investigation serves to identify any recognized environmental condition, as defined in ASTM Standard E 1527-13. This site assessment documents known and suspected HTRW sites discovered through a search and review of all reasonably attainable federal, state, and local government information and records. A site visit, interviews with relevant stakeholders, and review of aerial photographs are also mandated under the above standard.

1.5 Significant Assumptions

This report identifies known and suspected environmental concerns, both past and present based on availability of information at the time of the assessment. It is possible that unreported disposal of waste or illegal activities impairing the environmental status of the properties may have occurred which could not be identified.

1.6 Limitations and Exceptions

This document deviates slightly from the exact procedures outlined in ASTM E1527-13. Specifically, no "User Provided Information" nor "Non-Scope Services" were provided, and those sections of the report were omitted. Also, due to the layout of the overall document to which this report will be incorporated, it was decided that no appendices were to be generated for this report. Additionally, it should be noted that portions of this report were conducted by separate agency entities that did not have the ability to coordinate their efforts. However, this does not change the results or outcome of the report.

1.7 Special Terms and Conditions

No special terms or conditions with respect to ER 1165-2-132 and ASTM E 1527-13 standards were made.

Exhibit C

1.8 User Reliance

In accordance with ASTM E 1527-13 Section 7.5.2.1 “Reliance,” the environmental professional is not required to independently verify the information provided by various sources but may rely on the information unless there is actual knowledge that certain information is incorrect or unless it is obvious that certain information is incorrect based on other information obtained during the course of the investigation or otherwise actually known to the investigators conducting the assessment. At the present time there is no indication that the information provided by the database search is incorrect.

2.0 SITE DESCRIPTION

2.1 Location and Legal Description

The “property”, as defined by the referenced ASTM standard, in this case includes several different properties within the Nooksack River valley. For the purposes of this assessment, the proposed Nooksack project footprint will serve as the “property” under review (See Figure 6-2 in the main body text).

2.2 Site and Vicinity General Characteristics

The physical setting of the subject property and vicinity is detailed in Section 4.6.4.5 of the main feasibility report.

3.0 RECORDS REVIEW

3.1 Standard Environmental Records

A records search was conducted on October 16, 2015 using a variety of sources, including EPA’s National Priority List Mapper, EPA’s EnviroFacts database, the Washington State Department of Ecology’s (Ecology) Toxics Cleanup Program (TCP) database, and Ecology’s Facility/Site database. Below are the parameters and results of the records search.

Parameter	Source	Minimum Search Distance (mi.)	Results
Federal NPL	EPA NPL Mapper	1	None
Federal Delisted NPL	EPA NPL Mapper	0.5	None
Federal CERCLIS	EnviroFacts	0.5	None
Federal RCRA Generators	EnviroFacts	Property and Adjoining Properties Only	None
Federal RCRA TSDs	EnviroFacts	0.5	None
Federal RCRA Corrective Action Sites	EnviroFacts	1	None
Federal and State ICs Registry	Ecology TCP	Property Only	None
Federal Toxic Release Inventory	EnviroFacts	0.5	None

Exhibit C

Parameter	Source	Minimum Search Distance (mi.)	Results
State and Tribal Cleanup Sites	Ecology TCP	1	5 findings (see below)
State and Tribal Landfills and/or Solid Waste Disposal Facilities	Ecology Facility Search	0.5	None
State and Tribal UST Registry	Ecology TCP	Property and Adjoining Properties Only	None
State and Tribal LUST	Ecology TCP	0.5	1 finding (Friese Hide & Tallow)
State and Tribal Brownfields	Ecology TCP	0.5	None

The records search identified 5 state cleanup sites and 1 leaking underground storage tank within the applicable search distances. The records search also identified several other sites that, despite not being within the search radius, are worth discussion. Each of these sites will be discussed below.

Of the identified cleanup sites, only one is located within the project footprint, known as the Mount Property. This property is located at the eastern end of Rayhorst Road, 100 to 200 feet from the Nooksack River. In 2003, eight 55 gallon barrels of used motor oil were reported to Whatcom County as leaking into the ground on the property. Upon contact from the County, the owner disposed of the drums, excavated the contaminated soil, and stockpiled the soil onsite for eventual removal. The property was sold as-is in 2005, and the soil pile was not observed. However, the site cannot be verified as cleaned up by Ecology due to the lack of documentation. For the purposes of the proposed Corps project however, this site does not pose a risk despite its continued placement on the state cleanup list.

The leaking underground storage tank identified in the record search triggered a cleanup action at a site known as Friese Hide and Tallow, located immediately north of 1524 Slater Road. The company that owns the property conducts cattle hide curing for eventual shipment overseas. The hides are cured using only salt, with no other chemicals. Ecology conducted a preliminary assessment (PA) in 1989 verifying this claim, and further determined that the salt runoff was not in such quantities as to warrant further consideration by the agency. However, Ecology received several reports of leaking underground storage tanks between 1991 and 1992. Two USTs were subsequently removed voluntarily in 1991. No closure or removal report was filed until 2010, when the owner conducted a site assessment (SA), finding residual petroleum contamination. The contractor determined that the contamination had not migrated to groundwater, and did not appear to be migrating at all. However, as a result the residual contamination and the lack of proper UST closure documentation, Ecology is currently planning to conduct its own SA in order to close the site for good. Given the initial findings of the 2010 SA, and the fact that Ecology is planning their own investigation, it is determined that this site will not impact the proposed Corps project, and no further investigation is needed.

Two of the three remaining cleanup sites identified in the record search actually refer to the same site. The Recomp of Washington site (also known as the Thermal Reduction Landfill) is located at 1524 Slater Road, and was a permitted municipal scale solid waste handler from 1974 to 1989. The site had a solid waste incinerator and an onsite landfill where the resultant ash and other wastes were placed. The incinerator and landfill were closed in 1989 following a series of County violations, and the landfill was graded, covered in low permeability clay, and covered by an HDPE liner. A leachate collection system was installed downgradient from the closed landfill, and was connected to a POTW. Groundwater and leachate monitoring was initiated in 1988 and continued until 2001, with no exceedences of applicable standards. In 2004, Ecology stated that there was no further action (NFA) needed, and a restrictive covenant was filed for the property to preclude future ground disturbance. Ecology conducted a periodic review in 2011 and identified no emerging problems. As a result, this site will not impact the proposed Corps action.

The final identified cleanup site is known as the Wilder hazardous waste landfill. Originally part of the Recomp of Washington landfill, this site is located immediately north of the Recomp site and east of Friese Hide and Tallow. The Recomp landfill began accepting and placing hazardous waste in this area starting in 1976-77, and operated until 1979, when it was closed for County compliance violations. During that span, this section of the landfill accepted over 1000 oil/resin drums, solvents, asbestos, insecticide, lignosite, incinerator ash from the Recomp incinerator, and pentachlorophenol. This portion of the landfill was covered in 3-4 ft. of ash and 5 ft. of dirt when it was closed in 1979. Since then there have been very few investigations on the hazardous waste area, besides a 1991 NFA determination from Ecology and a 2003 EPA PA. The 2003 assessment/investigation concluded that the hazardous waste source was not controlled, although due to the relative impermeability of the surrounding soils, contamination does not appear to be migrating. EPA went on to say that the surface water pathway appeared to be the most significant migration pathway, and since contaminants were not actively migrating in any pathways, EPA would take no further action.

Despite this determination by EPA, the Wilder hazardous waste landfill has the potential to impact the proposed Corps project. The Corps is proposing to essentially extend the Nooksack floodplain by the removal of levees on the river. This will cause areas to be flooded that were not previously impacted. One of these areas is Claypit Pond, located immediately west and downgradient from the Recomp, Friese Hide and Tallow, and Wilder sites discussed above. Claypit Pond, also known as Brennan Pond, has historically had elevated concentrations of contaminants in sediment, although these concentrations were never so elevated as to pose a risk to human health. These contaminants were suspected to come from three sites to the east. It was suspected that surface water drainage through culverts was the mechanism by which these contaminants moved into the pond.

As of today, there is no reason to believe that contaminants form these sites are flowing into the pond, either because the source is controlled (Recomp, Friese Hide and Tallow) or because the surrounding soils have low permeability (Wilder). However, there is a possibility that inundating Claypit Pond will change the groundwater gradient, or more generally the overall hydrology, such that contaminants originating from the hazardous waste landfill will more easily flow into the pond. Therefore the Corps project would be impacted by contamination from one or more of the identified sites. This potential risk is addressed in the PSNERP Nooksack Risk Register and in the Memorandum for Record dated 8 December 2015.

The final site identified during the record search is known as the Lummi Shore Dump. Although this site was not identified within the search distance parameters, its alleged location prompted further investigation. Ecology landfill records indicate the existence of former landfill on Lummi reservation land, immediately east of the intersection of Lummi Shore Road and Scott Road, less than a mile from the Nooksack River, in the floodplain. An official Lummi Nation document states that the landfill operated from 1961 to 1972, and accepted household waste from tribal and nontribal sources. Ecology records indicate that EPA assessed the property under CERCLA in 1986 and determined "No further action". That same year, Ecology declined to assess the property, citing Federal authority and the EPA determination. No records of this site exists at EPA, and there does not appear to have been any action taken since 1986. Additionally, a review of aerial photographs from 1998 to the present show no obvious locations for a landfill, although there is a small structure and small pond east of the intersection that are possible candidates. Given the lack of specific information concerning the landfill's location, the lack of obvious significant ground disturbance in the area, the ¾ of a mile from the river, and the lack of specific proposed Corps action on that stretch of the river, the suspected site is not expected to impact the Corps project.

Exhibit C

Figure 1: HTRW Records Search Results

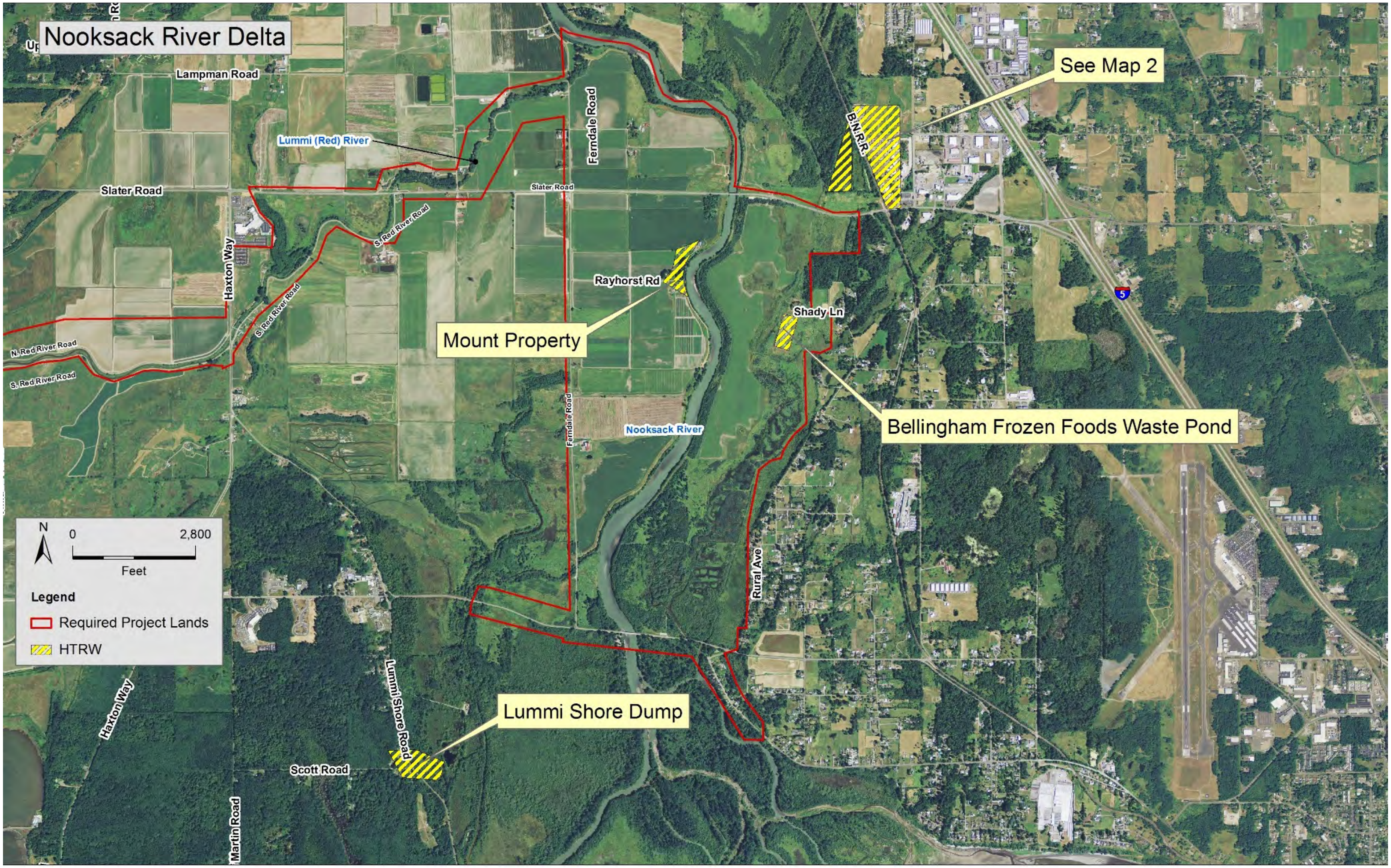
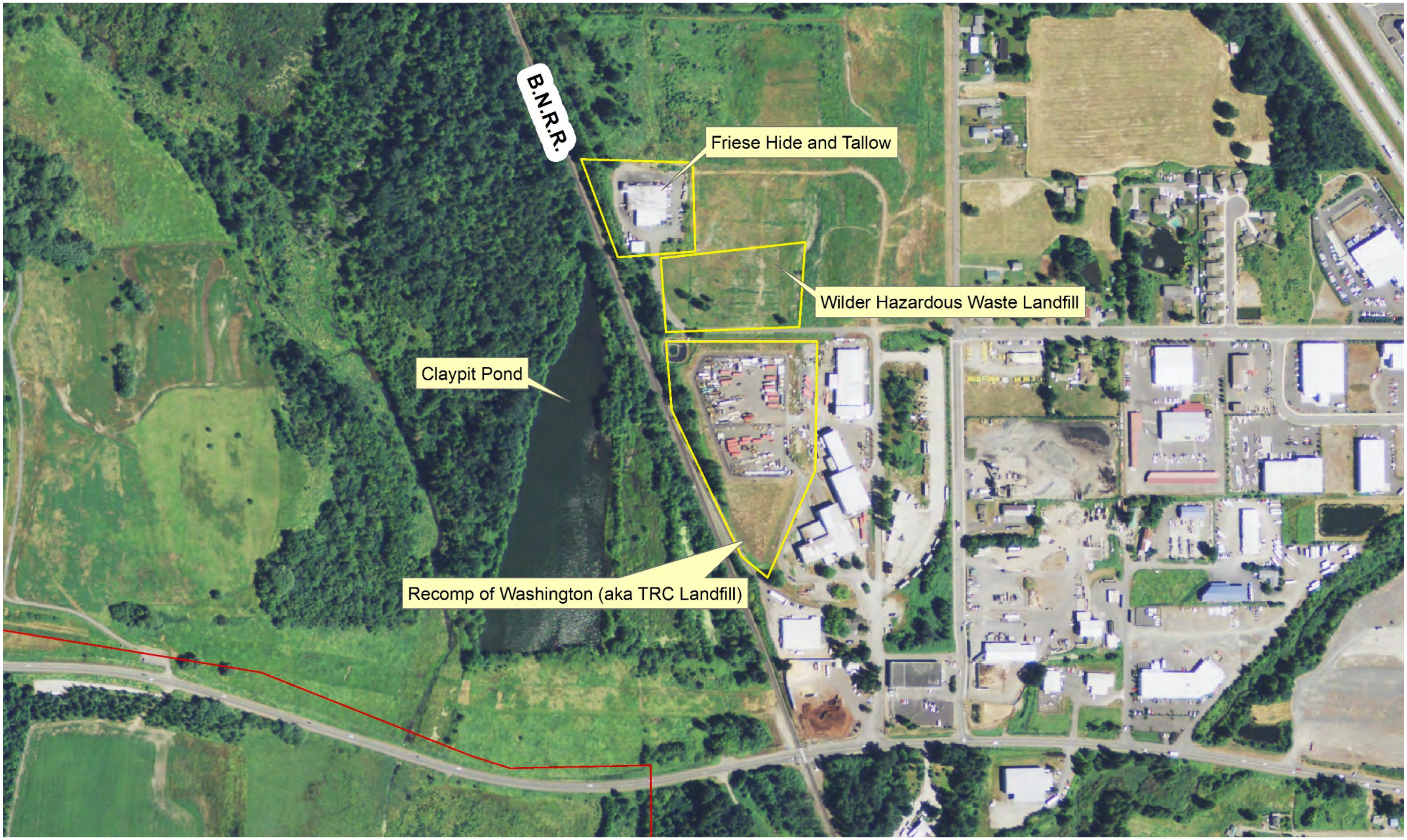


Exhibit C

Figure 2: HTRW Sites at 1524 Slater Road and Vicinity



3.2 Historical Records

A review of aerial photographs starting in 1963 to present land that is almost completely agricultural, with a small number of residences interspersed throughout. The only intensive changes within the project footprint are immediately east of the Nooksack River south of Slater Road. The aerial photographs from 1997 seem to indicate some sort of agricultural activity differing from the surrounding area; there were six circular green areas with several small buildings, as well as a small (approx. 4 acre) rectangular lagoon. Further investigation indicated that this was the Bellingham Frozen Foods Wastewater Treatment Facility. Wastewater is generated at a frozen vegetable plant in Bellingham, pumped into the rectangular lagoon to be treated, then sprayed into the six circular spray fields. The facility was purchased by the Washington State Department of Fish and Wildlife (WDFW) in 1998, and the spray fields were decommissioned. An environmental assessment was completed concerning the waste lagoon in 2003, and revealed no concerns. As a result, this site will not impact the proposed Corps project.

Numerous dairy farms were noted both in and around the project footprint during both the records search and aerial photograph review. Many of these facilities have National Pollutant Discharge Elimination System (NPDES) permits, and feature waste ponds related to the dairy. One facility, the Frank Moser Dairy, has a documented violation of this NPDES permit. While these discharges and waste ponds are not releases per the ASTM E1527-13 standard, it should be noted that the contents of these waste ponds may be unknown, and could be impacted in the case of inundation as a result of levee removal.

3.3 Additional Environmental Record Sources

There are no additional environmental record sources included in this assessment.

4.0 SITE RECONNAISSANCE

A site visit was conducted by the USFWS on February 1, 2011. The action area is currently owned by multiple landowners, and used primarily for agricultural purposes. There is potential for chemical or solvent storage at some of these sites, machinery and equipment repair areas, above ground storage tanks, older structures that may contain asbestos or lead. A visual survey of the action area revealed no known or suspected contaminant releases or spills. However, the Phase I site visit did not visit all parts of the action area, due to private property restrictions.

An interview was conducted in person by the USFWS on February 1, 2011 with Jeremy Freimund of the Lummi Nation, and Steve Seymour with WDFW. Telephone interviews were conducted by the USFWS with Carol Dorn and Gayle Garbush of Ecology, and Kye Iris with WDFW. No records of these interviews exist.

5.0 FINDINGS AND CONCLUSION

This Phase 1 Environmental Site Assessment was conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments,* and ER 1165-2-132: *HTRW Guidance for Civil Works Projects*. The assessment was initially conducted in 2011 by Cindy Schexnider, Specialist with the USFWS, and updated in 2015 by David Clark, remediation biologist at the Corps.

The assessment revealed several recognized environmental conditions in connection with areas adjacent to the project footprint, although all but one of these sites have no potential to impact the proposed project. The one exception is the Wilder Hazardous Waste Landfill site, located approximately a half mile east of the project footprint. There was determined to be a small risk of contaminants in the landfill being mobilized by flooding as a result of the proposed project. This risk is fully detailed in the Nooksack Risk Register, and in the project file Memorandum For Record dated 8 December 2015.

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Section 3 – North Fork Skagit River Delta

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Section 3: North Fork Skagit River Delta

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3-1 GENERAL – NORTH FORK SKAGIT RIVER DELTA

3-1.1 Overview of Restoration Site

The North Fork of the Skagit River empties into Skagit Bay just south of the town of La Conner. The proposed action is located between the former inlet of Dry Slough and the western terminus of the dike system near Rawlins Road. Extensive diking of the North Fork Skagit River has caused substantial loss of tidally influenced wetlands and their associated tidal channels. River levees have disconnected the river from its floodplain, eliminated off-channel habitats, reduced floodplain area, and constrained the river channel.

The North Fork Skagit site was selected to address River Delta restoration objectives to protect and restore freshwater input and tidal processes where major river floodplains meet marine waters. Target ecosystem processes include:

- Tidal flow
- Freshwater input (including alluvial sediment delivery)
- Erosion and accretion of sediments
- Distributary channel migration
- Tidal channel formation and maintenance
- Detritus recruitment and retention
- Exchange of aquatic organisms

The proposed restoration would remove or set back flood risk management dikes on both sides of the river, restore natural levees, and restore 256 acres of rare tidal freshwater marsh. Key restoration elements of this action include the following:

- Lower and breach dikes; construct new dikes to maintain existing level of flood risk management
- Excavate tidal channel network
- Remove shore armor, buildings, pavement, boat ramp, and roads

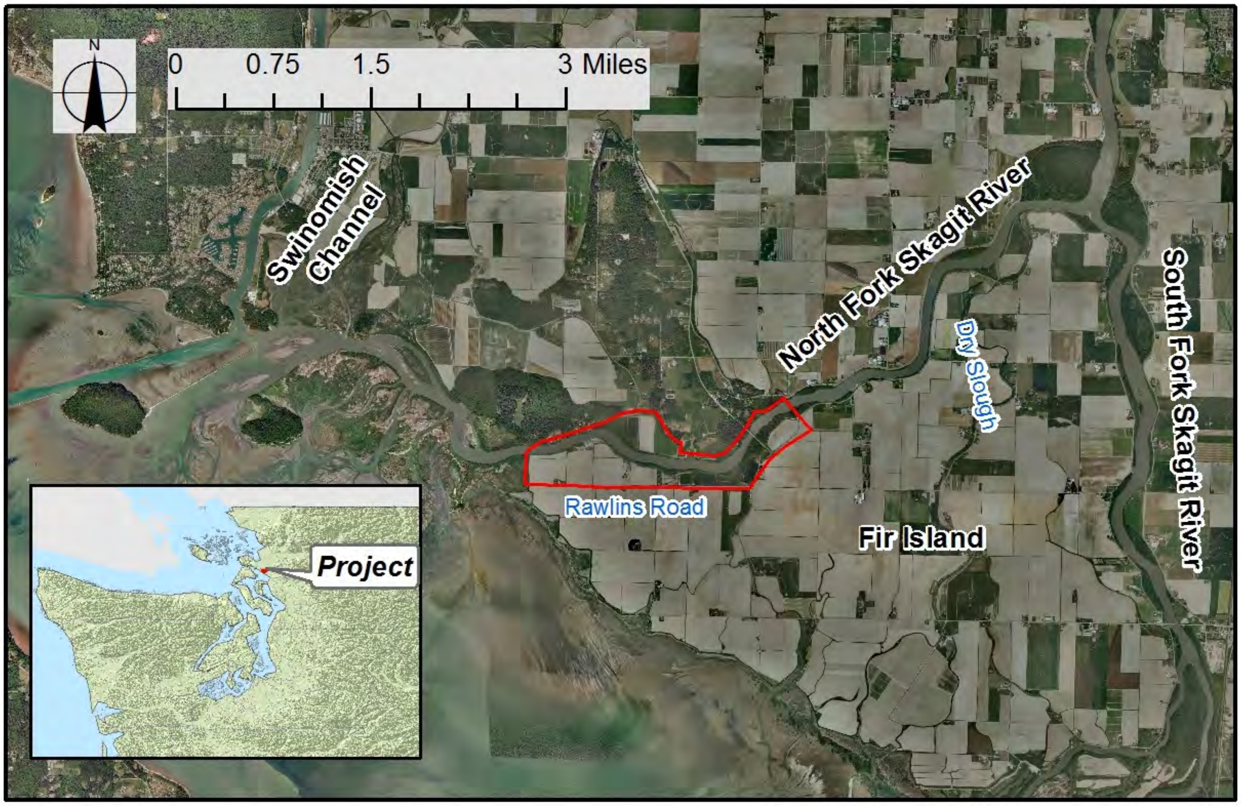

Figure 3-1-1. North Fork Skagit River and vicinity.

3-2 HYDROLOGY AND HYDRAULICS

The North Fork site lies in the estuary of the North Fork of the Skagit River. The Skagit River drains a watershed of over 2,700 square miles and splits into North and South Fork distributary channels about 10 miles upstream of the river mouths. The Skagit River downstream from Mount Vernon is fully confined by levees on both banks. The North and South Forks are similarly confined until they approach Skagit Bay maintaining a stable split in discharge. Both forks have aggraded similarly since observation in the 1970s. The North Fork Skagit River carries about 60 % of the flow at low flows. At high flows the split between the two forks is about 50/50. Average annual precipitation for this watershed is 101 inches per year. The site is located on the left bank of the North Fork Skagit River. Figure 3-2-1 shows a map of the watershed, including the North Fork of the Skagit River and the local drainages that affect The North Fork site.

The majority of the North Fork Skagit River Delta site lies below the 100-year flood elevation. The area is primarily agricultural. Blake's Resort, a small recreational vehicle (RV) park and dock, lies within the site adjacent to the left bank of the North Fork Skagit River. Highway 534, known as Best Road, crosses near the middle of the site and is a primary access to Fir Island, connecting it to Interstate 5 and State Route 20. The Best Road Bridge and its approaches are above the 100-year floodplain.

The intent of the project is to increase the frequency of both tidal flow and riverine flooding into the site for the purpose of habitat restoration. This will require removal of all structures and utilities within the project area. The project will lower the existing left bank levee and portions of the right bank shore armor, create a tidal channel network and build a setback levee on the left bank adjacent to Rawlins Road.

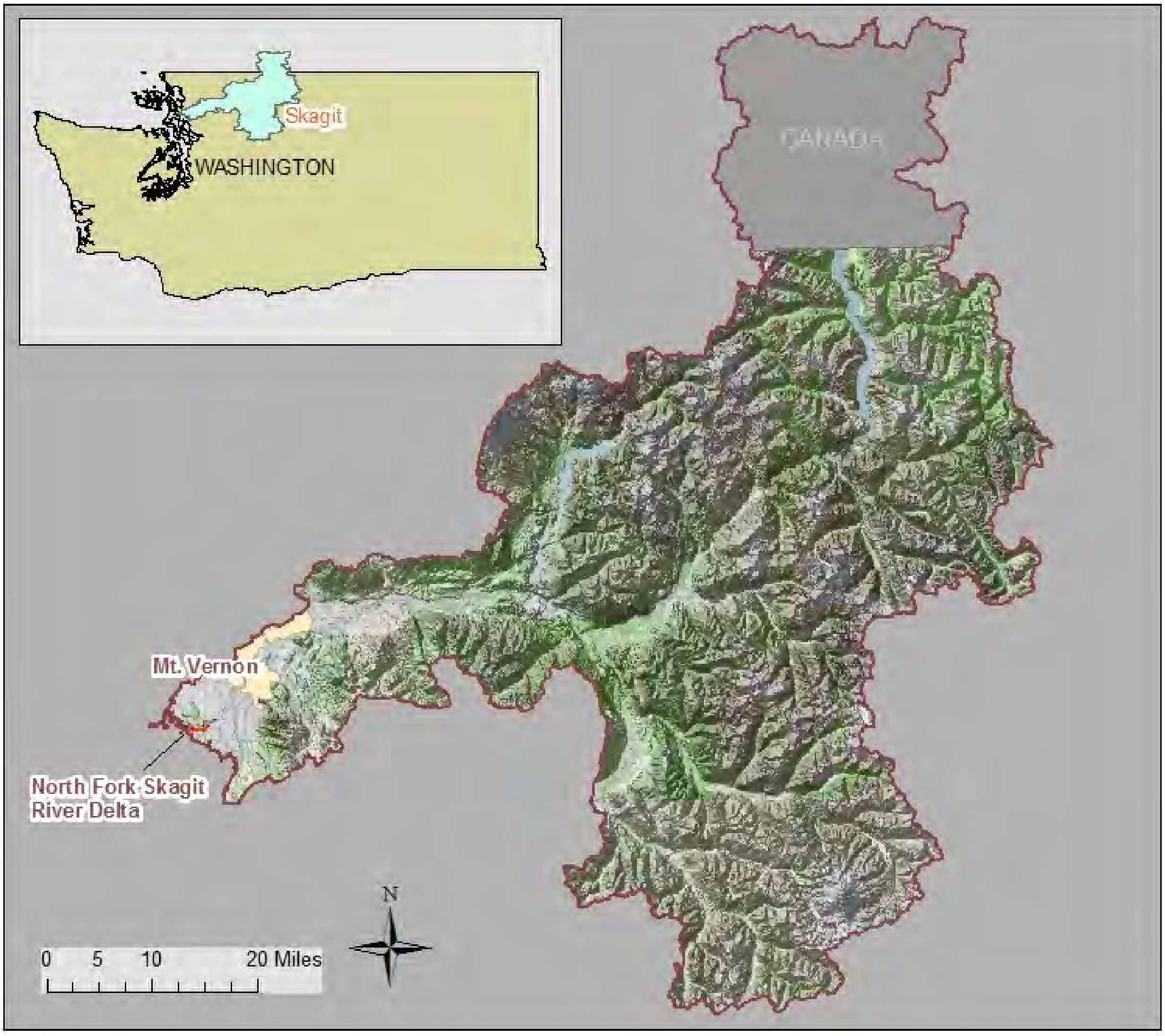

Figure 3-2-1. The North Fork Skagit River Delta site – Skagit River watershed.

The hydraulics and hydrology for all restoration sites in the Puget Sound Nearshore Ecosystem Restoration Project were evaluated using an area of potential hydraulic effects specific to the construction requirements for each particular site. The limits of the area for this site were established using 100-year Base Flood Elevations (BFEs) derived from a combination of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps and Flood Insurance Studies as well as U.S. Army Corps of Engineers (USACE) BFE determinations.

According to the 1% Annual Exceedance Probability (100-year) BFE as determined by the FEMA flood insurance mapping for unincorporated areas of Skagit County, community 530151 (revised 1985), the entire site lies well within the 100-year floodplain and away from floodplain boundaries. Figure 3-2-2 shows the area of potential hydraulic effects for the North Fork site as identical with the restoration site boundaries. The FEMA BFE varies between 21 feet (NAVD88) at the upstream limit of the site and 12 feet (NAVD88) downstream and depends primarily on flooding from the Skagit River. Although the river is tidally influenced in this reach, the coastal BFE of 12.2 feet (NAVD88) is lower than the elevations from river flooding for all but the most downstream 2000 feet of the leveed area.

This delineation of the area of potential hydraulic effects assumes that the planned lowering and setting back of levees will not substantially adversely affect levees or infrastructure on adjacent properties. There is expected to be some slight lowering of flood levels in the vicinity of the site because of the levee setback.

In addition, there may some short term changes in sedimentation patterns downstream of the site as the river adjusts to the expanded cross-section. During Project Engineering and Design (PED), the current one-dimensional hydrodynamic (HECRAS) model of the Skagit River developed as part of the Skagit River General Investigation will be updated to reflect with and without project geometry and to confirm the extent of hydraulic effects from the restoration. Additional modeling during PED will include consideration of tidal influences. A three-dimensional estuarine and coastal ocean model, developed at the Pacific Northwest National Laboratory (PNNL) with support from the Skagit Watershed Council and the Skagit River System Corporative, has been successfully used to model and evaluate restoration projects in the Skagit watershed and is available to use during the project design phase (Yang and Khangaonkar, 2006).

Figure 3-2-2. The North Fork Skagit River Delta: Area of potential hydraulic effects.

The Ecosystem Output Model (EOM), described in Appendix G utilized an area of restored process determined as follows:

The upland portion of each analysis area was delineated to ensure that the area included all stressor distributions within defined buffer distances from the shoreline. In the aquatic areas, the shape of the Analysis Area was determined by a combination of:

- The GIS area provided initially by the design team and the associated parcel map for the proposed action
- Ensuring an area encompassed all delineated tidal wetlands
- For any Analysis Area that extended through an aquatic area, boundaries were established approximately perpendicular to the shoreline orientation where the upland meets the shoreline.

The area of restored process at North Fork Skagit is shown in Figure 3-2-3 as 256 acres. For more information, please refer to Puget Sound Nearshore Ecosystem Restoration Project Fish and Wildlife Coordination Act Section 2(b) report in Appendix J, Environmental Compliance Documentation.

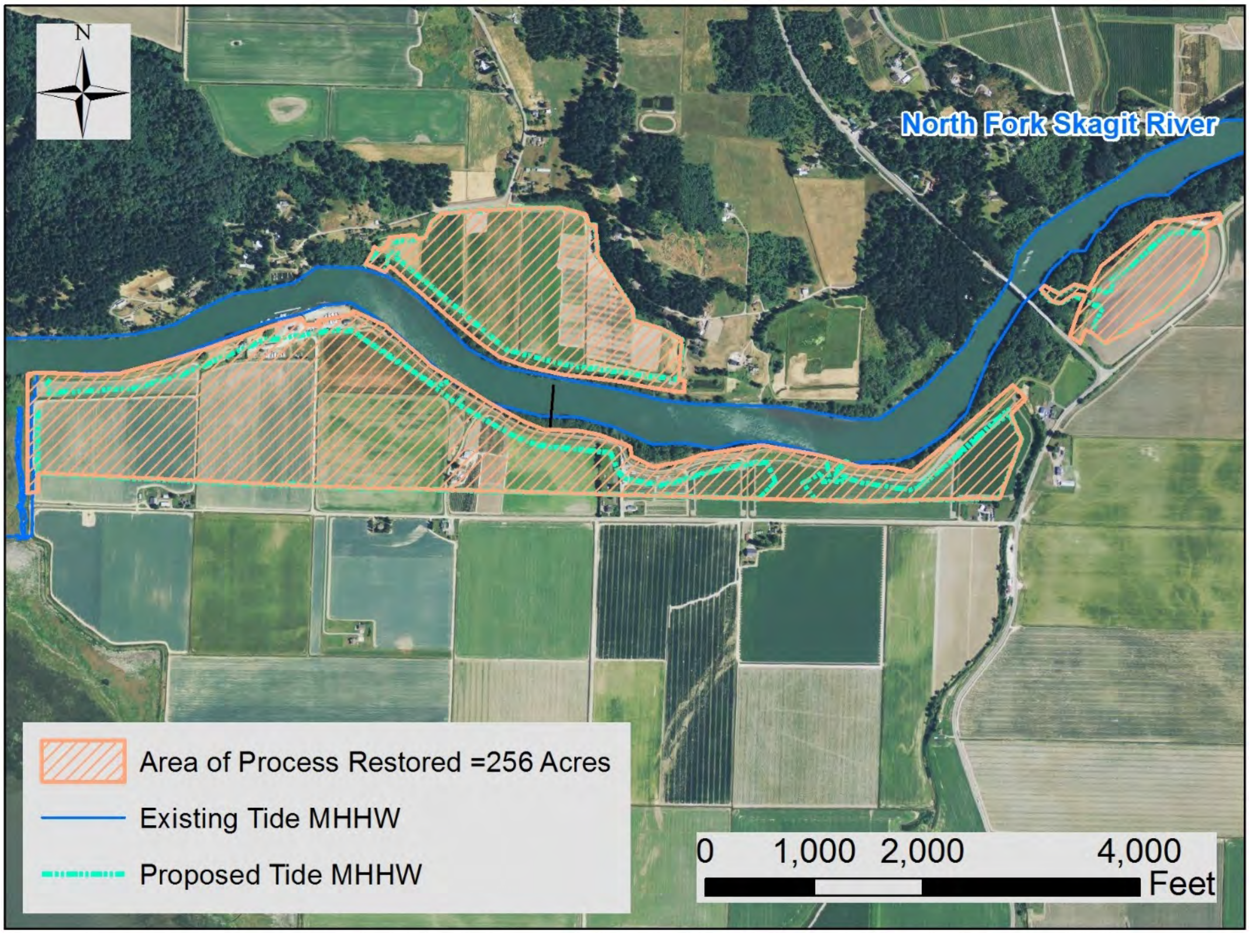

Figure 3-2-3. Area of process restored used in ecosystem output model at the North Fork Skagit River Delta.

3-2.1 Functional Design Requirements

This section describes the hydrologic and hydraulic setting for the site and the intended hydraulic consequences of the design features.

3-2.1.1 Consequences of flows exceeding discharge capacity of the project

The proposed action will restore the riverine floodplain and tidal connectivity along the lower reach of the North Fork of the Skagit River. This will require re-constructing a flood risk management levee further inland. The existing levee would be lowered and selectively breached to allow inundation of the estuarine emergent marsh and sustain back-channel habitat. Forested floodplain habitat would be created along the lowered levee adjacent to the mainstem river channel. A setback levee will be built to provide the same level of flood risk management as the existing left bank levee. The existing levee system around Fir Island is operated by Skagit Diking District 22. Skagit Diking District 22 is a connected levee system which, combined with a system of sea dikes, provides flood risk management to all of Fir Island (Figure 3- 2-4). Fir Island is a flat low-lying part of the Skagit Delta that relies on agriculture and tourism, including fishing, wildlife viewing, as well as popular and economically significant tulip and daffodil festivals. The Diking District 22 levee system has a 2% Annual Exceedance Probability (AEP) (50-year) residual risk based on levee screening conducted by the Corps. This levee system is part of the Corps PL84-99 Program and received a Levee Safety Action Classification (LSAC) of 4 – low risk in the 2014 Levee Screening Program fact sheet. Since the setback levee is to be built with the same residual risk as the current levee system, flood consequences for the remainder of Fir Island under the with-project condition would be the same as for the existing system.

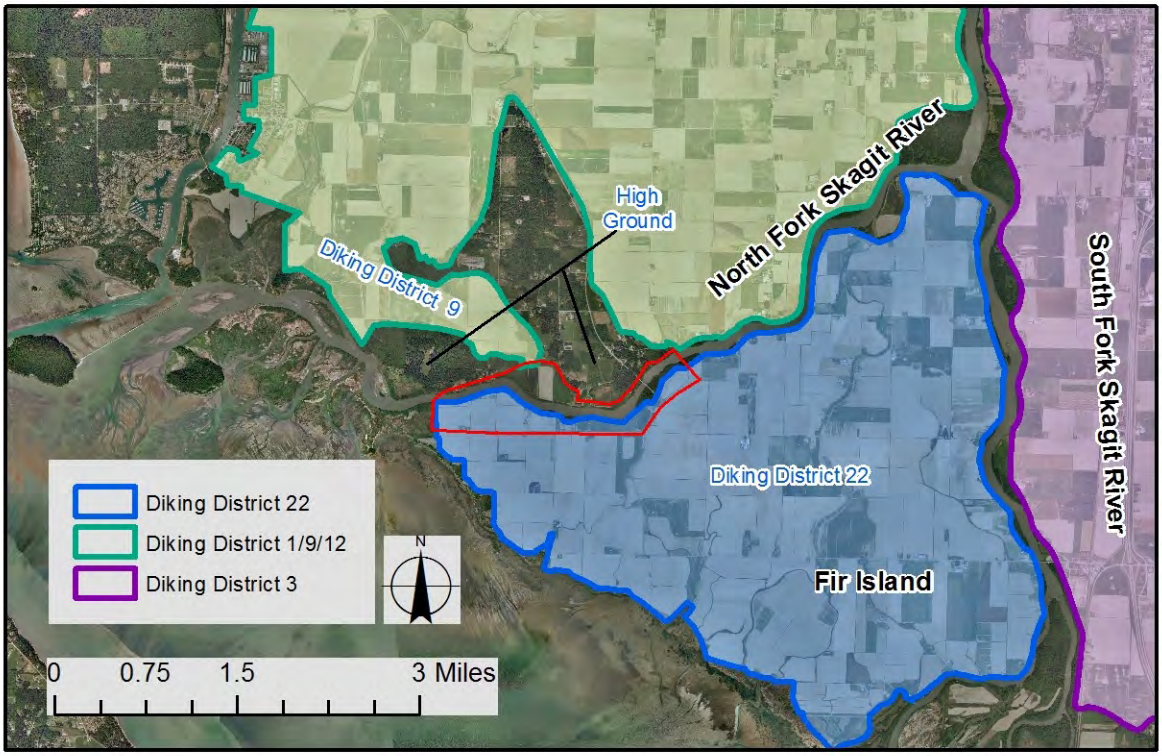

Figure 3- 2-4. North Fork Skagit River Delta levee system.

On the right bank of the North Fork Skagit River, the Diking District 9 levee with 0.2% AEP level of flood risk management lies just north of the project boundary and will not be affected by the project.

3-2.1.2 Project-induced changes obligating mitigation

No compensatory mitigation is included for this site as none is required. Implementation of restoration at this site would involve only minor construction activities in the aquatic environment. The restoration actions would have negligible, short-term construction related effects. All of these minor and temporary effects can be avoided and minimized through construction designs and standard best management practices (BMPs). Specific measurable and enforceable measures would be developed based on the specific effects of the project.

3-2.1.3 Discharge-frequency relationships

The site is located on both left and right banks of the North Fork Skagit River. The predictions for river discharge for this area are taken from the Skagit River Flood Risk Management Feasibility Study, Hydrology Technical Document (USACE 2013). The estimates for the Skagit River near Sedro-Woolley and at the Best Road Bridge are shown in Table 3-2-1. Low probability flood discharges at the site are significantly lower at the site due to significant overbank flow and the flow split at the junction of the North Fork and South Fork Skagit Rivers.

Table 3-2-1. Peak discharge – frequency predictions for mainstem Skagit River at Sedro-Woolley and Best Road Bridge using HECRAS routed regulated synthetic hydrographs.

Location	10% AEP (cfs)	2% AEP (cfs)	1% AEP (cfs)	0.2% AEP (cfs)
Sedro Woolley	133,000	197,400	235,700	325,400
Best Road Bridge	61,600	83,300	85,400	86,300

3-2.1.4 0.2% Annual Exceedance Probability (500-year) flood

The area of potential hydraulic effect for the North Fork site is dominated by fluvial flows with some influence from coastal flooding. The only critical infrastructure at this site is the Best Road Bridge. The 0.2% AEP flood elevations are essentially the same as the 1% AEP flood elevations, thus the 0.2% AEP flood will not be evaluated separately.

3-2.1.5 Stage-discharge relationships

Current stage-discharge relationships were computed for the North Fork Skagit River in the Skagit River Flood Risk Management Feasibility Study, Hydrology Technical Document (USACE 2013). Table 3-2-2 shows the computed elevations at the Best Road Bridge. The 2%, 1% and 0.2% AEP elevations are essentially the same. In order to forecast new stage-discharge relationships and effects on adjacent levees in the Skagit Delta, a revised hydraulic model will have to be implemented that reflects the proposed geometry of the site. This will be addressed during PED.

Table 3-2-2. Stage-discharge relations as shown in recent USACE Skagit River Flood Risk Management Feasibility Study for the North Fork Skagit River Delta site (USACE 2013).

Location	10% AEP Stage (feet NAVD88)	2% AEP Stage (feet NAVD88)	1% AEP Stage (feet NAVD88)	0.2% AEP Stage (feet NAVD88)
Best Road Bridge	17.5	21.0	21.0	21.0

3-2.1.6 Flow duration

At present, it is not anticipated that a flow duration analysis will be required at the site.

3-2.1.7 Flood inundation boundaries and flood stage hydrographs

Figure 3-2-5 shows the effective 1% AEP (100-year) flood inundation levels from the Skagit County FEMA Flood Insurance Study (FEMA 1985). In order to forecast any changes in flooding pattern, a revised hydraulic model will have to be implemented that reflects the proposed geometry. This will be addressed during PED.

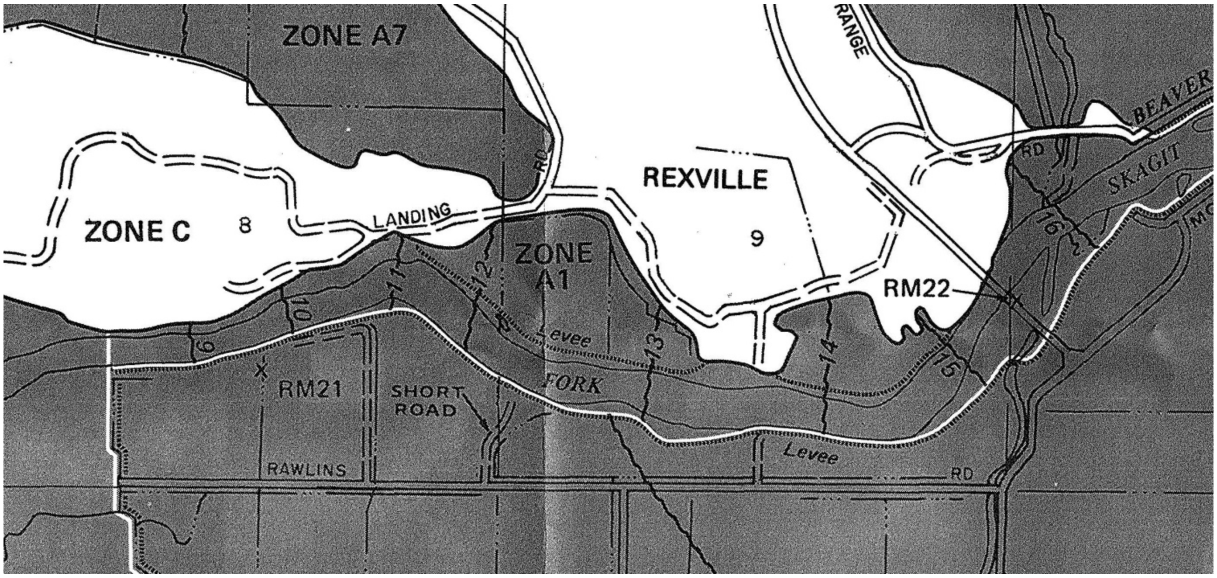

Figure 3-2-5. Effective FEMA 1% AEP (100-year) flood zone as adapted from Flood Insurance Rate Map (FIRM 530151 0425C) (FEMA 1985) (Elevations in NGVD29; NGVD29 + 3.79 feet = NAVD 88).

3-2.1.8 Reservoir yields

No reservoirs are planned as part of this site.

3-2.1.9 Risk and uncertainty analysis for sizing of the project under study

The existing levees would be lowered to elevations similar to those of the natural levees (about 13.5 feet above MLLW, 12 feet NAVD88), which are formed during flood events and exist further downstream. The setback levees will be built to provide the same level of flood risk management at Fir Island (AEP of 2%) as the existing levees.

Channel sizing:

Channels will be excavated to equilibrium dimensions as described in the *Applied Geomorphology Guidelines* (Attachment B). Channel top widths will vary between 30 and 100 feet, with depths between 4 and 12 feet below existing grade. A channel depth of 12 feet will allow for some infilling of the channels without impacting the performance of the restoration. How the new tidal channels will evolve is unknown, but the concept of natural evolution minimizes any risk because there are no specific discharge requirements for the new channels.

Sea Level Change

The North Fork site is located in the Whidbey Sub-basin of Puget Sound. Sea level change calculations for the Whidbey Sub-basin are based on the Seattle tide gauge and are calculated using the guidance in ER 1100-2-8162, Incorporating Sea Level Change in Civil Works Programs, and ETL-1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses and Adaptation (USACE 2013, 2014). Table 3-2-3 shows the range of sea level change projections for the 50-year project life as well as the 100-year horizon assuming a project start date in 2020. Changes are referenced to 1992, which is the midpoint of the most recent National Tidal Datum Epoch as established by NOAA. The high rate calculations indicate a sea level rise of 2.3 feet in 50 years after project start and a rise of 6.5 feet after 100 years.

The largest risk associated with sea level change at this site is the displacement of habitat upstream, with vegetated marshes becoming intertidal habitat and intertidal habitat becoming sub-tidal habitat. Tidal marshes can adapt to sea level change by building elevation to keep pace with the rising water levels, but this requires an adequate supply of sediment and/or organic matter accumulation. Future studies should include a sedimentation analysis to determine what impact the restoration will have on sedimentation rates and if there is sufficient sediment accumulation to keep pace with the projected sea level change. The sedimentation analysis will be used to fine tune initial channel design and to assess the anticipated environmental benefits from the project over the design life.

Table 3-2-3. Projected sea level change (feet) Seattle (Gauge 9447130). Source: USACE Sea-Level Change Curve Calculator (2015.46).

Year	Low (feet)	Intermediate (feet)	High (feet)
1992	0	0	0
1995	0.02	0.02	0.02
2000	0.05	0.06	0.08
2010	0.12	0.15	0.24
2020	0.19	0.26	0.48
2030	0.26	0.39	0.79
2040	0.32	0.53	1.18
2050	0.39	0.69	1.64
2060	0.46	0.87	2.17
2070	0.53	1.07	2.78
2080	0.6	1.28	3.47
2090	0.66	1.52	4.22
2100	0.73	1.77	5.05
2110	0.8	2.04	5.96
2120	0.87	2.32	6.94

Figure 3-2-6 shows the top of levee profile for the Diking District 22 levee on the left bank of the North Fork Skagit River along with riverine water surface profiles for 50% through 1% flood events. These water surface profiles were developed using HEC-RAS for the Skagit River Flood Risk Management Feasibility Study (USACE 2013b) and represent the without-project condition. In the figure, Station 52500 is at the upstream end of the site and Station 65300 is at the downstream end. Although the top of levee is well above the 1% AEP water surface in this part of the levee system, other parts of the system are more vulnerable, leading to the overall 2% AEP residual risk for the entire Diking District 22. The current coastal BFE of 12.2 feet NAVD88 affects the lowest 2000 feet of the site but is contained by the levee. The incrementally higher lines represent respectively the 100-year low, intermediate and high assumptions for sea level change at the North Fork site. For the highest, 100-year high rate of sea level change, the coastal BFE would overtop the lowest 5000 feet of the levee. At this elevation the sea dikes that protect Fir Island, and that join the levee at its downstream end would also be subject to overtopping, so both sides of the levee would be affected by the BFE. The most downstream end of the levee would begin to be overtopped by the base coastal flood in 2070 (about 50 years post-construction) for the high rate of sea level change and in 2140 (about 120 years after construction) for the intermediate rate of change. This performance will be reassessed in PED once levee crest elevations have been refined.

The intent of the project is to restore ecosystem process by removing stressors. For levees, the project is designed to meet the current level of residual risk for Diking District 22 (2% AEP) only. Since the project is not authorized for flood control, future adaptation of levees for sea level change is not within the project authority. Since the current Diking District 22 levee system includes long segments of coastal dikes, it is assumed that adaptation of the Fir Island levee system to sea level change would be undertaken as a system-wide measure either by the diking district or under a separate authority. Elements of the levee design that will be finalized in PED will include robustness considerations such as shallow side slopes and wide crest-widths. Some of these features may be useable by others to adapt to future sea level change.

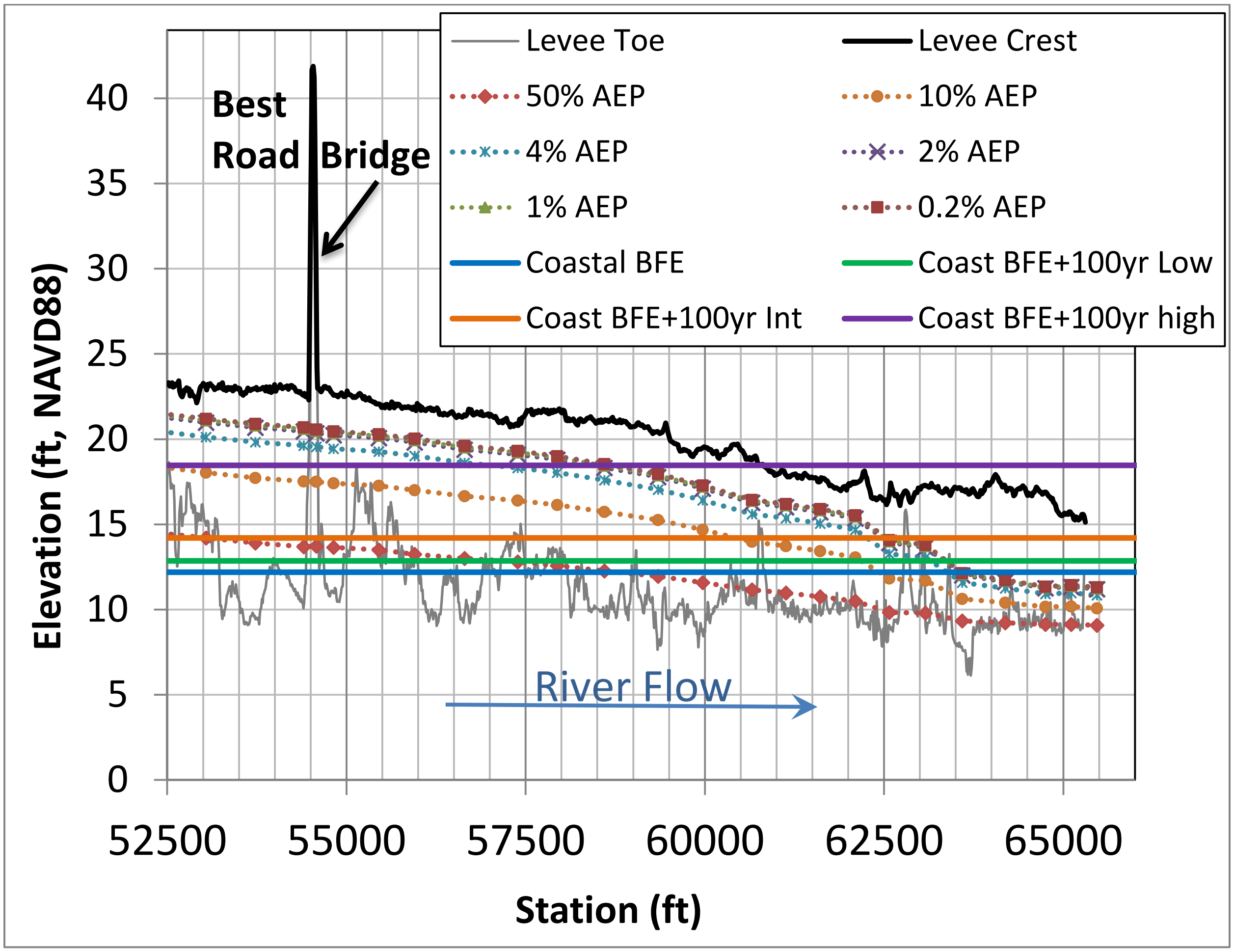

Figure 3-2-6. Effects of potential sea level change on the base flood elevation (without project) at the North Fork Skagit River Delta site.

Figure 3-2-7 shows the project features compared to current and possible future Mean Higher High Water (MHHW) elevations. The base level for the riparian berm breaches are below Mean Lower Low Water (MLLW), while the base levels for new tidal channels are at or slightly higher than the MLLW. The top of the riparian berms are above the current MHHW as well as above the 100 low and 100 year intermediate. For the 100-year high rate of sea level change, the riparian berms would be below MHHW at the end of 100 years.

The largest consequence associated with sea level change at this site is the displacement of habitat upstream, with freshwater habitat becoming intertidal habitat and intertidal habitat becoming sub-tidal habitat. Tidal marshes can adapt to sea level change by building elevation to keep pace with the rising water levels, with an adequate supply of sediment and/or organic matter accumulation. Further study in PED will include a sedimentation analysis to determine what impact the restoration will have on sedimentation rates, if sediment accumulation will keep pace with the projected sea level change, or if changes in habitat types are expected.

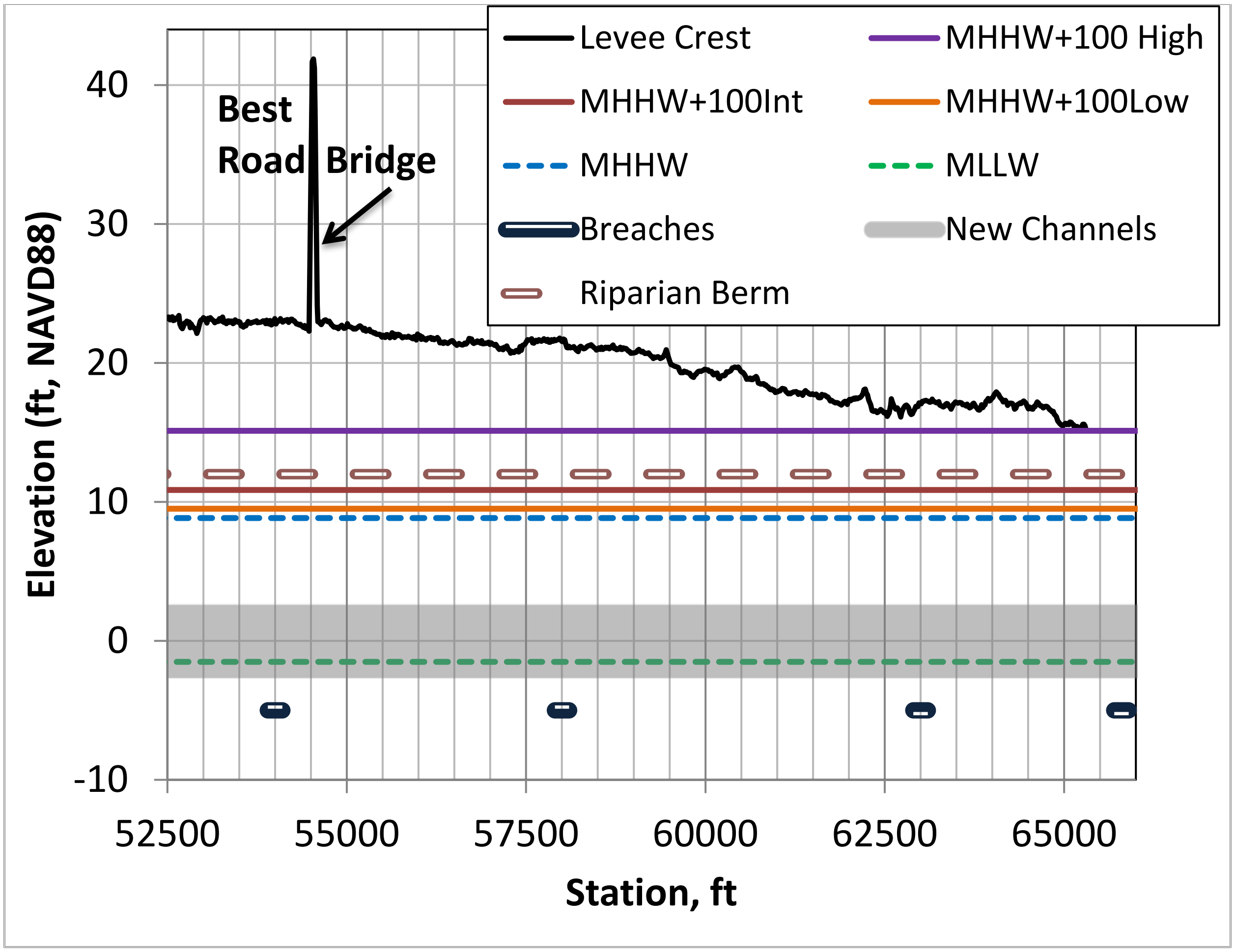

Figure 3-2-7. Effects of potential sea level change on diurnal tide fluctuations at the North Fork Skagit River Delta site.

<u>Climate Change</u>

ECB No. 2014-10 (USACE 2014) provides initial guidance for incorporating climate change information in hydrologic analyses in accordance with the USACE overarching climate change adaptation policy. There is a strong consensus among recent studies that future storm events in the Pacific Northwest region will be more intense and more frequent compared to the recent past (USACE 2015). The overall projected trends for the Pacific Northwest are summarized in the FR/EIS Section 3.6.5.1.

The Skagit River Basin Climate Science Report (Lee and Hamlet 2011), prepared for Envision Skagit and Skagit County, explores the potential impacts of climate change on Skagit River hydrology. The report describes potential changes in seasonal discharges (higher winter flows and lower summer flows) and increasing flood discharges. The Lee and Hamlet report forecasts flood peak increases could range from 4 to 64 percent by 2040, with an average of 23 percent, and 0 to 98 percent by 2080, with an average increase of 40 percent.

These projections would yield an average peak flow increase of 33% at the end of the project planning horizon in 2065. The risk of floods on the order of the current 1% AEP could increase to about 4% AEP. The frequency of flooding could increase significantly in the study area, however the amount of flow will vary by location. At the project site, since high flows cause so much overbank flow upstream, the relative increases in peak flow, water surface and flow speed are anticipated to be relatively minor. This is supported by the fact that the 0.2% AEP (500-year) water surface profile is on the order of 0.1 feet higher than the 1% AEP (100-year) water surface in the area of the project.

Higher winter flows, especially flood discharges, could result in an increase in sediment transport. In the Skagit River, discharges over 50,000 cfs at Mount Vernon now produce 21% of the annual sediment yield. Those flows are currently only exceeded 1% of the time on an annual basis. Should climate change result in higher discharges becoming more common, the sediment yield may increase accordingly. Higher sediment yields may cause increased deposition around the mouths of the North and South Fork Skagit Rivers. The potential impacts to river deposition are harder to estimate. That will depend on the future balance between sediment transport potential and the available sediment supply. The sedimentation analysis conducted during PED will provide an indication of whether more frequent high flow events will trend towards higher rates of sedimentation at the North Fork Site.

3-2.1.10 Water quality conditions

Water quality information has not been reviewed in detail for this site. The restoration is not anticipated to generate any long-term effects on surface water quality. Anticipated water quality effects are as follows:

- Construction-related turbidity and suspension of sediments may occur due to dike lowering and breaching. Sediment control will have to be carefully considered in the construction planning.
- Temporary increases in sedimentation may occur downstream of the site because of the release of sediment during the formation of any new distributary channels. These effects, together with other sedimentation issues, will be evaluated during PED.
- Levee lowering and channel construction may increase salinity within the site due to the increased tidal prism. Since the goal is to restore historic conditions, restoration of historic salinity patterns is presumed to be a desirable outcome. If needed, water quality sampling and analysis of water quality effects can take place during PED.

Rawlins Road, to the west of the site, was the subject of a study by Battelle (Yang and Khangaonkar, 2006) in connection with several restoration options around Fir Island. As part of the study, velocity, tidal elevation, salinity, and temperature time histories were collected at the northwest corner of the site using an S4 current meter mooring station. Instantaneous salinity and temperature profiles were also obtained near the station during instrument deployment and retrieval. The salinity measured during three weeks in June of 2005 averaged around 0.12 PSU (Practical Salinity Units) with occasional excursions to around 0.15 – 0.16 PSU.

3-2.1.11 Groundwater conditions

No groundwater information has been reviewed for this site. The lowering and breaching of dikes will allow an increased tidal prism within the site which may be accompanied by saltwater intrusion. Since the goal is to restore historical conditions, restoration of historical salinity patterns is presumed to be a desirable outcome.

3-2.1.12 Preliminary project regulation plan

The setback levees will be built to provide the same level of residual risk at Fir Island, an AEP of 2%, as the existing levees.

3-2.1.13 Preliminary real estate taking line elevations

The current real estate limits are delineated by the construction area, staging areas, and access roads and include the entire potential area of hydraulic effects. Real estate assumptions, valuations, and planning documents have been appropriately scaled for the current level of design. As additional surveys, modeling, and design are completed during the PED phase, the real estate documentation will be modified accordingly. For the current real estate status, refer to the Integrated Final Feasibility Report and Environmental Impact Statement (FR/EIS), Appendix C, *Real Estate Plan*.

3-2.1.14 Criteria for facility/utility relocations

Buildings, roads, utilities, and other hard structures/surfaces within the setback area, including Blake's Resort, will be removed. For further details on the utility relocations see Section 3-6.3.

3-2.1.15 Criteria for identification of flowage easements required for project function

No flowage easements are anticipated for this site. This will be reviewed and confirmed during PED.

3-2.1.16 Criteria in support of project OMRR&R requirements

Operations, maintenance, repair, replacement and rehabilitation needs associated with the hydraulic function of the site are as follows:

- This restoration concept relies on the natural evolution of the floodplain and channels, therefore no hydraulic performance maintenance is anticipated. If site specific objectives aren't being met with the process based restoration features, there may be some adaptive management required. Adaptive management costs are separate from O&MRRR.
- The North Fork site will require periodic levee maintenance, such as vegetation control. If erosion develops that threatens the reliability of the new setback levee, costs have been included that allow for the replacement of 300 feet of the new setback levee.

3-2.1.17 Environmental engineering considerations

In the context of hydrology and hydraulics, environmental engineering is taken to mean water supply and sanitation. Existing water supply or sanitation systems will be decommissioned.

3-2.2 Residual Flooding Consequences – With Project Flooding

This section discusses the predicted hydraulic conditions after construction of the proposed restoration.

3-2.2.1 Warning time of impending inundation

There will be no residences or infrastructure within the site, except for the Best Road Bridge. Aside from regional warnings for possible flooding, no warning system is planned.

3-2.2.2 Rate of rise, duration, depth, and velocity of inundation

Unsteady flow analysis and flood flow routing is likely to be required for this site, however no analysis of rate of rise and flow duration is planned for flood flows. The depths and velocities at the levee removals and in the tidal channel due to the combined effects of river flow and tidal prism will be evaluated during PED.

3-2.2.3 Historic, 1% and 0.2% exceedance (100-year and 500-year) flood extents

Setting back the levees is likely to decrease peak water levels within the project area, for the estimated 1% AEP (100-year) event. Preliminary hydraulic modeling using HECRAS 1D indicates that levee setback will reduce the 1% AEP flood elevations from 1 to 2 feet. This will be reviewed during PED once accurate survey information is available and soils explorations have been performed to quantify the potential for levee settlement. The levee design will be revised as appropriate. For the purposes of quantity estimation for feasibility, because of the uncertainties in topography and soils characteristics, the assumed levee cross section has the same crest elevations as the current levee.

Since the setback levee is to be built with the same level of flood risk management as the current levee system, flood consequences for the remainder of Fir Island for the with-project condition would be the same as for the existing system.

3-2.2.4 Access and egress problems created by flooding

Best Road and Bridge cross the site. The bridge and approaches are above the 1% AEP (100-year) flood elevation. There would be no loss of access or egress during flood events due to the project. Area flooding may limit access to the bridge during floods of approximately 2% AEP or larger due to low points in the Dike District 22 levee system outside the project area.

3-2.2.5 Potential for loss of life as a result of 3-2.2.1 through 3-2.2.3

The potential for loss of life as a result of the restoration is low. Areas within the site will be inundated more often for low return interval floods. However, the entire site lies within the 100-year floodplain and is not likely to be occupied by people during floods.

3-2.2.6 Identification of any potential loss of public services

There are no identified public services within the site.

3-2.2.7 Potential physical damages

Potential physical damages that can occur during flooding will be addressed by the hydraulic analyses conducted during PED. This will include an evaluation of erosion and sedimentation in the channel adjacent to the site and impacts to the Best Road Bridge abutments.

3-2.3 Project Induced Flooding – Change from Pre-Project Conditions

This section describes the effects of the site on flood elevations, flood patterns, and flood frequency.

3-2.3.1 Information categories required by 3-2.2

Flooding at the North Fork site is dominated by fluvial discharge from the Skagit River with some influence from coastal flooding and tides. Lowering of the levees at the site is anticipated to decrease peak water levels during 100-year floods by up to 2 feet. Water levels within the site during smaller flood events will be affected by the increased tidal prism and the availability of new inflow pathways to the site. The increased flow in the site is a goal of the restoration effort.

Since the setback levee is to be built with the same level of flood risk management as the current levee system, flood consequences for the remainder of Fir Island for the with-project condition would be the same as for the existing system.

3-2.3.2 Anticipated frequency of induced flooding

The proposed work is expected to alter the pattern and frequency of flooding of the site. Areas within the site will be inundated more often for lower return interval floods, which is one of the goals of the restoration effort.

3-2.4 Inundation Risk 0.2% Exceedance (500-year Return Interval) Flood

Work at the site is not anticipated to change the frequency of 0.2% AEP flooding. The 0.2% AEP (500-year) flood elevations within the North Fork site are expected to be reduced due to the levee setback. This will be reviewed during PED once accurate survey information is available and soils explorations have been performed to quantify the potential for levee settlement. Since the setback levee is to be built with the same level of flood risk management as the current levee system, 0.2% AEP flood consequences for the remainder of Fir Island for the with-project condition would be the same as for the existing system.

3-2.5 Hydraulic Studies

This section discusses the hydraulic studies, construction considerations, and instrumentation and monitoring needs for the site. The anticipated hydraulic studies at this site are summarized in Section 3-21.

3-2.5.1 Hydraulic roughness determinations

If a hydraulic roughness determination is required to complete hydraulic analyses, then roughnesses will be determined using a combination of aerial photographs and field surveys during PED. Roughnesses will be calibrated using high water marks if available.

3-2.5.2 Water surface profiles

Current water surface profiles as reported in the Skagit River Flood Risk Management Feasibility Study, Hydrology Technical Document (USACE 2013) include the presence of the levee system surrounding Fir Island. In order to predict the with-project water surface profiles, a revised hydraulic model will have to be implemented that reflects the proposed setback levee using accurate survey information. This will be addressed during PED.

To estimate a rough value of water surface elevation change at the project site, the existing one dimensional HECRAS model was modified to remove the levees, lower the riparian berm height to 12 feet NAVD88 and include a ground elevation rise at the setback levee location. The new tidal channels were not modeled for this order-of-magnitude estimate. In most locations, the water surface elevation was 0.5 ft to 1.5 feet lower for the with-project condition as compared to the without-project condition.

The HECRAS model was also modified include two dimensional flow areas and was run in the current configuration as well as with the levees removed down to the riparian berm elevation of 12 feet. The 2-D model predicted a somewhat larger water surface elevation drop of 1.5 to 2.5 feet.

Since no current survey and channel elevations are available for this area, the above analyses were only done to confirm the expectations that water surfaces will be reduced by the setback of the levee, and not to refine the levee design. The cost estimate includes a setback levee design with top of levee elevations similar to those of the existing levee. Until accurate topography, bathymetry and soils characterizations are available, refinement of the levee design is not practical.

Stage-discharge relationships

Current stage-discharge relationships as reported in the Skagit River Flood Risk Management Feasibility Study, Hydrology Technical Document (USACE 2013) include the presence of the levee system surrounding Fir Island. In order to predict the with-project water surface profiles, a revised hydraulic model will have to be implemented that reflects the proposed setback levee using accurate survey information. This will be addressed during PED.

3-2.5.3 Head loss

Other than the head losses that will be incorporated into the revised hydraulic model, no additional head loss studies are planned.

3-2.5.4 Flow and velocity

Flow and velocity information from the revised hydraulic model will be used to assess the possibility for sedimentation, scour, and bank erosion in and around the site.

3-2.5.5 Structural sizing needed to meet design capacity including slope protection

The hydraulic analysis conducted during PED will refine the size and slope protection required for the setback levees. A 3' thick riprap and hydroseeded 2" topsoil layer are included in the current phase.

3-2.5.6 Water control facilities

The hydraulic analysis conducted during PED will include the size and slope protection required for the setback levee. A 3' thick riprap and hydroseeded 2" topsoil layer are included in the current phase. In addition, culverts required for interior drainage from Fir Island will be also be designed. Three 30" culverts with tidegates are included in the cost estimate for Feasibility phase.

3-2.5.7 Energy dissipating facilities

No energy dissipation facilities are proposed. (Not applicable.)

3-2.5.8 Erosion control requirements

Construction

Planning during PED will evaluate the best and most cost-effective methods for excavation. Earthwork will likely be accomplished using land-based equipment, but some limited earthwork with water-based equipment may be needed to excavate existing bank protection. This will be verified during PED. Barge navigation and positioning have the potential to suspend or erode bottom sediments in the river, so if this occurs, appropriate in-water sediment control measures will need to be used to minimize impacts. These may include excavating during extreme low tides, installing silt curtains, or possibly using a containment structure for work in the dry. Excavation of interior tidal channels should occur prior to breaching the outlets to minimize sediment impacts to the waterways. Temporary roadways adjacent to waterways need to be engineered to minimize sediment impacts.

With Project

No erosion control is currently anticipated outside of the construction boundaries since the goal of the project is to reestablish natural erosion and sedimentation processes. The hydraulic analysis conducted during PED will evaluate whether erosion control or slope protection is needed in areas within or adjacent to the site because of flow changes caused by the restoration.

3-2.5.9 Existing and post-project sedimentation

Although the entire Skagit River Estuary is an active accretionary environment, it is also very dynamic. The North and South Forks of the Skagit River have, in the past, carried different proportions of the sediment load and may continue to shift in their relative transport capacities. Distributary channels in the estuary may shift or avulse as part of natural sedimentation patterns. If conditions at the North Fork site remain as they are presently, the area will continue to subside from lack of new sediment inflows. The breaching and lowering of dikes and the consequent development of a distributary channel network will allow increased tidal prism and sediment inflows at the site. The work is also likely to result in increased flows to the downstream channels. The amount and potential areas of flow changes and sedimentation will be addressed during PED.

3-2.5.10 Water control and order of work during construction

Construction should be sequenced, with work on the setback levee and the interior areas first and levee lowering last. For further considerations refer to Section 3-2.5.8.

3-2.5.11 Criteria for facility/utility relocations

Buildings, roads, utilities, and other hard structures/surfaces within the setback area, including Blake's Resort, will be removed. For further details on the utility relocations see Section 3-6.3.

3-2.5.12 Other facilities to meet project goals

No other facilities are required in order to meet project goals. (Not applicable.)

3-2.5.13 Instrumentation and monitoring

A combination of field surveys and aerial photographs will be used to document biological and physical changes to the landscape. Monitoring data can be used to refine adaptive management and corrective measures, as needed. Some of the key monitoring needs and opportunities are summarized in the FR/EIS in Section 6.6.

3-2.6 Coastal Studies

Coastal base flood elevations were calculated by FEMA as a part of the Skagit County Flood Insurance Study. The base flood elevations are calculated by combining the effects astronomical tide (caused by gravitational effects of sun and moon), storm surge (rise in water levels as a result of wind stress and low atmospheric pressure), waves breaking onto the shoreline, producing an additional water level rise at the beach (wave setup) and waves running up the beach (wave runup). The 1% annual exceedance coastal base flood elevation is at Elevation 12.19 (NAVD88).

The North Fork site is located along the North Fork of the Skagit River, approximately 3 miles upstream of the delta shoreline, and is only subjected to wind waves caused by local winds. Measurements at the nearby Whidbey Naval Air Station (Figure 3-2-8) show that the maximum wind speeds come from the southeasterly direction and rarely exceed 40 miles per hour. This could result in wave heights of 4 feet with a period of 4 seconds at the river delta shoreline; however, these waves would likely be attenuated by the time they reached the site. It is unlikely that wind waves are a significant forcing mechanism at this site. This site is chiefly dominated by diurnal tidal flows with periodic flooding from the North Fork Skagit River. The influence of wind wave activity, storm surge and wave setup will be evaluated further during PED.

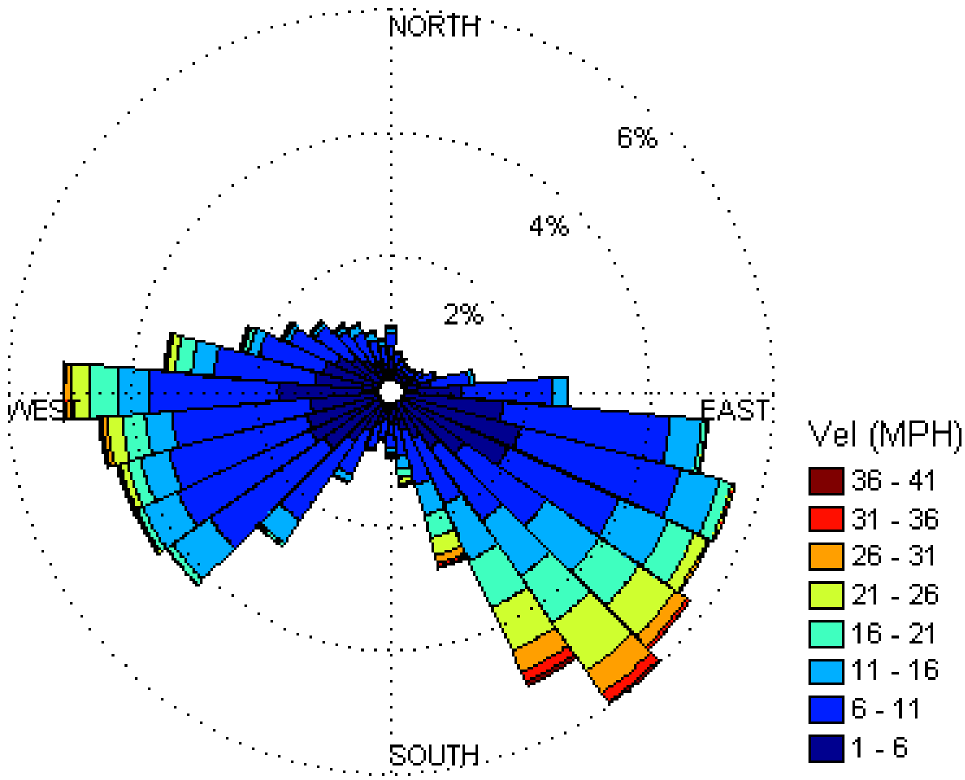

Figure 3-2-8. Wind Rose for Whidbey Naval Air Station

Plans formulated during the conceptual design phase for The North Fork site are based on a MHHW tidal datum of 8.84 feet (NAVD88). This datum is from the tide gauge at La Conner, Swinomish Slough (NOAA Gauge 9448558). Major tidal datums are summarized in Table 3-2-4. The final design tidal datums will be reviewed and established in PED.

Table 3-2-4. Major tidal datums for The North Fork site, La Conner, Swinomish Slough (Station 9448558)

Datum Description	Water Level (ft, NAVD88)
FEMA BFE (Coastal)	12.19
FEMA BFE (River)	12-21
Mean Higher-High Water (MHHW)	8.84
Mean High Water (MHW)	7.92
Mean Tide level (MTL)	4.55
Mean Sea Level (MSL)	4.45
National Geodetic Vertical Datum of 1929 (NGVD29)	3.79
Mean Diurnal Tide Level (DTL)	3.67
Mean Low Water (MLW)	1.19
North American Vertical Datum of 1988 (NAVD88)	0
Mean Lower Low Water (MLLW)	-1.51

A summary table for the anticipated coastal studies at this site is presented in Section 3-21.

3-2.6.1 Design of coastal shore protection projects (ER 1110-2-1407)

This site does not include coastal shore protection. (Not applicable.)

3-2.6.2 Effects on adjacent shores

Downstream of the site, the shoreline transitions from tidal freshwater wetlands to estuarine wetlands and finally to a river delta shoreline. The restoration is not likely to significantly alter the salinity and sedimentation patterns around the river delta. The effects on adjacent shores will be evaluated during PED.

3-2.7 Navigation Projects

The project at the North Fork site is unlikely to affect navigation channels. The southern entrance to the federally maintained Swinomish Navigation Channel is located approximately 3 miles downstream of the site. The Swinomish Channel is used by fishing and recreational boats and the channel is separated from flows from the Skagit River by a jetty that runs most of the way between Goat Island and McGlinn Island. (see Figure 3-2-9. Navigation chart for the vicinity of the North Fork Skagit River (Source: NOAA RNC Online).The project is not anticipated to have a significant impact on the navigation channel. The mouth of the North Fork Skagit River is over 7 miles east of commercial navigation channels in Puget Sound and separated from the Sound by Whidbey Island. Any potential impacts to navigation will be re-evaluated during PED.

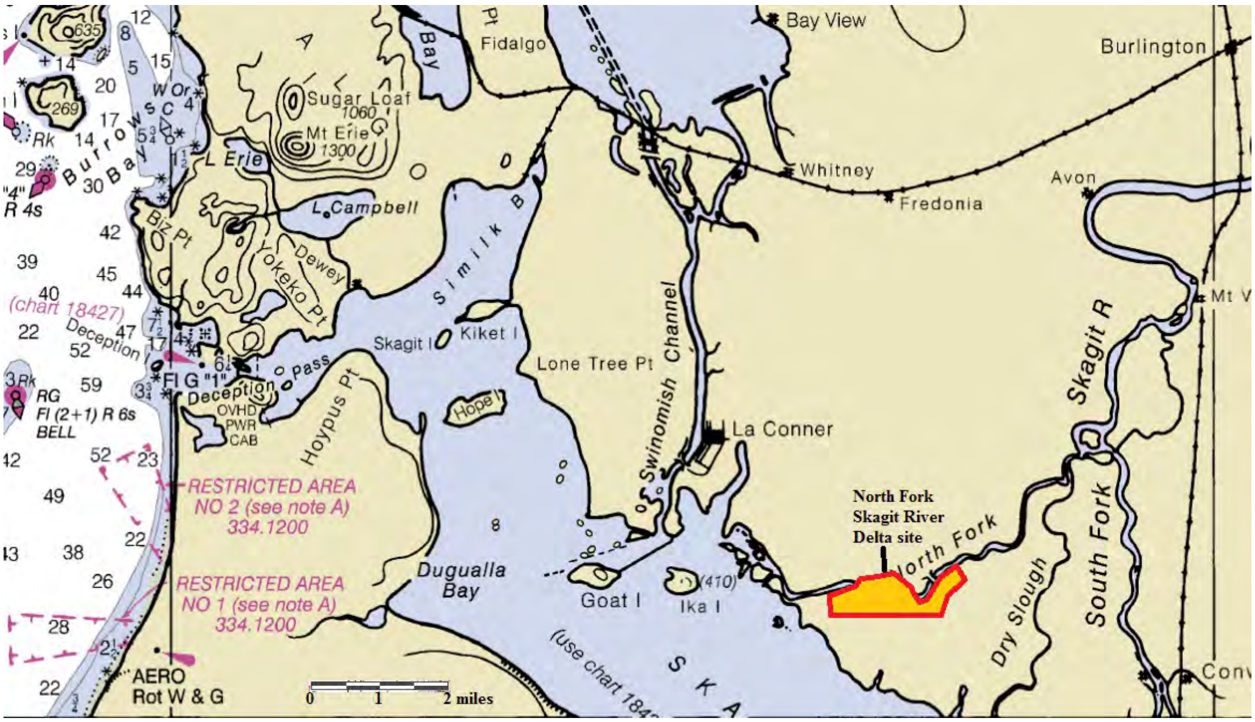

Figure 3-2-9. Navigation chart for the vicinity of the North Fork Skagit River (Source: NOAA RNC Online).

3-3 SURVEYING, MAPPING, AND OTHER GEOSPATIAL DATA REQUIREMENTS

This section describes surveying, mapping, and other geospatial data information to support preparation of the FR/EIS and the *Real Estate Plan* (Appendix C of FR/EIS). A brief outline of additional surveying and mapping required for subsequent design, plans and specifications, construction, and operations is also included.

3-3.1 Surveying, Mapping, and Other Geospatial Data Information Used

Geospatial data for the North Fork Skagit River Delta site were obtained primarily from remote sensing applications. No site-specific topographic, bathymetric, property, or utility surveys were conducted during the conceptual design phase. LiDAR, aerial imagery, and other geospatial data were used to delineate topographic features, determine surface elevations, and to estimate areas, volumes, lengths, and other dimensions of key features using CAD and/or ArcGIS. High-resolution LiDAR was obtained from the Puget Sound LiDAR Consortium (2005 LiDAR; 3m grid; State Plane projection in NAD83 [horizontal datum] and NAVD88 [vertical datum]; available at http://pugetsoundlidar.ess.washington.edu/lidardata/index.html). The Puget Sound Digital Elevation Model was used for combined bathymetry and topography of the Puget Sound lowland (Finlayson D.P., 2005; University of Washington; State Plane projection in NAD83 [horizontal datum] and NAVD88 [vertical datum]; available at http://www.ocean.washington.edu/data/pugetsound). Recent aerial photography (Aerials Express, 5/15/2009, 0.3m resolution, 2.45 m accuracy) was evaluated to determine recent site conditions. The conversion from Mean Lower Low Water (MLLW) to North American Vertical Datum (NAVD88) and to the NGVD29 datum was derived from the La Conner tide gauge (# 9448558).

Information on land ownership was derived from the Washington Public Lands Database. Additional parcel data, including parcel boundaries, were obtained from the Skagit County assessors' office (2010). Information on utilities, existing roadway geometry, and other site features was generally scaled off of aerial photographs because as-built drawings were not available. A site reconnaissance was performed in September 2010.

Designers consulted the Nearshore Geodatabase for additional site context. The Nearshore Geodatabase is available from the Washington State Geospatial Data Archive at: http://wagda.lib.washington.edu/data/geography/wa_state/#PSNERP. Metadata are provided in the *Geospatial Methodology Used in the PSNERP Comprehensive Change Analysis of Puget Sound* (Anchor QEA et al. 2009) (see Annex B). The geodatabase includes numerous datasets listed below:

- Shoreline
- Bathymetry
- Digital Elevation Model (DEM)
- LiDAR (terrestrial)
- Oblique aerial imagery (from the Washington Coastal Atlas)
- Hydrographic sheets
- Geology
- Slope stability
- Drift cells (net shore-drift)
- Streams
- Impervious surfaces
- Overwater structures
- Marinas
- Armoring
- Breakwaters/jetties
- Groins
- Levees
- Dams
- Nearshore fill
- Roads
- Railroads
- Land cover

Designers consulted the University of Washington Puget Sound River History Project 19th Century Coast Survey Topographic Sheets (2009) for information on historical geomorphic conditions. Conceptual designs were intended to replicate historical conditions and remove stressors to nearshore processes to the extent practicable and feasible. As a result, these datasets informed the selection of restoration strategies and features. Designers created additional GIS data layers (point files, line files, and polygon files) to represent civil design features, such as areas of lowland excavation, to be depicted on the plan view drawings. "Designers also created simple line drawings in CAD to represent typical sections and estimate quantity take-offs. Limited surface modeling was used to aid new levee and existing levee excavation quantity take-offs."

3-3.1.1 Additional survey and mapping required

Substantial additional information will be required during the PED stage to refine the design assumptions, confirm real estate requirements, and develop plans and specifications. Additional survey, mapping, and other geospatial data needs include the following:

- Property/Utility Survey – More detailed information on property boundaries and utilities will be needed to finalize the design and support real estate negotiations. The survey would also be useful in providing more accurate preliminary designs and quantities for excavations, utilities, and removal of existing features.

- Topographic/Bathymetric Survey – The conceptual design was based on LiDAR and aerial photos, which have inherent inaccuracies. Site-specific topographic and bathymetric survey data will be needed to refine design of key elements, confirm that target elevations are appropriate for the desired ecosystem components (low marsh, etc.), and develop detailed construction and demolition plans. Survey data could also be used as a baseline for pre- and post-construction modeling, including hydrodynamic modeling. A temporary tide gauge may be required in the early design stages to obtain site-specific tidal statistics.

3-3.1.2 Timeline for incorporation of new mapping or other geospatial data

Planning, design, and implementation are expected to take several years. The site-specific surveys identified above are standard components of the design process and should be completed in the early stages of PED to ensure that the design work proceeds efficiently. Incorporating these data into the design process is not expected to delay the restoration.

3-4 GEOTECHNICAL

This section describes the geologic setting of the site, previous and recommended studies, and proposed geotechnical explorations relevant to design features.

3-4.1 Geotechnical Information

3-4.1.1 Regional and Site Geology

Regional geologic mapping from the Washington State Department of Natural Resources (DNR) indicates the site is composed of nearshore deposits (Qn) and Skagit River alluvium (Qas) from the Holocene age (2004 Geologic Map of the Utsalady and Conway 7.5-minute Quadrangles, Skagit, Snohomish, and Island Counties, Washington). The nearshore deposits are commonly soft, gray to olive gray sand and silty sand with occasional lenses of organic material. The Skagit River alluvium consists of flood overbank deposits of grayish brown to gray sand, fine sandy silt, silt, and silty clay with minor peat. A section of the geologic map is shown in Figure 3-4-1.

The North Fork of the Skagit River was glaciated during the Vashon ice event about 15,000 years ago. Once the glacier retreated, the river built a broad alluvial plain. Icewater melt and flooding helped to deposit sediments in the forming delta and estuary. After the retreat of the glacier, the surface geology has been under the influence of both tidal action and flooding by the river.

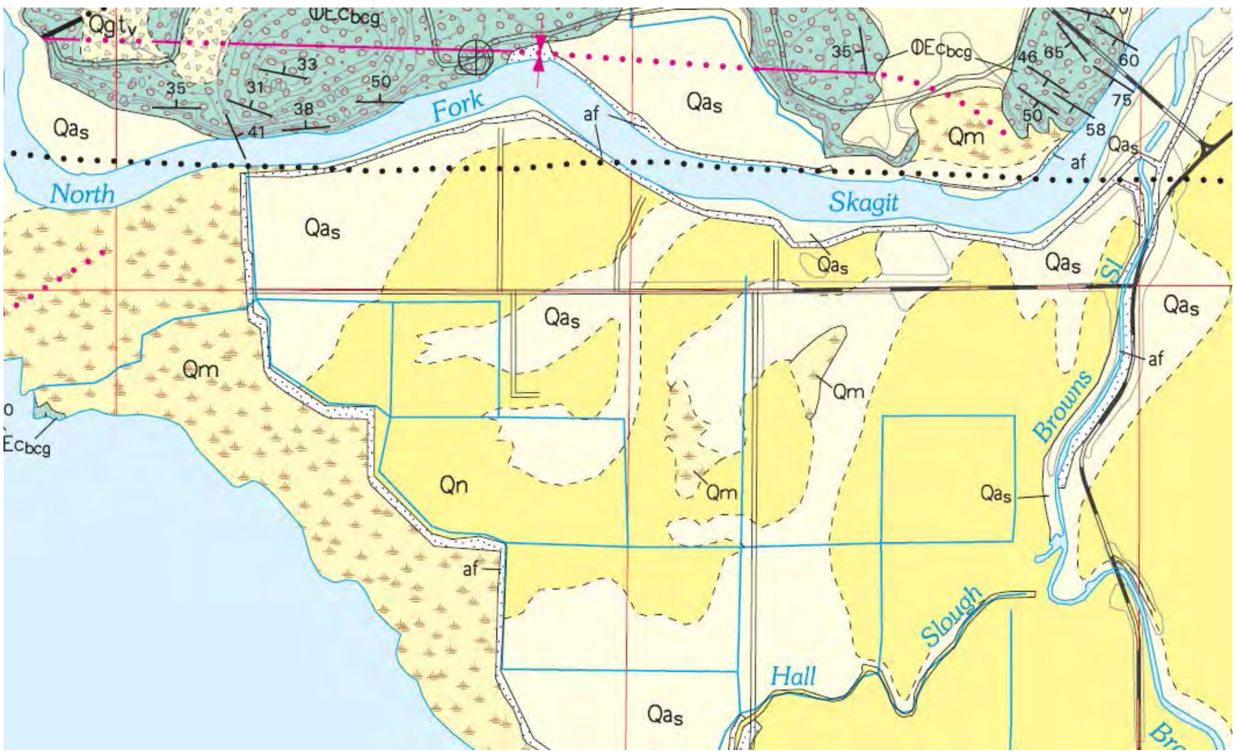

Figure 3-4-1. Geologic map of North Fork Skagit Delta.

Near-surface soils mapped in the *Soil Survey of Skagit County, Washington,* consist of Skagit silt loam, Sumas silt loam, Mt. Vernon very fine sandy loam, and Briscot fine sandy loam (NRCS 2012). Skagit silt loam is observed in floodplains and deltas and is described as silt and fine sand deriving from alluvium and volcanic ash. Sumas silt loam is observed in floodplains and deltas and is described as silt deriving from alluvium. Mt. Vernon very fine sandy loam is observed in floodplains and is described as ashy fine sand deriving from alluvium and volcanic. Briscot fine sandy loam is observed in floodplains and is described as fine sand and silt deriving from alluvium.

According to the Washington State Department of Ecology (Ecology) website, a boring was conducted nearby (400 feet to the east) in August 1994 (Ecology 2012). The boring was drilled to a depth of 42 feet with groundwater observed at 8 feet. The log recorded brown clay and silt from surface to 8 feet, gray sand with brown silt from 8 to 27 feet, and dark gray clay and silt from 27 feet to bottom of hole.

3-4.1.2 Completed explorations

At this time no subsurface explorations have been completed for this site. All subsurface information is based on soil surveys, geologic mapping, and available logs from Ecology. See Section 3-4.3 for the proposed subsurface exploration plan.

3-4.1.3 Selection of preliminary design parameters

Based upon research of soils and geology, subsurface soils on the site likely consist mostly of silts, clayey silts, and sandy silt. There are no plans to construct new structures. Therefore, no preliminary design parameters have been developed for this report. Soil parameters will need to be developed for the proposed setback levee once a suitable borrow source has been confirmed.

3-4.1.4 Geophysical investigations

No geophysical investigations have been conducted or are recommended. (Not applicable.)

3-4.1.5 Groundwater studies

No groundwater studies have been conducted. Groundwater elevation depends on local infiltration on Fir Island, flows from the Skagit River and the water surface elevation of Puget Sound. Groundwater elevations will be determined as part of the soils investigations to be conducted in PED.

3-4.1.6 Recommended instrumentation

No instrumentation is recommended. (Not applicable.)

3-4.1.7 Earthquake studies

No earthquake studies have been conducted or are recommended. There are no proposed structures or features requiring seismic design. (Not applicable.)

Earthquake loadings are not normally considered in analyzing the stability of levees because of the low risk associated with an earthquake coinciding with periods of high water. Depending on the severity of the expected earthquake, the importance of the levee, and the duration of flood event, seismic analyses to determine stability and liquefaction susceptibility may be required. This is not anticipated for this site.

3-4.1.8 Preliminary engineering analysis

The proposed setback levee should be designed in accordance with USACE Engineering Manual 1110-2-1913 Design and Construction of Levees. For levees constructed on soft subsurface conditions, stability and long-term settlement analyses are typically performed.

3-4.1.9 Excavatability analysis

Excavation will include removal of levees and reestablishment of a marsh channel network. No explorations or construction records were located, and therefore the levee embankment material is unknown. Based on soils and geology maps, the embankments likely consist of a combination of compact silt, clay, and sand. No bedrock or large boulders are anticipated; therefore, no blasting should be required. Majority of excavation will include the existing levee embankment composed of locally generated materials and native/agricultural soils within the proposed channels. Excavation of the levees and channels will likely be accomplished by excavator and/or bulldozer.

3-4.1.10 Anticipated construction techniques and limitations

The presence of the existing levee allows for construction of the setback levee year-round. Construction equipment shall be able to access the site by road. Track-mounted bulldozers, excavators, and end dumps will likely be used to construct the setback levee. Excavators and bulldozers will be used to lower the existing levee. Excavation of the breaches and levee lowering should be scheduled to coincide with periods of low water.

See Section 3-6 for additional construction notes.

3-4.1.11 Potential borrow sources and disposal sites

Material excavated to lower the existing levee and to excavate channels is unlikely to be suitable fill for construction of the proposed setback levee. Disposal soils from the existing levee will likely be used to create low levees to support riparian corridors or stability berms and transition slopes for the proposed levee. Offsite disposal and borrow sites are available within a reasonable distance from the site. Multiple borrow and disposal sites are located along the Interstate 5 corridor within 30 miles of the site. Project plans include a 30 mile haul distance for armor excavation, provision of fill material, a 20 mile haul distance for pavements and sidecast of levee and channel excavations. Borrow and disposal sites shall be confirmed during PED. The uncertainties associated with confirming suitable borrow and disposal sites have been captured in the cost risk register.

3-4.1.12 Potential sources of concrete and materials

The procurement of concrete or materials is not anticipated. (Not applicable.)

3-4.1.13 Suitability of concrete and materials

If concrete and additional materials are required, their suitability will be evaluated at later stages of design or during construction.

3-4.2 Additional Studies and Analysis

Additional studies and analysis to be completed during PED or subsequent phases of design include the following at a minimum:

- Geotechnical investigation including subsurface explorations, testing, and field reconnaissance
- Levee Design: stability, settlement, and seepage analysis

3-4.3 Additional Explorations and Testing

The proposed subsurface exploration plan consists of test pits or site probing along the existing levee embankments, proposed levee alignment, and along the channel excavation areas. Explorations for the proposed levee should be conducted in accordance with Engineering Manual 1110-2-1913. This will include a combination of test pits and borings along the levee alignment. Depth of borings and test pits for the levee should be a minimum of 10 feet and spaced approximately every 200 feet. Site probing in excavation areas will likely be less than 5 feet below the ground surface. Test pits will be accomplished with a backhoe or small excavator, and site probing uses manually operated small augers. The recommended boring method is mud rotary.

Sampling in the soil borings will be accomplished using standard penetration test (SPT) with samples taken typically every 2.5 feet for the top 25 feet and every 5 feet for the rest of the boring depth. Proposed soil lab testing will include moisture content, grain size analysis, and percent finer than #200 sieve. Atterberg limits and consolidation tests are recommended for cohesive soils, and unconfined compressive strength test for rock cores.

The subsurface exploration plan will be reevaluated and coordinated with hazardous and toxic material investigations during PED to include chemical sampling and testing (See Section 3-9).

3-4.4 Laboratory-testing Program and Evaluations

No laboratory testing or evaluation of materials has been completed at this time. Testing to be completed during PED is outlined in Section 3-4.2.

3-5 ENVIRONMENTAL ENGINEERING

This section describes environmental engineering factors relevant to the proposed design features.

3-5.1 Use of Environmentally Renewable Materials

At this design stage, use of environmentally renewable materials is not planned. However, if renewable materials are available they could be incorporated into the design. Specific details will be developed during subsequent design stages.

3-5.2 Design of Positive Environmental Attributes into the Project

The North Fork River Delta site was selected to address River Delta restoration objectives to protect and restore freshwater input and tidal processes where major river floodplains meet marine waters. The proposed action involves the lowering the existing levee and selectively breaching to allow inundation of the estuarine emergent marsh and sustain back channel habitat. Forested floodplain habitat will be created along the lowered dike adjacent to the mainstem river channel.

3-5.3 Inclusion of Environmentally Beneficial Operations and Management for the Project

Design and construction will incorporate sustainable and ISO 14000 compliant practices. The U.S. Army Corps of Engineers (USACE) Environmental Operating Principles (EOPs) are designed to provide direction on achieving better stewardship of air, water, and land resources while showing the connection between managing those resources and protecting environmental health. The EOPs are to ensure that USACE actions consider the environment and are sustainable now and in the future.

3-5.4 Beneficial Uses of Spoil or Other Project Refuse during Construction and Operation

Beneficial uses of spoil or other refuse are possible. Phase 2 emphasizes the use of material generated by levee lowering and channel excavation to create low berms in the forested wetland elevation range to increase the survival of the riparian forest in the subsided areas. Excavated material from all interior channels would be sidecast adjacent to the channels to create low discontinuous berms at elevations suitable to support a riparian woodland corridor. If spoils or other refuse materials are available for reuse, they could be incorporated into the design. Specific details will be developed during subsequent design stages.

3-5.5 Energy Savings Features of the Design

At this design stage, energy savings features have not been incorporated. In accordance with the EOPs, energy savings features will be a component of the design to the maximum extent practicable. The FR/EIS describes measures to minimize energy consumption for the purpose of reducing greenhouse gas emissions. These measures would also serve to minimize energy consumption.

3-5.6 Maintenance of the Ecological Continuity in the Project with the Surrounding Area and Within the Region

The restoration will increase ecological continuity within the site and with the surrounding area. This is one of several sites designed to restore the productivity and increase interconnectivity of the Puget Sound ecosystem.

3-5.7 Consideration of Indirect Environmental Costs and Benefits

All direct, indirect, and cumulative environmental costs and benefits were evaluated during the environmental impact assessment and alternatives analysis recorded in the Final FR/EIS.

3-5.8 Integration of Environmental Sensitivity into All Aspects of the Project

Construction will be conducted to ensure no long-term deleterious impacts to the ecosystem will occur. Best management practices will be incorporated into the contract documents. Best management practices will cover erosion and sediment control, stormwater management, spill response and hazardous material management, trash and debris management, air emissions from construction vehicles, and noise standards.

3-5.9 Use of Environmental Review Guide for Operations (ERGO) with Respect to Potential Future Environmental Problems

This is not a USACE operating facility. (Not applicable.)

3-5.10 Incorporation of Environmental Compliance Measures into the Project Design

All applicable laws and regulations will be followed during design and construction in accordance with the USACE contract documentation.

3-6 CIVIL DESIGN

This section discusses the key elements of the civil design including the selection of the site and evaluation of alternative layouts, alignments, and components.

3-6.1 Site Selection and Project Development

Extensive diking of the North Fork Skagit River has caused substantial loss of estuarine connectivity. The proposed restoration would set back flood risk management levees on both sides of the North Fork, from the former inlet of Dry Slough to the western terminus of the levee system near Rawlins Road. The action seeks to restore natural levees and create additional emergent marsh and riverine wetlands.

A brief description of the project is included in the Skagit Chinook Recovery Plan (SRSC and WDFW 2005). The plan lists this action as a project with a "long-term restoration horizon," meaning that it is generally less well developed and has uncertainties that must be addressed before implementation. The same plan includes a number of other setback projects proposed along the North Fork at Thein Farm, Rawlins Road Levee, and Blake's Bottleneck. A feasibility study of the Rawlins Road project has been conducted (Yang and Khangaonkar 2006). Given their geographical proximity, there is potential synergy between the North Fork levee setback and these other projects. The full restoration alternative presented here is a combination of the North Fork at Thein Farm, Rawlins Road Levee, and Blake's Bottleneck projects.

A restoration alternative that proposed less extensive work was considered but was not selected during cost effective analysis. The alternative selection process is documented in Chapters 4 and 5 of the FR/EIS.

Table 3-6-1 summarizes the key design elements associated with the proposed restoration. Annex 3-1 contains exhibits that depict the proposed restoration design elements.

Table 3-6-1. North Fork Skagit River Delta Key Design Elements.

Item	Description of Item	Quantity
Lower Levees and Build Riparian Berm	Excavate lowlands to lower 15,691 lf of existing levee to elevations similar to natural levees (13.5 ft MLLW, 12 ft NAVD 88 on the inboard side of the site, sloping down to 10.5 ft MLLW, 9.0 ft NAVD 88 on the main channel bank); Excavated material to be placed landward of existing levee to create 15,130 lf of floodplain berm approximately 100 to 150 ft wide (width to be determined by amount of material available) with the exception of 10,260 cy that will be used to block existing distributary channel west of Brown's Slough Road (described below)	211,820 cy
Excavate Breaches in Lowered Levee	Excavate lowlands to breach lowered levee in 4 locations. Breaches will be constructed to dimensions of 5th order channel; assume 50-foot wide benches at 7 ft NAVD88 (8.5 ft MLLW) installed on either side of the breach. At the end of the 50 ft bench, 10H:1V slopes extend up to between 9 .0 and 11.0 ft. This section results in an excavation of 45 cy/LF (at 9.0 ft NAVD88 top elevation) to 79 cy/lf (at 11.0 ft NAVD88 top elevation)	29,720 cy
Excavate Tidal Channel Network	Excavate 19,617 lf of tidal channels and sidecast generated material adjacent to the channels to create low berms that will support a riparian corridor. Excavation includes: 2,349 lf of second-order channel; assume 3 ft bottom width at elevation 2.0 ft, 3H:1V sideslopes, and average surface elevation of 6.5 ft 6,953 lf of third-order channel; assume 3 ft bottom width at elevation 0.0 ft, 3H:1V sideslopes, and average surface elevation of 6.5 ft 7,702 lf of fourth-order channel; assume 3 ft bottom width at elevation -2.0 ft, 3H:1V sideslopes, and average surface elevation of 6.5 ft 2,613 lf of fifth-order channel; assume 3 ft bottom width at elevation -5.0 ft, 5H:1V sideslopes, and average surface elevation of 6.5 ft	179,315 cy
Block Distributary Channel	Place excavated material from levee lowering to block distributary channel located 2,650 ft downstream of Best Road bridge; assume 158,240 sf area with an average depth of 1.75 ft	10,260 cy
Remove Shore Armor	Remove 16,140 lf of riprap armoring from existing levee (13,000 lf along south bank, 3,140 lf along north bank); assume entire length of levee is riprap composed of average of 5 ft height and 3 ft wide, with a density of 1.5 ton/cy	13,400 tons
Remove Buildings	Remove 17 buildings distributed throughout the project area including Blake's Resort and along Rawlins Road within the proposed levee lowering footprint; approximate area calculated from GIS	45,024 sf

Item	Description of Item	Quantity
Remove Pavement and Boat Ramp at Blake's Resort	Remove pavement at Blake's Resort; approximate area calculated from GIS	139,906 sf
	Remove boat ramp; assume 100 ft x 300 ft	30,000 sf
Remove Roads	Remove pavement from roads in newly setback area between lowered levee and new flood risk management levee; approximate area calculated from GIS	104,353 sf
Build New Levee	Construct new flood risk management levee beside Rawlins Road, Brown's Slough Road-Fir Island Rd, and Moore Road. Assume 12,317 lf. 25.6 cy/lf assumes levee crest from 15 to 22 feet NAVD 88, 15 ft crest width, 3H:1V sideslopes,1 foot overbuild for settlement and 1 foot of unsuitable soils. Levee construction includes the installation of three 100ft long interior drainage culverts with tide gates and 12,317 lf of riprap (69,920) covered with 2 feet of topsoil and hydroseeded on the riverward side (43,300 cy)	314,800 cy
Plant Vegetation	Plant riparian vegetation along slopes of lowered natural levee and sidecast berms and along realigned levee at Rawlins Road, Brown's Slough Road, and Moore Road. Assumes 85,238 sq. yds channel plantings, 66,416 sq. yds levee hydroseed, and 90,222 sq. yds sidecast plantings.	50 acres (approx.)

3-6.1.1 Basis of Design

The proposed action will restore the riverine floodplain and tidal connectivity along the lower reach of the North Fork of the Skagit River. This will require constructing a new flood risk management levee further inland. The existing levee would be lowered and selectively breached to allow inundation of the estuarine emergent marsh and sustain back channel habitat. Forested floodplain habitat would be created along the lowered levee adjacent to the mainstem river channel.

The primary stressors are armored levees preventing deltaic estuarine processes from occurring. Hydraulic processes related to frequency and depth of inundation are eliminated. Similarly, geomorphic processes such as sedimentation, channel avulsion and channel migration are prevented. This, combined with agricultural practices, has resulted in significant subsidence (2 to 4 feet) of the former emergent marsh/scrub- shrub habitat. Breaching and lowering of the levees to suitable elevations is intended to restore combined tidal/freshwater (low salinity) hydrology to support channel formation, emergent marsh, forested floodplains and scrub-shrub wetland community development. Specific process-based restoration objectives to be achieved with this action include: (1) tidal channel formation and maintenance; (2) tidal flow; (3) distributary channel migration; (4) erosion and accretion of sediments; and (5) exchange of aquatic organisms.

The action would create a continuous floodplain corridor along the length of the south bank of the North Fork, and an area of floodplain along the north bank of the North Fork (Exhibit A) by setting back the flood risk management levee. The project area includes the existing site footprint, the adjacent Rawlins Road setback project area, and Blake's Resort – a total project area of 310 acres. These expanded floodplains would increase flood capacity along the North Fork, and potentially lower flood levels in the project vicinity and to some extent upstream of the project site.

Inundation of Fir Island would continue to be prevented by replacement of flood risk management levees constructed along the southern edge of the site on the north side of Rawlins Road. The crest elevation of the new levees will be 21.5 feet MLLW (20 feet NAVD88 based on the La Conner tide gage), the same crest elevation as the levee it replaces. On the north side of the river, no new flood risk management levees are required as the setback area grades into rising land.

The existing levees would be lowered to elevations similar to that of the natural levees (about 13.5 feet MLLW, 12 feet NAVD88), which are formed during flood events and exist further downstream. This would restore the natural overtopping processes that occur during floods. Buildings, roads, utilities, and other hard structures/surfaces within the setback area, including Blake's Resort, will be removed. The material excavated during the lowering of the crest of the levee would be placed on the landward side of the existing levee to create a forested floodplain berm 100 to 150 feet wide. The width of the berm will be determined by the amount of material available. The berm will be constructed up to a maximum elevation of about 13.5 feet above MLLW (12 feet NAVD88).

Breaches through the lowered levees would allow unimpeded tidal inundation of the estuarine emergent marsh in the setback area. These breaches would be about 120 feet wide, with an additional 100 feet of the adjacent levee on either side lowered to 8.5 feet MLLW (7 feet NAVD88) to provide additional capacity at higher water levels. Channels running parallel to the mainstem river channel would drain the setback area through the breaches and create back channel habitat. Such habitat may take several decades to evolve unassisted. To accelerate their evolution, these breaches and channels will be excavated to equilibrium dimensions as described in the *Applied Geomorphology Guidelines* (Attachment B). Excavating the channels will also reduce the possibility of channel migration and the erosion of the flood risk management levee. Channel top widths would vary between 30 and 100 feet, with depths between 4 and 12 feet below existing grade. Approximately 20,000 lf of channel would be excavated. Material generated by channel excavation will be sidecast to increase heterogeneity of the setback area, help establish forested floodplain, and reduce handling and hauling costs.

During PED phase the PDT will examine trade-offs and engineer the recommended alternative to minimize costs while optimizing desired benefits. This will be accomplished once the site investigations and surveys recommended for PED have been completed.

Since 1860, land development on the delta has removed a large proportion of the estuary from the landscape, fundamentally altering the geomorphic processes that form and sustain delta ecosystems. The diking of distributary channels has had a significant impact on estuarine wetlands and tidal channels in the delta (Collins 1998). The 1886 topographic sheet (T-sheet) already showed extensive diking on the North Fork and on Fir Island. Post-restoration site conditions are intended to resemble or replicate the historical morphology to the extent feasible.

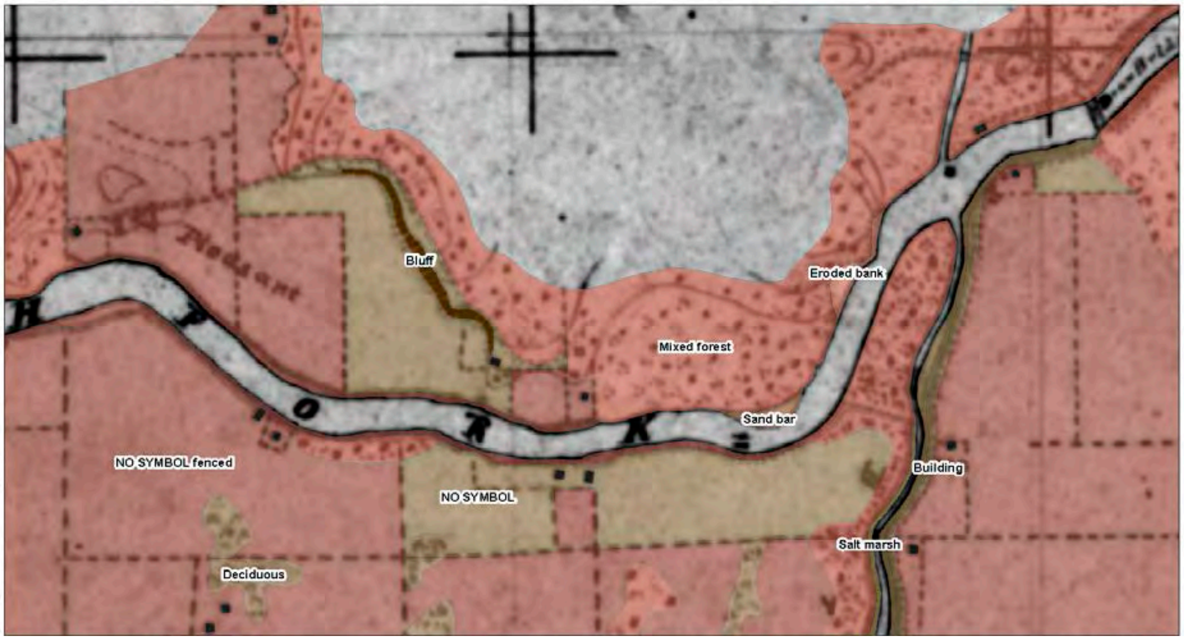

Figure 3-6-1. Historic map (T-Sheet) and river history project data.

3-6.1.2 Constructability

The present leveed nature of the site would allow for construction of the tidal channel network and the new levee year-round. The new levee would be constructed first with imported material. The new flood risk management levee and upper portions of the tidal channel network could be constructed primarily with upland equipment, including scrapers and end dumps. Excavators may be needed to create portions of the tidal channel network due to high groundwater levels.

Following construction of the new flood risk management levee, the existing levee adjacent to the North Fork may be lowered and widened primarily with upland equipment, provided this work occurs during the dry season. Breaches would require work with excavators.

Final levee lowering and breaching should be coordinated, including a plan for access as tidal waters enter the site.

It is assumed that construction access will likely be brought onsite by land, but further evaluation will be needed in succeeding stages of design. See Section 3-10 for additional information on construction procedures and Section 3-20 for the anticipated schedule for construction.

3-6.2 Real Estate

Real estate assumptions, valuations, and planning documents have been appropriately scaled for the current level of design. As additional surveys, modeling, and design are completed during PED, the real estate documentation will be modified accordingly. For the current real estate status, refer to the Final FR/EIS, Appendix C, *Real Estate Plan*.

3-6.3 Removals and Relocations

This action would require acquisition of several privately owned properties. Numerous buildings located along Rawlins Road would need to be removed. Blake's Resort, a small private RV park and boat launch would need to be removed. A cultural resource survey will be done in PED to determine if any of the standing structures to be removed are over 50 years of age and are eligible to the National Register or if there are national register eligible archaeological sites within the restoration area. Potentially, sanitary sewer or septic related to the existing buildings may require removal. Associated with the restoration is the removal of buried utilities and overhead power within the site boundaries. The cost estimate has been developed assuming that only utility removals will be required. In the unlikely case that it is necessary to reconnect or relocate utilities to properties outside the project boundary, this cost has been included in the cost risk register. Cultural resources costs related to the mitigation of National Register eligible historic properties are included in the overall project cost estimate.

Table 3-6-2. Facility / Utility Removals

Facility / Utility	Activity	Subsequent Design
Overhead power distribution and transmission lines	Determine locations and assess removal or relocation if applicable	Coordinate with utility owner on phasing of work.
Fiber Optic	Determine locations and assess removal or relocation if applicable	Coordinate with utility owner on phasing of work.
Gas lines	Determine locations and assess removal or relocation if applicable	Coordinate with utility owner on phasing of work.
Sanitary sewer septic systems	Determine locations and assess removal or relocation if applicable	Need for decommissioning analyzed during PED.
Water wells	Determine locations and assess protection or removal if applicable.	Need for decommissioning analyzed during PED.

3-7 STRUCTURAL REQUIREMENTS

This section discusses the structural elements of the proposed restoration including preliminary design requirements and criteria for bridges or roads, a description of major structures and construction considerations, and recommended analyses.

3-7.1 Functional Design Requirements and Technical Design Criteria

No new bridges or structures are planned as part of this restoration. (Not applicable.)

3-7.2 Survey, Hydrologic, Hydraulic, and Geotechnical Data Used

No new bridges or structures are planned as part of this restoration. (Not applicable.)

3-7.3 Site Selection Studies

The site selection is summarized in Section 3-6.

3-7.4 Major Structures

No new bridges or structures are planned as part of this restoration. (Not applicable.)

3-7.5 Describe Evaluation and Selection of Substructure Alternatives Based On Economy and Performance

No bridges or structures are planned as part of this restoration. (Not applicable.)

3-7.6 Construction Considerations

No new bridges or structures are planned as part of this restoration. (Not applicable.)

3-7.7 Stability Analyses

No bridges or structures are planned as part of this restoration. (Not applicable.)

3-7.8 Stress Analyses

No bridges or structures are planned as part of this restoration. (Not applicable.)

3-7.9 Thermal Stress Analyses

No bridges or structures are planned as part of this restoration. (Not applicable.)

3-7.10 Other Analyses

Not applicable.

3-7.11 Additional Studies, Tests, Analyses

Additional investigation and studies may be needed for other site requirements unrelated to the infrastructure. See Section 3-21 for a complete list of recommended additional studies and investigations.

3-8 ELECTRICAL AND MECHANICAL REQUIREMENTS

Electrical and mechanical structure requirements are not applicable to this site.

3-9 HAZARDOUS AND TOXIC MATERIALS

A Phase 1 Environmental Site Assessment was conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments,* and ER 1165-2-132: *HTRW Guidance for Civil Works Projects*. The Phase 1 Environmental Site Assessment report is attached in Annex 3-1.

The assessment revealed no evidence of recognized environmental conditions in connection with the proposed project footprint, nor any conditions at neighboring sites which have the potential to affect work at the North Fork site.

3-10 CONSTRUCTION PROCEDURES AND WATER CONTROL PLAN

The proposed restoration will involve earthwork and exposure of bare ground during lowering and breaching of levees, and excavation of channels. At this stage of design, it is assumed that standard best management practices will be implemented to control erosion and sedimentation and ensure construction areas are stabilized as needed to prevent adverse impacts. A standard temporary erosion and sediment control plan will be developed during PED.

The proposed restoration will not require in-water work during channel creation. For channel creation, work can be sequenced to avoid in-water work. Channel excavation will take place prior to breaching of the levees to reduce the likelihood of releasing sediments into downstream waters. Standard soil cover and stabilization practices will be implemented to stabilize the channels prior to introduction of water.

Specific measures for construction procedures and water control will vary depending on the location and nature of the work associated with each site. State and Federal resource agencies will impose specific timing restrictions on in-water work to protect fish and wildlife, and the Corps will adhere to conservation measures detailed in environmental compliance documents. In addition, specific measures may be required to protect downstream infrastructure or built environments. The erosion and water quality control plan will also need to consider and incorporate the findings of future analyses for hazardous and toxic materials at the site (as described in Section 3-9). A complete description of best management practices will be determined during PED.

3-11 INITIAL RESERVOIR FILLING AND SURVEILLANCE PLAN

The proposal is for ecosystem restoration. (Not applicable.)

3-12 FLOOD EMERGENCY PLANS FOR AREAS DOWNSTREAM OF CORPS DAMS

The proposal is for ecosystem restoration. (Not applicable.)

3-13 ENVIRONMENTAL OBJECTIVE AND REQUIREMENTS

Feasibility-level information to develop designs, plans, and specifications, and to execute construction and operations is included in the Project's supporting documents including the U.S. Fish and Wildlife Service report titled "Strategic Restoration Conceptual Design - Preliminary Environmental Contaminant, Cultural Resource, and Endangered Species Site Evaluations." The environmental information developed for the analysis in the FR/EIS provides additional environmental objectives and requirements for final site design development. As summarized in Section 3-6, Civil Design, substantial environmental information was developed for the FR/EIS regarding environmental problems, opportunities, and constraints such that the Corps could estimate costs of the restoration sites and prepare the Real Estate Plan. The Corps will adhere to requirements stated in the Endangered Species Act consultation documents, Clean Water Act Section 401 certification, and other site-specific environmental compliance documents. The Corps has prepared a Programmatic Agreement (PA) for Section 106 of the National Historic Preservation Act compliance. As outlined in the PA, cultural resource investigations are necessary in the PED phase to determine if National Register eligible historic properties are located in the restoration project area prior to construction. The Monitoring and Adaptive Management Plan will be used to determine whether the site is meeting environmental objectives after construction.

3-14 RESERVOIR CLEARING

The proposal is for ecosystem restoration. (Not applicable.)

3-15 OPERATION, MAINTENANCE, REPAIR, REPLACEMENT AND REHABILITATION (OMRR&R)

OMRR&R costs for the North Fork Skagit River Delta restoration are related to the following:

- Levee maintenance and repair including allowance for repair of up to 300 feet of levee once during a 50 year period.
- Vegetation costs such as removing invasive plants as well as site watering for plant establishment over the first 2-3 years prior to the start of adaptive management.

- Yearly culvert inspection and maintenance such as removal of debris and sediment.

Annual OMRR&R is estimated at $36,000 for the 50-year project period. Additional assessment of O&MRR&R activities will be conducted during PED.

The proposed setback levees will be designed in accordance with applicable USACE engineering manuals. Operation and maintenance of these structures will be necessary to ensure proper functioning of the structures and will be the responsibility of the local sponsor as detailed in the applicable O&M manual. Maintenance zones should extend 15 feet from both the riverward and landward setback levee toe. This maintenance zone should remain free of unwanted vegetation and unauthorized encroachments. Sod cover and riprap maintenance will be necessary to ensure proper functioning of erosion protection.

At the completion of the restoration project, an operation manual detailing proper maintenance practices for the setback levees will be provided to the local sponsor. Since the levees would be congressionally authorized and federally constructed, they will be ICW (Inspection of Completed Works) projects and automatically enrolled (eligible) in the PL 84-99 rehabilitation program.

3-15.1 33 CFR Part 208 Projects

The site is not a flood control project to be maintained and operated according to regulations in 33 CFR 208. (Not applicable.)

3-15.2 Channel or Basin Clean Out Projects

The restoration does not include channel or basin cleanout activities. (Not applicable.)

3-15.3 Multiple-Purpose, Complex Projects with Power Production

No power production is proposed. (Not applicable.)

3-15.4 Frequency and Cost of Maintenance Dredging

No maintenance dredging is proposed. (Not applicable.)

3-16 ACCESS ROADS

Temporary construction access roads will be needed to maximize the efficiency of earthwork operations and haul unsuitable materials offsite. It is assumed that construction access will likely be brought onsite by land. Further evaluation will be conducted in succeeding stages of design.

3-17 CORROSION MITIGATION

No new corrodible construction is proposed. (Not applicable.)

3-18 PROJECT SECURITY

The proposal is for ecosystem restoration. (Not applicable.)

3-19 COST ESTIMATES

The North Fork construction cost estimate of $99,299,000 (March 2016 dollars) consists of costs for removal of levee, shoreline armor, excavation of channels and construction of a setback levee along the

North Fork of the Skagit River. Other work includes levee breaching, demolition of existing structures, and plantings.

The largest cost driver is relocations including the setback levee ($58,987,000 construction cost, March 2016 dollars). Other substantial cost drivers include channel construction, the riparian planting along all the lowered dikes, and the demolition of existing structures.

Following a formal cost and schedule risks analysis, a project contingency of 36% was developed. The largest cost risk was the potential for construction modifications due to unexpected site issues. Also, there is a likelihood that newly built levees will settle and will require additional material to maintain the required prism. Schedule risks are entirely controlled by the potential for work to need to stop during the rainy seasons. Additional mobilizations and lost time would be incurred because of this.

There are non-cost related risks as well. There could be either erosion or sedimentation in excavated channels during river flooding. Channels will need to be watched following construction as part of the overall monitoring plan.

Opportunities to reduce the project cost include a potential for reductions in material prices. During PED, the team may design less expansive plantings or a more strategic planting scheme to reduce costs without sacrificing restoration success.

3-20 SCHEDULE FOR DESIGN AND CONSTRUCTION

The proposed restoration at the North Fork Skagit River Delta is considered to be relatively straightforward. Based on the low level of complexity, the anticipated design period for the site is approximately two years. This includes preparation of final design, plans and specifications, and the construction contract.

The anticipated construction period for Construction of the setback levee, removal of the river side levees and bank armoring levees, channel excavation, grading and construction of breaches is 2 years. Any in-water construction activities will take place during established work windows. Setback levee construction would occur first to maintain the integrity of the Dike District 22 levee system. Armor removal and lowering of the existing levee adjacent to the river may be accomplished primarily with upland equipment, provided this work occurs during the dry season. The present leveed nature of the site would allow for construction of the channel network within the site year-round. Construction would have to be sequenced with setback levee and interior marsh work first, breaches and levee lowering last.

Property acquisition and environmental compliance timelines are not included in this duration. The time required to complete these upfront activities is unknown, but is assumed to be relative to the length of the anticipated design period for the site as described above.

3-21 STUDIES TO BE COMPLETED IN PED

Table 3-21-1 summarizes recommended studies and additional investigations to be conducted at the site to support subsequent stages of design and implementation. Unless otherwise noted, these studies are recommended to take place during PED. In the table, studies are classified according to the following purposes:

- Data required for design, cost estimation or project compliance,
- design analysis to minimize project construction costs,
- design analysis to optimize environmental benefits,
- identification of induced flooding,
- and identification of actions needed for O&M.

Table 3-21-1. Studies Recommended for the North Fork Skagit River Delta Site

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Property Investigation/Survey	• Compile more detailed information on parcel ownership and property boundaries to finalize the design, confirm acquisition requirements, and support negotiations with property owners.	X	X			
Topographic/ Planimetric/ Bathymetric Survey	• Acquire site-specific topographic and bathymetric survey data to refine design of key project elements, develop detailed construction and demolition plans, and serve as a baseline for pre- and post-construction modeling, including hydrodynamic modeling.	X				
	• Install a temporary tide gage in the early design stages to obtain site-specific tidal statistics.	X				
Geomorphic Analysis	• Complete a geomorphic analysis of channel migration potential to consider channel response to dike setback and increased tidal influence and fine tune channels and breaches.		X	X		X
Hydraulic Analysis/Modeling	• Implement a revised hydraulic model for the Skagit River reflecting the proposed geometry and the levee setback to predict the with-project water surface profiles and confirm the extent and nature of hydraulic effects from the project.		X	X	X	X
	• Combine review of aerial photographs with field surveys to quantify channel topology and hydraulic roughness and inform geomorphic evaluation under restored conditions.	X				
	• Assess hydraulics at setback levee and effects of increased tidal prism to optimize levee design and quantify effects on adjacent shores.		X	X		

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Hydraulic Analysis/Modeling (cont.)	• Evaluate changes in salinity and flow patterns within, adjacent to and downstream of the site, if required.			X		X
	• Review the potential effects on water wells, septic systems, and groundwater seepage in the area of potential hydraulic effect.		X			
	• Formulate the detailed monitoring plan, including any required field surveys or instrumentation that will be used to evaluate the project's hydraulic performance.			X		X
Sedimentation Analysis/Modeling	• Analyze potential channel infilling and evolution of interior channels to refine channels dimensions and determine long-term stability of the site. This will likely require a sediment model.		X	X	X	X
	• Design erosion control and slope protection for the setback levee. Assess whether erosion control or slope protection is needed in or adjacent to the site because of flow changes caused by the restoration.		X			
	• Evaluate temporary increases in sedimentation upstream and downstream of the site during the formation of any new distributary channels.		X			
Coastal Engineering Studies	• Refine sea level projections using localized tide gauge data.	X				
	• Review and establish the final design tidal datums		X			
	• Conduct wind direction and wave run-up analysis.		X	X	X	
Geotechnical Investigation	• Complete a standard investigation to include subsurface explorations, testing, and field reconnaissance.	X	X			
	• Confirm borrow and disposal sites.	X				
	• Analyze levee design stability, settlement, and seepage.		X		X	X
Excavated Materials Study	• Evaluate the suitability of excavated materials for reuse.	X	X			
Structural Engineering	• Structural analysis of the bridge, bridge piers, and foundation.		X			

Type	Basic Requirements	Purpose				
		Required Data	Design/Costs	Design/Benefits	Inundation	O&M
Structural Engineering (cont.)	• Analysis for gravity, wind and seismic effects.		X			
	• Design of bridge deck and supporting structure for gravity, wind and seismic effects in accordance with criteria established in this report.		X			
Utility Survey	• Compile more detailed information on utilities to finalize the design and confirm acquisition requirements.	X	X			
Cultural Resources Investigation	• Complete surveys for archaeological and historic resources, particularly in areas proposed for excavation.	X	X	X		
Wetlands Investigation	• Document the location, extent, and character of wetlands.	X		X		
Cost Study	• Assess potential for cost and schedule reductions during refinement of restoration design.		X			
Environmental Compliance	• The Corps will coordinate with all relevant natural resource agencies during PED. Results of PED-phase studies will be provided to agencies and tribes as appropriate.			X		

3-22 DATA MANAGEMENT

Project documents, background materials, and digital files from the local sponsors were provided to the project team directly, through the State's Habitat Work Schedule, or via the Nearshore Sharepoint. The project team also used databases previously developed by and for the Puget Sound Nearshore Ecosystem Restoration Project including the Change Analysis and backing geospatial data (see Section 3-3.1.1 for additional detail).

Work products for the conceptual restoration designs were developed primarily in GIS and typical word processor and spreadsheet applications. GIS products for all action areas were collected in a single geodatabase that captured spatially referenced locations and sizes of major design elements.

3-23 USE OF METRIC SYSTEM MEASUREMENTS

This report uses United States customary units for design and construction measurements. To remain consistent with work conducted to date, the metric system of measurement was not used.

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ANNEX 3-1: EXHIBITS

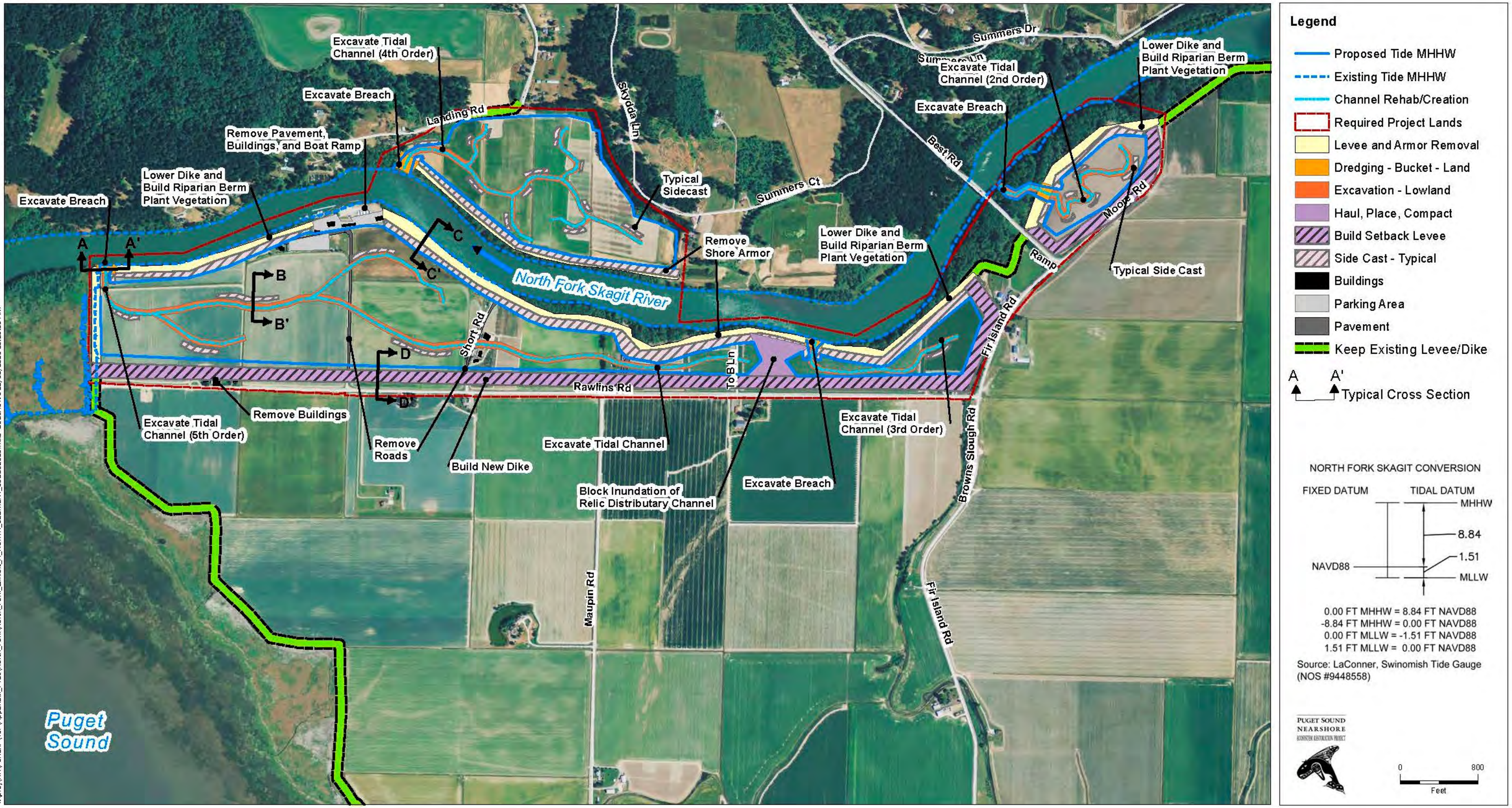
This annex contains a set of site-specific exhibits prepared for the proposed restoration. The exhibits include:

Exhibit A – Design Plan

Exhibit B – Design Sections

Exhibit C - Phase 1 Environmental Site Assessment

Exhibit A

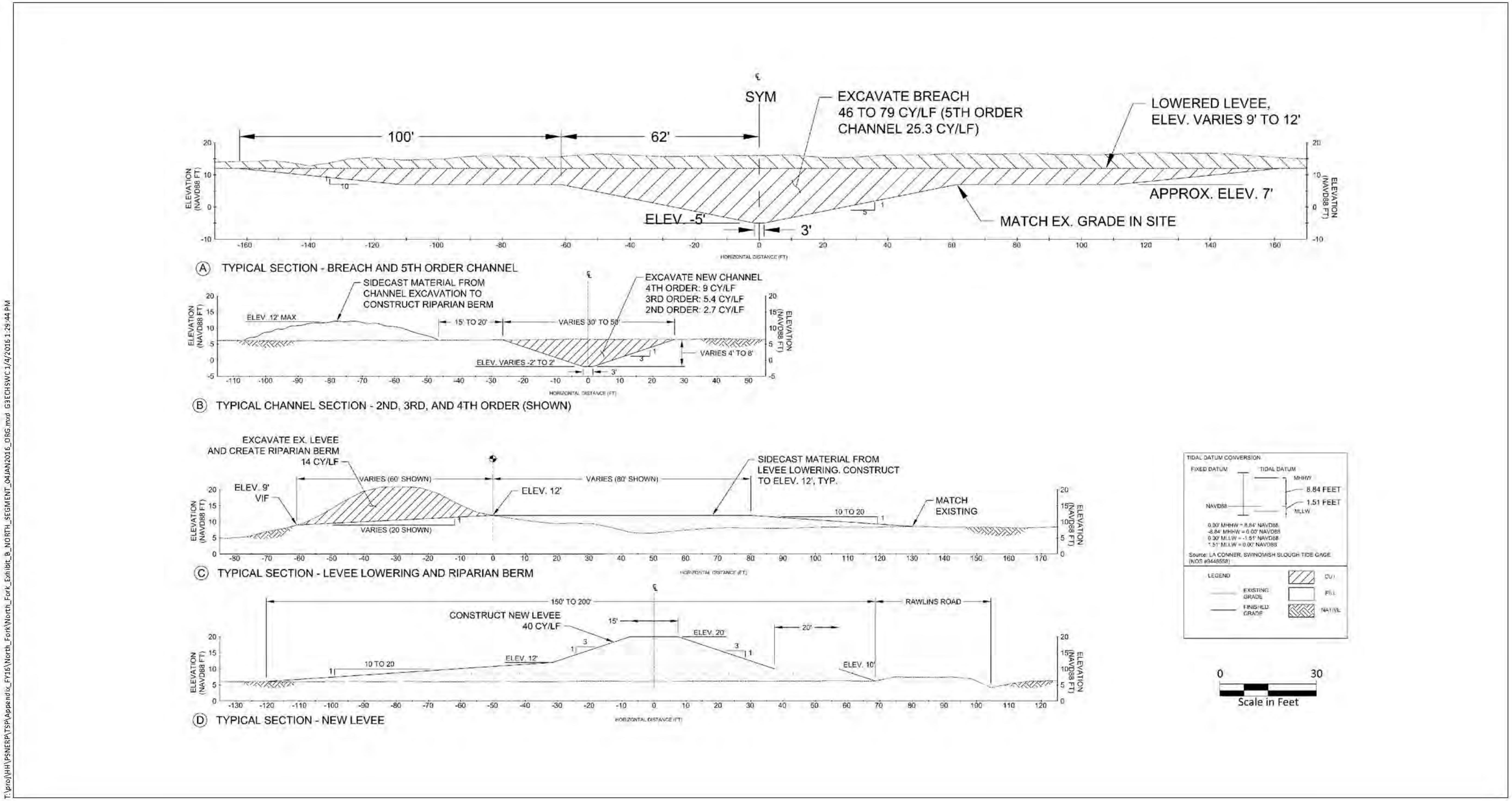


ORTHO_2013_NAIP_WASHINGTON

Site Name: North Fork Skagit River Delta

Lead Contractor: ESA
Design Lead: Anchor QEA, G. Sassen, ASLA
Revised: USACE Petroff/Campbell December 2015

Exhibit B



ORTHO_2013_NAIP_WASHINGTON

Site Name: North Fork Skagit River Delta

Lead Contractor: ESA
Design Lead: Anchor QEA, G. Sassen, ASLA
Revised: USACE Petroff/Campbell December 2015

Exhibit C

Puget Sound Nearshore Ecosystem Restoration Project
Feasibility Study
Hazardous, Toxic, and Radioactive Waste
Phase 1 Environmental Site Assessment

EXECUTIVE SUMMARY

The Seattle District Corps of Engineers (Corps), working collaboratively with the Washington Department of Fish and Wildlife (WDFW) as local sponsor, along with many other regional partners, has conducted a General Investigation (GI) to evaluate problems and potential solutions of ecosystem degradation and habitat loss in Puget Sound, Washington. The Puget Sound Nearshore Study (Nearshore Study) is authorized under Section 209 of the River and Harbor Act of 1962 (Pub. L. 87–874). The Corps and local sponsor are recommending implementation of restoration actions at three sites throughout the study area as the outcome of the Nearshore Study. Pursuant to Section 102(2)(C) of the National Environmental Policy Act (NEPA) of 1969, as amended, the U.S. Army Corps of Engineers is preparing an Integrated Feasibility Report/Environmental Impact Statement (FR/EIS) for the three restoration actions. The Phase 1 Environmental Site Assessment for the North Fork Skagit site is being conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments,* and ER 1165-2-132: *HTRW Guidance for Civil Works Projects,* except where noted below.

This assessment has revealed one potential recognized environmental condition in connection with the proposed project footprint, known as the Rexville Grocery site. However, due to the contaminants involved, degradation rates of those contaminants, topography, and the half mile distance between the Grocery and the subject property, this site is not expected to interact in any way with the proposed project. As a result, this assessment has revealed no evidence of recognized environmental conditions in connection with the proposed project footprint, nor any conditions at neighboring sites which have the potential to affect work at the project site.

Puget Sound Nearshore Ecosystem Restoration Project
Feasibility Study
Hazardous, Toxic, and Radioactive Waste
Phase 1 Environmental Site Assessment

1.0 INTRODUCTION

1.1 Involved Parties

The Corps is the lead Federal agency for the Puget Sound Nearshore Ecosystem Restoration Project (PSNERP) Report. The non-Federal, cost-sharing sponsor is the Washington Department of Fish and Wildlife (WDFW). As the non-Federal sponsor, WDFW contributes 50 percent of the total feasibility study costs in the form of cash or in-kind contributions; a feasibility cost sharing agreement was executed in 2001, with amendments.

1.2 Authority

The Puget Sound Nearshore Study (Nearshore Study) is authorized under Section 209 of the River and Harbor Act of 1962 (Pub. L. 87–874).

1.3 Guidance and Policy

Corps policy providing guidance for consideration of issues and problems associated with hazardous, toxic, and radioactive wastes (HTRW), as defined in this regulation, which may be located within project boundaries or may affect or be affected by Corps Civil Works projects is contained in ER 1165-2-132, *Hazardous, Toxic, and Radioactive Waste Guidance for Civil Works Projects*, which defines HTRW as "…any material listed as a 'hazardous substance' under the Comprehensive Environmental Response, Compensation, Liability Act (CERCLA)". ASTM International (ASTM) Standard E 1527-13 *Standard Practice for Environmental Site Assessments: Phase I Environmental Site Assessment Process* provides a comprehensive guide for conducting an HTRW Assessment. An assessment identifies known or suspected releases of hazardous substances (recognized environmental conditions) based on records review, site visit, and interviews.

1.4 Scope of Work

The complete investigation serves to identify any recognized environmental condition, as defined in ASTM Standard E 1527-13. This site assessment documents known and suspected HTRW sites discovered through a search and review of all reasonably attainable federal, state, and local government information and records. A site visit, interviews with relevant stakeholders, and review of aerial photographs are also mandated under the above standard.

1.5 Significant Assumptions

This report identifies known and suspected environmental concerns, both past and present based on availability of information at the time of the assessment. It is possible that unreported disposal of waste or illegal activities impairing the environmental status of the properties may have occurred which could not be identified.

1.6 Limitations and Exceptions

This assessment deviates slightly from the exact procedures outlined in ASTM E1527-13. Specifically, no "User Provided Information" nor "Non-Scope Services" were provided, and those sections of the report were omitted. Also, due to the layout of the overall document to which this report will be incorporated, it was decided that no appendices were to be generated for this report. Additionally, it should be noted that portions of this report were conducted by separate agency entities that did not have the ability to coordinate their efforts.

1.7 Special Terms and Conditions

No special terms or conditions with respect to ER 1165-2-132 and ASTM E 1527-13 standards were made.

1.8 User Reliance

In accordance with ASTM E 1527-13 Section 7.5.2.1 "Reliance," the environmental professional is not required to independently verify the information provided by various sources but may rely on the information unless there is actual knowledge that certain information is incorrect or unless it is obvious that certain information is incorrect based on other information obtained during the course of the investigation or otherwise actually known to the investigators conducting the assessment. At the present time there is no indication that the information provided by the database search is incorrect.

2.0 SITE DESCRIPTION

2.1 Location and Legal Description

The "property", as defined by the referenced ASTM standard, in this case includes several different properties on the southern bank on the North Fork Skagit River. For the purposes of this assessment, the proposed North Fork project footprint will serve as the "property" under review (See Figure 6-6 in the main body text of the feasibility report).

2.2 Site and Vicinity General Characteristics

The physical setting of the subject property and vicinity is detailed in Section 6.1.3 of the main feasibility report.

3.0 RECORDS REVIEW

3.1 Standard Environmental Records

A records search was conducted on May 13, 2014 and on November 2, 2015 using a variety of sources. These sources included EPA's National Priority List Mapper, EPA's EnviroFacts database, the Washington State Department of Ecology's (Ecology) Toxics Cleanup Program (TCP) database, and Ecology's Facility/Site database. Below are the parameters and results of the records search.

Parameter	Source	Minimum Search Distance (mi.)	Results
Federal NPL	EPA NPL Mapper	1	None
Federal Delisted NPL	EPA NPL Mapper	0.5	None
Federal CERCLIS	EnviroFacts	0.5	None
Federal RCRA Generators	EnviroFacts	Property and Adjoining Properties Only	None
Federal RCRA TSDs	EnviroFacts	0.5	None
Federal RCRA Corrective Action Sites	EnviroFacts	1	None
Federal and State ICs Registry	Ecology TCP	Property Only	None

Parameter	Source	Minimum Search Distance (mi.)	Results
Federal Toxic Release Inventory	EnviroFacts	0.5	None
State and Tribal Cleanup Sites	Ecology TCP	1	1 finding (Rexville Grocery)
State and Tribal Landfills and/or Solid Waste Disposal Facilities	Ecology Facility Search	0.5	None
State and Tribal UST Registry	Ecology TCP	Property and Adjoining Properties Only	None
State and Tribal LUST	Ecology TCP	0.5	1 finding (Rexville Grocery)
State and Tribal Brownfields	Ecology TCP	0.5	None

The records search did not identify any known or suspected contaminant releases within the project footprint. Ecology lists one state cleanup collocated with one leaking underground storage tank within a half mile of the project boundaries, known as the Rexville Grocery or the Little Country Store. Further investigation revealed that the leaking underground storage tank was reported as cleaned up in 1993, with 30 cy of contaminated soil being disposed of as well. However, Ecology could not verify the completeness of the cleanup and as a result, the site is still on Ecology's cleanup list. As of 2015, additional investigations by the State may occur. Despite the presence of this site approximately a half mile from the subject property, the proposed project would not affect, nor be affected by the Rexville Grocery cleanup investigation due to its distance from the project footprint. Additionally, the volatilization/degradation rates of the contaminants spilled (in this case gasoline and associated byproducts) over the course of 22+ years means there is an extremely low likelihood of these substances remaining in the groundwater in reportable quantities, much less migrating the half mile distance to the proposed Corps project. Finally, the topography of the area north of the river is not conducive to rapid groundwater flow into the proposed Corps project. No other sites of concern were noted during the records search.

3.2 Historical Records

A review of aerial photographs spanning 75 years show no major changes to land use on the subject property or its surroundings. The land is almost completely agricultural, with no intensive excavation noted, and no industrial activity present in the area.

3.3 Additional Environmental Record Sources

There are no additional environmental record sources included in this assessment.

4.0 SITE RECONNAISSANCE

A site visit was conducted by the USFWS in October 2010. The action area is currently owned by multiple landowners, and used primarily for agricultural purposes. The site visit noted the presence of several households, farms, and/or debris locations, but these were not accessed. Adjacent to the action area is an active RV/resort and boat launch, several transmission lines, and various roads. There is fill in the action area associated with existing levees, roads, and residences in the floodplain. There is likely other farming or residential debris, along with machinery and equipment repair areas on individual landowner properties. A visual survey of the action area revealed no known or suspected contaminant releases or spills. However, the Phase I site visit did not visit all parts of the action area, due to private property restrictions.

An interview was conducted with Mr. Stan Nelson of the Skagit County Diking District #22 on February 17, 2011 by telephone. No record of this interview exists, other than a statement that indicated the possibility of a former Skagit County garbage dump to the west of Brown's Slough Road (now called Fir Island Rd.). This site was not identified in the record search and not much is known about the status of the site, including its exact location. On October 13, 2015, a phone call was placed to Ms. Britt Pfaff-Dunton of the Skagit County Environmental Health Department to follow up on this anecdotal evidence. The respondent stated that while it was not uncommon for farmers to dispose of their trash and agricultural debris in Brown's Slough, Skagit County records show no historical landfills on Fir Island. Review of all available documentation did not identify the alleged landfill, and a review of historical aerial photographs show no evidence of the landfill or any dumping of any kind. Therefore, there are no suspected releases as a result of this alleged landfill.

5.0 FINDINGS AND CONCLUSION

This Phase 1 Environmental Site Assessment was conducted in conformance with the scope and limitations of ASTM E1527-13: *Standard Practice for Environmental Site Assessments*, and ER 1165-2-132: *HTRW Guidance for Civil Works Projects*. The assessment was initially conducted in 2010 by Ginger Phalen, biologist with the USFWS, and updated in 2015 by David Clark, remediation biologist at the U.S. Army Corps of Engineers (Corps).

This assessment has revealed one potential recognized environmental condition in connection with the proposed project footprint, known as the Rexville Grocery site. However, due to the contaminants involved, degradation rates of those contaminants, topography, and the half mile distance between the Grocery and the subject property, this site is not expected to interact in any way with the proposed project. As a result, this assessment has revealed no evidence of recognized environmental conditions in connection with the proposed project footprint, nor any conditions at neighboring sites which have the potential to affect work at the project site.

6.0 SOURCES

ASTM E1527-13: *Standard Practice for Environmental Site Assessments.*

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Attachment A – Cost Annex

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US Army Corps of Engineers®

COST ANNEX

FOR

Puget Sound Nearshore Ecosystem Restoration Project

Puget Sound, WA

Prepared for:

Seattle District, Seattle, WA

Prepared by:

USACE, Seattle District

Date: June 08, 2016

INTRODUCTION

The purpose of this appendix is to document and present the detailed cost estimate prepared in support of the Puget Sound Nearshore (PSNERP) GI. The project is intended to restore a more natural hydrology and environment at project sites throughout the Puget Sound. These individual projects are scattered throughout the Puget Sound, and the scope of work varies widely between them. The local sponsor for this project is the Washington State Department of Fish & Wildlife.

The basis of the cost estimates is conceptual design drawings and conceptual quantities prepared by the Project Delivery Team (PDT). Additional information developed by the PDT is incorporated into the estimate. This includes emails, phone calls, and in-person discussions. The MCACES estimate documents the basis of information used in development of costs, down to the lowest reasonable level. Guidance for preparation was obtained from ER 1110-2-1150 Engineering and Design (E&D) for Civil Works Projects, ER 1110-1-1300 E&D Cost Engineering Policy and General Requirements, ER 1110-2-1302 Civil Works Cost Engineering, and ETL 1110-2-573 E&D Construction Cost Estimating Guide for Civil Works. The cost estimates were prepared using Micro-Computer Aided Cost Estimating System MII version 4. Supporting cost libraries or databases were MII 2012-b English Cost Book, 2014 Region VIII Equipment library (EP 1110-1-8) and the 2015 National Labor Library rates for Seattle, Washington. Fuel rates were obtained online from gasbuddy.com.

The cost estimate was prepared at a level commensurate with the level of design detail, which should be considered a budget or class III estimate. Substantial additional design will be required at all sites that are contained within the PSNERP footprint.

Uncertainties regarding the cost estimate are documented in the Cost and Schedule Risk Analysis (CSRA) risk register report and based on a Formal risk analysis suitable for this stage in the planning process.

Quantities used in the cost estimate came from two sources: primarily they were developed by the applicable designer and delivered to the cost engineer, who then validated that they are reasonable. Additionally, limited quantities were developed by the cost estimator to support the estimate.

Lastly, this is a cost share project with the Washington State Department of Fish & Wildlife (WDFW) as the Local Sponsor. Federal costs are anticipated to be 65% of the Total Project Cost of the restoration features, with the balance to be WDFW's share.

PRICE LEVEL

The three categories of cost contained in the Total Project Cost Summary (TPCS) are "Estimated Cost," "Project First Cost," and "Total Project Cost." The estimated cost, which is the cost calculated in MCACES (MII), is based on a price level of October 2015. The Project First Cost, or in other words the value the project is actually authorized at, is also set at October 2015. Lastly, the date point of the Total Project Cost which is the cost the government will pay at the year of construction varies from 2020 to 2028. The sites are expected to be constructed over a period of decades and each site will have its own midpoint of construction.

Escalation is based on the September 2015 Civil Works Construction Cost Index System (CWCCIS), EM 1110- 2-1304.

It is assumed that the Seattle metropolitan area possesses a sufficiently large and diverse enough contractor, labor, equipment, and material base to support the project. The potential that this is not the case is considered within the risk analysis. Sources of cost information include MII 2012-b English Cost Book, 2014 Region VIII Equipment library (EP 1110-1-8) and 2015 Davis Bacon wage rates for Skagit County , Washington. Additionally, local vendor quotes for critical items were solicited and utilized for major components, or items that would otherwise be difficult to account for.

The cost of the selected plan is considered fair and reasonable, provided the construction is done by a prudent and well equipped contractor.

COST ESTIMATE STRUCTURE

The cost estimate for the project was prepared by the Cost Engineering Section within Seattle District. The overall structure of the cost estimate is dictated by the Civil Works – Work Breakdown Structure. This structure is followed down to the sub-feature level (e.g. feature 11 Levees and Floodwalls, followed by sub-feature 1101 Levees.) The remainder of the estimate structure is based on the expected construction methodology and phasing techniques as determined by the PDT.

A site specific discussion regarding cost, schedule, and risk is included within the Engineering Appendix. What follows is a discussion regarding the methodology used in estimate development throughout the program.

Project features in the total project cost summary (TPCS) are in accordance with the CWWBS:

01 Lands and Damages include the real estate acquisitions of project lands, easements and rights-of ways and PL 91-646 relocation costs. Additional real estate costs include: non-Federal’s sponsors cost for land surveys, title preparation, legal opinions and Federal costs of reviewing the non-Federal sponsor’s documents for legal sufficiency.

02 Relocation costs include new construction to modify existing public infrastructure. This will include the cost to construct new roads, bridges, levees and utilities All

demolition costs for existing infrastructures was determined to be a general construction cost and was assigned to the appropriate WBS item.

06 Fish & Wildlife Facilities. Since the main purpose of this project is ecosystem restoration all construction features not captured in 02 Relocations are summarized under WBS 06. This is reflected in the summary sheet of the TPCS, where 08, 09 & 11 project features are rolled into the 06 WBS item.

Typical 06 project features include efforts such as plantings, placement of large woody debris, removal of environmental stressors, and filling of agricultural ditches that prevent natural processes.

While not separately given their own line in the top sheet of the TPCS, Adaptive Management and Monitoring are both funded under WBS 06, and are explicitly noted for each project site. These are separately accounted for in MCACES, as they are based on scope provided by the PDT. Both of these are allowed under ER 1105-2-100 Sec. 3-5.b.(8). Implementation Guidance issued August 31, 2009 for Section 2039 of WRDA 2007 requires these activities be done, and that Monitoring can be cost shared for 10 years following construction completion.

Roads, Railroads & Bridges. While not separately given its own line on the summary sheet of the TPCS this project feature still captures construction costs in the supporting pages of the TPCS and the MCACES report. This includes all work to create new roadways such as pavement, earthwork shaping, guardrails and stripping. As well all work to create bridges including temporary access, bridge construction and bridge demo is also captured here.

Channels & Canals. While not separately given its own line on the summary sheet of the TPCS this project feature still captures construction costs in the supporting pages of the TPCS and MCACES report. This accounts for all costs to create tidal channels and canals that will restore areas to a more natural wetland state.

Levee and Floodwalls. While not separately given its own line on the summary sheet of the TPCS this project feature still captures construction costs in the supporting pages of the TPCS and MCACES report. Includes costs for demolition, installation, breaching, and lowering of all levees within a site. Other incidentals such as levee armoring are also accounted for here.

18 Cultural Resource Preservation. Includes calculated costs for work related to site surveys. These surveys may be done by the contractor and would evaluate historical structures and archaeological elements at the site. At the requirement of the Planning Team, an additional amount to account for Cultural Resource data recovery was included. This is allowed per ER 1105-2-100 App C C-4.d (10). The amount selected was 1% of all construction costs (not including PED, CM, Monitoring, or Adaptive Management) and is accounted for under the TPCS. Similar to Adaptive Management and Monitoring, this 1% additional cost is accounted for only within the TPCS. MCACES contains calculated values that

cover surveys, analyses, and reports, however, the 1% of construction costs is in addition to that.

30 Planning, Engineering and Design (PED). Provides the estimated engineering design costs for each project. In consultation with the Project Manager, three ranges for PED costs were determined based on the construction cost of the project. These categories were: projects between $0.1M and $10M, $10.1M to $99.9M, and $100M to $500M. Projects at the upper and lower ends of the spectrum had PED costs based on lump sum values, while those ranging from $10.1M to $99.9M had costs based on percentages. In general, design costs are somewhat correlated to project cost (as projects cost more, they are typically more complex). However, for lower construction cost projects, design cost estimates may under calculate the amount of work required, while for larger projects they may over calculate. Thus, set values were used for these extremes.

31 Construction Management (CM) provide the estimated CM or Supervision and Administration costs based on a percentage or lump sum of the construction cost features. The ranges used for this are identical to those used in 30 PLANNING, ENGINEERING, & DESIGN.

PROJECT CONTINGENCY

Contingencies are added to the cost estimates in the TPCS based on the results of the cost and schedule risk analysis performed in 2015. Further information about the CSRA can be found in the CSRA Report. The overall contingency of the project is 38.8%. Contingency values vary between sites but encompass a range of 36% to 46%. Results of the cost risk study yielded a percent contingency which has been added to the construction costs of the project. This value is not separate and distinct from the base cost of the project, but is a key component of it. As the project is developed, it is expected that the calculated cost of various features will increase as more becomes known regarding the sites. As more detail is available, contingency will be reduced as uncertainty and risk are lowered.

PROJECT ESCALATION

Escalation factors to the Effective Price Level Date and the Fully Funded Project Estimate Amount through the end of construction have also been included as part of the TPCS. The inflation was based on an assumed authorization date of October 1, 2015 and a mid-point of construction that varies between projects (2020-2028).

The date of authorization and schedule of projects are assumptions. If the date of authorization is delayed, the overall project schedule will be delayed. Similarly, the project schedule could be extended if funding does not match the original expectations.

PROJECT ASSUMPTIONS

At this phase of design there are still many unknowns and data gaps that won't be resolved until the project reaches the PED phase. Many assumptions were made and coordinated with the PDT to account for the design limitations. All assumptions were made with a slight conservatism already built in, knowing full well that a formal cost and schedule risk analysis would be used to capture the project contingency.

Key assumptions made while preparing construction costs:

- Disposal points for rock, fill, and construction debris are available within 30 miles for Nooksack and North Fork and within 60 miles for Duckabush.
- Sources of rock, fill, and general construction materials are available within 30 miles for Nooksack and North Fork and within 60 miles for Duckabush.
- Levee construction will require entirely new fill.
- Levees that are removed will have their fill disposed of off-site, unless explicitly noted otherwise.
- Existing roads and utilities will be replaced "in-kind" where demolished except for North Fork. Any changes in design codes must be incorporated when constructing new public infrastructure. However, if an existing road is two lanes, it will not be replaced with a four lane highway.
- All property acquisitions and easements can be achieved.
- There is sufficient workforce and equipment available to complete the project within the calculated timeframe.
- Work on any site within the PSNER Program will be performed entirely by one Prime Contractor, with multiple sub-contractors. All work will occur under one contract per site.
- Sub-contractors are assumed to do all non-supervisory work. This minimizes risk of alternate contracting arrangements.
- Staging areas are available close to the project elements.

Many of these assumptions were made by the PDT as a whole, but as they have significant cost impacts they are listed above.

DEVELOPMENT OF COSTS BY FEATURE

GENERAL

Most design information for PSNERP originally came from an Architect-Engineer hired by the Washington Department of Fish and Wildlife. A detailed discussion of this information was verified and updated is available in the appropriate Design appendices.

Quantities for each site are accounted for in the MII Estimate for each respective site. Note that there are variations and differences from the Quantity Estimates provided by the A/E that are accounted for in the estimate. This is done based on the USACE design team spot checking quantities and in some cases revising quantities based on current conditions and design standards.

Following is discussion of how the costs for each feature of work were developed.

O1 REAL ESTATE

Real estate acquisition costs along with relocations costs and internal administrative costs for land purchases and easements were provided to the Cost Engineer by Diane Hintz (NWS Real Estate). Real Estate provided their own contingency on such costs. Note that contingencies are not broken out separately in the TPCS for 01 REAL ESTATE. Please see the real estate appendix for further information.

02 RELOCATIONS

Relocations costs for each site were considered on a case by case basis. For each site, all features of work were evaluated to determine if it would be considered a relocation costs. Office of counsel, the Project Manager and Planner were involve in this discussion. Features of work that are typically considered for relocation costs are roads, bridges, levees and utilities. More specifically the cost for new construction was deemed a relocation cost. The demolition of such features of work was assigned to either WBS 06, 08, 09 or 11.

06 WILDLIFE FACILITIES & SANCTUARY

Quantities and rough scopes of what work will occur was provided by NWS Civil Design while being coordinated with NWS Environmental and Cultural Resources Branch (ECRB). For example, while vegetation areas were provided by Civil Design, the plant mix and spacing was provided by ECRB. Other elements, such as the large woody debris, were based on designs from other projects. Costs for this feature come from vendor quotes, RS Means items, and estimator judgment related to productivity. This account encompasses the widest array of different activities, but one feature that is consistent across most sites is building demolition. Little is known about the various structures being demolished within the project sites. Single story steel framed buildings were assumed, and the fact that this is likely to vary is considered in the risk analysis.

Plantings were given to Cost Engineering as an area, and a template plant spacing and species mix was provided by ECRB. A riparian mix was developed for upland areas, and a saltwater plantings mix was developed for areas that would be inundated.

ROADS, RAILROADS & BRIDGES

Information regarding road removal and installation, utilities, bridges, and railway construction activities would be done was provided to Cost Engineering by NWS Civil Design. Bridge information was coordinated with NWS Structures and NWS Geotechnical Design. Subsequently installation costs were developed based on RS Means production rates, custom crews and production rates based on site specific constraints, and vendor quotes for materials. Bridge design information was limited for the project and a standardized model was developed and used in all locations where road or rail bridge construction occurred.

This standardized bridge model includes large caissons being drilled to their appropriate depth, piers being installed, and heavy use of cranes to set girders into place. For new roads, 10" asphalt on top of 12" of crushed rock is assumed. Road construction includes crews to demo and replace hot mix asphalt, place pavement marking, provide traffic control and survey the site.

CHANNELS & CANALS

Quantities and rough scopes of what work will occur was provided by NWS Civil Design. Limited hydraulic design work was done by NWS Hydraulics and Hydrology (H&H) Branch. Excavation is done by tracked excavator or backhoe and is either side cast in low berms, used elsewhere on-site, or hauled to an off-site disposal facility. The channels are shallow and two cubic yard excavators are typically used.

LEVEES AND FLOODWALLS

Quantities and rough scopes of what work will occur was provided by NWS Civil Design. Fill material costs are based on vendor quotes (note that all fill is assumed to be purchased), and production rates and crew composition is based on RS Means items or calculated by the cost engineer. In certain areas, protective rip rap will be placed to armor the riverward slope. All grading information was developed from LIDAR surveys and contains a relatively high amount of uncertainty. This uncertainty is discussed and evaluated as part of the Cost & Schedule Risk Analysis.

Existing levees are either lowered by scrapers or excavators, and breaching is done by excavators. Levees that are breached typically are on islands and have their excess material graded out on-site in order to allow natural flow. New levee construction is done by crews consisting of excavators, dozers, and roller compactors that shape the levee into the proper prism.

Unusual construction methodologies or problematic design conditions were not assumed in the estimate. Settlement during or shortly after construction was not assumed to be an issue at project sites. The PDT does recognize that these issues are likely to occur at some of the projects, but lacked sufficient information to make a determination. The possibility that this will occur is evaluated in the Cost & Schedule Risk Analysis.

18 CULTURAL RESOURCE PRESERVATION

Requirements were specified by NWS Environmental and Cultural Resources Branch. Includes time for survey, analysis of each site and report preparation.

PROJECT SCOPE

The purpose of this project is to address and restore ecosystems through restoration of natural processes and restoration and/or re-creation of coastal wetlands and embayments. There are three sites in the recommended plan with locations throughout the Puget Sound. Restoration actions will occur near the mouth of the Duckabush River, Skagit River and Nooksack Rivers, respectively.

The proposed action to the Duckabush River Estuary would restore the natural geomorphology by removing major roadway obstructions, excavating channels, and removing fill. The action would remove two bridges that total 970 LF and associated approach road embankments along Highway 101. This will be followed by realigning Highway 101 onto a new 2100 LF bridge across the estuarine delta to restore tidal connection to the estuary. A surface street crossing (Shorewood Road) would be modified to tie into the new bridge structure. Adjacent fill at a distributary channel (Pierce Slough) would be removed. Multiple tidally influenced distributary river channels summing to 1,155 LF would be reestablished, and blind tidal channels summing to 4,200 LF would be excavated within the marsh areas.

The proposed action to the Nooksack River Delta would restore riverine and tidal flow as well as sediment transport and delivery processes throughout a substantial portion of the historical Nooksack River delta. This will be accomplished by a combination of construction features that would modify levees, roads, and other hydrological barriers. Construction of approximately 47,000 LF of new setback levees would provide flood risk management protection for active businesses, residences, farms, transportation infrastructure, and Lummi Nation lands in the project area. Features of work associated with levee setback construction include but are not limited to: 12,280 LF of levee removal, installation of 3 log jam structures, installation of 1 new water control structure and 9,980 LF of channel and berm regarding. Construction on existing infrastructure includes but is not limited to: removal of 6 bridges and approach road section and construction of 6 new bridges and approach road sections.

The proposed action to the North Fork Skagit River Delta would restore the riverine floodplain and tidal connectivity along the lower reach of the North Fork of the Skagit River. This will require constructing a new 12,300 LF flood risk management levee further inland. The 15,700 LF existing levee would be lowered and selectively breached in 4 locations to allow inundation of the estuarine emergent marsh and sustain back channel habitat. Forested floodplain habitat would be created along the lowered levee adjacent to the mainstem river channel. This will be done by excavating and creating 39,200 LF of tidal channels, removing 17 buildings in the flood plain, removing 344,300 SF of existing road and plant riparian vegetation over approximately 50 acres.

CONTRACTOR AND INDIRECT COST CONSIDERATIONS

The cost estimator assumed the work is done by a prime contractor who will subcontract most of the construction work. The reasoning for this is that the smaller locations are likely to be managed by Small Business contractors, and the larger ones are likely to be broken up into many smaller contracts.

Assuming a large contractor will be performing this work would be a very risky assumption. Contract acquisition strategy will be an important aspect of the PED phase. Specialty activities such as concrete placement, paving, and electrical/mechanical work are to be done by subcontractors hired by the prime contractor. No more than one level of sub-contractors is presently assumed. This arrangement makes for up to two levels of contracting and markup costs (job office overhead, home office overhead, profit, bond, and B&O tax).

PLANNING, ENGINEERING, AND DESIGN

The Planning, Engineering and Design (PED) costs are the costs from authorization until the site specific construction contract is awarded. This work includes detailed surveys, soil investigations and preparation of the plans and specifications to guide the contractor to construct the project. As noted above these values are established based on project sizes with three separate tiers.

CONSTRUCTION MANAGEMENT

The Construction Management (CM) costs are determined as a percent of the estimated construction costs. Similar to PED funding, this was broken into three tiers. The Planner in coordination with Cost Engineering selected a 9.5 percent contingency.

CONTINGENCY

Current regulations require analysis of schedule and costs risks. See the C&SRA Report Attachment for the Formal Cost Risk Analysis Study (C&SRA) documentation that was performed in 2012, and reevaluated in 2014 and 2015. The results of the cost risk study was a 38.8 percent project contingency (based on the 80% confidence interval) was appropriate for construction costs. A formal Monte Carlo simulation was done for each project site as required.

Contingency for 01 Real Estate costs was determined by NWS Real Estate. These values depend on the specific site and are available in the Real Estate Annex.

An output of the risk analysis is contingency, whose purpose is an added cost included in the cost estimate to cover unknowns. Risk drivers vary across the project, but certain elements were common across the project.

Unknowns across the project could include:

- Earthwork quantities were considered very speculative given the limited amount of design information available and the limited resolution of the LIDAR surveys. Variations of 20% could occur where levees and other major earthwork construction occurs.
- The bridge construction model used across the project utilizes a number of assumptions and there could be significant cost increases if these change during later design.
- A common assumption across all projects is that one Prime Contractor will be responsible for all construction. This was done to ease the estimating process and since there is not a way at present to determine how these will be contracted. It is likely that these sites will be split into two or more contracts. The risk to the base construction cost is relatively low: JOOH is calculated based on a percentage of the project cost and is not dependent on schedule, and sub- contractors are assumed to do all work on a site. However, there would be substantial impacts to overall project schedule.
- The available information on existing roads is minimal and there is a large degree of uncertainty as to what the existing roads are like and what exact changes will need to be made to accommodate new and raised levees. While the construction required for roads is well understood, additional detail will be needed related to items such as road width, depth of base and HMA, as well as how much road will be need to be relocated. This also applied to utilities, as information available was minimal at best.
- There may be difficulties in dealing with the large number of entities that will be affected by the project. State and federal highways, railroad mainlines, local utilities, and affected homeowner will all have concerns regarding project impacts to their property. This should not be viewed as a problem, as properly resolving stakeholder concerns is desirable. However, the scope and requirements of this is unknown, and may require extended time spent in the PED phase, or even changes to the project footprint to make reasonable accommodations.

PROJECT SCHEDULE

The overall project schedule and sequence was determined in coordination with Project Management. The anticipated sequence will start with Duckabush, followed by North Fork and finally end with Nooksack.

The site specific construction schedules were developed by the cost engineer based on MII calculated durations and logical sequencing of construction features. Project length depends upon the individual site and projected durations range from two to six years. Note that all durations are built on the assumption of a single contract, with concurrent construction activities. Breaking the contract into smaller elements would increase the duration.

Prior to construction start and after authorization, Planning, Engineering, and Design will occur. This is expected to take at least two years for Duckabush and North Fork and five years for Nooksack. PED will continue until all projects are contracted.

OPERATIONS, MAINTENANCE, REPAIR, REHABILITATION, AND REPLACEMENT COSTS

An OMRR&R estimate was prepared based on scope put together by the PDT. This cost is included in the TPCS as a stand-alone cost and does not roll up into the total project cost.

FINAL FEASIBILITY ESTIMATE

The final feasibility cost estimate as presented in the following Total Project Cost Summary (TPCS) for is as follows:

Cost of Puget Sound Nearshore Environmental Restoration Program
Washington State
2016 Feasibility Report

FY 2016 Price Level
$451,627,000
Fully Funded Amount
$539,839,000

Estimated Federal Cost
$350,896,000
Estimated Non-Federal Cost
$188,944,000

ATTACHMENTS

TPCS
MCACES REPORT
PROJECT SCHEDULES
CSRA REPORTS

WALLA WALLA COST ENGINEERING MANDATORY CENTER OF EXPERTISE

COST AGENCY TECHNICAL REVIEW

CERTIFICATION STATEMENT

For Project No. 106225

NWS – Puget Sound Nearshore Ecosystem Restoration
Feasibility Report

The Puget Sound Nearshore Ecosystem Restoration - Feasibility Study, as presented by Seattle District, has undergone a successful Cost Agency Technical Review (Cost ATR), performed by the Walla Walla District Cost Engineering Mandatory Center of Expertise (Cost MCX) team. The Cost ATR included study of the project scope, report, cost estimates, schedules, escalation, and risk-based contingencies. This certification signifies the products meet the quality standards as prescribed in ER 1110-2-1150 Engineering and Design for Civil Works Projects and ER 1110-2-1302 Civil Works Cost Engineering.

As of June 8, 2016, the Cost MCX certifies the estimated total project cost of:

FY 2016 Price Level: $451,627,000
Fully Funded Amount: $539,839,000

It remains the responsibility of the District to correctly reflect these cost values within the Final Report and to implement effective project management controls and implementation procedures including risk management throughout the life of the project.

®

Kim C. Callan, PE, CCE, PM
Chief, Cost Engineering MCX
Walla Walla District

**** TOTAL PROJECT COST SUMMARY ****

PROJECT: **Puget Sound Nearshore Ecosystem Restoration**
PROJECT NO: 106225
LOCATION: Washington State

DISTRICT: NWS Seattle District **PREPARED: 3/15/2016**
POC: CHIEF, COST ENGINEERING, John Dudgeon

This Estimate reflects the scope and schedule in report; PSNERP Feasibility Report

Program Year (Budget EC): 2016
Effective Price Level Date: 1 OCT 15

Civil Works Work Breakdown Structure		**ESTIMATED COST**				**PROJECT FIRST COST (Constant Dollar Basis)**						**TOTAL PROJECT COST (FULLY FUNDED)**			
WBS NUMBER *A*	Civil Works Feature & Sub-Feature Description *B*	COST ($K) *C*	CNTG ($K) *D*	CNTG (%) *E*	TOTAL ($K) *F*	ESC (%) *G*	COST ($K) *H*	CNTG ($K) *I*	**TOTAL ($K)** *J*	Spent Thru: **10/1/2015** ($K)	**TOTAL FIRST COST** ($K)	ESC (%)	COST ($K) *M*	CNTG ($K) *N*	FULL ($K) *O*
02	RELOCATIONS	$91,818	$38,682	42.1%	$130,500	0.0%	$91,818	$38,682	$130,500	$0	$130,500	12.5%	$103,320	$43,520	$146,841
06	FISH & WILDLIFE FACILITIES	$128,945	$50,331	39.0%	$179,276	0.0%	$128,945	$50,331	$179,276	$0	$179,276	19.1%	$153,453	$59,985	$213,438
18	CULTURAL RESOURCE PRESERVATION	$4,467	$1,811	40.5%	$6,278	0.0%	$4,467	$1,811	$6,278	$0	$6,278	17.3%	$5,241	$2,125	$7,367
						-						-			
						-						-			
						-						-			
						-									
	CONSTRUCTION ESTIMATE TOTALS:	$225,230	$90,824	40.3%	$316,054	0.0%	$225,230	$90,824	$316,054	$0	$316,054	16.3%	$262,015	$105,631	$367,646
01	LANDS AND DAMAGES	$25,651	$5,338	20.8%	$30,989	0.0%	$25,651	$5,338	$30,989	$0	$30,989	10.1%	$28,262	$5,873	$34,135
30	PLANNING, ENGINEERING & DESIGN	$53,103	$21,456	40.4%	$74,559	0.0%	$53,103	$21,456	$74,559	$0	$74,559	26.6%	$67,265	$27,140	$94,405
31	CONSTRUCTION MANAGEMENT	$21,397	$8,628	40.3%	$30,025	0.0%	$21,397	$8,628	$30,025	$0	$30,025	45.4%	$31,109	$12,545	$43,655
	PROJECT COST TOTALS:	$325,381	$126,246	38.8%	$451,627		$325,381	$126,246	**$451,627**	$0	**$451,627**	19.5%	$388,651	$151,188	$539,839

CHIEF, COST ENGINEERING, John Dudgeon

PROJECT MANAGER, Lynn Wetzler

CHIEF, REAL ESTATE, Patricia Fatherree

CHIEF, PLANNING, Valerie Ringold

CHIEF, ENGINEERING, JoAnn Walls

CHIEF, OPERATIONS, Elizabeth Coffey

CHIEF, CONSTRUCTION, Mark Slominski

CHIEF, CONTRACTING, Dave Williams

CHIEF, PM-PB, Valerie Ringold

CHIEF, PPMD & DDEPM, Olton Swanson

ESTIMATED FEDERAL COST:	**65%**	**$350,896**
ESTIMATED NON-FEDERAL COST:	**35%**	**$188,944**
ESTIMATED TOTAL PROJECT COST:		**$539,839**
Estimated OMRRR (FY2016)	**$**	**43,180**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Puget Sound Nearshore Ecosystem Restoration
LOCATION: Washington State
This Estimate reflects the scope and schedule in report; PSNERP Feasibility Report

DISTRICT: NWS Seattle District
POC: CHIEF, COST ENGINEERING, John Dudgeon
PREPARED: 3/15/2016

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared:			3/15/2016	Program Year (Budget EC):			2016					
		Effective Price Level:			10/1/2015	Effective Price Level Date:			1 OCT 15					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	RISK BASED CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Duckabush													
02	RELOCATIONS	$32,576	$14,985	46%	$47,561	0.0%	$32,576	$14,985	$47,561	2022Q1	12.2%	$36,538	$16,808	$53,346
06	FISH & WILDLIFE FACILITIES	$4,602	$2,117	46%	$6,719	0.0%	$4,602	$2,117	$6,719	2022Q1	12.2%	$5,162	$2,374	$7,536
06	MONITORING	$164	$41	25%	$205	0.0%	$164	$41	$205	2024Q1	36.8%	$224	$56	$280
06	ADAPTIVE MANANGEMENT	$1,473	$678	46%	$2,151	0.0%	$1,473	$678	$2,151	2024Q1	16.7%	$1,719	$791	$2,510
06	ROADS, RAILROADS & BRIDGES	$6,127	$2,818	46%	$8,945	0.0%	$6,127	$2,818	$8,945	2022Q1	12.2%	$6,872	$3,161	$10,034
18	CULTURAL RESOURCE PRESERVATION	$1,239	$570	46%	$1,809	0.0%	$1,239	$570	$1,809	2022Q1	12.2%	$1,390	$639	$2,029
	CONSTRUCTION ESTIMATE TOTALS:	$46,181	$21,209	46%	**67,390**		$46,181	$21,209	$67,390			$51,905	$23,829	$75,734
01	LANDS AND DAMAGES	$491	$63	13%	$554	0.0%	$491	$63	$554	2018Q4	5.1%	$517	$66	$583
30	PLANNING, ENGINEERING & DESIGN													
2.5%	Project Management	$1,155	$530	46%	$1,685	0.0%	$1,155	$530	$1,685	2018Q4	11.0%	$1,282	$589	$1,870
1.0%	Planning & Environmental Compliance	$462	$212	46%	$674	0.0%	$462	$212	$674	2018Q4	11.0%	$513	$235	$748
12.0%	Engineering & Design	$5,542	$2,545	46%	$8,087	0.0%	$5,542	$2,545	$8,087	2018Q4	11.0%	$6,150	$2,824	$8,975
1.0%	Reviews, ATRs, IEPRs, VE	$462	$212	46%	$674	0.0%	$462	$212	$674	2018Q4	11.0%	$513	$235	$748
0.5%	Life Cycle Updates (cost, schedule, risks)	$231	$106	46%	$337	0.0%	$231	$106	$337	2018Q4	11.0%	$256	$118	$374
1.0%	Contracting & Reprographics	$462	$212	46%	$674	0.0%	$462	$212	$674	2018Q4	11.0%	$513	$235	$748
3.0%	Engineering During Construction	$1,385	$636	46%	$2,021	0.0%	$1,385	$636	$2,021	2022Q1	26.1%	$1,747	$802	$2,549
2.0%	Planning During Construction	$924	$424	46%	$1,348	0.0%	$924	$424	$1,348	2022Q1	26.1%	$1,166	$535	$1,701
1.0%	Project Operations	$462	$212	46%	$674	0.0%	$462	$212	$674	2018Q4	11.0%	$513	$235	$748
31	CONSTRUCTION MANAGEMENT													
5.0%	Construction Management	$2,309	$1,060	46%	$3,369	0.0%	$2,309	$1,060	$3,369	2022Q1	26.1%	$2,913	$1,338	$4,250
2.0%	Project Operation:	$924	$424	46%	$1,348	0.0%	$924	$424	$1,348	2022Q1	26.1%	$1,166	$535	$1,701
2.5%	Project Management	$1,155	$530	46%	$1,685	0.0%	$1,155	$530	$1,685	2022Q1	26.1%	$1,457	$669	$2,126
	CONTRACT COST TOTALS:	$62,145	$28,378		**$90,523**		$62,145	$28,378	$90,523			$70,609	$32,248	**$102,856**
	OMRRR	*$4,178*	*$1,919*	*46%*	***$6,097***									

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Puget Sound Nearshore Ecosystem Restoration
LOCATION: Washington State
This Estimate reflects the scope and schedule in report; PSNERP Feasibility Report

DISTRICT: NWS Seattle District PREPARED: 3/15/2016
POC: CHIEF, COST ENGINEERING, John Dudgeon

Civil Works Work Breakdown Structure		**ESTIMATED COST**				**PROJECT FIRST COST (Constant Dollar Basis)**				**TOTAL PROJECT COST (FULLY FUNDED)**				
		Estimate Prepared:			**3/15/2016**	Program Year (Budget EC):			2016	FULLY FUNDED PROJECT ESTIMATE				
		Effective Price Level:			10/1/2015	Effective Price Level Date:			1 OCT 15					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Nooksack													
02	RELOCATIONS	$59,242	$23,697	40%	$ 82,939	0.0%	$59,242	$23,697	$82,939	2022Q2	12.7%	$66,782	$26,713	$93,495
06	FISH & WILDLIFE FACILITIES	$20,482	$8,193	40%	$ 28,675	0.0%	$20,482	$8,193	$28,675	2028Q1	26.3%	$25,872	$10,349	$36,221
06	MONITORING	$405	$101	25%	$ 506	0.0%	$405	$101	$506	2032Q1	95.2%	$790	$198	$988
06	ADAPTIVE MANANGEMENT	$790	$316	40%	$ 1,106	0.0%	$790	$316	$1,106	2032Q1	36.7%	$1,080	$432	$1,512
06	ROADS, RAILROADS & BRIDGES	$6,522	$2,609	40%	$ 9,131	0.0%	$6,522	$2,609	$9,131	2028Q1	26.3%	$8,238	$3,295	$11,534
06	CHANNELS & CANALS	$624	$250	40%	$ 874	0.0%	$624	$250	$874	2028Q1	26.3%	$788	$315	$1,103
06	LEVEES & FLOODWALLS	$41,249	$16,500	40%	$ 57,749	0.0%	$41,249	$16,500	$57,749	2028Q1	26.3%	$52,104	$20,842	$72,945
18	CULTURAL RESOURCE PRESERVATION	$1,971	$788	40%	$ 2,759	0.0%	$1,971	$788	$2,759	2028Q1	26.3%	$2,490	$996	$3,486
	CONSTRUCTION ESTIMATE TOTALS:	$131,285	$52,453	40%	**183,738**		$131,285	$52,453	$183,738			$158,145	$63,139	$221,284
01	LANDS AND DAMAGES	$13,809	$2,705	20%	$ 16,514	0.0%	$13,809	$2,705	$16,514	2022Q2	12.7%	$15,566	$3,049	$18,615
30	PLANNING, ENGINEERING & DESIGN													
2.5%	Project Management	$3,282	$1,311	40%	4,593	0.0%	$3,282	$1,311	$4,593	2022Q2	27.4%	$4,181	$1,671	$5,852
1.0%	Planning & Environmental Compliance	$1,313	$525	40%	1,838	0.0%	$1,313	$525	$1,838	2022Q2	27.4%	$1,673	$668	$2,341
12.0%	Engineering & Design	$15,754	$6,294	40%	22,048	0.0%	$15,754	$6,294	$22,048	2022Q2	27.4%	$20,071	$8,019	$28,090
1.0%	Reviews, ATRs, IEPRs, VE	$1,313	$525	40%	1,838	0.0%	$1,313	$525	$1,838	2022Q2	27.4%	$1,673	$668	$2,341
0.5%	Life Cycle Updates (cost, schedule, risks)	$656	$262	40%	$918	0.0%	$656	$262	$918	2022Q2	27.4%	$836	$334	$1,170
1.0%	Contracting & Reprographics	$1,313	$525	40%	1,838	0.0%	$1,313	$525	$1,838	2022Q2	27.4%	$1,673	$668	$2,341
3.0%	Engineering During Construction	$3,939	$1,574	40%	5,513	0.0%	$3,939	$1,574	$5,513	2028Q1	62.2%	$6,391	$2,553	$8,944
2.0%	Planning During Construction	$2,626	$1,049	40%	3,675	0.0%	$2,626	$1,049	$3,675	2028Q1	62.2%	$4,261	$1,702	$5,963
1.0%	Project Operations	$1,313	$525	40%	1,838	0.0%	$1,313	$525	$1,838	2022Q2	27.4%	$1,673	$668	$2,341
31	CONSTRUCTION MANAGEMENT													
5.0%	Construction Management	$6,564	$2,623	40%	9,187	0.0%	$6,564	$2,623	$9,187	2028Q1	62.2%	$10,650	$4,255	$14,905
2.0%	Project Operation:	$2,626	$1,049	40%	3,675	0.0%	$2,626	$1,049	$3,675	2028Q1	62.2%	$4,261	$1,702	$5,963
2.5%	Project Management	$3,282	$1,311	40%	4,593	0.0%	$3,282	$1,311	$4,593	2028Q1	62.2%	$5,325	$2,127	$7,452
	CONTRACT COST TOTALS:	$189,075	$72,730		**261,805**		$189,075	$72,730	$261,805			$236,376	$91,226	**$327,602**
	OMRRR	$25,194	*$10,066*	*40%*	***$35,260***									

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Puget Sound Nearshore Ecosystem Restoration
LOCATION: Washington State
This Estimate reflects the scope and schedule in report; PSNERP Feasibility Report

DISTRICT: NWS Seattle District
POC: CHIEF, COST ENGINEERING, John Dudgeon
PREPARED: 3/15/2016

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared: 3/15/2016				Program Year (Budget EC): 2016				FULLY FUNDED PROJECT ESTIMATE				
		Effective Price Level: 10/1/2015				Effective Price Level Date: 1 OCT 15								
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	North Fork													
06	FISH & WILDLIFE FACILITIES	$6,768	$2,436	36%	$9,204	0.0%	$6,768	$2,436	$9,204	2020Q2	8.4%	$7,333	$2,640	$9,973
06	MONITORING	$303	$76	25%	$379	0.0%	$303	$76	$379	2024Q1	36.8%	$415	$104	$518
06	ADAPTIVE MANANGEMENT	$1,507	$543	36%	$2,050	0.0%	$1,507	$543	$2,050	2024Q1	16.7%	$1,759	$633	$2,392
06	CHANNELS & CANALS	$1,324	$477	36%	$1,801	0.0%	$1,324	$477	$1,801	2020Q2	8.4%	$1,435	$516	$1,951
06	LEVEES & FLOODWALLS	$36,605	$13,178	36%	$49,783	0.0%	$36,605	$13,178	$49,783	2020Q2	8.4%	$39,662	$14,278	$53,941
18	CULTURAL RESOURCE PRESERVATION	$1,257	$453	36%	$1,710	0.0%	$1,257	$453	$1,710	2020Q2	8.4%	$1,362	$490	$1,852
	CONSTRUCTION ESTIMATE TOTALS:	$47,764	$17,162	36%	**$64,926**		$47,764	$17,162	$64,926			$51,965	$18,662	$70,627
01	LANDS AND DAMAGES	$11,352	$2,570	23%	$13,922	0.0%	$11,352	$2,570	$13,922	2019Q4	7.3%	$12,179	$2,757	$14,937
30	PLANNING, ENGINEERING & DESIGN													
2.5%	Project Management	$1,194	$429	36%	$1,623	0.0%	$1,194	$429	$1,623	2019Q4	15.4%	$1,378	$495	$1,873
1.0%	Planning & Environmental Compliance	$478	$172	36%	$650	0.0%	$478	$172	$650	2019Q4	15.4%	$552	$198	$750
10.0%	Engineering & Design	$4,776	$1,716	36%	$6,492	0.0%	$4,776	$1,716	$6,492	2019Q4	15.4%	$5,512	$1,981	$7,493
1.0%	Reviews, ATRs, IEPRs, VE	$478	$172	36%	$650	0.0%	$478	$172	$650	2019Q4	15.4%	$552	$198	$750
0.5%	Life Cycle Updates (cost, schedule, risks)	$239	$86	36%	$325	0.0%	$239	$86	$325	2019Q4	15.4%	$276	$99	$375
1.0%	Contracting & Reprographics	$478	$172	36%	$650	0.0%	$478	$172	$650	2019Q4	15.4%	$552	$198	$750
3.0%	Engineering During Construction	$1,433	$515	36%	$1,948	0.0%	$1,433	$515	$1,948	2020Q2	17.7%	$1,686	$606	$2,292
2.0%	Planning During Construction	$955	$343	36%	$1,298	0.0%	$955	$343	$1,298	2020Q2	17.7%	$1,124	$404	$1,528
1.0%	Project Operations	$478	$172	36%	$650	0.0%	$478	$172	$650	2019Q4	15.4%	$552	$198	$750
31	CONSTRUCTION MANAGEMENT													
5.0%	Construction Management	$2,388	$858	36%	$3,246	0.0%	$2,388	$858	$3,246	2020Q2	17.7%	$2,810	$1,010	$3,820
2.0%	Project Operation:	$955	$343	36%	$1,298	0.0%	$955	$343	$1,298	2020Q2	17.7%	$1,124	$404	$1,528
2.5%	Project Management	$1,194	$429	36%	$1,623	0.0%	$1,194	$429	$1,623	2020Q2	17.7%	$1,405	$505	$1,910
	CONTRACT COST TOTALS:	$74,162	$25,138		**$99,299**		$74,162	$25,138	$99,299			$81,666	$27,715	**$109,381**
	OMRRR	*$ 1,341.00*	*$482*	*36%*	***$1,823***									

Duckabush Causeway Replacement and Estuary Restoration

Budgetary estimate based on a FFR/EIS report- Duckabush Causeway Replacement and Estuary Restoration (#1012).

Estimated by	NWW, Cost Engineering Branch
Designed by	Seattle District
Prepared by	Anthony Rodriguez
Preparation Date	12/11/2015
Effective Date of Pricing	12/11/2015
Estimated Construction Time	480 Days

Description	Quantity	UOM	ContractCost	Escalation	Contingency	SIOH	ProjectCost
Project Cost Summary			50,358,498	0	0	0	50,358,498
Duckabush	1.00	LS	50,358,498	0	0	0	50,358,498
02 Relocations	1.00	EA	32,576,792	0	0	0	32,576,792
06 Fish and Wildlife Facilities	1.00	LS	4,601,909	0	0	0	4,601,909
06 Adaptive Management	1.00	EA	1,472,640	0	0	0	1,472,640
06 Monitoring	1.00	EA	163,000	0	0	0	163,000
08 Roads, RR & Bridges	1.00	EA	6,126,858	0	0	0	6,126,858
18 Cultural Resource Preservation	1.00	LS	1,239,009	0	0	0	1,239,009
OMRRR	1.00	LS	4,178,290	0	0	0	4,178,290

Labor ID: NLS2012 EQ ID: EP14R08 Currency in US dollars TRACES MII Version 4.3

Nooksack River Estuary
Nooksack River Estuary

BASIS OF COSTS: MII English Costbook and associated libraries, vendor pricing, and built crews.
SCOPE OF WORK: Final Feasibility Report
ESTIMATE CLASS; Conceptual, Level 4

Original Estimat: Jim Jetton, PE, CCE (NWW)
Revised Estimate: Anthony Rodriguez, CCC (NWS)

Estimated by	NWS, Cost Engineering Section
Designed by	NWS, Design Branch
Prepared by	Anthony Rodriguez, CCC
Preparation Date	3/17/2016
Effective Date of Pricing	3/17/2016
Estimated Construction Time	2,185 Days

Description	Quantity	UOM	ContractCost	Escalation	Contingency	SIOH	ProjectCost
Project Cost Summary			146,636,869	0	0	0	146,636,869
Nooksack River Estuary	1.00	LS	146,636,869	0	0	0	146,636,869
02 Relocation - Construction	1.00	EA	59,242,023	0	0	0	59,242,023
06 Fish and Wildlife Facilities	1.00	LS	10,482,385	0	0	0	10,482,385
06 Adaptive Management	1.00	EA	789,631	0	0	0	789,631
06 Monitoring	1.00	EA	564,000	0	0	0	564,000
08 Roads RR & BRidges	1.00	EA	6,521,785	0	0	0	6,521,785
09 Channels and Canals	1.00	LS	623,476	0	0	0	623,476
11 Levees and Floodwalls	1.00	LS	41,248,796	0	0	0	41,248,796
18 Cultural Resource Preservation	1.00	LS	1,971,088	0	0	0	1,971,088
OMRRR	1.00	LS	25,193,685	0	0	0	25,193,685

Labor ID: NLS2012 EQ ID: EP14R08 Currency in US dollars TRACES MII Version 4.3

North Fork Levee Setback
OBJECTIVE: Create tidal channels that will restore the area to a more natural condition.

BASIS OF COSTS: MII English Costbook and associated libraries, vendor pricing, and built crews.
SCOPE OF WORK: Final Feasibility Report
ESTIMATE CLASS; Conceptual, Level 4

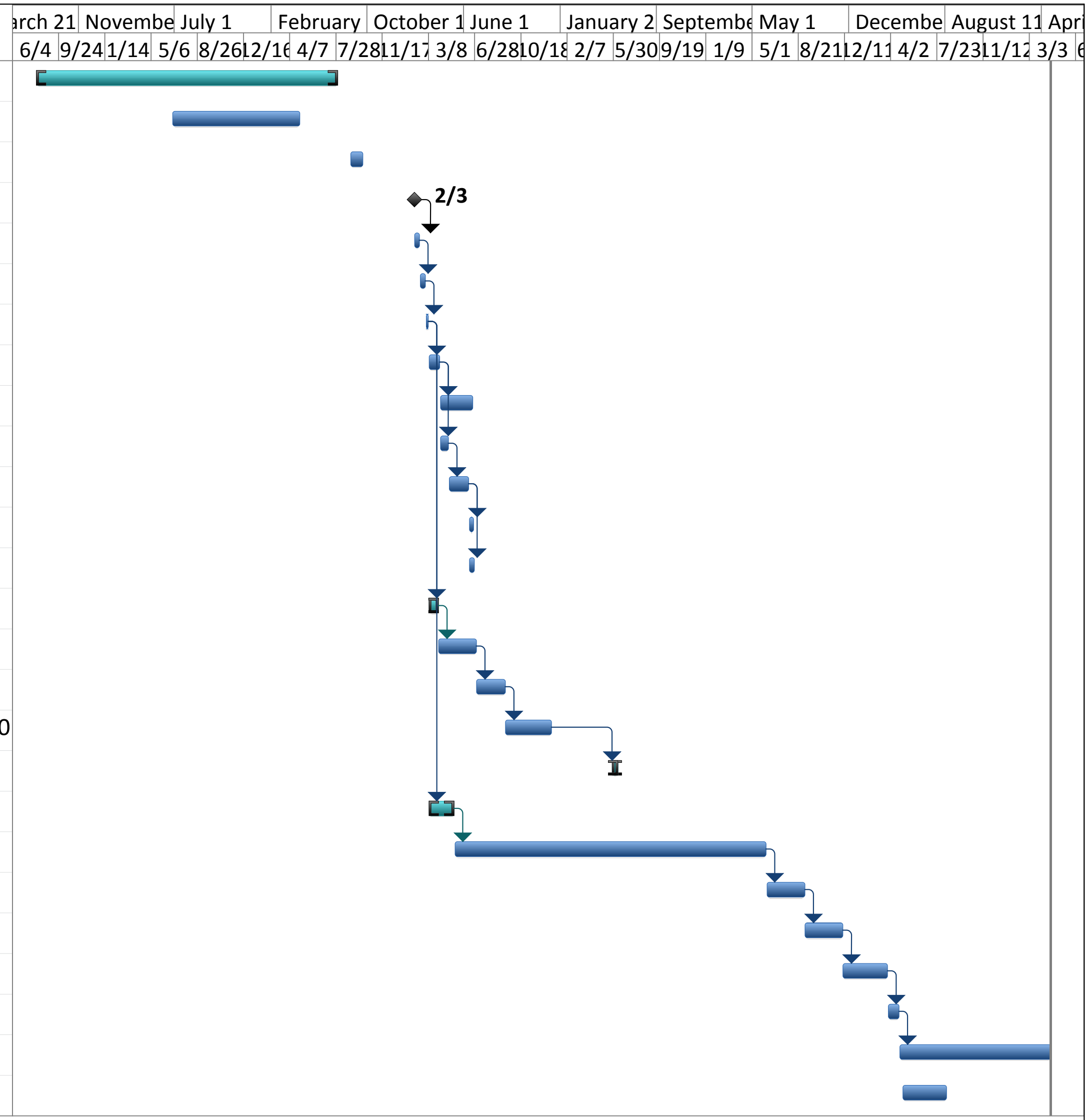
Original Estimate: Daniel Lowry, PE, CCC (NWS)
Revised Estimate: Anthony Rodriguez. CCC (NWS)

Estimated by	NWS, Cost Engineering Section
Designed by	NWS, Design Branch
Prepared by	Anthony Rodriguez, CCC
Preparation Date	3/16/2016
Effective Date of Pricing	3/16/2016
Estimated Construction Time	551 Days

Description	Quantity	UOM	ContractCost	Escalation	Contingency	SIOH	ProjectCost
Project Cost Summary			49,255,159	0	0	0	49,255,159
North Fork	1.00	LS	49,255,159	0	0	0	49,255,159
06 Fish and Wildlife Facilities	1.00	LS	6,768,044	0	0	0	6,768,044
06 Adaptive Management	1.00	LS	1,507,205	0	0	0	1,507,205
06 Monitoring	1.00	EA	451,000	0	0	0	451,000
09 Channels and Canals	1.00	LS	1,324,548	0	0	0	1,324,548
11 Levees & Floodwalls	1.00	EA	36,605,438	0	0	0	36,605,438
18 Cultural Resource Preservation	1.00	LS	1,257,015	0	0	0	1,257,015
OMRRR	1.00	EA	1,341,909	0	0	0	1,341,909

Currency in US dollars

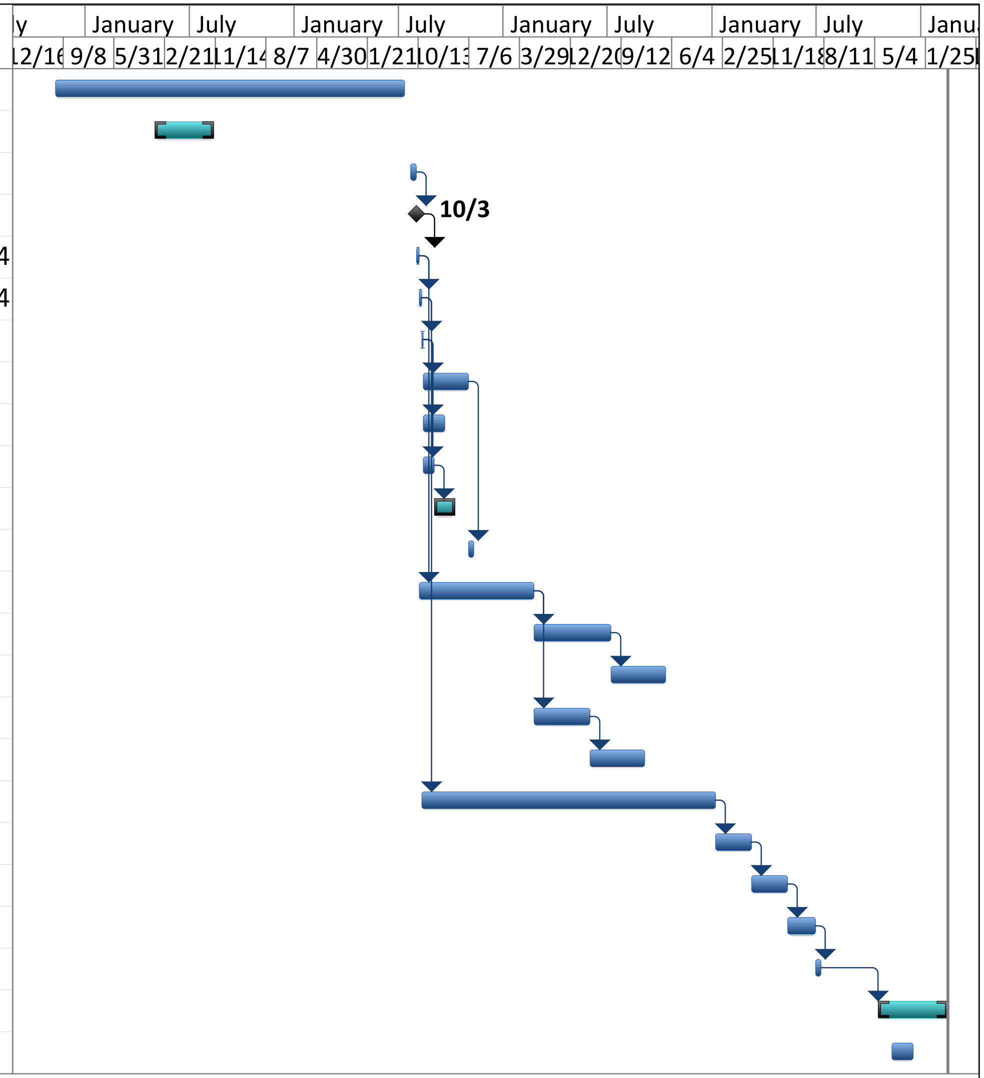
ID	Task Name	Duration	Start	Finish
1		522 days	Tue 8/1/17	Wed 7/31/19
2	PED	220 days	Wed 6/27/18	Tue 4/30/19
3	Solicitation	22 days	Sun 9/1/19	Mon 9/30/19
4	Start	0 days	Mon 2/3/20	Mon 2/3/20
5	Mobilize	10 days	Mon 2/3/20	Fri 2/14/20
6	Erosion Control	10 days	Mon 2/17/20	Fri 2/28/20
7	Clear and Grub	5 days	Mon 3/2/20	Fri 3/6/20
8	Excavation	20 days	Mon 3/9/20	Fri 4/3/20
9	Fill Placement	56 days	Mon 4/6/20	Mon 6/22/20
10	Large Channel Creation	15 days	Mon 4/6/20	Fri 4/24/20
11	Small Channel Creation	35 days	Mon 4/27/20	Fri 6/12/20
12	LWD Placement	8 days	Mon 6/15/20	Wed 6/24/20
13	Plantings	10 days	Mon 6/15/20	Fri 6/26/20
14	Temp Highway Road	17 days	Mon 3/9/20	Tue 3/31/20
15	Road Demo	66 days	Wed 4/1/20	Wed 7/1/20
16	New Pavement	50 days	Thu 7/2/20	Wed 9/9/20
17	New Utilities	80 days	Thu 9/10/20	Wed 12/30/20
18	Demo Utilties	10 days	Thu 5/27/21	Wed 6/9/21
19	Temp Construction Pad	45 days	Mon 3/9/20	Fri 5/8/20
20	High 101 Bridge Construction	540 days	Mon 5/11/20	Fri 6/3/22
21	Duckabush Road Bridge	66 days	Mon 6/6/22	Mon 9/5/22
22	Shorewood Road Bridge	66 days	Tue 9/6/22	Tue 12/6/22
23	Bridge Demo	78 days	Wed 12/7/22	Fri 3/24/23
24	Demobilize	20 days	Mon 3/27/23	Fri 4/21/23
25	Post Construction Monitoring	262 days	Mon 4/24/23	Tue 4/23/24
26	Adaptive Management	76 days	Mon 5/1/23	Mon 8/14/23



Project: Duckabush
Date: Tue 3/22/16

Task	External Milestone	Manual Summary Rollup	
Split	Inactive Task	Manual Summary	
Milestone	Inactive Milestone	Start-only	
Summary	Inactive Summary	Finish-only	
Project Summary	Manual Task	Deadline	
External Tasks	Duration-only	Progress	

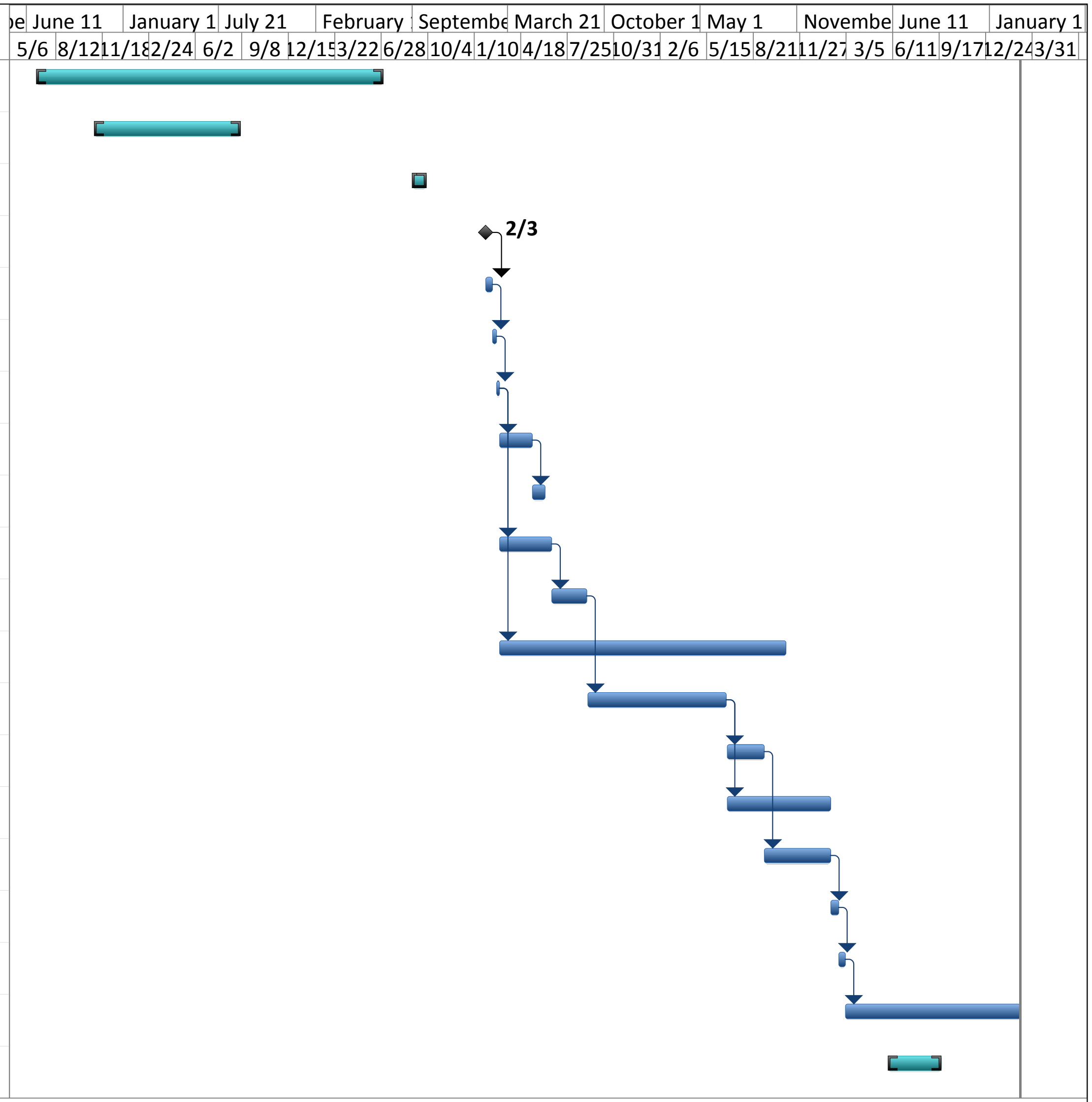
ID	Task Name	Duration	Start	Finish
1	PED	1309 days	Mon 7/29/19	Thu 8/1/24
2	Cultural Survey	220 days	Fri 1/1/21	Thu 11/4/21
3	Solicitation	22 days	Mon 9/2/24	Tue 10/1/24
4	Start	0 days	Thu 10/3/24	Thu 10/3/24
5	Mobilize	10 days	Thu 10/3/24	Wed 10/16/24
6	Erosion Control	10 days	Thu 10/17/24	Wed 10/30/24
7	Clear and Grub	5 days	Thu 10/31/24	Wed 11/6/24
8	Structure Demo	170 days	Thu 11/7/24	Wed 7/2/25
9	LWD Installation	82 days	Thu 11/7/24	Fri 2/28/25
10	Water Control Structure	42 days	Thu 11/7/24	Fri 1/3/25
11	Stormwater Pond	78 days	Mon 1/6/25	Wed 4/23/25
12	Lummi River Regrade	20 days	Thu 7/3/25	Wed 7/30/25
13	New Levees	430 days	Thu 10/17/24	Wed 6/10/26
14	New Utilities	288 days	Thu 6/11/26	Mon 7/19/27
15	Demo Utilties	205 days	Tue 7/20/27	Mon 5/1/28
16	New Pavement	210 days	Thu 6/11/26	Wed 3/31/27
17	Demo Pavement	205 days	Thu 4/1/27	Wed 1/12/28
18	Bridge Construction	1100 days	Thu 10/31/24	Wed 1/17/29
19	Bridge Demo	135 days	Thu 1/18/29	Wed 7/25/29
20	Remove Levee	135 days	Thu 7/26/29	Wed 1/30/30
21	Plantings	105 days	Thu 1/31/30	Wed 6/26/30
22	Demobilize	20 days	Thu 6/27/30	Wed 7/24/30
23	Post Construction Monitoring	262 days	Thu 5/22/31	Fri 5/21/32
24	Adaptive Management	80 days	Fri 8/1/31	Thu 11/20/31



Project: Nooksack
Date: Tue 3/22/16

Task		External Milestone		Manual Summary Rollup	
Split		Inactive Task		Manual Summary	
Milestone		Inactive Milestone		Start-only	
Summary		Inactive Summary		Finish-only	
Project Summary		Manual Task		Deadline	
External Tasks		Duration-only		Progress	

ID	Task Name	Duration	Start	Finish
1	PED	523 days	Sun 7/1/18	Wed 7/1/20
2	Cultural Survey	220 days	Thu 11/1/18	Wed 9/4/19
3	Solicitation	22 days	Tue 9/1/20	Wed 9/30/20
4	Start	0 days	Wed 2/3/21	Wed 2/3/21
5	Mobilize	10 days	Wed 2/3/21	Tue 2/16/21
6	Erosion Control	7 days	Wed 2/17/21	Thu 2/25/21
7	Clear and Grub	4 days	Fri 2/26/21	Wed 3/3/21
8	Building Demo	49 days	Thu 3/4/21	Tue 5/11/21
9	Demo Boat Ramp	19 days	Wed 5/12/21	Mon 6/7/21
10	Demo Paving	78 days	Thu 3/4/21	Mon 6/21/21
11	Demo Utilties	54 days	Tue 6/22/21	Fri 9/3/21
12	Traffic Control	432 days	Thu 3/4/21	Fri 10/28/22
13	New Levee	210 days	Mon 9/6/21	Fri 6/24/22
14	Channel Creation	56 days	Mon 6/27/22	Mon 9/12/22
15	Plantings	156 days	Mon 6/27/22	Mon 1/30/23
16	Remove Levee	100 days	Tue 9/13/22	Mon 1/30/23
17	Breach Dike	13 days	Tue 1/31/23	Thu 2/16/23
18	Demobilize	10 days	Fri 2/17/23	Thu 3/2/23
19	Post Construction Monitoring	263 days	Fri 3/3/23	Tue 3/5/24
20	Adaptive Management	80 days	Thu 6/1/23	Wed 9/20/23



Project: North Fork Schedule
Date: Tue 3/22/16

Task		External Milestone		Manual Summary Rollup	
Split		Inactive Task		Manual Summary	
Milestone		Inactive Milestone		Start-only	
Summary		Inactive Summary		Finish-only	
Project Summary		Manual Task		Deadline	
External Tasks		Duration-only		Progress	

US Army Corps
of Engineers®

COST AND SCHEDULE RISK ANALYSIS REPORT

FOR

Puget Sound Nearshore Ecosystem Restoration Project

Duckabush River Estuary, WA

Prepared for:

Seattle District, Seattle, WA

Prepared by:

USACE, Seattle District

Date: June 08, 2016

EXECUTIVE SUMMARY

The US Army Corps of Engineers (USACE), District, presents this cost and schedule risk analysis (CSRA) report regarding the risk findings and recommended contingencies for the Puget Sound Nearshore Environmental Restoration Project (PSNERP). In compliance with Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008, a *Monte-Carlo* based risk analysis was conducted by the Project Development Team (PDT) on remaining costs. The purpose of this risk analysis study is to present the cost and schedule risks considered, those determined and respective project contingencies at a recommended 80% confidence level of successful execution to project completion.

The purpose of this project is to address and restore ecosystems through restoration of natural processes and restoration and/or re-creation of coastal wetlands and embayments. There are three sites in the recommended plan with locations throughout the Puget Sound. Restoration actions will occur near the mouth of the Duckabush River, Skagit River and Nooksack Rivers, respectively. More specifically, this CSRA report will cover in detail the Duckabush River Estuary

The proposed action to the Duckabush River Estuary would restore the natural geomorphology by removing major roadway obstructions, excavating channels, and removing fill. The action would remove two bridges that total 970 LF and associated approach road embankments along Highway 101. This will be followed by realigning Highway 101 onto a new 2100 LF bridge across the estuarine delta to restore tidal connection to the estuary. A surface street crossing (Shorewood Road) would be modified to tie into the new bridge structure. Adjacent fill at a distributary channel (Pierce Slough) would be removed. Multiple tidally influenced distributary river channels summing to 1,155 LF would be reestablished, and blind tidal channels summing to 4,200 LF would be excavated within the marsh areas.

Specific to the Duckabush River Estuary, the current base cost estimate approximates $62M, including planning, engineering, design & construction management costs, but excluding contingency and escalation. The CSRA is calculated on the estimated remaining construction base cost of $46M expressed in FY2016 dollars. The CSRA base cost excluded lands, damages and PL 91-646 relocation costs of $491K, escalation and contingencies. Since the Real Estate office provided a separate 13% contingency for its real estate and PL 91-646 relocations requirements, Cost Engineering performed the CSRA on the estimated remaining construction costs. Based on the results of the risk analysis, Cost Engineering recommends a contingency value of $21M on the remaining construction work or approximately 46% of base project cost. The same 46% was then applied to engineering and design and construction management.

Cost estimates fluctuate over time. During this period of study, minor cost fluctuations can and have occurred. For this reason, contingency reporting is based on cost and percent values. Should cost vary to a slight degree with similar scope and risks, contingency percent values will be reported, cost values rounded.

Table ES-1. Construction Contingency Results

Base Case Construction Cost Estimate	**$46,200,000**	
Confidence Level	**Construction Value ($$) w/ Contingencies**	**Contingency (%)**
50%	$64,200,000	39%
80%	**$67,400,000**	**46%**
90%	$69,200,000	49%

KEY FINDINGS/OBSERVATIONS RECOMMENDATIONS

The PDT worked through the risk register on three separate occasions: June 2012, July 2014 and again in December 2015. That period of time allowed improved project scope definition, investigations, design and cost information, and resulted in revised risks in certain project areas. The key risk drivers identified through sensitivity analysis suggest a cost contingency of $17.8M and schedule risks adding another potential of $3.4M, both at an 80% confidence level.

A key risk, the Bridge Foundation remains uncertain due to a limited design and a lack of soil analysis. Concern has been mitigated by increasing the foundation at the bridge abutments. The current design includes two pile caps, four piers and four caissons at each abutment to mitigate any potential soil liquefaction. The PDT believe this approach is conservative. However it is very likely that the design of the bridge abutments will change as the project moves further in design.

Another key risk, the bridge cost model, remains uncertain. Since the design of the bridge is limited in scope and detail there is potential for the bridge design to change at the next phase of design. Additionally, many assumptions were made in order to estimate a cost for this feature of work. To mitigate this risk the PDT assumed a cost increase of 30% on the bridge cost model.

The last key risk regarding project funding, remains uncertain. This project will be competing with other projects on a national level for both funding and personnel resources. Factor in the large project cost and it is very likely that this project will be funded on a yearly basis and not one large lump sum. There is the risk that any delay in funding could impact the project resourcing and sequencing. Reducing funding leads to breaking down work packages to be executed in smaller outlay increments.

Cost Risks: From the CSRA, the key or greater Cost Risk items in terms of cost variability potential include:

- TL-5: Bridge & Foundation Placement – Several elements of the bridge scope are likely to change including bridge & foundation geometry.
- EST-3: Bridge Cost Model – The cost book items and construction assumptions may not be entirely accurate or applicable to the project. It is likely that there will be changes to the bridge cost model as the design progresses.

Moderate risks, when combined, can also become a cost impact.

- CA-1: Small Business Markups – The cost estimate assumes that all work will be done by a small business contractor. However, if the contract solicitation was restricted further such as 8A, HUBZONE, SDVOSB or Woman-Owned the expected home office overhead markup rate would likely be higher than what is already assumed in the cost estimate.
- TL-1: Earthwork Design Requirements – Project is in preliminary planning stages and no grading details are known. There is limited knowledge about what road embankment prisms will eventually be required.
- CON-1: Post-Award Contract Modifications – At this early state of design there is the likely risk that unsuitable soil conditions could be encountered once construction has commenced. This most certainly would result in a modification to the contract and could have impacts on cost as well as schedule.
- CON-3 Production Rate and Schedule – The cost estimate assumes that construction will take place in fair weather conditions. Conservative production rates were used throughout the cost estimate however it is impossible to predict the crew sizes the contractor will have on site or the actual production rates given the site constraints.

Schedule Risks: The schedule risk indicates some uncertainty of key risk items, time duration growth that can translate into added costs. Over time, risks increase on those out-year contracts where there is greater potential for change in new scope requirements, uncertain market conditions, and unexpected high inflation. The greatest risk is:

- PPM-5: Project Funding – Federal and non-Federal parties competing with other projects/programs for both funding and staff resources.

Moderate risks, when combined, can also become a time and resulting cost impact.

- EST-3: Bridge Cost Model (Estimate Risk Aggregate) – The assumed crew sizes and production rates to construct the bridge may be low, resulting in a longer construction duration.
- PR-3: Project Opposition – A pending lawsuit or failure to acquire land has the potential to stop the project and will require a project reevaluation.
- PPM-1: Project Scheduling – High volume of projects under the PSNERP authorization may present issues in terms of resource allocation and quality control.
- LD-1: Unknown Utilities: - Actual utility removal, decommissioning or relocation quantities will be determined during PED.
- RE-1: Cultural Resources – If cultural artifacts are uncovered during construction this will stop construction in the identified area.

Recommendations: Timely coordination and risk resolution between the Sponsor and USACE is needed in areas of project funding, bridge design and assumptions, earthwork design and production rates and schedules. The PDT must include the recommended cost and schedule contingencies and incorporate risk monitoring and mitigation on those identified risks. Further iterative study and update of the risk analysis throughout the project life-cycle is important in support of the remaining project work within an approved budget and appropriation.

US Army Corps
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COST AND SCHEDULE RISK ANALYSIS REPORT

FOR

Puget Sound Nearshore Ecosystem Restoration Project

Nooksack River Delta, WA

Prepared for:

Seattle District, Seattle, WA

Prepared by:

USACE, Seattle District

Date: June 08, 2016

EXECUTIVE SUMMARY

The US Army Corps of Engineers (USACE), District, presents this cost and schedule risk analysis (CSRA) report regarding the risk findings and recommended contingencies for the Puget Sound Nearshore Environmental Restoration Project (PSNERP). In compliance with Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008, a *Monte-Carlo* based risk analysis was conducted by the Project Development Team (PDT) on remaining costs. The purpose of this risk analysis study is to present the cost and schedule risks considered, those determined and respective project contingencies at a recommended 80% confidence level of successful execution to project completion.

The purpose of this project is to address and restore ecosystems through restoration of natural processes and restoration and/or re-creation of coastal wetlands and embayments. There are three sites in the recommended plan with locations throughout the Puget Sound. Restoration actions will occur near the mouth of the Duckabush River, Skagit River and Nooksack Rivers, respectively. More specifically, this CSRA report will cover in detail the Nooksack River Delta

The proposed action to the Nooksack River Delta would restore riverine and tidal flow as well as sediment transport and delivery processes throughout a substantial portion of the historical Nooksack River delta. This will be accomplished by a combination of construction features that would modify levees, roads, and other hydrological barriers. Construction of approximately 47,000 LF of new setback levees would provide flood risk management protection for active businesses, residences, farms, transportation infrastructure, and Lummi Nation lands in the project area. Features of work associated with levee setback construction include but are not limited to: 12,280 LF of levee removal, installation of 3 log jam structures, installation of 1 new water control structure and 9,980 LF of channel and berm regarding. Construction on existing infrastructure includes but is not limited to: removal of 6 bridges and approach road section and construction of 6 new bridges and approach road sections.

Specific to the Nooksack River Delta, the current base cost estimate approximates $189M, including planning, engineering, design & construction management costs, but excluding contingency and escalation. The CSRA is calculated on the estimated remaining construction base cost of $131M expressed in FY2016 dollars. The CSRA base cost excluded lands, damages and PL 91-646 relocation costs of $13.8M. Since the Real Estate office provided a separate 20% contingency for its real estate and PL 91-646 relocations requirements, Cost Engineering performed the CSRA on the estimated remaining construction costs. Based on the results of the risk analysis, Cost Engineering recommends a contingency value of $52M on the remaining construction work or approximately 40% of base project cost. The same 40% was then applied to engineering and design and construction management.

Cost estimates fluctuate over time. During this period of study, minor cost fluctuations can and have occurred. For this reason, contingency reporting is based on cost and percent values. Should cost vary to a slight degree with similar scope and risks, contingency percent values will be reported, cost values rounded.

Table ES-1. Construction Contingency Results

Base Case Construction Cost Estimate	**$131,300,000**	
Confidence Level	**Construction Value ($$) w/ Contingencies**	**Contingency (%)**
50%	$161,700,000	33%
80%	**$169,400,000**	**40%**
90%	$174,200,000	44%

KEY FINDINGS/OBSERVATIONS RECOMMENDATIONS

The PDT worked through the risk register on three separate occasions: June 2012, July 2014 and again in December 2015. That period of time allowed improved project scope definition, investigations, design and cost information, and resulted in revised risks in certain project areas. The key risk drivers identified through sensitivity analysis suggest a cost contingency of $43.5M and schedule risks adding another potential of $4.7M, both at an 80% confidence level.

A key risk, the bridge cost model, remains uncertain. Since the design of the bridge is limited in scope and detail there is potential for the bridge design to change at the next phase of design. Additionally, many assumptions were made in order to estimate a cost for this feature of work. To mitigate this risk the PDT assumed a cost increase of 30% on the bridge cost model.

Another key risk, bid competition, remains uncertain as well. This project is assumed to be solicited under one large contract. The number of prime contractors who can meet the bonding requirements will be limited. In addition, the current construction market in the Seattle area is very active. As a result of the market being saturated with work, contractors will not be as aggressive when trying to acquire new work. This risk will be reevaluated to as the project gets closer to solicitation.

Cost Risks: From the CSRA, the key or greater Cost Risk items in terms of cost variability potential include:

- CA-3: Bid Competition – If the market is saturated with work, contractors will place higher margins on the work they bid.
- EST-3: Bridge Cost Model – The cost book items and construction assumptions may not be entirely accurate or applicable to the project. It is likely that there will be changes to the bridge cost model as the design progresses.

Moderate risks, when combined, can also become a cost impact.

- CA-1: Small Business Markups – The cost estimate assumes that all work will be done by a small business contractor. However, if the contract solicitation was restricted further such as 8A, HUBZONE, SDVOSB or Woman-Owned the expected home office overhead markup rate would likely be higher than what is already assumed in the cost estimate.
- TL-1: Earthwork Design Requirements – Project is in preliminary planning stages and no grading details are known. There is limited knowledge about what road embankment prisms will eventually be required.
- TL-5: Bridge & Foundation Placement – Several elements of the bridge scope are likely to change including bridge & foundation geometry.
- CON-1: Post-Award Contract Modifications – At this early state of design there is the likely risk that unsuitable soil conditions could be encountered once construction has commenced. This most certainly would result in a modification to the contract and could have impacts on cost as well as schedule.
- CON-3 Production Rate and Schedule – The cost estimate assumes that construction will take place in fair weather conditions. Conservative production rates were used throughout the cost estimate however it is impossible to predict the crew sizes the contractor will have on site or the actual production rates given the site constraints.

Schedule Risks: The schedule risk indicates some uncertainty of key risk items, time duration growth that can translate into added costs. Over time, risks increase on those out-year contracts where there is greater potential for change in new scope requirements, uncertain market conditions, and unexpected high inflation. The greatest risk is:

- EST-1, 3 & 4: Estimate Risk Aggregate – The estimate risk aggregate corresponds to schedule risk impact as a result of inaccurate assumptions made in the cost estimate on risks such as: changes in soil conditions, bridge cost model or flooding and work windows. The likelihood of these risk impacts happening sequentially is very slim. Most likely if two or more estimate risks did occur there will be some concurrent work overlap, which is why a reasonable aggregate duration was used to capture the “most likely” worst case schedule impact should all three risks occur during the project duration. Of the three estimate risks the schedule impact from the bridge cost model is the greatest.

There is high uncertainty in the crew sizes and production rates used by the contractor.

Moderate risks, when combined, can also become a time and resulting cost impact.

- TL-2, 5, 7, 8 & 9: Technical Risk Aggregate – The technical risk aggregate corresponds to schedule risk impact as a result of uncertainty on risks such as: hydraulic structures, bridge & foundation placement, levee/road settlement, borrow source and storm water ponds. The likelihood of these risk impacts happening sequentially is very slim. Most likely if two or more technical risks did occur there will be some concurrent work overlap, which is why a reasonable aggregate duration was used to capture the "most likely" worst case schedule impact should all five risks occur during the project duration. Of the five technical risks the schedule impact from the bridge & foundation placement is the greatest. The design of the bridge foundations is very likely to change as the design progresses.
- LD-1: Unknown Utilities: - Actual utility removal, decommissioning or relocation quantities will be determined during PED.
- LD-2: Bridge/Structure Removal – There are 34 structures to be removed and additional structures could be present. The information on these structures is limited and the scope could increase as the design progresses.
- CON-1 & 2: Construction Risk Aggregate – The construction risk aggregate corresponds to schedule risk impact as a result of uncertainty on risks such as: post-award contract modifications and production rate and schedule. The likelihood of these risk impacts happening sequentially is very slim. Most likely if both construction risks did occur there will be some concurrent work overlap, which is why a reasonable aggregate duration was used to capture the "most likely" worst case schedule impact should all both risks occur during the project duration. Of the two technical risks the schedule impact from the post-award contract modifications is the greatest.
- PR-3: Project Opposition – A pending lawsuit or failure to acquire land has the potential to stop the project and will require a project reevaluation.

Recommendations: Timely coordination and risk resolution between the Sponsor and USACE is needed in areas of project funding, bridge design and assumptions, earthwork design and production rates and schedules. The PDT must include the recommended cost and schedule contingencies and incorporate risk monitoring and mitigation on those identified risks. Further iterative study and update of the risk analysis throughout the project life-cycle is important in support of the remaining project work within an approved budget and appropriation.

US Army Corps
of Engineers®

COST AND SCHEDULE RISK ANALYSIS REPORT

FOR

Puget Sound Nearshore Ecosystem Restoration Project

North Fork Skagit River Delta, WA

Prepared for:

Seattle District, Seattle, WA

Prepared by:

USACE, Seattle District

Date: June 08, 2016

EXECUTIVE SUMMARY

The US Army Corps of Engineers (USACE), District, presents this cost and schedule risk analysis (CSRA) report regarding the risk findings and recommended contingencies for the Puget Sound Nearshore Environmental Restoration Project (PSNERP). In compliance with Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008, a *Monte-Carlo* based risk analysis was conducted by the Project Development Team (PDT) on remaining costs. The purpose of this risk analysis study is to present the cost and schedule risks considered, those determined and respective project contingencies at a recommended 80% confidence level of successful execution to project completion.

The purpose of this project is to address and restore ecosystems through restoration of natural processes and restoration and/or re-creation of coastal wetlands and embayments. There are three sites in the recommended plan with locations throughout the Puget Sound. Restoration actions will occur near the mouth of the Duckabush River, Skagit River and Nooksack Rivers, respectively. More specifically, this CSRA report will cover in detail the North Fork Skagit River Delta

The proposed action to the North Fork Skagit River Delta would restore the riverine floodplain and tidal connectivity along the lower reach of the North Fork of the Skagit River. This will require constructing a new 12,300 LF flood risk management levee further inland. The 15,700 LF existing levee would be lowered and selectively breached in 4 locations to allow inundation of the estuarine emergent marsh and sustain back channel habitat. Forested floodplain habitat would be created along the lowered levee adjacent to the mainstem river channel. This will be done by excavating and creating 39,200 LF of tidal channels, removing 17 buildings in the flood plain, removing 344,300 SF of existing road and plant riparian vegetation over approximately 50 acres.

Specific to the North Fork Skagit River Delta, the current base cost estimate approximates $74M, including planning, engineering, design & construction management costs, but excluding contingency and escalation. The CSRA is calculated on the estimated remaining construction base cost of $48M expressed in FY2016 dollars. The CSRA base cost excluded lands, damages and PL 91-646 relocation costs of $11.4M. Since the Real Estate office provided a separate 23% contingency for its real estate and PL 91-646 relocations requirements, Cost Engineering performed the CSRA on the estimated remaining construction costs. Based on the results of the risk analysis, Cost Engineering recommends a contingency value of $17M on the remaining construction work or approximately 36% of base project cost. The same 36% was then applied to engineering and design and construction management.

Cost estimates fluctuate over time. During this period of study, minor cost fluctuations can and have occurred. For this reason, contingency reporting is based on cost and percent values. Should cost vary to a slight degree with similar scope and risks, contingency percent values will be reported, cost values rounded.

Table ES-1. Construction Contingency Results

Base Case Construction Cost Estimate	**$47,800,000**	
Confidence Level	**Construction Value ($$) w/ Contingencies**	**Contingency (%)**
50%	$62,400,000	30%
80%	**$65,000,000**	**36%**
90%	$66,600,000	39%

KEY FINDINGS/OBSERVATIONS RECOMMENDATIONS

The PDT worked through the risk register on three separate occasions: June 2012, July 2014 and again in November 2015. That period of time allowed improved project scope definition, investigations, design and cost information, and resulted in revised risks in certain project areas. The key risk drivers identified through sensitivity analysis suggest a cost contingency of $13.3M and schedule risks adding another potential of $3.9M, both at an 80% confidence level.

A key risk, bid competition, remains uncertain. This project is assumed to be solicited under one large contract. The number of prime contractors who can meet the bonding requirements will be limited. In addition, the current construction market in the Seattle area is very active. As a result of the market being saturated with work, contractors will not be as aggressive when trying to acquire new work. This risk will be reevaluated to as the project gets closer to solicitation.

The last key risk regarding project funding, remains uncertain. This project will be competing with other projects on a national level for both funding and personnel resources. Factor in the large project cost and it is very likely that this project will be funded on a yearly basis and not one large lump sum. There is the risk that any delay in funding could impact the project resourcing and sequencing. Reducing funding leads to breaking down work packages to be executed in smaller outlay increments.

Cost Risks: From the CSRA, the key or greater Cost Risk items in terms of cost variability potential include:

- CA-1: Small Business Markups – The cost estimate assumes that all work will be done by a small business contractor. However, if the contract solicitation was restricted further such as 8A, HUBZONE, SDVOSB or Woman-Owned the expected home office overhead markup rate would likely be higher than what is already assumed in the cost estimate.
- CA-3: Bid Competition – If the market is saturated with work, contractors will place higher margins on the work they bid.

Moderate risks, when combined, can also become a cost impact.

- TL-5: Levee/Road Settlement – The current design allows 1 foot of settlement. This amount will be re-examined in PED. If settlement occurs in excess of the amount anticipated, then additional materials would very likely be required.
- TL-6: Borrow Site – The MII estimate assumes that offsite disposal and borrow sites are available within 30 miles. However, it is likely that multiple sources will be needed, and, consequently, the haul distance may increase.
- LD-1: Unknown Utilities: - Actual utility removal, decommissioning or relocation quantities will be determined during PED.
- CON-1: Post-Award Contract Modifications – At this early state of design there is the likely risk that unsuitable soil conditions could be encountered once construction has commenced. This most certainly would result in a modification to the contract and could have impacts on cost as well as schedule.
- EST-2 Large Woody Debris & Plantings – The scope is very limited and quantities are broad. The 2012 design quantities for large woody debris and plantings were reviewed and updated. The current design quantities are expected to change again when this project enters PED.

Schedule Risks: The schedule risk indicates some uncertainty of key risk items, time duration growth that can translate into added costs. Over time, risks increase on those out-year contracts where there is greater potential for change in new scope requirements, uncertain market conditions, and unexpected high inflation. The greatest risk is:

- PPM-5: Project Funding – Federal and non-Federal parties competing with other projects/programs for both funding and staff resources.

Moderate risks, when combined, can also become a time and resulting cost impact.

- PPM-1: Project Scheduling – High volume of projects under the PSNERP authorization may present issues in terms of resource allocation and quality control.

- TL-2, 4, 5, 6 & 7: Technical Risk Aggregate – The technical risk aggregate corresponds to schedule risk impact as a result of uncertainty on risks such as: hydraulic structures, sea level change, additional armoring, levee/road settlement, borrow site and storm water ponds. The likelihood of these risk impacts happening sequentially is very slim. Most likely if two or more technical risks did occur there will be some concurrent work overlap, which is why a reasonable aggregate duration was used to capture the "most likely" worst case schedule impact should all five risks occur during the project duration. Of the five technical risks the schedule impact from additional armoring and storm water ponds is the greatest. The scope and magnitude for both risks is likely to change with the potential of additional armoring and additional storm water ponds.
- CON-1 & 2: Construction Risk Aggregate – The construction risk aggregate corresponds to schedule risk impact as a result of uncertainty on risks such as: post-award contract modifications and care and diversion of water. The likelihood of these risk impacts happening sequentially is very slim. Most likely if both construction risks did occur there will be some concurrent work overlap, which is why a reasonable aggregate duration was used to capture the "most likely" worst case schedule impact should all both risks occur during the project duration. Of the two technical risks the schedule impact from the post-award contract modifications is the greatest.
- EST-1, & 3: Estimate Risk Aggregate – The estimate risk aggregate corresponds to schedule risk impact as a result of inaccurate assumptions made in the cost estimate on risks such as: changes in soil conditions and flooding and work windows. The likelihood of these risk impacts happening sequentially is very slim. Most likely if both estimate risks did occur there will be some concurrent work overlap, which is why a reasonable aggregate duration was used to capture the "most likely" worst case schedule impact should both risks occur during the project duration. Of the two estimate risks the schedule impact flooding and work windows is the greatest because this risk has the potential to stop construction.

Recommendations: Timely coordination and risk resolution between the Sponsor and USACE is needed in areas of project funding, bridge design and assumptions, earthwork design and production rates and schedules. The PDT must include the recommended cost and schedule contingencies and incorporate risk monitoring and mitigation on those identified risks. Further iterative study and update of the risk analysis throughout the project life-cycle is important in support of the remaining project work within an approved budget and appropriation.

Attachment B – Applied Geomorphology Guidelines and Hierarchy of Openings

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Puget Sound Nearshore Ecosystem Restoration Project

Strategic Restoration Conceptual Engineering – Design Report

Applied Geomorphology Guidelines and Hierarchy of Openings

March 2011

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Applied Geomorphology Guidelines – Revised Draft Phase 2 Document
PSNERP Conceptual Design

December 20 2010

These Applied Geomorphology Guidelines will be used by the ESA team's conceptual designs. The Guidelines will be used as needed in the designs and to aid in quality control review. These guidelines may be revised to account for lessons learned during Phase 2 and for subsequent use.

The guidelines are intended only for conceptual design by the PSNERP team. These guidelines are established partly to provide a means of developing uniform designs, for quality control and precision, but also to facilitating future refinements. Further research and data collection are required to develop guidelines for broader application.

These guidelines use empirical models calibrated with data collected from field sites. Therefore, these guidelines are most useful when the site parameters lie within the range of the calibration data. Parameters include tide range, sediment and vegetation, fluvial effects, salinity (which affects plant types and geomorphology), and in some cases wave and littoral climate. Comprehensive data sets are not presently available for Puget Sound. The guidelines are based on both local data sets and data sets from other locations, with some adjustments, primarily for tide range. Therefore, the accuracy of the regressions provided here can be considered approximate. Historic data from the site (e.g. channel width from a T-Sheet) or nearby reference data (e.g. Hood's data for the Skagit, Barnard's data for Discovery Bay marshes) may be used instead of these guidelines.

The Guidelines are organized as follows:

1. Tides: Tide design parameters are identified for NOS tide stations selected to represent the varying tides in Puget Sound. Tide ranges are tabulated. Tidal datum conversion from Mean Lower Low Water (MLLW) to North American Vertical Datum (NAVD88) are provided at each tide station.

2. Tidal Marsh Channels: Regression lines and graphs are provided to relate channel geometry (channel cross-sectional area, width and depth) to marsh area and tidal prism. A set of regressions and graphs are provided for each tide station identified in (1), based on the tide range. A procedure is provided to estimate channel geometry with combined tidal and stream discharge.

3. Tidally-Influenced Fluvial Channels: Guidance for tidally influenced fluvial channels is to use historic data, remnant channel geometry and available published data on a site-specific basis.

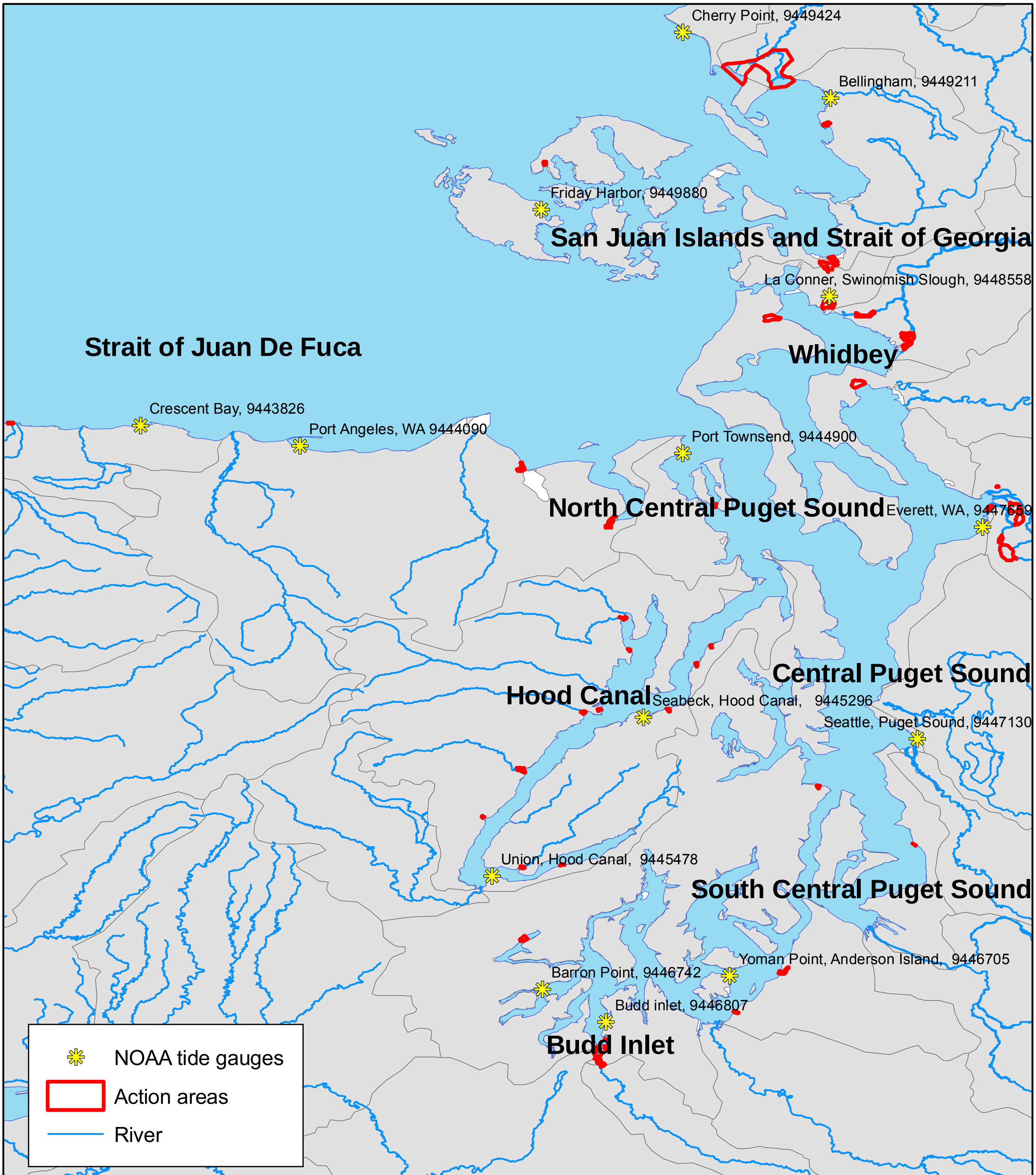
4. Tidal Inlets: A set of graphs are provided for tidal inlets where wave action and littoral drift affect the channel geometry and, in particular, limit the tide range. The graphs allow prediction of the tidal prism necessary for an open inlet and the size of the inlet cross section for a given tidal prism.

5. Beach Geometry: Guidance is provided to estimate the berm elevation of coarse sediment beaches.

1. Tides:

The Puget Sound Nearshore Ecosystem Restoration Project (PSNERP) has defined sub basins of Puget Sound (Figure 1). Tide stations have been selected to characterize the tidal regime for each sub basin. Since the tides vary along each arm of Puget Sound (for example, the tide is amplified with distance south through Hood Canal), several tide stations are indentified for each sub basin, as shown in Figure 1. Table 1 lists the tide stations and their tidal datums published by the National Ocean Service (NOS). A conversion between tidal datums and the project vertical datum (NAVD88) are provided. The conversions are those published by the NOS or are based on a review of the tidal benchmarks and provided by Pacific Surveying and Engineering (PSE) and ESA PWA as part of this project. The sources of the conversion and level of confidence are provided.

Each action should define the tidal datums and NAVD conversion used and the sources.



Source: NOAA

Note: Subregion names are bold with NOAA tide gauge reference number.

figure X

PSNERP

NOAA Tide Gauge Locations

PWA Ref# -

PWA

G:\2036_PSNERP\mxds\base_map

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Table 1: Tides Stations, tidal datums and NAVD conversions.

2036.01 Puget Sound
D.Kunz (PWA)
11/22/2010
Modfied B.Battalio, 12/9/2010
Puget Sound Tide Gages

				ft MLLW							ft NAVD							
Station	**NOS #**	**Start of Record**	**End of Record**	**MLLW**	**MLW**	**MSL**	**MTL**	**MHW**	**MHHW**	**MLLW to NAVD Conversion (ft)**	**MLLW**	**MLW**	**MSL**	**MTL**	**MHW**	**MHHW**	**Source of Datum Conversion**	**Conversion Level of Confidence***
Cherry Point	9449424	Nov-71	present	0.00	2.61	5.28	5.47	8.32	9.15	-0.96	-0.96	1.65	4.32	4.51	7.36	8.19	PSE Verrified	2
Bellingham	9449211	Mar-73	Jul-75	0.00	2.35	4.95	5.07	7.79	8.51	-0.48	-0.48	1.87	4.47	4.59	7.31	8.03	NOS Tidal Datums	4
Friday Harbor	9449880	Jan-32	present	0.00	2.29	4.55	4.70	7.11	7.76	-0.53	-0.53	1.76	4.02	4.17	6.58	7.23	PSE, WSDOT data	3
La Conner, Swinomish Slough	9448558	Jun-35	Feb-73	0.00	2.70	5.96	5.18	9.43	10.35	-1.51	-1.51	1.19	4.45	3.67	7.92	8.84	PSE Verrified	4
Crescent Bay	9443826	Sep-78	Nov-78	0.00	2.16	4.22	4.32	6.47	7.06	-0.42	-0.42	1.74	3.80	3.90	6.05	6.64	PSE Verrified	4
Port Angeles, WA	9444090	Aug-75	present	0.00	1.93	4.25	4.23	6.52	7.07	-0.43	-0.43	1.50	3.82	3.80	6.09	6.64	NOS Tidal Datums	4
Port Townsend	9444900	Dec-71	present	0.00	2.50	4.99	5.17	7.84	8.52	-1.11	-1.11	1.39	3.88	4.06	6.73	7.41	PSE Verrified	3
Everett, WA	9447659	Dec-76	Feb-96	0.00	2.80	6.48	6.51	10.21	11.09	-2.30	-2.30	0.50	4.18	4.21	7.91	8.79	NOS Tidal Datums	4
Seabeck, Hood Canal	9445296	Mar-35	Mar-78	0.00	2.99	6.75	6.76	10.54	11.49	-2.62	-2.62	0.37	4.13	4.14	7.92	8.87	PSE, VDATUM ONLY!!	1
Seattle, Puget Sound	9447130	Jan - 1899	Sep-88	0.00	2.83	6.64	6.66	10.49	11.36	-2.34	-2.34	0.49	4.30	4.32	8.15	9.02	NOS Tidal Datums	4
Union, Hood Canal	9445478	Mar-73	Mar-78	0.00	3.01	6.96	6.94	10.87	11.85	-2.84	-2.84	0.17	4.12	4.10	8.03	9.01	NOS Tidal Datums	4
Yoman Point, Anderson Island	9446705	Feb-78	Nov-96	0.00	2.94	7.71	7.75	12.55	13.48	-3.78	-3.78	-0.84	3.93	3.97	8.77	9.70	PSE, VDATUM ONLY!!	1
Barron Point	9446742	Sep-88	Mar-89	0.00	3.02	8.29	8.28	13.55	11.52	-4.08	-4.08	-1.06	4.21	4.20	9.47	7.44	PSE, VDATUM ONLY!!	1
Budd Inlet	9446807	Apr-96	Dec-96	0.00	3.07	8.31	8.30	13.53	14.48	-4.05	-4.05	-0.98	4.26	4.25	9.48	10.43	PSE, VDATUM ONLY!!	1

* Level of Confidence: 4 highest, 1 lowest.

"\\Mars\Projects\2036.01_PSNERP_Phase_1_Conceptual_Engineering\Applied Geomorphology Guidelines\NAVD MLLW Conversions\PSE Survey PSNERP_DATUM_CONVERSIONS 113010.pdf"

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2. Tidal Salt Marsh Channels (Channel Modification, Dike Removal, Hydraulic Connection)

Tidal marsh channels are often sized based on applied geomorphology, typically using hydraulic geometry or allometry (Williams et al, 2002; Hood, 2002). Unfortunately, existing data sets are not adequate to develop guidelines for Puget Sound, and research indicates large variation between systems and locations (Hood, 2007; 2002). Still, some basis is needed to size channels in the conceptual designs as these are key drivers of quantity and cost estimates. Therefore, the guidelines presented here can be considered more of an engineering method and not vetted from a scientific perspective.

Hydraulic geometry has been used primarily in the study of fluvial and tidal systems, where channel parameters such as stream width or depth are regressed with area of the watershed (used as a surrogate for tidal prism and discharge). The form of the equation is typically a power function:

$$Y = a*x^n,$$

Where x is a independent variable (eg marsh area or watershed area),Y is the dependent variable (tidal channel width or stream depth), a and n are empirically derived coefficients determined from a regression of the log-transformed independent and dependent variables.

The hydraulic geometry of tidal channel parameters has been investigated in Washington at the Chehalis estuary by Hood (2002) and at the Skagit delta by Hood (2007). In the Chehalis work, log-transformed slough outlet width and outlet depth are shown to scale tightly (r^2 >0.95 for both) with outlet length for the Chehalis river sloughs. However, when three other nearby systems are analyzed in a similar fashion, there are significant differences (95% confidence level) in the regression estimates for nearly all of the systems analyzed. Hood (2002) indicates that these differences are likely a result of watershed processes, such as run off or soils, and that these differences must be integrated into the development of a restoration project. Furthermore, two of the systems investigated (Willapa River and South Fork Willapa River sloughs) undergo the same tidal regime, but have somewhat differing hydraulic geometry scaling relations.

Similar scaling regressions were performed in the Skagit delta, but in this work, outlet channel depth was not included in the analyses (Hood, 2007). As above, there are significant differences in the scaling relationships between channel outlet width and marsh island area for similar, nearby locations. In the Skagit delta area, these differences are likely driven by sedimentation and discharge from the Skagit river (Hood, 2007).

Approximate Hydraulic Geometry for Puget Sound, Extrapolating San Francisco Bay Regressions

The most expeditious means of developing guidelines for sizing tidal marsh channels is to modify the guidelines for San Francisco Bay (Williams et al, 2002; PWA, 1995). San Francisco Bay data sets are large and have been used successfully in design of marshes from a few acres to thousands of acres. While Puget Sound marshes should have different geometry due to different sediments, salinities and plants and greater rainfall effects, the primary difference is believed to be driven by the larger tide ranges.

These regressions are intended to represent future equilibrium conditions. In most cases, these dimensions are recommended for construction, with modification for constructability and slope stability if important. Overall, channels can be expected to evolve along with the marsh and take decades to reach an equilibrium condition, largely depending on sediment supply and vegetation establishment.

To account for the larger tide range in the Puget Sound area (diurnal range 7' to 16' with an average of about 10.5'), we adjusted the regression lines for San Francisco Bay data. First, we compared the large San Francisco bay data set (typical diurnal tide range about 5.8ft) with the subset from southern San Francisco Bay where the tides are much larger (range about 8.8ft). We then calculated the change in regression lines between the two data sets, and related the differences to percent increase in tide range. We then prorated this increase based on the tide ranges in Puget Sound.

Figures 2, 3 and 4 show data from San Francisco Bay (Williams et al, 2002) and Discovery Bay (Barnard, project worksheets, 2010), and regression lines by Hood (2002) and PWA (2003). The recommended regressions are those in red. These are example regressions for one tide station.

The above methodology was applied to 14 tide ranges defined at tide gauges distributed throughout the study area. This resulted in adjusted regression lines for each of the tide stations that are listed in section 1. Fourteen graphs (one for each tide station) are provided in the Appendix. Each graph includes three lines:

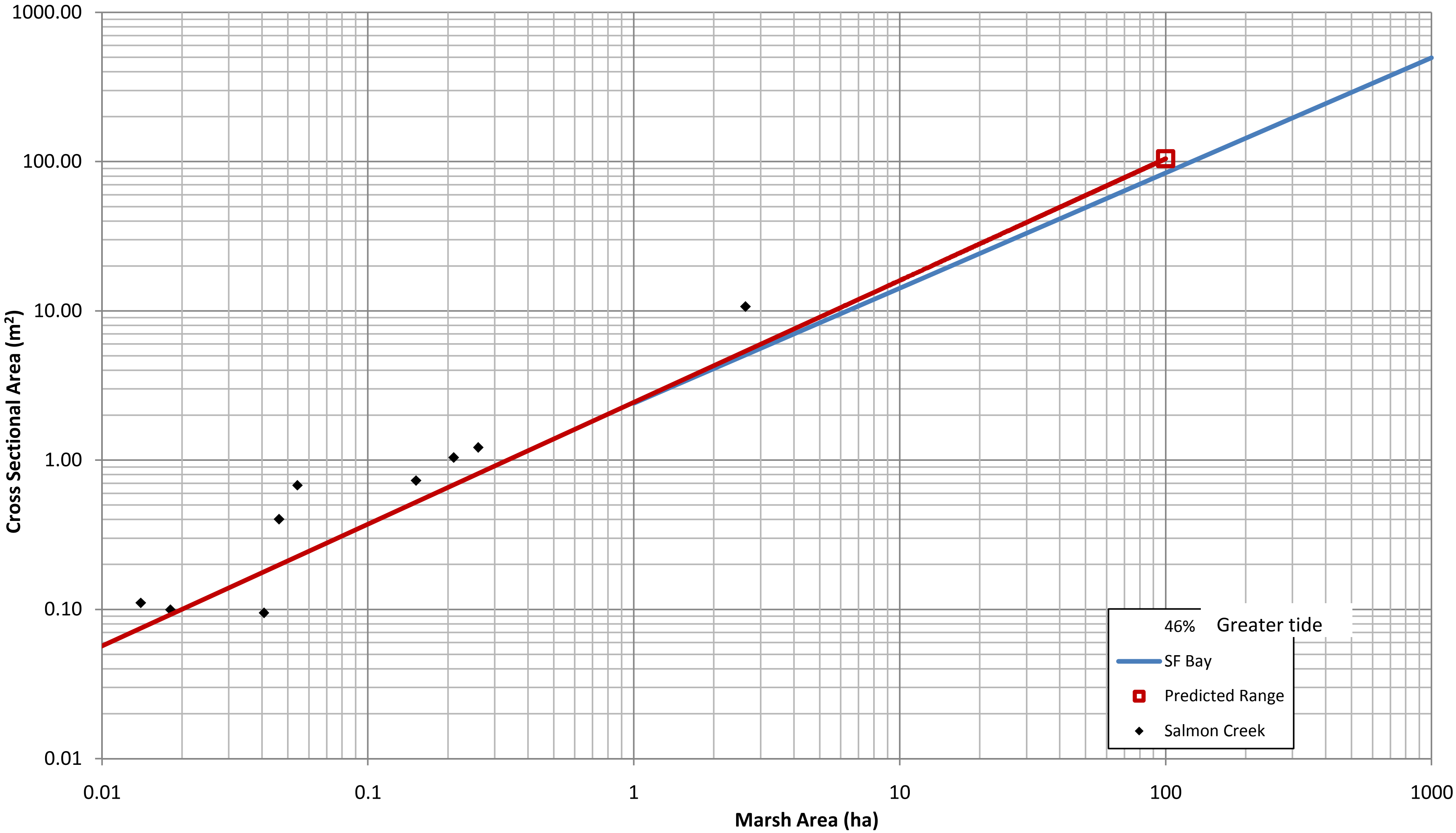
- Channel Cross section area (feet squared) vs. Marsh Area (acres);
- Channel Width (feet) vs. Marsh Area (acres); and,
- Channel Depth (feet) vs. Marsh Area (acres).

Upsizing to include stream discharge effects and additional tidal prism

The above discussion is based on tidal prism being the primary channel forming parameter, and uses marsh area as surrogate for tidal prism. Many Puget Sound marshes have significant freshwater inputs which add to the scouring power during ebb tides and therefore can be expected to increase the size of larger channels. To calculate the hydraulic geometry of a channel that incorporates fluvial discharge, the following methods are proposed. First, calculate the volume of water associated with fluvial discharge over the ebb period. Second, calculate the channel cross-sectional area from the marsh area. Third, using the Williams et al (2002) graph of tidal prism versus cross-sectional area:

a) locate the initial estimated cross-sectional area,

b) estimate the associated tidal prism,



Notes: Predicted width regressions are pro-rated based upon the depth and marsh area hydraulic geometry regressions of South SF bay and SF bay and tide range at given location. % greater tide refers to % greater than SF bay tides and is used to determine predicted range.
Source: Regression equations modified from Williams et al. (2002).

figure **2**
PSNERP

Channel area hydraulic geometry

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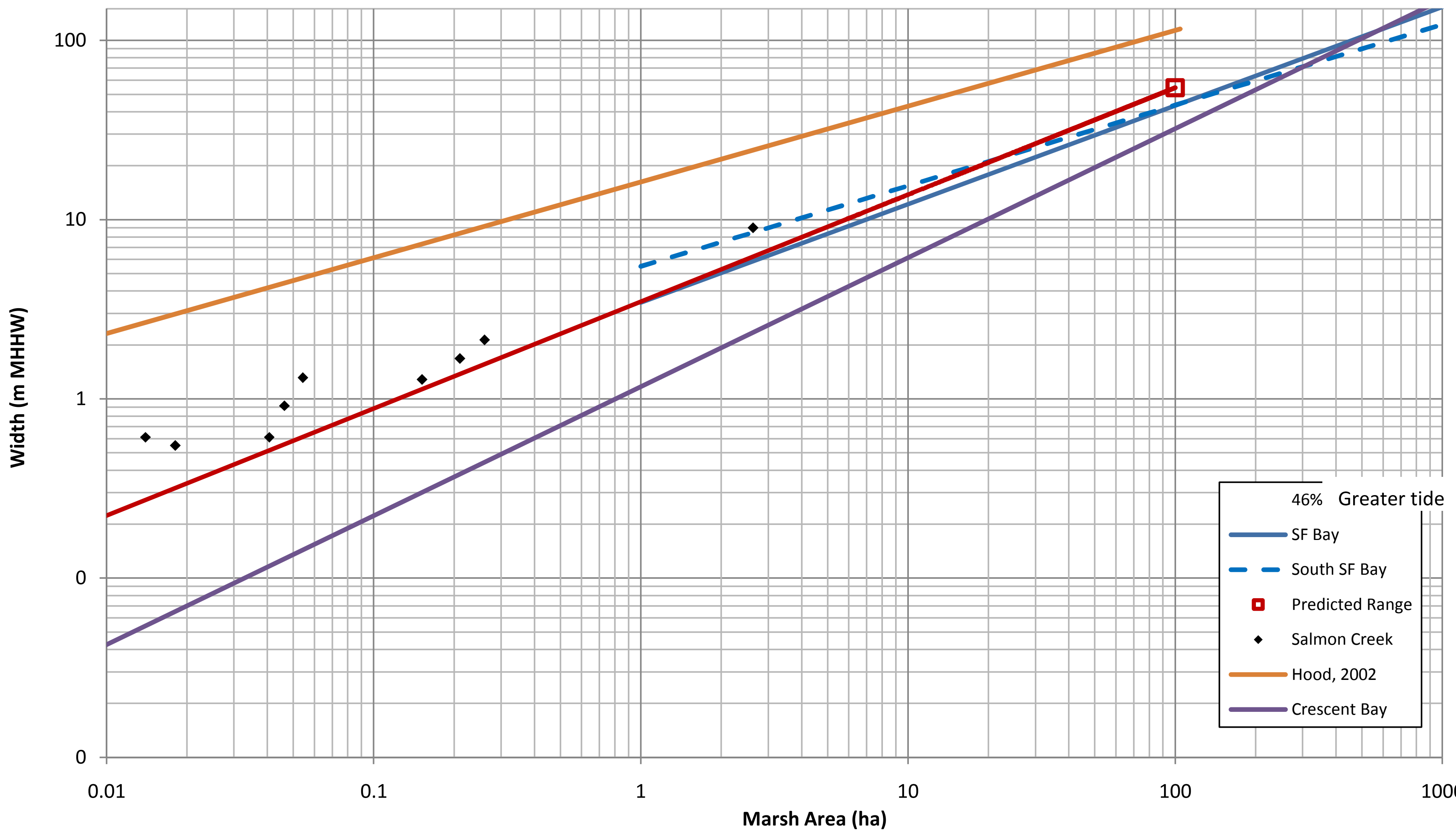

Notes: Predicted width regressions are pro-rated based upon the depth and marsh area hydraulic geometry regressions of South SF bay and SF bay and tide range at predicted location.% greater tide refers to % greater than SF bay tides and is used to determine predicted range.
Source: Regression equations modified from Williams et al. (2002).

figure **3**
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Channel Width hydraulic geometry

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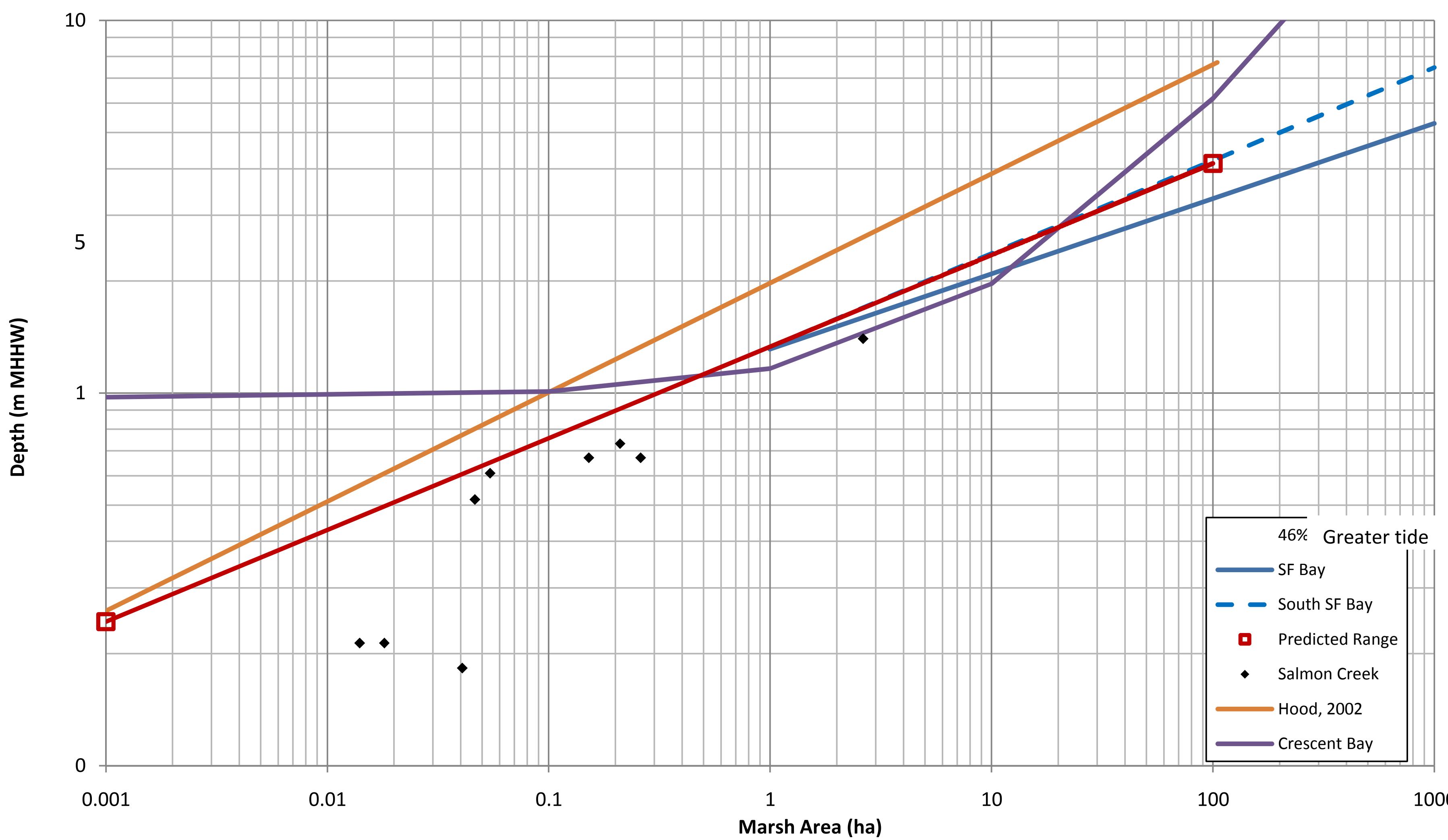

Notes: % greater tide refers to % greater than SF bay tides and is used to determine predicted range.
Source: Regression equations modified from Williams et al. (2002).

figure 4
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Channel depth hydraulic geometry

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c) add the fluvial volume to the tidal prism to get the increased effective tidal prism c,

d) and e) locate the corresponding adjusted cross-sectional area.

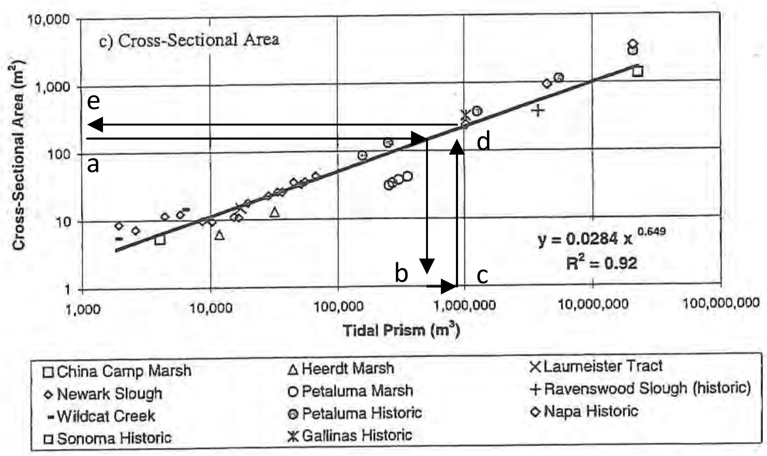


Figure 5: Tidal Prism vs. cross-section area (Williams et al, 2002); with example for adding minor drainage to increase effective tidal prism

The adjusted cross-sectional area can be accomplished by increasing the channel width, and assuming the depth does not increase.

The fluvial flow rate used in the calculation is selected by the designer. For summer conditions, this would be base flow. Otherwise, the estimated channel-forming flow, perhaps in the range of annual to 5 –year recurrence, can be used.

This procedure for increasing channel section for stream discharge can also be used to estimate the size of larger channels to convey the initial (post restoration) tidal prism of subsided sites. Typically, the future equilibrium tidal prism and channel dimensions are adequate and practical for restoration. However, the additional tidal prism for subsided sites can be estimated approximately as the site area times the difference between site grade and the MHHW elevation. This additional tidal prism can be added to get the expanded tidal prism "c", and the expanded channel dimensions estimated.

It should be noted that these are approximate dimensions intended to accommodate site evolution toward equilibrium, rather than equilibrium geometries.

Channel Order and Drainage Area

Channel order is a means of comparing the number of channels of different size within and between drainage networks. The hierarchy of channel segments starts with the smallest channels and increases in order when two channels of the same order connect. For example, when two first order channels join, the downstream segment is classed as second order. When two second order channels join, the downstream segment becomes third order. A first order channel joining a third order channel does not change the order of the downstream segment. The system is defined by the highest order of channel; for instance a tidal drainage network may be described as 'third order'.

Horton (1945) found that channel order is related to a number of metrics describing the channel network:

- number of channel segments;
- segment length;
- drainage area.

Generally these relations are semi-logarithmic as seen Figure 6 that plots drainage area with channel order for marshes in Snow Creek and Salmon Creek (Barnard, pers comm.).

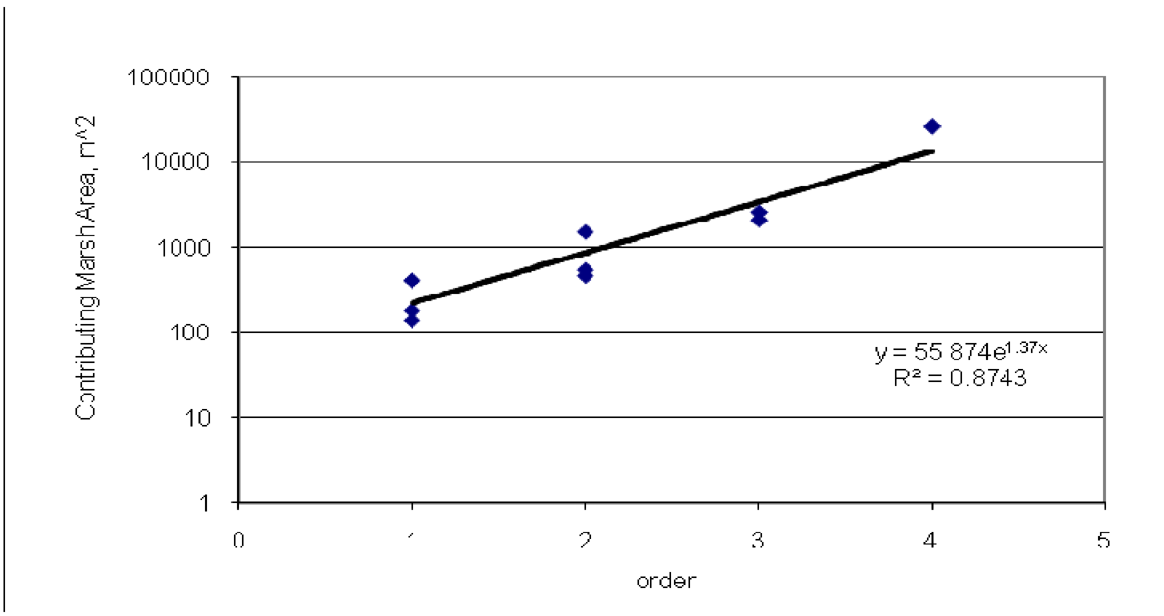


Figure 6: Regression of tidal channel order and marsh drainage area. Source: Salmon Creek data and analysis by Bob Barnard, WDFW.

General guidance can be provided from observations of natural channel systems of similar area; use should be made of local reference sites where possible. The following guidance is based upon marsh channel data and experience in constructing marsh channels:

a) Use the historical channel patterns, if it exists;
b) For drainage areas of 10 to 50 acres, a third or fourth order channel system should be adequate;
c) Second, third and fourth order channels should be excavated to equilibrium depth;

d) First order channels may not be practical to cut, especially if the site is subsided and expected to accrete sediment;
e) If the site is being graded, the marsh slope should be graded down below MHHW and sloped towards the channels to allow drainage and encourage channel network development.

3. Intertidal - Fluvial (channel modification, dike removal)

Given the paucity of data available, design guidelines for tidally-influenced fluvial channels is not practical within the time and budget constraints of this project. Hence, we recommend use of historic maps of the site, dimensions of remnant channels, and measurements at nearby reference sites if available.

4. Tidal Inlet (coarse sediment) (Channel Modification, Dike Removal, Hydraulic Connection)

Hydraulic geometry relationships between tidal prism and the cross-sectional area of the inlet channel are perhaps the most common criteria applied to predict the stability of tidal inlets (Battalio, et al, 2006). These are empirical relationships based on surveys of stable inlets and take the form:

$$A_e = C\,\Sigma^n$$

where A_e is the minimum cross-sectional area, Σ is the tidal prism, and C and n are empirically derived parameters. Jarrett (1976) examined earlier work by O'Brien (1931) for Pacific Coast inlets, and established relationships for sites along the Gulf, Pacific and Atlantic coasts. His results were further divided among inlets with and without jetties. Although the expressions established by Jarrett are considered the best available predictors for equilibrium cross-sectional areas, small inlets (small inlets can be defined as those with thalwegs near or above MLLW) tend to exhibit equilibrium area much larger than predicted by these tidal prism relationships (Hughes, 2002).

The cross-sectional area of the inlet channel, A_e, is related to the effective tidal prism by:

$$A_e = 0.65 k_a \,(C_I P)^{8/9}$$

where

$$C_I = \frac{W^{1/8}}{\left[g(S_S - 1)\right]^{1/2} d_e^{3/8}\, T}$$

W is the inlet width at mean tide level (meters), T is the tidal period (typically use semi-diurnal 12.4 hours, which is 44,640 seconds), d_e is the median grain size (in meters), g gravitational acceleration (9.81 m/sec^2), k_a is an empirical coefficient (with a best-fit value of 1.34), and P is the effective tidal prism (cubic meters). S_s is the specific gravity of the sediment (ρ_s / ρ_w) which is often taken to be around 2.6 for quartz and other rock.

The highest point in the channel thalweg typically occurs as the channel crosses the flood shoal and controls the low water elevation in the marsh. Relatively large wave events can induce a control at the receiving water side (eg. the ebb shoal or spit in Puget Sound) as well. Due to the complexities of ebb and flood shoal geometries and the difficulty in field data collection, the narrowest, deepest section of the inlets (aka "throat") are typically used as the reference section. Figure 7 shows the general relationships measured at the Crissy Field Lagoon "throat" in San Francisco Bay. The key parameter is the lagoon low water, which controls the effective tidal prism of the lagoon. The lagoon low water is variable, as it results from the sill elevation formed bay wave transport of littoral sediments against the scour of ebb tides. Inlet morphology also has an effect, which is greatly influenced by littoral drift parameters including structural controls such as reefs and jetties.

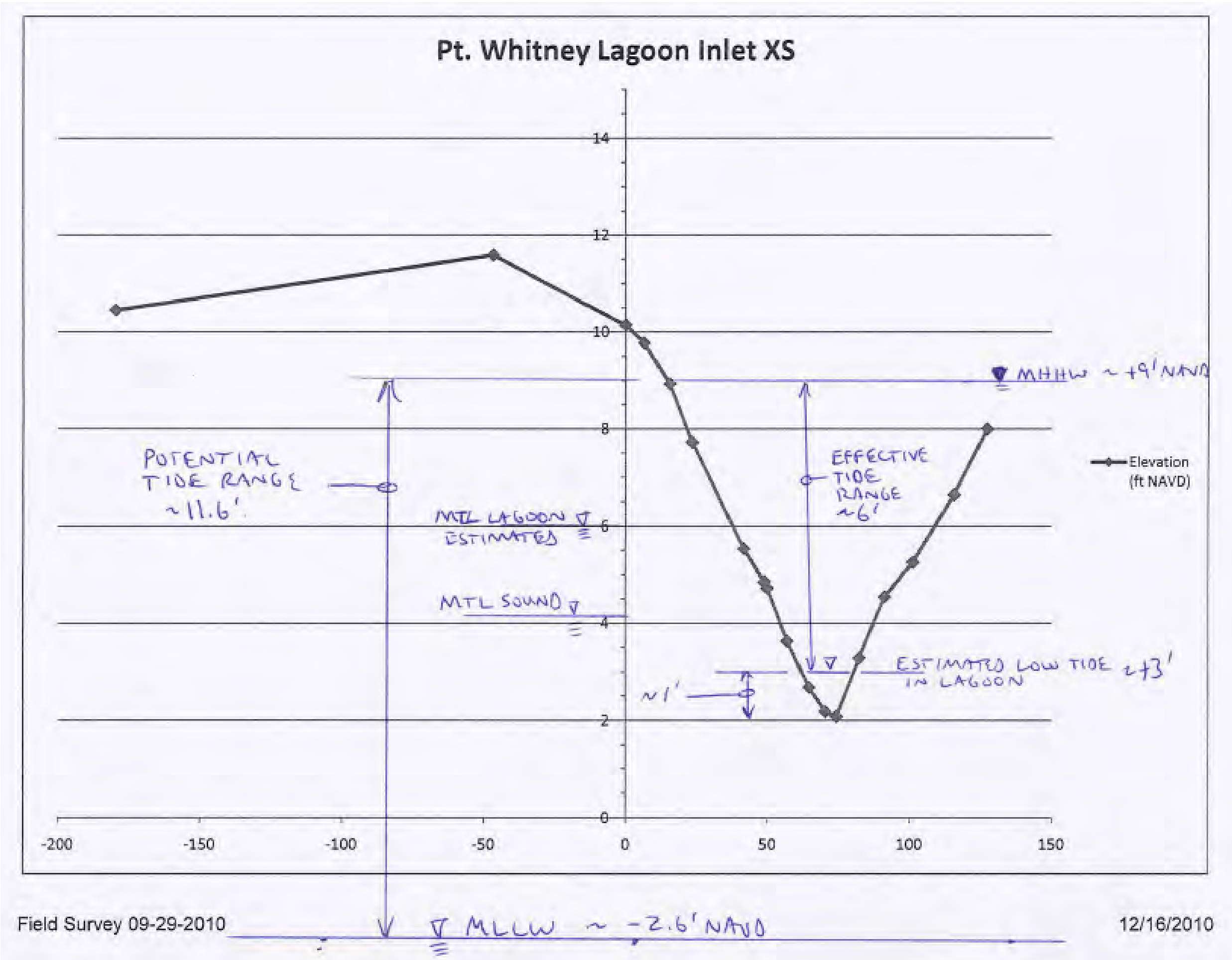


Figure 7. Effective tide range and inlet cross-section.

Considerable scatter in the data suggest that not all of the relevant processes are included in these simple relationships. Therefore, they should only be used as a first approximation and interpreted as representative of long-term average conditions. Significant variations in inlet cross-section can occur over the spring-neap tide cycle, during storms when wave attack is more intense, or following large flood events (DeTemple, 1999). This is especially true for small dynamic systems. A process-based tidal prism relationship developed by Hughes (2002) shows better agreement between small and large tidal inlets, and more promise for application to Puget Sound lagoons (Figure 8).

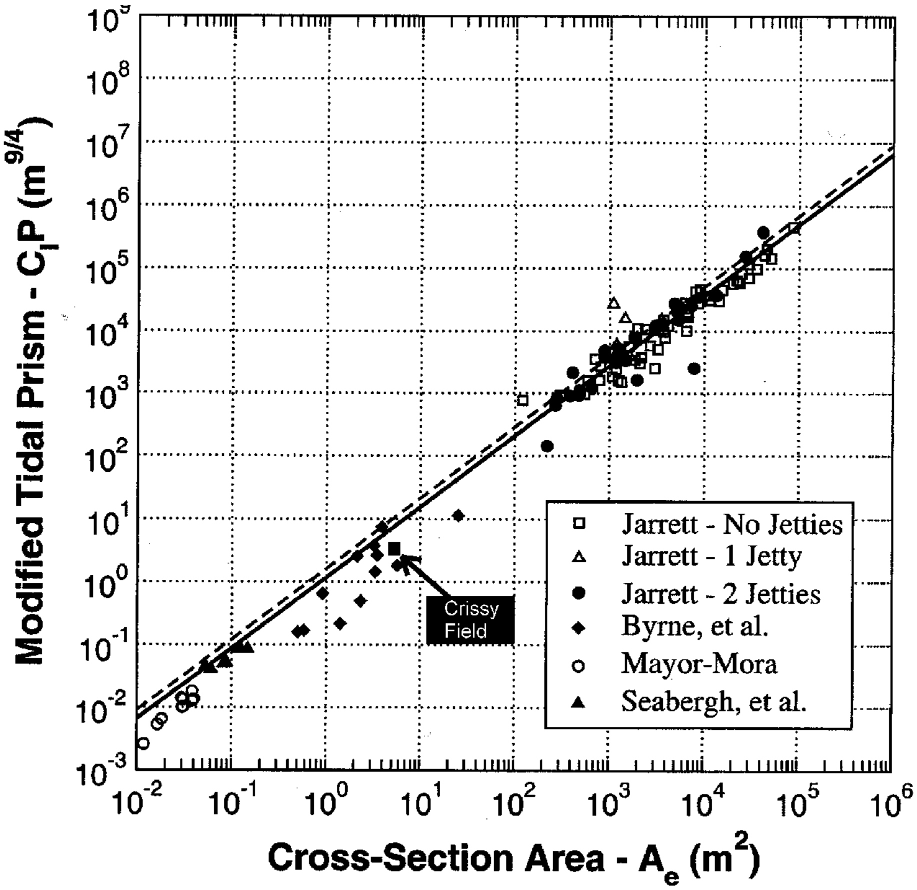


Figure 8. Hughes (2002) tidal prism and inlet cross-section relationship.

It should be noted that the sediments in Puget Sound are typically much coarser than those used to develop these empirical relationships. Coarser sediments tend to have greater porosity and hence can allow greater discharge through the berm, indicating smaller inlets may be likely. Coarser sediments also tend to form higher, steeper beach berms under wave action, which tend to resist deep thalwegs and hence shallower inlets can be expected. Finally, coarse sediments do not move as quickly for a given wave climate, and hence Puget Sound inlets will likely be more stable. In summary, we expect Puget Sound inlets to be shallower and more stable, with greater outflow through the littoral barrier.

Applications of the O'Brien and Johnson methods can be improved with accurate estimates of wave and tidal power. Figure 9 shows the corrected data in the form previously reported by Williams & Cuffe (1994) for California lagoons. We believe the version shown below can be used generally with contemporary wave power values (prior published versions had wave power values about 200 times too high (PWA, 1999)).

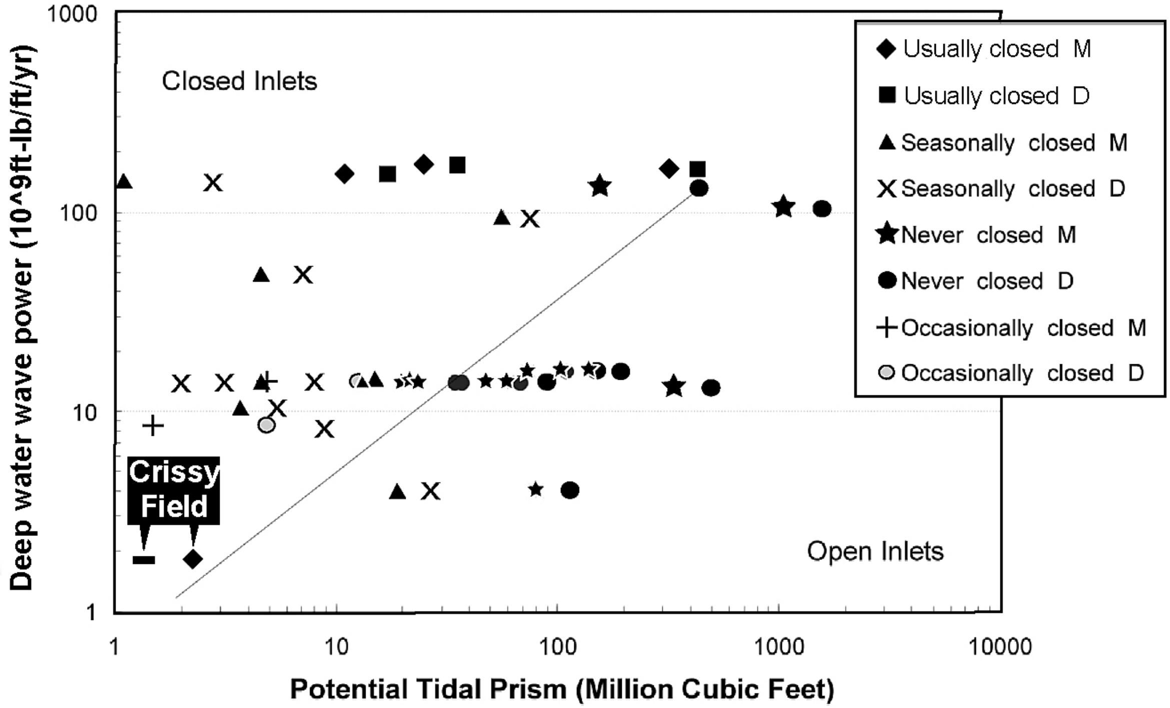


Figure 9. Power-based index of inlet closure potential with corrected annualized wave power. “M”=Mean tidal prism and “D”= diurnal tidal prism.

Ideally wave power would be calculated from the wave climate. Since much of the Sound is relatively deep with short fetches, relatively short wave periods can be expected. Exceptions are the Straits of and Juan De Fuca and Georgia where swell (Juan De Fuca)and large wind waves can occur (Strait of Georgia, Coulton et al, 2001): For these locations, more detailed calculations are recommended. For the Strait of Juan De Fuca, wave data are available. In most sites, wind wave hindcast using fetch limited parametric equations and wind climate is sufficient to define the deepwater wave climate. Simplified wave refraction using Snell’s law and diffraction using the methods of Goda (1985) are adequate. These data exist for some sub basins but otherwise may be beyond the scope of the PSNERP conceptual designs.

Wave power can then be used to estimate the minimum tidal prism needed to maintain an open inlet, using Figure 9. Note that Figure 9 is based on the potential tidal prism, which is typically the lagoon size multiplied by the potential tide range. Dividing by the tide range can then give an estimate of minimum lagoon size required for an open inlet. However, the actual tidal prism, often called the effective tidal prism, will be reduced by the littoral ridge built by waves. The effective tidal prism and tide range are “implicit” within inlet equations and can be the most challenging parameters to estimate. For open lagoons, a first estimate can be made by multiplying the lagoon area times the depth below MHHW (assuming the site bed is above MLLW). Note that the littoral sill often prevents full drainage, so using the existing grade as an estimate of the low tide may lead to an over-estimate of effective tidal prism and inlet area. For marshes, a first approximation can be based on the methods in Section 2 of this document. Also, like marshes, fluvial discharge tributary to the site can increase inlet size above that based on tidal prism alone. Once the effective tidal prism is estimated, it will be used to estimate the required inlet cross section geometry using Figure 8 and selected aspect ratios (width to depth).

The best available relationship for small tidal inlets in littoral systems is Hughes (2002; Figure 8). A review of this equation indicates that it is very sensitive to grain size, with larger grain sizes resulting in smaller predicted inlet cross sections. We recommend using a default of 1 mm which will help keep the equation within the range of data sources and bias the area calculation to the high side: In general, over-excavation of the inlet results in less risk of subsequent closure. Further research is needed to inform use for coarse sediment shores.

It should be noted that over-excavation will induce a perturbation that can reduce sediment supply to adjacent shores. Excavated sediment compatible with the littoral sediment should be placed down drift to mitigate the subsequent interruption of longshore transport during inlet evolution. For new inlets, placement of littoral sediments should be considered to mitigate the sediment deficit induced by flood and ebb shoal formation. The effect can extend updrift as well but to a lesser extent.

Once area is calculated, width and depth are selected. Ideally, an estimate of one of these parameters will be available. For example, the inlet width from an historic map can be used, and then the depth can calculated based on an assumed shape (see below).

The equation for a channel with a parabolic shape is as follows:

$$W=1.5A/d$$

Where W = width, A = area and d = depth.

Deepening of an existing inlet due to increased tidal prism can be estimated by prorating the existing depth using the square root of the estimated increase in cross-sectional area. Assuming that the width-to-depth ratio of the inlet throat remains the same, changes in depth of the throat can be estimated from:

$$\frac{d_{new}}{d_{old}} = \sqrt{\frac{A_{e\,new}}{A_{e\,old}}}$$

where d is the maximum depth at the throat, measured below $MTL_{lagoon,}$, and Ae is the area defined above for the Hughes (2002) relationship.

The depth of the inlet channel below Mean Tide Level (MTL) has been approximated using the relationship between depth and inlet area devised by Vincent and Corson (1981):

$$D_m = 0.5579(A_c)^{0.38}$$

where D_m is the depth at the throat of the inlet below MTL (ft) and A_c is the inlet below MTL (ft^2). This relationship was developed from data in Chesapeake Bay (Virginia, USA) which has a much smaller tide range, a semi-diurnal tide, and much finer sediment. Hence, this relationship should be used as an indicator to inform design judgment. For smaller systems, this equation may under-predict the depth due to its derivation with data from areas with smaller tide ranges. If this equation is used and predicts depths significantly above MLLW, over-excavation is recommended.

Reference site data can be used instead of or in addition to the methods proposed here.

5. Beach (coarse sediment) (bulkhead removal, groin removal)

The morphology of coarse sediment beaches includes a steep foreshore (swash zone) leading up to a flat terrace or berm (Bauer, 1974; Lorang, 2002). The profile morphology and terms are shown by the example in Figure 9 (Birch Bay, Whatcom County, Bauer, 1975). The swash zone slope is affected by sediment size and wave climate. For most Puget Sound locations, the swash zone has a typical slope around 7:1 with a range between 5:1 and 10:1 (horizontal: vertical). Steeper slopes can be expected for coarser and more uniform sized sediments, and higher wave exposure. The berm elevation is typically considered the result of wave runup that builds the berm to a level just below the annual maximum total water level (total water level is defined as the Puget Sound water level plus wave runup).

Figure 10 shows a conceptual profile of coarse beach dynamics (Bauer, 1975). Note that Figure 10 shows a berm configured to provide protection to inland development and includes extra volume as a "storm buffer," with the berm crest elevation about 6.5' above MHHW. Given PSNERP's focus on restoration, berm heights should typically not be over-built for protective purposes. Figures 11 and 12 provide an example of a reference site in Whatcom County (PWA, 2002). The berm crest is around 11.3' to 12.8' NAVD (converted from NGVD by adding 3.8'), and about 4 feet above MHHW. Note the wood above the berm indicating that the total water level exceeds the berm crest in natural conditions. Therefore, we recommend under-building the berm slightly to allow shaping by wave action. Slight under-building avoids the potential adverse effects of unnatural morphology and while limiting sediment demand to fill the "void" resulting removal of fill or armoring.

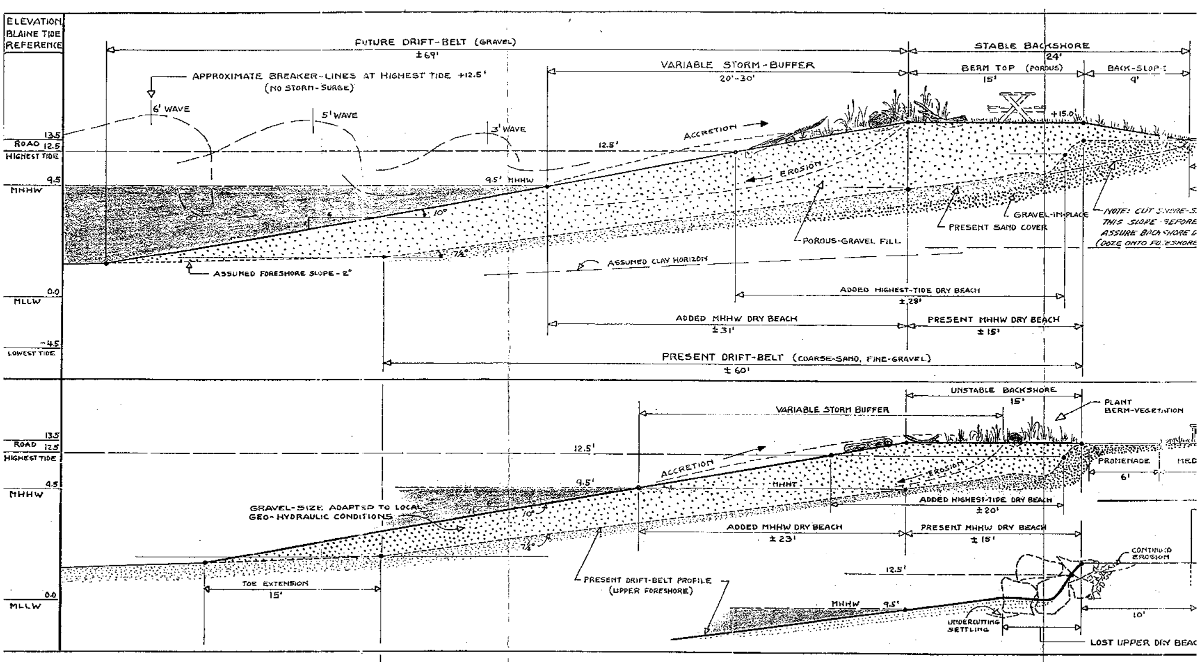
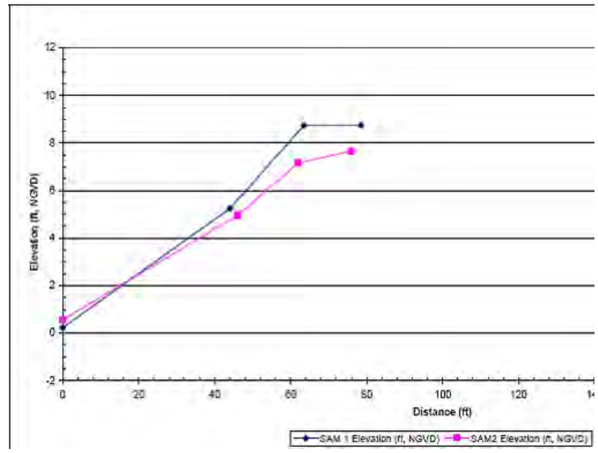


Figure 10. Description of coarse sediment beach profile morphology, for a protective berm (Bauer, 1975).



Figures 11 and 12. Semi-Ah-Moo Beach. Photograph of beach swash zone and berm (left); Elevation cross sections of swash zone slope and berm (right). Source, PWA, 1975.

For PSNERP conceptual designs, the geometry of nearby reference sites can be used to develop the restored profile and estimate quantities. Alternatively, some basic parameters should be sufficient. A slope of around 5:1 to 10:1 with a typical of 7:1 is recommended. This should be checked and adjusted based on consideration of local geometry, reference sites, size of sediment and wave exposure. The berm elevation can be estimated as the height of the annual wave runup (R) on the profile using the following equation:

$$
\begin{aligned}
R_{2\%} &= \text{Static Setup} &&+ \text{Runup} \\
&= 0.2\, H_0 &&+ 0.6r\,(m/(H_0/L_0)^{1/2})H_0
\end{aligned}
$$

The above equation is based on the surf similarity parameter / Iriabarren number; m is the beach slope, r is an empirical coefficient, H_0 is the significant deep water wave height and L_0 is the deep water wave length . The wave values used should be on the order of an annual to 5 year return period. (Note that the term “2%” for the runup does not refer to annual frequency, but rather the exceedance within an event, e.g. the significant exceedance is typically considered 33% and the rms 50%).

It is recommended that a composite slope of about 10:1 (m=0.1) is used to account for larger waves breaking offshore of the swash formed foreshore. Also, the result should be adjusted downward to account for the permeability of the coarse sediment and a factor of about r=0.8 is recommended.

Wave runup on natural beaches does not typically exceed about three times the wave height. For steeper waves on porous (gravel, cobble) sediments, the runup is reduced and a maximum of about two times the wave height can be expected. We therefore recommend that a reasonable range for runup in sheltered waters (not exposed to ocean swell) is between 0.5 and 2 times the wave height, and on coarse sediment shores (gravels and cobble) will not typically exceed 1 times the wave height.

Since the berm is formed by the total water level with an approximate annual exceedance, the more extreme wave runup value (1 to 5 year recurrence) should be added to a typical high tide, on the order of MHHW or MHHW with a surge / setup added: A setup due to meteorological effects can be on the order of 1 foot. Alternatively, an annual high water level can be combined with a smaller, nominal wave height likely to occur simultaneously with the high tide.

For cases where much larger waves break far offshore of the berm, wave setup for the larger offshore waves should be added. A static wave set up can be approximately estimated as 0.2 times incident wave height (FEMA, 2005). The groupiness and randomness of the waves also results in longer-period dynamics often called dynamic setup. Accounting for dynamic setup and combining with static setup and runup can be complex. However, for the conditions associated with PSNERP it is recommended that a total wave setup of about 0.3 times the deepwater wave height is a reasonable estimate of total setup due to larger waves breaking offshore.

We recommend a minimum beach berm elevation of 1.5 ft above MHHW.

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Appendix A: NAVD Conversions from Pacific Survey and Engineering, 2010

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Bob Battalio

From: Adam Morrow [AMorrow@psesurvey.com]
Sent: Tuesday, November 30, 2010 9:35 AM
To: Margaret Clancy
Cc: Bob Battalio
Subject: PSNERP Datum Conversions
Attachments: PSNERP_DATUM_CONVERSIONS.pdf

Margaret,

In an effort to wrap up our work on this project to date and provide you with an item that had been discussed in some detail over the past few months, we have attached a spreadsheet that details our Tidal-NAVD 88 vertical datum conversions that can be used for pre-selected sites. The spreadsheet includes conversions for areas that were noted as not available in Bob Battalio's previously emailed spreadsheet.

With a few exceptions, we were able to find consistent datum conversions for tidal regions throughout Puget Sound. Where we could not, we listed the applicable VDATUM conversion related to the reference tidal gauge. For your use, we also included a column that indicates our level of confidence for each conversion, based on the availability of published benchmark information and/or conflicts between published data and VDATUM results.

We hope that this proves useful for the design team in the continued efforts to provide 10% design documents for the project. We are ready and willing to respond to questions about this information as needed to help you complete your Phase 2 work.

I look forward to hearing from you in the near future about opportunities to continue to provide services on this project. From our research work to date, I suspect that we now have a good database of information from which we can provide cost estimates for necessary survey and base mapping work at each site if that is deemed necessary.

Thanks.

-Adam

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1812 Cornwall Avenue
Bellingham, WA 98225
(360) 671-7387
(360) 671-4685 (fax)

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	NOAA TIDAL REGION	POINTS IN COMMON (NGS)	Station Lat.	Station Long.	SEPARATION NAVD-TIDAL* US FEET	SEPARATION VDATUM** RESULT	TIDAL EPOCH	CONFIDENCE LEVEL***
Cherry Point	9449424	1	48 51.8 N	122 45.5 W	**0.96**	0.96	83-01	2
Friday Harbor	9449880	3 (WSDOT)	48 32.8 N	123 0.6 W	**0.53**	0.38	83-01	3
LaConner, SS	9448558	1	48 23.5 N	122 29.8 W	**1.51**	1.51	83-01	4
Crescent Bay	9443826	1	48 9.7 N	123 43.5 W	**0.42**	0.42	83-01	4
Pt Townsend	9444900	2	48 6.7' N	122 45.5' W	**1.11**	1.12	83-01	4
Seabeck, HC	9445296	0	47 38.5' N	122 49.7' W		**2.62**	83-01	1
Yoman Pt, AI	9446705	0	47 10.8' N	122 40.5' W		**3.78**	83-01	1
Barron Pt	9446742	0	47 9.4' N	123 0.5' W		**4.08**	83-01	1
Budd Inlet	9446807	0	47 5.9' N	122 53.7' W		**4.05**	83-01	1

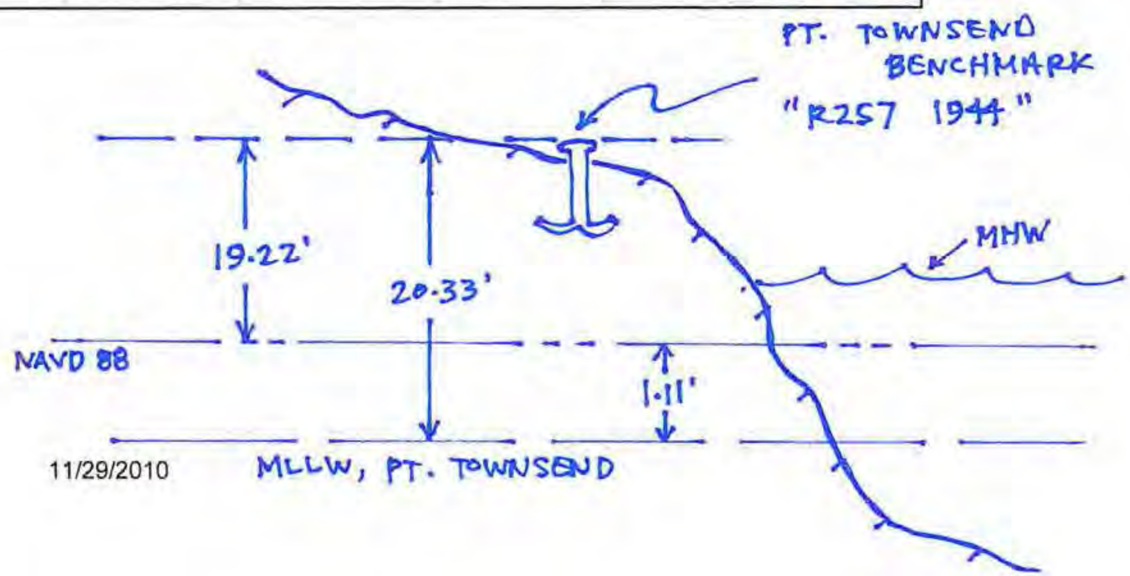
* Conversions determined from published elevations on benchmarks in common between NOAA, WSDOT and NGS.

** Conversions determined using VDATUM, a software tool developed jointly by NOAA's NGS, OCS and CO-OPS.

*** Level of confidence in conversion values based on # of sources found and agreement between sources. 4=highest, 1=lowest.

HOW TO APPLY CONVERSION:

Throughout the region, the 0 elevation plane of NAVD88 is above the 0 elevation plane of the various tidal datums. As a result, the NAVD88 elevation at any given point should reflect a smaller value than the local tidal datum elevation at the same point. To convert from tidal to NAVD88, subtract the separation value noted above in any given region. To convert from NAVD88 to local tidal datum, add the separation value. See sketch below:



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Appendix B: Graphs of Tidal Wetland Channel Dimensions vs. Marsh Area

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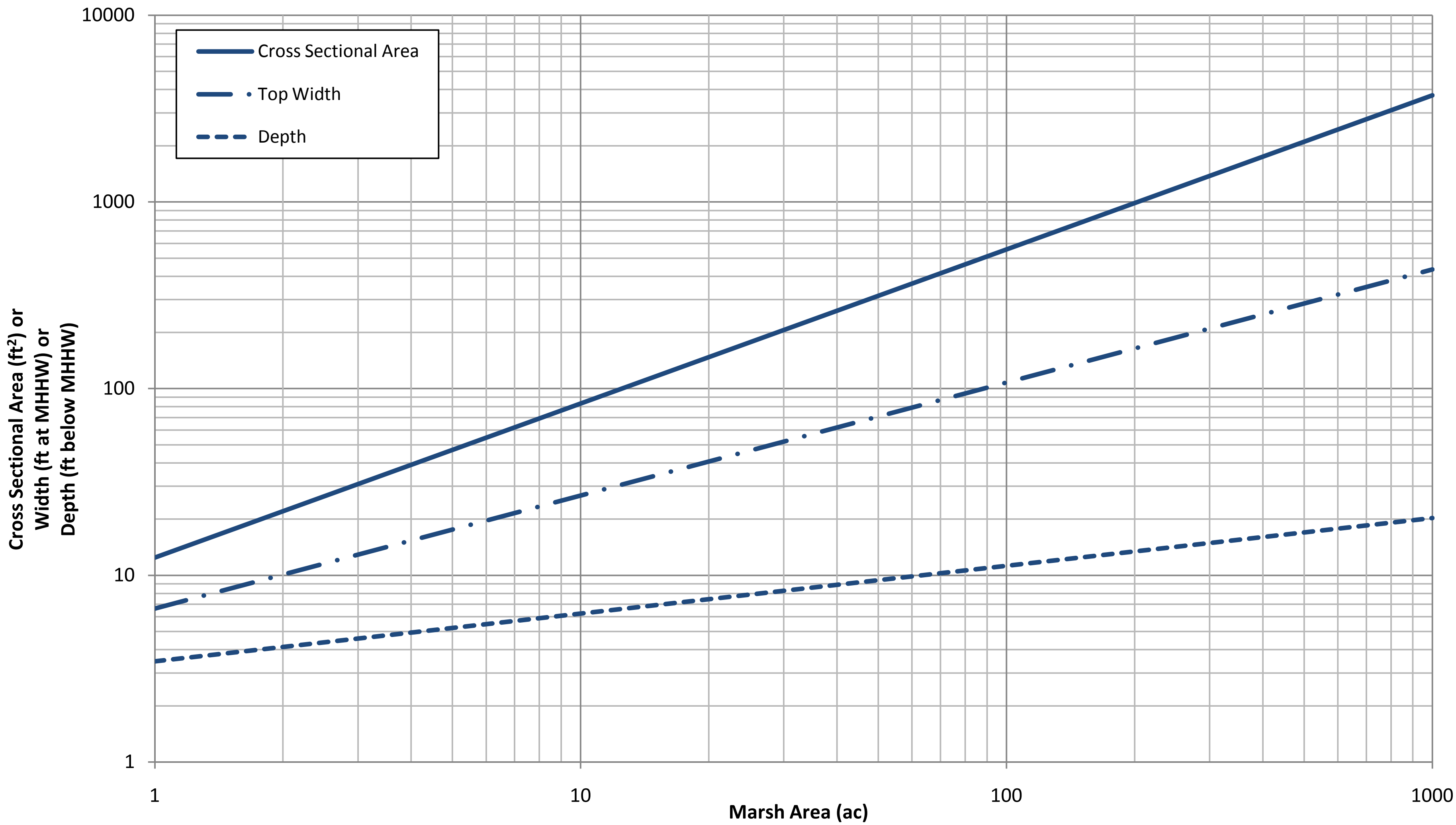

Tide Gage Station: Cherry Point # 9449424
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure ***1***

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Cherry Point

PWA Ref #: 2036.00

PWA

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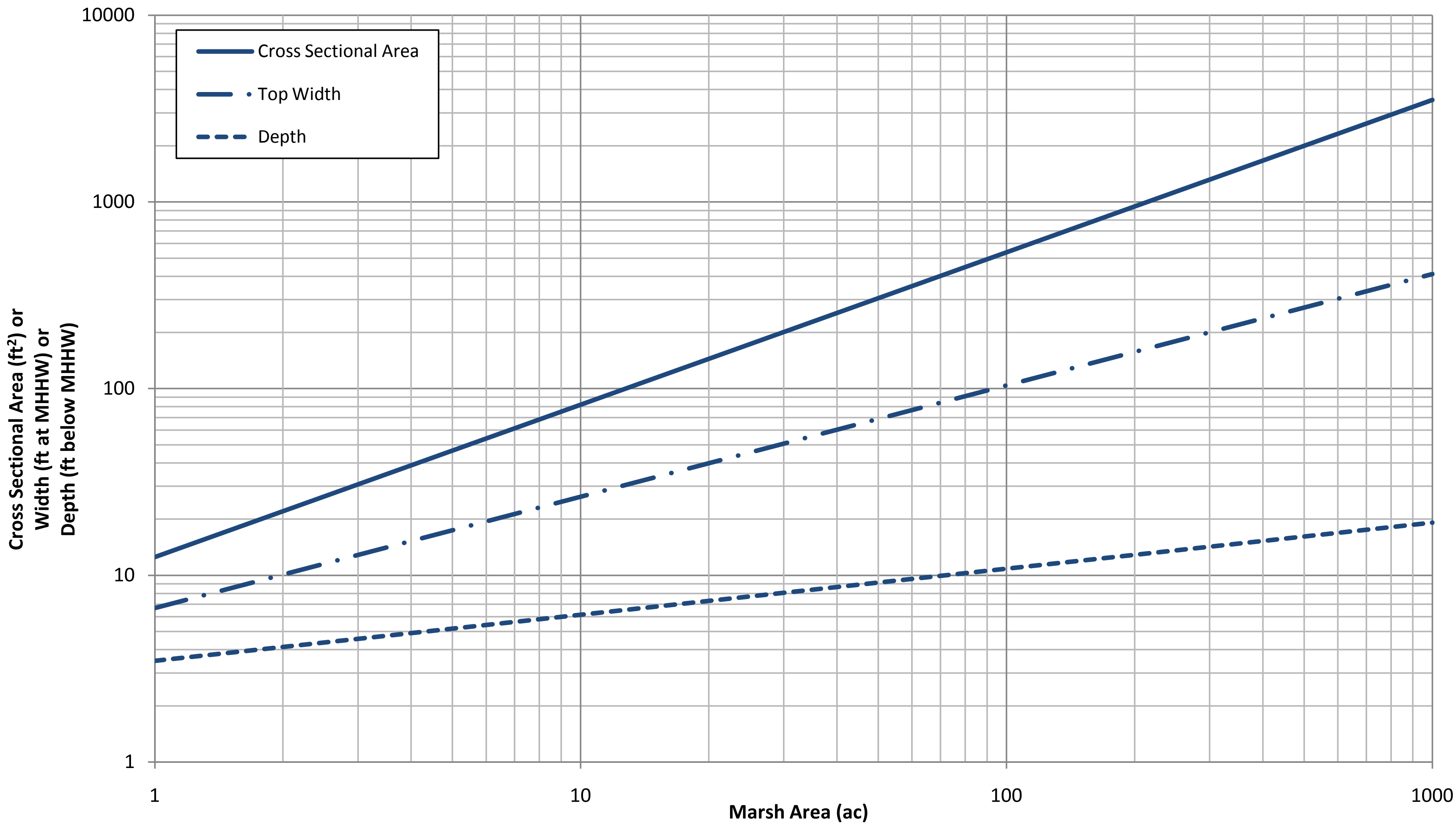

Tide Gage Station: Bellingham # 9449211
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure **2**

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Bellingham

PWA Ref #: 2036.00

PWA

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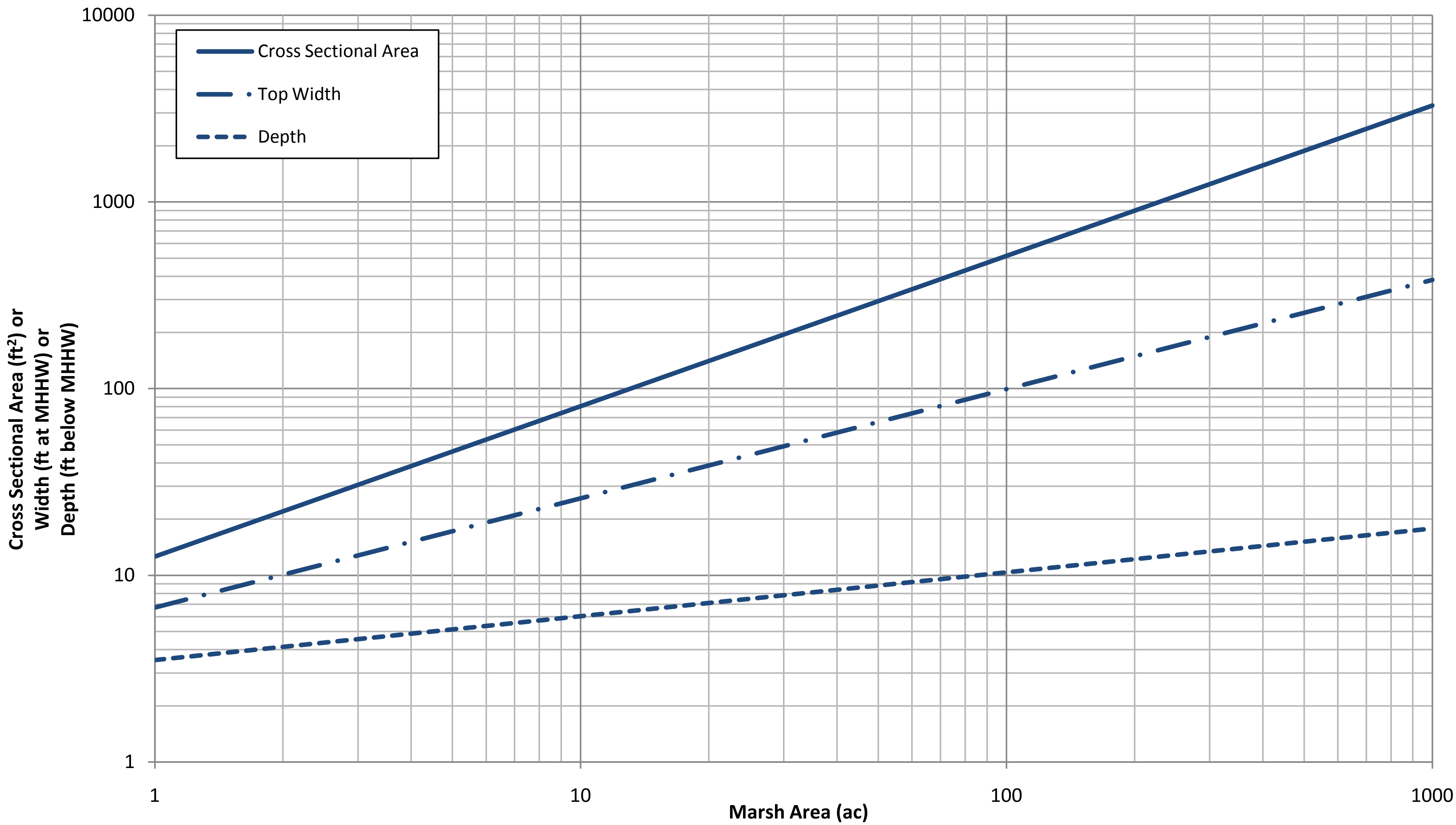

Tide Gage Station: Friday Harbor # 9449880
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure **3**

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Friday Harbor

PWA Ref #: 2036.00

PWA

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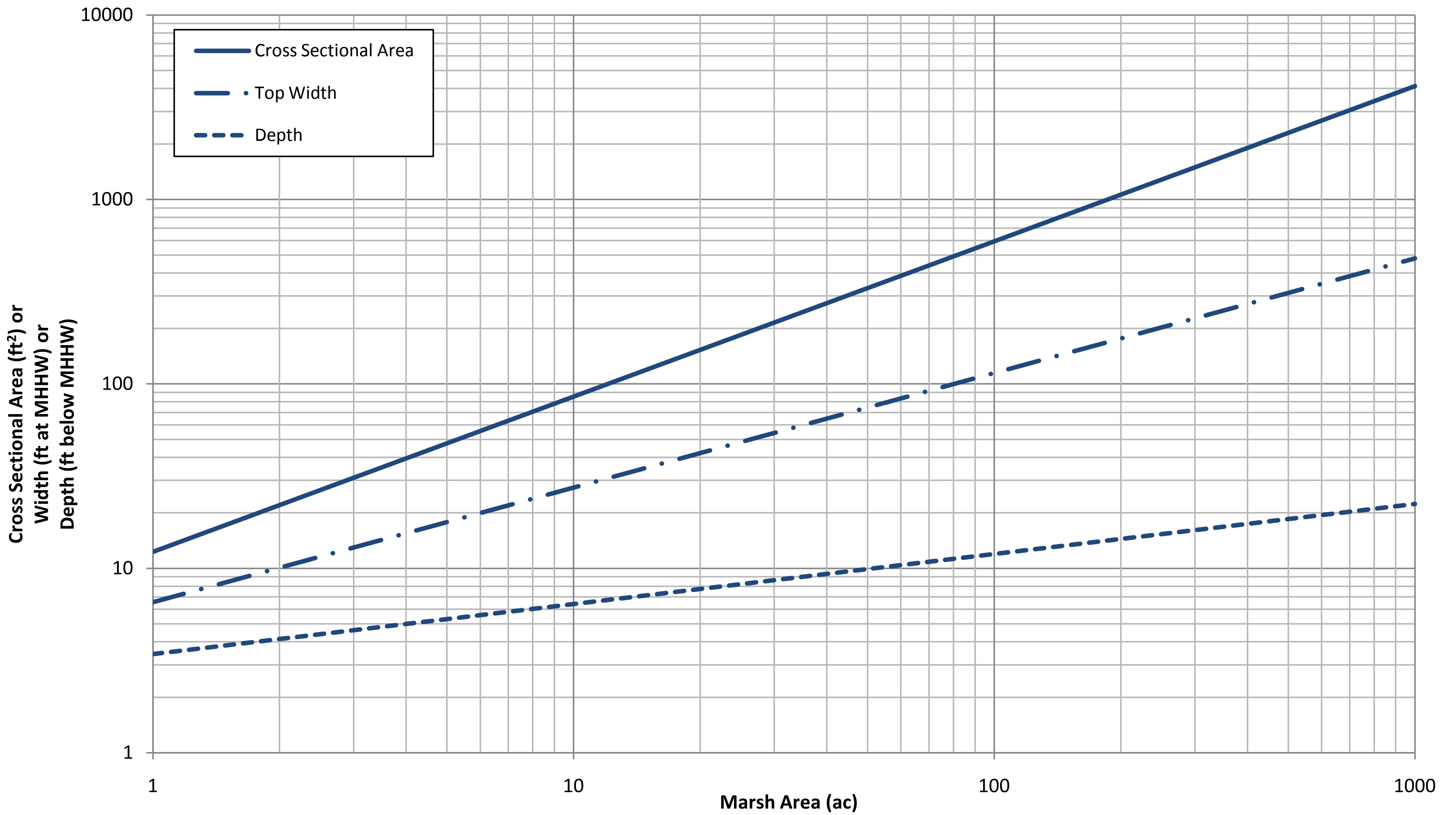

Tide Gage Station: La Conner, Swinomish Slough # 9448558
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure **4**

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for La Conner, Swinomish Slough

PWA Ref #: 2036.00

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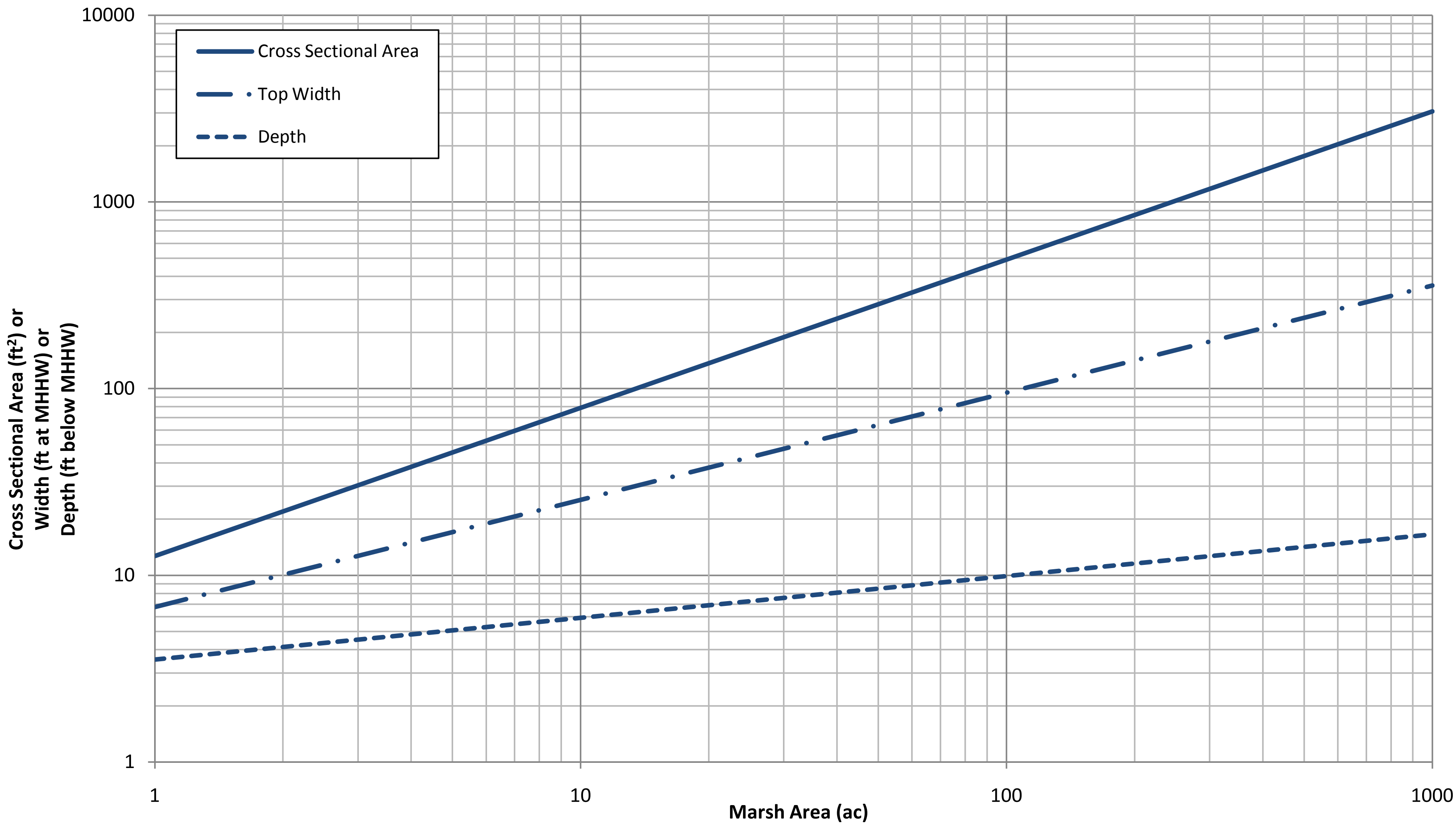

Tide Gage Station: Crescent Bay # 9443826
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure **5**

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Crescent Bay

PWA Ref #: 2036.00

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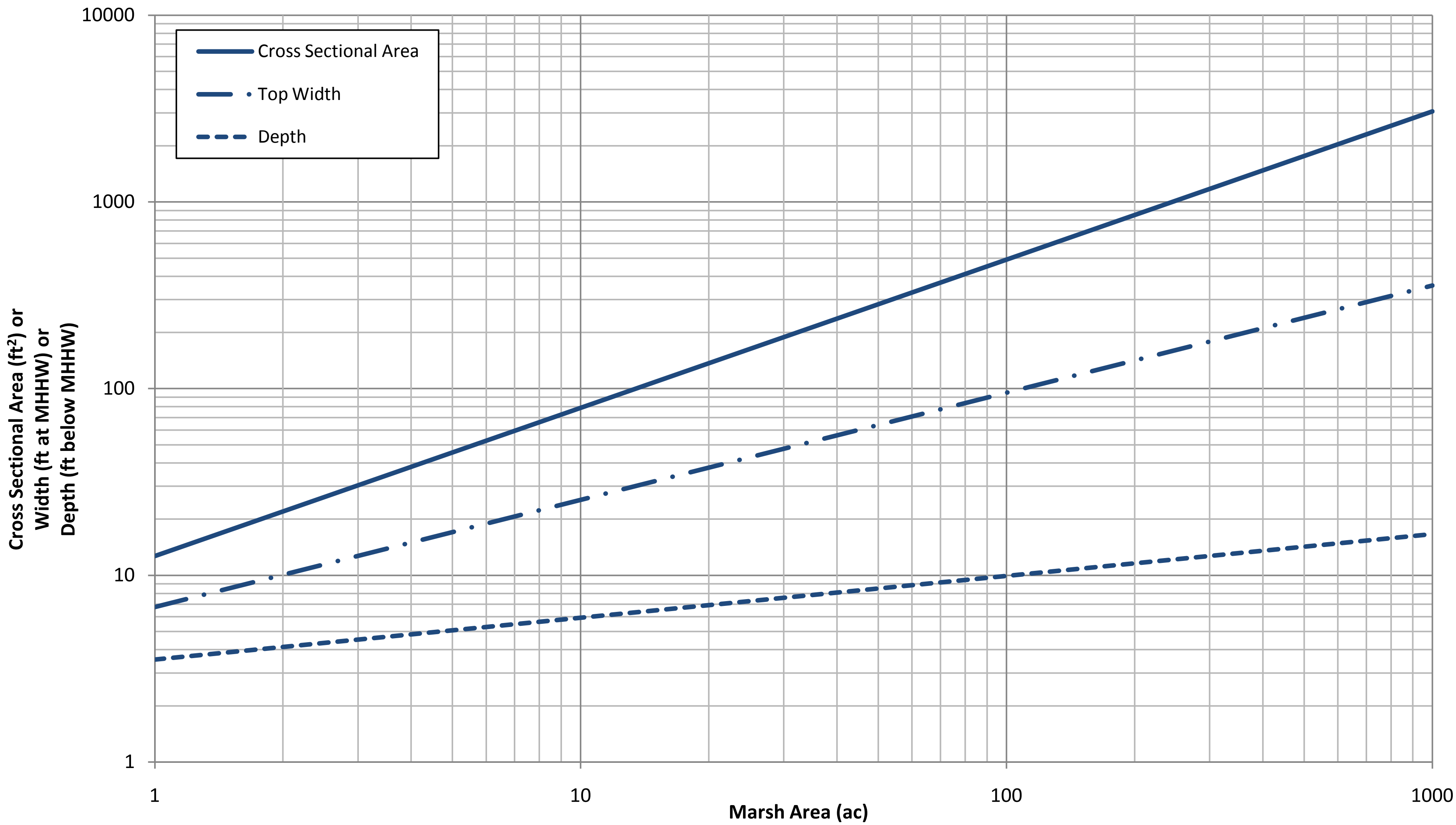


Tide Gage Station: Port Angeles, WA # 9444090
For 10% PSNERP design use only.

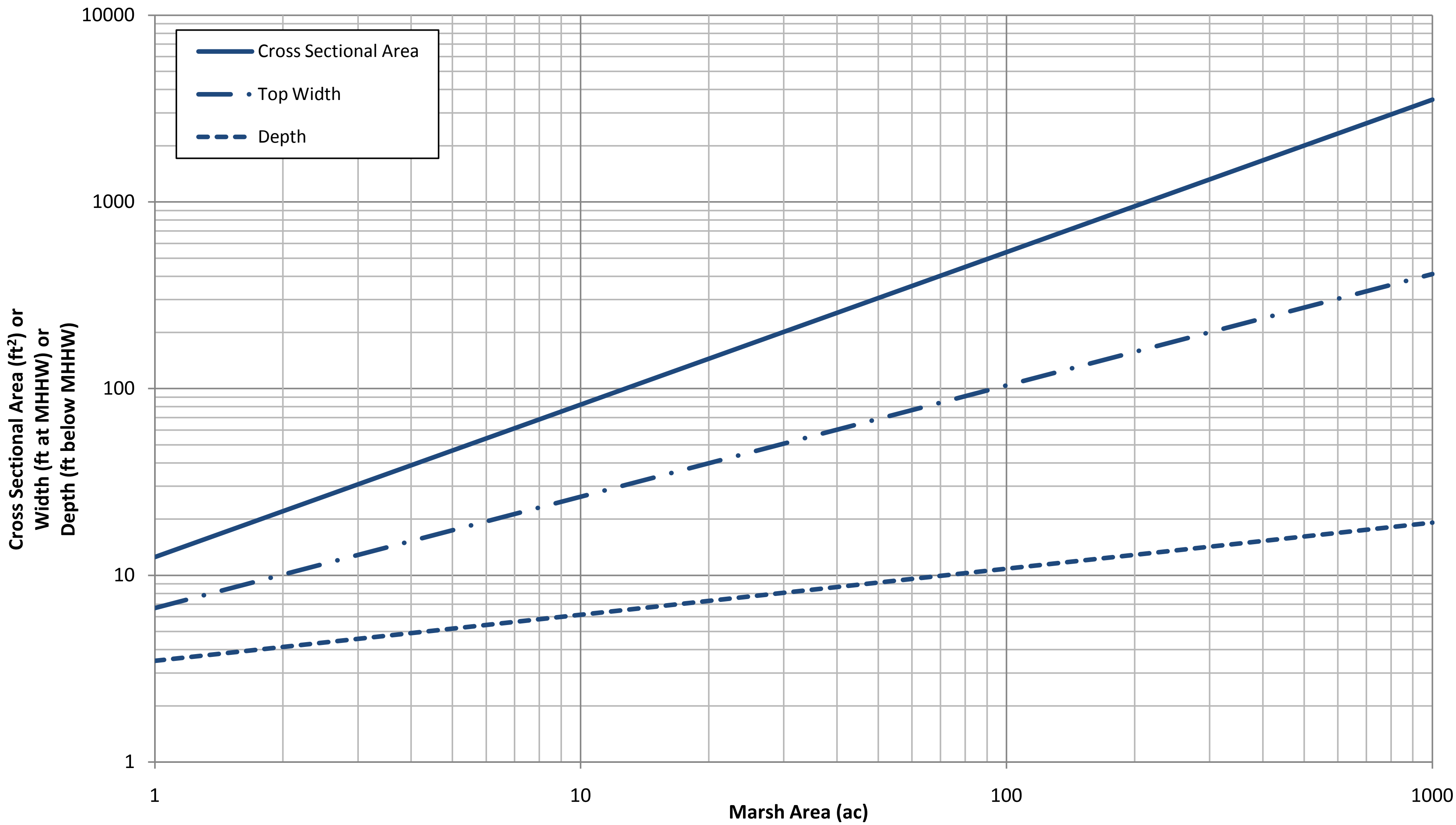
Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure ***6***

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Port Angeles, WA

PWA Ref #: 2036.00



Tide Gage Station: Port Townsend # 9444900
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

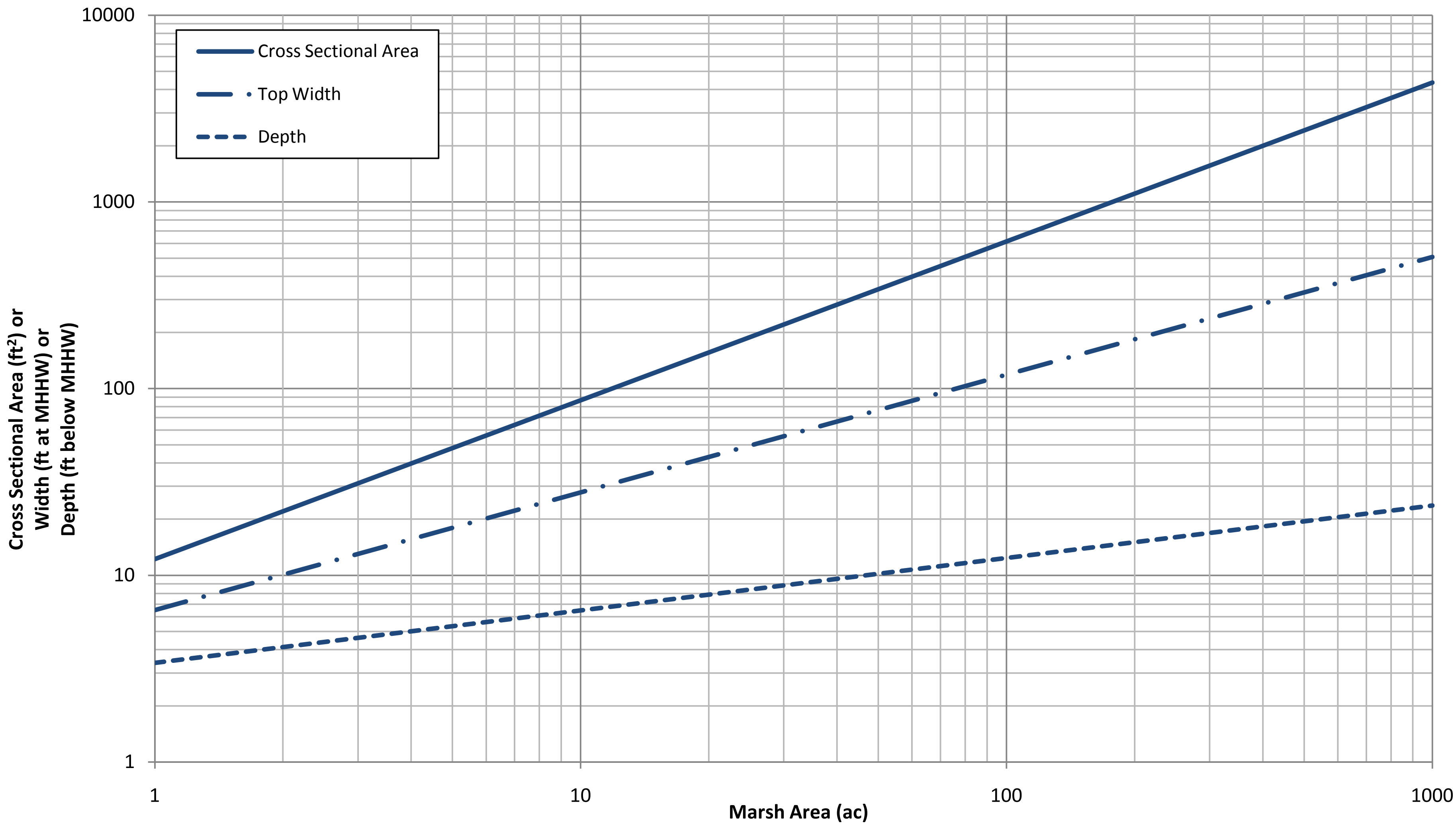
figure 7

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Port Townsend

PWA Ref #: 2036.00

PWA



Tide Gage Station: Everett, WA # 9447659
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure ***8***

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Everett, WA

PWA Ref #: 2036.00

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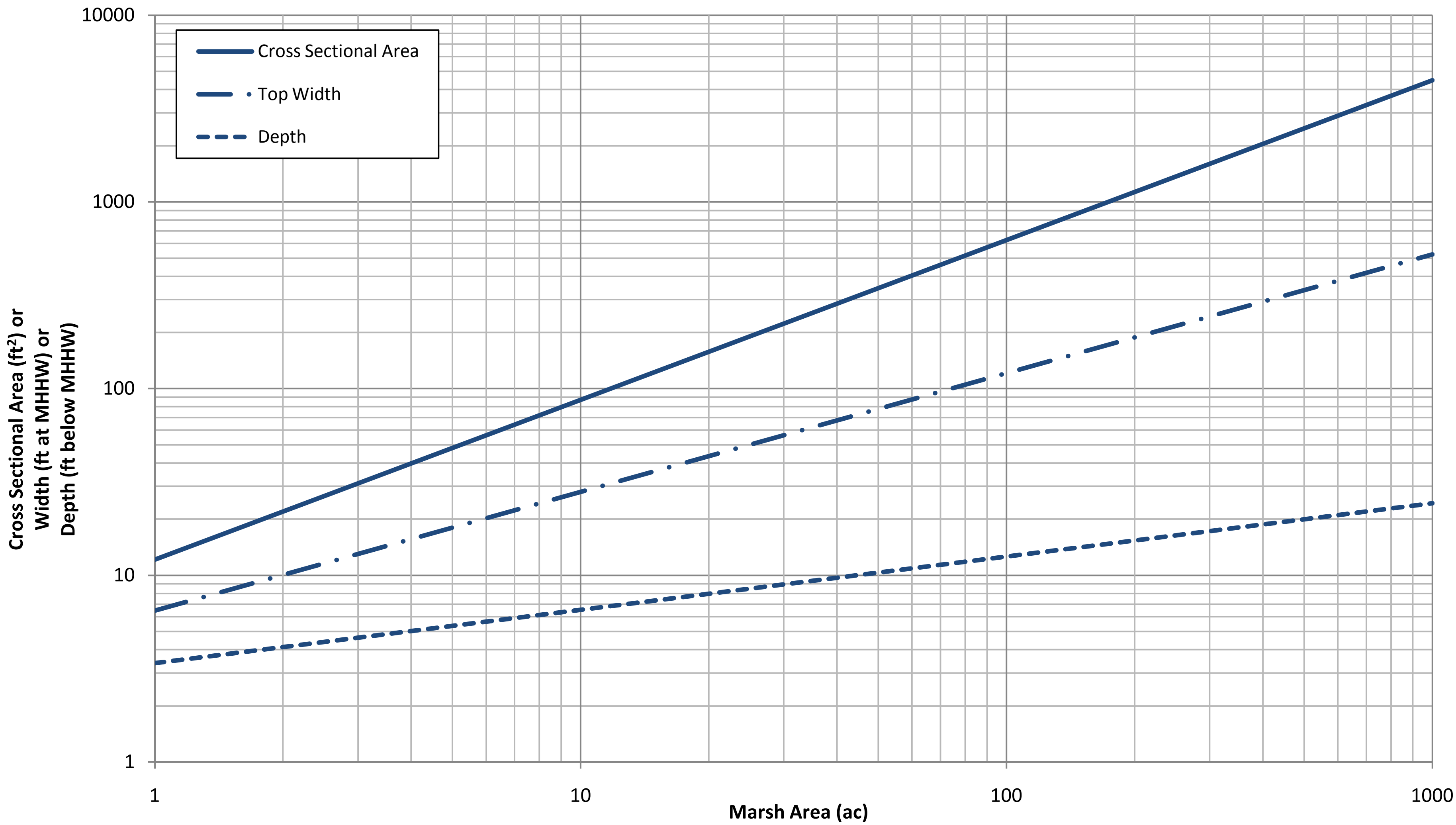

Tide Gage Station: Seabeck, Hood Canal # 9445296
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure **9**

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Seabeck, Hood Canal

PWA Ref #: 2036.00

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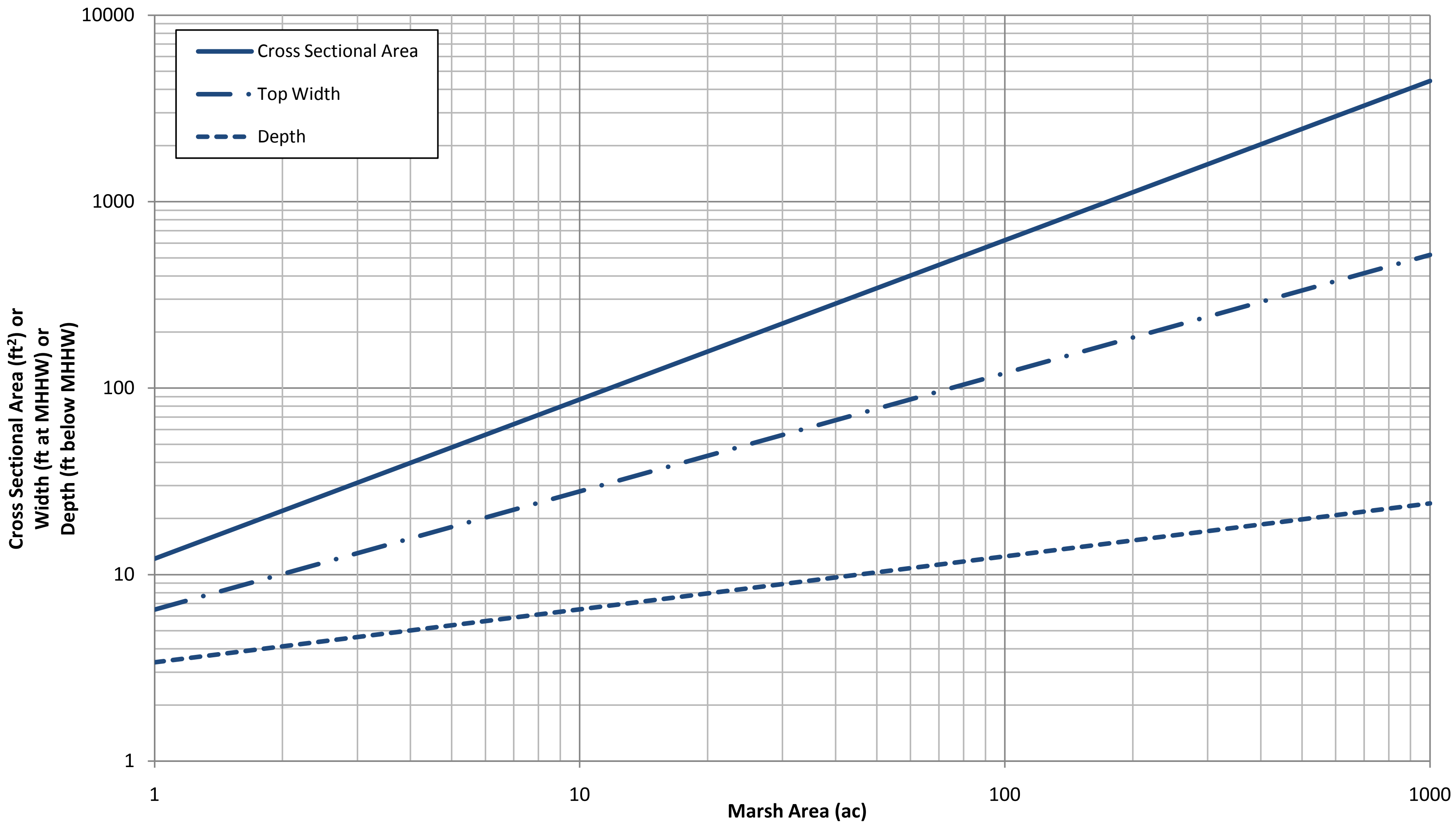

Tide Gage Station: Seattle, Puget Sound # 9447130
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure **10**

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Seattle, Puget Sound

PWA Ref #: 2036.00

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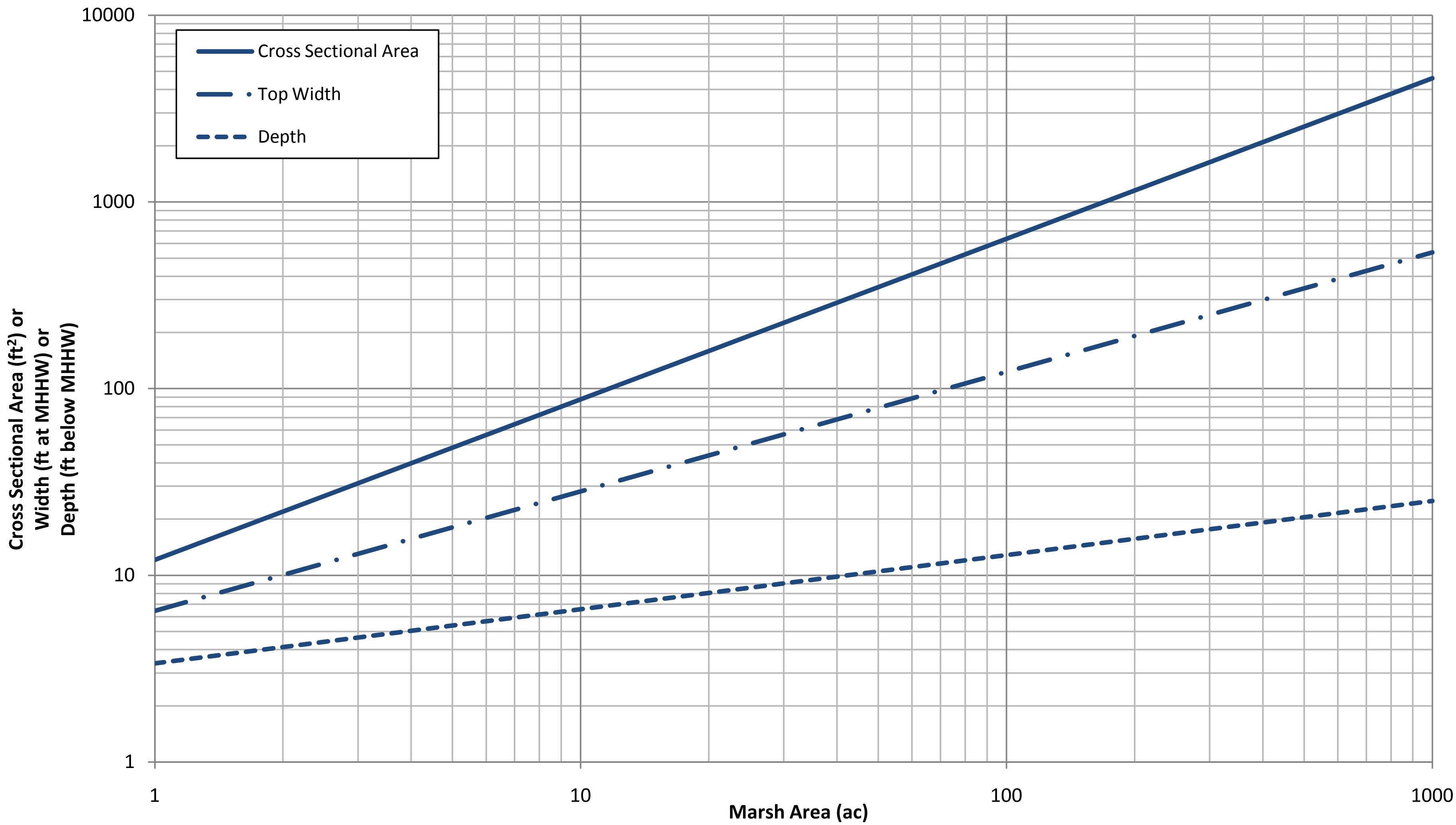

Tide Gage Station: Union, Hood Canal # 9445478
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure ***11***

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Union, Hood Canal

PWA Ref #: 2036.00

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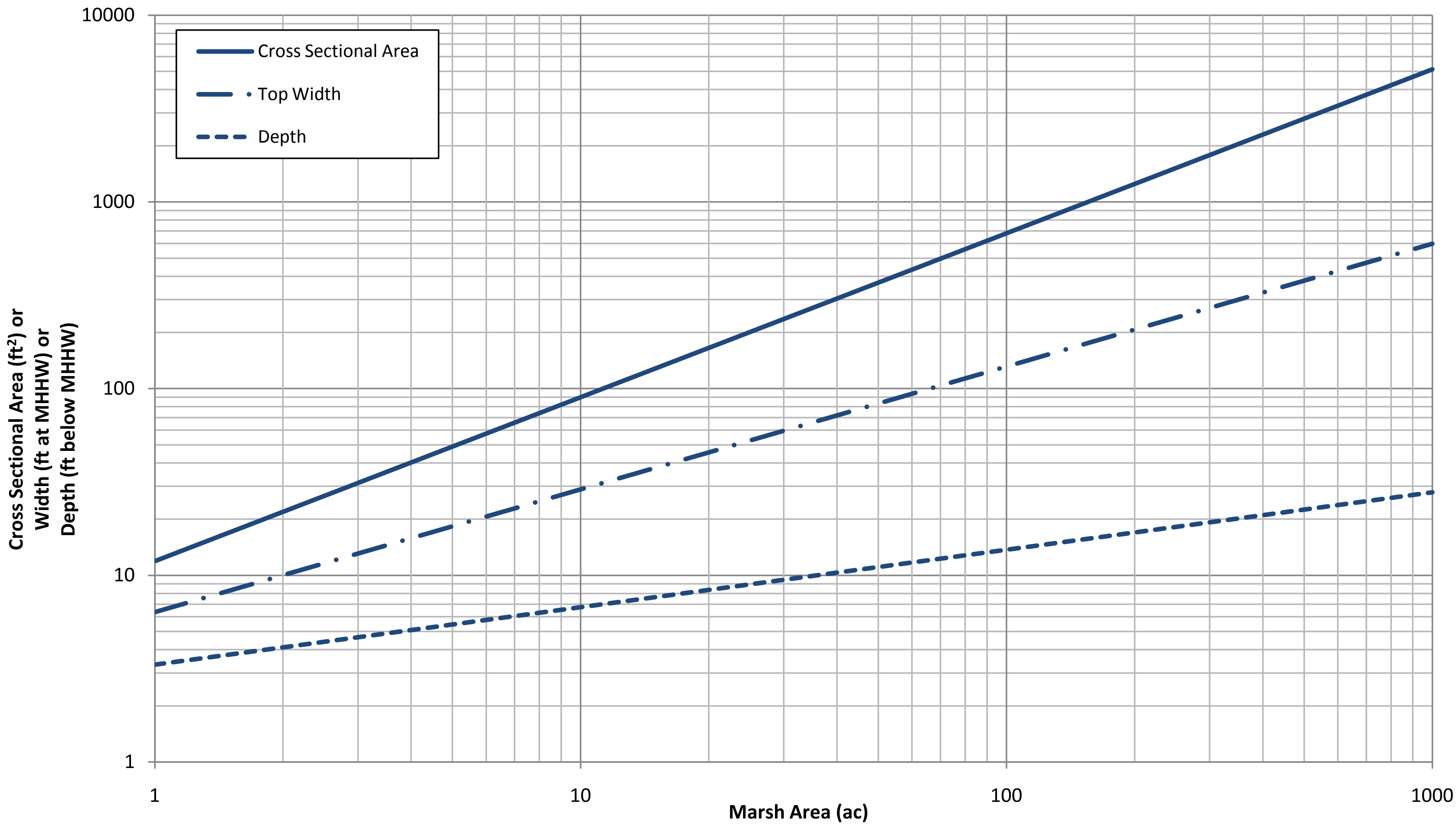

Tide Gage Station: Yoman Point, Anderson Island # 9446705
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure **12**

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Yoman Point, Anderson Island

PWA Ref #: 2036.00

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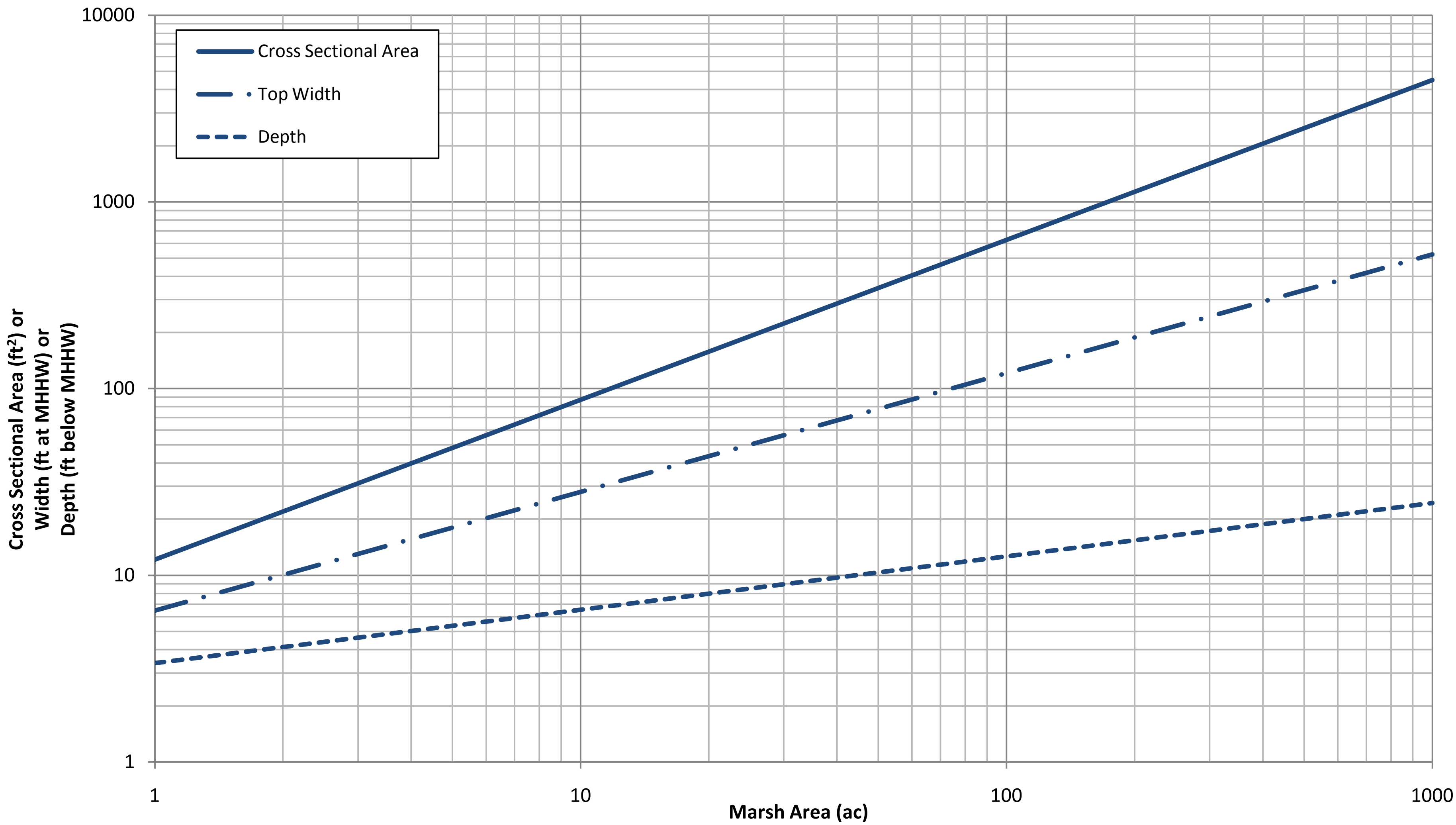

Tide Gage Station: Barron Point # 9446742
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure **13**

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Barron Point

PWA Ref #: 2036.00

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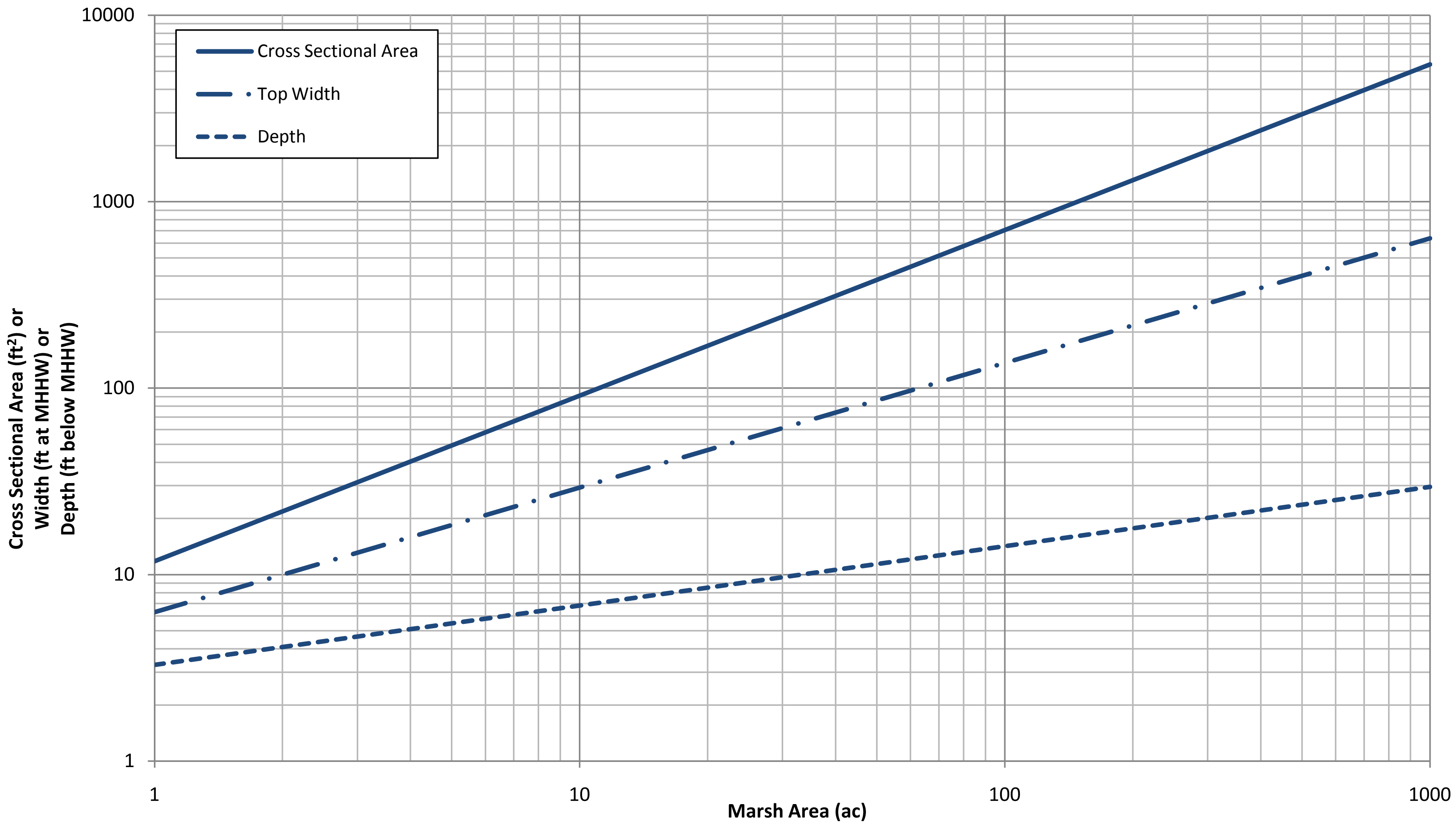

Tide Gage Station: Budd Inlet # 9446807
For 10% PSNERP design use only.

Source: Williams et al. (2002). Regression equations adjusted based on percent increase in diurnal tide range relative to San Francisco Bay.

figure ***14***

Puget Sound Nearshore Ecosystem Restoration Project

Hydraulic Geometry for Budd Inlet

PWA Ref #: 2036.00

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memorandum

date December 22, 2010

to Bob Barnard, Curtis Tanner, PSNERP
Conceptual Design Team

from Phil Williams and Jeremy Lowe

subject PSNERP - Hierarchy of Benefits

1. INTRODUCTION

The purpose of this memo is to describe a hierarchy of benefits that will likely accrue to the natural processes, structure, and function of an ecosystem for variously located and sized openings in crossings of tidal and tidally influenced fluvial channels. We describe benefits in terms of ecosystem process, structure and function. By understanding what these benefits are, and how they impact the nearshore system crossings can be designed to provide maximum benefits more efficiently.

There is a dearth of information regarding the ecological impacts of constructing bridges or culverts across tidally influenced areas in the scientific literature. While hydrological and hydraulic impacts, such as amount and extent of anticipated scouring and longshore transport of sediment, are carefully considered during crossing design, impacts to overall geomorphology and ecological function are not. This may be because many decisions establishing culvert or bridge crossing design practice were made prior to 1969, before the passage of federal and state statutes that require inclusion of environmental impacts. Almost all tidal channel crossings were, and sometimes still are, designed to simply optimize hydraulic conveyance for drainage or design floods at least cost.

The loss of connectivity that occurs when dikes are constructed across wetlands and floodplains is well documented. Embanked bridge crossings can generate similar environmental impacts because they too may restrict the flow of animals, water, sediment, organic plant material and detritus. Today, however, there is an opportunity to assess and rectify the impacts of existing structures through restoration. The question that will need to be addressed is:

> *'what are the tradeoffs between enhanced ecologic benefits and restoration costs for breaches or bridges larger than those required for hydraulic conveyance?'*

The hierarchy of benefits represents a new approach to crossing design by expanding its view from the minimum opening size that the hydraulics requires to one that considers how location and size of openings will impact the morphology and ecology of the ecosystem. This hierarchy of benefits will aid PSNERP decision makers by shedding light on whether a dike removal or a dike modification, and associated construction and monitoring costs, is warranted given particular parameters. It is a tool devised for this specific project, and its development was constrained by existing information and a short time horizon. It can be considered a starting place for cost-benefit analyses that incorporate the geomorphic and ecological aspects of ecosystem function.

2. CONCEPTUAL MODELS OF OPENINGS

PSNERP has described 21 management measures that that can be used to develop and evaluate Puget Sound nearshore restoration alternatives at individual sites. Management Measure 3 (MM3) (Clancy et al. 2009), describes in detail the need for and expected outcomes of dike removal or modification. One expected outcome is higher growth and survival of juvenile salmon in nearshore habitats. The connection between the restoration action (reintroducing the full tidal prism, flooding frequency and duration) and the goal (higher juvenile salmon survival rates) is expressed in a conceptual model that shows how the restoration action will likely restore processes and create structural changes that make the goal possible (see Figure 1).

Similarly, Management Measure 9 (MM9) (PSNERP 2009) describes the need for and expected outcomes of hydraulic modification. MM9 has comparable expected outcomes, and its conceptual model expresses how the restoration action (replace tide gate with open breach) will likely restore processes and create structural changes to improve salmon production and enhance other nearshore functions (see Figure 2).

Both dike removal or modification and hydraulic modification will result in a different type of opening across a tidally influenced area, such as a marsh or delta, than the constricted openings that currently exist. The impacts of the width, location and size of the new opening needs to be considered not only on the tidal and fluvial hydrology, but also on the geomorphic and ecologic processes of the tidally influenced area. This adds an additional dimension to the conceptual model because the rate at which the restoration goals can be achieved will be impacted by breach size.

3. IMPACTS OF CROSSING SIZE ON BARRIER ESTUARIES

Ecologic functioning of a number of barrier estuaries in the Puget Sound is constrained by road crossings. Typically, a road embankment has been constructed that follows the alignment of the natural barrier beach (Figure 3). The connection to tidal waters is often restricted to a single culvert or constricted bridge crossing. In addition, the inlet is fixed in location and high tide storm surge flows across the barrier beach are prevented by the embankment acting as a dike, reducing general flow over the marsh surface toward the bay front and eliminating wave action in the interior of the estuary.

The potential impacts of crossings on barrier estuaries are listed in Table 1 in terms of hydraulic and sedimentary processes and geomorphic and water quality impacts. The size of the inlet is often limited, which may partially or completely block the flow of water and mute the tide. This has implications for the location of head of tide and tidal prism volume. Small inlets may partially or completely block detritus, and large woody debris, and organic plant material from entering the estuary. Intertidal habitats inside the causeway may aggrade at a higher rate than areas outside due to the capture of sediment conveyed by floods from the watershed, or degrade when isolated from deposition of estuarine sediments brought in on the flood tide making these marshes more susceptible to the effects of sea level rise and geologic subsidence.

However these impacts do not occur in isolation. For example, within a barrier estuary alteration of the tidal signal has multiple hydrodynamic and geomorphic impacts including lowering of high tide elevations, raising low tide elevations, raising mean tide elevations, reducing the tidal frame, reducing the tidal prism in the marsh and reducing the tidal excursion. The structural and functional responses include isolation of marsh plains and conversion to fresher water habitats, a reduction in area of intertidal mudflat and sandflat habitat, siltation of tidal channels, an elevated water table affecting marsh to forest transition, a limited fluctuating water table affecting plant growth, atrophy of the channel system due to sedimentation and reduced channel connectivity, and passive advective transport of organisms into the estuary through baroclinic circulation.

The combination of embankment and reduced inlet size reduce both the area of habitat and habitat connectivity which in turn impacts all aspects of ecosystem function: distribution and abundance of species, community dynamics, productivity, and invasive species.

In restoring the ecosystem functions of these estuaries, the main tool is to decrease the hydraulic constriction due to the crossing and increase the habitat connectivity. The size of the opening will determine the type and amount of ecosystem processes that are impacted. The largest possible opening size will eliminate these impacts, while a small opening size will likely produce all of them. Intermediately sized openings will have impacts between these two endpoints.

3.1 Benefits of Increasing Bridge Crossing Size

To illustrate how much ecological benefits increase as opening size increases, we have carried out a first-cut qualitative assessment of five general categories of crossings as described below (see Figure 5):

1. Existing conditions. This assumes a raised embankment along the barrier beach and tidal flow restricted to a single culvert or narrow bridge crossing sized to drain the area landward of the barrier. Tidal regime will be strongly muted. All flows over the barrier beach will be blocked by the embankment.

2. Expand the inlet size with large culverts or bridge crossing to allow regular tidal inundation of the area landward of the barrier. The inlet crossing is designed to be the minimum size to allow the full average diurnal tidal range within the estuary based on the hydraulic geometry for tidal channels. However, tidal velocities will be greater than naturally occurring at the inlet requiring armoring to prevent scour and lateral migration. In addition storm surge tides will still be constricted. All flows over the barrier beach will be blocked by the embankment.

3. Expand the inlet size to allow for a naturally adjusting channel inlet to form. This would require a clear span bridge designed wide enough to allow a natural convex sided inlet channel that can adjust to storm surge tides. All flows over the barrier beach are blocked by the embankment.

4. Expand the inlet crossing to allow for lateral migration of the inlet channel. A bridge would be sized not only for the appropriate inlet channel morphology but also for historic migration width. Laterally meandering inlets have a tendency to ‘reset’ the estuarine drainage system and marsh habitats through bank erosion and migrating flood tide shoals All flows over the barrier beach are blocked by the embankment.

5. Complete removal of tidal barriers. This would include a bridge crossing to allow inlet migration and replacement of the embankment with an elevated causeway on pilings. The former road embankment would be graded down to natural beach crest elevations to allow for storm surge inundation and transport of large woody debris (LWD) into the estuary. The input of LWD creates habitat structure for all trophic levels from algae to invertebrates to fishes and wildlife; it allows for various species to seek shelter, find food, spawn, roost or nest. LWD also impacts sediment movement, potentially creating beach berms. More recently, LWD has been cited in facilitating tidal marsh succession acts by providing a nursery habitat for salt-intolerant species (Maser and Sedell).

Table 1 shows in detail how various process alterations impact ecosystem structure and function. Figures 5 uses this information to qualitatively assign values to restored processes according to opening size.

4. IMPACTS OF CROSSING SIZE AND LOCATION ON RIVER DELTAS

River deltas are dynamic geomorphic landscapes, with river distributary channels that evolve and migrate in response to major floods. They sustain a gradient of wetland habitat types from forested floodplains to forested tidal wetland to tidal marsh and mudflat. Roadways traverse river deltas at many locations in Puget Sound (Figure 4). Typically these have been constructed for convenience on embankments on the flat intertidal areas across the delta front and have concentrated river flows at a single bridge crossing location. Fixing the river channel in this way can significantly reduce the area of active delta. Upstream the river is restrained from avulsing into different distributary channels, resulting in a reduced variety of habitat types, and because of increased sediment deposition, the floodplain and former intertidal habitats aggrade. Downstream, single bridge crossings may partially or completely block the flow of sediment that sustains marsh habitats. Channelizing the outflow of riverine sediment along a single alignment forces delta progradation, changes salinity distribution and causes impacts to natural systems.

For instance, the size and location of bridge crossings are factors that will ultimately determine the viability of a salmon population. A population will become more viable if the size and location of the new opening adds new habitat, connects habitat and increases habitat capacity. New tidal or distributary channels will help to increase all three of these criteria, which alter the distribution and composition of life history strategies and result in an increase in viability.

4.1 Benefits of Increasing Bridge Crossing Size

To illustrate how ecologic benefits of river delta habits could be restored with increasing the size of bridge crossings we have conducted a first cut qualitative assessment of the four alternatives described below (see Figure 6):

1. Existing conditions. Assumes the roadway has been constructed on an elevated embankment that prevents tidal and river flows, and the bridge crossing itself has been sized to the typical design flood. Channel avulsions and distributary channel formation are restricted to the area downstream of the crossing. Elsewhere downstream of the embankment, tidal marshes are not replenished by sedimentation and relict distributary channels silt in. Upstream former intertidal wetlands convert to floodplains and the river channel is prevented from migrating or avulsing with river training structures that simplify habitat structure within the river channel.

2. Additional bridge crossing. The existing bridge crossing is duplicated at a location where a major distributary channel had been blocked off by the embankment. This would encourage a channel avulsion upstream and permit the main river to switch its course between two crossings, doubling the size of the active delta.

3. Extended bridge crossings allow for channel migration. Bridge spans are widened to allow for historic rates of lateral channel migration. Laterally meandering channels 'reset' the fluvial system through bank erosion and subsequent deposition in point bars. This introduces sediment and LWD into channels from stream banks, and promotes the exchange of nutrient-rich soils into the fluvial system. The erosion of banks, and subsequent deposition, results in a dynamic system with a mosaic of habitat types.

4. Extended bridge crossings with road on pilings. This would allow for restoring complete tidal exchange across the delta front. In addition it would allow removal of upstream river embankments allowing for restoration of fluvial processes acting across the delta.

Table 2 shows in detail how various process alterations impact ecosystem structure and function. Figures 6 uses this information to qualitatively assign values to restored processes according to opening size.

4.2 Benefits of Changing Bridge Crossing Location

The amount of ecological benefits derived from restoration efforts is not only influenced by the size of the opening, but also by its location within a watershed. The location of a crossing will be impacted by the tides (Figure 7). A qualitative assessment of tidal effects can be accomplished by expanding upon an approach published in Hydraulic Engineering Circular (Richardson, 2001) that is used to evaluate hydrological processes at crossings. This is, in large part, a measure of the distance from the head of tide to the crossing location. As the distance increases, the volume of tidal prism increases and, in turn, the discharge associated with each tidal cycle. Discharge drives the transport of fluvial and marine sediment in the estuary and scour at crossings. The distance from head of tide is also a measure of the crossing's effect on estuarine processes. Estuarine development (fill, dikes, land use) modifies the level of impact.

Qualitative categories of impact include (see Figure 7):

1. Low impact– the crossing is located near to the head of tide where tidal inundation occurs within the main channel banks, or where the tidally inundated marsh area is small.

2. Medium impact – this category encompasses most of the cases where the road embankment is built in the middle of the delta.

3. High impact– the crossing is located at the marine edge of a marsh, or encloses a large area principally below mean high water. These are cases where tidal volume is large and that significant inundated areas are funneled through a single opening, cutting off flow into distributary channels and over the marsh edge.

5. REFERENCES

Maser, C. and Sedell, J.R., 1994. From the Forest to the Sea: the Ecology of Wood in Streams, Rivers, Estuaries, and Oceans. St. Lucie Press, Delray Beach, FL, 200 pp.

Clancy et al., 2009. Management Measures for Protecting and Restoring the Puget Sound Nearshore. Technical Report 2009-01.

Richardson, 2001. Hydraulic Engineering Circular.

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Table 1. POTENTIAL ADVERSE IMPACTS OF CROSSINGS ON BARRIER ESTUARIES

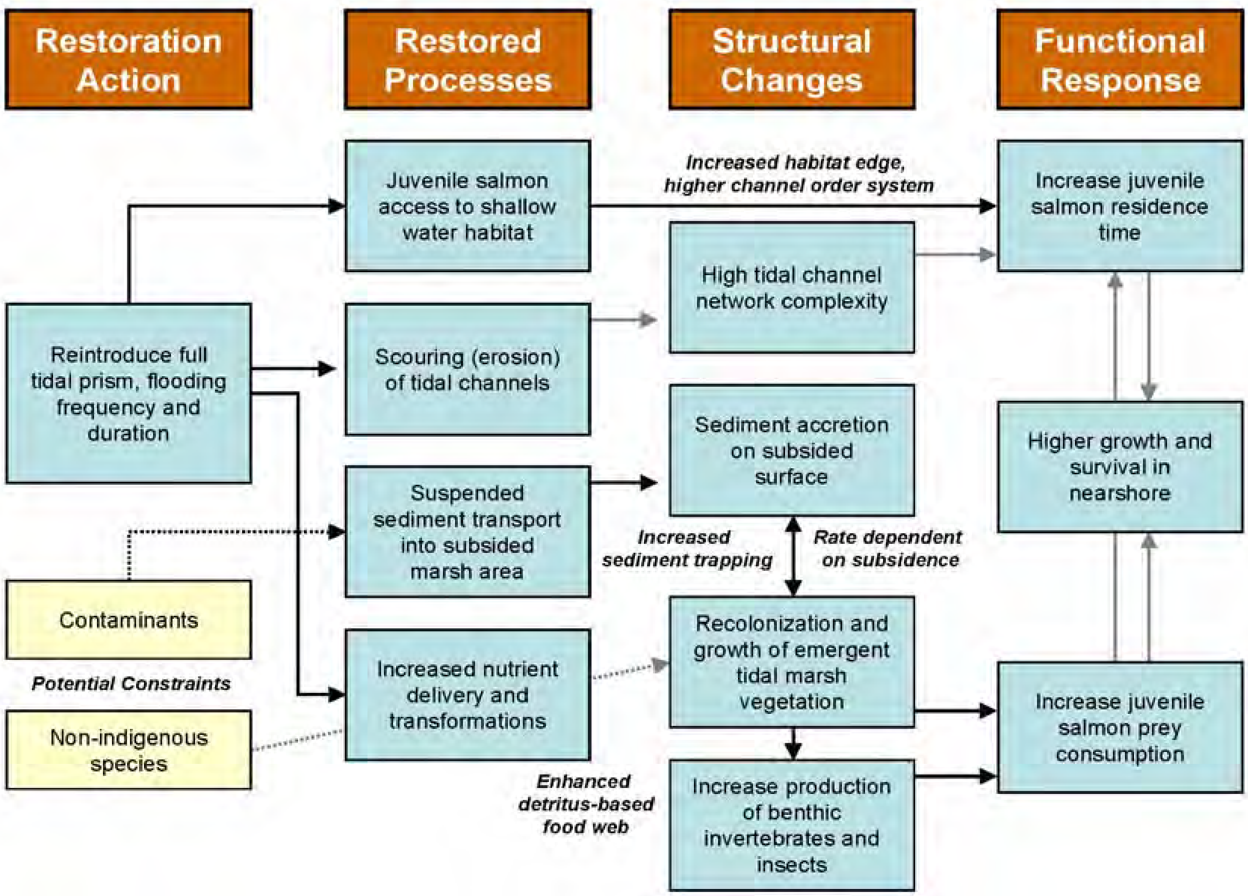
BARRIER ESTUARIES - Assumes culverted entrance, road embankment along beach alignment, watershed relatively small relative to estuary.

BARRIER ESTUARIES	**Process**	**Structural Impact**	**Functional Response**
HYDRAULIC/ HYDRODYNAMIC PROCESS IMPACTS	Alteration of tidal stage characteristics **(#2)**	Lowering of high tide elevations	Isolation of marsh plains, conversion to fresher habitats
		Raising low tide elevations	Reduction in area of intertidal mudflat/sandflat habitat
		Raising mean tide elevations	Water table elevated affecting marsh to forest transition
		Reduction in tidal frame	Water table fluctuation limited affecting plant growth
		Reduction in tidal prism in marsh	Channel system atrophies through sedimentation; reduced channel connectivity
		Reduced tidal excursion	Passive advective transport of organisms in and out of estuary diminished
	Alteration of salinity distribution **(#5)**	Vertical salinity stratification degraded through mixing	Reduction of passive transport of organisms into estuary through baroclinic circulation
		Salinity mixing zone length truncated	'Squeezing'and reduction of brackish zone habitats
	Elimination of storm surge overwash across beach **(#3, 4)**	Transport of large woody debris into marsh	Habitat heterogeneity reduced
		Mobilization of detritus due to storm surge wave action eliminated	Export of nutrients to estuary reduced
SEDIMENTARY PROCESS IMPACTS	Alluvial sedimentation altered by backwater affects	Fine sediment accumulates on marsh plain	Shift to upland habitats
		Coarse sediment accumulates in tidal channels	Loss of blind channel habitat
	Estuarine sedimentation limited by reduction in tidal flows **(#1)**	Reduced tidal prism reduces sediment delivery to marsh plain, causes lowering relative to tidal frame	Reduced productivity of marsh vegetation
		Increased turbidity in tidal channels due to loss of marsh plain sediment sink	Adverse affect on benthic organisms and eelgrass
GEOMORPHIC IMPACTS	Alteration of entrance channel morphology from broad shallow to narrow	Increased tidal velocity through entrance creates scour holes	Increased fish mortality
		Channel location fixed instead of lateral migration affecting ebb and flood shoal extent	Adverse affect on benthic organisms
		Fixed channel location may lead to permanent closure of confined marsh by longshore drift	Eliminates exchange of water, sediment, nutrients and organisms
	Atrophied tidal drainage system	Tidal channels shallower	Degraded estuarine habitat
		Dendritic tidal channel system becomes disconnected	Estuarine habitat degraded
	Marsh plain elevations changed	Lowered marsh plain	Reduced marsh productivity
		Areas raised by alluvial sedimentation	Change to freshwater or upland species
WATER QUALITY IMPACTS	Increased residence time **(#6)**	Reduction in tidal exchange	Algal blooms in marsh channels, anoxic in poorly drained holes
		Reduction in tidal excursion	Export of water column productivity to larger estuary limited
	Accumulation of toxics	Reduced tidal scouring allows accumulation of polluted sediments from watershed	Toxic affects on organisms
		Reduced residence time means concentration of dissolved pollutants in water column is higher	Toxic affects on organisms

Table 2. POTENTIAL ADVERSE IMPACTS OF CROSSINGS ON RIVER DELTAS

RIVER DELTAS - Assumes single bridge crossing across main river sized for major river flood on piers, road embankment across rest of delta.

RIVER DELTAS			
HYDRAULIC/ HYDRODYNAMIC PROCESS IMPACTS	Alteration of fluvial flows	Concentration of flood flows at one discharge point raises flood stages upstream	Shift from marshplain to floodplain ecologic processes
		Elimination of out of bank flows upstream increases discharge, scouring and flood velocities in main channel	Reduction of fish refuge habitat and shallow water habitat
	Alteration of estuarine tidal flows	Deeper main channel can extend tidal influence further upstream	Introduction of predators upstream
	Alteration of estuarine salinity distribution	Extension of single channel into deeper waters creates abrupt fresh to salt water mixing zone	Adverse impacts on anadramous migration
		Elimination of distributary channels alters spatial distribution of mixing zones across delta front.	Reduction in brackish zone, adverse impact on shellfish
		Elimination of distributary channels reduces linear extent of salinity transition zones	Reduction in anadramous fish habitat
SEDIMENTARY PROCESS IMPACTS	Alluvial sedimentation	Increased sedimentation on marshplain/floodplain upstream	Conversion from tidal marsh to floodplain habitats
		Reduced sediment delivery and erosion where distributary channels have been blocked	Loss of intertidal habitats
		Coarse sedimentation concentrated at mouth of single channel, instead of being distributed along multiple channels across delta front	Loss of habitat heterogeneity
	Estuarine sedimentation	Estuarine mudflats not replenished during flood events –fine alluvial sediments lost to deep water	Loss of intertidal mudflat/sandflat habitat
		Reduced flood tide suspended sediment concentrations reduce marshplain sedimentation rates	Loss of productivity and area of marshplain habitat
	Large wood accumulation	More export of large woody debris	Reduction in complexity of channel habitat
GEOMORPHIC IMPACTS	Spatial reduction of active delta	Reduction in area	Loss of benefits of large scale ecologic processes
		Simplification of deltaic system	Reduction in heterogeneity of habitats, loss of alternate migratory routes
		Disruption of natural gradient of wetland habits from floodplain to mudflat	Loss of connectivity of habitats, fragmentation of habitats
		Delinking of river channel from marshes	Adverse affect on migrating fish
	Main river channel changes	Deeper river channel	Simplification of fish habitat
		Channel location fixed	Reduction in habitat complexity derived from meandering processes
		Extension of delta lobe to deeper water reducing channel slope, increasing in-channel sediment deposition	Loss of watershed derived nutrients to estuarine system
	Distributary channel changes	Remnant distributary channel atrophies	Loss of channel edge habitat and migration routes
	Marshplain system changes	Marshplain erosion	Loss of marsh area, conversion to mud/sand flat
		Marshplain lowering	Reduction of productivity
	Mudflat changes	Mudflat lowering	Loss of mudflat habitat



Source: PSNERP Management Measures (2009)

figure ***1***

PSNERP Concept Engineering

Conceptual model of dike removal or modification

PWA Ref# 2036.01

PWA

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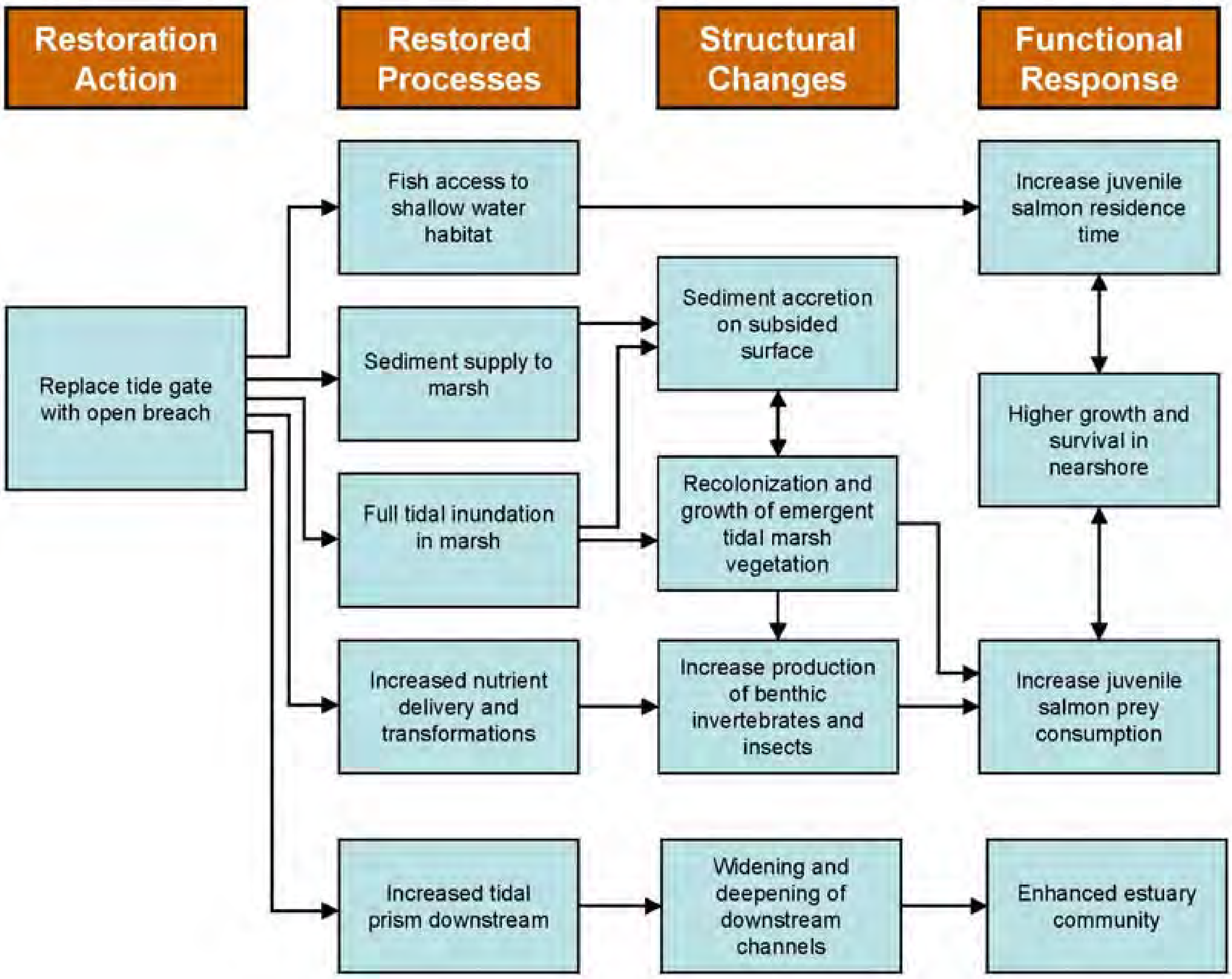
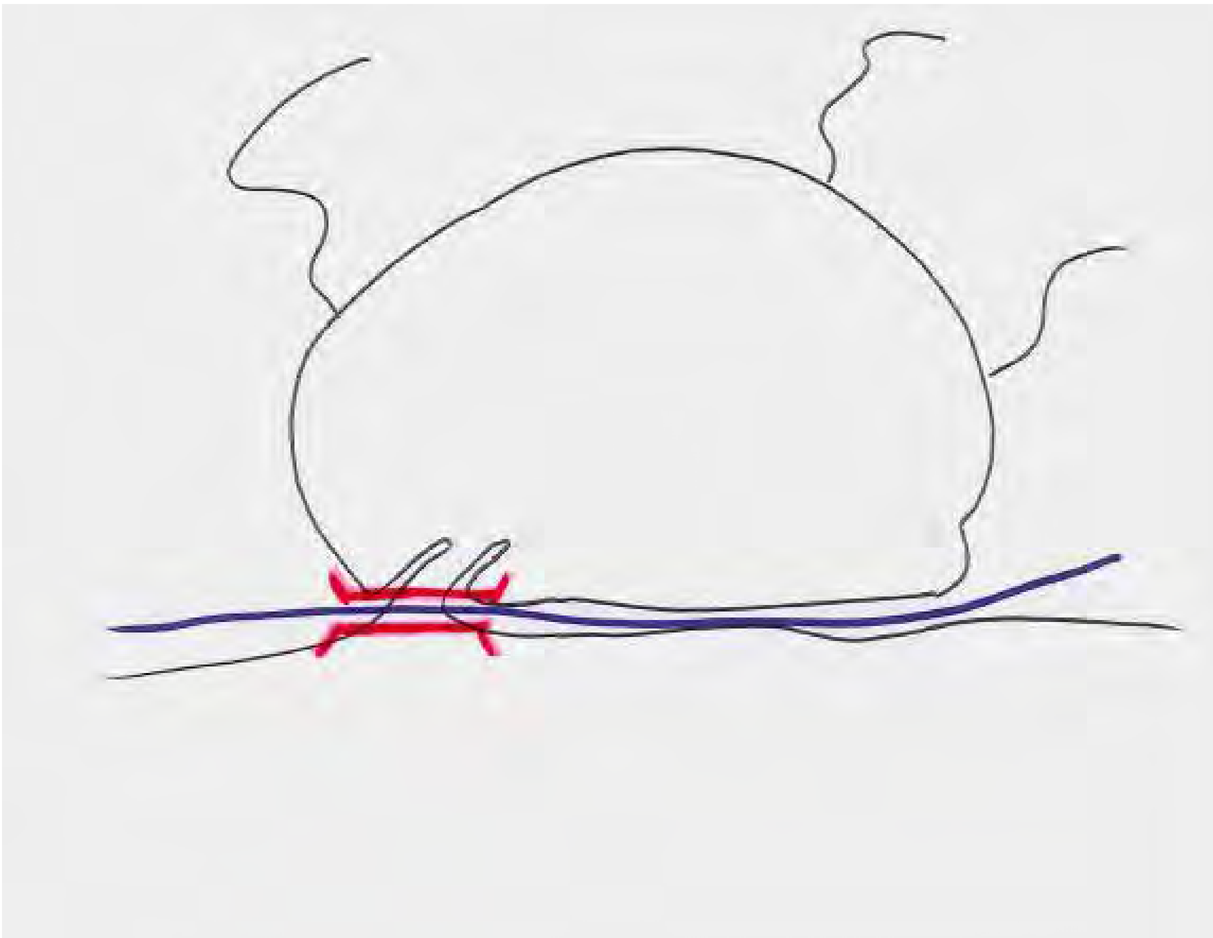


Source: PSNERP Management Measures (2009)	*figure* **2** *PSNERP Concept Engineering* Conceptual model of hydraulic modification PWA Ref# 2036.01 — PWA

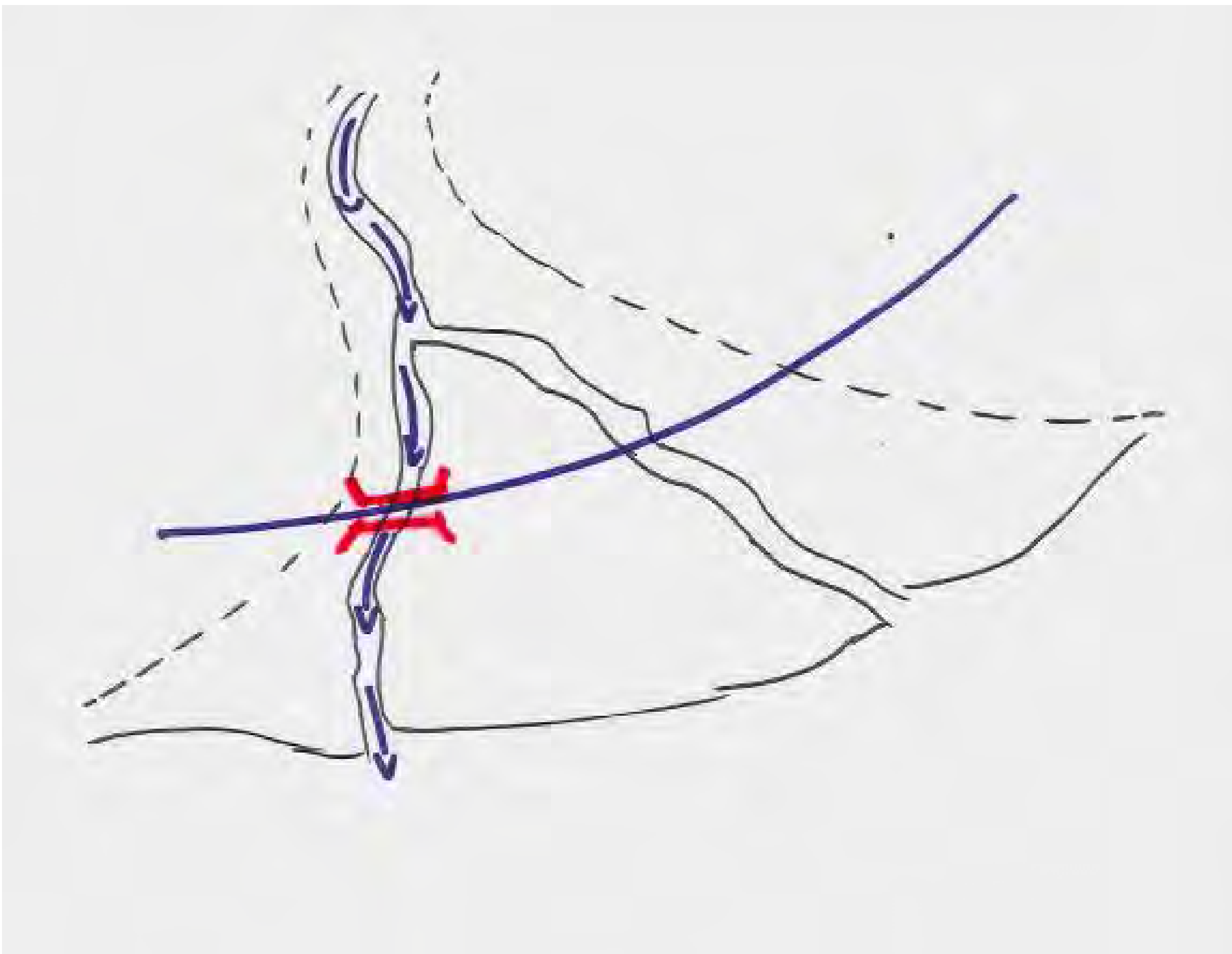


Source:

figure ***3***
PSNERP Concept Engineering

General layout of barrier estuary crossing

PWA Ref# 2036.01



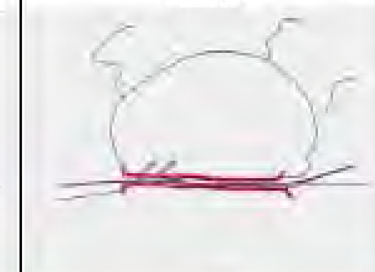
Source:

figure ***3***
PSNERP Concept Engineering

General layout of river delta crossing

PWA Ref# 2036.01

BARRIER ESTUARIES

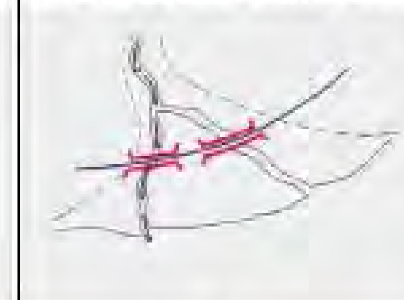
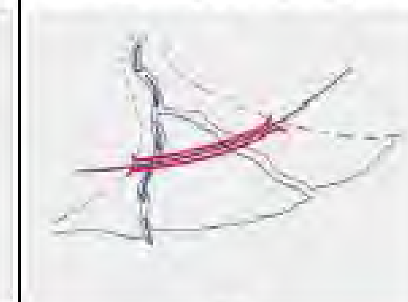
	Restored Process	Culvert	Culvert sized to allow inundation of marsh plain	Bridge sized to inlet channel morphology	Bridge sized for migration width	Causeway
						
	Fish Access to Shallow Water Habitat / Connectivity		0.6	0.8	1.0	1.0
1	Estuarine Sediment Supply to Marsh		0.8	0.9	0.9	1.0
2	Full Tidal Inundation in Marsh		0.7	1.0	1.0	1.0
3	Increased Nutrient Delivery to the Bay		0.4	0.7	0.8	1.0
	Increased Tidal Prism Downstream		0.7	1.0	1.0	1.0
4	Disturbance Regimes (LWD)/ Habitat Heterogeneity		0.0	0.2	0.4	1.0
5	Salt Water Mixing		0.4	0.9	1.0	1.0
6	Tidal Exchange Processes over Time		0.6	0.7	0.8	1.0
	Total Benefit		4.2	6.2	6.9	8.0

Source:

figure **5**
PSNERP Concept Engineering

Benefits of widening crossings of a barrier estuary

PWA Ref# 2036.01

Restored Process	Culvert not feasible	Bridge sized to 10-100 year flood	Two Bridges	Two Bridges sized for migration width	Two Bridges and Causeway
					
Fish Access to Shallow Water Habitat / Connectivity		0.1	0.4	0.6	0.8
Estuarine Sediment Supply to Marsh		0.1	0.3	0.5	1.0
Distributary Channel Formation		0.1	0.4	0.6	0.8
Increased Nutrient Delivery to the Bay		0.2	0.4	0.6	[illegible]
Channel Migration		0.0 0.0	0.1	0.4	0.8
Salt Water Mixing		0.2	0.6	0.6	0.8
	Total Benefit	0.7	2.2	3.3	5.2

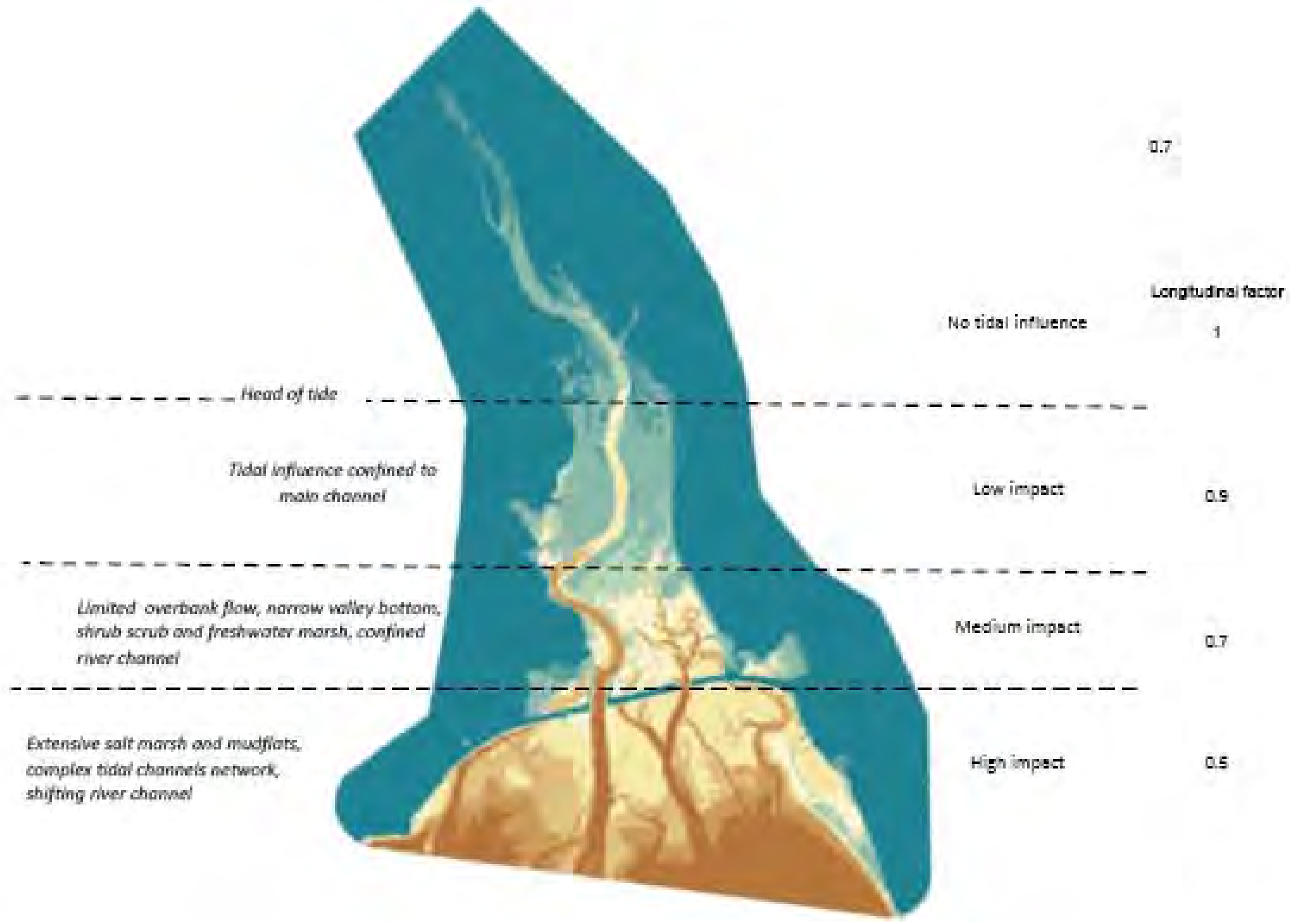
Source:

figure **6**
PSNERP Concept Engineering

Benefits of widening crossings of a river delta

PWA Ref# 2036.01

PWA



Source:

figure ***7***
PSNERP Concept Engineering

Location of crossing

PWA Ref# 2036.01

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