APPENDIX A
MANAGEMENT OF DREDGING RESIDUALS

TABLE OF CONTENTS

List of Tables

ACRONYMS/ABBREVIATIONS

≤	less than or equal to
BST	Bellingham Shipping Terminal
D/F	dioxin/furan
DMMP	Dredged Material Management Program
Ecology	Washington State Department of Ecology
EDR	Engineering Design Report
gm/cm^3	grams per cubic centimeter
Hg	mercury
mg/kg	milligrams/kilogram
MLLW	mean lower low water
ng/kg	nanograms/kilogram
ppt	parts per trillion
PRDI	Pre-remedial Design Investigation
SMS	Sediment Management Standards
SWAC	surface-weighted average concentration
TEQ	toxicity equivalent concentration
Waterway	Whatcom Waterway

1 INTRODUCTION

The engineering design for the Whatcom Waterway (Waterway) Cleanup in Phase 1 Site Areas includes the application of best management practices to address dredging residuals. The evaluation presented in this appendix has been incorporated into the design of the cleanup action as described in the main text of the Engineering Design Report (EDR).

The generation of dredge residuals is inherent to the dredging process, whatever the method (USACE 2008a; USACE 2008b; Patmont and Palermo 2007; Bridges et al. 2010). These residuals result from the loose sediment that re-deposits on the surface during each dredging pass. Best management practices include use of appropriate dredging methods and equipment, use of appropriate dredge pass thicknesses, use of cleanup pass dredging as appropriate, and placement of clean cover material to mix with the dredging residuals. These actions collectively minimize the resulting quantity and concentration of contaminants remaining in the completed dredge area.

The evaluation presented herein provides an estimate of the quantity and quality of dredging residuals to be generated during dredging in areas within Unit 1C, near the Bellingham Shipping Terminal (BST). Also included are design recommendations for management of these dredging residuals. As described in the Cleanup Action Plan (Ecology 2007), post-dredging residual sediment contamination for Unit 1C will include the use of best practices and placement of clean sand cover. Based on the evaluation conducted in this Appendix, a sand cover thickness of 6 inches will be applied within this area after dredging.

This appendix does not address dredging residuals to be generated within the Inner Waterway. In the Inner Waterway, dredging is to be followed by placement of an engineered cap. This sequential cap placement addresses any residuals that may be produced during dredging. Therefore, no additional residuals management evaluation is required in these areas.

2 METHODOLOGY

The evaluation of the resulting post-dredge residual chemical concentration and the required thickness of the post-dredge cover within Unit 1C of the Outer Waterway was determined using guidance provided by the U.S. Army Corps of Engineers (USACE) (USACE 2008a). The approach taken is the mass-balance method for determining final concentration of the remaining material as a mixed layer of cover material and residual material. This methodology has been utilized for many sediment cleanup projects (e.g., on the Grasse River, Hudson River, Fox River, and for the Lower Willamette Group in Portland Harbor).

Within Unit 1C, four sediment cores were placed during pre-remedial design investigations, each representing conditions in respective quadrants of the unit's footprint (Anchor QEA 2010). For purposes of this discussion, each quadrant is referenced by the sediment core label.

Required dredge elevations for each quadrant were established based on a review of historical dredge depths (Anchor QEA 2012) and the analytical results of the sediment cores. The total depth of dredging is assumed to have an allowable over-dredge of one foot.

2.1 Target Contaminant Levels

The cleanup levels established in the Whatcom Waterway Consent Decree (Ecology 2007) specify compliance with the sediment quality standard (SQS) for total mercury. The mercury SQS is 0.41 milligrams per kilogram (mg/kg).

Dioxin/furan (D/F) compounds are present in surface and subsurface sediments within most of Puget Sound's urban bays. These compounds are derived from multiple historical and ongoing sources. The Washington State Department of Ecology (Ecology) has conducted sampling and issued a final data report documenting a regional background concentration of D/Fs in Bellingham Bay of 15 ng/kg (Ecology 2015).

Current surface D/F concentrations within Unit 1C surface sediments were measured as part of the Pre-remedial Design Investigation (PRDI) (Anchor QEA 2010). Surface sediments collected during that study indicated existing concentrations equal to 14.8 ng/kg TEQ. To comply with Ecology anti-degradation requirements under the Sediment Management Standards (SMS) regulations, cleanup and residuals management within Unit 1C will need to result in concentrations equal to or less than15 ng/kg.

2.2 Model Parameterization

Analysis of dredging residuals requires estimation of the quantity and quality residuals generated during each dredging pass. Primary factors affecting the dredge residuals calculations include the following:

- In situ chemical concentration of the target dredge material
- In situ bulk density of the target dredge material
- Dredge cut thickness
- Presence or absence of debris content or hard bottom conditions (affecting potential bucket loss)
- Over-dredge allowance

The effectiveness of the residuals management strategy depends on the following factors:

- Target post-remedy surface concentration
- Thickness of the final production or cleanup pass dredge cut
- Thickness of cover material
- Bulk density of cover material
- Chemical concentration of cover material

Table A-1 contains descriptions of the input parameters for the dredging residuals analysis, including the source of each parameter, and the value assumed for this evaluation.

Table A-1
Dredge Residual Calculation Parameter Input Values

Parameter	Source	Input Value or Range
In situ dry density of target dredge material	Derived from laboratory results of samples from in-water borings (PRDI)	0.63 gm/cm3
Required dredge cut thickness	Survey data and results of vibracores (PRDI)	4 to 12 feet

Parameter	Source	Input Value or Range
Residual loss	Figure 1 of Patmont and Palermo (2007)	3.5% to 7.5%
Depth of allowable over-dredge	Typical range for environmental dredging	1 foot
Target chemical concentration (required values)	Target concentrations based on cleanup levels as defined in the Cleanup Action Plan (Ecology 2007) and SMS anti-degradation requirements	≤0.41 mg/kg for mercury ≤15 ng/kg for D/F
Thickness of production dredge passes	Value is solved for iteratively based on targeted cleanup levels	2.0 to 7.0 feet
Thickness of final production/ cleanup dredge passes	Required dredge cut thickness less the thickness of the initial production dredge pass	2 to 5 feet
Thickness of post-dredge sand cover material	Expected value based on past experience with similar projects	minimum of 6 inches
Bulk density of post-dredge cap material	Assumed value for loose, pluviated sand	1.47 gm/cm3
Chemical concentration of cover material	Estimated based on past experience with common borrow source material	See Table A-2

Notes:
≤ = less than or equal to
D/F = dioxin/furan
gm/cm^3 = grams per cubic centimeter
mg/kg = milligrams per kilogram
ng/kg = nanograms per kilogram
PRDI = *Pre-Remedial Design Investigation* (Anchor QEA 2010)
SMS = Sediment Management Standards

2.3 Residuals Calculations

The dredging residuals management was analyzed for each quadrant within Unit 1C, and then for the surface-weighted average concentration (SWAC) of Unit 1C. This analysis was performed using two different assumptions for the quality of the cover material. Table A-2 provides the initial concentration estimates for mercury and D/Fs for the two evaluated cover materials (Cover 1 and cover 2). These two cover materials are intended to represent the range in naturally occurring mercury and D/F concentrations that may be present in quarry sands used for residuals management.

Table A-2
Typical Chemical Concentrations of Cover Material

Material	Concentration of Mercury (mg/kg)	Concentration of Dioxins/Furans (ng/kg)
Cover 1	0.07	1.0
Cover 2	0.00	2.0

Notes:
mg/kg = milligrams per kilogram
ng/kg = nanograms per kilogram

The following represents the formulation for a two-pass production dredge scenario used for the analysis.

Equations for removal in two dredge passes:

$$C_T = \frac{C_S \cdot S \cdot \rho_S + C_{R1,2} \cdot R_2 \cdot \rho_{R2}}{R_2 \cdot \rho_{R2} + 1}$$

$$C_{R1,2} = \frac{C_1 \cdot R_1 \cdot \rho_{R1} + C_2 \cdot L_2 \cdot \rho_2}{R_1 \cdot \rho_{R1} + L_2 \cdot \rho_2}$$

Where:

C_T = targeted/resulting chemical concentration
S = sand thickness (6 inches)
L = thickness of the production lift
R = thickness of dredge residual (percentage of previous production pass and residual thickness, if present)
C = in situ chemical concentration
ρ = in situ dry density

(subscripts denote the production pass and/or layer; 1 = initial pass; 2 = subsequent pass; S = residual cover material; R = residual sediment)

Table A-3

Surface Area of Quadrants for Unit 1C

Quadrant	Surface Area (ft^2)	Percent of Total Surface Area (%)
1C-105	41,175	17.5
1C-106	68,625	29.1
1C-107	47,250	20.0
1C-108	78,750	33.4

Notes:

ft^2 = square feet

3 DISCUSSION AND RESULTS

Table A-4 presents the results of residual management estimates for mercury. Wood debris was reported in the boring logs. Therefore, residuals estimates considered that debris could increase the potential for generated residuals. The bucket loss for this evaluation is estimated to range between 3.5 and 7.5 percent of the dredge volume and is based on the case histories compiled by Patmont and Palermo (2007). The range of values represents a typical range for bottom conditions that might be anticipated in the presence of significant debris. A mid-range estimate of the percent residuals generated during dredging was selected to be approximately 6 percent.

For each quadrant of Unit 1C, a residuals management analysis was performed for the residual generation of 3.5, 6, and 7.5 percent to determine the resulting average surface concentration of each cover material (see Table A-4). Table A-5 shows the resulting SWACs for mercury for a minimum cover thickness of 6 inches for each cover material.

Table A-4
Residuals Management Evaluation for Each Unit 1C Quadrant – Hg

		Percent Residuals Generated					
		3.5%		6%		7.5%	
Quadrant	Neatline (feet, MLLW)	Cover 1 (mg/kg)	Cover 2 (mg/kg)	Cover 1 (mg/kg)	Cover 2 (mg/kg)	Cover 1 (mg/kg)	Cover 2 (mg/kg)
1C-05VC[1]	-36	0.092	0.028	0.126	0.070	0.156	0.100
1C-06VC	-40	0.111	0.048	0.137	0.078	0.151	0.094
1C-07VC[1]	-40	0.142	0.082	0.207	0.151	0.247	0.196
1C-08VC	-40	0.155	0.091	0.210	0.149	0.241	0.182

Notes:
1. A minimum of two production dredge passes are required.
Hg = Mercury
mg/kg = milligrams per kilogram
MLLW = mean lower low water

Table A-5
Residuals Management Evaluation for Unit 1C – Hg SWAC

Quadrant	Surface-weighted Average Concentration at 6% Generated Residuals (mg/kg)		Percent of Targeted Concentration (0.41 mg/kg)	
	Cover 1[1]	Cover 2[1]	Cover 1	Cover 2
1C-05VC[1]	0.126	0.07	30.7%	17.1%
1C-06VC	0.137	0.078	33.4%	19.0%
1C-07VC[1]	0.207	0.151	50.5%	36.8%
1C-08VC	0.21	0.149	51.2%	36.3%
Unit 1C SWAC	**0.18**	**0.12**	**43.9%**	**29.3%**

Notes:

1. Values represent a percent residuals generation of 6%.

Hg = Mercury

mg/kg = milligrams per kilogram

SWAC = Surface weighted average concentration

The above results show that none of the scenarios analyzed are expected to exceed the 0.41 mg/kg site-specific cleanup level established for mercury.

Tables A-6 and A-7 show the resulting D/F concentration estimates for each quadrant and for the overall Unit 1C SWAC. This analysis assumes a minimum placed cover thickness of 6 inches (with each of the two evaluated cover materials differing in its initial D/F concentrations).

The evaluation results (Table A-6) show that none of the scenarios analyzed are expected to exceed the existing surface concentration (15 ng/kg) of D/Fs. The worst-case analysis (where 7.5 percent residuals are generated and Cover 2 is placed in the more heavily impacted quadrant 1C-05VC) yields an estimated concentration of 5.50 ng/kg, which is well below starting surface concentrations. Table A-7 shows that, under the more probable case of 6 percent generated residuals and use of a residuals cover material consistent with Cover 2 (2.0 ng/kg initial D/F TEQ), the final D/F SWAC within Unit 1C will be 3.31 ng/kg.

Table A-6
Residuals Management Analysis for each Unit 1C Quadrant – D/Fs

Quadrant	Neatline (feet, MLLW)	Percent Residuals Generated					
		3.5%		6%		7.5%	
		Cover 1 (ng/kg)	Cover 2 (ng/kg)	Cover 1 (ng/kg)	Cover 2 (ng/kg)	Cover 1 (ng/kg)	Cover 2 (ng/kg)
1C-05VC	-36	2.48	3.40	3.80	4.65	4.65	5.50
1C-06VC	-40	1.66	2.59	2.17	3.06	2.50	3.35
1C-07VC	-40	1.63	2.52	2.47	3.30	3.10	3.87
1C-08VC	-40	1.50	2.45	1.91	2.82	2.17	3.06

Notes:
D/F = dioxin/furan
MLLW = mean lower low water
ng/kg = nanograms per kilogram

Table A-7
Residuals Management Analysis for Unit 1C – D/Fs SWAC

Quadrant	Surface-weighted Average Concentration (ng/kg)		Percent of Targeted Concentration (14 ng/kg)	
	Cover 1[1]	Cover 2[1]	Cover 1	Cover 2
1C-05VC[1]	3.80	4.65	27.1%	33.2%
1C-06VC	2.17	3.06	15.5%	21.9%
1C-07VC[1]	2.47	3.30	17.6%	23.6%
1C-08VC	1.91	2.82	13.6%	20.1%
Unit 1C SWAC	**2.43**	**3.31**	**17.4%**	**23.6%**

Notes:
1. Values represent a percent residuals generation of 6%.
D/F = dioxin/furan
ng/kg = nanograms per kilogram

The above results demonstrate that the placement of a 6-inch layer of residuals management cover will result in final surface concentrations that comply with the site-specific cleanup levels for mercury, and that comply with SMS anti-degradation requirements for D/Fs.

4 RESIDUALS MANAGEMENT RECOMMENDATIONS

Based on the results presented in the previous section, we recommend the following residuals management strategies:

Table A-8
Residuals Management Design Recommendations

Quadrant	Recommendations
1C-05VC	• Removal to -36 feet MLLW with an over-dredge allowance of 1 foot • At least one production dredge pass and one final production/cleanup pass • Minimum 6-inch clean cover placement
1C-06VC	• Removal to -40 feet MLLW with an over-dredge allowance of 1 foot • At least one production dredge pass and one final production/cleanup pass • Minimum 6-inch clean cover placement
1C-07VC	• Removal to -40 feet MLLW with an over-dredge allowance of 1 foot • At least one production dredge pass and one final production/cleanup pass • Minimum 6-inch clean cover placement
1C-08VC	• Removal to -40 feet MLLW with an over-dredge tolerance of 1 foot • At least one production dredge pass and one final production/cleanup pass • Minimum 6-inch clean cover placement

Note:
MLLW = mean lower low water

5 REFERENCES

Anchor QEA (Anchor QEA, LLC), 2010. *Pre-Remedial Design Investigation Data Report.* Whatcom Waterway Site Cleanup. Prepared for Port of Bellingham. August 2010.

Anchor QEA, 2012. *Preliminary Design Conceptual Report.* Whatcom Waterway Site Cleanup. Prepared for Port of Bellingham. 2012.

Bridges, T.S., K.E. Gustavson, P. Schroeder, S.J. Ells, D. Hayes, S.C. Nadeau, M.R. Palermo, and C. Patmont, 2010. *Dredging Processes and Remedy Effectiveness: Relationship to the 4 Rs of Environmental Dredging*. Integrated Environmental Assessment and Management. February 10, 2010.Ecology (Washington State Department of Ecology), 2015. *Bellingham Bay Regional Background Sediment Characterization Draft Data Evaluation and Summary Report*. Publication No. 15-09-044. January 2015.

Ecology (Washington State Department of Ecology), 2015. *Bellingham Bay Regional Background Sediment Characterization Final Data Evaluation and Summary Report*. Publication No. 15-09-044. February 2015.

Patmont, C. and M.R. Palermo, 2007. *Case Studies of Environmental Dredging Residuals and Management Implications*. Paper D-066, in: E.A. Foote and G.S. Durell (Conference Chairs), *Remediation of Contaminated Sediments—2007*. Proceedings of the Fourth International Conference on Remediation of Contaminated Sediments. Savannah, Georgia. January 2007.

USACE (U.S. Army Corps of Engineers), 2008a*. Technical Guidelines for Environmental Dredging of Contaminated Sediments.* ERDC/EL TR-08-29. U.S. Army Corps of Engineers. September, 2008.

USACE, 2008b. *The 4 Rs of Environmental Dredging: Resuspension, Release, Residuals, and Risk*. Prepared by Todd S. Bridges, Stephen Ells, Donald Hayes, David Mount, Steven C. Nadeau, Michael R. Palermo, Clay Patmont, and Paul Schroeder, ERDC/EL TR-08-4. U.S. Army Corps of Engineers. January 2008.

APPENDIX B
GEOTECHNICAL EVALUATION

TABLE OF CONTENTS

Tables

Figures

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
Anchor QEA	Anchor QEA, LLC
ARCS	Assessment & Remediation of Contaminated Sediments
ASCE	American Society of Civil Engineers
bgs	below ground surface
BST	Bellingham Shipping Terminal
CSZ	Cascadia Subduction Zone
DE	design event
GDM	Geotechnical Design Manual
GP	Georgia-Pacific Corporation
H:V	horizontal to vertical
LL	liquid limit
LRFD	Load and Resistance Factor Design
MCE	maximum considered earthquake
MHHW	mean higher high water
MLLW	mean lower low water
MSL	mean sea level
NOAA	National Oceanic and Atmospheric Administration
NRDI	Non-remedial Design Investigation
OCR	over-consolidation ratio
PGA	peak ground acceleration
PI	plasticity index
PRDI	Pre-remedial Design Investigation
Project	Whatcom Waterway Cleanup in Phase 1 Site Areas
psf	pounds per square foot
SDC	Seismic Design Category
SPT	Standard Penetration Test
USGS	U.S. Geological Survey
Waterway	Whatcom Waterway
WSDOT	Washington State Department of Transportation

1 INTRODUCTION

This report presents the results of the geotechnical engineering evaluation performed by Anchor QEA, LLC (Anchor QEA), for the remedial design for the Whatcom Waterway (Waterway) Cleanup in Phase 1 Site Areas Project (Project). The proposed remedial actions include dredging and capping of areas within the Waterway that have waterfront or overwater structures, ongoing upland operations, capped and uncapped slopes created through dredging, and areas with unfavorable subsurface conditions. The specific areas that were evaluated are summarized in the next section (see Figures B-1 and B-2).

1.1 Locations Studied

The proposed work requires remediation efforts adjacent to several waterfront structures and shorelines and within an actively managed navigation channel. Presented is a brief description of the existing conditions of the areas studied and the work to be performed.

1.1.1 Outer Waterway

This region of the Waterway includes the dredging in front of the Bellingham Shipping Terminal (BST) at Berths 1 and 2, located near the mouth of the Waterway. The pier is approximately 1,400 feet in length and is primarily a timber-pile supported concrete and asphalt deck.

1.1.2 Inner Waterway

This region of the Waterway includes the dredging in front of the Georgia-Pacific Corporation (GP) dock, nearshore dredging and capping within dredge Unit 2A, and dredging and capping in the open-water area of the multi-use Waterway within the northeastern portion of dredge Unit 2C. The GP dock occupies approximately 1,400 feet of shoreline along the southeast side of the Waterway and is primarily a timber-pile supported wood dock. The Waterway in this area is maintained as a multi-purpose navigation channel. The mudline elevations within the channel generally trend deeper from the northeastern end out toward Bellingham Bay and range from -18 to -35 feet mean lower low water (MLLW). The northeastern end is described as a tidal flat and is not actively maintained for navigation purposes.

1.1.3 South Shoreline

The region referred to as the South Shoreline includes the clarifier bulkhead and pile-supported clarifier tank, as well as an approximately 100-foot section of shoreline immediately to the northeast. The clarifier bulkhead and tank will be removed and the shoreline is to be re-graded and capped. The existing clarifier bulkhead is approximately 270 feet in length.

1.1.4 Central Waterfront

The Central Waterfront includes three key shoreline regions – former Chevron property, other Central Waterfront properties (including the Maple Street bulkhead), and the Meridian Pacific property (ASCE 7-10).

- **Former Chevron property:** This shoreline region is located on the northern side of the Waterway at the western end of the shoreline and consists of approximately 400 feet of existing bulkhead, a timber-pile supported wood dock, and wooden fender piles. Areas with riprap are present along the shoreline.
- **Other Central Waterfront properties:** This shoreline region is located on the northern side of the Waterway and includes vessel moorage, a ramp for boat haul-out and launch, and the existing Maple Street bulkhead. Areas with riprap are present along the adjacent shorelines to the east and west of the existing Maple Street bulkhead. Construction will include a replacement bulkhead (designed to meet ASCE 7-10) at the location of the existing Maple Street bulkhead and paving the upland shoreline area just west of the Meridian Pacific property boundary to support crane operations. Shoreline Structures (i.e., sheetpile walls) will be constructed in the areas immediately east and west of the existing Maple Street bulkhead.
- **Meridian Pacific property:** This shoreline region is located to the east of the other Central Waterfront properties and work involves stabilization of an existing concrete bulkhead as part of the remedial actions.

1.1.5 Log Pond

The Log Pond (Unit 4) is located along the southeast side of the Waterway and lies between the BST and the GP dock. Shoreline features include approximately 300 lineal feet of an upper and lower (two-step) timber bulkhead that are spaced approximately 14 feet apart along the BST warehouse. The BST warehouse is set back into the upland area approximately 10 feet from the upper bulkhead. The remainder of the shoreline area includes over-steepened slopes covered in debris with timber piling present in some locations. Shoreline debris removal and capping will be conducted for this unit and includes placement of a rock buttress at the existing BST timber bulkhead.

1.2 Previous Studies

The analyses preformed as part of this geotechnical evaluation are based on the subsurface explorations and findings included in the Pre-remedial Design Investigation (PRDI) and the Non-remedial Design Investigation (NRDI) performed by Anchor QEA, as referenced in the Engineering Design Report. The following other studies were also consulted:

- CH2M Hill. *Draft Report: Conceptual Design of Aerated Lagoon – Phase 1*. May 1977.
- Converse, Davis, Dixon. *Final Report: Geotechnical Exploration – Phase II: Proposed Submarine Outfall and Secondary Treatment Lagoon, Bellingham Bay, Washington*. March 1978.
- Anvil Corporation. *Report of Settlement Monitoring of Dike. Interim Report No. 3*. January 1979.
- GeoEngineers. *Geotechnical Engineering Services, Clarifier Bulkhead*. January 1989.
- W.D. Purnell and Associates. *Engineering Geology and Geotechnical Investigation of Proposed Pier and Retaining Wall*. July 1988.
- Hart Crowser. *Sediment Core Logs*. September 1996.
- Remediation Technologies, Inc. *Roeder Avenue Warehouse Feasibility Analysis*. October 1996.
- Anchor Environmental, L.L.C. *Addendum No. 4: Georgia-Pacific Aerated Stabilization Basin (ASB) Supplemental Data Collection Work Plan/Sampling and Analysis Plan (SAP) Remedial Investigation/Feasibility Study, Whatcom Waterway Site, Bellingham, Washington*. June 2003.

1.3 Report Structure

This report consists of three main sections:

- Section 1 provides a brief overview of the areas studied and data sources referenced for the geotechnical evaluation.

- Section 2 discusses the subsurface conditions underlying the site.
- Section 3 discusses the geotechnical engineering analyses performed for each of the following design elements:
  - *Dredge Prism Slide Slopes* – Includes a discussion of the impact on shoreline features and evaluates slope stability for dredge cuts.
  - *Cap Design* – Includes estimates for consolidation of the soft sediments underlying the cap, an evaluation bearing capacity for cap placement, and the stability of caps on slopes.
  - *Source Control and Replacement Bulkhead Design* – Includes active, passive, and seismic earth pressures for source control and replacement bulkhead designs, and allowable pullout stress for tieback design at the Maple Street replacement bulkhead.
  - *Geotechnical Engineering Recommendations for Other Structural Elements* – Includes active, passive, and soil modulus parameters for the structural evaluation of existing, pile supported, over-water structures (i.e., GP dock and BST dock).
  - *Slope Stability* – Includes an evaluation of the factor of safety for post-dredge and post-cap scenarios of dredge prism side slopes, shorelines, and shoreline source control and bulkhead structures.
  - *Seismicity* – Includes an evaluation of seismic hazards for source control and replacement bulkhead design as per ASCE 2010 and effects of seismicity with respect to remedial design features.

2 SITE GEOTECHNICAL CONDITIONS

Subsurface geotechnical conditions at the site were investigated by Anchor QEA as part of the PRDI and NRDI. For these two investigations, 54 borings were performed with depths below ground surface (bgs) ranging from approximately 25 to 90 feet. This information was supplemented with explorations conducted by others in the project area. The deepest exploration was performed by others upland of the GP dock and was approximately 170 feet bgs (Purnell 1988). Using the available surface explorations, geologic cross-sections of the locations studied were developed and are presented in Figures B-3.1 through B-3.6.

2.1 Subsurface Soil Conditions

Whatcom Waterway is located along the banks of Whatcom Creek, where it enters Bellingham Bay. The upland areas at the site were historically developed by placing dredge and other fill materials atop the native soils. Underlying the fill materials are alluvial soils that are attributed to deposition from Whatcom Creek. Underlying the alluvial soils are deposits of glacial outwash and glaciomarine drift with interbedded sand lenses/layers. The glaciomarine drift is underlain by bedrock.

General descriptions of the soil layers identified from the borings advanced at this site are presented below in order from the ground surface downward.

2.1.1 Fill

Fill encountered in borings completed at the site is identified as loose to dense, light brown to dark gray, primarily gravel and sand with variable silt content and frequent wood debris. This material is observed in the upland soil borings and is present behind the existing shoreline bulkheads and is assumed to extend to the shoreline areas. The material is unstratified and is locally variable. The thickness of this layer ranges from approximately 15 to 30 feet, with an approximate elevation range from -6 to -20 feet MLLW at the base of the fill along the Central Waterfront shoreline and from approximate elevation range -6 to -16 feet MLLW upland of the existing clarifier bulkhead and GP dock at the South Shoreline area.

2.1.2 Organic Silt

Organic silt encountered in borings completed at the site is identified as very soft to soft, dark gray to black, with varying sand and gravel content and wood. This deposit is observed in nearly all in-water locations and varies in thickness from approximately 3 to 20 feet but is typically 8 to 10 feet thick in most areas. The water content of this material varies from about 80 to more than 300 percent with an average value of approximately 150 percent. During sampling of this material, the static weight of the drill rods was sufficient to advance the sampler, meaning a blow count of zero (blows per foot for the Standard Penetration Test [SPT]) was recorded. The elevation of the bottom of this unit ranges from approximate elevation -42 feet MLLW in the middle of the outer Waterway to elevation -20 feet MLLW east of the existing clarifier bulkhead.

2.1.3 Sand

Sand encountered in borings completed at the site is identified as loose to dense, light to dark gray, fine grained sand with little to moderate silt content, trace shells, and occasional gravel. This deposit is observed primarily in upland soil borings and was rarely observed to be continuous between borings. The thickness of this unit ranges from approximately 0 to 15 feet and the bottom elevation of this unit ranges from approximate elevation -25 to -44 feet MLLW.

2.1.4 Silty Sand

Silty sand encountered in borings completed at the site is identified as loose to medium dense, gray, with varying gravel and silt content. Thin, discontinuous layers of this deposit (up to 10 feet thick) were observed in most areas of the Waterway. Thicker deposits (greater than 10 feet) were observed along the southeast side of the Waterway from approximately the middle of the GP dock toward Bellingham Bay. The bottom of this unit ranges from approximate elevation -15 to -45 feet MLLW.

2.1.5 Clay

Clay encountered in borings completed at the site is identified as very soft to stiff, gray, with variable silt, sand, and gravel content and wood. The material is predominately a low to medium plasticity clay with water content ranging from approximately 15 to 40 percent.

From consolidation test data collected for the PRDI and from data collected by others, the clay is expected to have an over-consolidation ratio (OCR) between 1.5 and 3. This deposit is locally referred to as Bellingham Glaciomarine Drift and was observed in nearly all borings. Thickness of the deposit in and near the Waterway ranges from approximately 10 to more than 80 feet. Undrained strength tests conducted in this unit indicate the clay is highly variable, with peak undrained strengths ranging from 250 to 2,000 pounds per square foot (psf) and no clear correlation between shear strength and depth. The bottom elevation of this unit ranges from -26 feet MLLW to elevations below -70 feet MLLW

2.1.6 Bedrock

Bedrock encountered in borings completed at the site is identified as unweathered to weathered, gray sandstone (Chuckanut Formation). Rock was encountered in soil borings near the clarifier bulkhead with the contact depths trending deeper toward Bellingham Bay. The shallowest observed elevation was -26 feet MLLW and occurred at the east end of the clarifier bulkhead. The weathered zone of the bedrock was not well delineated and is assumed to be a minimum of 5 feet thick. Mapping of the bedrock contact was previously performed by GeoEngineers (1989). In general, the elevation of the bedrock contact deepens from the eastern most end of the Waterway toward the mouth of the Waterway and from the south shore of the Waterway toward the middle of the Waterway. Bedrock was not encountered in borings completed on the north side of the Waterway.

2.2 Sea Level and Nearshore Groundwater

Since Whatcom Waterway is open to Bellingham Bay, it is subjected to tidal fluctuations and seasonal variations in tides. According to the National Oceanic Atmospheric Administration (NOAA), this region of Puget Sound experiences a mean higher high water (MHHW) of +8.51 feet and a mean sea level (MSL) of +4.95 feet. Both values are measured relative to the MLLW, which serves as the vertical datum at +0.00 feet (NOAA 2003).

The groundwater elevations nearest to the shoreline are influenced by the tide as seawater transmits through the soil during high and low water stages. For analysis purposes, the groundwater elevation for both shorelines of the Waterway was assumed to be static and equal to the sea level. For high and low water scenarios, the MHHW and MLLW values were assumed, respectively. An MSL value of +5.0 feet MLLW was assumed for the analysis of long-term slope stability. A mid-range value between the MHHW and MLLW was rounded to +4 feet MLLW and assumed for analysis of the design-level seismic events. Table 2-1 presents a summary of the water condition assumed for each analysis.

Table 2-1
Water-level Assumptions for Analyses Performed

Analysis Performed	Short Term (feet; MLLW)	Long Term (feet; MLLW)[1]
Earth Pressures	+0.0	+5.0
Seismic Earth Pressures	+4	---
Slope Stability[2]	+0.0, +8.5	+5.0
Seismic Slope Stability	+4	---
Seismic Hazards	+4	---

Notes:
1. Long-term analysis is valid for static conditions only.
2. Both high- and low-water scenarios were analyzed.

3 DISCUSSION OF GEOTECHNICAL ENGINEERING ANALYSIS

This section discusses and present the results of the analyses performed for the design elements mentioned in Section 1.3:

- Dredge prism side slopes
- Cap design
- Source control and replacement bulkhead wall design
- Slope stability
- Seismicity

The various remedial components were evaluated by computing factors of safety during the analysis. Target factor of safety values assumed for the design of dredge prism side slopes, wall designs, slope stability analyses, and seismic hazards are based on various references (Duncan and Wright 2005; WSDOT 2011; Fang 1991), as described in each subsection below.

Conclusions of the analyses are provided at the end of each subsection where recommendations for the remedial design are provided.

3.1 Dredge Prism Side Slopes

Dredging is planned at the locations shown on Figure B-2. The analysis of the submerged dredge prism side slopes includes areas that are capped and not capped to assess stable slope angles for the range of conditions present throughout the Waterway. The stability of the submerged dredge prism side slopes was evaluated using limit equilibrium methods implemented by the Rocscience SLIDE 6.0 software (SLIDE). Further discussion of slope stability using SLIDE, including evaluation of nearshore dredge prism side slopes, is provided in Section 3.5.

3.1.1 Stability of Submerged Dredge Prism Side Slopes

Assessment of the dredge design includes slope stability analysis of dredge prism side slopes for short-term and long-term scenarios for slopes that are capped and uncapped. Submerged side slopes with a horizontal to vertical (H:V) slope of 2H:1V and 3H:1V were assessed to evaluate the factor of safety.

Geologic models were constructed for side slopes of 2H:1V and 3H:1V and representative soil parameters were applied based on the cross sections presented on Figures B-3.1 through B-3.6. In both models, the organic silt and clay soil layers are assumed to be the surface layers present. A 4- to 6.5-foot-thick granular cap will be placed as part of cap construction in the open-water and shoreline areas of the site. All dredge prism side slopes were assumed to be fully submerged.

The analysis considered both short-term and long-term stability. For the capped slope, the most critical short-term condition is the scenario immediately after the full thickness of the cap has been placed, but before the underlying sediment has fully consolidated (i.e., the undrained case). For purposes of the slope stability analysis of the short-term condition, it was assumed that the underlying soils would behave in undrained conditions and without significant strength gain from consolidation. This assumption is conservative because strength gain would occur during placement of the first layer of capping material and increase as subsequent lifts are placed during construction.

The long-term scenario is represented by drained behavior conditions following capping and considering strength gain from consolidation. The soils are assumed to behave under drained conditions. Table 3-1 summarizes the input values for the slope stability model for the short-term and long-term scenarios.

Table 3-1
Input Parameters for Slope Stability Analysis of Dredge Prism Side Slopes

		Short Term			Long Term	
Soil Layer	Total Unit Weight (pcf)	Minimum Undrained Shear Strength	Undrained Strength Ratio (Su/σ_v[4])	Internal Friction Angle (deg)	Effective Cohesion (psf)	Effective Internal Friction Angle (deg)
CAP[1]	120	0	0	30	0	30
Organic SILT[2]	90	60	0.40	0	5	25
CLAY[3]	115	250	0.38	0	0	30

Notes:
1. Values based on best professional judgment using a conservative estimate of the friction angle as placed.
2. Values are estimated from in situ Vane Shear Tests (PRDI 2010) and published literature of high water content materials (Edil and Fox 2000).
3. Values are estimated from Standard Penetration Tests and Consolidated-Undrained Triaxial Compression Tests (PRDI 2010), studies performed by others, and published literature of clayey soils (Kulhawy and Mayne 1990).
4. An undrained strength ratio is the undrained shear strength of the soil at a depth divided by the effective overburden stress at that depth.

The results of the slope stability analyses are summarized in Table 3-2 and are expressed in terms of the factor of safety.

Table 3-2
Factor of Safety Matrix for Slope Stability Results

Scenario Analyzed	Target Factor of Safety	2H:1V	3H:1V
Capped Slope – Short Term	1.3	1.18	1.60
Capped Slope – Long Term	1.5	1.18	1.68
Uncapped Slope – Short Term	1.3	1.28	2.06
Uncapped Slope – Long Term	1.5	1.26	1.75

3.1.2 Conclusions

For short-term loading conditions, a slope stability factor of safety of 1.3 is typically used. When assessing long-term loading conditions, the factor of safety is typically higher; in this case, it is 1.5. As indicated in Table 3-2, the factor of safety for a 3H:1V dredge prism side slope meets the target factors of safety requirements for loading conditions assessed, and 3H:1V side slopes are used in design of the dredge prism.

3.2 Cap Design

Caps will be constructed in the Inner Waterway and along the shoreline of the Central Waterfront area and former GP West property, and within the Log Pond. Shoreline and open-water caps consist of placement of up to 6.5 feet of sand, filter, and armor materials. Table 3-3 summarizes the thickness of the various cap components, and the assumed unit weight for each material.

Table 3-3
Properties of Sediment Cap

Layer	Total Unit Weight (pcf)	Maximum Inner Waterway Thickness (feet)	Maximum Nearshore Thickness (feet)	Log Pond Thickness (feet)
Armor	140	0	2.5	2.0
Filter/Gravel	125	1.5	1.5	1.5
Sand	120	2.5	2.5	2.5

3.2.1 Bearing Capacity

Bearing capacity for the caps were evaluated using methods described in Appendix C of the Assessment & Remediation of Contaminated Sediments (ARCS) Program *Guidance for In situ Subaqueous Capping of Contaminated Sediments* (Palermo et al. 1998). When cap material is placed on the surface of soft sediments, there is a potential for a bearing capacity failure directly through the in situ sediment. The initial cap lift thickness must be thin enough to prevent a bearing capacity failure resulting from the weight of the cap.

In typical foundation design problems, a factor of safety of 3.0 is used for calculations where there is potential for structural damage or impact to human safety. This is the suggested factor of safety presented in the ARCS guidance. However, the guidance does not distinguish between short-term and long-term bearing capacity considerations. Because of the transient nature of short-term loading, lower factors of safety are often considered acceptable in geotechnical engineering design. Experience on other capping projects has shown that a factor of safety of 3.0 can be overly conservative when considering construction lift thickness. Because life, safety, and structural stability are not design considerations, and due to the short duration of construction, a factor of safety of 1.5 was considered appropriate for use in this analysis for evaluating the design cap lift thickness. Subaquatic cap placement has been successfully demonstrated at multiple sites when designed using a bearing capacity factor of safety of 1.5.

This analysis evaluates the steady state, short-term stability of the cap, and soft sediments during construction. Once the cap has been placed, consolidation of fine-grained in situ sediments will occur which will increase the shear strength of the sediment. Thus, the long-term stability of the cap against bearing capacity failure will be greater than the short-term stability.

The in situ sediments must have sufficient internal strength to prevent local shear failure. To evaluate this condition, the ultimate bearing capacity was calculated with the Terzaghi equations for local failure (Palermo et al. 1998) using an undrained shear strength model for the organic silt consistent with that used in the slope stability evaluation. Based on a conservative undrained shear strength ratio of 0.4 (Edil and Fox 2000), this equates to an average near-surface undrained shear strength of 20 psf for the soft sediments, which is similar to that measured at other sites.

$$q_{ult} = \left(\frac{2}{3}\right) s_u * N_c$$

Where:

q_{ult} = ultimate bearing capacity of sediment (psf)

s_u = undrained shear strength of in situ sediments (psf)

N_c = Bearing capacity factor (dimensionless) = 5.14 for continuous strip footing (Terzaghi and Peck 1967)

This equation applies to a cap placed on the surface of a cohesive soil with an angle of internal friction, ϕ, equal to zero.

The ultimate undrained shear strength was calculated as follows:

$$q_{ult} = \left(\frac{2}{3}\right) \cdot 20 \cdot 5.14 = 68\ psf$$

A factor of safety of 1.5 was used to compute the allowable bearing capacity:

$$\mathrm{q_{all}} = \left(\frac{q_{ult}}{\mathrm{FOS}}\right)$$

Where:

q_{all} = Allowable bearing capacity (psf)

FOS = Factor of Safety = 1.5

$$\mathrm{q_{all}} = \left(\frac{68}{1.5}\right) = 45\ psf$$

The initial cap lift thickness that could be supported by the lowest strength in situ sediments without causing internal shear failure was calculated using the allowable bearing capacity and the following equation:

$$\mathrm{h} = \left(\frac{\mathrm{q_{all}}}{\gamma'}\right)$$

Where:

h = lift thickness

γ' = buoyant unit weight of cap material, if submerged (pcf)

$\gamma' = \gamma - \gamma_w$

γ = average total unit weight of cap material (pcf) ≈ 130 pcf

γ_w = unit weight of water (62.4 pcf)

γ' = 130 pcf – 62.4 pcf = 67.6 pcf

$$h = \frac{45\ psf}{67.6\ pcf} = 0.7\ feet$$

The analysis above, which uses the minimum undrained shear strength measured in the field, indicates a cap lift thickness of approximately 0.7 foot (approximately 8 inches) can theoretically be placed while maintaining an adequate factor of safety against bearing capacity failure during construction.

An observational approach will be implemented during construction to evaluate the performance of the recommended cap lift thickness and to evaluate the possibility of localized bearing capacity failures, should they occur. This observational approach will include review and evaluation of the contractor and Port of Bellingham progress surveys. Should results of the survey review indicate that localized bearing capacity failures may be occurring, then contractor means and methods for placement of the material will be revisited. Change in means and methods may include requirement for placement of material at slower rates or placement of material in thinner lift intervals. These provisions will be included in the design documents (plans and specifications) for the project.

3.2.2 Consolidation

The load from the cap will result in consolidation of the underlying sediments. The compressible layers that exist at the site are the organic silt and clay. Compressible properties for the organic silt were estimated using empirical correlations and the index properties measured during the PRDI. The compression index was estimated using the following empirical correlation specific for organic silts (Das 2006):

$$C_C = 0.0115 \cdot w_N$$

Where:
$C_{C=}$ Compression index (unitless)
w_N = Average in situ water content (unitless) = 150 percent

The average in situ void ratio was estimated to be 3.6 and was determined using the above average water content and a specific gravity of 2.4; both values were based on data collected during the PRDI. Compressibility properties for clay were calculated using consolidation tests performed by Converse, Davis, Dixon, and Associates (1979). The compression index for each of the two layers was divided by one plus the void ratio ($1+e_0$) to obtain the compression ratio. The resulting values are 0.38 and 0.07 for organic silt and clay, respectively.

To assess consolidation of the cap, geologic profiles were developed near study locations where capping is planned. At these locations, consolidation was evaluated where the organic silt deposits are expected to be thickest. In general, post-dredge thicknesses of the organic silt were thinnest in the northeast end of the Waterway and trended thicker along the Waterway alignment toward the southwest. The organic silt was found to be thickest near shoreline structures. Results of the consolidation analysis are presented in Table 3-4.

Table 3-4
Results of Consolidation Analysis

Location	Cap Type	Post-dredge Organic Silt Thickness (feet)	Anticipated Range of Long-term Settlement (inches)
Central Waterfront	Nearshore	5 to 13	20 to 40
Clarifier Shoreline	Nearshore	10 to 14	30 to 40
GP Dock	Nearshore	13 to 17	up to 15
Inner Waterway	Inner Waterway	8 to 11	up to 20
Log Pond	Nearshore	0	15 to 35

3.2.3 Conclusions for Cap Design

Based on the bearing capacity analysis performed, caps can be constructed with an acceptable factor of safety for placement of an initial lift thickness of 0.7 foot (or approximately 8 inches) of cap material (i.e., clean sand). Additional lifts can be placed following the full placement of the initial lift in the area were capping is being performed.

3.3 Retaining Wall Design

The recommendations contained in this section are provided in support of the Maple Street bulkhead and East Central Waterfront and West Central Waterfront walls (waterfront walls) that are located along the respective shorelines adjacent to the Maple Street bulkhead.

3.3.1 Lateral Earth Pressures for Shoreline Structures

Lateral earth pressures were estimated using the soil parameters presented in Table 3-4. In the development of these recommended earth pressure parameters, two cases for the static earth pressures were evaluated:

- **Post-dredge:** This case represents a temporary condition were the unbraced wall height is at a maximum. Capping has not yet been performed meaning the additional passive earth pressure provide by the cap is not present. The clay and organic silt are assumed to behave undrained. Both clay and organic silt exists waterward of the Maple Street bulkhead while only clay is upland of the Maple Street bulkhead and on both sides of the waterfront walls. A factor of safety of 1.3 is recommended for structural analyses of this case.

  A factor of safety of 1.3 is appropriate for this evaluation due to the implementation of additional engineering controls that further mitigate the risk of slope movement or structure displacement. These engineering controls include a requirement for offset of surcharge associated with upland operations, upland soil removal adjacent to the wall to reduce active loading, and limited exposure time in the temporary condition through requirement of cap material placement immediately following completion of dredging activities.

- **Post-cap:** This case represents the final configuration of the shoreline and the condition immediately after placement of the cap material. Capping will take place in front of all bulkheads meaning an additional passive earth pressure will be provided. The clay and organic silt are assumed to behave undrained. Both clay and organic silt exists waterward of the Maple Street bulkhead while only clay is upland of the Maple Street bulkhead and on both sides of the waterfront walls. A factor of safety of 1.5 is recommended for structural analyses of this case.

The earth pressure theory assumed for both static cases is Rankine theory (Fang 1991). Rankine earth pressure theory assumes that there is no interface wall friction between the structural element and the soil. This generally produces estimates for the active and passive earth pressure that are conservative. Earth pressure diagrams for these two static cases are shown on Figure B-4.1 and B-4.2 for the Maple Street bulkhead and Figure B-4.4 and B-4.5 for the waterfront walls.

Additionally, seismic earth pressures were developed for scenarios where the backfill soil is both non-liquefied and liquefied. For the non-liquefied case, the Mononobe and Okabe methodology was used to determine the seismic increment exerted on the walls during a design-level earthquake event.

More detailed discussion of seismicity, which includes ground motion parameters, liquefaction, and post-liquefaction residual strength and provides further discussion of the seismic earth pressures, is presented in Section 3.6. The seismic earth pressures for the Maple Street bulkhead are shown on Figure B-4.3 and on Figures B-4.6 and B-4.7 for the waterfront walls.

3.3.2 Tie-backs for Maple Street Replacement Bulkhead

The Maple Street replacement bulkhead will require tie-backs to support the wall. The retained soil behind the wall includes both granular and cohesive soil units. The cohesive clay layer is expected to be encountered at an approximate elevation of -12 feet MLLW. During seismic loading conditions, the granular soil layer is susceptible to liquefaction; therefore, the tie-back should be bonded in the clay unit. For planning purposes, an ultimate pull-out stress of 1,000 psf should be assumed for bonding within the clay unit. A factor of safety of 2.0 should be used with this value. Higher ultimate adhesion values may be achievable in the field depending on the contractor’s installation methods.

A pilot test was performed prior to completion of the design effort associated with the Maple Street bulkhead to confirm the recommended pull-out stress and to evaluate the potential for long-term creep and down-drag on the sheetpile wall due to the vertical component of stress in the anchors. The tests specified for the pilot test program for pull-out stress and creep potential were consistent with those described in the Federal Highway Administration ground anchor and anchor system technical manual (FHWA 1999). A total of four tiebacks were tested for the pilot test program—two in the granular fill layer above the clay and two in the clay unit. Based on the results of the testing program, the recommended ultimate pull-out stress of 1,000 psf for clay resulted in an acceptable tie-back performance during testing. Results of the pilot test program will be summarized in a reference document that will be made available to the contractors that provide bids for this project. Additional verification testing of tie-back anchor capacity will also be performed during construction to verify that design criteria are being achieved.

3.3.3 Conclusions for Design of Shoreline Structures

The recommended earth pressures for the Maple Street replacement bulkhead and waterfront walls are presented in Figures B-4.1 to B-4.7.

Stability of the Maple Street replacement bulkhead and waterfront walls were also evaluated using limit equilibrium to check embedment requirements for global stability. For the Maple Street replacement bulkhead during seismic loading, the minimum total wall length and toe elevation of the bulkhead should be approximately 60 feet and elevation -45 feet MLLW, respectively to achieve an acceptable factor of safety for global stability. Stability of the waterfront walls require a minimum total wall length of approximately 30 feet and toe elevation of -15 feet MLLW, respectively to achieve an acceptable factor of safety for global stability.

For planning purposes, an ultimate pullout stress of 1,000 psf should be assumed for bonding in the clay. Bonding should not be performed above an elevation of -12 feet MLLW due to the presence of potentially liquefiable soils above this elevation.

3.4 Geotechnical Engineering Recommendations for Other Structures

3.4.1 Earth Pressures and Soil Modulus for Pile-supported Structures

Dredging near the GP dock and BST dock could result in an unbalanced lateral earth pressure at the face of the outer most timber piles. Earth pressures and soil modulus parameters were developed to allow the structural engineer to assess potential structural issues related to the unbalanced lateral earth pressure on the dock and pier.

Analysis of the BST pier considered both Berths 1 and 2, where different dredge depths are planned (see Figure B-2). The maximum over-dredge allowance of 2 feet (for permitting purposes) is assumed for all cases.

Soil modulus parameters were requested by the structural engineer for use in their model. Estimates of horizontal soil modulus for both cohesive and cohesionless soils were performed using the guidance provided in the American Association of State Highway and Transportation Officials (AASHTO) 2010 Load and Resistance Factor Design (LRFD) Bridge Design and Specifications. The soil modulus parameters and estimate of the effective pile width for the passive earth pressure resistance of the BST and GP docks are provided on Figures B-4.8 to B-4.10.

3.4.2 Pile Foundation Design

Pile installation at the Central Waterfront region of the site includes the following:

- Replacement of mooring and dolphin piles at various locations within the Inner Waterway area
- Installation of new fender piles waterward of the Maple Street replacement bulkhead

3.4.3 Pile Selection

A variety of pile types are commonly used to support structures. Broadly categorized, pile types typically used to support heavy loads include continuous flight auger (CFA), drilled shafts, and driven piles. Although installation of CFA and drilled shaft piles typically causes less ground vibration than driven pile installation, these pile types require removal, management, and disposal of site soils, which can be costly if soils are contaminated. Driven piles typically require minimal management of site soils and will be considered for the support of structures for this project.

There are three classes of driven piles commonly used for foundation support: timber, concrete, and steel. Table 4-4 presents the pros and cons for each of these types of driven piles. Based on site-specific considerations, steel pipe piles are being considered for foundation support for this project.

Table 4-4
Driven Pile Selection Considerations

Pile Type	Advantages	Disadvantages
Timber	• Comparatively low in initial cost • Permanently submerged piles are resistant to decay • Easy to handle	• Difficult to splice • Vulnerable to damage in hard driving; both pile head and toe may need protection • Intermittently submerged piles are vulnerable to decay unless treated
Precast Concrete	• High load capacities • Corrosion resistance obtainable • Hard driving possible	• Unless pre-stressed, vulnerable to handling damage • Relatively high breakage rate, especially when piles are to be spliced • Considerable displacement • Difficult to splice when insufficient length ordered
Steel H-Piles	• Available in various lengths and sizes • High capacity • Small soil displacement • Easy to splice • Able to penetrate through light obstructions • Pile toe protection will assist penetration through harder layers and some obstructions	• Vulnerable to corrosion where exposed HP section may be damaged or deflected by major obstructions • Allowable capacity should be reduced in corrosive environments • Use as a friction pile in granular materials can result in cost overruns
Steel Pipe Piles	• Closed end pipe can be internally inspected after driving • Low soil displacement for open end installation • Open end pipe with cutting shoe can be used against obstructions • Open end pipe can be cleaned out and driven further • High load capacities • Relatively easy to splice	• Soil displacement for closed end pipe

3.4.3.1 Vertical Pile Capacity

Dolphin and fender systems will utilize pile foundations. Pile foundations carry vertical compressive loads by a combination of friction along the pile sides and by end bearing at the tip. Vertical uplift loads are resisted by friction alone. The top of the clay layer is observed at elevations ranging approximately from elevations -22 to -33 feet MLLW for in-water piles.

This clay layer provides the majority of the compressive and uplift capacity for in-water pile design. For planning purposes, factored (i.e., allowable) vertical compressive and uplift capacities were developed for 16-, 18-, 24- and 30-inch diameter steel pipe piles in accordance with AASHTO (2010). Figure B-4.11 at the end of this report presents the factored nominal pile capacity as a function of pile tip elevation (MLLW) for each pile diameter. To minimize the potential for group effects, horizontal pile spacing should be at a minimum of 2.5 times the pile diameter or 30 inches, whichever is greater.

The embedment depth determined using Figure B-4.11 is based on a strength limit state. Because of the compressibility of the clay unit, a pile group loaded to the strength limit state could settle on the order of 2 to 3 inches if larger diameter piles are used (the 24- and 30-inch diameter). If lesser settlement is desired, longer piles than estimated from Figure B-4.9 will be necessary.

3.4.3.2 Lateral Pile Capacity

We understand that LPILE computer software will be used to evaluate the lateral response of piles. Table 4-5 provides recommended LPILE parameters for static design. Table 4-6 provides the recommended elevation ranges over which these parameters should be used at various locations around the site.

Table 4-5
Static Input Parameters for Lateral Pile Analysis Using LPILE

Parameter	Fill	Fill (Submerged)	Cap Material	Organic Silt	Sand	Clay
Soil Type	Sand	Sand	Sand	Soft Clay	Sand	Soft Clay
K Value (pci)[1]	90	40	60	N/A	20	N/A
Effective Soil Weight (pcf)[2]	125	58	62	28	58	52
Internal Friction Angle (deg)	35	32	35	N/A	30	N/A
Undrained Shear Strength (psf)	N/A	N/A	N/A	250[5]	N/A	500[4]
Strain Factor (Strain at 50% Max Stress)	N/A	N/A	N/A	0.02	N/A	0.01

Notes:

1. pci – pounds per cubic inch
2. pcf – pounds per cubic foot
3. Value is for top of layer. Increase this value by 10 psf per foot of pile embedment into this layer.
4. Value is for top of layer. Increase this value by 20 psf per foot of pile embedment into this layer.

Table 4-6
Recommended Elevation Ranges of Soil/Sediment Units for Static Analysis Using LPILE

		Estimated Range of Elevations of Soil Units (Feet; MLLW)					
Pile Type	Pile Location	Fill	Fill (Submerged)	Cap Material	Organic Silt	Sand	Clay
Mooring and Dolphin Replacement	Former Chevron Shoreline	N/A	N/A	-10 to -14	-14 to -33	N/A	-33 to pile toe
	South Shoreline Source Control Structure	N/A	N/A	-6 to -12	-12 to -25	N/A	-25 to pile toe
	North Shoreline Source Control Structure	N/A	N/A	-10 to -14	-14 to -22	N/A	-22 to pile toe
Fender Piles	Maple Street Bulkhead	N/A	N/A	-8 to -14	-14 to -19	N/A	-19 to pile toe

3.4.3.3 Pile Vertical Spring Constant

For the moorings and dolphins, the structures are not expected to be sensitive to settlement and loading is assumed to be rapid. The total factored load is understood to be 200 kips in axial compression for batter piles. For these structures, a vertical spring constant of 300 to 500 kips/in is recommended and is based on the evaluation of 16- to 30-inch diameter steel pipe piles. If the use of smaller or larger diameter piles is desired, appropriate spring constants can be developed at the request of the structural engineer.

3.4.3.4 Pile Installation

The recommended pile capacities provided in this report are based on observed soil conditions; the soil conditions may vary in consistency and type at actual pile installation locations. It is anticipated that piles will be advanced using vibratory methods and will encounter soft to stiff cohesive soils, which could make pile advancement difficult. It is important to bear in mind that excessive vibrating can damage the piles. A reasonable selection of the pile size and vibratory hammer can reduce pile damage during advancement. If the contractor elects to discontinue vibrating due to refusal or slow advancement prior to reaching or nearing the design tip elevation, consultation with the geotechnical engineer is recommended to determine the shorter piles' adequacy for carrying design loads.

Furthermore, it is recommended that a geotechnical engineer be present during pile vibrating activities. The engineer will observe the contractor's operation, collect and interpret the installation data, and observe all pile installation. With careful observation of pile installation operations, it is possible to monitor variations in subsurface conditions and verify that the required penetration depths and capacities are achieved. Pilings that may be subjected to potential vertical loads will be proof tested at completion of installation.

3.4.4 Conclusions for Geotechnical Engineering Recommendations for Other Structures

For the structural assessment of the GP dock and BST pier, diagrams that include earth pressures and soil modulus parameters are presented on Figures B-4.8 to B-4.10.

For the sizing of piles for planning purposes, use Figure B-4.9 for estimates of pile capacity as a function of tip elevation.

3.5 Slope Stability

The remediation includes dredging near waterfront facilities and shorelines. Dredging removes sediments that support the toe of the slope and hence the resisting force against a potential sliding mass. To assess slope stability, geologic models were developed for each of the facilities described in Section 1.1 of this report.

Slope stability modeling was performed using Rocscience SLIDE 6.0 software that utilizes limit equilibrium methods of analysis. The soil model for limit equilibrium analysis is a rigid, perfectly-plastic soil model. The assumptions inherent to this model are that the anticipated sliding mass remains rigid (i.e., non-deformable) and the soil strength along the slip plane is fully mobilized at failure. While this analysis method does not directly represent the true behavior of the soil during a slope failure, it is intended to provide a reasonable indication of the overall stability of a slope and is generally accepted as the standard of practice for this type of assessment.

The inter-slice force functions used in the analysis were Morgenstern-Price (1965) and Spencer (1967). These two methods satisfy both force and moment equilibrium and have been used in common practice for more than 40 years. For each loading condition and respective wall condition analyzed, both methods were applied to a suite of potential failure planes that pass beneath the toe of the sheetpile wall. The failure plane with the lowest factor of safety is then compared to the respective design criteria.

In addition to the slope stability analysis performed for the shorelines and waterfront structures, an assessment of post-dredge sloughing of slopes underneath the GP dock and BST pier was performed. The intent of the analysis is to estimate a range of long-term, post-dredge stable slope angles.

The Maple Street bulkhead waterfront walls were also assessed for the seismic condition. The factors of safety criteria for slope stability assessment are summarized in Table 3-7.

Table 3-7
Slope Stability Factor of Safety Criteria

Condition	Description	Criteria
Short Term, Static	Also referred to as the temporary case, the short-term static condition is represented by two scenarios: 1. post-dredge 2. post-cap Modeling of the short-term condition assumes undrained shear strength parameters for cohesive soil layers and drained strength parameters for cohesionless soil layers.	Minimum FS = 1.3 (Duncan and Wright 2005) Surcharge = 250 psf above Maple Street Bulkhead Surcharge = 100 psf for shoreline slopes
Long Term, Static	This condition represents the final, post-construction configuration and assumes sufficient consolidation of subgrade soils as well as slow failure, such that drained conditions are appropriate. Modeling of these conditions assumes drained shear strength parameters for all soil layers.	Minimum FS = 1.5 (Duncan and Wright 2005) Surcharge = 250 psf above Maple Street Bulkhead Surcharge = 100 psf above shoreline slopes
Seismic	A pseudostatic slope stability analysis is performed. The seismic coefficient is as assumed to be one-half the spectral acceleration at a 0-second period of the design response spectrum (K_h = 0.121 g). Modeling of the seismic condition assumes undrained shear strength parameters for cohesive soil layers and drained strength parameters for cohesionless soil layers.	Minimum FS = 1.1 (WSDOT 2011) Surcharge = 250 psf for Maple Street Bulkhead Surcharge = 100 psf for East and West Central Waterfront Walls

Note:

FS = factor of safety
K_h = horizontal seismic coefficient

3.5.1 Information and Assumptions

For the cohesionless soils (e.g., fill, sand, and silty sand), strength parameters were estimated using blow counts from in situ SPTs (ASTM D 1586). A blow-count to friction angle empirical correlation from published literature was used to estimate the effective internal friction angle of the soils (Kulhawy and Mayne 1990). The unit weight of the material was estimated using typical values for the soil types identified in the boring logs.

For the clay, test results from consolidated undrained tri-axial compression tests with recorded pore pressure measurements (CU-TX; ASTM D 4767) were used to estimate an effective internal friction angle (ϕ') for analysis of long-term conditions where drained behavior is assumed. The CU-TX results were also used with one dimensional oedometer tests (ASTM D 2435) to develop an undrained strength ratio and an estimate of the OCR of the clay.

The normally consolidated strength ratio derived from the CU-TX results produces a design value of 0.22.

The oedometer test results of samples collected during the PRDI and by others reveals that the typical range of OCRs for the clay is about 1.5 to 3. An average OCR of 2.0 is assumed for clay, which results in an average design value of 0.38 for the strength ratio of the over-consolidated clay.

These values were used with a minimum undrained shear strength of 250 psf for the analysis of submerged dredge prism side slopes and a value of 500 psf for the analysis of nearshore slopes and shorelines, which have been subjected to relatively higher overburden stresses.

For seismic analysis, the minimum undrained shear strength value of 500 psf was increased by 30 percent since loading is expected to be rapid. The resulting minimum shear strength value is 700 psf for nearshore slopes and shorelines. Additionally, the clay unit is not predicted to exhibit significant strength loss during a seismic event and therefore post-seismic strength reductions are not applied (Idriss and Boulanger 2004).

For the organic silt, index properties from Atterberg Limits (ASTM D 4318) were used with published correlations to estimate an undrained strength ratio that was used for modeling. An OCR of 1.0 (i.e., a normally consolidated state) was assumed (Edil and Fox 2000). The undrained shear strength ratio is estimated to be 0.40 based on the literature correlations. This estimated value represents a lower bound of the empirical correlation used; therefore, the value is considered conservative. The strength ratio for organic silt was used with a minimum shear strength of 60 psf, which is consistent with results measured for soft sediments at similar sites and lower than the average results of the Vane Shear testing performed at this site.

The soil properties assumed are summarized in Table 3-8.

Table 3-8
Soil Parameters Assumed for Slope Stability Analysis

Soil Stratum	Strength Type	Total Unit Weight γ_t (lb/ft^3)	Effective Internal Friction Angle ϕ' (deg)	Minimum Undrained Shear Strength	Undrained Strength Ratio (S_u/σ_v')
Fill	Mohr-Coulomb	120	31 to 35	0	0
Organic Silt	Undrained	90	0	60	0.40
Organic Silt (drained)	Mohr-Coulomb	90	25	0	0
Sand	Mohr-Coulomb	110	28 to 32	0	0
Silty Sand	Mohr-Coulomb	115	31 to 33	0	0
Clay (in-water)	Undrained	115	0	250	0.38
Clay (nearshore)	Undrained	115	0	500	0.38
Clay (seismic)	Undrained	115	0	700	0.38
Clay (drained)	Mohr-Coulomb	115	30	0	0
Bedrock	Mohr-Coulomb	150	42	0	0
Clean Sand (cap)	Mohr-Coulomb	120	30	0	0
Filter and Armor (cap)	Mohr-Coulomb	130	38	0	0

Notes:
deg = degrees
lb/ft^3 = pounds per cubic feet

3.5.2 Slope Stability Results

The Maple Street bulkhead and East and West Central Waterfront walls were analyzed for short-term, long-term, and seismic loading conditions (including the post-liquefaction condition). The remaining locations were analyzed for short-term and long-term loading conditions only because these areas do not support shoreline structures where life safety concerns exist. For these other areas, an assessment of permanent seismic slope displacements was performed to assess the magnitude of potential slope movements during a seismic event. The discussion of this evaluation and presentation of results is reserved for Section 3.6.

The results for the slope stability analysis of the locations studied are presented in Table 3-9.

Table 3-9
Slope Stability Results for Shorelines and Waterfront Structures

Locations Studied	Targeted Short-term FS[1]	Targeted Long-term FS[1]	Short-term FS[1] (Post-dredge)[2]	Short-term FS[1] (Post-cap)	Long-term FS (Post-cap)	Dynamic FS[1]	Post-liquefaction FS[1,2]
Former Chevron[5] Property	1.3	1.5	1.31	1.39	1.56	N/A	N/A
Maple Street Bulkhead	1.3	1.5	1.32	1.63	2.18	1.1	1.5
Other Central Waterfront (east)[5]	1.3	1.5	1.30	1.38	1.54	N/A	N/A
South[3] Shoreline	1.3	1.5	1.30	1.34	1.51	N/A	N/A
Log Pond[3,4] Bulkhead	1.3	1.5	N/A	1.42	1.53	N/A	N/A

Notes:
1. FS = Factor of safety against slope movement
2. No surcharge loading is assumed.
3. A 3H:1V slope is assumed.
4. Failure planes are restricted to those waterward of the bulkhead.
5. Results are for the source control structures.

As observed in the above table, the results of the slope stability analysis meet the targeted factors of safety. Slopes for the South Shoreline and the rock buttress of the Log Pond should be 3H:1V or shallower. Representative outputs from the slope stability analysis are presented in Figures B-5.1 to B-5.2.

3.5.3 Conclusions for Slope Stability Assessment

The slope stability analyses demonstrate that the studied locations meet the target factors of safety discussed at the beginning of this section. For the South Shoreline and rock buttress of Log Pond bulkhead, slopes with a 3H:1V have an acceptable factor of safety. Source control structures and the Maple Street bulkhead also have an acceptable factor of safety for designs where sheetpiles are driven to the minimum tip elevations provided in Section 3.3.

3.6 Seismicity

The project location lies in a seismically active region and is characterized by four principal sources for strong ground shaking (earthquakes) – three associated with the Cascadia Subduction Zone (CSZ) and one resulting from relatively shallow crustal zones.

The seismic hazard analysis performed is based on the seismic site class and associated ground motion parameters developed using American Society of Civil Engineers Standard 7 2010 (ASCE 7-10; the code) with supplemental guidance from U.S. Geological Survey (USGS) resources. The ASCE 7-10 procedure for developing design-level ground motion parameters results in a seismic demand that is similar to the demand from an earthquake with a 10 percent probability of occurrence in 50 years (i.e., 475-year event). The ASCE 7-10 procedure is commonly used nationwide and has been successfully used on many projects in the greater Puget Sound area.

3.6.1 Seismic Parameters

Code-based seismic design is typically used when upland structures are present and life-safety is a concern. Seismic design criteria for remedial actions have not been developed. In light of this, the seismic analysis of the remedial design considered ASCE 7-10 to be appropriately conservative for the evaluation of non-structural elements even though life-safety is not a concern.

Ground motion parameters were developed using ASCE 7-10 specifications with guidance from USGS resources.

3.6.1.1 Seismic Site Class and Design Category

The method chosen for determination of the seismic site class utilized SPT blow counts (N_i) from the geotechnical borings provided in the PRDI and NRDI reports. The average uncorrected SPT blow count for the upper 100 feet of soil ($\overline{N}$) was calculated to be less than 15 blows per foot for the borings nearest to the Maple Street bulkhead and on the South Shoreline near the existing clarifier bulkhead. Therefore, the structural design of the Maple Street replacement bulkhead should be based on a site class E. The site class designation E is also assumed for the assessment of permanent seismic displacements of capped submerged slopes, nearshore capped shorelines, and regions of the Central Waterfront shoreline where source control structures will be installed.

3.6.1.2 Ground Motion Parameters

The response spectrum and ground motion parameters for five-percent damping were developed using the code. The response spectral accelerations parameters for short and 1-second periods corresponding to Whatcom Waterway are presented below:

$$\mathbf{S}_s = 0.959 \text{ g}$$

$$\mathbf{S}_1 = 0.377 \text{ g}$$

The peak ground acceleration (PGA) of the maximum considered earthquake (MCE) for a site class E was determined in accordance with the code. The value for the site dependent PGA for the MCE is 0.361 (g). This value is used for the evaluation of liquefaction potential for structural design as specified in ASCE 7-10 code, and is greater than the acceleration expected during an event with a 475-year return interval.

Assuming the risk category is I, II, or III, the design of the Maple Street bulkhead and waterfront walls should be developed for a Seismic Design Category (SDC) of D.

3.6.2 Seismic Hazards

The seismic hazards evaluated for the design of the Maple Street bulkhead and waterfront walls includes:

- Surface fault rupture
- Liquefaction potential
- Post-liquefaction stability
- Seismic slope stability
- Permanent seismic slope displacements

Recommendations to mitigate the risks associated with these hazards are provided in Section 3.6.5. Seismic issues for dredging and capping were evaluated to estimate potential permanent seismic slope displacements during a design-level event.

3.6.2.1 Surface Fault Rupture

Mapping efforts of active faults in Washington State has been performed by the USGS (2002). Those maps have been adapted and included in the WSDOT Geotechnical Design Manual (GDM; WSDOT 2011). The WSDOT maps and USGS online resources show no known active faults occur within 2 miles of the site, the minimum distance required by the code for considerations of fault rupture. Therefore, surface deformations as a result of surface fault rupture are not anticipated.

3.6.2.2 Liquefaction Potential

As is true at most shoreline sites in Puget Sound, ground deformations from liquefaction during strong shaking are considered a pre-existing hazard that has been present at the site prior to any remedial efforts. The remedial efforts will not contribute additional seismic hazards at the site. In some cases, increased resistance to lateral spreading (e.g., the Central Waterfront shoreline) will result because the installation of bulkheads and replacement of soft or loose nearshore soils with sand and gravel cap materials will reduce susceptibility to deformation.

The phenomenon of liquefaction most commonly occurs in saturated, loose, cohesionless soils during ground shaking. When subjected to rapid loading (i.e., earthquakes) a saturated soil that is unable to drain during loading will generate excess pore water pressure as it attempts to densify. Liquefaction occurs when the excess pore water pressure reduces the effective stress enough to result in a loss of soil strength.

The soil conditions adjacent to the Maple Street bulkhead and waterfront walls were evaluated for strength-loss potential using the criteria proposed by Bray and Sancio (2006). This methodology evaluates the index properties of the soil to determine if an assessment of liquefaction potential is needed. The method is applied by comparing the plasticity index (PI) and the ratio of in situ water content (w_c) to the soil's liquid limit (LL) against a historical database of other soils that are known to have experienced strength-loss during strong shaking. Soils with a PI greater than 18 and a ratio of w_c/LL less than 0.80 are not considered susceptible to significant strength-loss, while soils with a PI less than 12 and a w_c/LL ratio greater than 0.85 are considered susceptible. Based on the Bray and Sancio criteria, the granular soils upland of the bulkhead wall (i.e., fill and sand) and above the clay contact are considered susceptible to strength-loss while the clay and organic silt are not. The Bray and Sancio evaluation indicates the fill and sand upland of the Maple Street bulkhead and waterfront walls are susceptible to strength-loss during an earthquake.

The fill and sand adjacent to the Maple Street replacement bulkhead and waterfront walls were evaluated for liquefaction susceptibility using two simplified procedures, Youd et al. (2001) and Idriss and Boulanger (2008). Both methods are considered deterministic procedures where a soil is considered liquefiable at a 16 percent likelihood of liquefaction triggering. For the Central Waterfront shoreline, liquefiable soils are present between the assumed upland water elevation and the contact with the top of the clay layer (ranges from -6 feet MLLW at the northeastern end to -20 feet MLLW at the southwestern end). Just upland of the Maple Street bulkhead, the potentially liquefiable layer is approximately 16 feet thick.

3.6.2.3 Post-liquefaction Stability

Ground motions that result in significant levels of horizontal acceleration are believed to occur before the strength-loss associated with soil liquefaction. Therefore, the assessment of slope stability for the post-liquefaction scenario considers only the residual shear strength of the soil resulting from the onset of liquefaction and does not include a seismic loading coefficient for the slope.

Estimates of the residual strength for a post-liquefied soil were made using the methodology proposed by Idriss and Boulanger (2008). Because the behavior of a liquefied soil is believed to be similar to a viscous fluid, the strength is described in terms of an undrained shear strength ratio. The resulting parameter is a unitless constant and is defined as the soil's undrained shear strength at a particular depth divided by the vertical effective stress of the overburden at that depth. The undrained shear strength ratio for potentially liquefiable soil at the Maple Street bulkhead is 0.11. The results of the post-liquefaction stability analysis are presented in Section 3.6.2.4.

3.6.2.4 Seismic Slope Stability

The seismic coefficient was estimated using guidance from the WSDOT GDM and is taken as one-half of the 0-second period design spectral acceleration (0.121g). The Maple Street bulkhead is found to require a minimum embedment of 37 feet (i.e., total wall length of approximately 60 feet) to result in the target seismic factor of safety of 1.1. This resulting configuration produces the following factors of safety for global stability:

- Short-term static (post-cap) = 1.63
- Long-term static = 2.18
- Post-liquefaction = 1.5

Seismic slope stability of the waterfront walls requires embedment to an elevation of approximately -15 feet MLLW (i.e., total wall length of about 28 feet). The resulting configuration produces the following factors of safety for global stability:

- Short-term static (post-cap) = 1.38
- Long-term static = 1.54
- Post-liquefaction = 1.2

3.6.3 Seismic Earth Pressure

The recommended earth pressure diagrams for seismic scenarios are presented in Figures B-4.3. For a scenario were soils do not liquefy, a horizontal pseudostatic acceleration (k_h) is assumed to be one-half of the 0-period spectral acceleration (0.121 g). This horizontal seismic coefficient and a vertical pseudostatic acceleration (k_v) of 0 (g) were used in a Mononobe-Okabe (Mononobe and Matsuo 1929; Okabe 1926) method of analysis. Guidance from Kramer (1996) was used to develop seismic earth pressures for a saturated backfill that is not free-draining (i.e., restrained pore water condition).

For the seismic earth pressure case where the backfill liquefies, it was assumed that any significant ground motions that would result in additional horizontal loading will have occurred prior to the liquefying of the upland soils; therefore, k_v and k_h are taken as 0 (g). The calculation of the earth pressure uses the undrained shear strength ratio of 0.11 that was determined using the methodology proposed by Idriss and Boulanger (2008). The recommended factor of safety can be reduced to 1.1 for the seismic event.

3.6.4 Permanent Seismic Slope Displacements

The permanent seismic slope displacement of the shorelines, submerged slopes, and the Maple Street replacement bulkhead were evaluated for the ASCE 7-10 design event (DE) using the methodology proposed in Bray and Travasarou (2007). All analysis performed was based on the final, post-construction condition of the remedial design (i.e., post-cap). Two required parameters for the analysis, yield coefficient and slope height were derived using limit equilibrium procedures. Other required input parameters were obtained from the response spectrum of the DE developed from the ASCE 7-10 code, USGS online resources, and SPT blow counts from nearby borings.

For the shorelines and submerged slopes, a representative section of each region of the Waterway was selected for analysis. The Maple Street bulkhead and waterfront walls and associated slopes were analyzed separately. Table 3-10 summarizes the results of the analyses performed for each region of the Waterway.

Table 3-10
Results of Permanent Seismic Slope Displacement Assessment

Location	Yield Coefficient (g)	Slope Height (feet)	Period of Sliding Mass[1] (s)	Spectral Acceleration at Period of Sliding Mass[2] (g)	Degraded Period of Sliding Mass (s)	Spectral Acceleration at Degraded Period of Sliding Mass (g)	Range of Permanent Seismic Slope Displacements[3] (feet)
Inner Waterway	0.113	22	0.13	0.47	0.20	.60	0.3 to 1
Log Pond	0.088	17	0.11	0.44	0.17	0.55	0.3 to 1
West Central Waterfront	0.105	36	0.17	0.57	0.26	0.61	0.3 to 1
East Central Waterfront	0.110	35	0.19	0.58	0.29	0.61	0.25 to 1
Maple Street Bulkhead	0.170	33	0.18	0.56	0.27	0.61	0 to 0.5

Notes:

1. Estimated using empirical correlation with SPT blow-count (Imai and Tonouchi 1982).
2. Estimated from the design response spectrum developed using ASCE 7-10 code.
3. Range represents an 84 percent to 16 percent probability of exceedance (Bray and Travasarou 2007).

Slope displacements for shorelines and in-water slopes generally range from 0.3 to 1.0 foot. This general range is within the 3-foot threshold that is generally accepted for earthen embankments (Duncan and Wright 2005). The permanent displacement of the slope with the Maple Street bulkhead is estimated to be approximately 0.5 foot and less than 1 foot for the waterfront walls. The Maple Street bulkhead and waterfront walls could have noticeable damage following a seismic event but would not be expected to fail catastrophically because deformations are precicted to be less than 12 inches (Kramer 1996). Therefore, the permanent deformations are considered acceptable given a non-collapse seismic performance criteria. However, repairs may be necessary for shoreline structures following a design-level or larger earthquake.

3.6.5 Seismic Hazard Considerations for Dredge Slope and Shoreline Design

Seismic hazards from strong ground shaking can cause surface deformations. For the Inner Waterway and shoreline slopes, the design features potentially affected by seismic hazards are sediment caps.

Potential cap thinning as a result of tolerable permanent seismic displacements was evaluated. For the nearshore slopes and in-water dredge prism side slopes, displacements are estimated to be 0.3 to 1.0 foot as previously discussed. Thus, thinning could occur, but a full exposure of underlying sediment would not be expected because caps are thicker than the estimated displacement.

To mitigate potential risks associated with a design-level or larger earthquake, a contingency cap inspection and repair program could be developed. Inspection could include visual, bathymetric, or probing surveys. Repairs would likely entail placing additional cap materials in areas identified to be deficient as a result of the inspection. Given the uncertain nature of seismic risk and uncertainty in the ability to predict potential effects from an earthquake, this contingency inspection and repair program would provide a sufficient level of assurance that seismic risks can be appropriately addressed.

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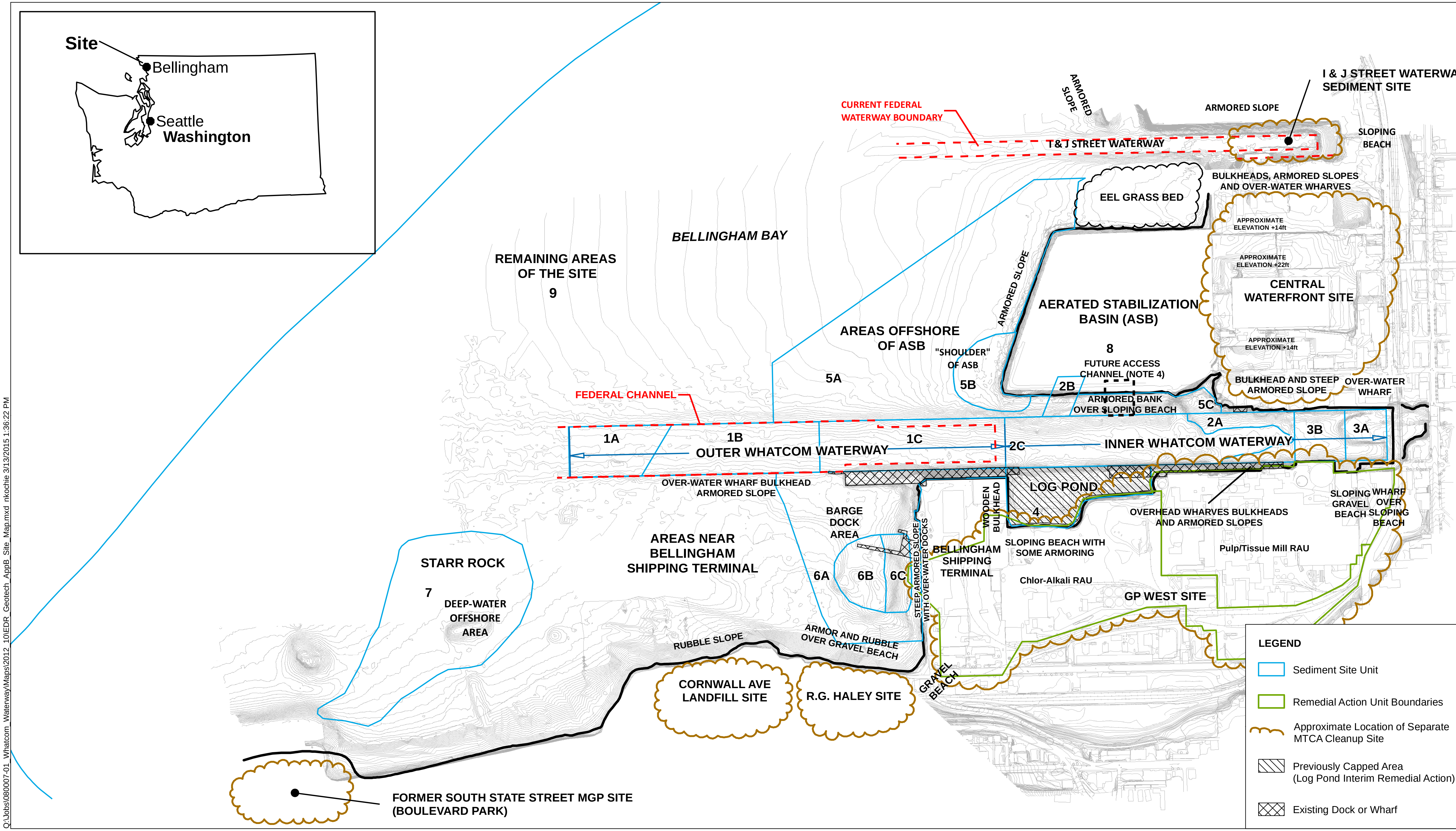
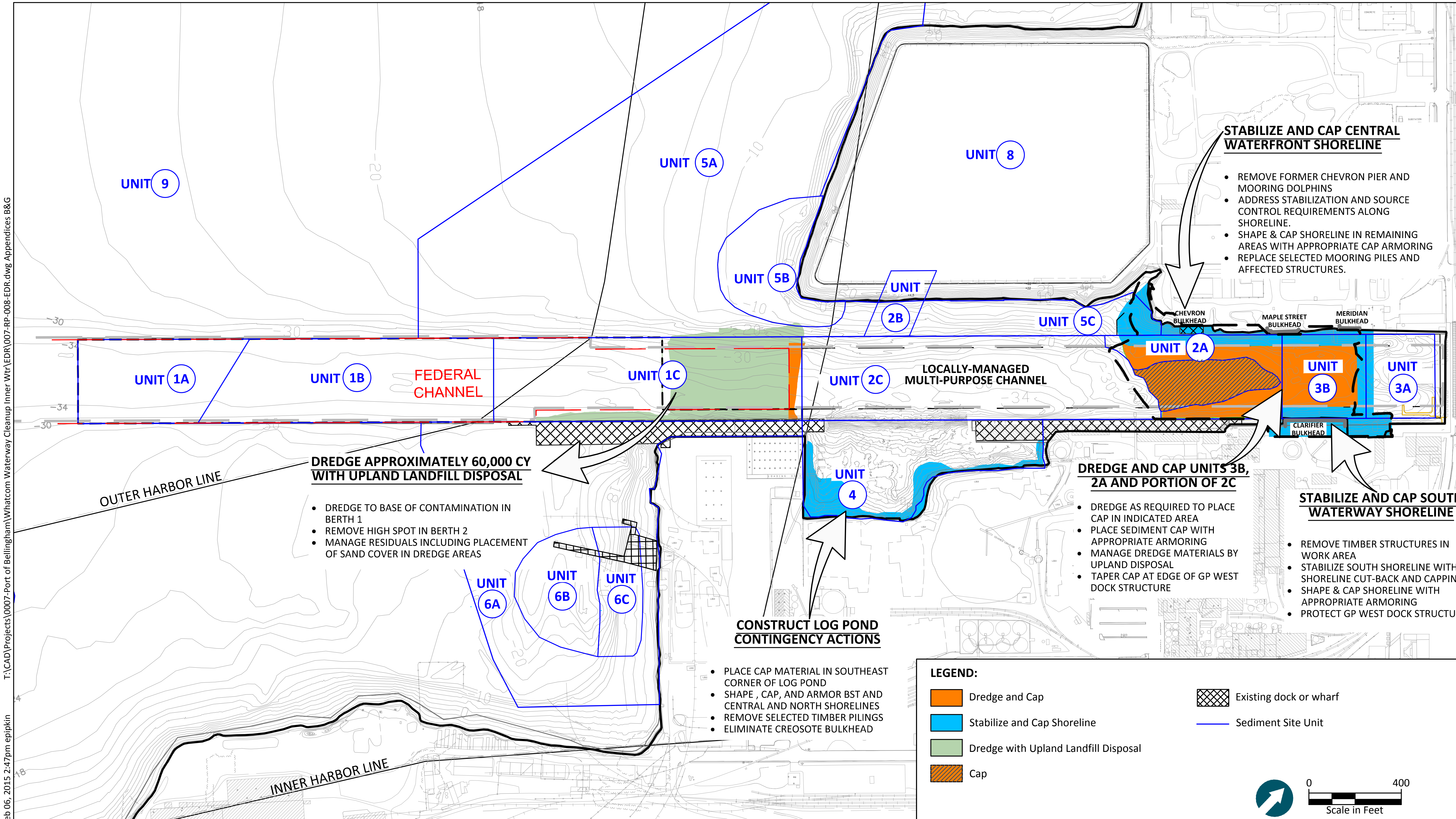
FIGURES



NOTES:
1. Site units are shown based on those in Figure 2-3 Cleanup Action Plan, Whatcom Waterway Site, September 2007. Unit 9 boundary updated based on PRDI findings.
2. Horizontal datum: Washington State Plane North, NAD 83 Feet.
3. Vertical datum: Mean Lower Low Water (MLLW).
4. Unit 2B was established in the Cleanup Action Plan based on the anticipated marina access channel location. This location will be adjusted during final design.
5. Remedial Action Unit (RAU) boundaries were defined in the Final Cleanup Action Plan for the GP West Pulp and Tissue Remedial Action Unit (Aspect 2014).

Feet
0 240 480 720 960

Figure B-1
Site Vicinity Map
Appendix B - Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas - Final EDR



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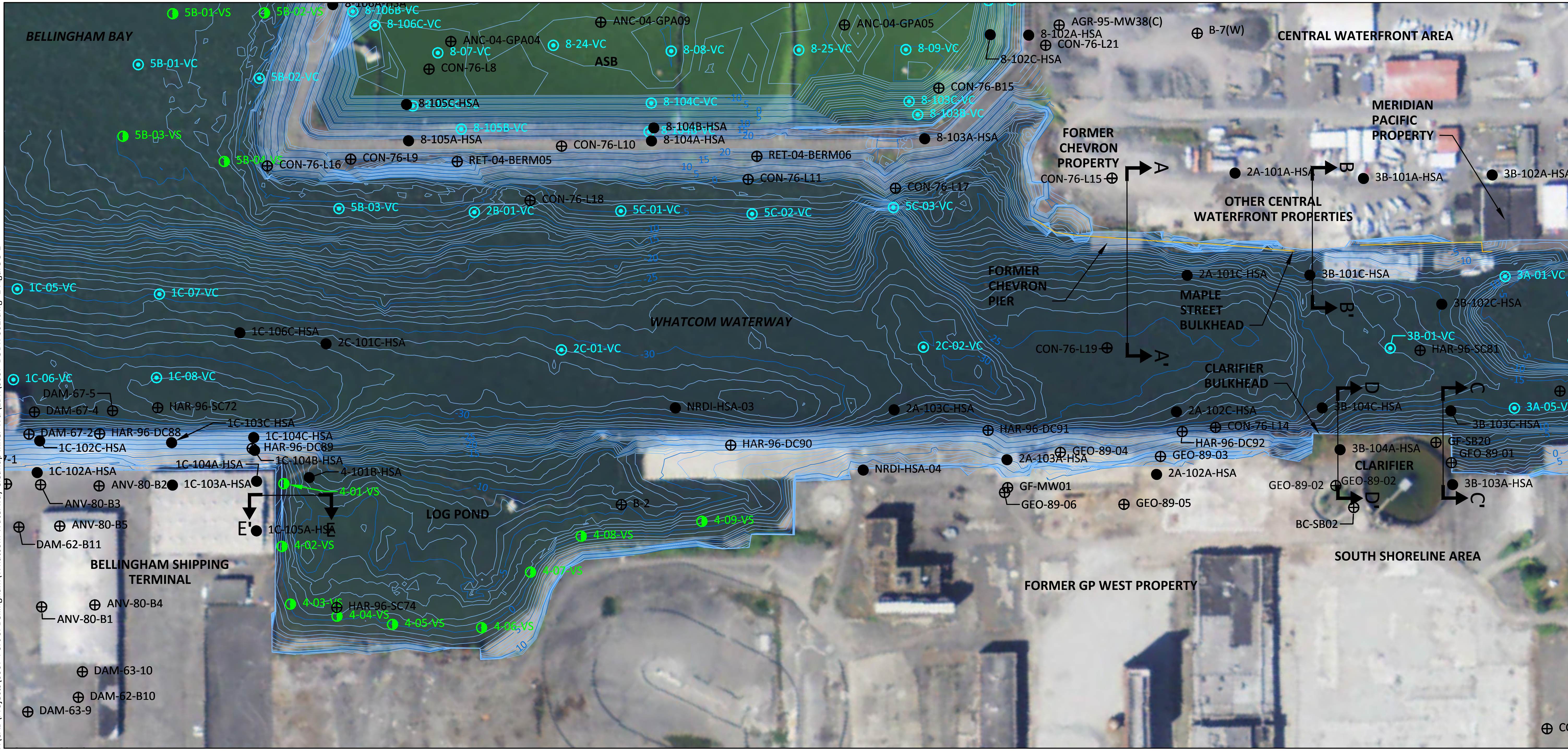
SOURCE: Figure 6-5 of Exhibit 1 of the First Amendment to the Whatcom Waterway Site Consent Decree (2011).
HORIZONTAL DATUM: Washington State Plane North, NAD 83 Feet.
VERTICAL DATUM: Mean Lower Low Water (MLLW).

Figure B-2
Construction Project for Phase 1 Areas
Appendix B - Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase Site 1 Areas - Final EDR



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HORIZONTAL DATUM: Washington State Plane North, NAD83.
VERTICAL DATUM: Mean Lower Low Water (MLLW).

NOTE:
Exploration locations from other firms are approximated from figures in other reports. Accuracy is limited and should not be exact.

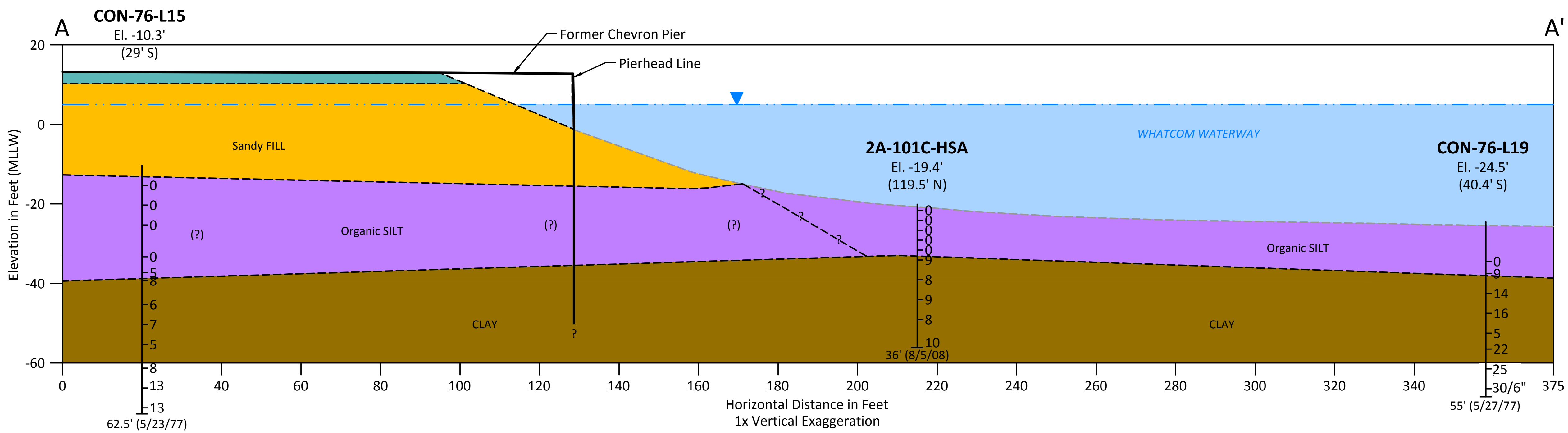
LEGEND:
- Soil Boring (Anchor QEA)
- Subsurface Exploration (Other firm)
- Vibracore (Anchor QEA)
- Vane Shear (Anchor QEA)

A A'
Geologic Cross Section Location and Designation

0 200
Scale in Feet

ANCHOR QEA

Figure B-3.1
Exploration and Cross Section Plan View
Appendix B - Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas - Final EDR



LEGEND:

Rip-rap and Asphalt

Sandy FILL - Loose to medium dense, variable moisture, multi-colored, gravelly SAND

Organic SILT- Very soft, black to dark gray, very sandy organic SILT with wood fragments

CLAY - Medium stiff, gray to light gray, low to high plastic CLAY with variable silt and gravel content and thin layers/lenses of fine sand

Existing Ground Surface

Mean Sea Level (+5 MLLW)

3D-104A-HSA — Boring ID
El. 25.5' — Surface Elevation (feet, MLLW)
(54' W) — Offset Distance in Feet
— Boring Location
18 — Uncorrected Blow Count (blows/feet)
— Bottom of Boring
30.0' (10/8/10)
— Completion Date
— Depth of Boring in Feet

0 30
Scale in Feet

NOTES:
1. Geologic contacts are inferred between borings.
2. Soil descriptions are general. Actual conditions will vary locally.

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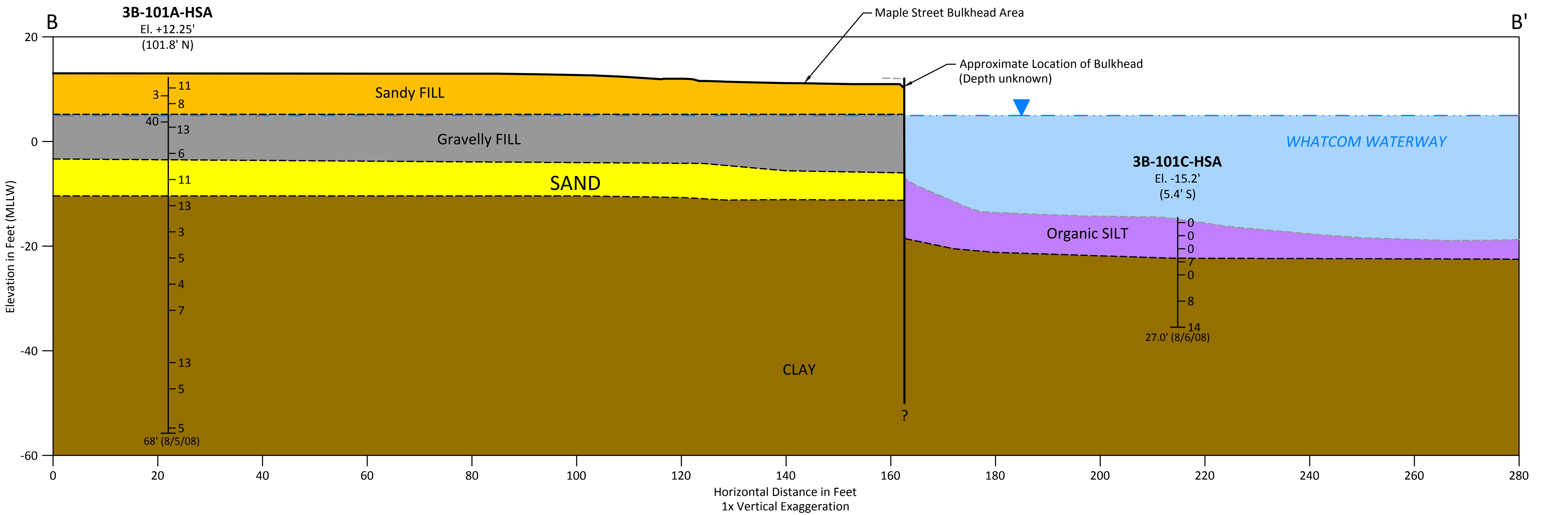
Figure B-3.2
Geologic Cross Section A-A'
Appendix B - Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas - Final EDR



LEGEND:

Sandy FILL - Loose to medium dense, variable moisture, multi-colored, gravelly SAND

Gravelly FILL - Medium dense to dense, dark gray with brown and light gray, sandy GRAVEL with silt

SAND - Medium dense, dark gray, silty poorly graded SAND with shell fragments

Organic SILT- Very soft, black to dark gray, very sandy organic SILT with wood fragments

CLAY - Medium stiff to stiff, moist, gray, CLAY with thin layers of silty sand

Existing Ground Surface

Mean Sea Level (+5 MLLW)

3D-104A-HSA — Boring ID
El. 25.5' — Surface Elevation (feet, MLLW)
(54' W) — Offset Distance in Feet
— Boring Location
18 — Uncorrected Blow Count (blows/feet)
— Bottom of Boring
30.0' (10/8/10)
— Completion Date
— Bottom of Boring in Feet

0 20
Scale in Feet

NOTES:
1. Geologic contacts are inferred between borings.
2. Soil descriptions are general. Actual conditions will vary locally.

T:\CAD\Projects\0007-Port of Bellingham\Whatcom Waterway Cleanup Inner Wtr\EDR\0007-BODR-002.dwg 3 Figure 3-3
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Figure B-3.3
Geologic Cross Section B-B'
Appendix B - Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas - Final EDR

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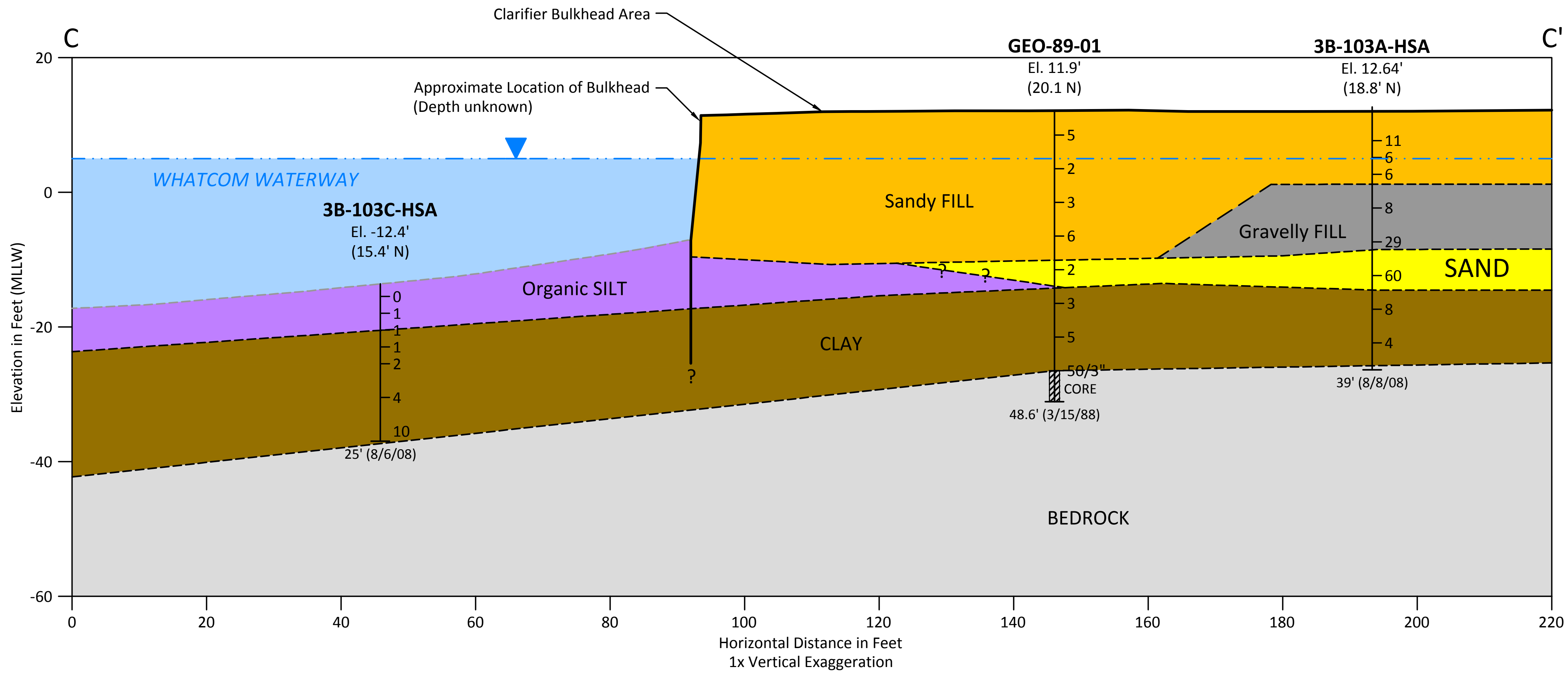


LEGEND:

Sandy FILL - Loose to medium dense, variable moisture, multi-colored, gravelly SAND

Gravelly FILL - Loose to medium dense, light to dark brown, coarse sandy GRAVEL with silt and wood fragments and shells.

SAND - Loose to dense, black and gray, gravelly SAND

Organic SILT- Very soft, black to dark gray, very sandy organic SILT with wood fragments

CLAY - Medium stiff , dark gray, clayey SILT to silty CLAY

BEDROCK - Gray Sandstone (Chuckanut Formation)

Existing Ground Surface

Mean Sea Level (+5 MLLW)

3D-104A-HSA — Boring ID
El. 25.5' — Surface Elevation (feet, MLLW)
(54' W) — Offset Distance in Feet
— Boring Location
18 — Uncorrected Blow Count (blows/feet)
— Bottom of Boring
30.0' (10/8/10)
— Completion Date
— Bottom of Boring in Feet

0 20
Scale in Feet

NOTES:

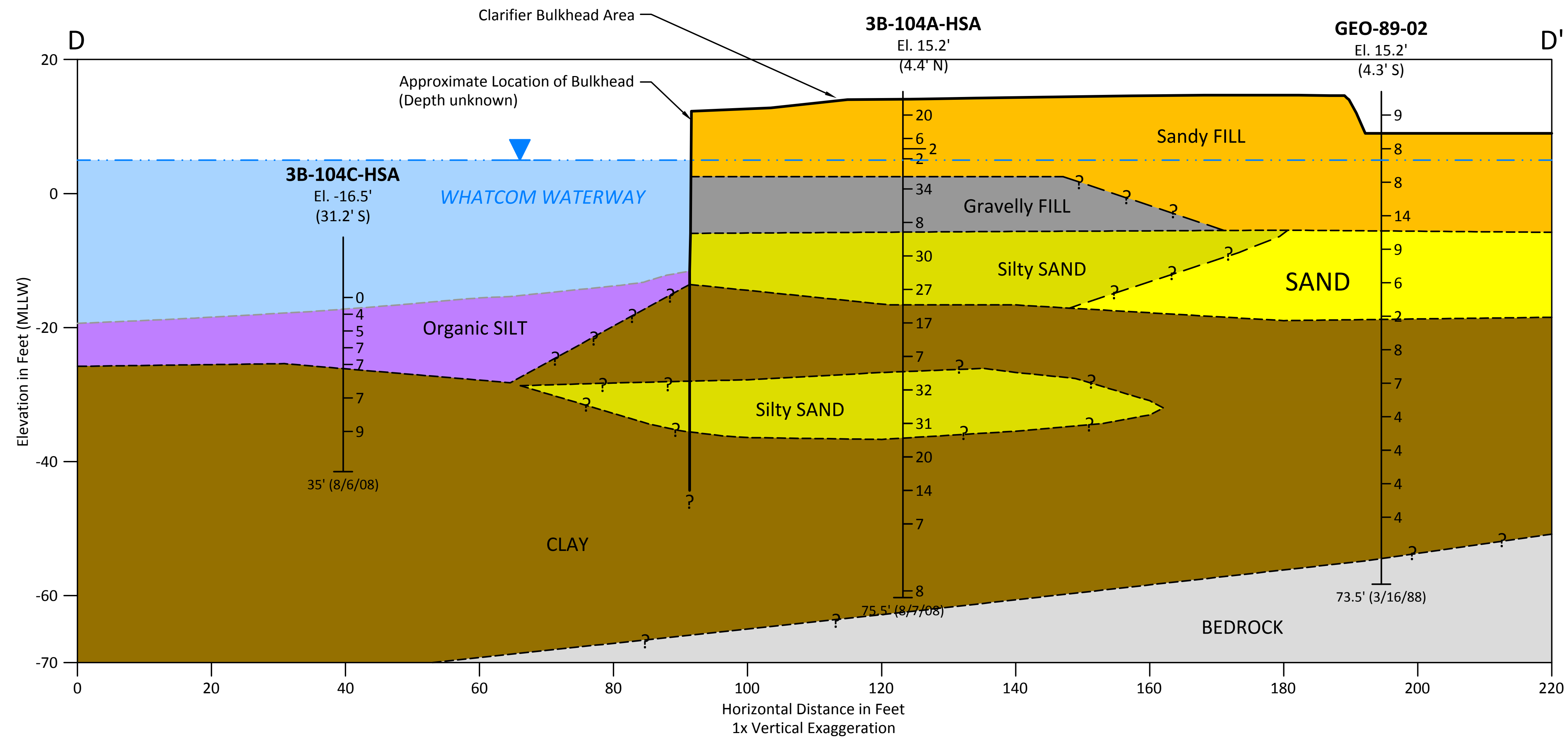
1. Geologic contacts are inferred between borings.
2. Soil descriptions are general. Actual conditions will vary locally.

T:\CAD\Projects\0007-Port of Bellingham\Whatcom Waterway Cleanup Inner Wtr\EDR\0007-BODR-002.dwg 4 Figure 3-4

Feb 06, 2015 2:51pm epipkin

ANCHOR QEA

Figure B-3.4
Geologic Cross Section C-C'
Appendix B - Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas - Final EDR



LEGEND:

- Sandy FILL - Loose to medium dense, variable moisture, multi-colored, gravelly SAND
- Gravelly FILL - Loose to medium dense, light to dark brown, coarse sandy GRAVEL with silt and wood fragments and shells.
- Silty SAND - Medium dense, dark brown to dark gray, silty SAND with shell fragments.
- SAND - Loose to dense, black and gray, gravelly SAND
- Organic SILT- Very soft, black to dark gray, very sandy organic SILT with wood fragments
- CLAY - Medium stiff to stiff, gray, silty CLAY with variable sand content and thin sand layers
- BEDROCK - Gray Sandstone (Chuckanut Formation)
- Existing Ground Surface
- Mean Sea Level (+5 MLLW)

3D-104A-HSA — Boring ID
El. 25.5' — Surface Elevation (feet, MLLW)
(54' W) — Offset Distance in Feet
— Boring Location
18 — Uncorrected Blow Count (blows/feet)
— Bottom of Boring
30.0' (10/8/10) — Completion Date
— Bottom of Boring in Feet

0 20
Scale in Feet

NOTES:

1. Geologic contacts are inferred between borings.
2. Soil descriptions are general. Actual conditions will vary locally.

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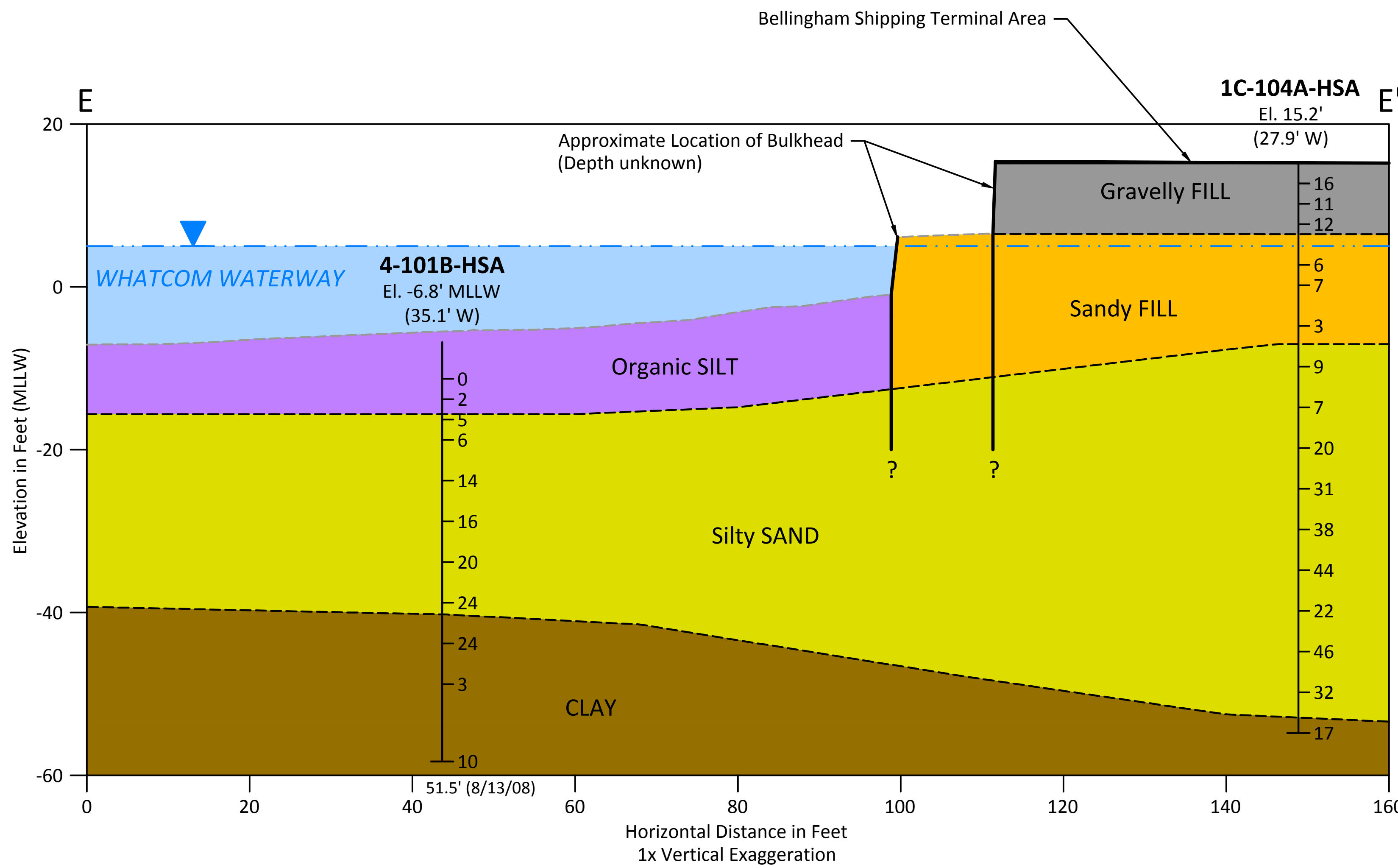
Figure B-3.5
Geologic Cross Section D-D'
Appendix B - Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas - Final EDR



LEGEND:

Gravelly FILL - Loose to medium dense, light to dark brown, coarse sandy GRAVEL with silt and wood fragments and shells.

Sandy FILL - Loose to medium dense, variable moisture, multi-colored, gravelly SAND

Organic SILT- Very soft, black to dark gray, very sandy organic SILT with wood fragments

Silty SAND - Medium dense, dark brown to dark gray, silty SAND with shell fragments.

CLAY - Medium stiff to stiff, gray, silty CLAY with variable sand content and thin sand layers

Existing Ground Surface

Mean Sea Level (+5 MLLW)

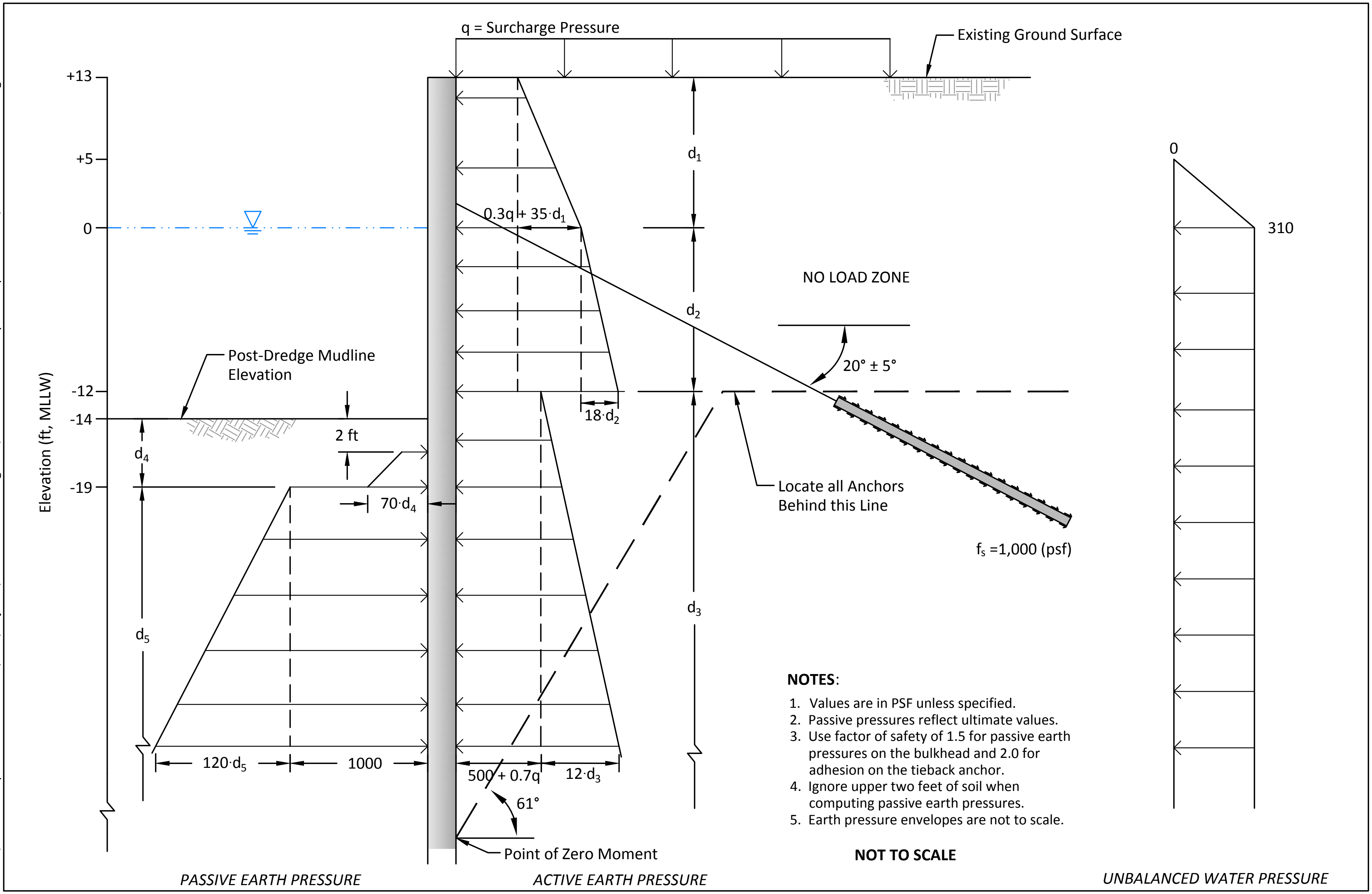
3D-104A-HSA — Boring ID
El. 25.5' — Surface Elevation (feet, MLLW)
(54' W) — Offset Distance in Feet
— Boring Location
18 — Uncorrected Blow Count (blows/feet)
— Bottom of Boring
30.0' (10/8/10)
Completion Date
Bottom of Boring in Feet

0 20
Scale in Feet

NOTES:
1. Geologic contacts are inferred between borings.
2. Soil descriptions are general. Actual conditions will vary locally.

ANCHOR QEA

Figure B-3.6
Geologic Cross Section E-E'
Appendix B - Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas - Final EDR



NOTES:

1. Values are in PSF unless specified.
2. Passive pressures reflect ultimate values.
3. Use factor of safety of 1.5 for passive earth pressures on the bulkhead and 2.0 for adhesion on the tieback anchor.
4. Ignore upper two feet of soil when computing passive earth pressures.
5. Earth pressure envelopes are not to scale.

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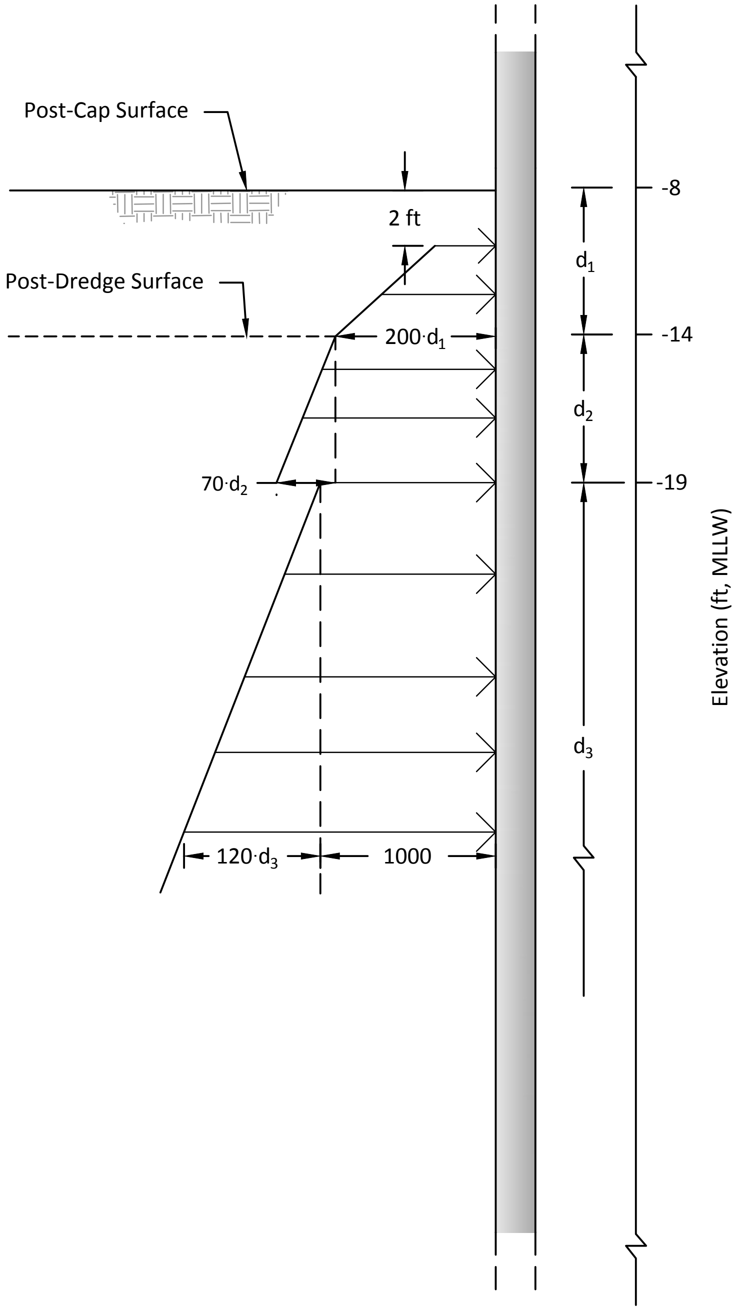
Feb 04, 2015 12:11pm chewett

Figure B-4.1
Earth Pressures at Maple Street Bulkhead (Post-Dredge Conditions)
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



PASSIVE EARTH PRESSURE

NOTES:

1. Values are in PSF unless specified.
2. Passive earth pressures reflect ultimate values.
3. Use FS = 1.5 for computing allowable passive earth pressures.
4. Ignore upper two feet of soil when computing passive earth pressures.
5. Earth pressures envelopes are not to scale.

NOT TO SCALE

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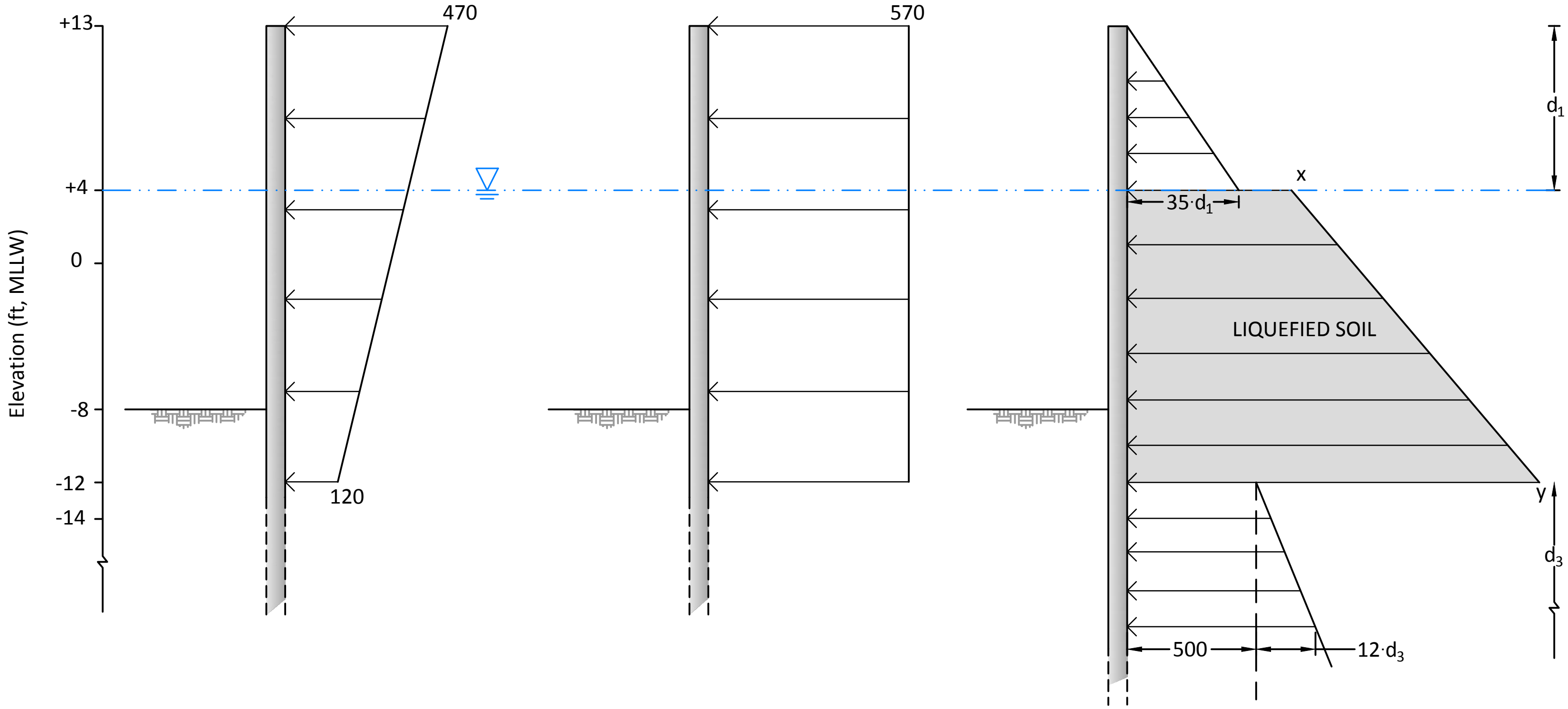
Figure B-4.2

Passive Earth Pressures at Maple Street Bulkhead (Post-Cap Conditions)

ANCHOR QEA



SEISMIC INCREMENT ONLY [2]

SEISMIC PLUS STATIC [2, 3]

LIQUEFIED CASE [2,4]

NOTES:

1. Values are in PSF unless specified.
2. Use with Figure B-4.2 for passive earth pressures.
3. Use with Figure B-4.1 for active earth pressures at elevations greater than -12' MLLW.
4. Values reflect seismic and static earth pressures. Use static active earth pressures below -12' MLLW.
5. Increase the allowable static passive earth pressure by one-third (1/3) for seismic design.

NOT TO SCALE

LIQUEFIED SOIL PRESSURE RECOMMENDATIONS			
	LOW	MID	HIGH
x	325	540	830
y	900	1500	1550

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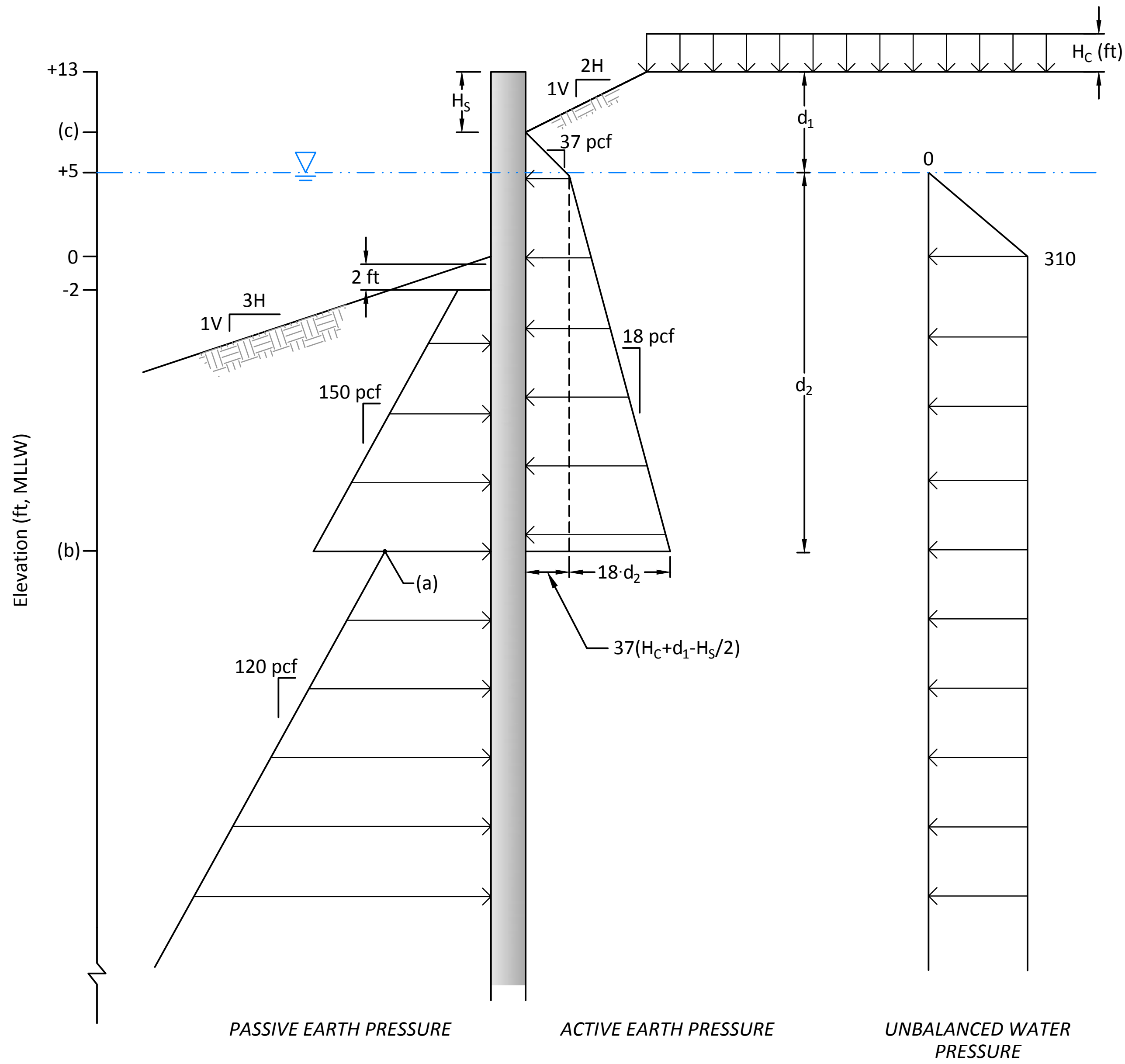
Figure B-4.3
Seismic Earth Pressures for Maple Street Bulkhead
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



PARAMETER	FORMER CHEVRON PROPERTY	SOUTH SHORELINE	NORTH SHORELINE
(a) (psf)	3450	2800	2800
(b) (ft, MLLW)	-20	-14	-10
Maximum depth of (c)	+5	+5	+0

NOTES:
1. Values are in PSF unless specified.
2. Passive earth pressures reflect ultimate values for post-dredge conditions.
3. Use FS = 1.3 for computing allowable passive earth pressures.
4. Ignore upper two feet of soil when computing passive earth pressures.
5. Earth pressures envelopes are not to scale.
6. Values for active and passive earth pressures were developed using formulations for cohesive soils as presented in Fang (91).

NOT TO SCALE

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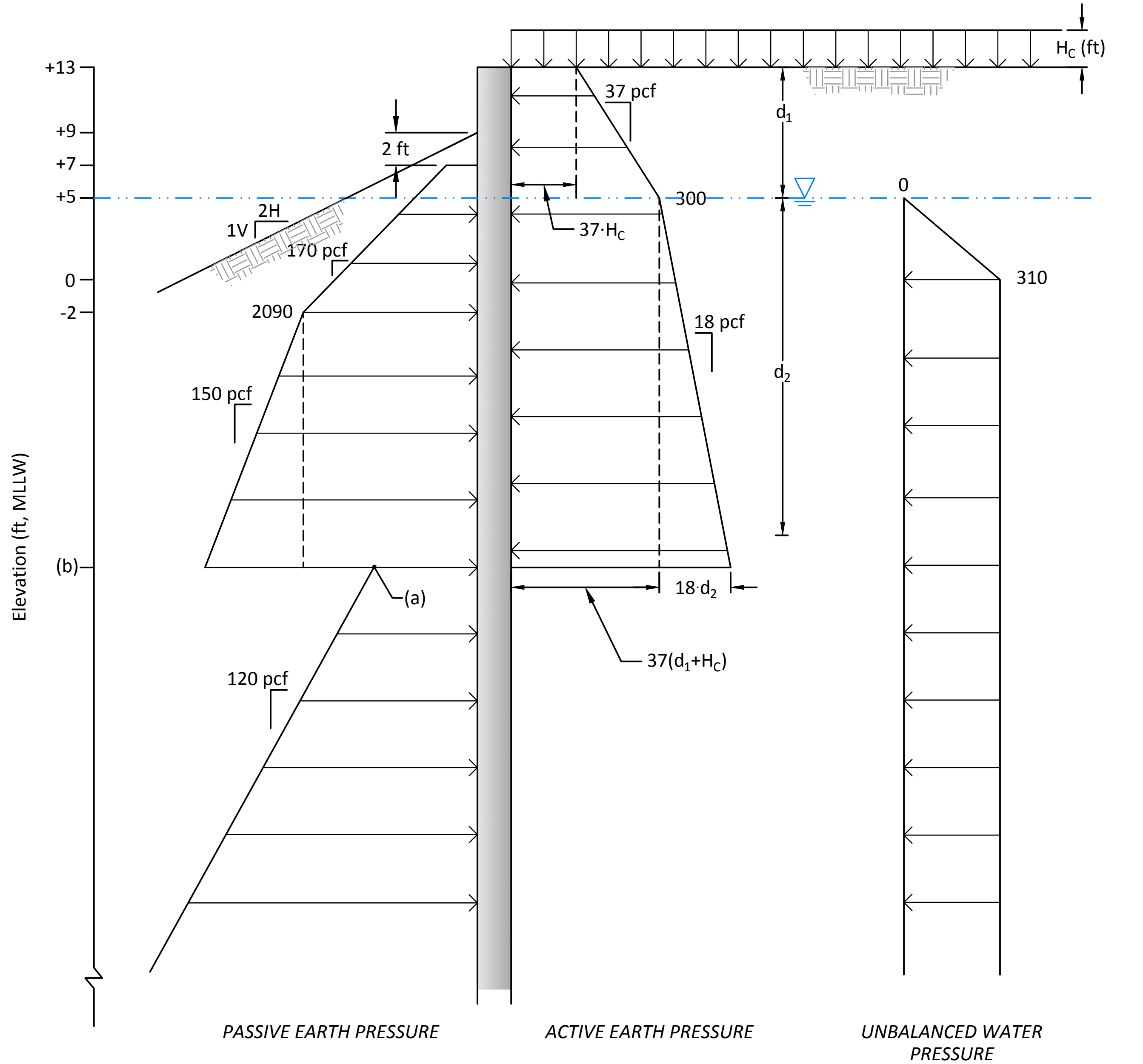
Figure B-4.4
Post-Dredge Earth Pressures for East and West Waterfront Walls
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR

ANCHOR QEA



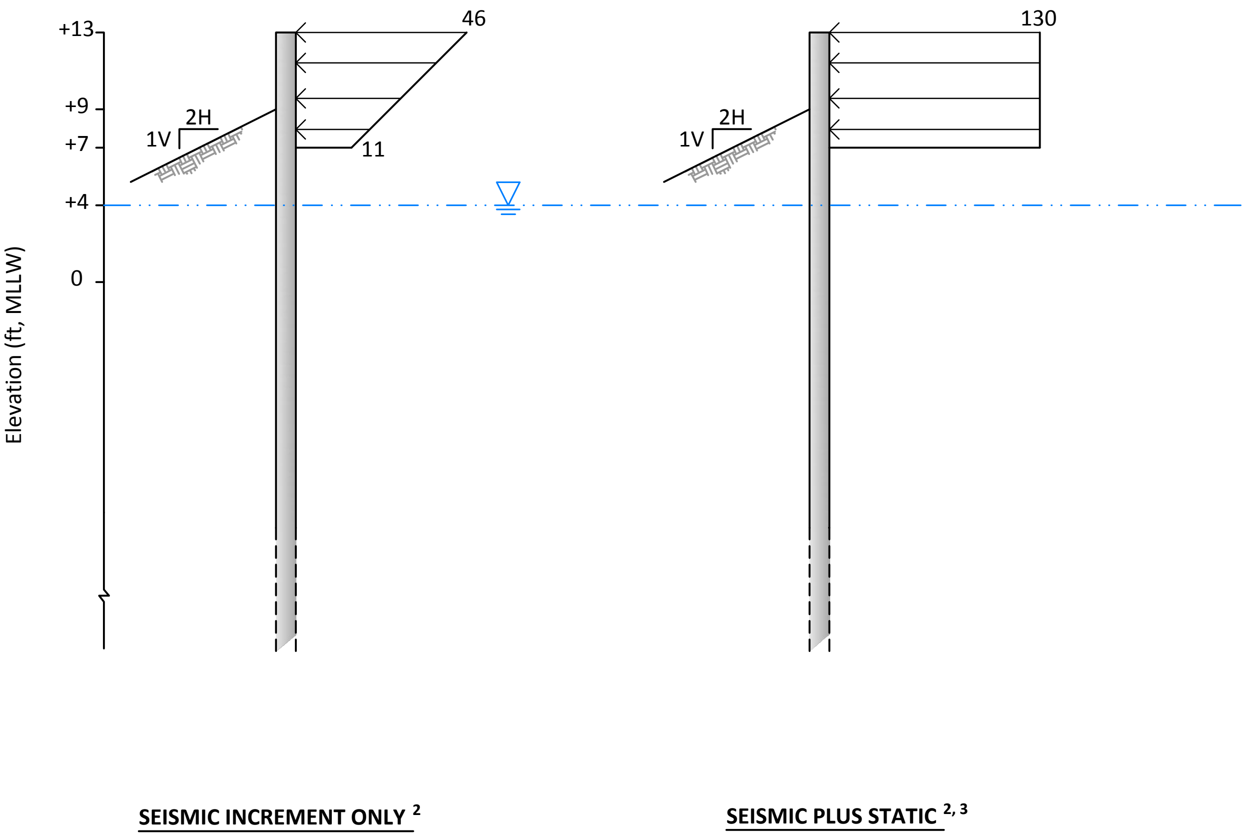
PARAMETER	FORMER CHEVRON PROPERTY	SOUTH SHORELINE	NORTH SHORELINE
(a) (psf)	3450	2800	2800
(b) (ft, MLLW)	-20	-14	-10

NOTES:

1. Values are in PSF unless specified.
2. Passive earth pressures reflect ultimate values for post-cap conditions.
3. Use FS = 1.5 for computing allowable passive earth pressures.
4. Ignore upper two feet of soil when computing passive earth pressures.
5. Earth pressures envelopes are not to scale.
6. Values for active and passive earth pressures were developed using formulations for cohesive soils as presented in Fang (91).

NOT TO SCALE

Figure B-4.5
Post-Cap Earth Pressures for East and West Waterfront Walls
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR

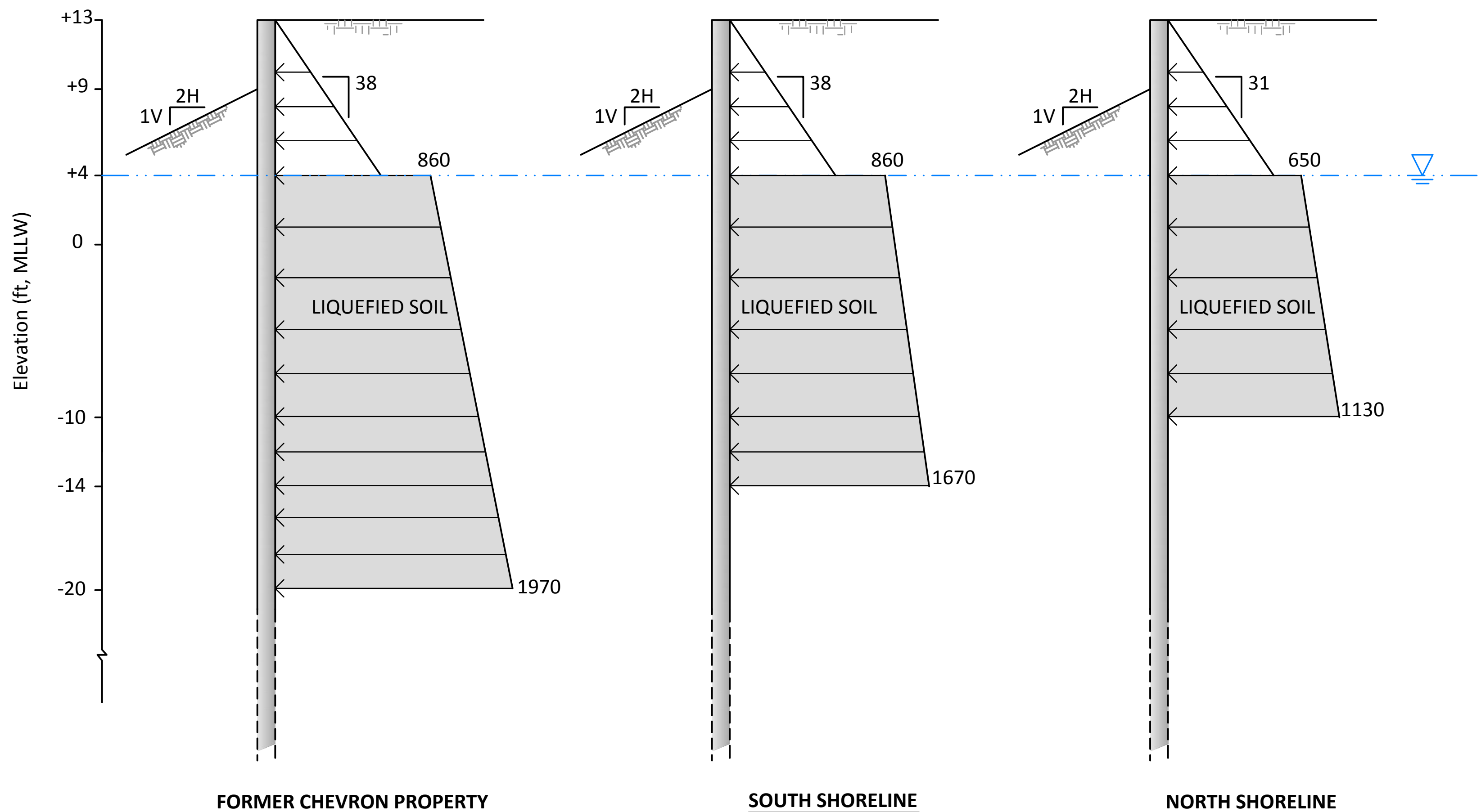


NOTES:

1. Values are in PSF unless specified.
2. Use with Figure B-4.5 for passive earth pressures.
3. Use with Figure B-4.4 for active earth pressures.
4. Use with Figure B-4.4 for active earth pressures at elevations greater than +7' MLLW.
5. Increase the allowable static passive earth pressure by one-third (1/3) for seismic design.

NOT TO SCALE

Figure B-4.6
Seismic Earth Pressures for East and West Waterfront Walls (Non-Liquified)
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



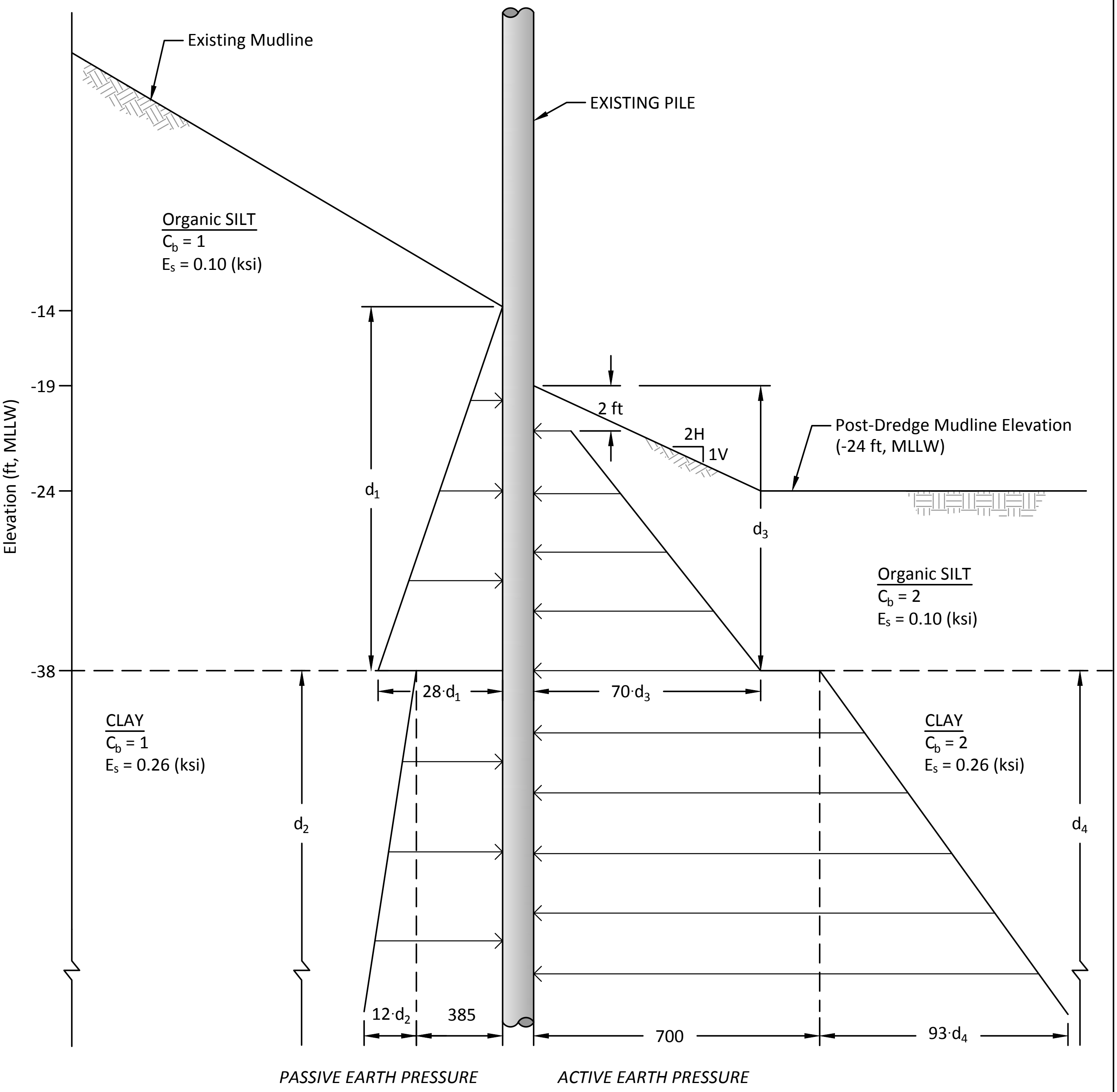
NOTES:

1. Values are in PSF unless specified.
2. Use with Figure B-4.5 for passive earth pressures.
3. Increase the allowable static passive earth pressure by one-third (1/3) for seismic design.
4. For seismic conditions, active earth pressure below liquefied soil can be neglected.

NOT TO SCALE

ANCHOR QEA

Figure B-4.7
Seismic Earth Pressures for East and West Waterfront Walls (Liquified)
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



LEGEND:

C_b Effective Pile Width

E_s Soil Modulus (ksi)

N_h Rate of Increase in Soil Modulus with Depth (ksi/ft)

NOTES:

1. Values are in PSF unless specified.
2. Passive pressures reflect ultimate values.
3. For passive earth pressures use FS = 1.5
4. Pressure envelopes are not to scale.
5. Ignore upper 2 feet of passive earth pressure.

NOT TO SCALE

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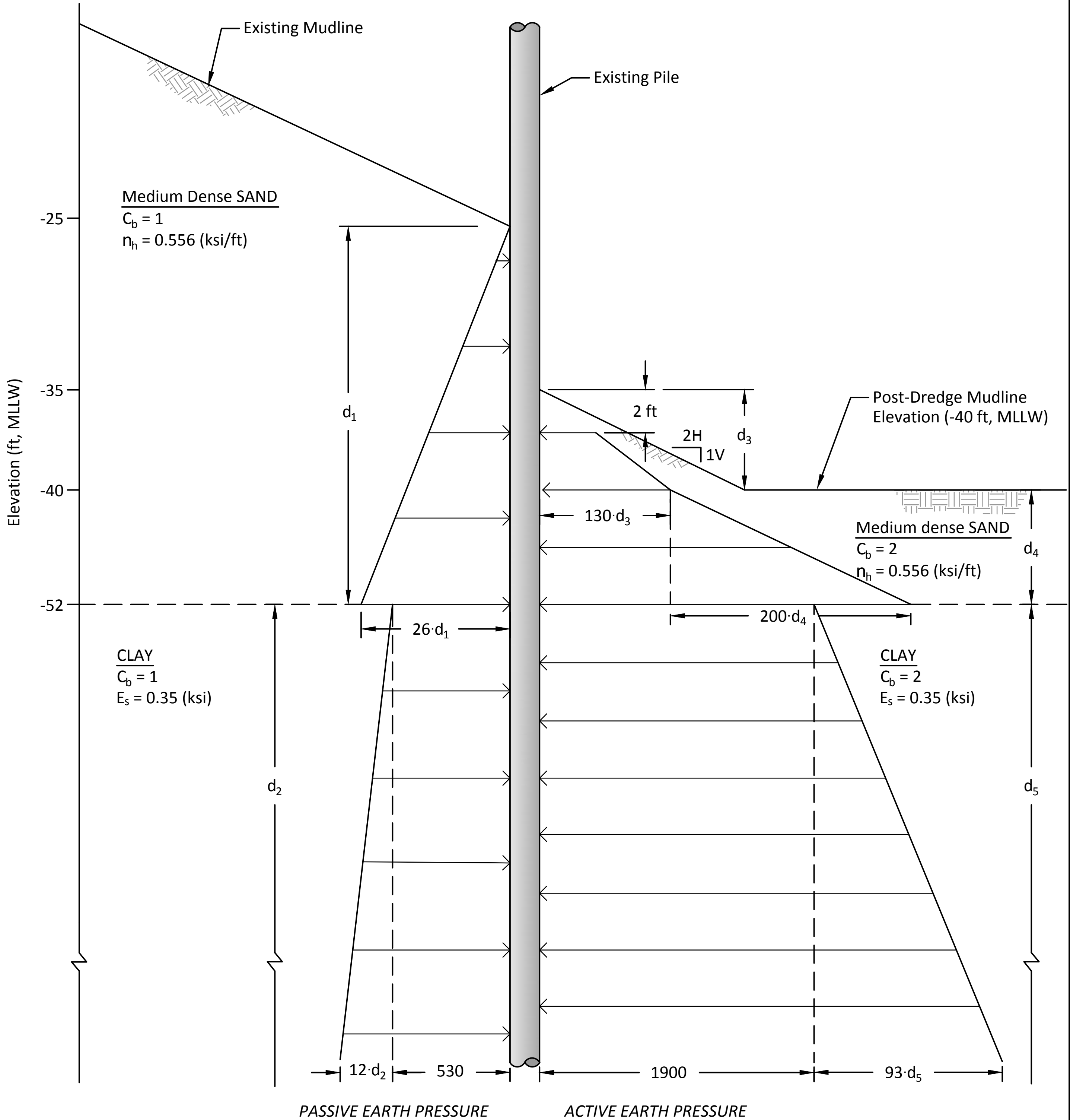
Figure B-4.8

Earth Pressures for G-P Dock Section



LEGEND:

C_b Effective Pile Width

E_s Soil Modulus (ksi)

N_h Rate of Increase in Soil Modulus with Depth (ksi/ft)

NOTES:

1. Values are in PSF unless specified.
2. Passive pressures reflect ultimate values.
3. For passive earth pressures use FS = 1.5
4. Pressure envelopes are not to scale.
5. Ignore upper 2 feet of passive earth pressure.

NOT TO SCALE

Figure B-4.9

Earth Pressures for BST Dock - Berth 1

Appendix B – Geotechnical Evaluation

Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR

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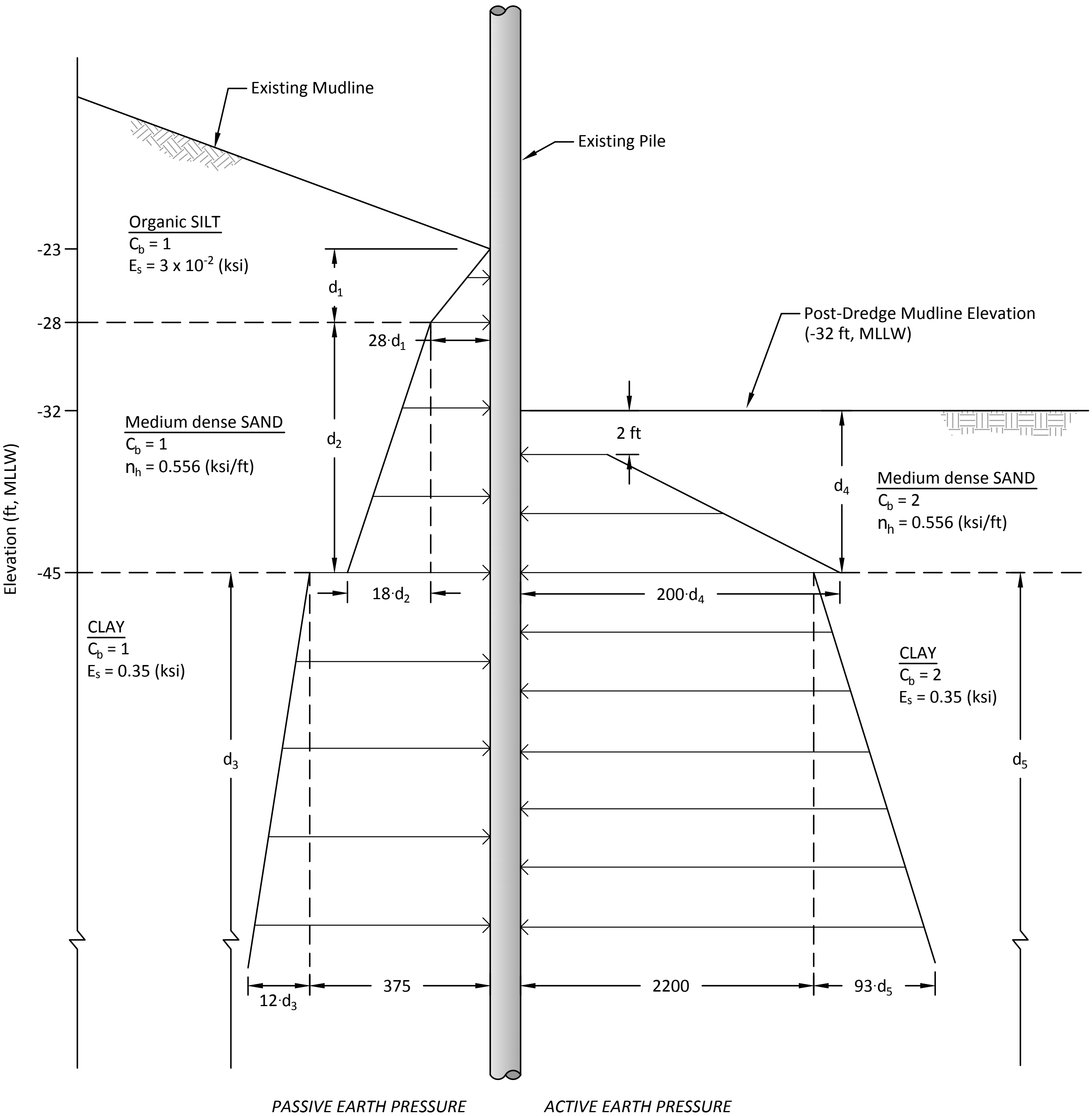

LEGEND:

C_b	Effective Pile Width
E_s	Soil Modulus (ksi)
N_h	Rate of Increase in Soil Modulus with Depth (ksi/ft)

NOTES:

1. Values are in PSF unless specified.
2. Passive pressures reflect ultimate values.
3. For passive earth pressures use FS = 1.5
4. Pressure envelopes are not to scale.
5. Ignore upper 2 feet of passive earth pressure.

NOT TO SCALE

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Figure B-4.10
Earth Pressures for BST Dock - Berth 2
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR

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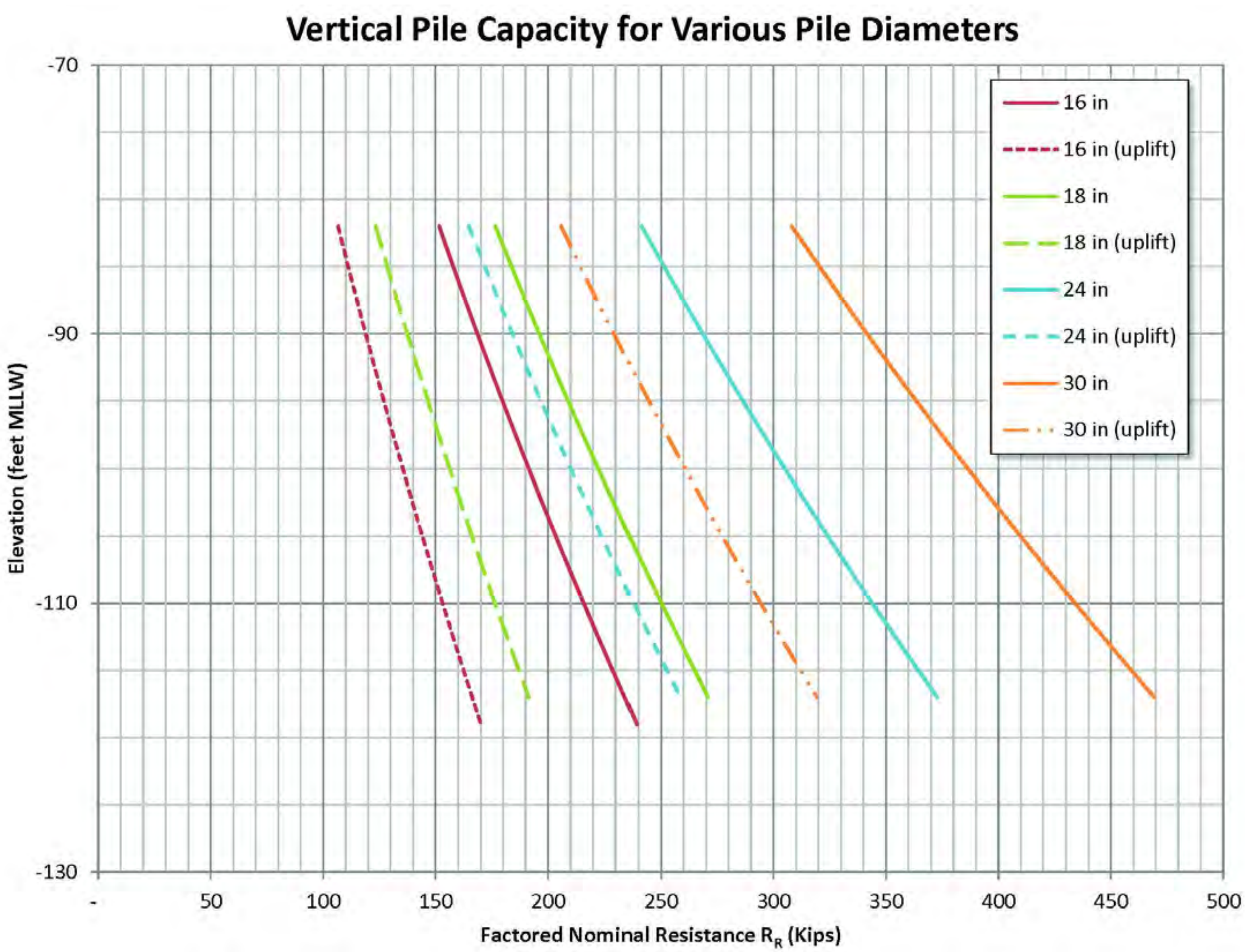
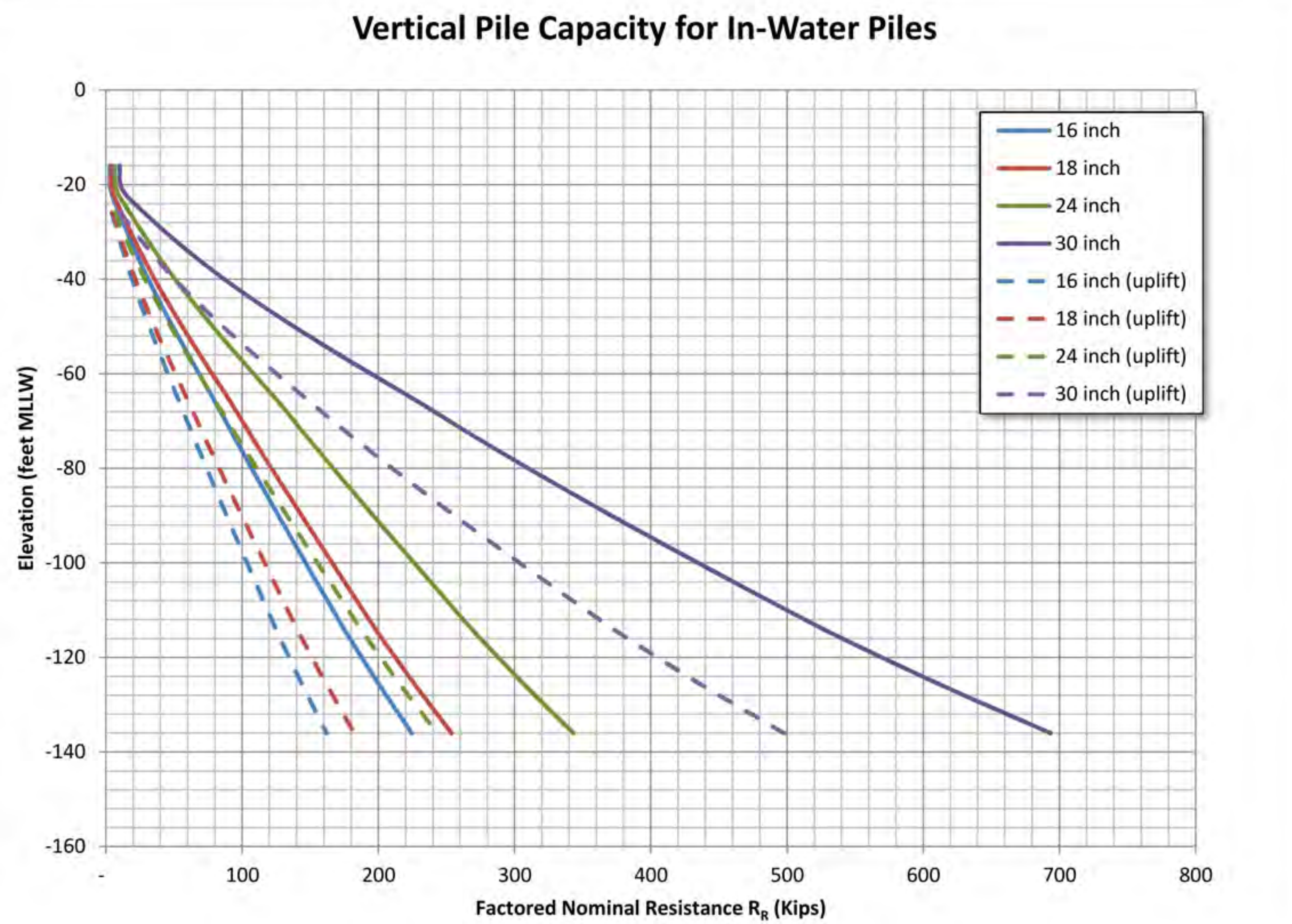


NOTES:

1. The diagram is to be used for planning and permitting purposes only and supports a strength limit state for fender and dolphin pile designs.
2. Pile spacing should be a minimum of 2.5 times the pile diameter.
3. Factored nominal resistance is based on steel pipe piles.

ANCHOR QEA

Figure B-4.11
Vertical Pile Capacity for Upland Piles
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR

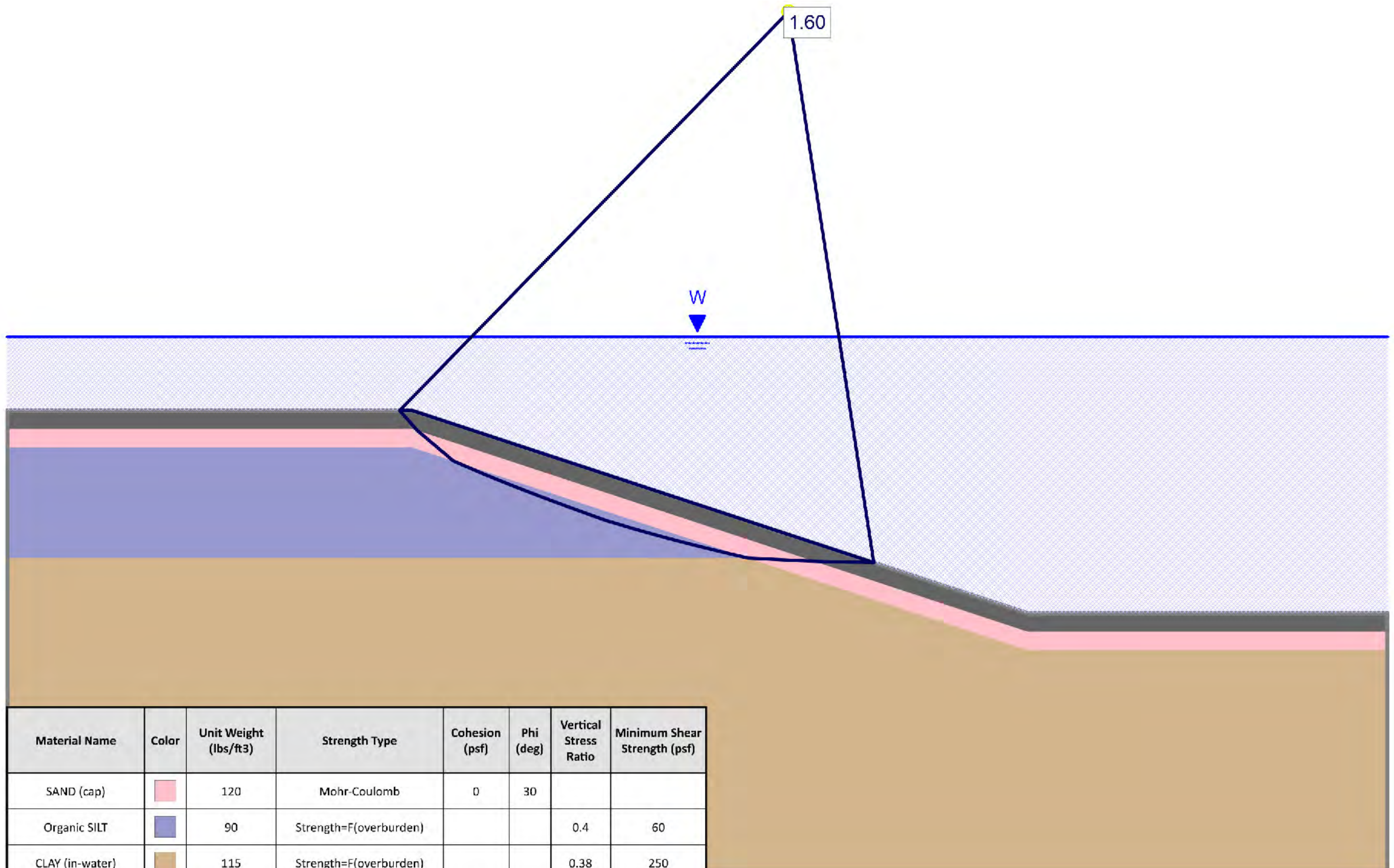


NOTES:

1. The diagram is to be used for planning and permitting purposes only and supports a strength limit state for the fender and dolphin pile designs.
2. Pile spacing should be a minimum of 2.5 times the pile diameter.
3. Factored nominal resistance is based on steel pipe piles.

ANCHOR QEA

Figure B-4.12
Vertical Pile Capacity for In-Water Piles
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)
SAND (cap)		120	Mohr-Coulomb	0	30		
Organic SILT		90	Strength=F(overburden)			0.4	60
CLAY (in-water)		115	Strength=F(overburden)			0.38	250
FILTER AND ARMOR		130	Mohr-Coulomb	0	38		

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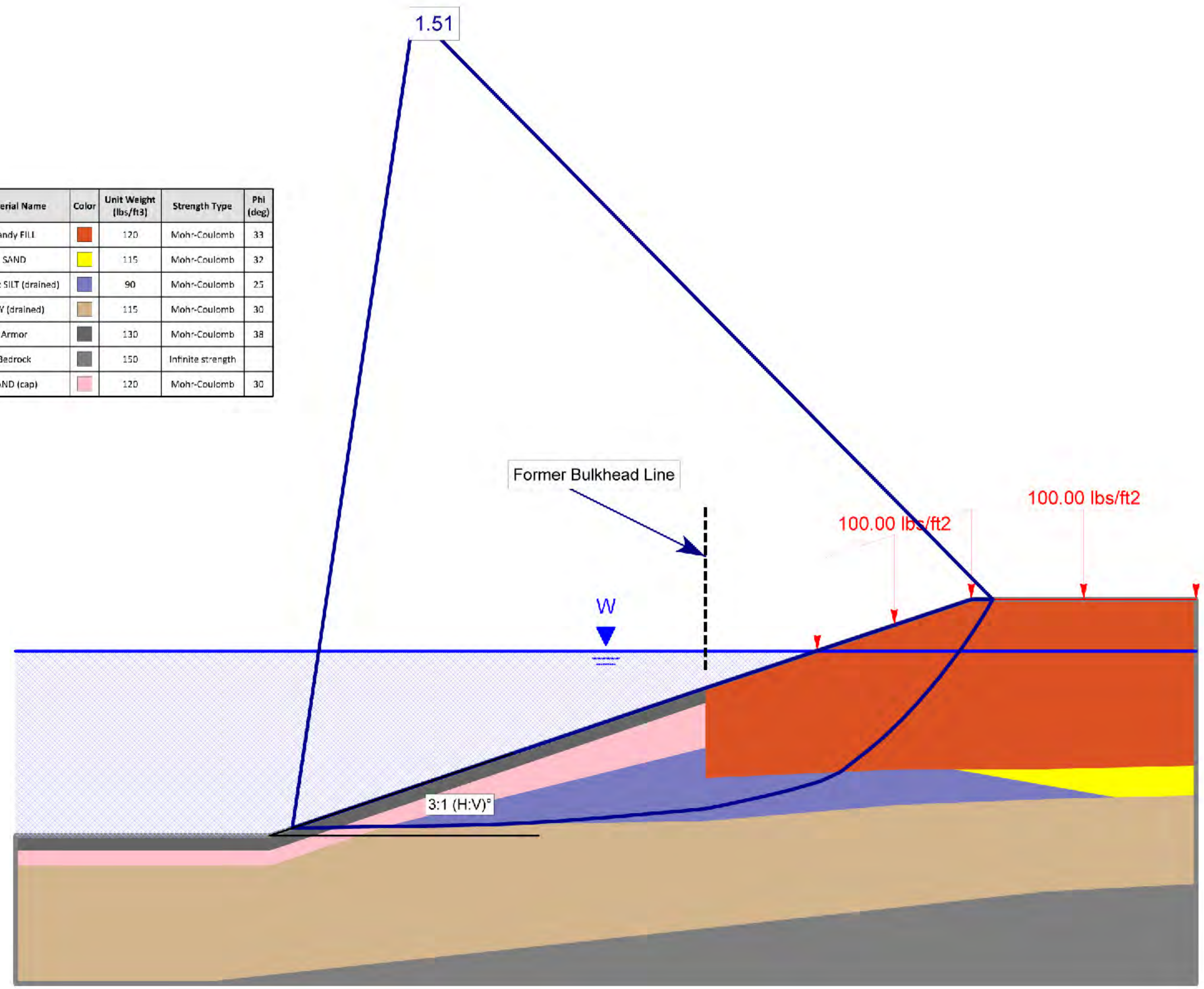
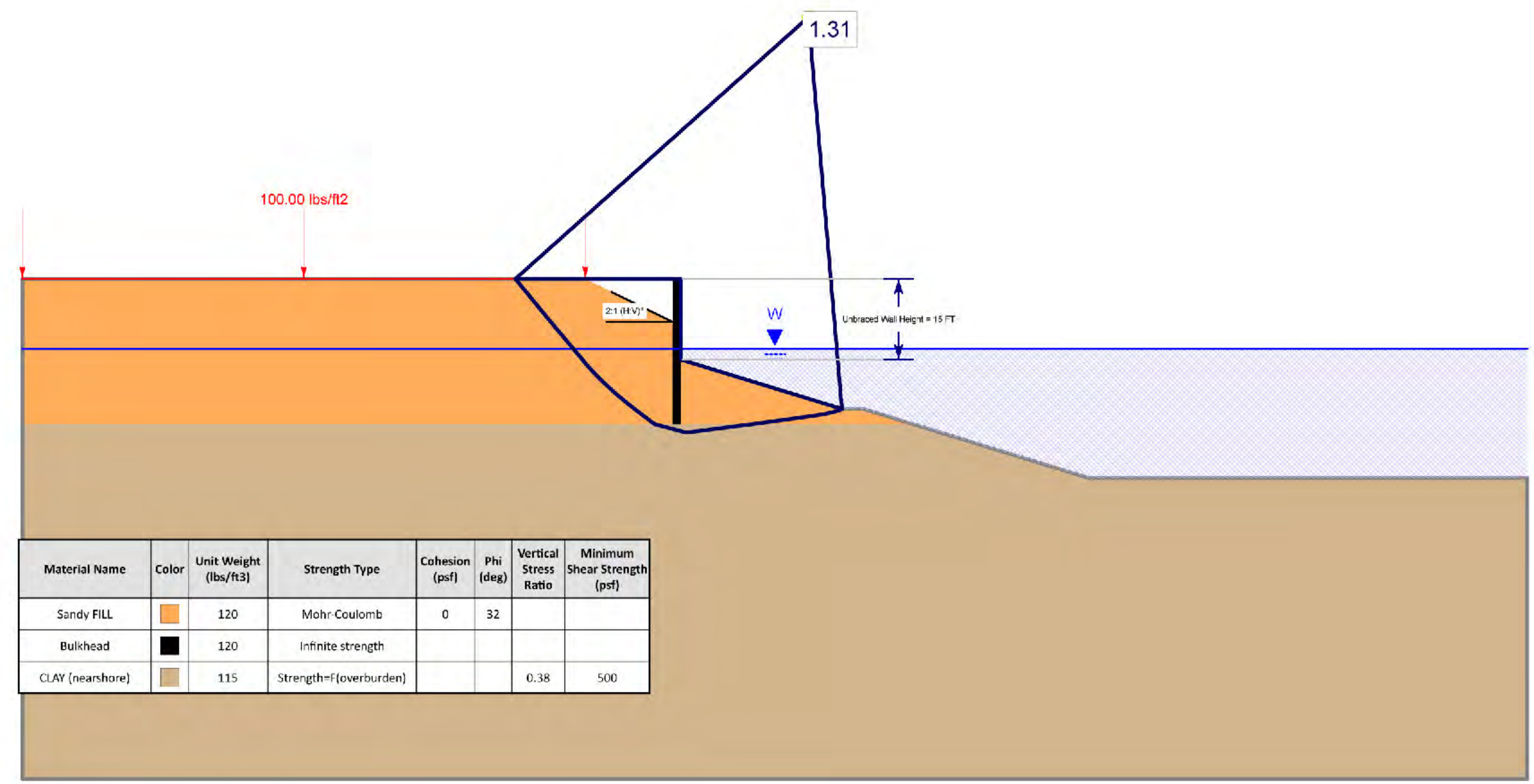
Figure B-5.1
In-water Dredge Prism Side Slopes – Post Cap – Short Term
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Phi (deg)
Sandy FILL		120	Mohr-Coulomb	33
SAND		115	Mohr-Coulomb	32
Organic SILT (drained)		90	Mohr-Coulomb	25
CLAY (drained)		115	Mohr-Coulomb	30
Armor		130	Mohr-Coulomb	38
Bedrock		150	Infinite strength	
SAND (cap)		120	Mohr-Coulomb	30

Figure B-5.2
South Shoreline – Post Cap – Long Term
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)
Sandy FILL		120	Mohr-Coulomb	0	32		
Bulkhead		120	Infinite strength				
CLAY (nearshore)		115	Strength=F(overburden)			0.38	500

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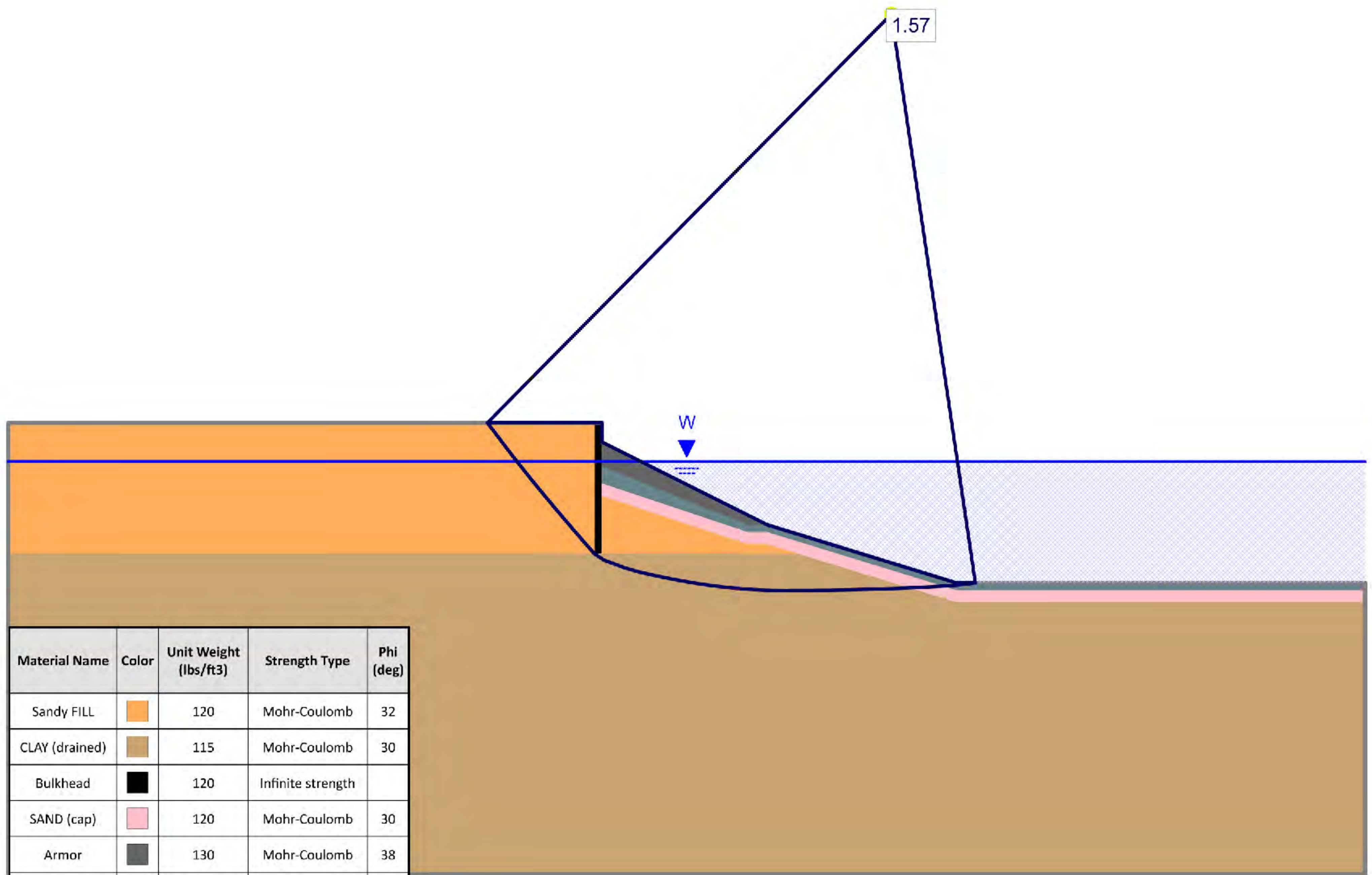
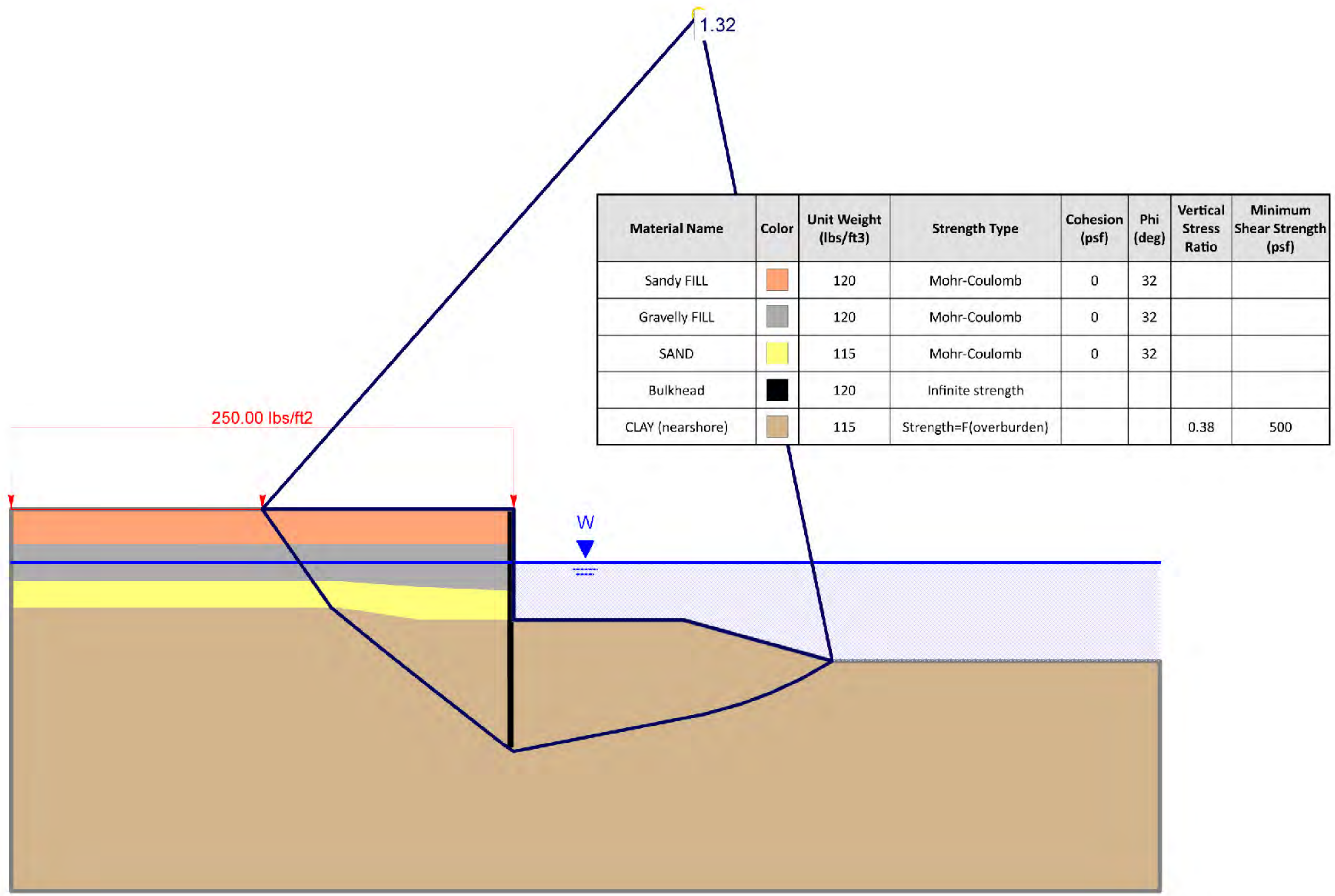
Figure B-5.3
Central Waterfront Shoreline Source Control Structures – Post Dredge – Short Term
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Phi (deg)
Sandy FILL		120	Mohr-Coulomb	32
CLAY (drained)		115	Mohr-Coulomb	30
Bulkhead		120	Infinite strength	
SAND (cap)		120	Mohr-Coulomb	30
Armor		130	Mohr-Coulomb	38
Filter/Gravel		125	Mohr-Coulomb	35

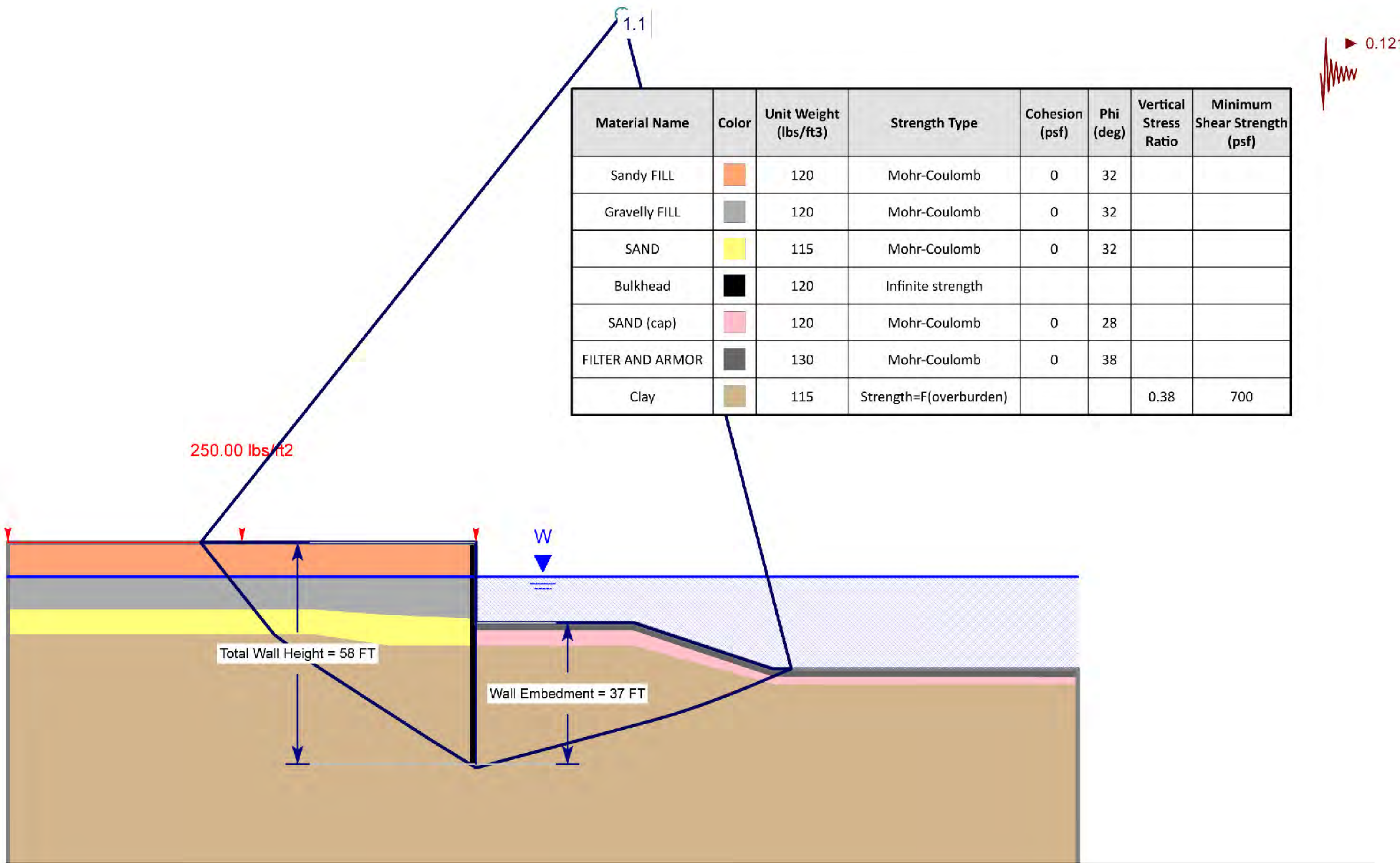
Figure B-5.4
Central Waterfront Shoreline Source Control Structures – Post Cap – Long Term
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)
Sandy FILL		120	Mohr-Coulomb	0	32		
Gravelly FILL		120	Mohr-Coulomb	0	32		
SAND		115	Mohr-Coulomb	0	32		
Bulkhead		120	Infinite strength				
CLAY (nearshore)		115	Strength=F(overburden)			0.38	500

ANCHOR QEA

Figure B-5.5
Maple Street Bulkhead – Post Dredge – Short Term
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)
Sandy FILL		120	Mohr-Coulomb	0	32		
Gravelly FILL		120	Mohr-Coulomb	0	32		
SAND		115	Mohr-Coulomb	0	32		
Bulkhead		120	Infinite strength				
SAND (cap)		120	Mohr-Coulomb	0	28		
FILTER AND ARMOR		130	Mohr-Coulomb	0	38		
Clay		115	Strength=F(overburden)			0.38	700

Figure B-5.6
Maple Street Bulkhead – Post Cap – Seismic
Appendix B – Geotechnical Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR

APPENDIX C
COASTAL EVALUATION

TABLE OF CONTENTS

List of Tables

List of Figures

List of Attachments

ACRONYMS AND ABBREVIATIONS

°	degree
3-D	three-dimensional
ACES	Automated Coastal Engineering System
CEM	Coastal Engineering Manual
CERC	Coastal Engineering Research Center
cfs	cubic feet per second
CHE	Coast & Harbor Engineering
H:V	horizontal to vertical
MHHW	mean higher high water
MLLW	mean lower low water
MOST	Method of Splitting Tsunamis
NOAA	National Oceanic and Atmospheric Administration
propwash	propeller wash
USACE	U.S. Army Corps of Engineers
Waterway	Whatcom Waterway

1 INTRODUCTION

As part of the Whatcom Waterway (Waterway) Cleanup in Phase 1 Site Areas, a coastal engineering evaluation was conducted to develop design criteria for nearshore cap and shoreline stabilization within the Inner Waterway and Log Pond areas. The evaluations included current and wave data collection, tidal circulation modeling, and wave transformation modeling. This appendix outlines:

- Data used to complete the evaluation
- Methodology employed
- Design criteria developed for nearshore engineered cap areas and shoreline stabilization
- Results of the evaluation

2 DATA USED FOR ANALYSIS

The coastal engineering evaluation for Whatcom Waterway used multiple data sources for wave modeling and hydrodynamic modeling. Previous engineered capping efforts were also reviewed. The different data sources used in these evaluations are outlined in Table C-1.

Table C-1
Summary of Data Used for Evaluation

Data Type	Data Source (Author and Date)	Purpose of Data
Hourly wind	Bellingham International Airport (1973-2006)	Wave modeling input
Hourly wind	Sandy Point Shores (2004-2006)	Wave modeling input
Flow meter	USGS	Hydrodynamic model input
Design elevations	Anchor QEA 60% Design Drawings (2012)	Depths
Datum	NOAA Station No. 9449211	Water levels (see Table C-2)
Tidal constituents	n/a	Hydrodynamic model input
Wave data	Two wave meters (March-May 2008); collected by Evans Hamilton	Calibrate/validate wave model and hydrodynamic model
Current data	Four current meters (March-May 2008); collected by Evans Hamilton	Calibrate/validate hydrodynamic model
Bathymetry	Survey, Wilson Engineering (2008 and 2009)	Existing bathymetry
Bathymetry	Pre-Log Pond cap survey (Wilson)	Current cap thicknesses

The various datums for this Project in reference to mean lower low water (MLLW) are listed in Table C-2.

Table C-2
Datum Elevations (NOAA Station No. 9449211)

Tide Level	Meters (MLLW)	Feet (MLLW)
Highest Observed (1/5/1975)[1]	3.177	10.4
Ordinary High Water[2]	3.116	10.2
Mean Higher High Water	2.594	8.5
Mean High Water	2.375	7.8
Mean Tide Level	1.546	5.1
Mean Sea Level	1.510	5.0
Mean Low Water	0.718	2.4
Mean Lower Low Water	0.000	0.0
Lowest Observed (12/30/1974)[3]	-1.057	-3.5
NAVD 88	0.147	0.5

Notes:
1. Station No. 9449211 was active from March 30, 1973, to July 21, 1975. Tidal predictions for the area have been higher and lower than those observed.
2. Ordinary high water evaluated as part of the Bellingham Shipping Terminal Bulkhead and Pier Repair and Replacement Project (not determined from referenced NOAA Station)

NOAA = National Oceanic and Atmospheric Administration
MLLW = mean lower low water
NAVD 88 = North American Vertical Datum of 1988

3 METHODS OF ANALYSIS

The armor size requirements throughout the Waterway are based on the combined influences of waves, currents, and propeller wash (propwash). This appendix provides a summary of the coastal evaluation conducted to determine stable sediment and rock sizes within the Waterway due to waves and tidal currents. A detailed discussion of the propwash evaluation is provided in Appendix D.

Tasks conducted as part of this work are described in the following sections:

- Defined general design criteria for the Project
- Developed design wave heights and periods (wave modeling)
- Developed estimates of near-bed velocities due to tidal and riverine currents (hydrodynamic modeling)
- Determined stable armor size and layer thickness under influence of design hydrodynamic forces
- Considered the potential impacts of tsunami events in addition to the cap design evaluations for wind and wave erosion and propwash

3.1 General Design Criteria

Site-wide basis of design criteria for coastal engineering considerations include the following:

- The 100-year return period event was used to evaluate armor stability under hydrodynamic forcing for the Project. The 100-year recurrence interval is the general standard for engineered cap design per guidance outlined in *Guidance for Subaqueous Dredged Material Capping* (USACE 1998) and *Contaminated Sediment Remediation Guidance for Hazardous Waste Sites* (USEPA 2005).
- Wave conditions at the Project Site were based on results of numerical modeling for 100-year recurrence interval events determined through evaluation of long-term wind data.
- Tidal and riverine currents at the Project Site (bed-velocities) were estimated using numerical modeling for a greater than 100-year recurrence interval event for freshwater inflow and spring tide conditions (largest elevation difference between subsequent high and low tides). The design fresh water inflow from Whatcom Creek

was taken as the sum of the estimated 100-year flow (extrapolated from gage data collected upstream of Whatcom Falls) and the maximum flow out of the control structure at Lake Whatcom (at the headwaters of Whatcom Creek). This represents almost twice the 100-year flow in the creek estimated from long-term gage data alone.

- Stable sediment and armor size for shoreline areas impacted by waves were calculated using guidance in the U.S. Army Corps of Engineers (USACE) Coastal Engineering Manual (CEM) (USACE 2002) assuming a "no damage" (no movement) condition. This is a conservative assumption in terms of size of armor.
- Stable sediment and armor size for areas impacted by currents (tidal and propwash) are defined as the $D_{95\ size}$ fraction of a normally distributed sediment gradation. This is a conservative assumption relative to USACE guidance (USACE 1998), which evaluates stable sediment and armor size based on the D_{50}.[1] of the sediment and armor size distribution.
- A potential rise in sea level of 2.4 feet by 2100 was considered as part of remedial design. This potential sea level rise is consistent with the evaluation documented in the *Waterfront District Redevelopment Final Draft EIS* (Port of Bellingham 2010) completed by the Port in July 2010. This would result in a future predicted mean higher high water (MHHW) elevation of 11.4 feet MLLW (in comparison to the current MHHW elevation of 8.5 feet based on MLLW defined by the current tidal epoch).

3.2 Design Wave Conditions

Design wind waves parameters (100-year return interval) were estimated by Coast & Harbor Engineering (CHE) using numerical modeling. Wave modeling for 100-year wave conditions was completed using predicted 100-year winds speeds from 240-degree (southwest) and 270-degree (south) directions, which represent the most impactive angles of wave attack for the Waterway. Wave model runs were also run under both MLLW and MHHW tide conditions (based on the National Oceanic and Atmospheric Administration [NOAA] tidal datum at Bellingham, Washington).

[1] The D_{50} of a sediment/rock gradation is by definition equal to or less than the D_{95} of the same gradation.

Additional details regarding this modeling effort have been documented in a memorandum developed by CHE for Anchor QEA that is provided as Attachment 1 to this appendix (CHE 2012).

Three models were used for the modeling: simulating waves nearshore (SWAN), HWAVE, and HWAVE Spectral. A nested modeling approach was used. Wave heights (100-year) within Bellingham Bay were developed using a large scale model of Bellingham Bay and adjacent waterbodies for wind directions of interest (SWAN). This information was then used as input to a smaller Project-scale model to provide high-resolution predictions of 100-year wave heights within the Waterway (HWAVE). The largest 100-year wave heights and periods predicted in each area of the Waterway (for each wave direction of interest) were then used to calculate the required shoreline stabilization requirements (armor size).

In areas outside the breaking wave zone (deeper water), wave heights and periods predicted by the model were used to estimate near-bed velocities due to waves in the Waterway. These velocities were calculated based on stream wave theory developed by Robert G. Dean (Dean and Dalrymple 1991).

3.3 Tidal Currents

Near-bed current velocities were estimated using three-dimensional (3-D) hydrodynamic modeling (using the SELFE model) conducted by CHE (CHE 2012). Details regarding this modeling effort have been documented in a memorandum developed by CHE for Anchor QEA that is provided as Attachment 1 to this appendix (CHE 2012). The input conditions for the model consisted of spring tide conditions (largest predicted elevation difference between subsequent high and low tides) and an extreme flow event from Whatcom Creek. The flow in Whatcom Creek was estimated as the sum of the 100-year flow of Whatcom Creek estimated using Lake Whatcom Gage Station No. 12203500 at Whatcom Creek (approximately 1,200 cubic feet per second [cfs]) and the maximum flow through the control structure located at Lake Whatcom (approximately 1,200 cfs). This resulted in an estimated flow of 2,400 cfs, which is twice the magnitude of the 100-year flow estimated from long-term gage data and, therefore, represents an extreme event beyond the 100-year recurrence interval.

3.4 Stable Armor Size

Stable armor size was determined through joint evaluation of the impacts of tidal currents, waves, and propwash in open water and nearshore areas. The stable armor size was chosen as the maximum calculated armor size in each area of the Waterway based on all these evaluations. Stable armor size varies throughout the Waterway depending on which hydrodynamic process is more impactful in that area. The following sections describe methodology and results (Section 4) for stable armor size due to impacts from waves and currents. The evaluation of stable armor size due to propwash is described in a separate attachment (Appendix D).

3.4.1 Breaking Waves

The armor sizing was calculated based on methods outlined in the CEM (USACE 2002). An overview of these calculations is provided below.

The 50 percent passing stone weight can be calculated using Equations 1 through 7:

$$W_{50} = w_r \left[\frac{H_s}{N_s \left(\frac{w_r}{w_w} - 1 \right)} \right]^3 \qquad (1)$$

Where:

W_{50}	=	median weight of the armor stone
w_r	=	unit weight of armor stone (assumed to be quarrystone or granite)
H_s	=	significant wave height
w_w	=	unit weight of water (saltwater)
N_s	=	stability number (higher value between Dutch and Coastal Engineering Research Center [CERC] methods; page 4-4-6 of USACE 1992)

CERC N_s =

$$N_{s-zero} = \frac{1.45}{1.27}(\cot\theta)^{\frac{1}{6}} \tag{2}$$

Where:

θ = is the slope of the shore

Dutch N_s =

$$N_s = 6.2 * P^{0.18} * \left(\frac{S}{\sqrt{N}}\right)^{0.2} * (\xi_z)^{-0.5} \tag{3}$$

Where:

P = permeability coefficient (P = 0.4; Figure 4-4-2b of USACE 1992)

S = damage level (S = 2, start of damage; Table 4-4-1 of USACE 1992)

N = number of waves (N = 7,000; page 4-4-3 of USACE 1992)

ξ_z = surf similarity parameter

ξ_{om} = mean surf similarity parameter defined as:

$$\xi_{om} = \frac{\tan\theta}{\sqrt{s_{om}}} \tag{4}$$

Where:

s_{om} = mean wave steepness and is defined as:

$$s_{om} = \frac{2\pi}{g}\frac{H_s}{{T_m}^2} \tag{5}$$

Where:

T_m = mean wave period and is defined as:

$$T_m = T_s\left(\frac{0.67}{0.80}\right) \tag{6}$$

It is necessary to multiply by the shallow-water correction factor, which is 1.2 (page 4-4-4 of USACE 1992):

$$\text{Corrected Dutch } N_s = N_s * 1.2 \tag{7}$$

Once the median weight of the armor stone (W_{50}) is calculated based on the significant wave height and period, the required armor gradation (W_{max}, W_{min}, W_{85}, and W_{15}) can be solved for using Equations 8 through 11.

Weight of the largest stone:

$$W_{\max} = 4W_{50} \tag{8}$$

Weight of the smallest stone:

$$W_{\min} = \frac{1}{8}W_{50} \tag{9}$$

Percentage of total weight of gradation contributed by stones of lesser weight:

$$W_{85} = 1.96W_{50} \tag{10}$$

$$W_{15} = 0.4W_{50} \tag{11}$$

Using the weight of the stones, the Equation 12 can be used to solve for the dimensions of the armor stones, where the subscript *x* indicates the percentage of the weight of the total gradation contributed by stones of lesser weight.

$$D_x = \left(\frac{W_x}{w_r}\right)^{\frac{1}{3}} \tag{12}$$

In most cases, the armor material is too large to place directly onto cap material or native sediment (due to risk of sediment piping) and, therefore, a filter material is needed. The filter layer sizing for the slopes was calculated using a method outlined in the Automated Coastal Engineering System (ACES) (USACE 1992) and is shown in Equations 13 and 14. The D_{15} of the armor layer is used to find the D_{85} of the filter, then the D_{85} of the filter can be used to solve for the D_{50} and D_{15} of the filter.

$$\frac{D_{15(Armor)}}{D_{85(Filter)}} = 4 \tag{13}$$

$$\frac{D_x}{D_{50}} = e^{(0.01157\,(x) - 0.5785)} \tag{14}$$

The thicknesses of the layers are based on standards outlined in the CEM, which states that the thickness of the armor should be two times the D_{50} of the armor stone. The thickness of the filter is found by dividing the thickness of the armor layer by 4; however, the filter must have a minimum thickness of 1 foot.

The extent of the armor is based on how far a wave will travel up the slope (run up) and how far down the breaking wave will impact the slope. The run up was calculated based on methods outlined in ACES (USACE 1992) (Equations 15 to 18). The lower limit of riprap protection was based on guidance provided in the USACE CEM (2002), which defines the lower extent of shoreline armoring in the breaking wave zone in relation to the design wave height at the toe of the structure. The lower armor extent is equal to 1.5 times the height of the design wave below the expected extreme low water.

$$R_{\max} = H_{mo}\left(\frac{a\xi}{1+b\xi}\right) \tag{15}$$

Where:

$R_{\max}$	=	maximum wave run up (feet)
a	=	1.022
b	=	0.247
H_{mo}	=	energy-based zero-moment wave height (feet) defined as:

$$H_{mo} = \frac{H_s}{\exp\left[C_o\left(\frac{d}{gT_p^{\,2}}\right)^{-C_1}\right]} \tag{16}$$

Where:

C_o	=	0.00089 (page 4-4-9 of USACE 1992)
C_1	=	0.834 (page 4-4-9 of USACE 1992)
g	=	gravity (32.2 feet/s^2)
H_s	=	significant wave height (feet)
d	=	depth at toe of structure (feet)
T_p	=	period of peak energy density of the wave spectrum defined as:

$$T_p = \frac{T_s}{0.80} \tag{17}$$

Where:

T_s	=	significant wave period
ξ	=	surf similarity parameter defined as:

$$\xi = \frac{\tan\theta}{\left(\frac{2\pi H_{mo}}{gT_p^{\,2}}\right)^{1/2}} \tag{18}$$

Where:

θ = slope angle

3.4.2 Currents

Currents in the Waterway are caused by tidal influence and fresh water flows from Whatcom Creek. CHE developed and implemented a 3-D flow circulation model (SELFE) of Bellingham Bay and the Waterway to predict the bottom velocities caused due to tidal circulation and the estimated 100-year flow from Whatcom Creek. Details regarding this modeling effort are also documented in a memorandum (CHE 2012) developed by CHE and provided as Attachment 1 to this appendix.

The velocity outputs from the model were then used to calculate stable stone sizes based on a method outlined in Hydraulic Design of Flood Control Channels (USACE 1994) (see Equations 19 and 20).

$$\frac{D_{30}}{h} = S_f C_s \left[\left(\frac{w_w}{w_r - w_w} \right)^{1/2} \left(\frac{\bar{u}}{\sqrt{K_1 g h}} \right) \right]^{5/2} \tag{19}$$

Where:

D_{30} = the diameter in which 30 percent of the sediment passes

h = depth

S_f = safety factor (minimum of 1.1)

C_s = stability coefficient for incipient motion (0.3 for angular stone and 0.38 for rounded stone)

$\bar{u}$ = mean flow velocity

g = gravity

K_1 = defined as:

$$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \varphi}} \tag{20}$$

Where:

θ = channel side wall slope

ϕ = angle of repose of blanket armor (approximately 40 degrees [°] for riprap)

The gradation and filter sizing for this analysis are calculated the same as for the wave armor requirements detailed in Section 2.1.

3.5 Consideration of Potential Tsunami Events

An evaluation was completed to determine the potential impacts on the engineered cap due to a tsunami event. The direct impacts of a tsunami event on an engineered cap are challenging to quantify due to several factors:

- Tsunamis are unpredictable extreme events.
- There are currently no design standards to address potential tsunami events.
- The greatest impact due to tsunami events is often associated with debris carried by the tsunami (particularly during the withdrawal stage, where the tsunami wave pulls out debris towards the water). Predicting what type of debris may be entrained in the water and where exactly it will impact the Site cannot be defined with certainty and, therefore, cannot be quantified at the level sufficient to develop a basis for design standards.

Based on these tsunami-event impact factors, the information contained within this section is not intended to define design criteria for the sediment caps. Impacts of extreme events such as tsunamis will be addressed through monitoring and contingent actions as described in the Compliance Monitoring and Contingency Response Plan (Appendix G).

The current evaluation focuses on the potential impacts from a tsunami event due to currents and water levels caused by the passage of the tsunami based on modeling completed by NOAA for Bellingham Bay (Walsh et al. 2004).

3.5.1 National Oceanic and Atmospheric Administration Modeling

NOAA has conducted tsunami inundation modeling for the Bellingham area based on a Cascadia Subduction Zone Earthquake[2] (Priest et al. 1997). The model used to determine inundation and in-water velocities is called Method of Splitting Tsunamis (MOST) (Titov and Gonzalez 1997). The model uses non-linear shallow water equations to simulate the propagation and inundation of the long period waves typically produced by tsunamis. It has been extensively tested and validated for both far field and near field applications and is very accurate in predicting arrival times and initial wave heights. As part of the model, wave amplitudes and depth-averaged velocities are calculated in two horizontal directions. Although the accurate calculation of velocities in tsunamis is still an active area of research, MOST represents the current state of technology for practical applications.

Figure C-1, taken from Walsh et al. 2004, shows predicted inundation areas and associated water depths in the Bellingham area (including the Waterway) predicted by the MOST model. The upland areas surrounding the Waterway are predicted to be inundated from 0 to 1.5 feet (0.5 meters) during the modeling tsunami event, which is conservatively assumed to occur at high tide (approximately +9 feet MLLW)[3].

The time series of tsunami waves in the Bellingham Bay at simulated tide gage location No. 5 shown on Figure C-1 indicates wave heights ranging from 9.8 feet (3 meters) to -9.8 feet (-3 meters) relative to a MHHW. The approximate wave length between the crest and trough of the wave is 1.5 hours. Figure C-2 shows current velocities predicted by the MOST model for the same area. The Waterway Site is predicted to experience between 0 and 5 feet per second (1.5 meters per second) depth-averaged current velocities (maximum predicted).

[2] The earthquake modeled represents a 9.1 event (based on the Richter scale). Additional parameters of the modeling event can be found in Priest et al. 1997.

[3] Tsunami modeling conducted at high tide may not be conservative for predicted current velocities.

There are limitations of the modeling that are important to keep in mind when applying the results to the evaluation of impacts to the engineered cap at the Waterway Site:

- The model is based on a tsunami that is generated in the Pacific Ocean and propagates into the Bellingham Bay area. Local seismic activity or landslides could result in a different tsunami event.
- The resolution of the topography data used to represent upland areas around the Site was based on 5-foot contour data. The resolution of the topography data (5 feet) is larger than the predicted inundation depths in the Project area (less than 2 feet). Therefore, the inundation depths at the Waterway are accurate to about 5 feet based solely on the resolution of the topographic data set.
- The model does not include influence of tidal oscillations, which could increase the wave height or decrease the wave trough (draw-down) elevation over the course of the tsunami event.
- The model was run at a static tidal elevation relative to MHHW (approximately +9 feet MLLW). Depth-averaged current velocities within the Bellingham Area due to a tsunami that occurs at lower tidal elevations may be higher than estimated by the current model (Walsh et al. 2004).

3.5.2 Comparison to 100-year Events and Propwash

Predicted water surface elevations due to 100-year storm event, depth-averaged current velocities due to tidal circulation and 100-year creek flow, and near-bed velocities due to propwash were compared to water surface elevations and current velocities predicted due to a tsunami event. These comparisons are discussed in the following sections.

3.5.2.1 Water Surface Elevations

Water surface elevations in the Waterway are dominated by tidal action and onshore winds (storm surge). Extreme tide elevation is approximately +12 feet MLLW in Bellingham Bay (based on tidal predictions in Bellingham Bay). Storm surge based on 100-year onshore wind speeds could add up to another 1 foot of still water elevation (Ippen 1966) along the shoreline, for a total storm tide of approximately +13 feet MLLW.

Predicted inundation depths due to a tsunami event (based on NOAA modeling) could be as high as 1.5 feet above existing ground elevation (based on still water elevation at MHHW; +9 feet MLLW). The ground elevation within the Waterway varies from +13 to +16 feet MLLW. Therefore, water surface elevations due to tsunami could be as high as +17.5 feet MLLW (using +16-foot MLLW ground elevation). Since the topography used in the NOAA model was based on 5-foot contours; the predicted wave elevations in Bellingham Bay (at Location 5 shown in Figure C-1) were compared directly to topography within the Waterway Site. Waves as high as approximately 10 feet above MHHW were predicted at this location. Given that MHHW is +9 feet MLLW in this location, the water surface elevation would be approximately +19 feet MLLW at the Waterway Site. This is a slightly higher water surface elevation than calculated using inundation depth (Figure C-1). This discrepancy is likely due to lack of resolution in the topography data used to determine inundation depths by Walsh et al. (2004).

Based on these predictions, the water surface elevation due to tsunami inundation could be as much as 6 feet higher than that caused by a typical 100-year wind wave event (occurring at extreme high tide).

3.5.2.2 Current Velocities

Depth-averaged current velocities due to tidal currents are not directly comparable to those from tsunami bores due to differences in the vertical distribution of flow. Tsunami bores, in contrast to tidal flows, generate large amounts of vorticity and turbulence and are very effective at distributing energy throughout the water column (Li et al. 2010). Therefore, the vertical distribution of current in a tsunami bore is more uniform from top to bottom, whereas the typical vertical distribution of current for open channel flows is logarithmic tending to zero at the bed. Therefore, the depth-averaged velocity for tidal flows will be higher in magnitude than the near-bed velocity and the depth-averaged velocity for tsunami bores will be similar in magnitude to the near-bed velocity. Direct comparison of the depth-averaged quantities will result in a conservative evaluation (since the depth-averaged velocity is an over-prediction of actual near-bed velocities).

Maximum depth-averaged current velocities within the Waterway due to spring tide conditions and a greater than 100-year flow from Whatcom Creek are generally less than 3.5 feet per second (see Attachment 1) for most of the Waterway. At the head of the Waterway, the flow out of Whatcom Creek (and tidal flows due to flood tide) is constrained by the Roeder Street bridge piers and shallower water depths. Depth-averaged current velocities in this area can reach 19 feet per second directly under the bridge, with velocities in the range of 10 feet per second just downstream of the bridge location (see Attachment 1). Near-bed current velocities due to propwash (see Appendix D) can range from approximately 3 to 12 feet per second at various locations throughout the Waterway. Depth-averaged current velocities due to tsunami (NOAA modeling at high tide) can range from 0 to 5 feet per second. It is possible that depth-averaged velocities in Whatcom Waterway could be higher than predicted by current modeling if the tsunami were to occur at lower tidal elevations. However, velocities due to 100-year design criteria and propwash in the Waterway are more than 200 percent higher than the maximum predicted velocities based on existing modeling of a tsunami at high tide (Walsh et al. 2004).

Current velocities due to tides and creek flows are predicted to be more than twice as high as those predicted by a tsunami event at the head of the Waterway and at and just downstream of the Roeder Bridge (defined as a natural recovery area). For the remainder of the Waterway, predicted propwash velocities (near-bed) are approximately equal to or much higher than current velocities anticipated due to a tsunami event.

3.5.3 Discussion

A tsunami event similar to the one modeled by NOAA (Walsh et al. 2004) has the capacity to increase water levels at the Waterway Site by up to 6 feet higher than a typical 100-year wind-wave event. This would have direct impacts to upland areas and would potentially have some impact to shoreline armor rock at the top of the slope or at the landside connection of the armored slope.

Near-bed currents induced by passage of the tsunami waves would have the most impact to the engineered cap (excluding direct impact by debris). Current velocities predicted by the tsunami event are less than velocities due to tidal and current flows (at the head of the

Waterway just south of the bridges) and by propwash velocities (in all areas of the Waterway except the Log Pond). The engineered cap has not been designed specifically for a tsunami event. However, it has been designed to remain stable under other flow regimes (tidal and creek flows and propwash), which have velocity magnitudes higher than those predicted by a tsunami.

Potential impacts to the engineered cap due to a tsunami not considered as part of this evaluation include:

- Tsunami caused by local seismic activity or landslides
- Damage to the engineered cap due to ship grounding (due to draw-down during a tsunami) or direct impact by floating or sunken debris
- Damage to shoreline armor due to breaking tsunami waves or moving bore fronts
- Redistribution of sediments due to liquefaction

4 RESULTS AND DISCUSSION

Results of the wave and current modeling described above and associated armor requirements for deep water and nearshore areas within the Waterway are described in the following sections. Due to varying depths and shoreline slopes, the different areas of the Waterway were evaluated separately based on their specific criteria. Additional armor requirements based on propwash are discussed in detail in Appendix D.

4.1 Outer Waterway

The tidal bottom velocities in this area are near zero and the bottom velocity from a 100-year wave (height of 5.3 feet and period of 5 seconds) is 0.6 feet per second, which, when compared to the propwash velocities (see Appendix D), is low. Therefore, the bottom armor stone sizing for the Outer Waterway was dominated by vessel propeller scour potential. See Appendix D for armor sizing requirements in this area of the Waterway.

4.2 Inner Waterway

The tidal bottom velocities in this area are near zero and the bottom velocity from a 100-year wave (height of 2.4 feet and period of 4 seconds) is 0.6 feet per second, which, when compared to the propwash velocities (see Appendix D), is low. Therefore, the bottom armor stone sizing for the Inner Waterway was dominated by vessel propeller scour potential. See Appendix D for armor sizing requirements in this area of the Waterway.

The area near the mouth of Whatcom Creek in the Inner Waterway (area south and downstream of the Roeder Street bridge) experiences higher bottom velocities due to the extreme flow from the creek used in the model (2,400 cfs). Based on the hydrodynamic model results, a bottom velocity of 3.3 feet per second is found in this area. Based on this velocity, the required D_{50} for the stable bottom material in this small area would need to be 1 inch. This area of high velocity is within the natural recovery area and outside the footprint of active remediation for Phase 1 work.

4.3 South Shoreline

The South Shoreline consists of sloping shorelines and bulkheads. On the South Shoreline's sloped areas, the critical armor design for the upper portions of the slope was based on breaking waves, while the critical armor design for the lower portions of the slope was based on propwash analysis results (see Appendix D). For shoreline areas within the breaking wave zone, the armor size was based on a wave of 2.3 feet and a period of 4 seconds (CHE 2012). The breaking waves on the shoreline require an armor stone size D_{50} of 7.5-inches[4]; however, a D_{50} of 9 inches has been specified to fit closer to a standard specification and to provide for a factor of safety of approximately 1.2. The 9-inch stone requires a minimum filter stone size D_{50} of 1.1 inches; however, a poorly graded filter material centered around a D_{50} of 3 inches will be used to eliminate the need for a second filter layer. The bottom bed (Inner Waterway) requires an armor size D_{50} of 3-inch material based on propwash analysis (see Appendix D).

The upper elevation of the sloped shorelines on which the armor material will be placed varies based on the elevation of the top of bank. The low elevation for the armor extends to a minimum elevation of -8 feet MLLW. The lower extent is based on a 2.3-foot storm wave and an extreme low-tide prediction[5] of -4.3 feet MLLW.

The armor and filter layer will be placed on top of a sand layer covering the Inner Waterway and the shoreline areas. The isolation cap modeling (see Appendix E) specifies a minimum sand layer thickness of 2 feet required for the Waterway to address contaminant mobility, and the design includes consideration of an over-placement allowance of 0.5 feet. The thickness of the filter layer will be 1 foot, with an over-placement allowance of 0.5 feet. The thickness of the armor layer will be 1.5 feet, with an over-placement allowance of 1.0 feet. This results in a total engineered cap thickness of 4.5 to 6.5 feet of sand, filter, and armor placed on the slope above -8 feet MLLW, and a thickness of 3 to 4 feet of sand and filter placed on the slope below -8 feet MLLW.

[4] Armor stone size is based on an assumption of "no-damage," which results in a conservatively large armor rock size for a particular wave condition (USACE 2002).
[5] Tide predictions for Bellingham Bay are based on the gage at Cherry Point, Washington (Station No. 9449424).

Table C-3 summarizes the armor results for the South Shoreline.

Table C-3
South Shoreline Armor Results, Summary

Criterion	Result
Design Slope	3H:1V1
Design Wave Height	2.3 feet
Design Wave Period	4 seconds
Calculated Slope Armor (D_{50})	0.6 feet (7.5 inches)
Calculated Slope Filter (D_{50})	0.07 feet (1 inch)
Specified Slope Armor (D_{50})	0.75 feet (9 inches)
Specified Slope Filter (D_{50})	0.25 feet (3 inches)1
Upper Extent of Armor	Top of bank (varies)
Upper Extent of Filter	Top of bank (varies)
Lower Extent of Armor	-8 feet MLLW
Lower Extent of Filter[2]	To bottom of slope

Notes:
1. A 9-inch armor stone actually requires a filter with a minimum D_{50} of 1.1 inches; however, a D_{50} of 3 inches (well-graded material) has been specified to fulfill the bottom armor requirements (see Appendix D).
2. The filter layer acts as armor for the sand cap below -8 feet MLLW.

H:V = horizontal to vertical

4.4 Central Waterfront Shoreline

The Central Waterfront Shoreline is mostly the same as the South Shoreline (see Section 4.2.3 and Table C-3). The main differences are that the final Central Waterfront Shoreline slope varies from 2 horizontal to 1 vertical (H:V) to 3H:1V and the shoreline has a flat shelf that is at an elevation of -8 feet MLLW and is 35 feet wide in front of the Maple Street concrete bulkhead. In this area, the armor layer will cover the entire shelf, with the top of the armor layer at -8 feet MLLW.

The lower extent of the armor stone for the majority of the Central Waterfront is the same as the South Shoreline at -8 feet MLLW. However, the extent is lower in front of the Maple Street concrete bulkhead, extending down to an elevation of -20 feet MLLW (see Figures 14b and 15c, Section F in the main document) due to propwash concerns in this area (see Appendix D).

4.5 Log Pond

The Log Pond has no vessel traffic and the tidal currents are minimal; therefore, the armor requirements are based solely on waves. A design wave with a height of 2.7 feet and a period of 4.2 seconds was used to determine the required the Log Pond armor size. The armor material requires a D_{50} of 9 inches for the 2H:1V slope along the upland connection to the slope. Milder slopes waterward of this area (3H:1V and milder) require a D_{50} of 7 inches to prevent significant movement by breaking waves. The Log Pond cap for the 2H:1V slope area was designed using a damage level slightly higher than for the Inner Waterway shorelines (S=4; see equation 3, van der Meer 1987) to allow for some minimal movement of single armor rocks during the design storm event, but not significant damage to the structure as a whole (Table 4-4-1; USACE 1992). This was done to balance stabilization with habitat concerns along the Log Pond shoreline. For simplified construction and material specifications, the armor and filter sizes for the Log Pond will be the same as the South Shoreline: armor with a D_{50} of 9 inches and a filter material with a D_{50} of 3 inches. This results in a safety factor of 1.2 for slopes greater than 3H:1V, but only a safety factor of 1.0 (based on minimal movement of single armor rocks[6]) for slopes equation to 2H:1V.

The horizontal placement extents for the proposed armor and filter material for the Log Pond was based on the thickness remaining from the previously placed sand cap (Anchor Environmental 2001) and was not based on the methods described in Section 2.1. The upper elevation of the armor is based on the elevation at the top of the bank, which is approximately 16 feet MLLW. The armor slope was extended from the top of the bank at a 2H:1V slope down to existing grade, then extended out horizontally into the Log Pond until the existing sand cap thickness exceeded approximately 2 feet; this approach was used to ensure protection of the areas where the existing sand cap had been significantly eroded.

[6] It is likely that, during the 100-year return period event, a small number of armor rocks will be displaced from their positions but the armored slope itself will not fail. The damage level associated with structural failure is 10 (van der Meer 1987).

The Log Pond will also include additional sand cap material placement in the southern corner to re-establish the thickness of the sand cap in this portion of the Log Pond that had experience significant loss of the sand cap material (see Section 9.2 of the main report and Appendix E). This additional sand cap placement will have a minimum thickness of 2 feet and will be covered by the filter and armor layers to prevent future erosion in this corner of the Log Pond. The thickness of the armor layer will be 1.5 feet, with an over-placement allowance of 0.5 feet. The thickness of the filter layer will be 1 foot, with an over-placement allowance of 0.5 feet. This results in a total thickness of 2.5 to 3.5 feet for the armor and filter layers throughout the Log Pond (see Figure 12).

Table C-4 summarizes the armor results for the Log Pond.

Table C-4
Log Pond Results, Summary

Criterion	Result
Design Slope	Varies 2H:1V to 5H:1V
Design Wave Height	2.7 feet
Design Wave Period	4.2 seconds
Calculated Slope Armor (D_{50})	Varies 0.58 feet (7 inches) to 0.75 feet (9 inches)
Calculated Slope Filter (D_{50})	Varies 0.07 feet (0.8 inch) to 0.09 feet (1.0 inch)
Specified Slope Armor (D_{50})	0.75 feet (9 inches)
Specified Slope Filter (D_{50})	0.25 feet (3 inches)[1]
Upper Extent of Armor/Filter	Top of bank (varies)
Lower Extent of Armor/Filter	Based on previous cap (varies)

Notes:
1. A 9-inch armor stone actually requires a filter with a minimum D_{50} of 1.1 inches; however, a D_{50} of 3 inches (well-graded material) has been specified to fulfill the bottom armor requirements (Appendix D).

H:V = horizontal to vertical

4.6 Uncertainty Discussion

The analysis of wind-driven waves uses calculations and models developed by coastal engineering experts over the past century. The evaluations and modeling described in this appendix use conservative methodologies intended to result in robust design standards for the current Project. Potential uncertainties, such as those associated with weather data and model assumptions, have been addressed where possible through the use of conservative assumptions as described below.

One source of uncertainty is the weather data on which wind forecasts are based. Prediction of extreme winds is based on a statistical evaluation of long-term wind datasets derived from nearby locations. This is the most valid basis on which to base estimates of future wind speeds and return periods. However, reliable local weather observations are available for a shorter duration than the conservative design return period (100 years). Therefore, estimated return periods for extreme winds may change over time as additional weather data are collected. These estimates may also change due to changes in weather patterns resulting from such factors as potential climate change. However, the magnitude of sustained extreme winds is not linearly related to the estimated return period (e.g., the magnitude of a 100-year storm is only slightly greater than that of a 50-year storm). Conservatism was included in the modeling evaluation by using a high return period (100 years) for design, along with other conservative assumptions as described below.

Second, a local long-term wave monitoring dataset was not available for Bellingham Bay. This did not affect the evaluation, as standard methods are available for estimating wave conditions using wind data and hindcasting methods. Further, the wave and current models were calibrated using site-specific data (wave and current measurements) to validate model predictions. There was good agreement between predicted and observed wave conditions, indicating that the numerical modeling methods are appropriate for use on the project.

Third, wind waves in some areas of the project may be affected by the presence of obstructions (e.g., ships or barges) or structures (e.g., bulkheads). These effects can result in localized reductions to wave forces, or can magnify wave forces through wave reflection. Modeling scenarios considered these potential effects. Evaluations utilized scenarios intended to estimate the maximum potential wave erosional force.

To account for uncertainty in the evaluation, conservative assumptions have been made toward the design of the armor size and armor cap geometry:

- Design water levels were set at lowest low water and highest high water, which results in conservative estimates of breaking wave heights for armor design and for vertical extents (above and below the design water level) of armor required.
- The damage level for cap stability was set to “no damage,” which produces a conservatively high armor size and layer thickness for the cap armor.
- The analysis of tidal currents and creek flows from Whatcom Creek was conducted conservatively, combining low-water conditions with peak estimated creek discharge rates. This may over-estimate current forces for this area of the Site.

The evaluation of potential tsunami impacts is an evolving area of research. The analysis described in Section 3.5 was conducted using best available science. Uncertainties associated with that evaluation are discussed in Section 3.5.

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FIGURES

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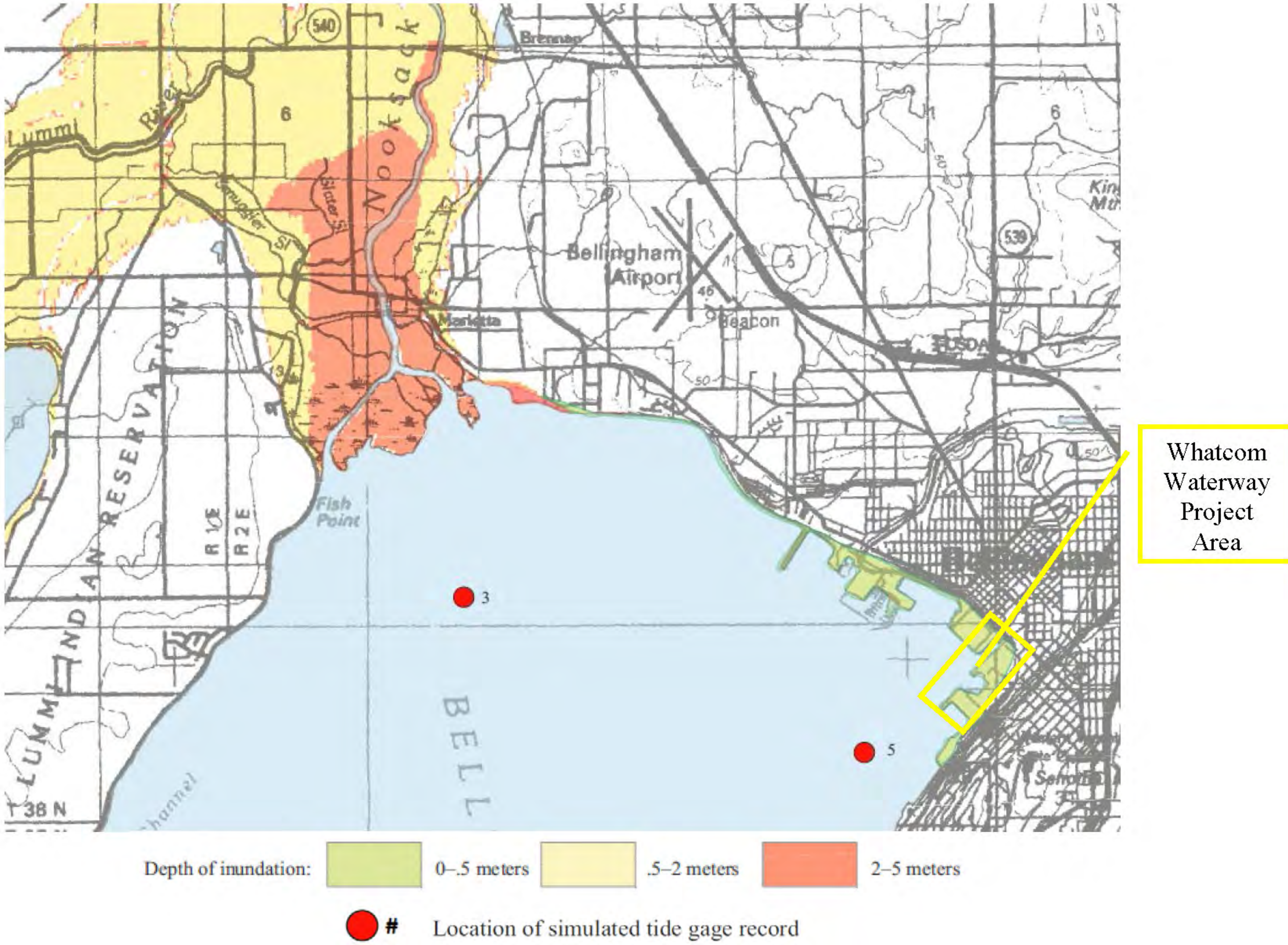

Figure C-1
Tsunami Inundation Areas and Depths as Predicted by Walsh et al. 2004 (Figure adapted from Walsh et al. 2004)
Appendix C – Coastal Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR

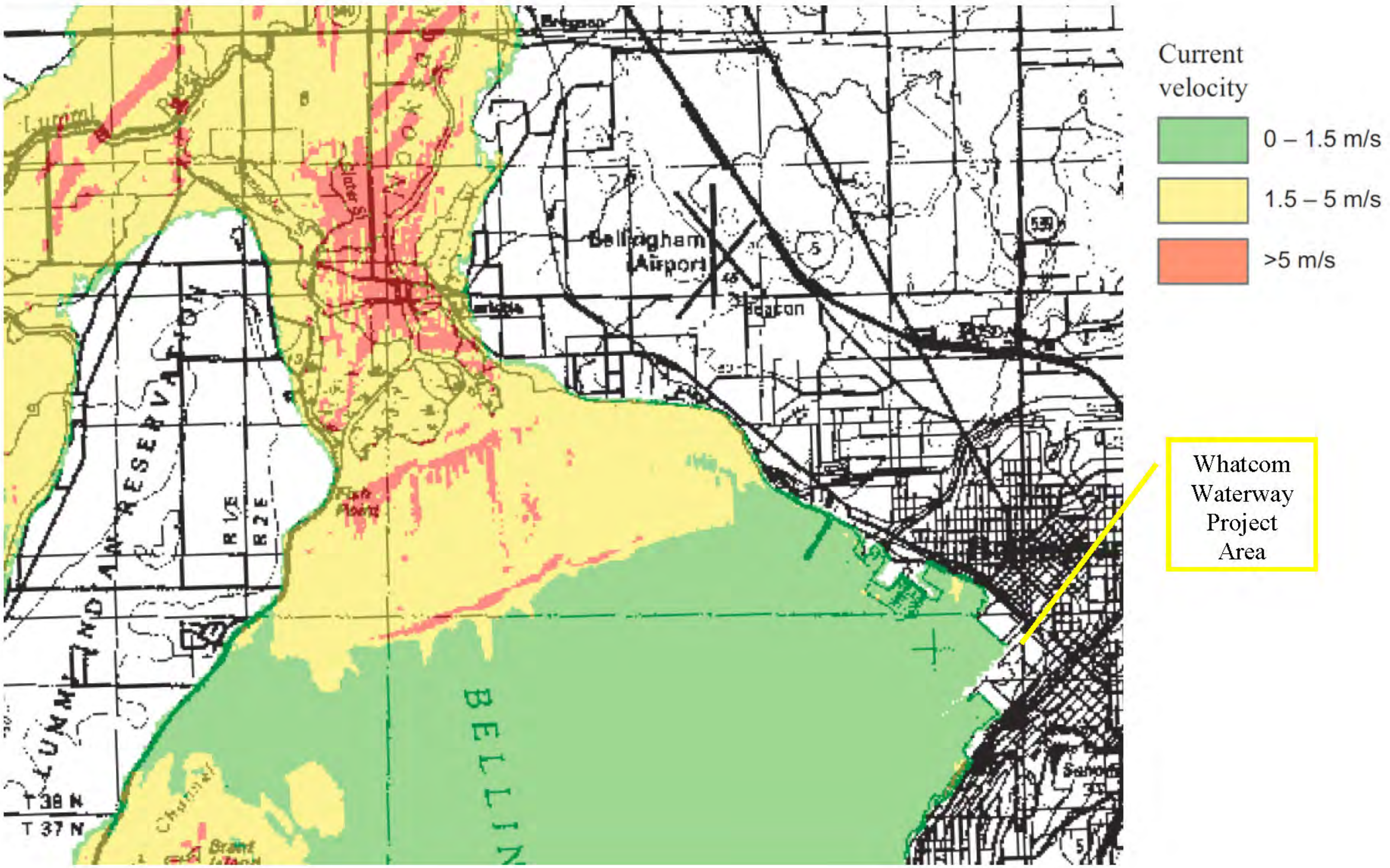

Figure C-2
Tsunami Inundation Areas and Depths as Predicted by Walsh et al. 2004 (Figure adapted from Walsh et al. 2004)
Appendix C – Coastal Evaluation
Whatcom Waterway Cleanup in Phase 1 Site Areas – Final EDR

ATTACHMENT 1

WHATCOM WATERWAY PHASE 1 CLEANUP PROJECT
Port of Bellingham 60% Design Hydrodynamic Modeling and Coastal Engineering Analysis

COAST & HARBOR
ENGINEERING

Technical Report

WHATCOM WATERWAY PHASE 1 CLEANUP PROJECT
Port of Bellingham 60% Design Hydrodynamic Modeling and Coastal Engineering Analysis

Prepared for:

AnchorQEA

This document is to assist with 60% design of the Port of Bellingham's Whatcom Waterway Cleanup Project. This document presents the results of hydrodynamic modeling and coastal engineering analysis conducted by CHE. This document is not to be used for purposes of final engineering design, or for construction documents.

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TABLE OF CONTENTS

FIGURES

TABLES

Technical Report
Whatcom Waterway Phase 1 Cleanup Project, Port of Bellingham 60% Design Hydrodynamic Modeling and Coastal Engineering Analysis

1. Introduction

This brief technical report summarizes the results of hydrodynamic modeling and coastal engineering analysis conducted by Coast & Harbor Engineering (CHE) to assist AnchorQEA with 60% design of the Whatcom Waterway Phase 1 Cleanup Project. The scope of CHE's hydrodynamic numerical modeling included two-dimensional (2-D) wave refraction, diffraction, and reflection; three-dimensional (3-D) tidal circulation and Whatcom Creek flows, propwash velocity and stability of sediment subjected to propwash, and tabulation of size of sediment at the bottom of the waterway and on side slopes at the edge of the waterway in designated areas, and at the bottom and shoreline in areas within the log pond.

Prior to conducting modeling and analysis, CHE prepared and coordinated the Technical Memorandum *Whatcom Waterway Phase 1 Cleanup Hydrodynamic Modeling Cases* (CHE 2011) with AnchorQEA. This technical memorandum described the physical conditions, input parameters, and modeling scenarios that were used for hydrodynamic modeling herein. The current brief report does not repeat these characteristics, input parameters and modeling scenarios; but rather, directs interested readers to the October 2011 CHE technical memorandum.

2. Wave Modeling

2.1. General

Wave generation, propagation to the project site, and interaction with bottom sediment and waterfront structures was conducted using 2-D numerical modeling software, SWAN and HWAVE. Waves were simulated (numerically modeled) with two steps, using a large area modeling domain (Step 1), and high resolution, nested numerical modeling domain (Step 2). Wave spectral information developed in SWAN is passed to HWAVE at open water boundaries of the nested grid. Bathymetry of the large and nested modeling domains is shown in Figure 1 in color format.

For Phase 1 project wave modeling was conducted for a 100-year return period wind speed approaching from 240 degrees True North (TN) and from 270 degrees TN. Waves were modeled with tide levels at MHHW and lower. Wave modeling results at high tide level are presented below in Section 2.1 as plan views of significant wave heights over the modeling domains and wave heights and periods extracted at the requested locations. Wave parameters at requested locations were extracted for each modeling scenario by locating numerical wave gauges at appropriate nodes of the modeling grid, shown in Figure 2, which output wave height, period, and direction.

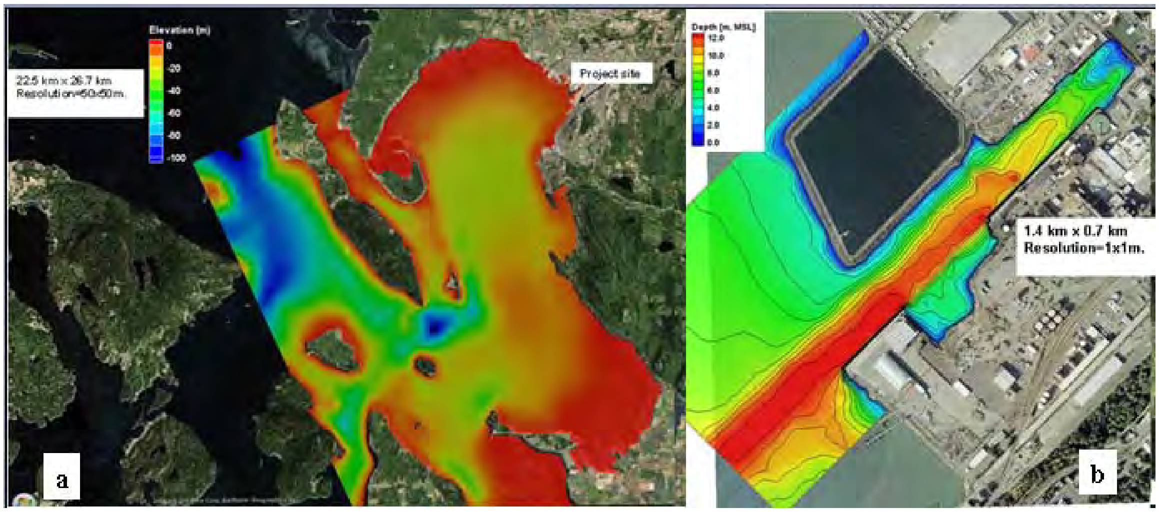


Figure 1. Large (a) and nested (b) wave numerical modeling domains

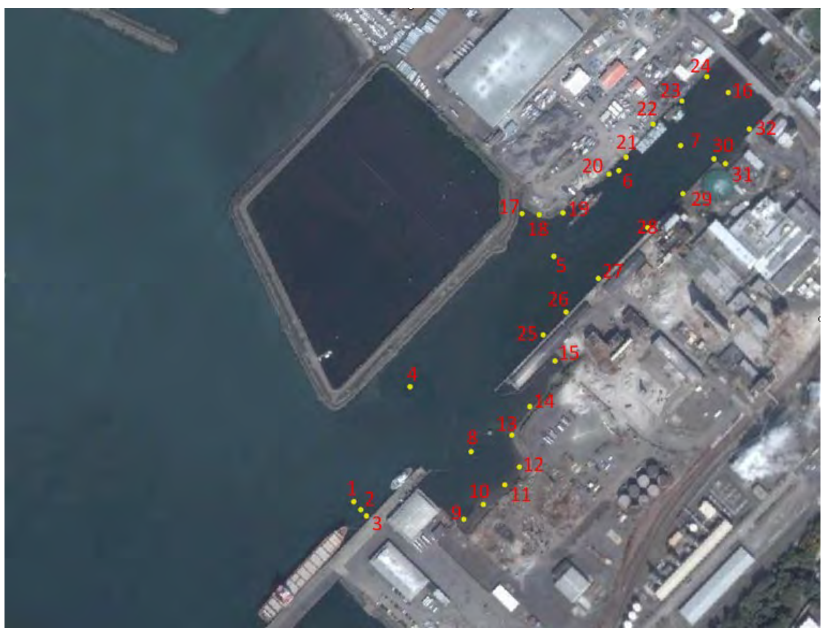


Figure 2. Wave model information extraction points

2.2. Wave Model Output

Results of HWAVE modeling with the high resolution grid to simulate significant wave heights during the 100-year return period storms are shown in Figures 3 and 4. Wave height is shown in color format. Red color indicates higher waves and blue color indicates smaller or no waves. Figure 3 shows the wave height pattern over the modeling domain during the design storm approaching from 240 degrees. Similarly,

Figure 4 shows wave heights for the design storm approaching from 270 degrees. Color patterns represent the distribution of wave heights, not wave fronts. Vectors in the figures show wave direction at grid points.

Modeled wave heights, wave periods, and corresponding water depths at the requested locations (see Figure 2 above) on the modeling grid are presented in Table 1 for wave direction 240 degrees, and in Table 2 for 270 degrees.

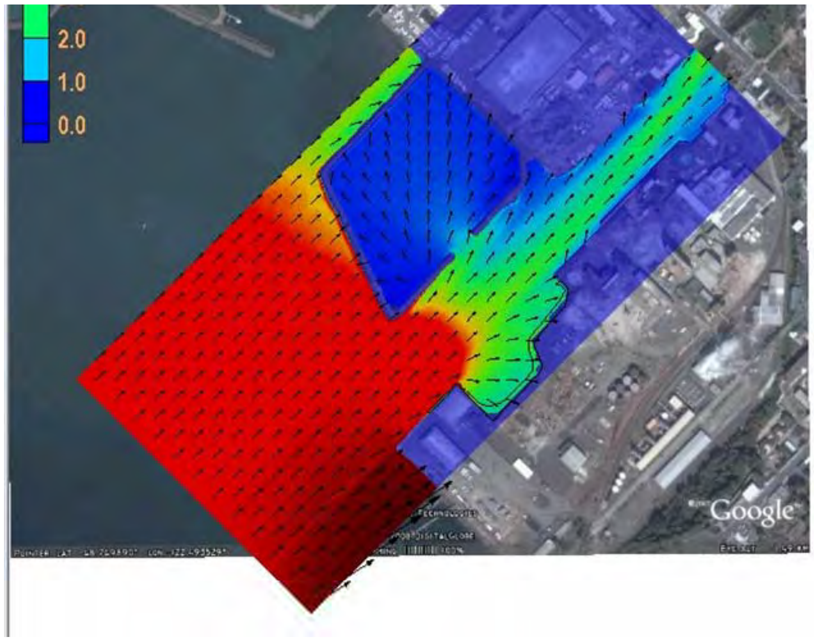


Figure 3. Wave height and direction in Whatcom Waterway, wave storm approaching from 240 degrees

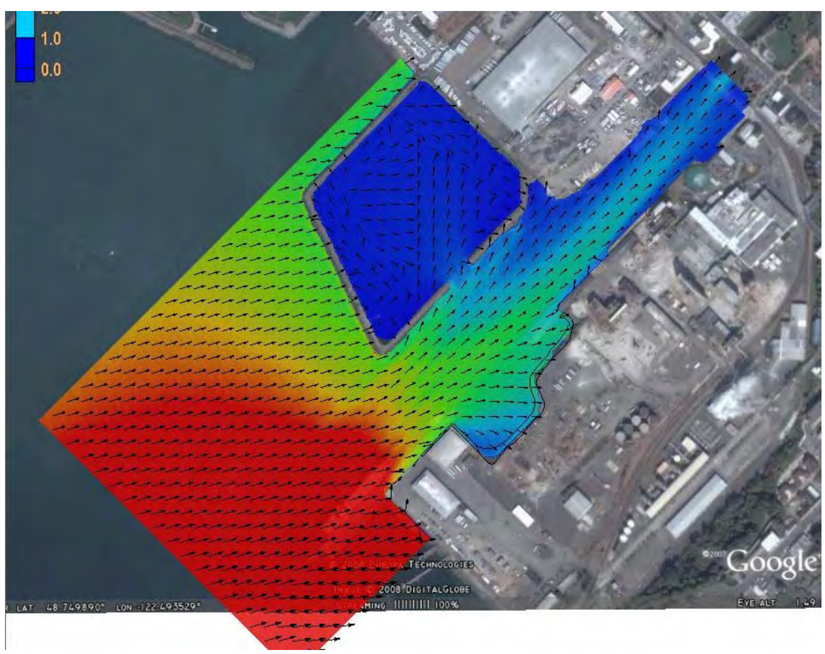


Figure 4. Wave height and direction in Whatcom Waterway, wave storm approaching from 270 degrees

Table 1. Wave height and period and water depth, waves from 240 degrees

	100-Yr Return Period, direction 240 deg				
	Significant Wave Height, Period, Depth at Extractions Points				
	Point Coordinates				
	UTM - meters				
Point	**X**	**Y**	**Depth (ft)**	**Hs (ft)**	**Tp (s)**
1	537228.0	5399365.1	50.8	5.34	4.47
2	537239.3	5399352.2	50.8	5.18	4.47
3	537248.5	5399342.1	40.0	4.92	4.47
4	537319.2	5399551.0	36.8	3.79	4.11
5	537552.4	5399761.9	36.8	1.28	4.47
6	537657.9	5399900.7	19.2	0.61	4.47
7	537758.1	5399941.2	31.8	2.30	3.86
8	537418.2	5399446.1	20.4	2.78	4.22
9	537406.4	5399336.6	5.3	1.64	5.62
10	537438.1	5399360.6	7.6	1.65	4.22
11	537472.8	5399392.3	7.1	1.63	4.22
12	537496.5	5399421.3	5.9	1.78	4.22
13	537484.3	5399472.8	8.3	2.45	4.47
14	537513.4	5399519.0	7.9	1.84	4.47
15	537554.3	5399592.8	12.1	1.49	4.47
16	537835.3	5400027.0	13.7	2.04	4.22
17	537500.8	5399831.0	14.9	0.27	3.98
18	537528.6	5399829.3	11.6	0.44	3.94
19	537567.2	5399832.2	14.3	0.57	3.44
20	537641.9	5399894.9	10.5	0.59	3.76
21	537669.7	5399922.3	11.5	0.56	3.69
22	537713.3	5399975.9	3.7	0.62	3.21
23	537760.2	5400013.2	18.0	0.61	2.98
24	537800.5	5400052.6	16.9	1.07	1.20
25	537535.1	5399634.9	30.1	1.64	4.47
26	537572.2	5399671.9	29.0	1.39	4.21
27	537624.6	5399726.3	28.7	1.16	4.10
28	537704.7	5399808.3	13.5	0.72	3.76
29	537761.7	5399863.4	-0.1	0.65	3.65
30	537812.1	5399919.6	7.2	0.95	4.47
31	537830.8	5399911.9	2.8	0.66	4.73
32	537869.4	5399967.8	15.6	0.63	4.47

Table 2. Wave height and period and water depth, waves from 270 degrees

	100-Yr Return Period, direction 270 deg				
	Significant Wave Height, Period, Depth at Extractions Points				
	Point Coordinates				
	UTM - meters				
Point	**X**	**Y**	**Depth (ft)**	**Hs (ft)**	**Tp (s)**
1	537228.0	5399365.1	50.8	4.30	4.22
2	537239.3	5399352.2	50.8	4.10	4.22
3	537248.5	5399342.1	40.0	3.96	4.22
4	537319.2	5399551.0	36.8	1.81	3.76
5	537552.4	5399761.9	36.8	0.69	3.98
6	537657.9	5399900.7	19.2	0.26	3.76
7	537758.1	5399941.2	31.8	0.90	3.76
8	537418.2	5399446.1	20.4	1.79	4.22
9	537406.4	5399336.6	5.3	0.54	5.01
10	537438.1	5399360.6	7.6	0.75	4.22
11	537472.8	5399392.3	7.1	0.94	4.22
12	537496.5	5399421.3	5.9	1.32	4.22
13	537484.3	5399472.8	8.3	1.61	4.22
14	537513.4	5399519.0	7.9	1.42	4.22
15	537554.3	5399592.8	12.1	1.33	3.76
16	537835.3	5400027.0	13.7	0.82	3.76
17	537500.8	5399831.0	14.9	0.20	3.98
18	537528.6	5399829.3	11.6	0.28	3.35
19	537567.2	5399832.2	14.3	0.25	3.21
20	537641.9	5399894.9	10.5	0.22	3.84
21	537669.7	5399922.3	11.5	0.20	3.67
22	537713.3	5399975.9	3.7	0.27	3.23
23	537760.2	5400013.2	18.0	0.31	3.01
24	537800.5	5400052.6	16.9	0.26	2.78
25	537535.1	5399634.9	30.1	1.36	3.99
26	537572.2	5399671.9	29.0	1.10	3.90
27	537624.6	5399726.3	28.7	0.95	3.87
28	537704.7	5399808.3	13.5	0.39	3.76
29	537761.7	5399863.4	-0.1	0.31	3.60
30	537812.1	5399919.6	7.2	0.38	4.47
31	537830.8	5399911.9	2.8	0.17	2.40
32	537869.4	5399967.8	15.6	0.26	4.47

3. Bottom Material Stability in Log Pond

Modeling and analysis were conducted to determine stability of bottom and shoreline sediment subjected to the design wave storm at the Log Pond area. The area known as the Log Pond is an embayment on the south side of the Waterway and east of the Bellingham Shipping Terminal (BST). Sediment stability analysis was determined according to the ability of sediment to resist hydrodynamic forces generated in the design storm by wave orbital velocities for the non-breaking wave conditions, and swash velocities in the wave breaking area and surf zone. For this purpose, the bottom orbital velocities and wave swash velocities were determined from wave modeling and compared to threshold velocities of sediment motion. Graphical results of material stability analysis at a water level of high tide and wave direction from 240 degrees (most severe conditions) for the Log Pond are shown in Figure 5. The figure presents the pattern of minimum stable size in color format: red color represents a larger size of sediment and blue color represents a smaller size. The figure shows that along the shoreline at the wave breaking area stability criteria require material size to be in the range of riprap. This result is reasonable and is supported by local data.

Figure 5. Stable material size according to location, wave direction from 240 degrees

4. Flow Circulation Modeling

4.1. General

Flow velocities in Whatcom Waterway were simulated for the condition of a combined tidal series of neap to spring tides and storm discharge from Whatcom Creek. The model simulates unsteady flow in three dimensions. Storm discharge consists of the 100-year return period discharge from both the controlled and uncontrolled areas of the drainage. The peak discharge modeled at the mouth of Whatcom Creek was 2400 c.f.s.

Modeling was conducted with the 3-D flow circulation model SELFE. The model was validated and calibrated previously based on field data collected in Bellingham Bay; and specifically, in Whatcom Waterway. Modeling was conducted on several modeling grids, including a large modeling grid and two nested grids, shown in Figure 6. The minimum size of the element in the modeling grid was in the range of 4-6 ft.

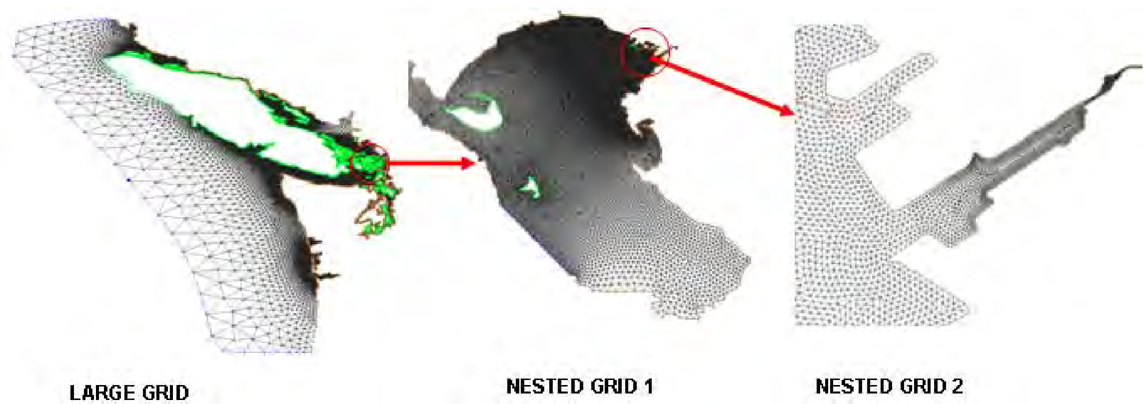


Figure 6. SELFE numerical modeling grids for Whatcom Waterway hydrodynamic modeling

4.2. Modeling Results

Modeling results were extracted from the modeling output of depth-averaged velocity and velocity at the bottom boundary in the modeling domains. Specifically, upon the request from Anchor QEA, the modeling results were extracted in areas 3A-MNR and 3B (see Figure 7 for area locations).

Modeled depth-averaged velocity distribution over the modeling domain of Nested Grid 2 is shown in Figure 8 in color format. Modeled bottom velocity distribution over the modeling domain of Nested Grid 2 is shown in Figure 9. Maximum velocities for the two cases were extracted from the results and are 19 ft/sec (5.8 m/s) and 17.06 (5.2 m/s), respectively. As shown in the figures, the peak velocity occurs at the channel bottom between the two bridges.

Figure 7. Bottom effects study areas within Whatcom Waterway

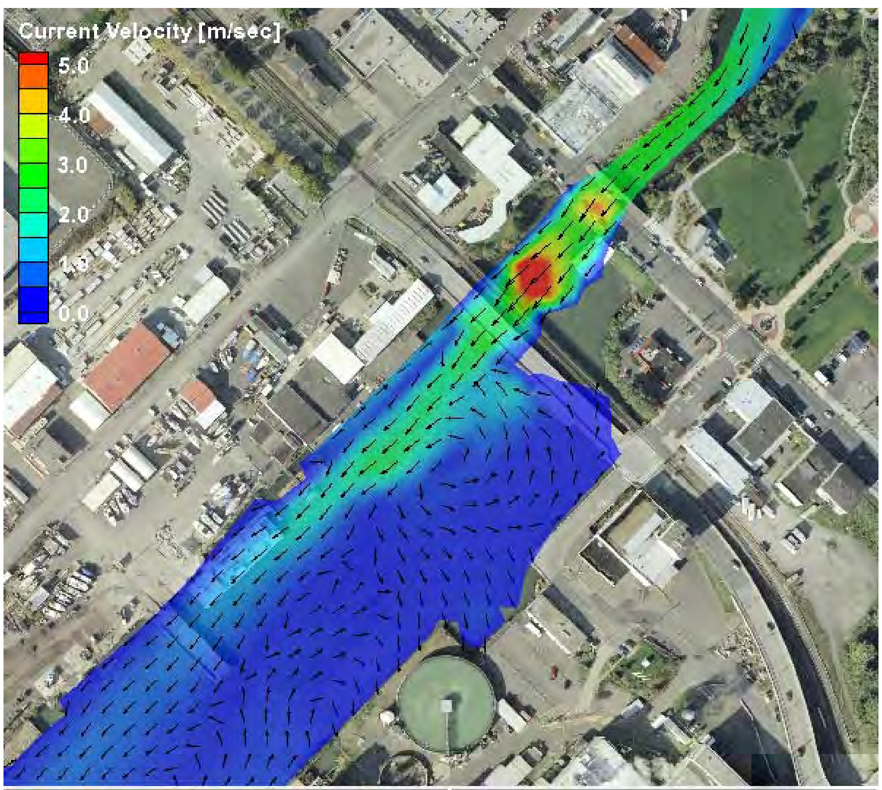


Figure 8. Velocity pattern at time of peak flow, depth-averaged velocity

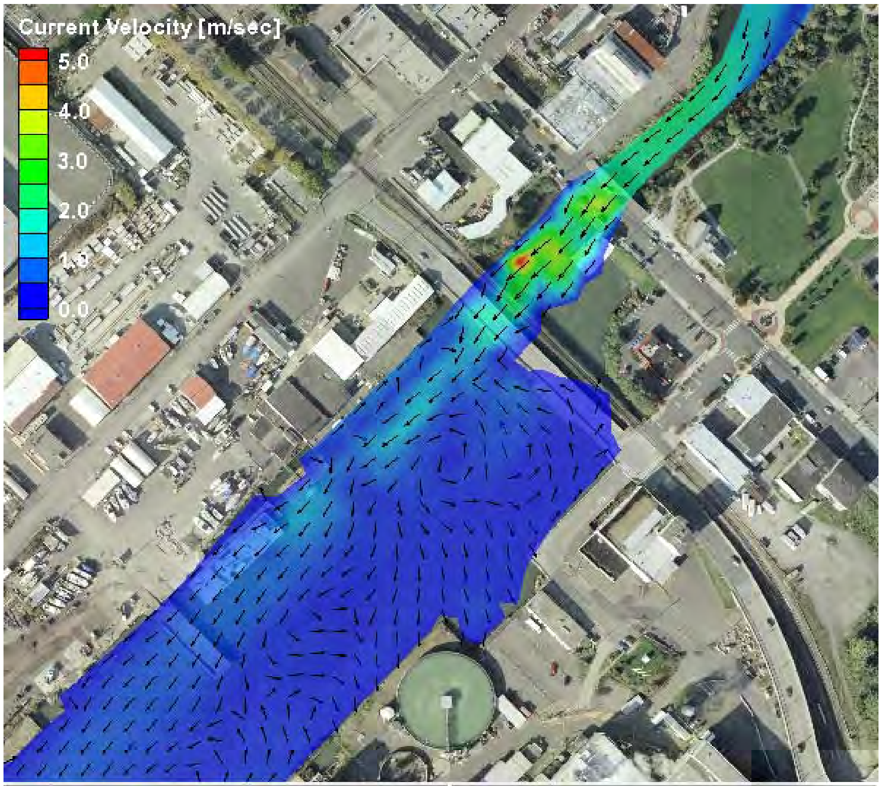


Figure 9. Velocity pattern at time of peak flow, bottom velocity

5. Propeller Wash and Material Stability

5.1. General

Vessel propulsion is a potential source of velocity (propwash) that, under certain circumstances, may impinge on the bottom and side slopes of Whatcom Waterway. Propwash analysis was performed with the model JETWASH for vessel, propulsion, water depth, and bottom slope conditions that were specified by AnchorQEA.

For the purpose of modeling propwash effect, Whatcom Waterway was divided into five distinct areas, as shown in Figure 7. Detail of the propwash analysis area at Bellingham Shipping Terminal is shown in Figure 10.

Twenty-five propwash scenarios were modeled. Most of the modeling scenarios were described in CHE's Technical Memorandum dated October 24, 2011, and are presented in Table 3. Two additional modeling scenarios were requested by AnchorQEA during the study. Modeling scenarios simulated in the current study are described in Table 3.

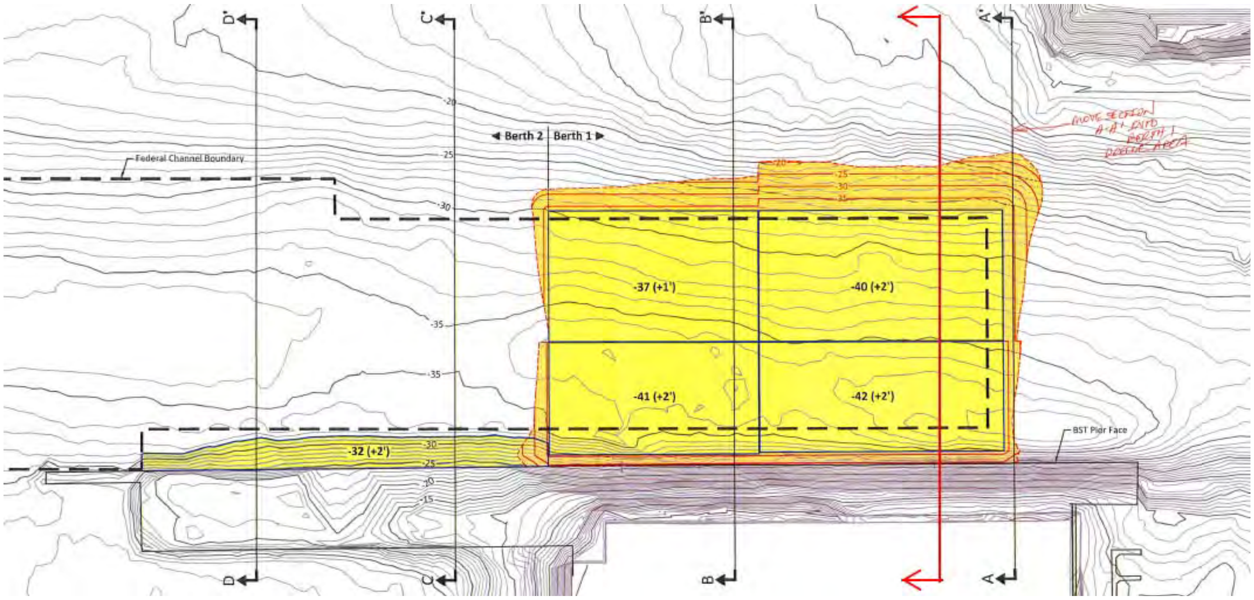


Figure 10. Propwash study areas at Bellingham Shipping Terminal

Table 3. Waterway design areas and propwash modeling scenarios

Parameter	Tractor Tug	Cargo	Cargo	Puget S. Tug	Fishing Boat
Representative Vessel	Garth Foss	Star "O" Main	Star "O" Thruster	Swinomish	Aleutian Falcon
Applied h.p.	4000	1151	1206	722	1,500
% available	50%	10%	60%	85%	33% / 50%
Prop diam.	5.89′	21.6′	9.19′	5.83′	9.67′
Prop r.p.m.	51	58	150	289	150 / 225
Prop draft	12.5′	26.5′	32.8′	11.5′	8.17′
Area 1C elev.	-41′	-41′	-41′	N / A	N / A
Under BST elev.	-41′	-41′	-41′	N / A	N / A
Slope under BST elev.	-41′	-41′	-41′	N / A	N / A
Side slope under BST	2 : 1	2 : 1	2 : 1	N / A	N / A
Area 2C elev.	-25′	N / A	N / A	N / A	N / A
Area 2A elev.	N / A	N / A	N / A	-20	20′
Area 3B	N / A	N / A	N / A	-18	18′

5.2. Propwash Model Output

The JETWASH model was run to determine the maximum near-bottom velocity for each case. Near-bottom velocity is the horizontal mean velocity at a distance of 26 cm above the bottom. Mean velocity is the time average of the turbulent, fluctuating velocity. The mean near-bottom velocity magnitude varies with distance from the propeller. The peak of the mean velocity magnitude is listed in column 6 for each modeling scenario in columns 1 through 5 in Table 4.

Table 4. Propeller wash parameters and stable particle size

Jet Wash Results Nov 2011 Area	Vessel	Propulsion	Location	Depth (ft)	Peak Bottom Velocity [1] (fps)	Median Particle size [2] (in)
1C-BST	**Star O**	main	flat-bottom	41	5.8	2.3
		bow thruster	slope section (a) [3]	41	9.1	5.1
			slope section (b) [4]	41	7.1	3.3
		stern thruster	slope section (a)	41	7.3	3.4
			slope section (b)	41	5.5	2.1
	Tractor Tug	Voith-Schneider	flat-bottom	41	4.1	1.2
Under BST	**Star O**	main	flat-bottom	32	7.3	3.5
		bow thruster	slope section (a)	32	9.1	5.1
			slope section (b)	32	7.1	3.3
		stern thruster	slope section (a)	32	7.3	3.4
			slope section (b)	32	5.5	2.1
	Tractor Tug	Voith-Schneider	flat-bottom	32	6.0	2.4
Slope under BST	**Star O**	main	flat-bottom	41	5.8	2.3
		bow thruster	slope section (a)	41	9.1	5.1
			slope section (b)	41	7.1	3.3
		stern thruster	slope section (a)	41	7.3	3.4
			slope section (b)	41	5.5	2.1
	Tractor Tug	Voith-Schneider	flat-bottom	41	4.1	1.2
			slope section (a)	41	5.4	2.0
			slope section (b)	41	5.6	2.1
2C	**Tractor Tug**	Voith-Schneider	flat-bottom	25	9.4	5.5
2A	**Puget Sound Tug**		flat-bottom	20	6.8	3.0
	Aleutian Falcon		flat-bottom	20	3.3	0.8
3B	**Puget Sound Tug**		flat-bottom	18	9.0	5.1
	Aleutian Falcon		flat-bottom	18	5.7	2.2

Notes:

1. Peak bottom velocity is highest mean velocity computed for segment of profile under consideration, at height of 26 cm above bottom.
2. Stable median particle size assumes particles have a size distribution centered on stated size and with grading = 2 and comprise a layer at least 1.0 ft thick
3. Slope Section (a) is the part of the underwater slope seaward from the vertical cut in the cross-section
4. Slope Section (b) is the part of the under pier slope above the vertical cut in the cross-section

The relationship of near-bottom velocity and stable particle size was developed from a compilation of several published relationships and validated with field experience by CHE. The relationship developed by CHE through this compilation of others' research is graphed in Figure 11. The regression equation expressing particle size is the basis of calculating the median sediment size (column 7 of Table 4) at threshold of motion corresponding to the velocity in column 6.

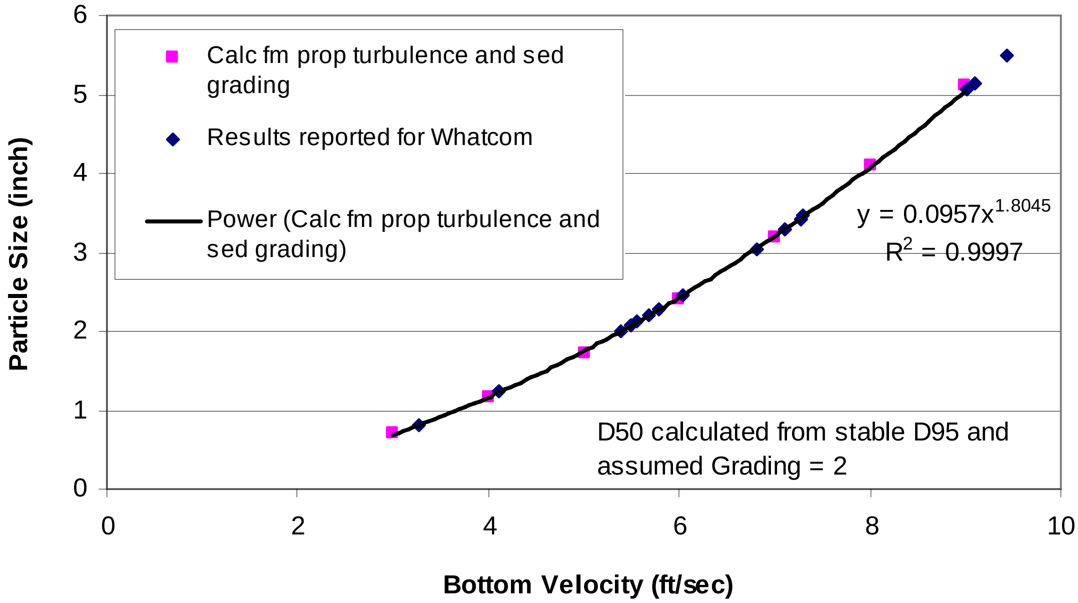


Figure 11. Near-bottom velocity and corresponding stable particle size

6. Reference

CHE. October 24, 2011. Whatcom Waterway Phase 1 Cleanup Hydrodynamic Modeling Cases. CHE Technical Memorandum prepared for AnchorQEA.

APPENDIX D
PROPWASH EVALUATION

TABLE OF CONTENTS

List of Tables

List of Figures

List of Attachments

ACRONYMS AND ABBREVIATIONS

2-D	two-dimensional
Anchor QEA	Anchor QEA, LLC
BST	Bellingham Shipping Terminal
CHE	Coast & Harbor Engineering
D_{50}	Median diameter of sediment in a gradation based on mass; typically determined through sieve analysis
H:V	horizontal to vertical
hp	horsepower
MLLW	mean lower low water
NAVD 88	North American Vertical Datum of 1988
Port	Port of Bellingham
propwash	propeller wash
Waterway	Whatcom Waterway

1 INTRODUCTION

Existing and proposed future vessel operations within Whatcom Waterway were identified and evaluated to determine impacts and define design criteria for proposed engineered capping areas within the Whatcom Waterway (Waterway). Anchor QEA, LLC (Anchor QEA), developed vessel operation scenarios and designed required cap armoring. Coast & Harbor Engineering (CHE) completed propeller wash (propwash) modeling based on developed scenarios to evaluate near-bed velocities and stable sediment and rock sizes. This appendix provides an overview of the vessel operations considered, the methodology employed, and results of the propwash analyses.

2 DATA USED FOR ANALYSIS

Table D-1 lists the data used for the propwash evaluation and Table D-2 outlines the datum elevations used in this analysis (based on mean lower low water [MLLW]).

Table D-1
Summary of Data Used for Evaluation

Data Type	Data Source (author and date)	Purpose of Data
Port Vessel Usages	Port (see Table D-31)	Vessels expected to be used in Whatcom Waterway
Design Elevations and Offsets	Anchor QEA 60% Design Drawings (2012)	Depths and offsets
Datums	NOAA Station No. 9449211	Depths
GP Dock and BST Area Operations	Dan Stahl (Port), 2008-2010	BST and South Shoreline operational vessel specifications
Local Operations	Local Operators, June 2012	Central Waterfront vessel specifications
Tug Operations	Western Towboat Company communications, August 2012	Tug boat operations within Whatcom Waterway

Notes:
BST = Bellingham Shipping Terminal
NOAA = National Oceanic and Atmospheric Administration

Table D-2
Datum Elevations (Station No. 9449211)

Tide Level	Meters (MLLW)	Feet (MLLW)
Highest Observed (1/5/1975)[1]	3.177	10.4
Mean Higher High Water	2.594	8.5
Mean High Water	2.375	7.8
Mean Tide Level	1.546	5.1
Mean Sea Level	1.510	5.0
Mean Low Water	0.718	2.4
Mean Lower Low Water	0.000	0.0
Lowest Observed (12/30/1974)[1]	-1.057	-3.5
NAVD 88	0.147	0.5

Notes:
1. Station No. 9449211 was active from March 30, 1973, to July 21, 1975. Tidal predictions for the area have been higher and lower than those observed.

MLLW = mean lower low water
NAVD 88 = North American Vertical Datum of 1988

3 VESSEL OPERATIONS AND SCENARIOS

Typical vessel operations within Whatcom Waterway were developed through interviews and personal conversations with the Port of Bellingham (Port), existing and potential future operators, and tug operators (see Table D-1). Table D-3 provides a list of the design vessels identified for the propwash evaluation. Based on this information, operational areas were identified that had similar vessel operations: the Outer Waterway/Bellingham Shipping Terminal (BST), the Inner Waterway/South Shoreline, the Inner Waterway/Central Waterfront shoreline, and the Log Pond (see Figure D-1). Vessel operations in these areas were combined with anticipated water depths (post-dredging and capping) to develop scenarios for propwash modeling. These scenarios are described below for each of the defined operational areas within the Waterway.

Table D-3
Summary of Vessel Information

Vessel Category	Vessel Name or Class	Length (feet)	Draft (feet)	Propeller Diameter (feet)	Horsepower (hp)
Puget Sound Tug	Dunlap Towing Swinomish	74	11	5.83	850
Tractor Tug	Garth Foss	100	13.0	5.89	8,000
Break Bulk	Star "O" class	653	39	21.6-main 9.19-thruster	11,510-main 2,010-thruster
Commercial Fishing	Aleutian Falcon	232	8.17	9.67	1,500

3.1 Outer Waterway and Bellingham Shipping Terminal

The design vessels identified for this area include a Tractor Tug- ("Garth Foss") and Star "O"-type cargo vessels. The Star "O" has two propulsion systems that were considered: the main prop and side-thruster. Specific information regarding each vessel, including prop draft and applied horsepower are provided in Table 3 of Attachment 1 (CHE 2012a). The bed elevation in this area was set at the post-dredge elevation of -41 feet MLLW for the purpose of propwash evaluation (see Figure 7 in the main document). In addition to a flat bottom at -41 feet MLLW, two slope conditions were evaluated. Vessel operations included docking and undocking maneuvers of both vessels along the entire length of the BST, including use of available mooring dolphins to the north and south of the BST dock (see Figure D-1).

3.2 Inner Waterway and South Shoreline

The design vessels identified for this area include a Puget Sound Tug ("Swinomish") and a representative large fishing vessel ("Aleutian Falcon"). Specific information regarding each vessel, including prop draft and applied horsepower are also provided in Table 3 of Attachment 1 (CHE 2012a). The bed elevation in this area was set at the proposed top of cap elevation of -20 feet MLLW for the purpose of propwash evaluation (see Figure 11 in the main document). Vessel operations included transit of both vessels through the Inner Waterway and docking and undocking maneuvers of both vessels along the entire length of the shoreline (GP dock and clarifier areas) (see Figure D-1).

3.3 Inner Waterway and Central Waterfront Shoreline

The design vessel identified for this area is the Puget Sound Tug ("Swinomish"). Specific information regarding this vessel provided in Table 3 of Attachment 1 (CHE 2012a). The bed elevation in this area was set at the proposed top of cap elevation of -20 feet MLLW for the purpose of propwash evaluation (see Figures 14a and 14b in main document). Vessel operations included transit of both vessels through the Inner Waterway and docking and undocking maneuvers of both vessels along the entire length of the shoreline (see Figure D-1). In addition to vessel operations within the deep water areas along the shoreline, several specific along-shore operations were considered based on anticipated uses in this area in the future. These operations include tug/barge operations that will be docking barges of various sizes lateral to the shoreline at a series of mooring piles along the shoreline (see Figures 14a and 14b in the main document). The barges considered varied in width from 35 to 70 feet to take into account the range of potential offset distances that the tug would be located from the shoreline if tied alongside the waterward-side of the barge while at berth. Slope conditions were defined by the proposed top of cap location as shown in Figures 14a and 14b in the main document. Typical slope conditions used in propwash modeling represent conditions shown in Sections D and F (Figures 15b and 15c in main document).

3.4 Log Pond

The Log Pond has no assumed vessel traffic; therefore, no propwash evaluation was completed for this area. Armor design in this area was developed based on storm wave impacts. See Appendix C for details on Log Pond armor based on wave impacts.

4 OVERVIEW OF PROPWASH MODELING

A two-dimensional (2-D) modeling tool called JETWASH was used by CHE to evaluate near-bottom velocities due to propwash from design vessels (see Attachment 3, CHE 2007). These velocities are required for calculating shear stress on the bed. The JETWASH model simulates the velocity field created by propulsion systems and accounts for interaction of the velocity jet with the bottom boundary. Additional information regarding the model and its application are provided in Attachment 3 (CHE 2007).

5 RESULTS AND DISCUSSION

The following sections outline the results of the propwash analysis (near bed velocities and stable sediment/rock size) conducted by CHE (CHE 2012a, 2012b).

5.1 General

Table D-4 shows a summary of the propwash modeling results for all scenarios, regardless of area. Modeling results include predicted near-bed velocities induced by the prop under defined scenarios and estimated stable sediment/rock size under predicted velocities (see Attachment 3 for methodology).

Table D-4
Propwash Velocities and Sediment Sizes, Summary

Area	Vessel	Propulsion	Location	Depth (feet)	Peak Bottom Velocity[1] (feet/sec)	Median Particle Size[2] (inch)
Outer Waterway	Star "O" Class	Main	Flat bottom	41–32	5.8–7.3	2.3–3.5
Outer Waterway	Star "O" Class	Bow thruster	Slope (a)[3]	41	9.1	5.1
Outer Waterway	Star "O" Class	Bow thruster	Slope (b)[4]	41	7.1	3.3
Outer Waterway	Star "O" Class	Stern thruster	Slope (a)	41	7.3	3.4
Outer Waterway	Star "O" Class	Stern thruster	Slope (b)	41	5.5	2.1
Outer Waterway	Tractor Tug	Voith Schneider	Flat bottom	41	4.1	1.2
Inner Waterway	Puget Sound Tug	Main	Flat bottom	20	6.8	3.0
Inner Waterway	Aleutian Falcon	Main	Flat bottom	20	3.3	0.8
South Shoreline	Puget Sound Tug	Main	Flat bottom	20	4.9	1.7
Central Waterfront	Puget Sound Tug	Main	Slope	3-20	11.6	8.0

Notes:
1. Peak bottom velocity is highest mean velocity computed by the model for segment of profile under consideration, at height of 10.24 inches above bottom (see Attachment 3).
2. Stable median particle size assumes particles have a size distribution centered on stated size and compose a layer at least 1 foot thick.
3. Slope Section (a) is the part of the underwater slope seaward from the vertical cut in the cross-section.
4. Slope Section (b) is the part of the under-pier slope above the vertical cut in the cross-section.

5.2 Outer Waterway/Bellingham Shipping Terminal

The remedy proposed for the Outer Waterway/BST includes dredging to remove all contaminated sediment within the navigation channel (see Figures 7 and 8a in the main report). No engineered capping is proposed in the open-water portion of the Outer Waterway. A residuals management cover is anticipated to be placed in this area, but is intended to mix with any remaining contaminated sediment residuals through propwash and other hydrodynamic forces. Therefore, a propwash evaluation was not needed to determine stable sediment size within the Outer Waterway area.

Potential re-suspension of under-pier sediments due to propwash from vessels using the BST berth area is a possibility. The under-pier areas of the BST are to be remediated as part of Phase 2 cleanup. The Port plans on implementing institutional controls for vessels using the BST to minimize the risk for under-pier re-suspension. Long-term monitoring required for Phase 1 will identify whether recontamination of the Outer Waterway occurs; if the areas are recontaminated, the Phase 2 cleanup will address those areas.

The transition slope from the dredged area in the Outer Waterway into the Inner Waterway will be covered with an engineered cap to ensure stability of the transition slope (post-dredging) under propwash forces from vessels operating in the navigation channel adjacent to the BST. A 2-foot-thick sand layer will be placed on the cut slope with a filter and armor system placed on top. The filter material will have a D_{50} of 3 inches and be well graded; it will be placed with a minimum thickness of 1 foot. The armor layer will have a minimum thickness of 1.5 feet and have a D_{50} of 9 inches. Table D-5 summarizes the armor requirements and specifications for the Outer Waterway transition slope cap based on stated design vessels and operations.

Table D-5
Outer Waterway Transition Slope Results, Summary
Vessels: Puget Sound Tug

Armor or Filter Requirement	Specification
Design Velocities	6.6 to 9.4 feet per second
Calculated Slope Armor (D_{50})	1.6 to 6 inch
Specified Slope Armor (D_{50})	9 inches
Specified Slope Filter (D_{50})	3.0 inch (well graded)

5.3 Inner Waterway

The Puget Sound Tug transiting through the Inner Waterway results in the largest stable sediment size (as estimated by the modeling effort). The final capped Waterway will have a surface elevation no higher than -20 feet MLLW (see Figure 9b in main report), which will be constructed using a D_{50} armor size of 3 inches (minimum 1 foot thick) placed on top of the isolation sand layer designed for this area.

Table D-6 summarizes the armor requirements and specifications for the Inner Waterway.

Table D-6
Inner Waterway Results, Summary
Design Vessels: Aleutian Falcon and Puget Sound Tug

Armor or Filter Requirement	Specification
Design Velocities	3.3 to 6.8 feet per second
Calculated Bottom Armor (D_{50})	0.25 feet (3.0 inch)
Specified Bottom Armor/Filter (D_{50})	3.0 inch (well graded)

5.3.1 South Shoreline

The South Shoreline consists of sloping shorelines and existing vertical bulkheads (see Section 7.3 of the main report). For all shoreline areas, propwash impacts are the critical design factor for subtidal elevations and wave impacts are the critical design factor for intertidal areas (see Appendix C for the coastal engineering evaluation).

Design vessels accessing this shoreline area include the Puget Sound Tug and Aleutian Falcon (see Attachment 2). Predicted near-bed velocities along the southern shoreline for each of these vessels are 4.9 feet per second and 3.3 feet per second, respectively. These expected maximum near-bed velocities require a stable armor size of less than 3-inches, which is the required armor size based on vessel operations in the open water portion of the Inner Waterway (Section 4.2.2). Therefore, the specified bottom armor for the Inner Waterway of 3 inches will be used on the South Shoreline slopes up to an elevation of -8 feet MLLW. Above -8 feet MLLW, the 3-inch material will be used as a filter for the required intertidal armor (9-inch armor rock), which is required for shoreline stability under storm induced waves (see Appendix C). The 9-inch armor rock will be extended to the toe of the slope (-20 feet MLLW) at the transition between the clarifier area and the GP dock (see Figure 11) to account for direct propwash impacts to that transition slope from vessels anticipated to operate at the GP dock.

Table D-7 summarizes the armor requirements and specifications for the South Shoreline.

Table D-7
South Shoreline Results, Summary
Design Vessel: Puget Sound Tug

Armor or Filter Requirement	Specification
Design Velocities	3.3 to 6.8 feet per second
Calculated Slope Armor (D_{50})	0.25 feet (3.0 inch)
Specified Lower Slope Armor/Filter (D_{50})[1]	3.0 inch (well graded)
Specified Upper Slope Armor	0.75 feet (9.0 inch)

Notes:
1. The lower armor/filter will cover the entire slope; below -8 feet MLLW, this specification acts as the armor and above -8 feet MLLW, this specification acts as a filter material for the upper armor, except where noted on Figure 9.

5.3.2 Central Waterfront Shoreline

The Central Waterfront shoreline consists of sloping shorelines (ranging from 2 horizontal to 1 vertical [2H:1V] to 3H:1V) and existing vertical bulkheads (see Section 7.3 of the main report). The design vessel along this shoreline is the Puget Sound Tug, which will be used to bring barges of various sizes into and out of this area (see Attachment 2). Operational parameters that were used to evaluate near-bed velocities along the slope include size of barge, offset of barge from the shoreline and mooring piles, tug beam, and prop draft of the tug. Based on the various scenarios, armor requirements along the majority of the Central Waterfront shoreline are the same as required along the South Shoreline (3-inch-sized material or smaller will be stable under estimated propwash velocities). The one exception is the subtidal slope in front of the Maple Street bulkhead. A proposed new bulkhead will be constructed in front of the existing bulkhead, and will be fronted by a proposed 35–foot-wide shelf at -8 feet MLLW elevation (see Figure 14b in main document). This shelf extends the subtidal slope out farther into the Waterway at this location and, as a result, requires the design tug to operate directly over the subtidal slope area. This operational need results in higher propwash velocities (predicted to be approximately 11.6 feet per second) along the subtidal slope fronting the Maple Street bulkhead as compared to sloped areas along other portions of the Central Waterfront shoreline. These velocities require a stable D_{50} armor size of at least 8-inches. Therefore, the 9-inch armor rock required for shoreline stability under wave impact (above -8 feet MLLW) will be extended from -8 feet MLLW to -20 feet MLLW in front of the Maple Street bulkhead (see Figures 14b and 15c in the main document). The remainder of the Central Waterfront shoreline will be armored similarly to the Southern Shoreline. The 9-inch armor rock (to protect against wave impacts) will be placed from top of slope down feet to -8 feet MLLW and then the lower slope armor of 3-inch material will be placed from -8 feet MLLW down to the toe of slope to provide cap stability from propwash forces.

Table D-8 summarizes the armor requirements and specifications for the South Shoreline.

Table D-8
Central Waterfront Results, Summary (see Attachment 2)
Design Vessel: Puget Sound Tug

Armor or Filter Requirement	Specification
Design Velocities	3.9 to 11.6 feet per second
Calculated Slope Armor (D_{50})	0.8 to 8.0 inch
Specified Lower Slope Armor/Filter (D_{50})[1]	3.0 inch (well graded)
Specified Upper Slope Armor[2]	0.75 feet (9.0 inch)

Notes:
1. The lower armor/filter will cover the entire slope; below -8 feet MLLW, this specification acts as the armor and above -8 feet MLLW, this specification acts as a filter material for the upper armor.
2. The upper armor will extend down, over the lower slope armor, to -20 feet MLLW in front of the Maple Street bulkhead only.

5.4 Uncertainty Discussion

The evaluation of propwash forces uses well-established and validated modeling methods to estimate potential erosive forces from vessel operations. The sources of uncertainty in that evaluation result from both the reliability of the vessel information, and the assumptions used for modeling site operational scenarios.

Future vessel operations were estimated using both past observations, as well as an evaluation of potential future vessel operations. The vessel operations information has been obtained from local experts with direct experience with Whatcom Waterway navigation activities, including operations managers at local tug and barge companies, and Port planners and engineers. This information on past and potential future vessels was used along with a capacity-based analysis to identify a range of design vessels for evaluation, and to identify reasonable and upper-probable operational parameters for modeling. All reasonable attempts were made to bracket a range of vessel types and operational conditions representative of likely future conditions within the Site. However, future changes in propulsion systems or significant changes in land use could result in vessel or operational characteristics beyond those evaluated in this report.

Modeling scenarios developed for the analysis took vessel operational uncertainty into account by using conservative operational criteria for the propwash simulations. The operation parameters for each modeling scenario (e.g., percent power used for bow thrusters and actual tug operations) were established at conservative values that may overestimate typical or upper probable propwash forces. These scenarios are anticipated to drive sediment mobilization in the Whatcom Waterway (due to propwash) to a larger extent than a single emergency maneuver or event. However, some extreme handling situations (e.g., emergency maneuvers under extreme conditions) could occur outside of the evaluated range on an infrequent basis. These emergency conditions are generally not used for design. Potential impacts of these extreme conditions are addressed through monitoring and contingencies.

Additional uncertainties associated with the propwash modeling include defining transitions between operational areas, understanding the duration of each operation (e.g., how long the vessel uses its bow thruster at 100% power), and the choice of representative water depths for the simulations. As with uncertainties in operational information, conservative assumptions were used when developing the simulations to offset these additional uncertainties. Simulations assumed steady state conditions for vessels transiting Whatcom Waterway (i.e., infinite duration of operations in one location), and water depths chosen for the simulations in each of the operational areas were conservatively low (i.e., shallower depths within each operational area).

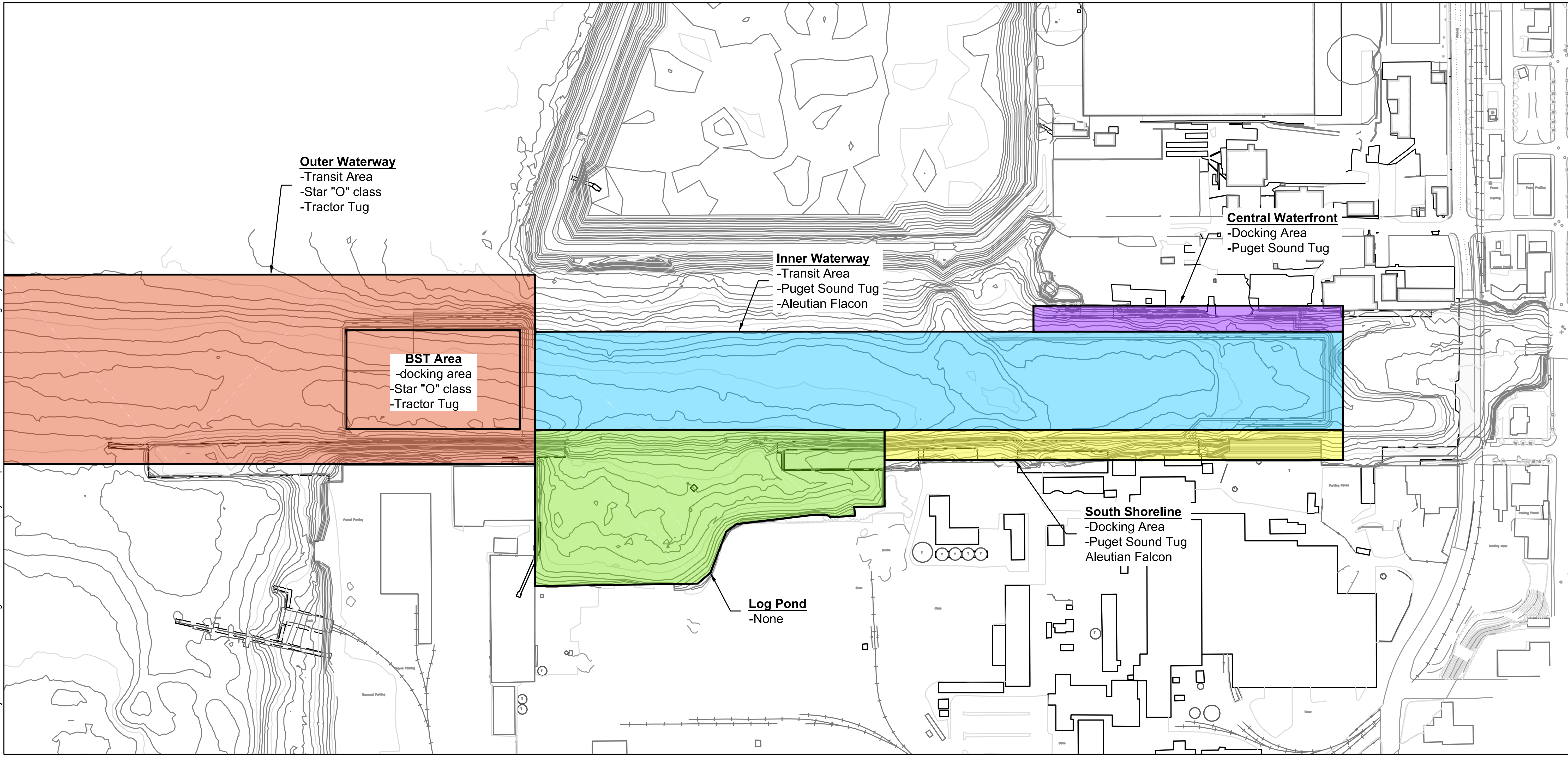
6 REFERENCES

CHE (Coast and Harbor Engineering), 2007. *Technical Memorandum 5 – Lower Fox River OUs 2-5, Jetwash Model Boundary Layer Development Analysis*. March 22, 2007.

CHE (Coast and Harbor Engineering), 2012a. *Whatcom Waterway Phase 1 Cleanup Project.* Port of Bellingham 60% Design Hydrodynamic Modeling and Coastal Engineering Analysis. January 2012.

CHE (Coast and Harbor Engineering), 2012b. *Whatcom Waterway Phase 1 Cleanup Project.* Propwash Modeling of Puget Sound Tug for 90% Design Prepared for Anchor QEA. September 2012.

FIGURES



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SOURCE: Drawing prepared from bathymetric survey provided by Wilson Engineering.
HORIZONTAL DATUM: Washington State Plane North, NAD83, Feet.
VERTICAL DATUM: Mean Lower Low Water (MLLW), Feet.

ANCHOR QEA

Figure D-1
Vessel Operational Areas
Appendix D - Propwash Evaluation
Whatcom Water Cleanup in Phase 1 Site Areas - Final EDR

ATTACHMENT 1
TECHNICAL REPORT
PORT OF BELLINGHAM 60% DESIGN
HYDRODYNAMIC MODELING AND
COASTAL ENGINEERING ANALYSIS

WHATCOM WATERWAY PHASE 1 CLEANUP PROJECT
Port of Bellingham 60% Design Hydrodynamic Modeling and Coastal Engineering Analysis

COAST & HARBOR ENGINEERING

Technical Report

WHATCOM WATERWAY PHASE 1 CLEANUP PROJECT
Port of Bellingham 60% Design Hydrodynamic Modeling and Coastal Engineering Analysis

Prepared for:

AnchorQEA

This document is to assist with 60% design of the Port of Bellingham's Whatcom Waterway Cleanup Project. This document presents the results of hydrodynamic modeling and coastal engineering analysis conducted by CHE. This document is not to be used for purposes of final engineering design, or for construction documents.

Prepared by:

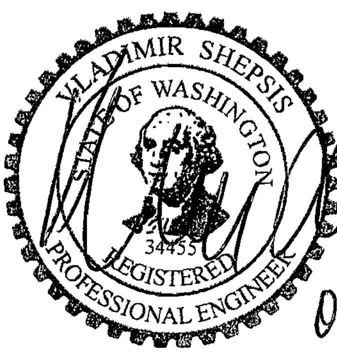

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TABLE OF CONTENTS

FIGURES

TABLES

COAST & HARBOR ENGINEERING

Technical Report
Whatcom Waterway Phase 1 Cleanup Project, Port of Bellingham 60% Design Hydrodynamic Modeling and Coastal Engineering Analysis

1. Introduction

This brief technical report summarizes the results of hydrodynamic modeling and coastal engineering analysis conducted by Coast & Harbor Engineering (CHE) to assist AnchorQEA with 60% design of the Whatcom Waterway Phase 1 Cleanup Project. The scope of CHE's hydrodynamic numerical modeling included two-dimensional (2-D) wave refraction, diffraction, and reflection; three-dimensional (3-D) tidal circulation and Whatcom Creek flows, propwash velocity and stability of sediment subjected to propwash, and tabulation of size of sediment at the bottom of the waterway and on side slopes at the edge of the waterway in designated areas, and at the bottom and shoreline in areas within the log pond.

Prior to conducting modeling and analysis, CHE prepared and coordinated the Technical Memorandum *Whatcom Waterway Phase 1 Cleanup Hydrodynamic Modeling Cases* (CHE 2011) with AnchorQEA. This technical memorandum described the physical conditions, input parameters, and modeling scenarios that were used for hydrodynamic modeling herein. The current brief report does not repeat these characteristics, input parameters and modeling scenarios; but rather, directs interested readers to the October 2011 CHE technical memorandum.

2. Wave Modeling

2.1. General

Wave generation, propagation to the project site, and interaction with bottom sediment and waterfront structures was conducted using 2-D numerical modeling software, SWAN and HWAVE. Waves were simulated (numerically modeled) with two steps, using a large area modeling domain (Step 1), and high resolution, nested numerical modeling domain (Step 2). Wave spectral information developed in SWAN is passed to HWAVE at open water boundaries of the nested grid. Bathymetry of the large and nested modeling domains is shown in Figure 1 in color format.

For Phase 1 project wave modeling was conducted for a 100-year return period wind speed approaching from 240 degrees True North (TN) and from 270 degrees TN. Waves were modeled with tide levels at MHHW and lower. Wave modeling results at high tide level are presented below in Section 2.1 as plan views of significant wave heights over the modeling domains and wave heights and periods extracted at the requested locations. Wave parameters at requested locations were extracted for each modeling scenario by locating numerical wave gauges at appropriate nodes of the modeling grid, shown in Figure 2, which output wave height, period, and direction.

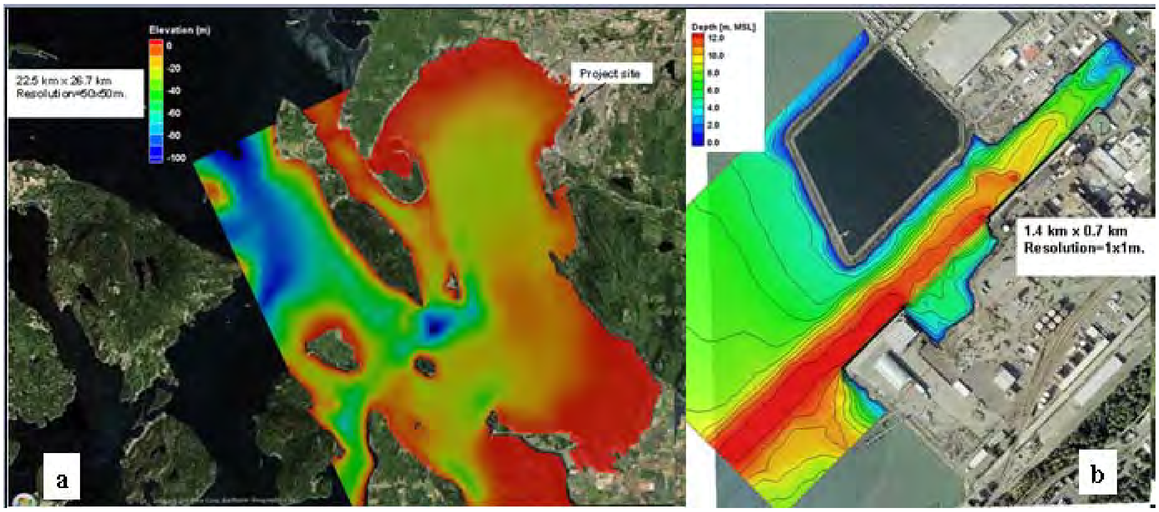

Figure 1. Large (a) and nested (b) wave numerical modeling domains

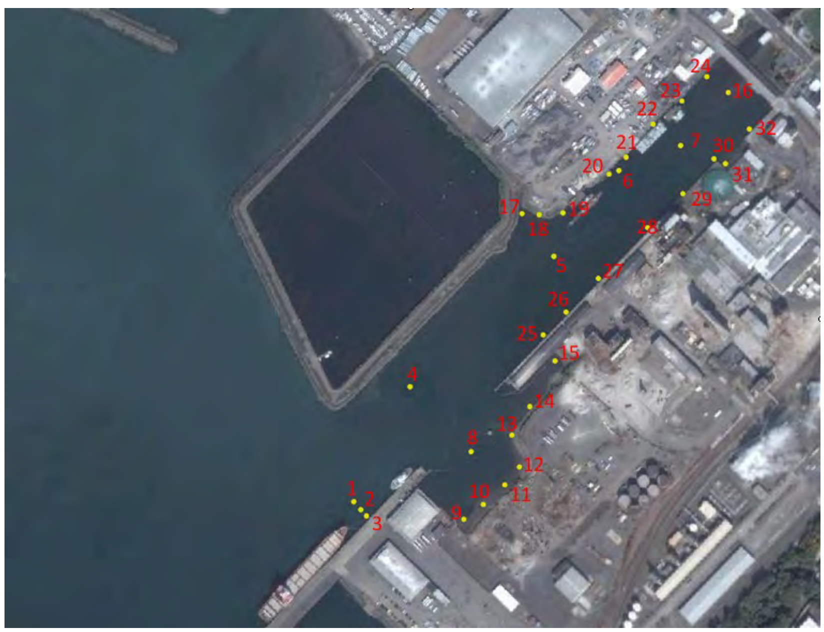

Figure 2. Wave model information extraction points

2.2. Wave Model Output

Results of HWAVE modeling with the high resolution grid to simulate significant wave heights during the 100-year return period storms are shown in Figures 3 and 4. Wave height is shown in color format. Red color indicates higher waves and blue color indicates smaller or no waves. Figure 3 shows the wave height pattern over the modeling domain during the design storm approaching from 240 degrees. Similarly,

Figure 4 shows wave heights for the design storm approaching from 270 degrees. Color patterns represent the distribution of wave heights, not wave fronts. Vectors in the figures show wave direction at grid points.

Modeled wave heights, wave periods, and corresponding water depths at the requested locations (see Figure 2 above) on the modeling grid are presented in Table 1 for wave direction 240 degrees, and in Table 2 for 270 degrees.

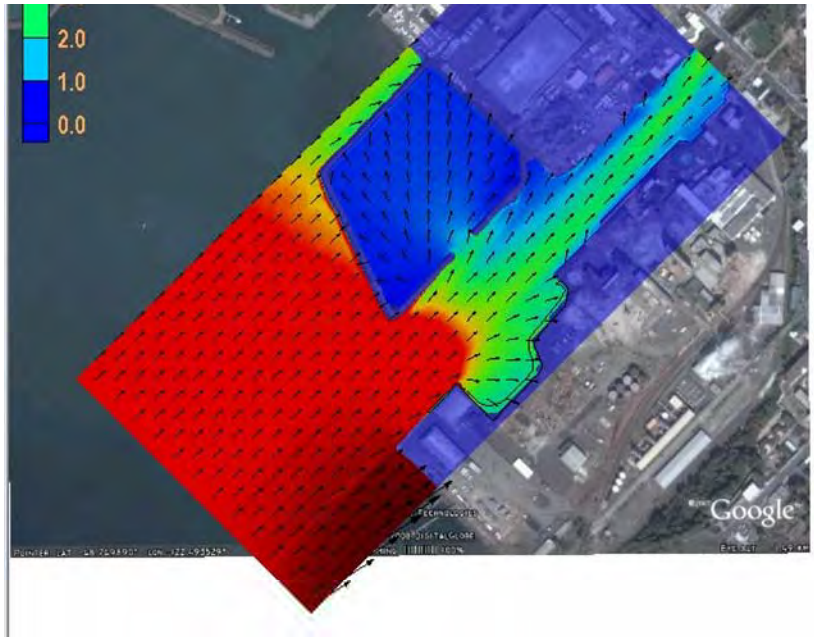

Figure 3. Wave height and direction in Whatcom Waterway, wave storm approaching from 240 degrees

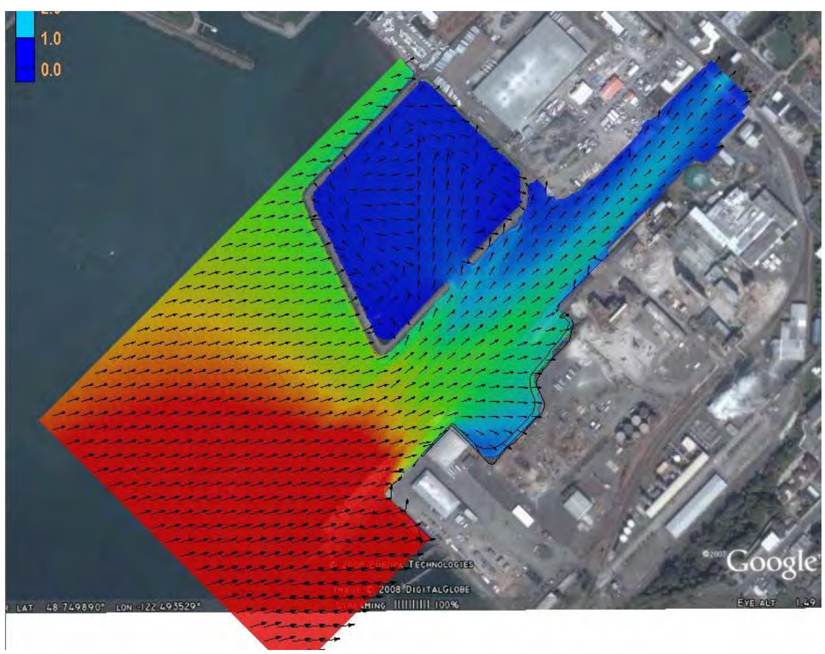

Figure 4. Wave height and direction in Whatcom Waterway, wave storm approaching from 270 degrees

Table 1. Wave height and period and water depth, waves from 240 degrees

	100-Yr Return Period, direction 240 deg				
	Significant Wave Height, Period, Depth at Extractions Points				
	Point Coordinates				
	UTM - meters				
Point	**X**	**Y**	**Depth (ft)**	**Hs (ft)**	**Tp (s)**
1	537228.0	5399365.1	50.8	5.34	4.47
2	537239.3	5399352.2	50.8	5.18	4.47
3	537248.5	5399342.1	40.0	4.92	4.47
4	537319.2	5399551.0	36.8	3.79	4.11
5	537552.4	5399761.9	36.8	1.28	4.47
6	537657.9	5399900.7	19.2	0.61	4.47
7	537758.1	5399941.2	31.8	2.30	3.86
8	537418.2	5399446.1	20.4	2.78	4.22
9	537406.4	5399336.6	5.3	1.64	5.62
10	537438.1	5399360.6	7.6	1.65	4.22
11	537472.8	5399392.3	7.1	1.63	4.22
12	537496.5	5399421.3	5.9	1.78	4.22
13	537484.3	5399472.8	8.3	2.45	4.47
14	537513.4	5399519.0	7.9	1.84	4.47
15	537554.3	5399592.8	12.1	1.49	4.47
16	537835.3	5400027.0	13.7	2.04	4.22
17	537500.8	5399831.0	14.9	0.27	3.98
18	537528.6	5399829.3	11.6	0.44	3.94
19	537567.2	5399832.2	14.3	0.57	3.44
20	537641.9	5399894.9	10.5	0.59	3.76
21	537669.7	5399922.3	11.5	0.56	3.69
22	537713.3	5399975.9	3.7	0.62	3.21
23	537760.2	5400013.2	18.0	0.61	2.98
24	537800.5	5400052.6	16.9	1.07	1.20
25	537535.1	5399634.9	30.1	1.64	4.47
26	537572.2	5399671.9	29.0	1.39	4.21
27	537624.6	5399726.3	28.7	1.16	4.10
28	537704.7	5399808.3	13.5	0.72	3.76
29	537761.7	5399863.4	-0.1	0.65	3.65
30	537812.1	5399919.6	7.2	0.95	4.47
31	537830.8	5399911.9	2.8	0.66	4.73
32	537869.4	5399967.8	15.6	0.63	4.47

Table 2. Wave height and period and water depth, waves from 270 degrees

	100-Yr Return Period, direction 270 deg				
	Significant Wave Height, Period, Depth at Extractions Points				
	Point Coordinates				
	UTM - meters				
Point	**X**	**Y**	**Depth (ft)**	**Hs (ft)**	**Tp (s)**
1	537228.0	5399365.1	50.8	4.30	4.22
2	537239.3	5399352.2	50.8	4.10	4.22
3	537248.5	5399342.1	40.0	3.96	4.22
4	537319.2	5399551.0	36.8	1.81	3.76
5	537552.4	5399761.9	36.8	0.69	3.98
6	537657.9	5399900.7	19.2	0.26	3.76
7	537758.1	5399941.2	31.8	0.90	3.76
8	537418.2	5399446.1	20.4	1.79	4.22
9	537406.4	5399336.6	5.3	0.54	5.01
10	537438.1	5399360.6	7.6	0.75	4.22
11	537472.8	5399392.3	7.1	0.94	4.22
12	537496.5	5399421.3	5.9	1.32	4.22
13	537484.3	5399472.8	8.3	1.61	4.22
14	537513.4	5399519.0	7.9	1.42	4.22
15	537554.3	5399592.8	12.1	1.33	3.76
16	537835.3	5400027.0	13.7	0.82	3.76
17	537500.8	5399831.0	14.9	0.20	3.98
18	537528.6	5399829.3	11.6	0.28	3.35
19	537567.2	5399832.2	14.3	0.25	3.21
20	537641.9	5399894.9	10.5	0.22	3.84
21	537669.7	5399922.3	11.5	0.20	3.67
22	537713.3	5399975.9	3.7	0.27	3.23
23	537760.2	5400013.2	18.0	0.31	3.01
24	537800.5	5400052.6	16.9	0.26	2.78
25	537535.1	5399634.9	30.1	1.36	3.99
26	537572.2	5399671.9	29.0	1.10	3.90
27	537624.6	5399726.3	28.7	0.95	3.87
28	537704.7	5399808.3	13.5	0.39	3.76
29	537761.7	5399863.4	-0.1	0.31	3.60
30	537812.1	5399919.6	7.2	0.38	4.47
31	537830.8	5399911.9	2.8	0.17	2.40
32	537869.4	5399967.8	15.6	0.26	4.47

3. Bottom Material Stability in Log Pond

Modeling and analysis were conducted to determine stability of bottom and shoreline sediment subjected to the design wave storm at the Log Pond area. The area known as the Log Pond is an embayment on the south side of the Waterway and east of the Bellingham Shipping Terminal (BST). Sediment stability analysis was determined according to the ability of sediment to resist hydrodynamic forces generated in the design storm by wave orbital velocities for the non-breaking wave conditions, and swash velocities in the wave breaking area and surf zone. For this purpose, the bottom orbital velocities and wave swash velocities were determined from wave modeling and compared to threshold velocities of sediment motion. Graphical results of material stability analysis at a water level of high tide and wave direction from 240 degrees (most severe conditions) for the Log Pond are shown in Figure 5. The figure presents the pattern of minimum stable size in color format: red color represents a larger size of sediment and blue color represents a smaller size. The figure shows that along the shoreline at the wave breaking area stability criteria require material size to be in the range of riprap. This result is reasonable and is supported by local data.

Figure 5. Stable material size according to location, wave direction from 240 degrees

4. Flow Circulation Modeling

4.1. General

Flow velocities in Whatcom Waterway were simulated for the condition of a combined tidal series of neap to spring tides and storm discharge from Whatcom Creek. The model simulates unsteady flow in three dimensions. Storm discharge consists of the 100-year return period discharge from both the controlled and uncontrolled areas of the drainage. The peak discharge modeled at the mouth of Whatcom Creek was 2400 c.f.s.

Modeling was conducted with the 3-D flow circulation model SELFE. The model was validated and calibrated previously based on field data collected in Bellingham Bay; and specifically, in Whatcom Waterway. Modeling was conducted on several modeling grids, including a large modeling grid and two nested grids, shown in Figure 6. The minimum size of the element in the modeling grid was in the range of 4-6 ft.

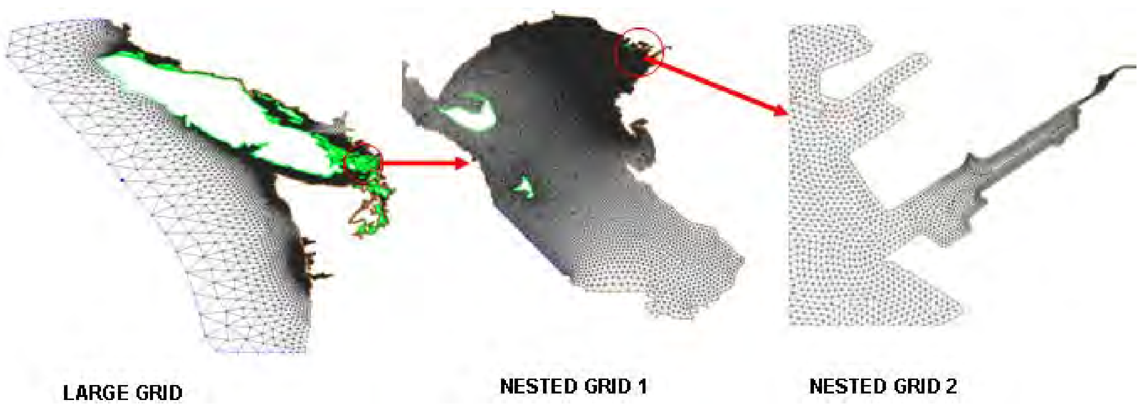

Figure 6. SELFE numerical modeling grids for Whatcom Waterway hydrodynamic modeling

4.2. Modeling Results

Modeling results were extracted from the modeling output of depth-averaged velocity and velocity at the bottom boundary in the modeling domains. Specifically, upon the request from Anchor QEA, the modeling results were extracted in areas 3A-MNR and 3B (see Figure 7 for area locations).

Modeled depth-averaged velocity distribution over the modeling domain of Nested Grid 2 is shown in Figure 8 in color format. Modeled bottom velocity distribution over the modeling domain of Nested Grid 2 is shown in Figure 9. Maximum velocities for the two cases were extracted from the results and are 19 ft/sec (5.8 m/s) and 17.06 (5.2 m/s), respectively. As shown in the figures, the peak velocity occurs at the channel bottom between the two bridges.

Figure 7. Bottom effects study areas within Whatcom Waterway

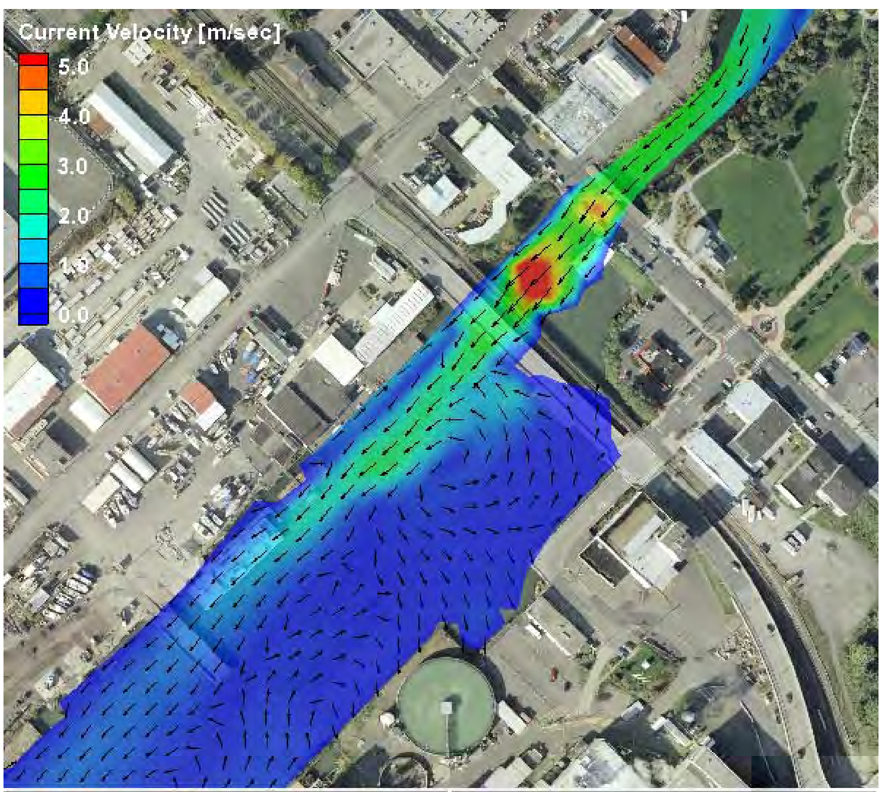

Figure 8. Velocity pattern at time of peak flow, depth-averaged velocity

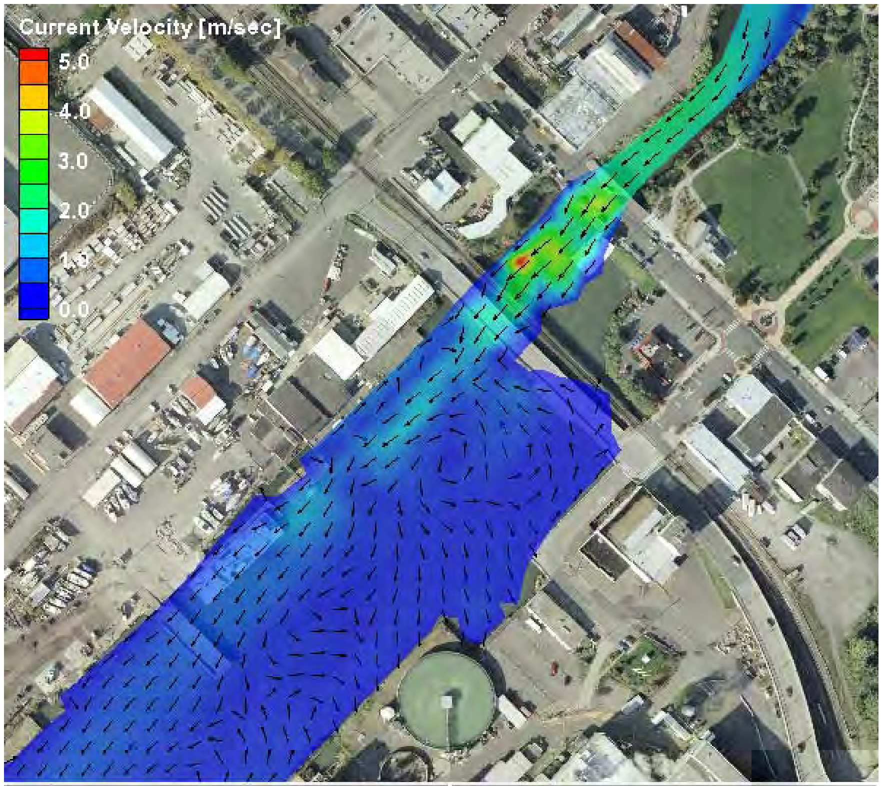

Figure 9. Velocity pattern at time of peak flow, bottom velocity

5. Propeller Wash and Material Stability

5.1. General

Vessel propulsion is a potential source of velocity (propwash) that, under certain circumstances, may impinge on the bottom and side slopes of Whatcom Waterway. Propwash analysis was performed with the model JETWASH for vessel, propulsion, water depth, and bottom slope conditions that were specified by AnchorQEA.

For the purpose of modeling propwash effect, Whatcom Waterway was divided into five distinct areas, as shown in Figure 7. Detail of the propwash analysis area at Bellingham Shipping Terminal is shown in Figure 10.

Twenty-five propwash scenarios were modeled. Most of the modeling scenarios were described in CHE's Technical Memorandum dated October 24, 2011, and are presented in Table 3. Two additional modeling scenarios were requested by AnchorQEA during the study. Modeling scenarios simulated in the current study are described in Table 3.

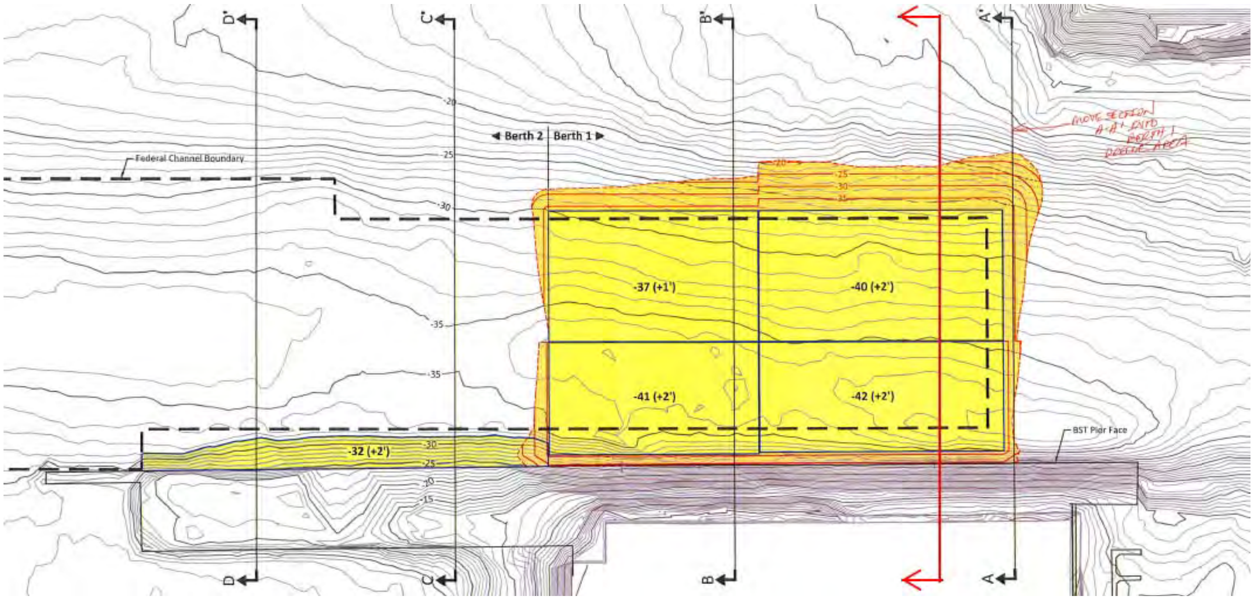

Figure 10. Propwash study areas at Bellingham Shipping Terminal

Table 3. Waterway design areas and propwash modeling scenarios

Parameter	Tractor Tug	Cargo	Cargo	Puget S. Tug	Fishing Boat
Representative Vessel	Garth Foss	Star "O" Main	Star "O" Thruster	Swinomish	Aleutian Falcon
Applied h.p.	4000	1151	1206	722	1,500
% available	50%	10%	60%	85%	33% / 50%
Prop diam.	5.89′	21.6′	9.19′	5.83′	9.67′
Prop r.p.m.	51	58	150	289	150 / 225
Prop draft	12.5′	26.5′	32.8′	11.5′	8.17′
Area 1C elev.	-41′	-41′	-41′	N / A	N / A
Under BST elev.	-41′	-41′	-41′	N / A	N / A
Slope under BST elev.	-41′	-41′	-41′	N / A	N / A
Side slope under BST	2 : 1	2 : 1	2 : 1	N / A	N / A
Area 2C elev.	-25′	N / A	N / A	N / A	N / A
Area 2A elev.	N / A	N / A	N / A	-20	20′
Area 3B	N / A	N / A	N / A	-18	18′

5.2. Propwash Model Output

The JETWASH model was run to determine the maximum near-bottom velocity for each case. Near-bottom velocity is the horizontal mean velocity at a distance of 26 cm above the bottom. Mean velocity is the time average of the turbulent, fluctuating velocity. The mean near-bottom velocity magnitude varies with distance from the propeller. The peak of the mean velocity magnitude is listed in column 6 for each modeling scenario in columns 1 through 5 in Table 4.

Table 4. Propeller wash parameters and stable particle size

Jet Wash Results Nov 2011 Area	Vessel	Propulsion	Location	Depth ft	Peak Bottom Velocity [1] fps	Median Particle size [2] in
1C-BST	**Star O**	main	flat-bottom	41	5.8	2.3
		bow thruster	slope section (a) [3]	41	9.1	5.1
			slope section (b) [4]	41	7.1	3.3
		stern thruster	slope section (a)	41	7.3	3.4
			slope section (b)	41	5.5	2.1
	Tractor Tug	Voith-Schneider	flat-bottom	41	4.1	1.2
Under BST	**Star O**	main	flat-bottom	32	7.3	3.5
		bow thruster	slope section (a)	32	9.1	5.1
			slope section (b)	32	7.1	3.3
		stern thruster	slope section (a)	32	7.3	3.4
			slope section (b)	32	5.5	2.1
	Tractor Tug	Voith-Schneider	flat-bottom	32	6.0	2.4
Slope under BST	**Star O**	main	flat-bottom	41	5.8	2.3
		bow thruster	slope section (a)	41	9.1	5.1
			slope section (b)	41	7.1	3.3
		stern thruster	slope section (a)	41	7.3	3.4
			slope section (b)	41	5.5	2.1
	Tractor Tug	Voith-Schneider	flat-bottom	41	4.1	1.2
			slope section (a)	41	5.4	2.0
			slope section (b)	41	5.6	2.1
2C	**Tractor Tug**	Voith-Schneider	flat-bottom	25	9.4	5.5
2A	**Puget Sound Tug**		flat-bottom	20	6.8	3.0
	Aleutian Falcon		flat-bottom	20	3.3	0.8
3B	**Puget Sound Tug**		flat-bottom	18	9.0	5.1
	Aleutian Falcon		flat-bottom	18	5.7	2.2

Notes:
1. Peak bottom velocity is highest mean velocity computed for segment of profile under consideration, at height of 26 cm above bottom.
2. Stable median particle size assumes particles have a size distribution centered on stated size and with grading = 2 and comprise a layer at least 1.0 ft thick
3. Slope Section (a) is the part of the underwater slope seaward from the vertical cut in the cross-section
4. Slope Section (b) is the part of the under pier slope above the vertical cut in the cross-section

The relationship of near-bottom velocity and stable particle size was developed from a compilation of several published relationships and validated with field experience by CHE. The relationship developed by CHE through this compilation of others' research is graphed in Figure 11. The regression equation expressing particle size is the basis of calculating the median sediment size (column 7 of Table 4) at threshold of motion corresponding to the velocity in column 6.

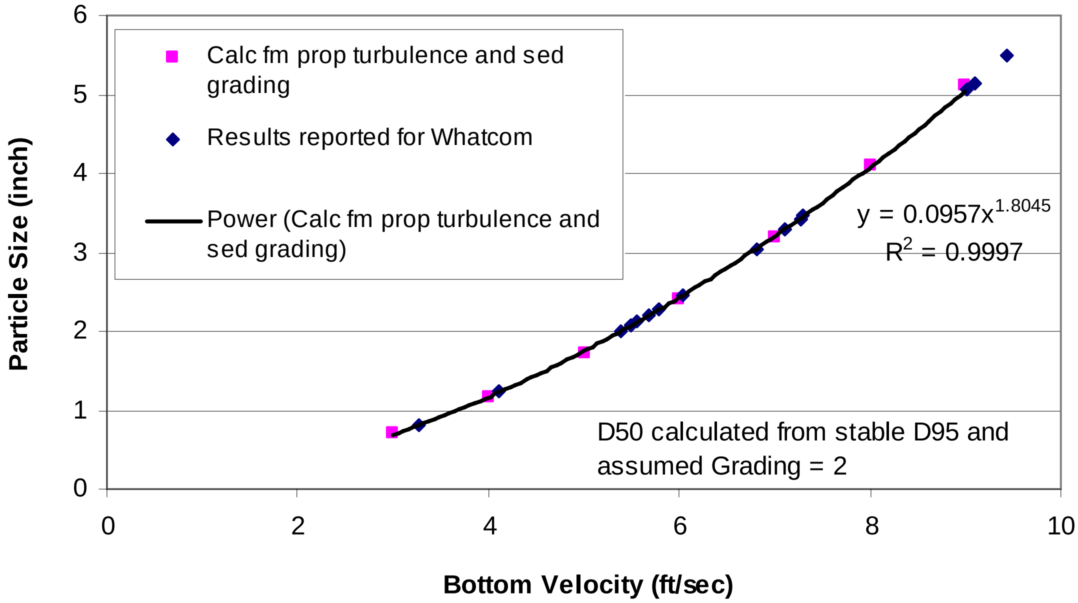

Figure 11. Near-bottom velocity and corresponding stable particle size

6. Reference

CHE. October 24, 2011. Whatcom Waterway Phase 1 Cleanup Hydrodynamic Modeling Cases. CHE Technical Memorandum prepared for AnchorQEA.

ATTACHMENT 2
TECHNICAL MEMORANDUM
PROPWASH MODELING OF PUGET SOUND TUG FOR 90% DESIGN

COAST & HARBOR ENGINEERING

Technical Memorandum

Whatcom Waterway Phase 1 Cleanup Project
Propwash Modeling of Puget Sound Tug for 90% Design

1. Introduction

This brief technical memorandum summarizes the results of propwash modeling conducted by Coast & Harbor Engineering (CHE) to assist AnchorQEA with the Whatcom Waterway Cleanup and Marina Development Phase I Project (Anchor QEA Project Number: 080007-01). The scope of CHE's numerical modeling included two-dimensional (2-D) propwash velocity and stability of sediment subjected to propwash, and tabulating the size of sediment at the bottom of the waterway and on sideslopes at the edge of the waterway for specific cases of slope configuration, vessel orientation, and water level.

Input specified by AnchorQEA for CHE to conduct modeling and analysis consisted of six cases, referred to as Runs 1 through 6. Channel and slope geometry and dimensions, and tug position and orientation were illustrated in the file *Attachment A Propwash Test Matrix Rev1 8 30 2012.xlsx*. The current memorandum summarizes these characteristics, input parameters, and modeling scenarios in tables and graphics in the following sections[1].

2. Propwash Modeling

2.1. General

Modeling was conducted with a 2-D steady model JETWASH (Shepsis, Fenical and Tirindelli 2005). Cross-sections illustrating the bottom profiles and tug positions near the head of Whatcom Waterway were provided by AnchorQEA in an email dated August 30, 2012. The cross-section configuration were either a sideslope of 3 H : 1 V extending down from a bulkhead at the channel edge to the channel bottom at -20 ft, or a flat bench at elevation -8 ft at the channel edge, then a 3 H : 1 V slope downward to the channel bottom at -20 ft. The portion of the waterway that is the subject of this modeling and analysis is shown in Figure 1.

The tug dimensions represent a Puget Sound tug - a hypothetical vessel, with some dimensions patterned after the tug SWINOMISH. Please note that dual propellers were coded into the modeled vessel (Puget Sound tug); whereas, the actual SWINOMISH has a

[1] For more description of the propwash model and modeling methodology as well as modeling results of waves, currents, and sediment stability at Whatcom Waterway, the reader is referred to CHE Technical Report *Port of Bellingham 60% Design Hydrodynamic Modeling and Coastal Engineering Analysis*, dated January 30, 2012.

single propeller. Vessel and propulsion parameters are listed in Table 1. Propwash model parameters and assumptions are listed in Table 2.

Figure 1. Whatcom Waterway area of propwash modeling

Table 1. Vessel modeling parameters

Parameter	Puget S. Tug
Vessel draft	11 ft
Applied h.p.	722
% available	85%
Prop diam.	5.83 ft
Prop r.p.m.	289
Prop draft	8.1 ft
Number of props	Two
Ducted/nonducted	Ducted
Rudder/no rudder	No rudder

Table 2. Propwash modeling scenarios

Parameter	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6
Tide level above MLLW	0′	7′	0′	3.2′	0′	0′
Vert distance from prop center to bottom	11.9′	18.9′	11.9′	6.3’	11.9’	11.9’
Horiz distance from prop center to toe of slope	4.9’	4.9’	38.9’	(over slope)	8.9’	89.0’

2.2. Propwash Model Output

The JETWASH model was run to determine the maximum near-bottom velocity and the two-dimensional pattern of near-bottom velocity for each case. Near-bottom velocity is the horizontal mean velocity at a distance of 26 cm above the bottom. A tilted bottom is coded to calculate velocity in a plane parallel to the channel sideslope where the velocity field intersects the slope. Figures 2 through 8 show the pattern of velocity magnitude, with insets to illustrate the bottom configuration of the model run.

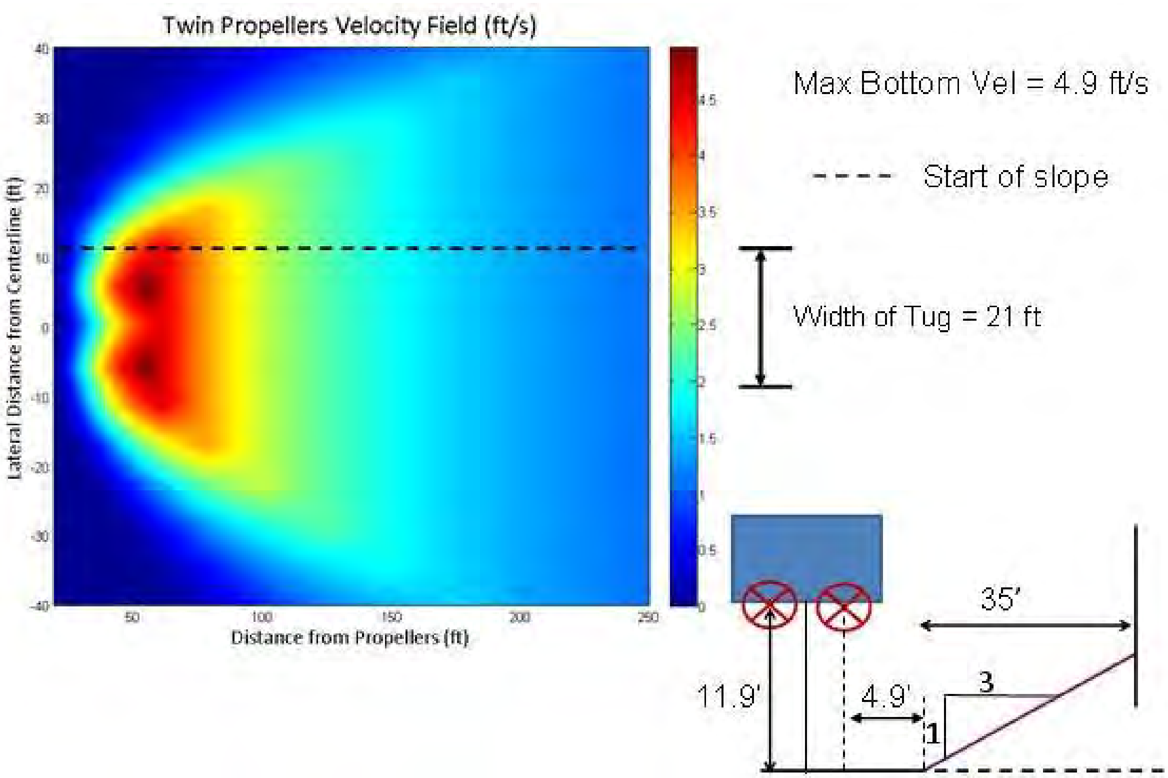

Figure 2. Whatcom Waterway near-bottom propwash velocity, Run 1

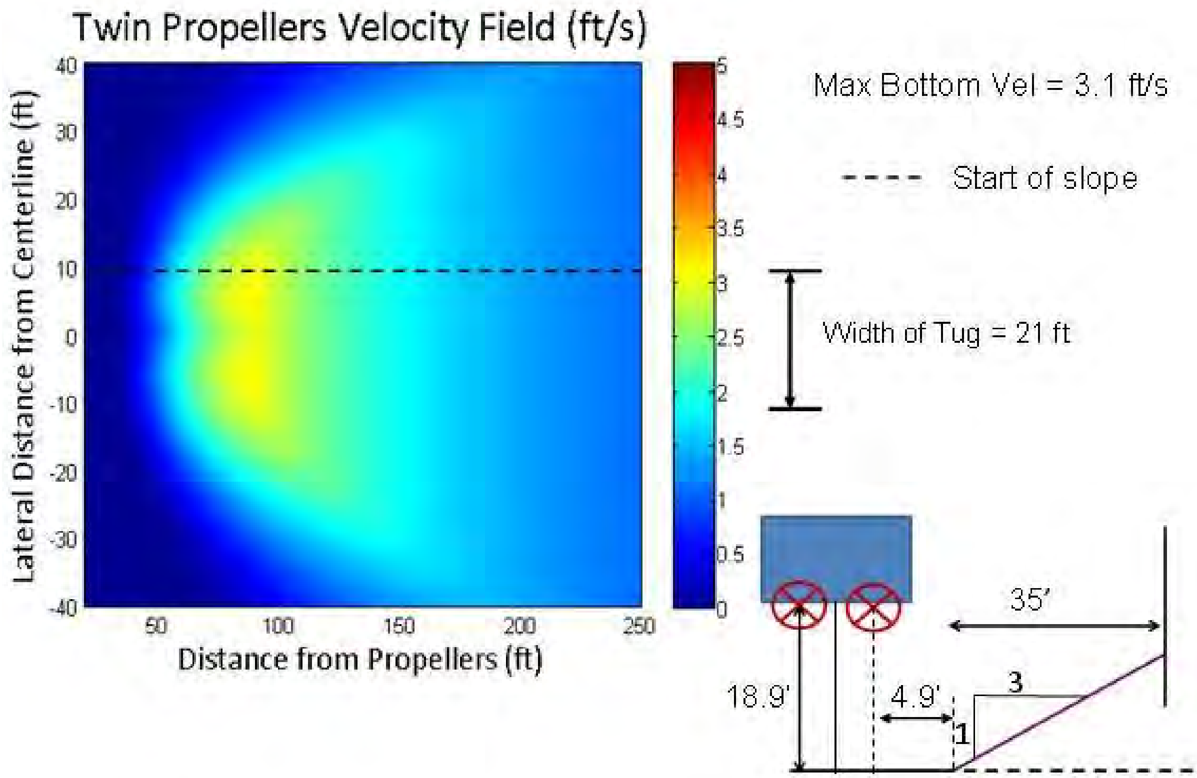

Figure 3. Whatcom Waterway near-bottom propwash velocity, Run 2

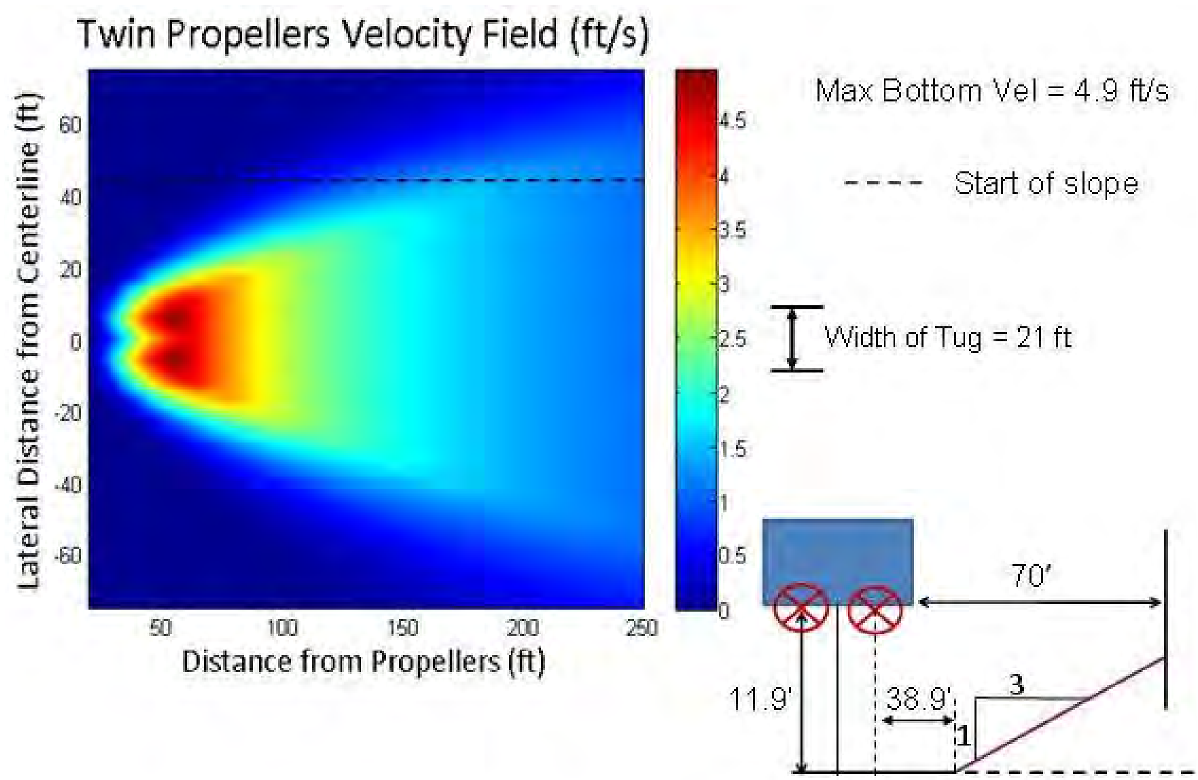

Figure 4. Whatcom Waterway near-bottom propwash velocity, Run 3

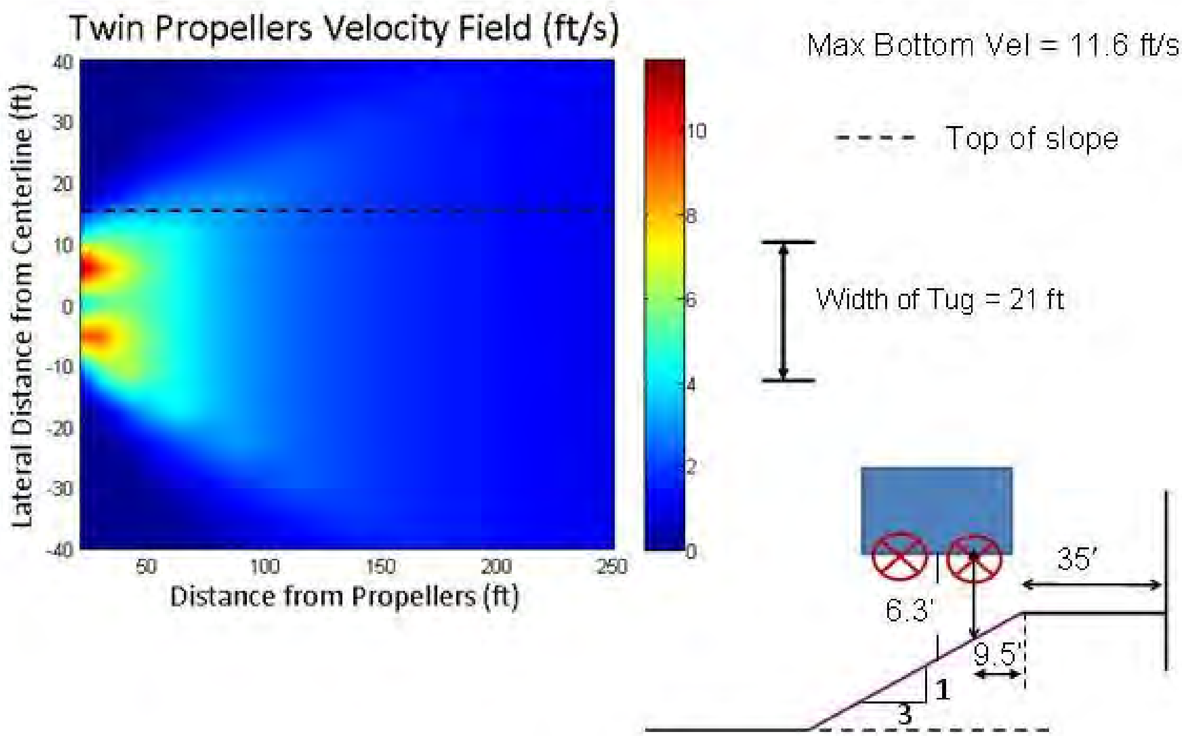

Figure 5. Whatcom Waterway near-bottom propwash velocity, Run 4

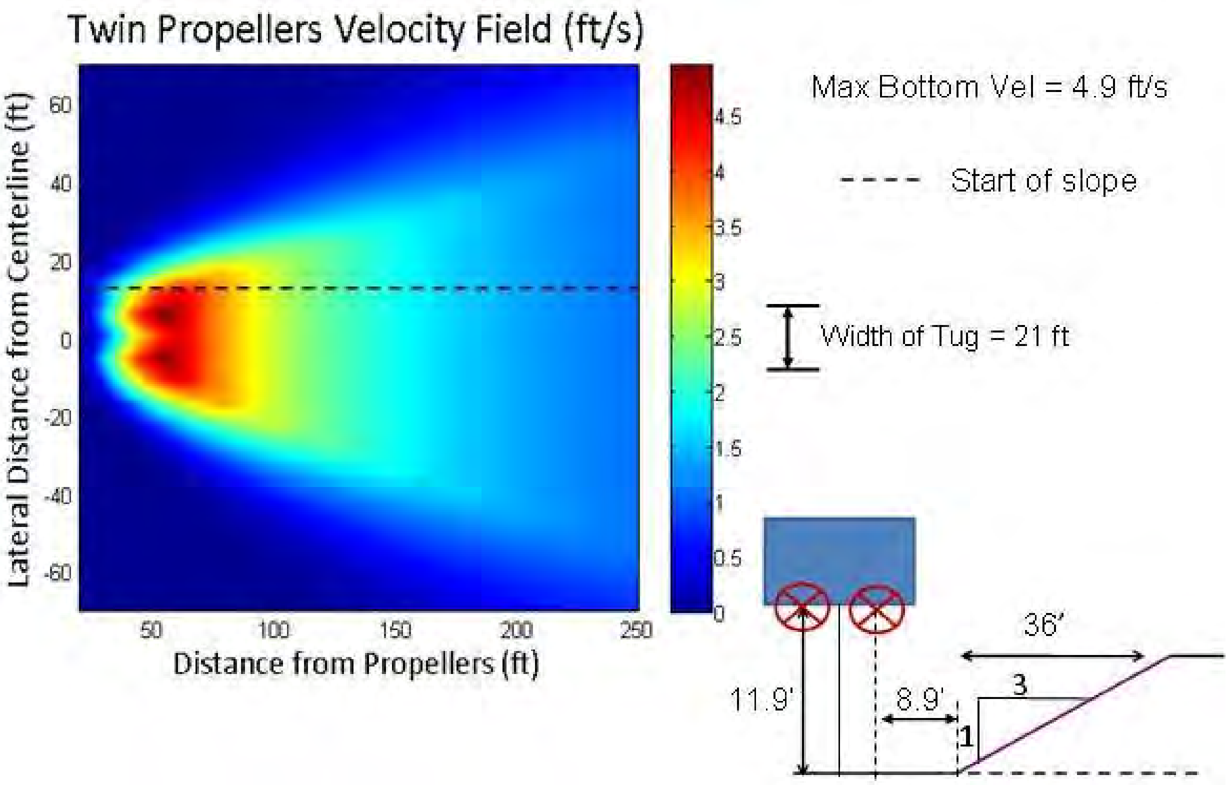

Figure 6. Whatcom Waterway near-bottom propwash velocity, Run 5

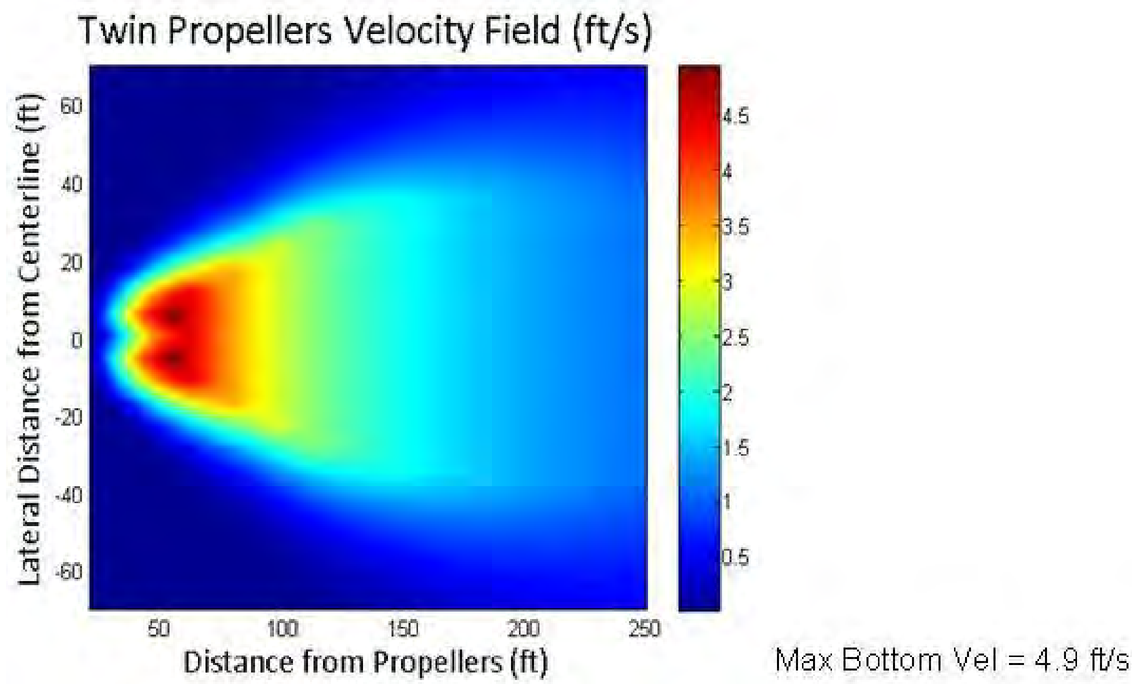

Figure 7. Whatcom Waterway near-bottom propwash velocity, Run 6

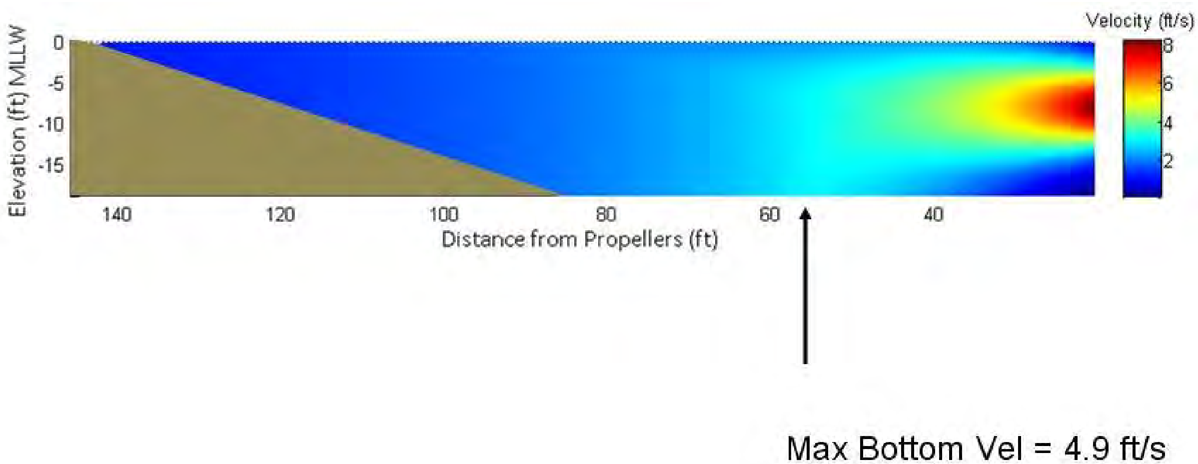

Figure 8. Whatcom Waterway vertical slice through axis of propwash velocity, Run 6

3. Stable Sediment Size

The stable particle size was computed for this modeling of propwash using the relationship between of near-bottom velocity and stable particle size previously developed and validated with field experience by CHE. The resulting regression equation expressing particle size is the basis of calculating the median sediment size (column 3 of Table 3) at threshold of motion corresponding to the velocity in column 2.

Table 3. Velocity and stable sediment size

Parameter	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6
Maximum Near-Bottom Velocity (ft/sec)	4.9	3.1	4.9	11.6	4.9	4.9
Sediment Size at Threshold of Motion (in)	1.7	0.8	1.7	8.0	1.7	1.7

4. References

Shepsis, V., S. Fenical, M. Tirindelli. 2005. Contaminated Sediment Capping: Propwash Modeling and Application. Presented at Western Dredging Association Conference, 2005.

ATTACHMENT 3
TECHNICAL MEMORANDUM NO. 5
JETWASH MODEL BOUNDARY LAYER DEVELOPMENT ANALYSIS

Technical Memorandum 5

JETWASH Model Boundary Layer Development Analysis

A boundary layer is the zone of flow in the immediate vicinity of the bottom surface in which the motion of the fluid is affected by the frictional resistance exerted by the bottom. Schematically, the boundary layer for propwash flow is shown in Figure 1.

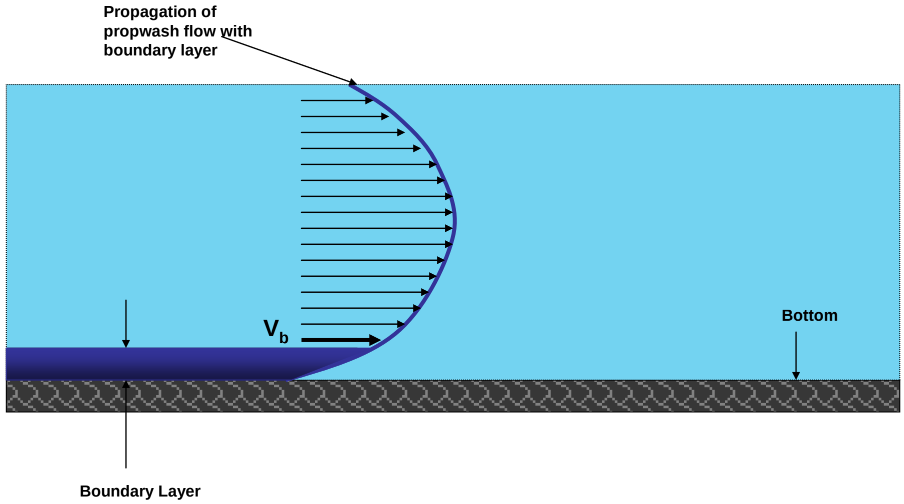

Figure 1. Schematic of propagation of propwash flow with boundary layer

For still water (when flow velocity equals zero) the boundary layer does not exist. The boundary layer forms as a consequence of the boundary's frictional resistance applied to the flowing fluid.

Theoretically, it should take some period of time to form a fully developed boundary layer after flow suddenly starts in still water (for example, a boat producing propwash moves over the lake). If it were possible to measure propwash velocity during boundary layer development, it is likely that this velocity would be larger than the velocity at the same elevation in the case with fully developed the boundary layer. Figure 2 shows schematically the theoretical differences in propwash velocities for flows with and without a boundary layer.

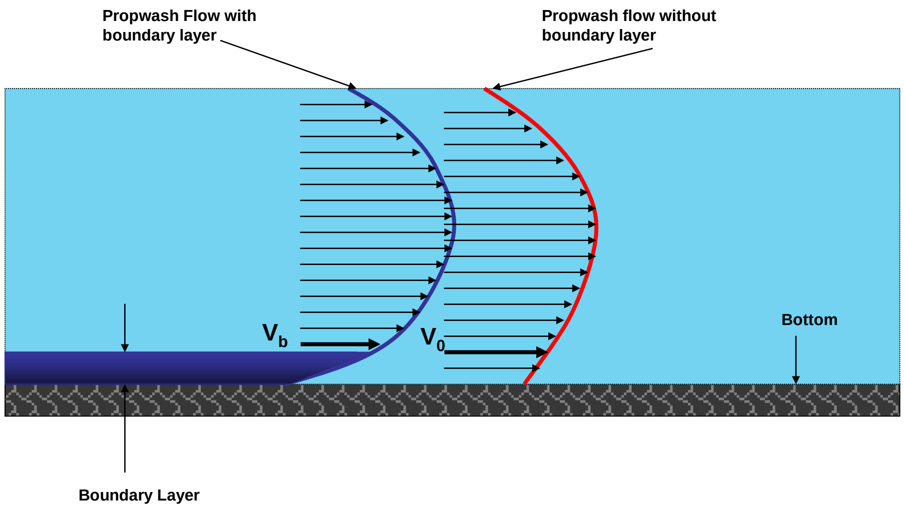

Figure 2. Propagation of propwash flow with and without a boundary layer

The JEWASH model does not account directly for existence of a boundary layer. It does; however, include other conservative factors that indirectly account for the boundary layer. Therefore, there has been concern that actual propwash flow with a non-established boundary layer (initial impingement of the jet at the bottom) may affect bottom sediment (cap material) with larger shear stress than that calculated with velocities predicted by JETWASH at specified heights above the bottom.

At present, no methods exist for assessing boundary layer development for conditions such as propeller wash impinging on the sediment bed. Shear stress in the non-fully developed boundary layer is a fundamental theoretical problem that cannot be solved in the scope of current study. However, it can be demonstrated that JETWASH results are sufficiently conservative to compensate for boundary layer development effects. To do this, Coast & Harbor Engineering (CHE) assumed that shear stress at the bottom is proportional to bottom flow velocity at a small distance above the bed. JETWASH velocity results calculated for cases near a bottom boundary and with no bottom boundary were compared. It can be reasonably assumed that near-bottom velocities during boundary layer development will not be greater than those from the no bottom boundary case. The goal of the following discussion is to 1) demonstrate that JETWASH conditions with a boundary produce higher velocities at water depths near the boundary than the no-boundary conditions. From this, it can then be demonstrated that 2) bottom shear stress is greater in JETWASH than for the comparable estimates for the period during boundary layer development due to the built-in conservatism in JETWASH. A height above the sediment bed of 15 cm was selected as the height at which velocities were compared, and is defined herein as the "near-bottom" velocity from which bottom shear stress is calculated. This height is sufficiently close to the bottom so that shear stress estimates will be conservative, both under developing and developed boundary layer conditions.

Denoting instantaneous actual bottom propwash velocity without a boundary layer (infinite water depth) as V_0 and JETWASH predicted velocity at the same elevation as Vj, the evaluation can be summarized as follows:

- If $V_0 > V_j$, then JETWASH may not be conservative enough and additional analysis of sediment stability is required.
- If $V_0 < V_j$, JETWASH is conservative enough and can be used for the design of cap layer.

The objective of the analysis is to determine the difference between propwash velocity at the bottom with a not fully-developed boundary layer and propwash velocity (of the same source) predicted by the JETWASH model.

No reliable and commonly accepted methods (formulae, models) exist to compute flow velocity for developing boundary layer conditions during jet impingement. Therefore, the above described JETWASH comparison tests were applied. The test includes computing near-bottom velocity with JETWASH at the existing water depth (boundary layer included), and at the same elevation but with the bottom moved to infinite depth. Figure 3 schematically shows the infinite depth concept for this evaluation.

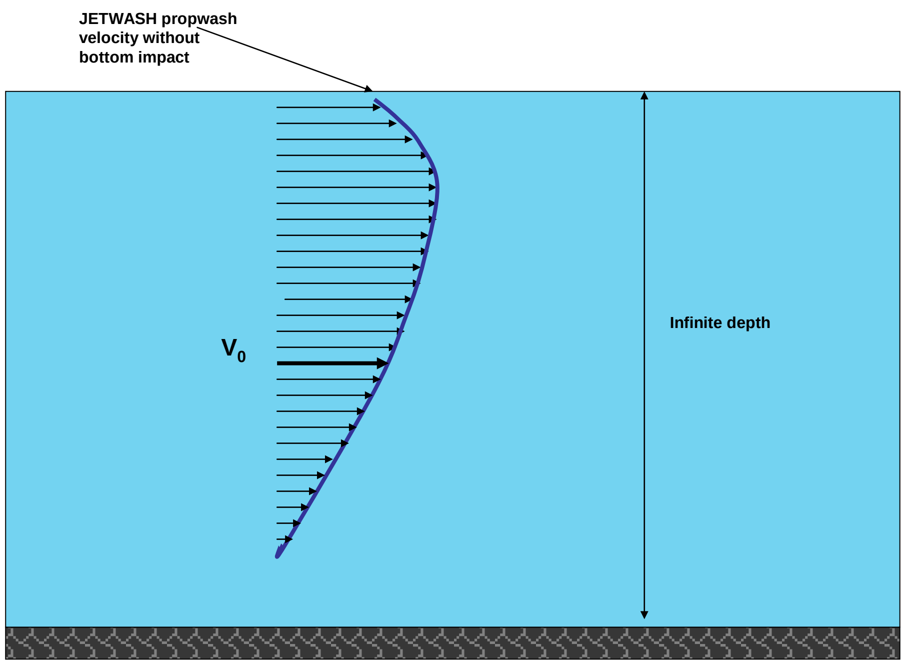

Figure 3. Schematic of computational test, flow velocity at a fixed elevation with bottom moved to infinite depth

A standard definition for a bottom boundary layer is: "…zone of flow in the immediate[1] vicinity of bottom surface in which the motion of the fluid is affected by the frictional resistance exerted by the bottom" (Middleton and Southard 1984). Therefore, velocity V_0 is not affected by frictional resistance of the bottom, and we can assume V_0 is equal

[1] For certain conditions it may influence much of the water column

to or greater than the near-bed velocity in developing boundary layer conditions. This evaluation includes computing bottom velocity with JETWASH for various conditions (including a Fox River example that is used in Technical Memo 3) and repeating the computations for cases with the bottom at such depth that it does not affect the flow. JETWASH algorithms are designed such that the propeller-induced total flux for each case (depth-limited and infinite depth) is equal (assuming identical prop conditions). Therefore, JETWASH forces the depth-limited case to include increased near-bottom velocities to compensate for areas in the infinite depth case that are below the natural water depth in the depth-limited case but where flux still occurs (i.e., JETWASH reflects the additional flux back up into the near-bottom layer of the depth-limited case; thus, the built in near-bottom conservatism). The results of a sample computation are presented in Table 1. Figure 1 is an example of velocity profile computations at 5.9 ft behind the prop showing velocities at heights of 5 to 30 cm above the bottom for 5-ft water depth conditions. The built-in conservatism of JETWASH can be seen in the depth-limited profile, which does not fit the 'typical' near-bottom velocity profile where there is a rapid decrease near-bottom. This is due to the reflection of the infinite-depth case velocities back into the boundary layer.

Table 1, Results of JETWASH computational test

Monterey, 5-ft water depth	**Velocity at 15 cm above bottom (ft/sec)**	
Distance behind prop (ft)	**With bottom**	**No bottom effect**
4.8	5.907	5.907
5.9	11.489	9.549
7.1	13.205	10.600

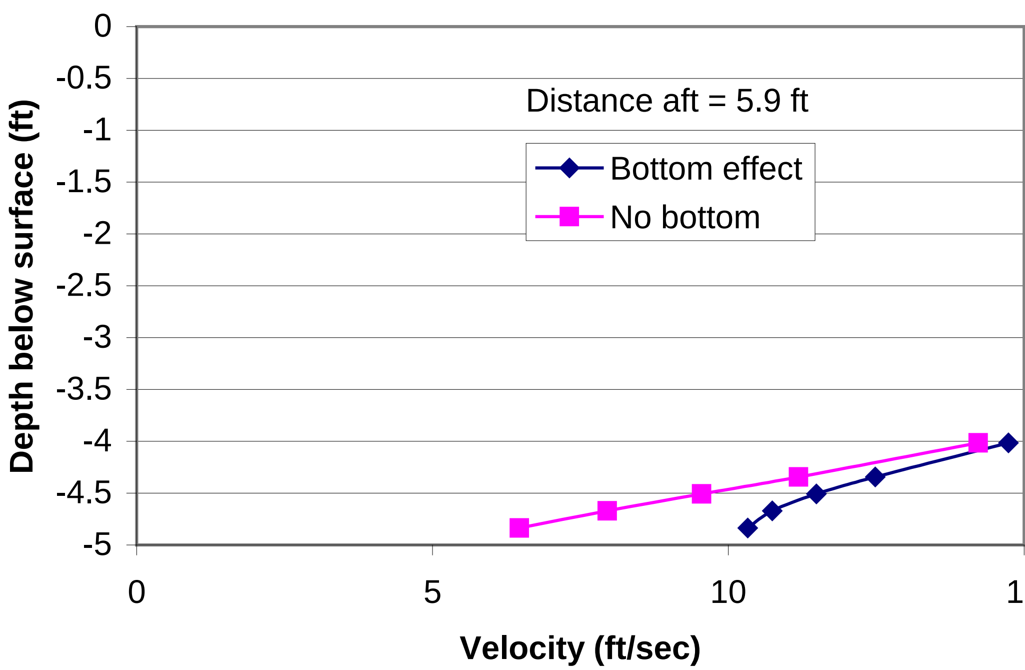

Figure 4. Example of JETWASH computational test, Monterey boat in 5-ft depth

Based on the computational test, it is concluded that because the JETWASH model incorporates enough conservative assumptions, the calculated bottom velocity is higher than that for the case of a non-developed boundary layer. Based on these computational tests and the JETWASH algorithms that inherently increase near-bottom velocities to account for flux balance, it can be concluded that the JETWASH assumptions will always provide conservative bottom shear stress during both developing and developed boundary layer calculations. Therefore, the JETWASH model is considered appropriately conservative for the Fox River propwash analysis without modifications.

REFERENCE

Middleton, Gerard V. and Southard, John B. 1984. Mechanics of Sediment Movement. S.E.P.M. Short Course No. 3, 2nd Edition. SEPM Tulsa, OK 74159-0756

APPENDIX E
CONTAINANT MOBILITY MODELING

TABLE OF CONTENTS

List of Tables

List of Figures

Attachments

ACRONYMS AND ABBREVIATIONS

μg/kg	micrograms/kilograms
C_0	porewater concentration
CAP	Cleanup Action Plan
cm	centimeter
cm^2/sec	Centimeters squared per second
cm/yr	centimeters per year
D/F	dioxin/furan
DMMP	Dredged Material Management Program
Ecology	Washington State Department of Ecology
g/cm^3	grams per centimeters cubed
L/kg	liters per kilogram
mg/kg	milligrams per kilogram
mg/L	milligrams per liter
ng/kg	nanograms per kilogram
OC	organic carbon
Pb	Lead
ppt	parts per trillion
RI/FS	Remedial Investigation/Feasibility Study
SMS	Sediment Management Standards
SQS	Sediment Quality Standard
TEQ	toxicity equivalents quotient
TOC	total organic carbons
USACE	U.S. Army Corps of Engineers
USEPA	U.S. Environmental Protection Agency
USGS	U.S. Geological Survey
Waterway	Whatcom Waterway

1 INTRODUCTION

This appendix presents the analysis of contaminant mobility that was performed in support of sediment cap design for the Inner Whatcom Waterway (Waterway) areas (sediment site Units 2A and 3B and portions of 2C). The cap modeling analysis focused on evaluating potential contaminant mobility through sediment caps over long-term steady-state conditions. This analysis was performed to verify that the caps will be effective over the long-term.

This appendix does not include additional contaminant mobility evaluations for the Log Pond. Contaminant mobility within the Log Pond areas was previously evaluated (Anchor Environmental 2001a) prior to implementation of the Interim Action in that area. Subsequent empirical monitoring of cap pore-water (Anchor Environmental 2001b, Anchor Environmental 2002, and RETEC 2006) has demonstrated that the cap in that area remains protective with respect to contaminant mobility. Additionally, groundwater fate and transport modeling was completed as part of the remedial investigation of the adjacent GP West site. That work included evaluating the beneficial impact of additional cap material placement on the fate and transport of mercury in nearshore groundwater. Results of that testing are described separately in the *Shoreline Groundwater Modeling Assessment* (Aspect 2012), included as Appendix J to this EDR. Findings of that study confirmed that the existing cap effectiveness would be enhanced by the placement of additional cap materials in nearshore areas.

The implementation of Log Pond contingency actions in that area includes measures to resist physical cap disturbance from wind/wave erosion. Areas of the cap that included surface recontamination will be capped with a minimum thickness of two feet of clean sand, plus additional material to address potential wind/wave erosion.

1.1 Methodology

Engineered sediment isolation capping (capping) is a remedial technology for containing contaminants in sediments and preventing or reducing the potential exposure and mobility of those contaminants from the sediment. Capping involves placing a subaqueous covering, or cap, of suitable material over contaminated sediment that remains in place. Caps are one of the most commonly evaluated and implemented remedial technologies for contaminated sediments (EPA 2005a; Palermo et al. 1998a); their effectiveness as a remedial option has been demonstrated by numerous successful projects.

The engineering basis for sediment isolation cap design is unique for each application and depends on site-specific conditions and project objectives. A one-dimensional (i.e., vertical) contaminant transport model is typically used to evaluate the long-term performance of an isolation sediment cap. Contaminant transport through a cap is driven by advective or diffusive forces. While the amount of advection varies according to the presence (or lack) of groundwater upwelling, diffusion is an ever-present condition driven by concentration gradients. These models can be used to conservatively estimate the thickness and type of material (e.g., permeability and organic carbon content) that would effectively reduce the flux of contaminants from the underlying sediments (Palermo et al. 1998a).

The cap modeling analysis for the Inner Waterway was conducted using conservative assumptions to assess whether capping would be an effective technology from a contaminant mobility perspective. A series of calculations were performed to evaluate the characteristics of the chemical isolation component of a subaqueous cap necessary to appropriately contain mercury in areas identified for in situ capping. Based on the presence of dioxin/furans (D/Fs) in subsurface sediments throughout Bellingham Bay from various sources, the protectiveness of the cap for mercury was also evaluated for performance with these compounds.

The mobility evaluations were performed using a minimum cap design thickness of 2 feet. The 2-foot design thickness was evaluated under long-term steady-state conditions to assess potential concentrations of mercury that could come to reside in the cap surface once steady state was achieved. This evaluation was also conducted to evaluate the potential fate of D/Fs under the same cap conditions.

Cap modeling was conducted under four different scenarios as summarized in Table 1-1. The four modeled scenarios (i.e., Scenarios A through D) assessed the protectiveness of the cap under a range of conditions expected to occur at the Site.

- Scenario A – "Base Case": This scenario simulated a typical cap section in open-water areas, with a typical consolidation.

- Scenario B – "Increased Consolidation": This scenario evaluated protectiveness of the design cap for areas with a higher level of sediment consolidation. This scenario addresses expected conditions in areas of especially soft sediment, or areas where a thick armor sequence is to be applied over the cap isolation layer.
- Scenario C – "High Groundwater Flux": This scenario evaluates the sensitivity of the model to increases in groundwater flux through the capping area. During the proposed Project, some decreases in groundwater flux are anticipated in Central Waterfront shoreline areas immediately offshore of areas where containment structures are to be installed. However, in other areas the groundwater flux could increase due to changes proposed within the site (e.g., higher flux could occur in areas adjacent to installed containment wall structures).
- Scenario D – "High Dioxin/Furan": In one area along the south shoreline, subsurface soils in nearshore areas were found to contain elevated concentrations of dioxin/furan compounds. A test scenario was included to evaluate the impact of these deep soil concentrations on the potential protectiveness of the subtidal sediment caps.

The cap modeling in each of the above four scenarios focused on the following:

- Sediment caps within the Inner Waterway (sediment site Units 2A and 3B and portions of 2C) and the South Shoreline
- Protectiveness of the cap design for mercury, as the primary contaminant of concern within the Whatcom Waterway site
- Protectiveness of the cap design with D/F compounds which are also present in sediments within Bellingham Bay
- Physical isolation of the contaminated sediment from the aquatic environment
- Reduction or elimination of the flux of dissolved contamination into the upper layers of the cap such that cap performance criteria are not exceeded

Table 1-1
Mobility Evaluation Scenarios

Scenario	Scenario Description	Purpose
A	Base Case	Evaluate a typical cap for open-water areas
B	Increased Consolidation	Test effects of higher consolidation in thicker armor areas or in areas of especially soft sediment
C	High Groundwater Flux	Evaluate sensitivity of results to changes in groundwater flux
D	High Dioxin/Furan	Evaluate the impact of elevated dioxin/furan concentrations present in localized nearshore on capping protectiveness

1.2 Document Organization

This appendix is divided into the following three sections:

- Section 2 describes the performance criteria used in the capping analysis
- Section 3 describes the capping analysis
- Section 4 describes the capping analysis results

2 PERFORMANCE CRITERIA

The U.S. Environmental Protection Agency's (USEPA's) *Contaminated Sediment Remediation Guidance for Hazardous Waste Sites* (USEPA 2005a) confirms that capping can provide short-term and long-term effectiveness and permanence. Caps can be designed to ensure their stability and ability to control contaminant migration such that the long-term effectiveness and permanence of capping alternatives can be similar to that provided by other alternatives, including removal.

Detailed guidance has been developed by the U.S. Army Corps of Engineers (USACE) and the EPA for in situ capping as a remedial alternative for contaminated sediments. The documents *Contaminated Sediment Remediation Guidance for Hazardous Waste Sites* (EPA 2005a), *Guidance for Subaqueous Dredged Material Capping* (Palermo et al. 1998a), and *Assessment and Remediation of Contaminated Sediments (ARCS) Program Guidance for In Situ Subaqueous Capping of Contaminated Sediments* (Palermo et al. 1998b)) provide detailed procedures for site and sediment characterization, cap design, cap placement operations, and cap monitoring. These guidance documents, in particular Appendix B of Palermo et al. 1998a, provide the technical basis for the design of the chemical isolation layer presented in the following sections.

As described in the *Guidance for In Situ Subaqueous Capping of Contaminated Sediments* (Palermo et al. 1998a):

> *"If a cap has a properly designed physical isolation component, contaminant migration associated with the movement of sediment particles should be controlled. However, the vertical movement of dissolved contaminants by advection (flow of ground water or pore water) through the cap is possible, while some movement of contaminants by molecular diffusion (movement across a concentration gradient) over long periods usually is inevitable. However, in assessing these processes, it is important to also assess the sorptive capacity of the cap material, which will act to retard contaminant flux through the cap, and the long-term fate of capped contaminants that may transform through time. Slow releases of dissolved contaminants through a cap at low levels will generally not create unacceptable exposures. If reduction of*

> *contaminant flux is necessary to meet remedial action objectives, however, a more involved analysis to include capping effectiveness testing and modeling should be conducted as a part of cap design*."

To evaluate the potential effectiveness of sediment caps for controlling contaminant migration from the sediments, the maximum concentrations of representative contaminants in the inner waterway relative to a cap were evaluated. The representative contaminants evaluated in this analysis include:

- Mercury
- D/Fs congeners and D/Fs toxic equivalents quotient (TEQ)

Sediment cleanup levels for mercury were defined in the Whatcom Waterway Cleanup Action Plan (CAP) (Ecology 2007). The mercury cleanup level was established at the Sediment Quality Standard (SQS) consistent with Washington State Department of Ecology's (Ecology's) Sediment Management Standards (SMS) regulations. The SQS for mercury is 0.41 mg/kg. The cleanup level applies to the bioactive zone, which in Bellingham Bay typically represents the top 12 centimeters (cm) of sediment.

D/F compounds are present in surface and subsurface sediments within most of Puget Sound's urban bays. These compounds are derived from multiple historical and ongoing sources. Ecology has conducted sampling and issued a final data report documenting a regional background concentration of D/Fs in Bellingham Bay of 15 ng/kg (Ecology 2015).

3 CONTAMINANT MOBILITY ANALYSIS

This section describes the cap model and the model inputs used to assess the chemical isolation effectiveness. The analytical steady-state model used in this analysis was developed on behalf of the USACE and has been published in peer-reviewed journals and publications such as the *Journal of Soil and Sediment Contamination* (Lampert and Reible 2009) and *Guidance for In Situ Subaqueous Capping of Contaminated Sediments* (Palermo et al. 1998a).

As discussed in Section 1, the primary objective of a cap is physical and chemical containment of the underlying contaminated sediments. The chemical isolation component of the cap should therefore control the movement of contaminants by dissolved phase advection and diffusion. Diffusion is a very slow process in which ionic and molecular species in water are transported by random molecular motion across a concentration gradient. Advection refers to the flow of sediment porewater or underlying groundwater resulting from consolidation of the contaminated sediment layer due to cap placement or upward flow of groundwater. Advection transports dissolved contaminants and colloidally bound fractions (e.g., complexed with dissolved organic matter).

3.1 Model Description

This assessment used an analytical steady-state approach for modeling contaminant concentrations and fluxes in sediment caps, which was developed by Dr. Reible from the University of Texas at Austin (Lampert and Reible 2009). Steady-state predictions are useful for assessing long-term contaminant profiles within a subaqueous cap, although the time to reach the steady-state concentrations predicted by the model will vary depending on the chemical characteristics of the contaminant, sediment geochemical conditions, and subsurface hydrology.

The model makes the conservative assumption that the underlying sediment concentration remains constant over time (i.e., that the capped sediment represents a continuing subsurface source). The model incorporates the traditional groundwater transport mechanisms of advection, diffusion, dispersion, partitioning between the aqueous and sorbed (sediment or cap material) phases. As applied herein, the model assumes that contaminant flux is not affected by biodegradation or geochemical processes, other than adsorption (the model can simulate biodegradation, but this degradation rate was assumed to be zero for this conservative evaluation). The model accounts for mass transfer processes at the sediment-water interface, including biological mixing.

The analytical steady-state model has been used to support the evaluation and design of sediment caps at numerous sites around the United States, including Commencement Bay, the Tittabawassee River (Tittabawassee & Saginaw River Team 2011) and Onondaga Lake, New York (Parsons and Anchor QEA 2011). Details on the model structure and underlying theory and equations are provided in Lampert and Reible (2009). The spreadsheet version of the model is publicly available and can be found at http://www.ce.utexas.edu/reiblegroup/downloads.html.

3.2 Model Inputs

The steady-state model includes a number of parameters that describe the properties of the chemical isolation cap material and chemical mass transfer rates associated with processes such as bioturbation, groundwater flow, and sediment deposition. Reible's steady-state model estimates the chemical concentrations in the surficial (bioturbation) sediment layers of a cap once steady-state conditions are achieved in the cap. As the dissolved contaminants move upward through the cap, they are predicted to undergo biodegradation while at the same time partitioning onto the cap material. Bioturbation mixes the surface layer, further reducing concentrations.

The model calculates the chemical concentrations in the bioturbation layer as a balance between the flux from the underlying chemical isolation layer, the flux leaving the bioturbation layer, and the benthic boundary layer in the overlying water column. To calculate the overall fluxes noted above, the model requires the input values defined in Table 3-1.

Table 3-1
Model Parameter Inputs

Symbol	Scenario A Value(s)	Scenario B Value(s)	Scenario C Value(s)	Scenario D Value(s)	Units	Comments
Contaminant Properties						
$LogK_d/LogK_{oc}$[1]	See Table 3-2				L/kg-OC	Equilibrium partitioning coefficient/OC partitioning coefficient
Dw	See Table 3-2				cm^2/sec	Water diffusivity
λ_1	See Table 3-2				$year^{-1}$	Cap decay rate
λ_2	See Table 3-2				$year^{-1}$	Bioturbation layer decay rate
Sediment Properties						
C_o[1]	See Table 3-3			See Table 3-4	mg/L	Porewater chemical concentration of underlying sediments
$(f_{oc})_{bio}$	0				mg/L	Biological active zone fraction OC
ρ_{DOC}	0				mg/L	Colloidal OC concentration
V	65.8	65.8	131.6	32.9	cm/yr	Darcy velocity
V_{dep}	0				cm/yr	Depositional velocity
h_{bio}	12				cm	Bioturbation layer thickness
D_{bio}^{pw}	100				cm^2/sec	Porewater biodiffusion coefficient, Dbiopw
D_{bio}^{p}	1				cm^2/sec	Particle Biodiffusion Coefficient Dbiop
Cap Properties						
--	60.96				cm	Cap placed depth conventional cap placed depth
G or C	G				NA	Cap Materials -Granular (G) or Consolidated Silty/Clay (C)
--	0 0				cm	Cap consolidation depth
--	15.24	30.48	15.24	15.24	cm	Consolidation of Underlying sediment

Symbol	Scenario A Value(s)	Scenario B Value(s)	Scenario C Value(s)	Scenario D Value(s)	Units	Comments
ε	0.4				--	Porosity of cap sediments; 0.4 is a typical value for sand, based on past experience
P_b	2.6				g/cm^3	Particle density of cap sediments
$f_{oc\ eff}$	0.0006				fraction	TOC of cap material (typical value shown for quarry sand)
z	6				cm	Depth of interest
$f_{oc\ (z)}$	2.95				%	Fraction organic carbon at depth of interest

Notes:
1. Chemical-specific parameter; see Table 3-2 for K_d, K_{oc}, water diffusivity, and λ, and Tables 3-3 and 3-4 for C_o.

cm = centimeter
cm^2/sec = centimeters squared per second
cm/hr = centimeters per hour
cm/yr = centimeters per year
C_o = porewater concentration
g/cm^3 = grams per cubic centimeter
L/kg = liters per kilogram
mg/L = milligrams per liter
OC = organic carbon
TOC = Total organic carbon

Based on Lampert and Reible (2009), the steady-state model is most sensitive to:

- Groundwater velocity
- Partitioning coefficient
- Biodegradation rates, where applicable (these are assumed to be zero in the current simulations)
- Concentration of contaminant in sediments underlying the cap

The current evaluation used conservative assumptions to assess whether capping would be an effective technology for the site. The analytical steady-state model described in Lampert and Reible (2009) was used in the analysis. It was assumed that the cap chemical isolation material comprised a granular material, such as sand, with minimal organic carbon. The steady-state model was used to evaluate capping performance for worst-case conditions within the capping area, using the highest sediment concentrations measured in surface or shallow subsurface sediments. This analysis was performed for each of the contaminants (described in Section 2.1) under a 2-foot-thick chemical isolation layer.

Table 3-2 lists the chemical-specific model input parameters such as partitioning coefficients and biodegradation rates.

Table 3-2
Chemical-specific Model Inputs

Chemical	$Log_{10}K_d$ and $Log_{10}K_{oc}$	Water Diffusivity (cm^2/year)	Assumed Anaerobic Biodegradation Rate (per year)	Assumed Aerobic Biodegradation Rate (per year)
Mercury	3.8	0.0000068	Zero	Zero
2,3,7,8-TCDD	6.68	0.0000055	Zero	Zero
1,2,3,7,8-PeCDD	6.53	0.0000052	Zero	Zero
1,2,3,4,7,8-HxCDD	7.67	0.0000050	Zero	Zero
1,2,3,6,7,8-HxCDD	7.18	0.0000050	Zero	Zero
1,2,3,7,8,9-HxCDD	7.18	0.0000050	Zero	Zero
1,2,3,4,6,7,8-HpCDD	7.86	0.0000048	Zero	Zero
OCDD	8.06	0.0000046	Zero	Zero
2,3,7,8-TCDF	6.00	0.0000056	Zero	Zero
1,2,3,7,8-PeCDF	6.67	0.0000053	Zero	Zero
2,3,4,7,8-PeCDF	6.39	0.0000053	Zero	Zero
1,2,3,4,7,8-HxCDF	6.88	0.0000051	Zero	Zero
1,2,3,6,7,8-HxCDF	6.88	0.0000051	Zero	Zero
2,3,4,6,7,8-HxCDF	6.88	0.0000051	Zero	Zero
1,2,3,7,8,9-HxCDF	6.88	0.0000051	Zero	Zero
1,2,3,4,6,7,8-HpCDF	7.27	0.0000049	Zero	Zero
1,2,3,4,7,8,9-HpCDF	7.27	0.0000049	Zero	Zero
OCDF	7.86	0.0000047	Zero	Zero

Partitioning of contaminants between the aqueous and sorbed (i.e., sediment) phases is described in the model using chemical-specific equilibrium partition coefficients (K_d). For organic compounds, the partition coefficient is expressed on an organic carbon basis (K_{oc}) and is used in conjunction with $f_{oc\,(z)}$ to calculate K_d. Selection of the K_d value for mercury was based on individual site-specific K_d estimates (Aspect and Anchor QEA 2011). A summary of the data used to derive this value is attached as Attachment 1. Organic carbon partitioning coefficient (K_{oc}) values for D/Fs for use in the model were based on a review of literature sources. A summary of this literature review is provided as Attachment 2.

Mercury and D/Fs were both assumed to be subject to no biodegradation or geochemical immobilization processes, though these assumptions are likely conservative. Additionally, the cap evaluation was conducted without accounting for natural recovery mixing and burial processes, though these are known to occur within Bellingham Bay.

The following is a discussion of each of the model parameter inputs.

- **Cap Thicknesses:** Reasonable assumptions were made about the potential cap thickness in Units 2A, 3B, and a portion of 2C. The modeled minimum cap thickness is 2 feet; this conservatively does not include any over-placement of cap material (typically 6 to 12 inches) thickness. Nor does it consider the placement of additional armoring materials over the cap, though these are to be provided in all cap areas. The cap consolidation depth is assumed to be zero from past experience because the cap materials consolidate as they are placed during construction and the minimum placement thickness is met (i.e., 2 feet).
- **Maximum Chemical Concentrations:** Historical site data were consulted to determine appropriate conservative input sediment chemical concentrations for each COC (Anchor QEA 2003; Anchor QEA 2009; Anchor and Hart Crowser 2000; RETEC 2006).
  - The maximum concentration for each COC from all sediment datasets was chosen as input to the model (Table 3-3).
  - The South Shoreline upland soil D/F data is shown in Table 3-4.
- **Isolation Sand Properties:** A porosity of 0.4 total organic carbon (TOC) of less than 0.1 percent, and particle density of 2.6 grams per centimeters cubed (g/cm^3) were used

as typical values for granular cap sand and blended cap sand and gravel mix. The specific TOC used in the analysis was 0.06 percent, based on data from source quarries in the Pacific Northwest.

- **Infinite Source Assumption:** The concentration of the contaminant within the underlying sediment is conservatively assumed to maintain its starting porewater concentration (C_0) for all time without degradation or depletion due to transport into the cap.
- **Darcy Velocities:** Estimates of Darcy velocities (from which seepage velocities were calculated) for the inner waterway were obtained from the results of an extensive MODFLOW modeling effort developed as part of the Remedial Investigation/Feasibility Study (RI/FS) for the Central Waterfront project. (MODFLOW is a three-dimensional finite-difference groundwater flow model developed by the U.S. Geological Survey [USGS].) This modeling incorporated data from groundwater investigations conducted throughout the area.
  - Darcy velocity was approximated by assuming the most conservative (i.e., maximum) value obtained from the areas that were covered by the modeling (65.8 centimeters per year [cm/yr]); the resulting Darcy velocity used in the model was 65.8 cm/yr. This value was used in the base case cap (Scenario A) and the high consolidation scenario (Scenario B).
  - Scenario C assumes a high darcy velocity (131.6 cm/yr) at two times the maximum value noted above to account for higher groundwater flow due to proposed changes to shoreline areas where sheetpile containment walls may be installed.
  - Scenario D uses a darcy velocity of 32.9 cm/yr to account for the presence of a clay soil horizon approximately 15 to 20 feet below ground surface within the GP West site. This clay layer tends to reduce groundwater flux through subtidal sediments in this area.
- **Total Organic Carbon:** The average TOC for sediments for the inner waterway was found to be 2.95 percent
- **Biodegradation Rates:** As is the case with all metals, the biodegradation of mercury was assumed to be zero. Biodegradation of dioxins/furans was also conservatively assumed to be zero

- **Depositional Velocity:** Deposition of cleaner sediments on the cap was conservatively assumed to be zero. This assumption is conservative because contaminant concentrations in suspended sediments at the site are generally below any of the applicable sediment criteria; thus, any permanent deposition of clean sediments will tend to increase the effectiveness of a cap.
- **Partitioning Coefficients:** A site-specific K_d value for mercury was calculated by pairing historic bulk sediment data with collocated porewater data. The resulting K_d value of 3.8 is more conservative than the value given in the EPA's *Partition Coefficients for Metals in Surface Water, Soil, and Waste* (EPA 2005b). A comprehensive review of D/Fs K_{oc} values cited in the literature was conducted. K_{oc} values used in the cap modeling were derived from K_{ow} values sourced from a draft document by the EPA entitled *Exposure and Human Health Reassessment of 2,3,7,8-Tetrachlorodibenzo-p-Dioxin (TCDD) and Related Compounds National Academy Sciences (NAS) Review*, as part of the ongoing Dioxin Reassessment being conducted by the EPA. K_{ow} values were used to calculate corresponding K_{oc} values using the empirically derived relationship between K_{oc} and K_{ow} derived by DiToro (1985). Using these partitioning coefficients, and assuming equilibrium partitioning conditions, the maximum sediment concentrations were converted into porewater concentrations.
- **Colloidal Organic Carbon Concentration:** The dissolved organic carbon in sediment and cap interstitial waters was assumed to be zero.
- **Porewater Biodiffusion Coefficient:** The effective diffusion coefficient in biologically active layer based on interstitial water. There is very little guidance for this parameter although measurements have shown 10^{-3}-10^{-5} cm^2/sec as reasonable estimates.
- **Particle Biodiffusion Coefficient:** Effective particle diffusion coefficient in biological active layer and typical values are between 1 and 9 for freshwater and estuarine systems.
- **Underlying Sediment Consolidation:** This indicates the total volume of porewater expressed into the cap layer. The migration of a contaminant expressed with this porewater may be considerably less than the total consolidation due to sorption-related retardation in the cap material.

- Six inches of consolidation was used for capping Scenarios A, C, and D.
- Twelve inches of consolidation was used under Scenario B to simulate potential consolidation in areas of especially soft sediment or in areas where a thickner armor sequence is to be installed above the cap.

- **Depth of Interest:** This is set as the center of the bioactive zone (6 cm) for sediment concentration estimates and below the cap surface (1 cm) for porewater concentration estimates.
- **Water Diffusivity:** Diffusivity of the pure contaminant in water based on molecular weight of each compound.

Table 3-3

Scenarios A, B, and C: Initial Maximum Bulk Sediment Concentrations for Chemical Isolation Layer Modeling and Calculated Initial Porewater Concentrations

Chemical of Concern	Maximum Detected Concentration of Underlying Bulk Sediment (mg/kg, µg/kg)[1]	Maximum Estimated Concentration in Underlying Sediment Porewater C_o (µg/L, pg/L)
Mercury	4.3 mg/kg dry wt	0.68 µg/L
Dioxins		
2,3,7,8-TCDD	0.005 µg/kg dry wt	0.038 pg/L
1,2,3,7,8-PeCDD	0.006 µg/kg dry wt	0.061 pg/L
1,2,3,4,7,8-HxCDD	0.011 µg/kg dry wt	0.772 pg/L
1,2,3,6,7,8-HxCDD	0.145 µg/kg dry wt	0.328 pg/L
1,2,3,7,8,9-HxCDD	0.029 µg/kg dry wt	0.066 pg/L
1,2,3,4,6,7,8-HpCDD	3.52 µg/kg dry wt	1.63 pg/L
OCDD	35.6 µg/kg dry wt	10.5 pg/L
Furans		
2,3,7,8-TCDF	0.040 µg/kg dry wt	1.36 pg/L
1,2,3,7,8-PeCDF	0.051 µg/kg dry wt	0.367 pg/L
2,3,4,7,8-PeCDF	0.076 µg/kg dry wt	1.05 pg/L
1,2,3,4,7,8-HxCDF	0.147 µg/kg dry wt	0.655 pg/L
1,2,3,6,7,8-HxCDF	0.035 µg/kg dry wt	0.157 pg/L

Chemical of Concern	Maximum Detected Concentration of Underlying Bulk Sediment (mg/kg, µg/kg)[1]	Maximum Estimated Concentration in Underlying Sediment Porewater C_o (µg/L, pg/L)
2,3,4,6,7,8-HxCDF	0.038 µg/kg dry wt	0.170 pg/L
1,2,3,7,8,9-HxCDF	0.032 µg/kg dry wt	0.141 pg/L
1,2,3,4,6,7,8-HpCDF	0.715 µg/kg dry wt	1.29 pg/L
1,2,3,4,7,8,9-HpCDF	0.054 µg/kg dry wt	0.098 pg/L
OCDF	2.27 µg/kg dry wt	1.05 pg/L

Notes:

1. D/Fs congener concentrations from maximum D/Fs bulk sediment concentration with a sum TEQ equal to 138 ng/kg TEQ.

µg/kg = micrograms per kilogram
µg/L = micrograms per liter
C_0 = porewater concentration
D/F = dioxin/furan
mg/kg = milligrams per kilogram
ng/kg = nanograms per kilogram
pg/L = picograms per liter
TEQ = toxicity equivalent quotient

For the sediment-based criteria, the model was used to first determine the maximum porewater concentration and then determine the equivalent bulk sediment concentration using partitioning. D/F congeners were modeled individually and then summed to calculate a total D/F TEQ value within the sediment bioactive zone.

Table 3-4

Scenario D: Dioxin/Furan Initial Maximum Bulk Sediment Concentrations for Chemical Isolation Layer Modeling and Calculated Initial Porewater Concentrations

Chemical of Concern	Maximum Detected Concentration of Underlying Bulk Sediment (µg/kg)[1]	Maximum Estimated Concentration in Underlying Sediment Porewater C_o (pg/L)
Dioxins		
2,3,7,8-TCDD	0.002 µg/kg dry wt	0.016 pg/L
1,2,3,7,8-PeCDD	0.290 µg/kg dry wt	2.98 pg/L

Chemical of Concern	Maximum Detected Concentration of Underlying Bulk Sediment (μg/kg)[1]	Maximum Estimated Concentration in Underlying Sediment Porewater C_o (pg/L)
1,2,3,4,7,8-HxCDD	2.050 μg/kg dry wt	1.49 pg/L
1,2,3,6,7,8-HxCDD	1.500 μg/kg dry wt	3.39 pg/L
1,2,3,7,8,9-HxCDD	1.315 μg/kg dry wt	2.97 pg/L
1,2,3,4,6,7,8-HpCDD	10.250 μg/kg dry wt	4.75 pg/L
OCDD	7.600 μg/kg dry wt	2.24 pg/L
Furans		
2,3,7,8-TCDF	0.058 μg/kg dry wt	1.98 pg/L
1,2,3,7,8-PeCDF	0.052 μg/kg dry wt	0.369 pg/L
2,3,4,7,8-PeCDF	0.056 μg/kg dry wt	0.774 pg/L
1,2,3,4,7,8-HxCDF	0.081 μg/kg dry wt	0.359 pg/L
1,2,3,6,7,8-HxCDF	0.039 μg/kg dry wt	0.174 pg/L
2,3,4,6,7,8-HxCDF	0.002 μg/kg dry wt	0.009 pg/L
1,2,3,7,8,9-HxCDF	0.026 μg/kg dry wt	0.116 pg/L
1,2,3,4,6,7,8-HpCDF	0.155 μg/kg dry wt	0.279 pg/L
1,2,3,4,7,8,9-HpCDF	0.018 μg/kg dry wt	0.032 pg/L
OCDF	0.390 μg/kg dry wt	0.181 pg/L

Notes:

1. = D/Fs congener concentrations from maximum D/Fs bulk sediment concentration (average of BH-SB02 4 to 8 feet interval and field duplicate) with a sum TEQ equal to 924 ng/kg TEQ.

μg/kg = micrograms per kilogram

ng/kg = nanograms per kilogram

C_0 = porewater concentration

D/F = dioxin/furan

pg/L = picograms per liter

TEQ = toxicity equivalents quotient

4 RESULTS AND DISCUSSION

Tables 4-1 through 4-4 presents the results for cap Scenarios A through D of the maximum concentration estimates in surface sediment in the waterway and for nearby upland soil D/F concentrations using the conservative assumptions of this analysis using the steady-state cap model at a depth of interest at the center of the bioactive zone, and the maximum concentration estimates of porewater at 1 cm below the cap surface (i.e., near the cap-surface water interface). This is a steady-state analysis in which the time to steady-state is very long (thousands of years). In addition, the modeling conservatively assumes no deposition of sediment on top of the cap.

The cap design analysis, using conservative modeling assumptions (i.e., no new sediment deposition, no included over-placement allowance thickness, infinite source assumption, biodegradation rates, and partitioning coefficients), indicates that a 2-foot-thick layer of sand, as part of an overall engineered cap design, including armor as necessary, would provide an appropriate level of contaminant isolation for a range of scenarios:

- Scenario A: The base case cap scenario is expected to comply with the mercury SQS (0.41 mg/kg), with a significant margin of safety. The base case cap would also be effective at isolating D/F compounds, with the estimated concentrations of these compounds at long-term steady-state conditions estimated to be less than typical background concentrations of these compounds within Puget Sound.
- Scenario B: The high consolidation scenario evaluates the protectiveness of the capping in areas where consolidation may be higher than that simulated in Scenario A. These include areas with especially soft sediments and areas where a thicker armor layer is to be applied over the top of the cap isolation layer. The model concentration estimates for this scenario are similarly protective as those of Scenario A.
- Scenario C: The high groundwater flux scenario evaluates a higher darcy velocity relative to the base cap by a factor of two for areas where groundwater flux may increase due to shoreline sheetpile wall installation in nearby areas, and resultant groundwater diversion through the cap placement areas. The model results show that the Scenario C cap is expected to comply with the mercury SQS (0.41 mg/kg) with a significant margin of safety. This scenario would also be effective at isolating D/F

compounds, with the estimated concentrations of these compounds at long-term, steady-state conditions estimated to be less than typical background concentrations of these compounds within Puget Sound

- Scenario D: The south shoreline cap scenario uses the base case model inputs and an area specific darcy velocity coupled with high D/F upland soil concentrations and the model predicts the sediment D/F TEQ may marginally exceed the natural background by a factor of 1.3. This finding is not considered significant, as the results would not result in area-wide changes to average sediment quality throughout the Project area. As the 2-foot cap thickness for range of scenarios was found to be protective, thicker cap sequences were not evaluated. As described in the EDR, the final achieved cap thicknesses achieved during construction will be greater than 2 feet due to cap material over-placement, and due to the thicknesses of cap armoring that will be placed over the cap to provide protection against wind/wave erosion and propwash.

Table 4-1
Scenario A (Base Case):
Summary of Chemical Isolation Layer Modeling Results

Chemical of Potential Concern	Sediment Concentration at Steady State (Within Bioactive Zone) W_{bio}	Porewater Concentration at Steady State C_{bio} (Top of Bioactive Zone)	Time to Reach Steady State (years)
Mercury	0.21 mg/kg	0.01 μg/L	4,460
Dioxins			
2,3,7,8-TCDD	0.0642 ng/kg	3.94E-04 pg/L	2,132
1,2,3,7,8-PeCDD	0.076 ng/kg2	–	1,496
1,2,3,4,7,8-HxCDD	0.0107 ng/kg	–	20,793
1,2,3,6,7,8-HxCDD	0.153 ng/kg	–	6,705
1,2,3,7,8,9-HxCDD	0.0306 ng/kg	–	6,705
1,2,3,4,6,7,8-HpCDD	0.354 ng/kg	–	32,885
OCDD	0.107 ng/kg	–	51,977
Furans			
2,3,7,8-TCDF	0.0734 ng/kg	–	436
1,2,3,7,8-PeCDF	0.0180 ng/kg	–	2,093

Chemical of Potential Concern	Sediment Concentration at Steady State (Within Bioactive Zone) W_{bio}	Porewater Concentration at Steady State C_{bio} (Top of Bioactive Zone)	Time to Reach Steady State (years)
2,3,4,7,8-PeCDF	0.307 ng/kg	–	1,086
1,2,3,4,7,8-HxCDF	0.163 ng/kg	–	3,390
1,2,3,6,7,8-HxCDF	0.0391 ng/kg	–	3,390
2,3,4,6,7,8-HxCDF	0.0423 ng/kg	–	3,390
1,2,3,7,8,9-HxCDF	0.0351 ng/kg	–	3,390
1,2,3,4,6,7,8-HpCDF	0.0744 ng/kg	–	8,435
1,2,3,4,7,8,9-HpCDF	0.00564 ng/kg	–	8,435
OCDF	0.00686 ng/kg	–	32,977
D/F TEQ	1.56 (ng/kg)	–	--

Notes:

W_{bio} = Sediment Concentration at Steady State

C_{bio} = Porewater Concentration at Steady State

mg/kg = milligrams per kilogram

ng/kg = nanograms per kilogram

µg/L = micrograms per liter

pg/L = picograms per liter

Table 4-2
Scenario B (High Consolidation): Summary of Chemical Isolation Layer Modeling Results

Chemical of Potential Concern	Sediment Concentration at Steady State (Within Bioactive Zone) W_{bio}	Porewater Concentration at Steady State C_{bio} (Top of Bioactive Zone)	Time to Reach Steady State (years)
Mercury	0.21 mg/kg	0.01 µg/L	4,459
Dioxins			
2,3,7,8-TCDD	0.0642 ng/kg	3.94E-04 pg/L	2132
1,2,3,7,8-PeCDD	0.076 ng/kg	–	1495
1,2,3,4,7,8-HxCDD	0.0107 ng/kg	–	20792
1,2,3,6,7,8-HxCDD	0.153 ng/kg	–	6705
1,2,3,7,8,9-HxCDD	0.0306 ng/kg	–	6705
1,2,3,4,6,7,8-HpCDD	0.354 ng/kg	–	32884
OCDD	0.107 ng/kg	–	51976
Furans			
2,3,7,8-TCDF	0.0734 ng/kg	–	435
1,2,3,7,8-PeCDF	0.0180 ng/kg	–	2093
2,3,4,7,8-PeCDF	0.307 ng/kg	–	1086
1,2,3,4,7,8-HxCDF	0.163 ng/kg	–	3390
1,2,3,6,7,8-HxCDF	0.0391 ng/kg	–	3390
2,3,4,6,7,8-HxCDF	0.0423 ng/kg	–	3390
1,2,3,7,8,9-HxCDF	0.0351 ng/kg	–	3390
1,2,3,4,6,7,8-HpCDF	0.0744 ng/kg	–	8435
1,2,3,4,7,8,9-HpCDF	0.00564 ng/kg	–	8435
OCDF	0.00686 ng/kg	–	32977
D/F TEQ	1.56 (ng/kg)	–	--

Notes:
W_{bio} = Sediment Concentration at Steady State
C_{bio} = Porewater Concentration at Steady State
mg/kg = milligrams per kilogram
ng/kg = nanograms per kilogram
µg/L = micrograms per liter
pg/L = picograms per liter

Table 4-3
Scenario C (High Groundwater Flux): Summary of Chemical Isolation Layer Modeling Results

Chemical of Potential Concern	Sediment Concentration at Steady State (Within Bioactive Zone) W_{bio}	Porewater Concentration at Steady State C_{bio} (Top of Bioactive Zone)	Time to Reach Steady State (years)
Mercury	0.4 mg/kg	0.02 ug/L	2,466
Dioxins			
2,3,7,8-TCDD	0.1270 ng/kg	7.79E-04 pg/L	1159
1,2,3,7,8-PeCDD	0.1504 ng/kg	–	810
1,2,3,4,7,8-HxCDD	0.0213 ng/kg	–	11228
1,2,3,6,7,8-HxCDD	0.3021 ng/kg	–	3621
1,2,3,7,8,9-HxCDD	0.0606 ng/kg	–	3621
1,2,3,4,6,7,8-HpCDD	0.7018 ng/kg	–	17711
OCDD	0.2120 ng/kg	–	27928
Furans			
2,3,7,8-TCDF	0.1450 ng/kg	–	237
1,2,3,7,8-PeCDF	0.0357 ng/kg	–	1135
2,3,4,7,8-PeCDF	0.6074 ng/kg	–	589
1,2,3,4,7,8-HxCDF	0.3223 ng/kg	–	1833
1,2,3,6,7,8-HxCDF	0.0774 ng/kg	–	1833
2,3,4,6,7,8-HxCDF	0.0837 ng/kg	–	1833
1,2,3,7,8,9-HxCDF	0.0695 ng/kg	–	1833
1,2,3,4,6,7,8-HpCDF	0.1473 ng/kg	–	4548
1,2,3,4,7,8,9-HpCDF	0.0112 ng/kg	–	4548
OCDF	0.0136 ng/kg	–	17738
D/F TEQ	3.08 ng/kg	–	--

Notes:
W_{bio} = Sediment Concentration at Steady State
C_{bio} = Porewater Concentration at Steady State
mg/kg = milligrams per kilogram
ng/kg = nanograms per kilogram
μg/L = micrograms per liter
pg/L = picograms per liter

Table 4-4

Scenario D (High Dioxin/Furan): Summary of Chemical Isolation Layer Modeling Results

Chemical of Potential Concern	Sediment Concentration at Steady State (Within Bioactive Zone) W_{bio}	Porewater Concentration at Steady State C_{bio} (Top of Bioactive Zone)	Time to Reach Steady State (years)
Dioxins			
2,3,7,8-TCDD	0.0132 ng/kg	8.12E-05 pg/L	3677
1,2,3,7,8-PeCDD	1.8179	–	2593
1,2,3,4,7,8-HxCDD	1.0442	–	36219
1,2,3,6,7,8-HxCDD	0.7931	–	11680
1,2,3,7,8,9-HxCDD	0.6953	–	11680
1,2,3,4,6,7,8-HpCDD	0.5186	–	57527
OCDD	0.0115	–	91276
Furans			
2,3,7,8-TCDF	0.0538	–	749
1,2,3,7,8-PeCDF	0.0091	–	3620
2,3,4,7,8-PeCDF	0.1132	–	1878
1,2,3,4,7,8-HxCDF	0.0448	–	5893
1,2,3,6,7,8-HxCDF	0.0217	–	5893
2,3,4,6,7,8-HxCDF	0.0011	–	5893
1,2,3,7,8,9-HxCDF	0.0145	–	5893
1,2,3,4,6,7,8-HpCDF	0.0081	–	14727
1,2,3,4,7,8,9-HpCDF	0.0009	–	14727
OCDF	0.0006	–	57810
D/F TEQ	5.16 ng/kg	–	

Notes:

W_{bio} = Sediment Concentration at Steady State
C_{bio} = Porewater Concentration at Steady State
mg/kg = milligrams per kilogram
ng/kg = nanograms per kilogram
µg/L = micrograms per liter
pg/L = picograms per liter

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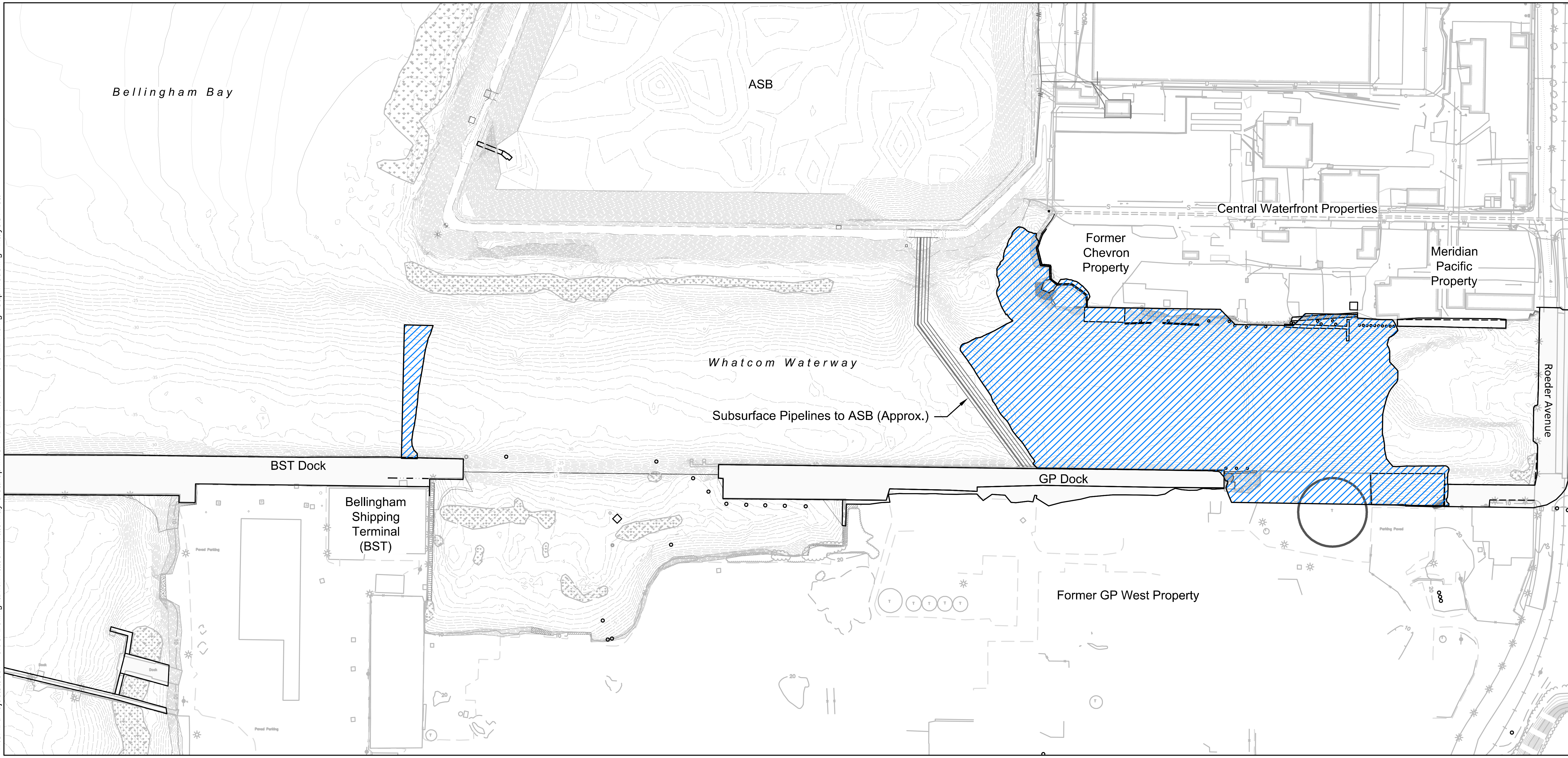
FIGURES



T:\CAD\Projects\0007-Port of Bellingham\Whatcom Waterway Cleanup Inner Wtr\EDR\007-RP-010-EDR.dwg Cap Modeling Analysis Area

Feb 06, 2015 3:18pm epipkin

SOURCE: Survey provided by Wilson Engineering.
HORIZONTAL DATUM: Washington State Plane North, NAD83, U.S. Feet.
VERTICAL DATUM: Mean Lower Low Water (MLLW).

LEGEND:

Cap Modeling Analysis Area

ANCHOR QEA

Figure 2-1
Cap Modeling Analysis Area
Appendix E - Contaminant Mobility Modeling
Whatcom Waterway Cleanup in Phase 1 Site Areas -Final EDR

ATTACHMENT 1
DERIVATION OF SITE-SPECIFIC MERCURY PARTITIONING COEFFICIENT

Aspect CONSULTING

Project No.: 070188-001-10

June 17, 2011

To: Brian Sato, Department of Ecology Northwest Regional Office

cc: Brian Gouran, Port of Bellingham

From: Steve Germiat and Joe Morrice, Aspect Consulting
Mike Riley and Mark Larsen, Anchor QEA

Re: **Site-Specific Nearshore Mercury Partition Coefficient for Use in Modeling Groundwater Screening Levels Protective of Sediment and Surface Water**
Georgia-Pacific West Site, Bellingham, Washington

This memorandum describes the approach and results for developing a Site-specific mercury partition coefficient to be applied in nearshore areas to model groundwater screening levels for the GP West Site that are protective of the marine environment (sediment and surface water). The modeling predicts the attenuation of Site-associated contaminants in Fill Unit groundwater along the flow path between a nearshore monitoring well and the sediment bioactive zone (point of exposure). The modeling approach is described in a memorandum submitted to Ecology entitled *Modeling Approach to Assess Groundwater Screening Levels Protective of Sediment and Surface Water*, *Georgia-Pacific West Site, Bellingham, Washington,* dated March 31, 2011, and prepared by Aspect Consulting and AnchorQEA (Aspect and Anchor QEA, 2011).

One component of the contaminant flow path attenuation considered in the modeling is sorption, which is assessed using contaminant-specific partition (or distribution) coefficients (K_d). The K_d relates the partitioning of a contaminant between the solid and aqueous phases, and can be defined as the ratio of the contaminant concentration on the solid phase (mg/kg) to the contaminant concentration in the aqueous phase (mg/L) at equilibrium with the solid phase. The K_d value is expressed in units of L/kg and, if expressing values in logarithmic space, log L/kg.

The Fill Unit groundwater flow path presented in the modeling travels through both the nearshore upland and waterway sediment – from a nearshore upland monitoring well to the sediment porewater bioactive zone (refer to Aspect and Anchor QEA, 2011). Mercury is the principal contaminant being evaluated in the modeling. As such, it is appropriate to develop a site-specific mercury K_d for the modeling that integrates collocated soil and groundwater data from the nearshore area of the upland GP West Site, and collocated sediment and sediment porewater data from the Whatcom Waterway Site. For consistency, the same mercury K_d value would also be applied for sediment cap recontamination modeling as part of the Whatcom Waterway remedial design. The K_d value(s) and other relevant fate and transport characteristics applicable to other upland areas of the Site are being evaluated separately and will be discussed as part of the forthcoming GP West Site Feasibility Study.

In the modeling to derive nearshore groundwater screening levels, the K_d value is applied in two partitioning scenarios along the groundwater flow path which have opposite effects in terms of the "conservatism" of the modeling assessment:

(1) K_d controls contaminant sorption from groundwater onto the aquifer matrix (solid) – i.e., contaminant attenuation – retarding groundwater transport of contaminants toward the sediment bioactive zone; and

(2) K_d defines an acceptable sediment porewater concentration that, assuming equilibrium partitioning, is protective of the sediment quality target(s) for the bioactive zone.

The upland nearshore groundwater screening level is derived as the sediment porewater concentration protective of sediment quality[1] multiplied by the model-predicted attenuation factor between a nearshore monitoring well and the bioactive sediment layer (refer to Aspect and Anchor QEA, 2011).

The following sections outline the data used to quantify the site-specific mercury K_d value to be used in the referenced nearshore modeling assessment, and a discussion of the results.

Site-Specific Data Evaluation

Data used to estimate a site-specific mercury K_d for this modeling assessment include empirical testing results for GP West site soil and groundwater, and Whatcom Waterway site sediment and porewater:

- In-water sediment /porewater data include twelve pairs of collocated in-water sediment/porewater samples from the Log Pond and the Whatcom Waterway:
  - Log Pond: Six Sediment/porewater sampling events conducted during interim action cap monitoring stations within intertidal areas of the Log Pond cap.
  - Whatcom Waterway: Six sediment and porewater sampling stations analyzed as part of the Pre-Remedial Design Investigation study (Anchor QEA, 2010).
- Upland nearshore soil/groundwater data include six pairs of collocated soil/groundwater samples from nearshore upland groundwater monitoring wells (Aspect, 2010; Aspect, 2011):
  - Law-1 area: Three monitoring wells (L1-MW01, L1-MW02, and L1-MW03) in the Law-1 area.
  - Downgradient of the caustic plume: Three monitoring wells (CP-MWA3, CP-MWB3, and CP-MWC3) beyond the extent of elevated pH in groundwater.

The sample locations are shown on Figure 1.

For the shoreline monitoring well data sets, the soil data are from samples collected within each well's screened interval and groundwater data are the average detected mercury concentrations from repeated sampling events. When mercury in porewater or groundwater was not detected at concentrations above the laboratory reporting limit, a concentration equal to one-half the detection limit was used for this evaluation; as noted below, this assumption has minimal effect on the estimated K_d values. Table 1 summarizes the site-specific data and individual K_d estimates. Appendix A provides a more detailed presentation of the data used.

[1] Or surface water cleanup level if more stringent.

An implicit assumption of this approach is that mercury concentrations in the aqueous and solid phases at each sampling location are in equilibrium. The equilibrium assumption is fundamental to derivation of cleanup levels for soil based on groundwater protection in MTCA (Chapter 173-304 WAC), and for sediment in the Sediment Management Standards (Chapter 173-204 WAC), and is therefore considered reasonable for this assessment.

Determination of Site-Specific K_d Value

As presented in Table 1, the calculated geometric mean of the individual site-specific mercury K_d estimates is 6,900 L/kg. This represents a log K_d value of 3.8. The value is robust in that it is based on a large data set containing 18 pairs of soil/sediment and porewater/groundwater samples. Furthermore, the estimated log K_d value changes only slightly (to 3.7) if data pairs with one or more non-detected values (aqueous phase) are excluded from the data set (Table 1).

The calculated site-specific mercury log K_d is the same value as reported in EPA (2005) for soil-to-water partitioning of mercury (log K_d of 3.8 based on a sample size of 17) and the site-specific data overlap with a smaller data set (2 samples) of EPA-published K_d values for sediments (EPA, 2005).

Summary

A site-specific mercury K_d of 6,900 L/Kg K_d value (log K_d value of 3.8) is calculated from a sizeable site-specific data set collected from nearshore areas of the GP West Site. The K_d estimate is considered robust in that it is based on the large site-specific data set, is not significantly affected by non-detect values, and is consistent with EPA-published K_d values for soils and sediments.

The derived site-specific K_d value is proposed for use in nearshore groundwater modeling described in Aspect and Anchor QEA (2011). Fate and transport characteristics for mercury in other upland areas of the Site with differing geochemical properties are being evaluated separately and will be discussed in the Feasibility Study after completion of bench-scale testing and other evaluations.

Please contact us if you have questions.

Attachments:

Table 1 – Site-Specific Estimates of Mercury K_d (Nearshore Groundwater)
Figure 1 – Upland and In-water Sample Locations for Site-Specific Mercury Data
Appendix A – Sampling Data Supporting Site-Specific Mercury K_d Value

References Cited

Anchor QEA, 2010, Pre-Remedial Design Investigation Data Report, Whatcom Waterway Site Cleanup, Bellingham, Washington, August 2010.

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Limitations

Work for this project was performed and this memorandum prepared in accordance with generally accepted professional practices for the nature and conditions of work completed in the same or similar localities, at the time the work was performed. It is intended for the exclusive use of the Port of Bellingham for specific application to the referenced property. This memorandum does not represent a legal opinion. No other warranty, expressed or implied, is made.

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Table 1 - Site-Specific Estimates of Mercury Kd (Nearshore Groundwater)

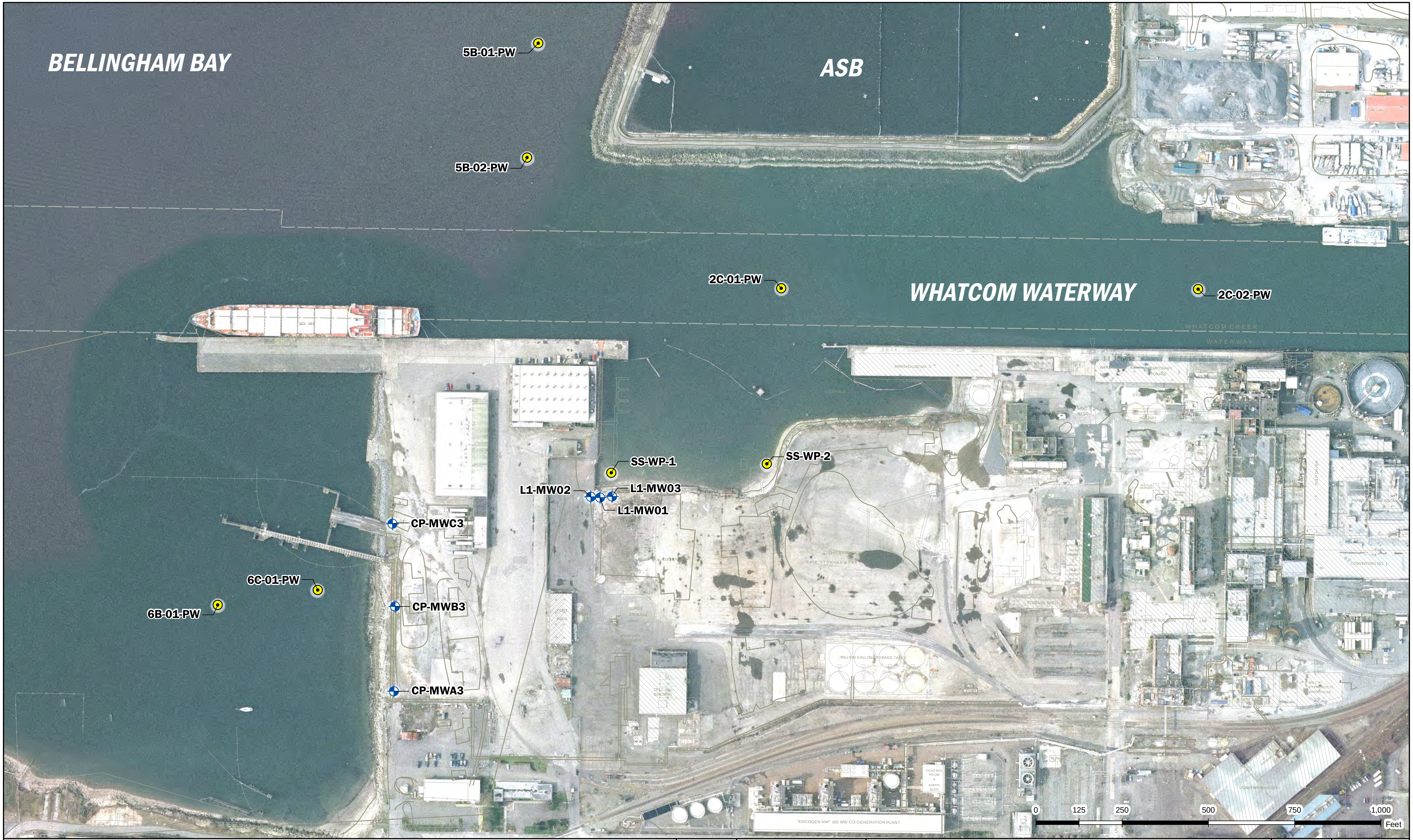
Sampling Station	Waterway Site Unit or GP West Site area	Soil or Sediment Mercury (mg/kg)	Pore-Water or Groundwater Mercury (mg/L)	Kd-Estimates Kd (L/Kg)	Kd-Estimates Log Kd Log(L/kg)	Notes
Whatcom Waterway Sediment and Porewater Sampling Data (see Table A-1 in Appendix A)						
2C-02-PW	Unit 2C (Waterway)	0.437	0.00005	8,733	3.9	1,2
2C-01-PW	Unit 2C (Waterway)	0.35	0.00005	7,000	3.8	1
5B-01-PW	Unit 5B (ASB Shoulder)	0.857	0.00005	17,133	4.2	1,2
5B-02-PW	Unit 5B (ASB Shoulder)	1.25	0.00005	25,000	4.4	1,2
6B-01-PW	Unit 6B (Barge Dock)	0.3	0.00005	6,000	3.8	1
6C-01-PW	Unit 6C (Barge Dock)	0.333	0.00005	6,667	3.8	1
RET-05-WP1 Y5	Unit 4 (Log Pond, Southwest Corner)	0.6525	0.00000118	552,966	5.7	2
SS-WP-1 Y2	Unit 4 (Log Pond, Southwest Corner)	0.08	0.0000721	1,110	3.0	
SS-WP-1 Y1	Unit 4 (Log Pond, Southwest Corner)	0.025	0.0000059	4,237	3.6	3
RET-05-WP2 Y5	Unit 4 (Log Pond, Central Section)	0.09	0.00000277	32,491	4.5	
SS-WP-2 Y2	Unit 4 (Log Pond, Central Section)	0.15	0.0000013	115,385	5.1	1
SS-WP-2 Y1	Unit 4 (Log Pond, Central Section)	0.035	0.0000074	4,730	3.7	3
Nearshore Soil and Groundwater Data from the GP West Site (see Table A-2 in Appendix A)						
L1-MW01	Law-1 Nearshore Area	2.37	0.00753	315	2.5	1
L1-MW02	Law-1 Nearshore Area	1.72	0.0185	93	2.0	
L1-MW03	Law-1 Nearshore Area	0.15	0.0000245	6,000	3.8	
CP-MWA3	Downgradient of Caustic Plume	0.02	0.000000548	36,530	4.6	1
CP-MWB3	Downgradient of Caustic Plume	0.026	0.000177	147	2.2	
CP-MWC3	Downgradient of Caustic Plume	0.023	0.000000888	25,915	4.4	
Mean Estimate (All Data; N=18) [4]				6,900	**3.8**	
Mean Estimates (Excluding Sample Pairs with ND Water Values; N=9) [4]				4,500	**3.7**	

Notes:

Anchor QEA (2010): Pre-Remedial Design Investigation Data Report. Prepared for the Port of Bellingham.

RETEC (2006): Remedial Investigation and Feasibility Study for the Whatcom Waterway Site. Prepared for the Port of Bellingham.

1. Porewater or groundwater mercury was not detected in one or more measurement. Concentration equal to 1/2 the reporting limit assumed for non-detects.
2. Sediment value based on two or more adjacent samples located in immediate vicinity of pore-water sampling location.
3. Sediment mercury concentration was not detected. Sediment concentration estimated based on an assumed concentration equal to 1/2 the reporting limit.
4. Kd values are expressed as the geometric mean of the Kd estimates.



Upland Soil/Groundwater Sample Location

In-water Sediment/Porewater Sample Location

MILL NORTH

TRUE NORTH

45°

Aspect CONSULTING

Upland and In-Water Sample Locations for Site-Specific Mercury Data

GP West Site - Bellingham, WA

DATE: Jun 2011	PROJECT NO. 070188
DESIGNED BY: SJG	
DRAWN BY: PPW	FIGURE NO. 1
REVISED BY: - - -	

Path: T:\projects_8\Port_of_Bellingham\Deivered\SampleLocs_Site-Specific_Mercury.mxd

APPENDIX A

Data Supporting Site-Specific Mercury Kd Estimate

Table A-1 - Paired Sediment and Porewater Analytical Data

Pore Water Data						Sediment Data			Kd Estimates		
Sampling Station	Waterway Site Unit	Data Source	Measured Porewater Data Hg (ug/L)	Measured Porewater Data Hg (mg/L)	Unit Estimate Hg (mg/L)	Sampling Stations	Sample Results Hg (mg/kg)	Unit Estimate Hg (mg/kg)	Kd (L/Kg)	Log Kd Log(L/kg)	Notes
2C-02-PW	Unit 2C (Waterway)	Anchor QEA, 2010	0.1U	0.0001U	0.00005	2C-03-SS	0.6	0.437	8,733	3.94	1,2
						5C-02-SS	0.31				
						3A-04-SS	0.4				
2C-01-PW	Unit 2C (Waterway)	Anchor QEA, 2010	0.1U	0.0001U	0.00005	2C-01-SS	0.3	0.35	7,000	3.85	1
						2C-02-SS	0.4				
5B-01-PW	Unit 5B (ASB Shoulder)	Anchor QEA, 2010	0.1U	0.0001U	0.00005	5B-01-SS	0.76	0.857	17,133	4.23	1,2
						5B-02-SS	1.1				
						5B-05-SS	0.71				
5B-02-PW	Unit 5B (ASB Shoulder)	Anchor QEA, 2010	0.1U	0.0001U	0.00005	5B-03-SS	0.5	1.25	25,000	4.40	1,2
						5B-04-SS	0.61				
						5B-06-SS	2.64				
6B-01-PW	Unit 6B (Barge Dock)	Anchor QEA, 2010	0.1U	0.0001U	0.00005	6B-02-SS	0.3	0.3	6,000	3.78	1
						6B-03-SS	0.3				
						6B-05-SS	0.3				
6C-01-PW	Unit 6C (Barge Dock)	Anchor QEA, 2010	0.1U	0.0001U	0.00005	6C-01-SS	0.3	0.333	6,667	3.82	1
						6C-02-SS	0.4				
						6B-05-SS	0.3				
RET-05-WP1 Y5	Unit 4 (Log Pond, Southwest Corner)	RETEC, 2006	0.00118	0.00000118	0.00000118	SS-W1	2	0.6525	552,966	5.74	2
						SS-W3	0.13				
						SS-W5	0.3				
						SS-W8	0.18				
SS-WP-1 Y2	Unit 4 (Log Pond, Southwest Corner)	RETEC, 2006	0.0721	0.0000721	0.0000721	SS-WP-1 Y2	0.08	0.08	1,110	3.05	
SS-WP-1 Y1	Unit 4 (Log Pond, Southwest Corner)	RETEC, 2006	0.0059	0.0000059	0.0000059	SS-WP-1 Y1	0.05 U	0.025	4,237	3.63	3
RET-05-WP2 Y5	Unit 4 (Log Pond, Central Section)	RETEC, 2006	0.00277	0.00000277	0.00000277	RET-05-WP2 Y5	0.09	0.09	32,491	4.51	
SS-WP-2 Y2	Unit 4 (Log Pond, Central Section)	RETEC, 2006	0.0026 U	0.0000026 U	0.0000013	SS-WP-2 Y2	0.15	0.15	115,385	5.06	1
SS-WP-2 Y1	Unit 4 (Log Pond, Central Section)	RETEC, 2006	0.0074	0.0000074	0.0000074	SS-WP-2 Y1	< 0.07	0.035	4,730	3.67	3

Notes:

Anchor QEA (2010): Pre-Remedial Design Investigation Data Report. Prepared for the Port of Bellingham.

RETEC (2006): Supplemental Remedial Investigation and Feasibility Study for the Whatcom Waterway Site. Prepared for the Port of Bellingham.

1. Pore-water mercury was not detected. Kd estimate based on assumed concentration equal to 1/2 the reporting limit.
2. Sediment value based on two or more adjacent samples located in immediate vicinity of pore-water sampling location.
3. Sediment mercury concentration was not detected. Sediment concentration estimated based on an assumed concentration equal to 1/2 the reporting limit.

Table A-2 - Paired Soil and Groundwater Analytical Data

Data Source	Location Name	Soil Sample Data: Sample Date	Soil Sample Depth Interval	Mercury in mg/kg	Groundwater Sample Data: Screen Depth Interval (ft)	Dissolved Mercury in mg/L: 9/28/2009	3/29/2010	12/19/2010	1/31/2011	Kd Estimates: Kd (L/Kg)	Log Kd Log(L/kg)	Notes
Aspect, 2011	L1-MW01	12/16/10	7 to 9 feet	1.02	8.5-13.5	NA	NA	0.000460	0.014600			
	L1-MW01	12/16/10	11 to 13 feet	3.72						315	2.5	
Aspect, 2011	L1-MW02	12/17/10	11 to 12 feet	3.37	10-15	NA	NA	0.035000	0.002090			
	L1-MW02	12/17/10	13 to 14 feet	0.075						93	2.0	
Aspect, 2011	L1-MW03	12/15/10	15 to 16 feet	0.147	14-19	NA	NA	0.000025	0.0000240	6,000	3.8	
Aspect, 2010	CP-MWA3	09/16/09	11 to 13 feet	0.020	9-14	0.0000010 U	0.0000006 J	NA	NA	36,530	4.6	1
Aspect, 2010	CP-MWB3	09/17/09	13 to 15 feet	0.026	11-16	0.0003060	0.0000479	NA	NA	147	2.2	
Aspect, 2010	CP-MWC3	09/16/09	12 to 14 feet	0.023	10-15	0.0000017 U	0.0000009 J	NA	NA	25,915	4.4	1

NA: Not analyzed.
U: Not detected at associated reporting limit. J: Estimated concentration.
1. Groundwater mercury was not detected. Kd estimate based on assumed concentration equal to 1/2 the reporting limit.

ATTACHMENT 2
LITERATURE REVIEW OF DIOXIN/FURAN PARTITIONING COEFFICIENTS

Literature Review of Partitioning Estimates for Dioxin/Furan Compounds

Congener		log Kow						
		KOWWIN (EPI Suite)		Experimental and theoretical values				
Analyte Name	CAS	KOWWIN estimated value	Experimental Database	log Kow Govers & Krop (1998)	Range/values cited in Mackay (1992)	Range/values cited by ATSDR (1994, 1998)	Range cited in EPA (2003 draft); Tbl A-1	EPA (2003 draft) selected value
Dioxins								
2,3,7,8-TCDD	1746-01-6	6.92	6.8	6.96	5.38 - 8.93	6.8 - 8.7	5.38 - 8.93	6.80
1,2,3,7,8-PeCDD	40321-76-4	7.56	6.64	7.50	--	--		6.64
1,2,3,4,7,8-HxCDD	39227-28-6	8.21	7.80	7.94	7.3 - 10.89	9.19 - 10.4	7.79 - 10.44	7.80
1,2,3,6,7,8-HxCDD	57653-85-7	8.21	--	7.98	--	--	--	7.30
1,2,3,7,8,9-HxCDD	19408-74-3	8.21	--	7.87	--	--	--	7.30
1,2,3,4,6,7,8-HpCDD	35822-46-9	8.85	8.00	8.40	7.92 - 11.98	9.69 - 11.38	7.92 - 11.98	8.00
OCDD	3268-87-9	9.50	8.20	8.75	7.33 - 13.08	8.78 - 13.37	7.5 - 13	8.20
Furans								
2,3,7,8-TCDF	51207-31-9	6.29	6.53	6.46	5.82 - 7.70	5.82	5.82 - 6.53	6.1
1,2,3,7,8-PeCDF	57117-41-6	6.94	6.79	6.99	--	6.79	--	6.79
2,3,4,7,8-PeCDF	57117-31-4	6.94	6.92	7.11	6.92 - 7.60	6.92	6.92 - 7.82	6.5
1,2,3,4,7,8-HxCDF	70648-26-9	7.92	--	7.53	7.70	--	--	7.0
1,2,3,6,7,8-HxCDF	57117-44-9	7.92	--	7.57	--	--	--	7.0
1,2,3,7,8,9-HxCDF	72918-21-9	7.58	--	7.76	--	--	--	7.0
2,3,4,6,7,8-HxCDF	60851-34-5	7.92	--	7.65	--	--	--	7.0
1,2,3,4,6,7,8-HpCDF	67562-39-4	8.23	7.92	8.01	7.90 - 9.25	7.92	7.92 - 9.25	7.4
1,2,3,4,7,8,9-HpCDF	55673-89-7	8.23	--	8.23	6.90	--	--	7.4
OCDF	39001-02-0	8.87	8.60	8.60	7.05 - 13.93	8.20	7.0 - 13	8.00

Final recommended values

Literature Review of Partitioning Estimates for Dioxin/Furan Compounds

Congener		log Koc															
		KOCWIN (EPI Suite)				empirically derived based on correlation to log Kow						Experimentally derived K_{OC} values			
						Karickhoff 1979 *[log Koc = log Kow - 0.21]*			Di Toro 1985 *[log Koc = 0.983 log Kow + 0.00028]*						
Analyte Name	CAS	MCI method	From log Kow	Experimental Database	Range cited in EPA (2003 draft); Tbl A-1	Using KOWWIN (EPI Suite) log Kow	Using Govers & Krop (1998) log Kow	Using EPA (2003 draft) log Kow	Using KOWWIN (EPI Suite) log Kow	Using Govers & Krop (1998) log Kow	Using EPA (2003 draft) log Kow	Walters (1989)	Lodge (2002)	Fan (2006)	Frankki (2007)
Dioxins															
2,3,7,8-TCDD	1746-01-6	5.40	4.83	6.50	3.06 - 8.50	6.71	6.75	6.59	6.80	6.84	**6.68**	6.66	> 7.1	4.14	--
1,2,3,7,8-PeCDD	40321-76-4	5.62	4.74	--		7.35	7.29	6.43	7.43	7.37	**6.53**	--	--	--	6.43
1,2,3,4,7,8-HxCDD	39227-28-6	5.84	5.38	--	5.02 - 7.10	8.00	7.73	7.59	8.07	7.81	**7.67**	--	--	--	7.61
1,2,3,6,7,8-HxCDD	57653-85-7	5.84	5.61	--	--	8.00	7.77	7.09	8.07	7.84	**7.18**	--	--	--	7.45
1,2,3,7,8,9-HxCDD	19408-74-3	5.84	5.61	--	--	8.00	7.66	7.09	8.07	7.74	**7.18**	--	--	--	7.90
1,2,3,4,6,7,8-HpCDD	35822-46-9	6.07	5.49	--	5.47 - 7.80	8.64	8.19	7.79	8.70	8.26	**7.86**	--	--	--	7.33
OCDD	3268-87-9	6.29	5.61	--	5.92 - 7.90	9.29	8.54	7.99	9.34	8.60	**8.06**	--	--	--	7.20
Furans															
2,3,7,8-TCDF	51207-31-9	5.14	4.68	--	5.20 - 7.50	6.08	6.25	5.89	6.18	6.35	**6.00**	--	--	--	--
1,2,3,7,8-PeCDF	57117-41-6	5.37	4.83	--	--	6.73	6.78	6.58	6.82	6.87	**6.67**	--	--	--	--
2,3,4,7,8-PeCDF	57117-31-4	5.37	4.83	--	5.59 - 7.40	6.73	6.90	6.29	6.82	6.99	**6.39**	--	--	--	--
1,2,3,4,7,8-HxCDF	70648-26-9	5.59	5.45	--	--	7.71	7.32	6.79	7.79	7.40	**6.88**	--	--	--	--
1,2,3,6,7,8-HxCDF	57117-44-9	5.59	5.45	--	--	7.71	7.36	6.79	7.79	7.44	**6.88**	--	--	--	--
1,2,3,7,8,9-HxCDF	72918-21-9	5.59	5.26	--	--	7.37	7.55	6.79	7.45	7.63	**6.88**	--	--	--	--
2,3,4,6,7,8-HxCDF	60851-34-5	5.59	5.45	--	--	7.71	7.44	6.79	7.79	7.52	**6.88**	--	--	--	--
1,2,3,4,6,7,8-HpCDF	67562-39-4	5.81	5.45	--	6.00 - 7.90	8.02	7.80	7.19	8.09	7.87	**7.27**	--	--	--	--
1,2,3,4,7,8,9-HpCDF	55673-89-7	5.81	5.45	--	--	8.02	8.02	7.19	8.09	8.09	**7.27**	--	--	--	--
OCDF	39001-02-0	6.04	5.83	--	6.00 - 7.40	8.66	8.39	7.79	8.72	8.45	**7.86**	--	--	--	--

Final recommended values

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Please note the following disclaimers regarding the EPA 2003 draft document:

"EPA announced the release of the final Reanalysis of Key Issues Related to Dioxin Toxicity and Response to NAS Comments, Volume 1, in a February 17, 2012, Press Release. This document provides hazard identification and dose-response information on 2,3,7,8- tetrachlorodibenzo-p-dioxin (TCDD) and the most up-to-date analysis of non-cancer health effects from TCDD exposure. The report also include a reference dose (RfD) and a detailed and transparent description of the underlying data and analyses. EPA will complete Reanalysis, Volume 2, containing the full dioxin cancer assessment, as expeditiously as possible. In Volume 2, EPA will complete the evaluation of the available cancer mode-of-action data, and will augment the cancer dose-response modeling, including justification of the approaches used for dose response modeling of the cancer endpoints, and an associated quantitative uncertainty analysis."

DISCLAIMERS:

Volume 1 (noncancer) of the Reanalysis contains some descriptive cancer information. The cancer information in Volume 1 should not be used for regulatory or risk management decision-making.
Volumes 1 and 2 of the Reanalysis will supersede the 2003 draft dioxin Reassessment.
The 2003 draft dioxin Reassessment includes a disclaimer that the document should not be cited or quoted. As such, information in this draft document should not be used for regulatory or risk management decision-making.

http://cfpub.epa.gov/ncea/cfm/recordisplay.cfm?deid=209690