HYDRAULIC MODELING REPORT

WCD Marine Yard

British Columbia

05/13/2024

Prepared by:

Kiewit Engineering Group Inc

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British Columbia

May 2024

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1.0 INTRODUCTION

1.1 Project Description

Kiewit is developing a Marine Yard along the Fraser River in Coquitlam, BC. It is located just downstream of the Port Mann bridge. This Marine yard will be used to load and unload barges, as well as for temporary storage of construction materials.

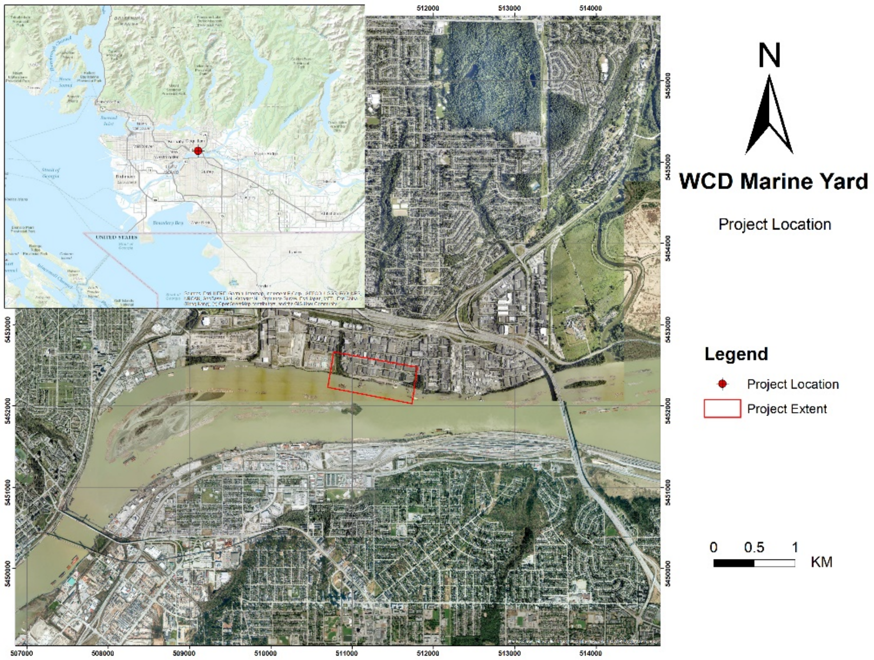

Exhibit 1.1: Project Location

1.2 Purpose

The purpose of this hydrotechnical analysis is to provide a design that meets the criteria for the required life of the Marine Yard. Given the uncertainty in hydrology and hydraulics, combined with the uncertainty of climate change as well as future natural and manmade impacts, it is not expected that this hydrotechnical analysis will provide the answers to all questions that come up during final design. It will aid the team in developing a marine yard configuration that meets the design requirements while also being resilient and economically efficient.

1.3 Scope of Work

The hydrotechnical scope of work includes:

- Revetment design. The revetment design considers riverine conditions during large discharges (design event) and shipping induced waves and water movements. Expected high water levels during the design event are determined.
- Scour depth estimates for the foundations at the proposed structure locations.
- Calculation of hydraulic loadings at the foundation elements.

1.4 Datum Conversion

The HEC-RAS model is in the CGVD13 datum. The conversion from a geodetic datum (like CGVD) to Chart Datum (CD) is site specific as the CD changes along the Fraser River. For the project location, the conversion factors between the datums are listed in table 1.1.

Table 1.1: Datum Conversion

From: To:	Chart Datum	CGVD28 (HTv2.0)	CGVD2013
Chart Datum	-	-1.135	-1.012
CGVD28	+1.135	-	-0.123
CGVD2013	+1.012	+0.123	-

Unless otherwise noted, all elevations in this report refer to the CGVD2013 geodetic datum.

2.0 CLIMATE CHANGE

2.1 Climate Change Impacts

Both Sea Level Rise (SLR) and changing flow conditions on the Fraser River can change hydraulic conditions at the project location. According to VFPA (2020), the recommended design life for new shoreline construction is 50 years. Per FLNRORD (2014), the design event for a 50-year design life is a 100-year event. VFPA (2020) indicates the relative SLR in the pacific Northwest Region is less than the global average. The report indicates a SLR at the end of the century between 0.5m and 0.75m. The end of the century exceeds the design life of the facility, and a 0.5m SLR was assumed for the project. The HEC-RAS model obtained from the Fraser Basin Council included the inflow and tidal data for a 100-year (1% AEP) event with 0.5m SLR and a climate reflecting 2050 conditions. Moderate climate change impacts have been considered by using the 2050 conditions. These data were used to estimate the hydraulic design parameters for the facility.

3.0 HYDRAULIC MODEL

3.1 Model Source

The HEC-RAS model was obtained from the Fraser Basin Council (FBC). The model included the results of the historical 200-year (0.5% AEP) flood. However, based on the VFPA requirements to include the impacts of climate change, the model was updated to account for the Freshet Climate Change year 2050, 1% AEP with 0.5 m SLR. The flow data for this scenario was included in the model obtained from FBC.

3.2 Model Extent

The FBC model extent covers from north of Hope, BC to the Salish Sea and includes several tributaries and floodplains. For the design development, the model extent was reduced to a section on the Fraser River, roughly between the Port Mann and Patullo bridges (short model). This short model was used to evaluate the impact of design decisions without necessitating running the full model for each design iteration.

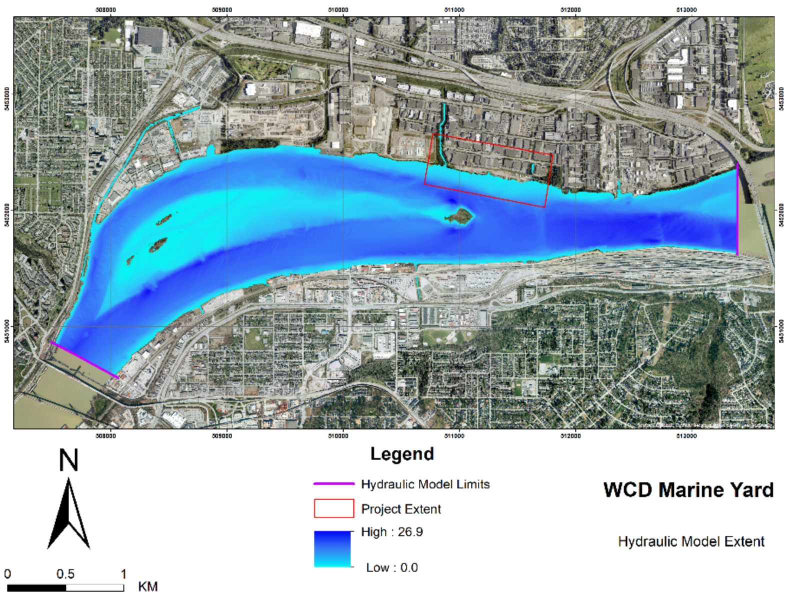

Exhibit 3.1: Short model extents

3.2.1 Boundary Conditions

The boundary conditions for the short model were extracted from the full model. The model results for the historical 200-year (0.5% AEP) flood were used to calibrate the short model. The full FBC HEC-RAS model was re-run, using boundary conditions based on an AEP of 1% with a Sea Level Rise (SLR) of 0.5m and a moderate impact of climate change on the Fraser River Discharge.

The boundary conditions for the short model were extracted from the full model results. The upstream boundary condition flow hydrograph was broken up in two sections. One section for the faster flows in the main channel, and a section for the shallower sections outside the main channel. The downstream boundary condition was the stage hydrograph of a point on the downstream boundary of the short model.

3.2.2 Model validation

The results of the short model were compared to the results of the full model for the 200-year event. The energy grade slopes of both upstream boundary conditions were iterated until the short model and full FBC model water velocity and depth aligned closely. At the project location, the short model showed a maximum water surface elevation of 4.09m. The full model showed an elevation of 4.11m. This difference is within the expected range of accuracy for a hydraulic model at this scale.

3.3 Terrain Modifications

The proposed dredging activities and land infill will impact the bathymetry of the Fraser River. The terrain configuration shown below depicts the dredging and infill activities that are included in the final yard layout. These changes in the terrain were included within the simulation to evaluate the impact on design hydrotechnical parameters.

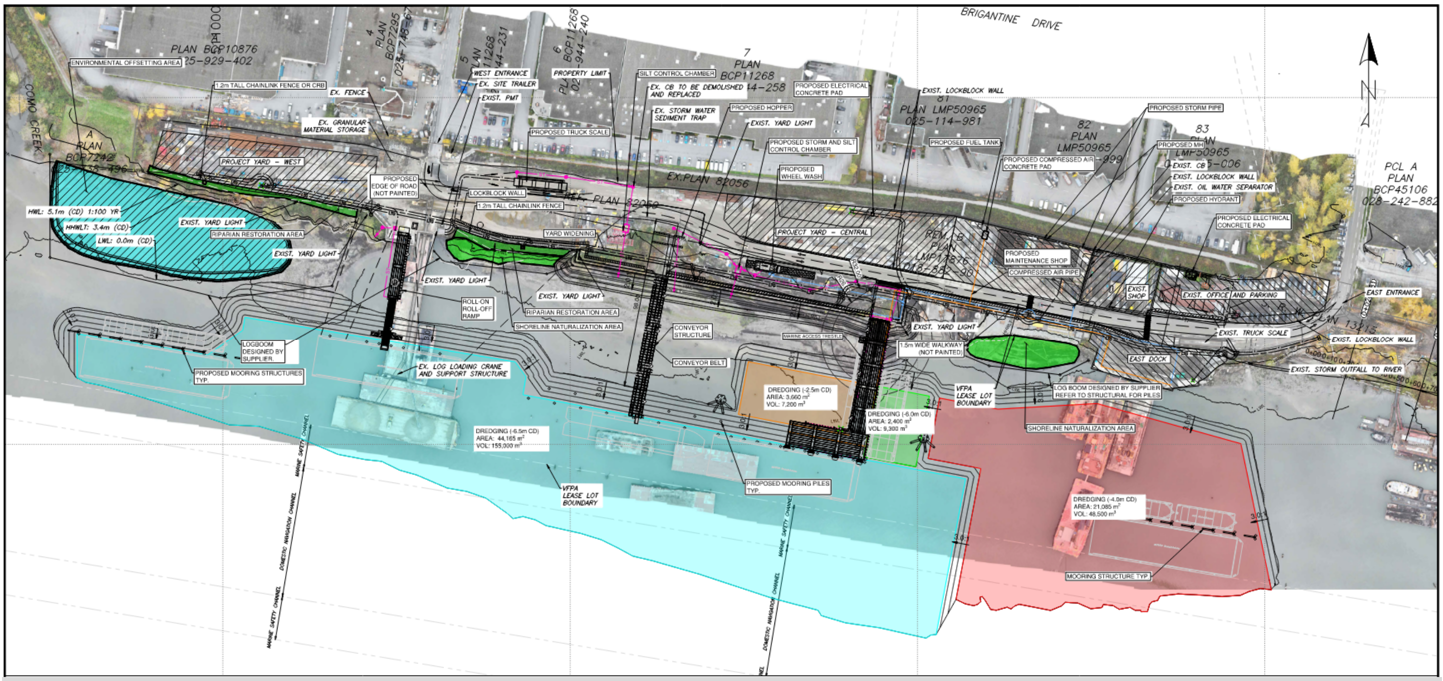

Exhibit 3.2: Marine Yard Site Plan

3.3.1 Terrain Roughness

No changes were made to the terrain roughness values in the short model. The terrain roughness for the modified areas were set to match the existing stream bed roughness values.

3.3.2 Terrain Break lines

Break lines from the full model were incorporated into the short model. Additional break lines were added to accurately reflect the outlines of the dredging activities.

3.3.3 Marine Access Trestle

A trestle structure is proposed near the east side of the property. An existing structure is currently used by a Kiewit-Ledcor Partnership and located along Commissioner Street in Vancouver Harbor that will be repurposed. A new trestle structure, similar to the existing Commissioner Trestle that is supported by 1067 mm piles, will be constructed on 1219 mm diameter piles. Given the total width of the river and the relatively small pier sizes required for the trestle structure, the piers were not included in the model geometry.

3.3.4 Bottleneck Infill

Towards the center of the property, a small bight creates a bottleneck for the operations in the Marine Yard. The infill of this bight was included in the terrain model.

3.3.5 Conveyor Structure

The Conveyor Structure will be constructed on 610 mm, 910 mm, and 1219 mm diameter piles. Therefore, and similarly to the Marine Access Trestle case, these piers were not included in the terrain model.

3.3.6 Roll-on/Roll-off Ramp

The Roll-on/Roll-off (RoRo) will be installed downstream of the Marine Access Trestle and the Conveyor Structure. The RoRo ramp will consist of an approach structure supported by piles and an articulated ramp. The towers to support the hydraulic cylinders to move the ramp up and down will be supported on piles. Similar to the trestle and the conveyor structures, the relatively small piles will not be incorporated into the terrain model.

3.3.7 Habitat Compensation

The habitat compensation will be formed by a tidal bench in the river on the west side of the property. The bench design was added to the terrain model and thus it was incorporated into the short model. Adding this geometry, allowed assessing the impact of this terrain modification on local hydraulic variables when compared to the existing conditions.

3.4 Model Run

No changes were made to the model run parameters between the full and short models.

The run time for the full model ramp up plan was about half an hour and a full model run was approximately 30 hours. For the short model, the ramp up plan and full model ran in 30 seconds and 10 minutes respectively.

3.5 Model Results

The proposed dredging, land infill, and marsh bench habitat compensation have limited impact on near-shore velocities, where velocities vary only slightly between existing and proposed conditions. In Exhibit 3.3, existing velocity contours are shown in red while proposed velocity contours are shown in yellow.

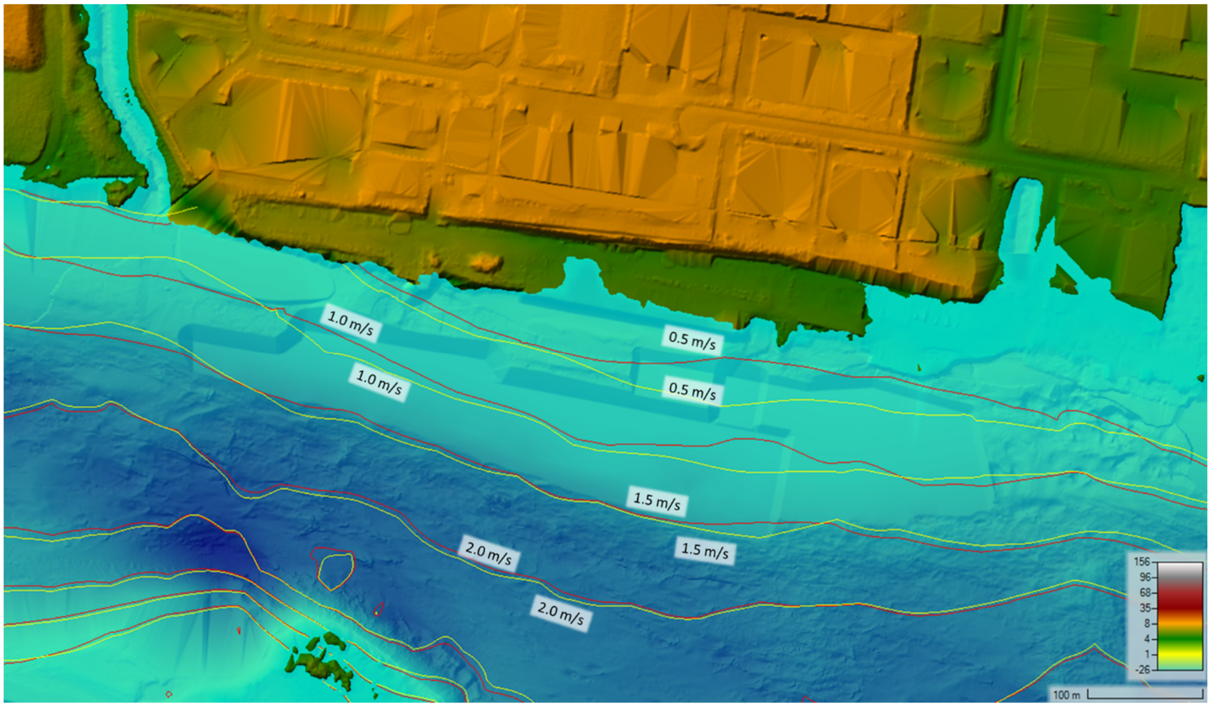

Exhibit 3.3: HEC-RAS model velocity comparison

4.0 SHIP INDUCED WAVES AND WATER MOVEMENT

The Fraser River has relatively low velocities near the project location. During large discharge events, the velocities at the project area range from 0.5 m/s to 1.6 m/s. VFPA (2020) indicates that wind induced waves are not a concern at the facility. However, shipping induced waves and water movement are expected to induce hydraulic loadings on the proposed facilities. The largest waves are expected from passing vessels, not from ingress and egress operations at the facility. The main shipping channel (Queens Reach) in the Fraser is on the south side of the river, while the facility is located on the north side of the river. A secondary navigation channel (Sapperton Channel) is located closer to the facility (See Exhibit 4.1). It is assumed that the traffic closer to the facilities, in the Sapperton Channel, will result in the critical loading conditions.

Tugboats operating near the facility can cause high water velocities. The tugboats are expected to operate near the facility in a safe manner, avoiding damages to both the revetment and the foundation structures. The revetment design and scour protection is designed to provide operational flexibility, not protect against the full thrust of the tugboats. Per the VFPA (2020) guidance, the primary ship wave, return current, secondary ship waves, and propeller jet were calculated using CIRIA C683 (2007).

4.1 Design Vessel

For the wave action, a large ship that is likely to navigate in the area was considered for design. Therefore, the dimension for the design vessel were based on the DB General, a 300'x100' barge with a draft of 8'.

L_s= 91.5, Ship Length (m)

B_s= 30.5, Ship beam Bs (m)

T_s= 2.5, draught (m)

The larger the distance between the shipping channel and the facility, the more the waves will attenuate before reaching the facility. The following parameters are used (see Figure 4.1 to Figure 4.3):

y= 150, Ship position relative to the fairway (shipping channel) centerline (m)

y_s= 100, Ship position relative to the bank (m)

A_c= 6667, cross-sectional area of the waterway (m²)

h= 6, water depth of fairway in the dredging area (m)

b_b= 420, bottom width of fairway (m)

b_w= 620, top width of fairway (m)

$\alpha = 1.14$, slope angle of the bank (radians)

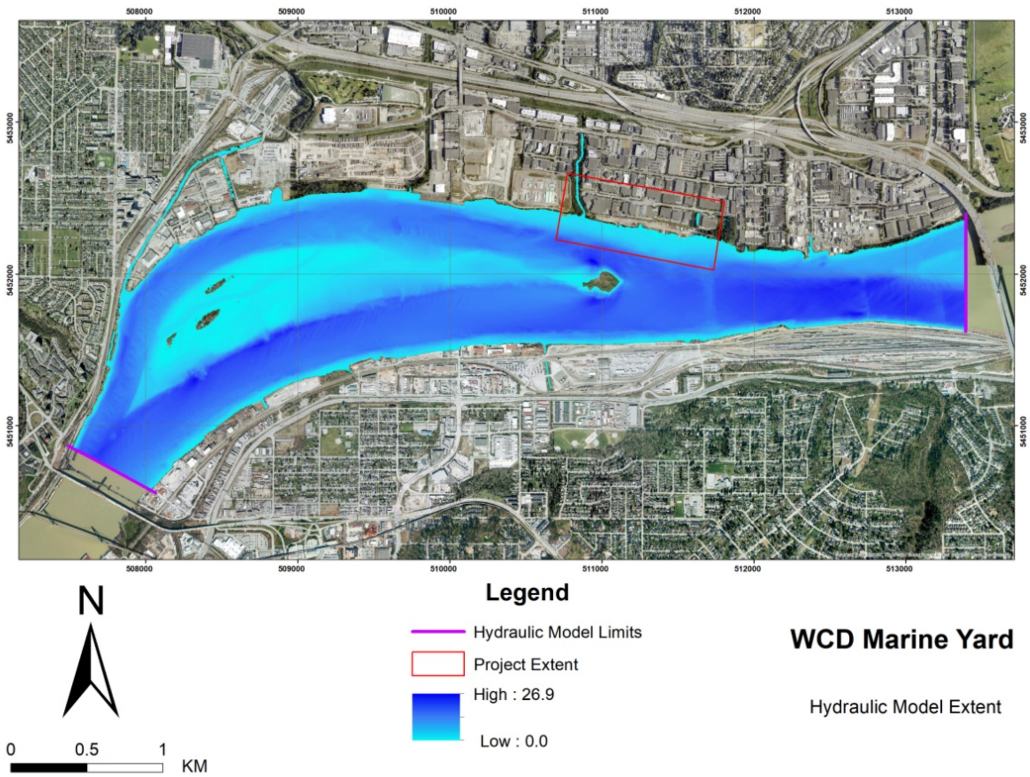

Exhibit 4.1: Channel Location

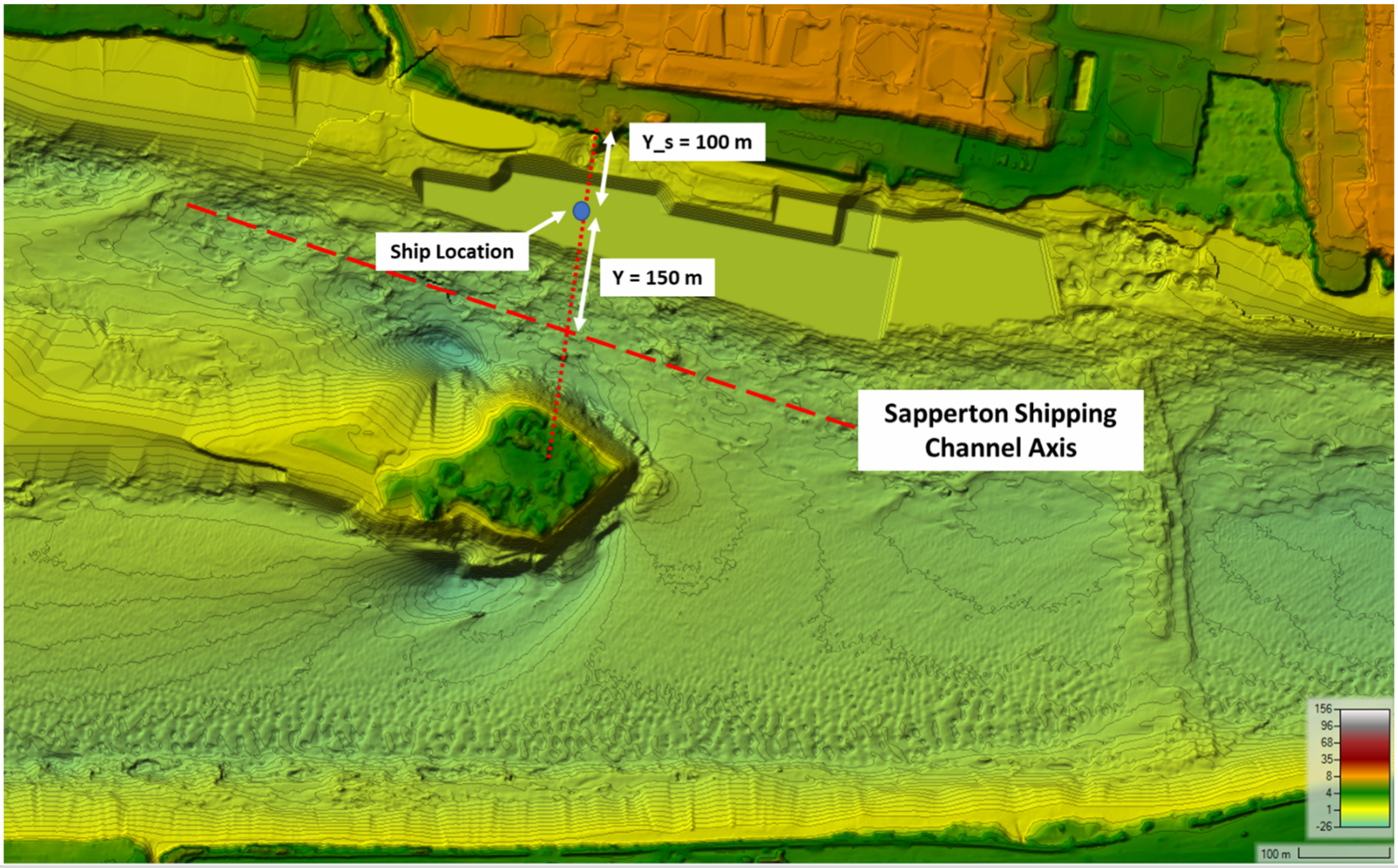

Exhibit 4.2: Distances to shipping channel

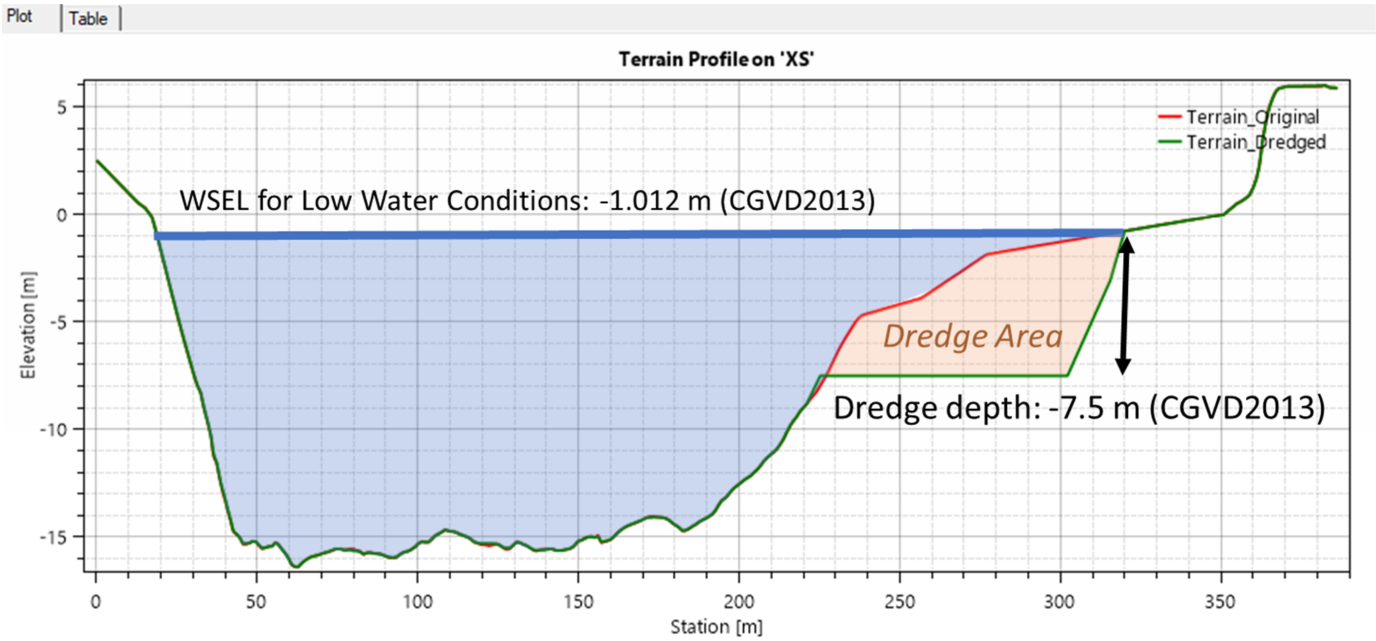

Exhibit 4.3: Channel Cross Section

4.2 Primary ship wave

The transversal front wave, water level depression alongside the ship, and transversal stern wave were evaluated according to the methodology described in CIRIA C683 paragraph 4.3.4.

The largest waves and highest return flow velocities induced by ships will occur during low flow conditions (the low-low water level, 0 m at Chart Datum). Converted to CGVD2013, a water level of -1.0 m was used in the calculations.

4.2.1 Limit Speed and Actual Speed

The limit speed of the vessel, V_L is assumed to be 10 knots or 5 m/s. This results in an actual speed (V_s) of 7.5 knots (3.75 m/s), using a reduction friction factor for waves (f_w) of 0.75, which corresponds to loaded ships.

4.2.2 Water Level Depression

Moving vessels create a depression wake along the vessel that could migrate to shore. The mean water level depression (Δh) can be calculated by equation 4.173 of CIRIA (C683):

$$\Delta \mathrm{h} = \frac{{V_s}^2}{2g}\left[\alpha_s(\frac{A_c}{{A^*}_c})^2 - 1\right]$$

α_s= 1.1, Factor to express the effect of the sailing speed V_s relative to its maximumV_L (-)

$$\alpha_s = 1.4 - 0.4\frac{V_s}{V_L}$$

A_c = 2749, Cross-sectional area of the fairway in the undisturbed situation (m^2) as follows:

$$A_c = (b_b * h) + (h^2 * cot(\alpha))$$

${A^*}_c$= 2615, Cross-sectional area of the fairway next to the ship (m^2) as follows:

$${A^*}_c = b_b(h - \Delta h) + cot\alpha(h - \Delta h)^2 - A_m$$

Δh = 0.15, previously defined for these parameters by iteration (m).

A_m = 68.63, Vessel's submerged x-section (m^2). Considering a beam width of 30.5m, draught of 2.5 and 0.9 as midship coefficient. $A_m = C_{midship} * beam * draught$

The calculated mean water level depression is 0.16 m. The maximum water level depression ($\Delta\hat{h}$) is a function of the mean depression and is calculated using equation 4.175 of CIRIA (C683).

$$\Delta\hat{h}/\Delta h = \begin{cases} 1 + 2{A^*}_w & for\ b_w / L_s < 1.5 \\ 1 + 4{A^*}_w & for\ b_w / L_s \geq 1.5 \end{cases}$$

${A^*}_w = y\frac{h}{A_c}$ (-)

b_w= 620, top width of fairway (m)

L_s= 91.5, ship Length (m)

The calculated maximum water level depression is 0.37 m.

4.2.3 Front Wave Height

The height of the front wave (Δh_f) can be calculated using equation 4.177 of CIRIA (C683)

$$\Delta h_f = 0.1\Delta h + \Delta\hat{h}$$

Δh = 0.15, mean water level depression (m)

$\Delta\hat{h}$= 0.37, maximum water level depression (m/s)

Based on the water level depressions calculated in Section 4.2.2, the front wave height is 0.39 m.

4.2.4 Stern Wave Height

The stern wave height (z_{max}) can be calculated using equation 4.179 of CIRIA (C683)

$$z_{max} = 1.5\,\Delta\hat{h}$$

$\Delta\hat{h}$= 0.37, maximum water level depression (m/s)

The height of the stern wave is 0.56m.

4.3 Return Current

For river cross sections (A_c) that are large compared to the submerged vessel cross section (A_m), the flow field should be considered 2-dimensional. The calculation methods are based on 1D flow fields. Given the large river cross section relative to the submerged cross-sectional area of a vessel, the use of a 1D flow field will be a limitation in the computation of the design velocities while the return current is not expected to result in critical design velocities.

4.4 Secondary ship waves

Secondary waves, their wavelength, and period were evaluated according to the methodology described in CIRIA C683 paragraph 4.3.4.2. This methodology is limited to conditions where the sailing speed is relatively small compared to the river depth ($V_s/\sqrt{(gh)} < 0.8$. The calculated actual speed (V_s) is 3.8 m/s, and the river depth (h) in the channel is 14 m, resulting in $V_s/(gh)^{0.5} < 0.32$.

$$H_i = 1.2\,\alpha_i h \frac{\left(\frac{y_s}{h}\right)^{-\frac{1}{3}} {V_s}^4}{(gh)^2}$$

H_i = Height of secondary wave (m)

α_i = 1, coefficient, corresponding to tugs and loaded ships (-)

y_s = 100, Ship position relative to the bank (m)

h = 6, river depth (m)

V_s = 3.75, actual vessel speed (m/s)

g = 9.81, gravitational constant (m/s^2)

The resulting secondary wave height is 0.16 m. From equation 4.185 of CIRIA (C683), the wavelength (L_i) of the secondary wave is 6 m.

$$L_i = 4.2\frac{V_s^2}{g}$$

From equation 4.186, the wave period (T_i) of the secondary wave is 2 seconds.

$$T_i = 5.1\frac{V_s}{g}$$

4.5 Propeller jet

The methodology described in CIRIA C683 paragraph 4.3.4.3 relies on applied power to the tugboat propellers during low-speed operations. The most powerful (1,788 HP) tugboat used at the facility (Tim MacKenzie) has a propeller diameter of 61.5”. This tugboat information was considered for the propeller jet velocities.

The following parameters were considered for its evaluation:

P = 1,300,000, Applied power (W)

D_p = 1.6, Actual diameter of propeller (m)

D_0 = 1.1, Effective diameter or propeller, considering propeller without nozzle (m)

z_p = 2.7, Distance between the propeller axis and the bed (m)

ρ_w = 9810, Density of water (N/m^3)

Equation for velocity behind propeller (efflux velocity):

$$u_{p,0} = 1.15\left(\frac{p}{\rho_w {D_0}^2}\right)^{1/3}$$

This results in a propeller efflux velocity of 5.5 m/s. Note that there is no scour estimated from this velocity as the ship will not stay in the same place long enough to cause propeller efflux related erosion.

The maximum velocity along the riverbed ($u_{p,\max bed}$) is calculated using equation 4.190 of CIRIA (C683).

$$u_{p,\max bed} = c\, u_{p,0}\left(D_0/z_p\right)^n$$

$u_{p,0}$ = 5.5, Propeller efflux velocity (m/s)

D_0 = 1.1, Effective diameter or propeller

z_p = 2.7, Distance between propeller centerline and the bed (m)

c = 0.3, Empirical coefficient (Blaauw and Van der Kaa, 1978)

n = 1, Empirical coefficient (Blaauw and Van der Kaa, 1978)

The maximum bed velocity is 0.7 m/s.

5.0 MARINE STRUCTURES

5.1 Structure Description

Currently, three marine structures are planned in the Fraser River along the Marine Yard. The Marine Trestle Structure will be the most upstream structure. While the Conveyor and Roll-on/Roll-off structure will be protected from floating drift and ice flows by the Marine Access Trestle, neither structure should rely on the Marine Access Trestle being in place to allow for future operational flexibility. As a result, the scour and hydraulic loading at all structures is identical.

5.2 Hydrotechnical Results

The results of the hydraulic modeling (HEC-RAS) and the shipping induced waves and water movement are summarized in table 5.1. The HEC-RAS model results shown are maximum values during the model run. The design flow is the flow for the 100-year event with the assumption of moderate climate change impact and a 0.5m SLR.

Table 5.1: Marine Structures Hydraulic Modeling Results

Parameter	HEC-RAS	Shipping Induced
Design Flow	19,400 m^3/s	
Depth Averaged Maximum Velocity	**1.6 m/s**	
Maximum Bottom Velocity	**1.6 m/s**	0.7 m/s
Depth in waterway opening	**13.0 m**	
Maximum Water Surface Elevation*	**4.1 m**	
Froude Number	**0.15**	
Wave Height		**0.6 m**

* Elevations are reported on the CGVD2013 datum

Numbers shown in bold are the critical conditions and have been used in the analysis.

5.3 Scour

5.3.1 General Scour

Contraction scour (also called general scour) occurs when the velocity of the water increases due to a reduction in flow area. The Fraser River contracts while flowing under the Port Mann Bridge, then expands along the project location. Contraction scour is therefore not considered for this project.

5.3.2 Local Scour

Local Scour occurs because of turbulence and vortices caused by obstructions in the river. Typical riverine obstruction related to bridges are abutments (Abutment scour) and piers (Pier scour). No abutment-like features are included in the design. The structural design will have piers between 1.07 m and 1.83 m in diameter and assumed to be spaced at least 4 meters apart. Dolphin piles with a diameter between 0.61 m and 1.22 m will also be included in the design. Conservatively, an angle of attack of 5 degrees was assumed. Additionally, based on an interview with the yard’s operation team it is unlikely that debris will be removed in a timely manner if it was to accumulate against the structure. Therefore, debris accumulation on the scour is included by using an effective pier width that includes debris.

Using equation 7.3 from HEC-18, the required parameters to assess pier scour (y_s) are the following:

$$\frac{y_s}{a} = 2.0K_1K_2K_3\left(\frac{y_1}{a}\right)^{0.35} Fr_1{}^{0.43}$$

where:

y_1 = 1.7, Flow depth directly upstream of the pier (m)

K_1 = 1.0, Correction factor for pier nose shape (-)

K_2 = varying from 1.3 to 1.43, Correction factor for angle of attack (-). There is one K_2 per each column size. It has been considered that at a minimum, columns will be spaced 4 meters apart. K_2 was obtained with the following equation:

$$K_2 = (\cos\theta + \frac{L}{a}\sin\theta)^{0.65}$$

K_3 = 1.1, Correction factor for bed condition (-)

a = varying from 610 to 1067 mm, Pier width (m)

Fr_1 = varying from 0.10 to 0.15 mm, Froude Number directly upstream of the pier as obtained from the HEC-RAS model.

a^* = varying from 773 to 1197 mm, equivalent pier width for debris impact assessment (m). Obtained using equation 4.16 from TAC (2001).

$$a^* = \frac{0.5\,bt + a(y_1 - 0.5t)}{y_1}$$

b = varying from 1.83 to 5.49, Width of debris accumulation (m). It has been considered that width of debris includes the pier's own diameter plus one diameter to both the right and left of each column.

t = 2, Depth of debris accumulation (m), assumed constant for all piers.

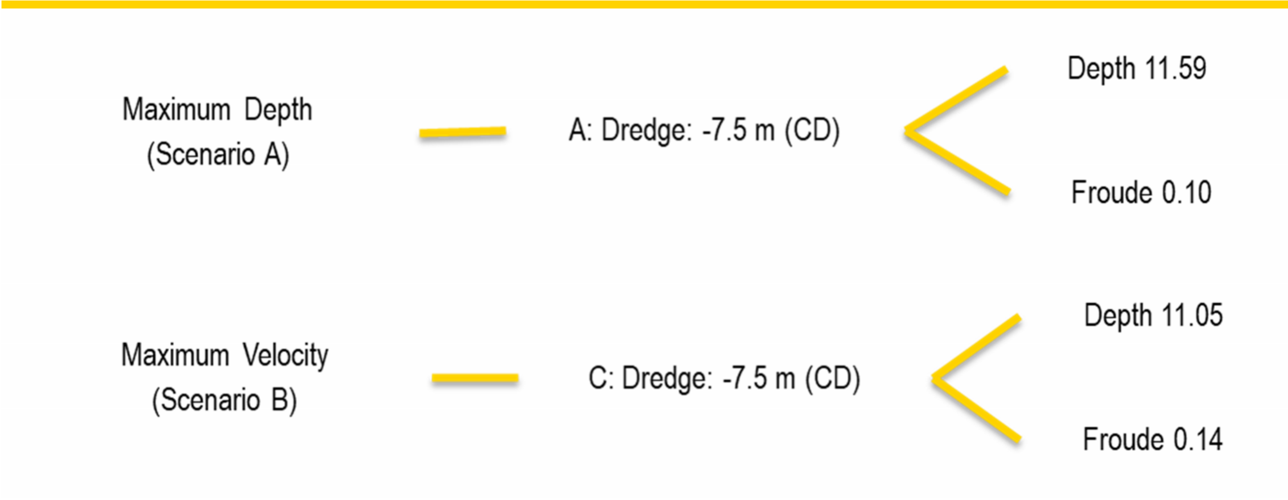

Exhibit 5.1: Evaluated scenarios for local scour due to piers

Table 5.2: Local scour calculated parameters (b and a*) and scour results

Diameter (mm)	b (m)	a* (m)	Scour Scenario A (m)	Scour Scenario B (m)
610	1.83	0.72	2.62	2.99
910	2.73	1.07	3.40	3.88
1067	3.20	1.25	3.77	4.30
1219	3.66	1.43	4.11	4.69
1829[1]	5.49	2.14	3.58	4.19

Accounting for debris accumulation and different dredge scenarios, the maximum local scour depth that could be developed ranges from 2.62 m to 4.69 m for the area where the dredge reaches -7.5 m (CD), and from 3.58 m to 4.19 m for the Roll-on/Roll-off ramp location. Local scour depth will be given as a recommendation given these obtained values.

5.3.3 Natural Scour

Scour is a natural phenomenon due to processes of channel adjustment. The effect of natural scour processes on a structure depends on the location of the structure relative to the active channel. If the structure is in a river bend, higher velocities, and secondary currents in the outside of bend can result in deeper sections of river along the outer banks. The project location is on the inside of a slight bend. Bend scour is therefore not considered for this project.

Rivers are also subject to bedform scour due to bed load transport in the river. Bed load in a river moves down the river as a series of dunes with troughs and crests. The troughs and crests move through the river as a wave, with the wavelength and height being a function of the water velocity. As the trough moves along a structure, this can increase the total scour depth. Due to the lower velocities along the banks, the bedform scour will be minimal.

The empirical equation on bed form for ripples, dunes, and transition is given by Karim (1999):

$$\frac{\delta}{y} = \left[\frac{\left\{S - 0.0168\left(\frac{D_{50}}{y}\right)^{0.33} F^2\right\}\left(\frac{L}{y}\right)^{1.20}}{0.47F^2}\right]^{0.73}$$

This equation has been refactored considering manning's equation as follows:

$$\frac{\delta}{y} = 1.735\left[\frac{gn^2}{y^{\frac{1}{3}}} - 0.0168\left(\frac{D_{50}}{y}\right)^{0.33}\right]^{0.73}\left(\frac{L}{y}\right)^{0.876}$$

[1] The 1829 piles are only used at the towers of the Roll-on/Roll-off ramp. Location specific velocity and depth were used to determine the scour at these piles. The design velocity was 0.6 m/s, and the depth is 4.5 m.

Where:

δ = bedform height (m)

y = 10.54, flow depth (m)

D_{50} = 0.000035, median size of bed sediments (m); according to Rennie (2004).

g = 9.81, gravity acceleration (m/s^2)

n = 0.02, manning's roughness (-).

The ratio of bedform height to depth is 0.06, resulting in a maximum bedform height of 0.65 m. The scour is half the wave height, resulting in bedform scour of 0.32 m. Bedform scour depth will be given as a recommendation given this obtained value.

5.3.4 Channel Degradation

Channel degradation is the progressive lowering of the channel. Channel degradation is commonly associated with man-induced changes in the river or upstream watershed. No long-term channel degradation has been included in the scour calculations.

5.3.5 Total Scour

Considering the impact of each scour process individually and adding the impacts might be overly conservative. TAC chapter 4.4.8 indicates that natural scour should only be added to the contraction and local scour if warranted by site conditions. It further states that adding local scour to the maximum general scour could result in overly conservative results as scour patterns from the different sources might overlap. Furthermore, for natural scour, bedform scour tends to occur on the inside of bends, whereas bendway scour occurs along the outside bank.

5.3.6 Scour Depth

Based on the results obtained in section 5.2.2, recommended design scour from piers is shown in Table 5.3 along with bedform scour results. The total scour depth is the sum of the bedform scour and the pier scour. The scour depth is considered a design case condition. Due to the uncertainty in scour processes, the depths have been rounded up to the nearest 0.5m.

Table 5.3: Scour depth results

Diameter (mm)	Design Pier Scour Depth (m)	Bedform Scour Depth (m)	Total Scour Depth (m)
610	3.5	0.5	4.0
910	4.0	0.5	4.5
1067	4.5	0.5	5.0
1219	5.0	0.5	5.5
1829	4.5	0.5	5.0

5.4 Hydrotechnical Design Parameters

5.4.1 Design Water Elevation

The design event is the 100-year (1% AEP) discharge on the Fraser River with a SLR of 0.5m and moderate climate change. The peak water elevation is 4.1 m CGVD2013, corresponding to 5.1 m CD at the Marine Structures.

5.4.2 Longitudinal Stream Pressure

The longitudinal stream pressure on the piers during the alternative scour scenario is calculated based on the channel conditions during the design event. Using the equation provided in Section 3.11.4.1 in CSA S6 and considering the largest pier size of 1067 mm, the required parameters to assess the longitudinal stream pressure are the following:

$$P_{longitudinal} = C_D \rho \frac{Av^2}{2}$$

$P_{longitudinal}$= Longitudinal stream force (kN)

C_D = 0.7, Round pier (-)

ρ = 1, Density of water (ton/m^3)

y = 13.0, Depth of approach flow (m)

a = Pier diameter (m) A = area of a pier exposed to flowing water (m^2)

v = 1.6, Maximum channel velocity at the project location obtained with the HEC-RAS model (m/s)

Table 5.4: Longitudinal Stream Force

Diameter (mm)	Exposed Area (m^2)	Longitudinal Stream Force (kN)
610	7.9	**7.1**
910	11.8	**10.6**
1067	13.9	**12.4**
1219	15.8	**14.2**

5.4.3 Lateral Stream Pressure

The lateral stream pressure on the piers during the alternative scour scenario is calculated based on the channel conditions during the design event. Using the equation provided in Section 3.11.4.2 in CSA S6, the required parameters to assess the lateral stream pressure are the following:

$$P_{lateral} = C_L \rho \frac{Av^2}{2}$$

$P_{lateral}$= Lateral stream force (kN)

C_L = 0.5, 5-degree angle of attack assumed (-)

ρ, y, a, A and v are defined in section 5.4.2

Table 5.5: Lateral Stream Force

Diameter (mm)	Exposed Area (m^2)	Lateral Stream Force (kN)
610	7.9	**5.1**
910	11.8	**7.6**
1067	13.9	**8.9**
1219	15.8	**10.1**

5.4.4 Wave Action

Wave action on the foundation elements is included based on paragraph 3.11.5 of CSA S6. The required wave height necessary for its evaluation is considered as the Primary Ship wave.

$$F_{wave} = 10H_w^2$$

F_{wave}= kN, Wave impact force

H_W = Wave Height [Primary ship wave] (m)

The calculated wave height has a value of 0.6m, resulting in a wave impact of 4 kN. The impact of this force acts at $H_w/2$ = 0.3m above the still water elevation.

5.4.5 Wind

As stated in CIRIA C683 paragraph 4.11, wind generated waves in the Fraser River are limited and typically do not govern in terms of shore protection design. Therefore, it has not been evaluated or included in this report.

5.4.6 Debris and Floating Drift

The Fraser River transports many trees during the freshet season. A floating log will be used to estimate the impact load on the structure. The characteristic log is assumed to be 10 meters long, with a diameter of 0.6m and a density of 610 kg/m^3. The reported HEC-RAS velocities (See table 5.1) are depth averaged velocity and should be multiplied by 1.2 to calculate the water surface velocity. It can be assumed that the log will travel at the water surface velocity.

5.4.7 Ice Loading

While the Fraser River typically transports large pieces of ice in the winter months and during freshet season, it is less common for the river to freeze over at the project location.

5.4.7.1 Ice Thickness

The expected ice thickness can be calculated with the Stefan equation:

$$T = C\sqrt{FDD}$$

T = cm, Ice Thickness

C = -, coefficient based on local conditions

FDD = °C, Freezing Degree Days

A Freezing Degree Day is the absolute average temperature of day with below freezing temperatures. The FDD is the sum of each individual day for a season. The climate atlas of Canada provides the average FDD for regions and municipalities in Canada. However, designs should be based on events that are much less likely to happen than every other year. The climate atlas also provides the data underlying their maps, including for the city of New Westminster. While the Marine Yard is not in New Westminster, the close proximity makes it reasonable to assume identical climatological conditions.

The city of New Westminster has a 64-year record of FDD, starting in 1950. Three years show no freezing degree days, and the maximum FDD is 248 °C in 1950. The next largest FDD is 120 °C in 1985. The data fitted best to an Exponential function, and the FDD with a return interval of once in 100 years is 186 °C.

Using the RCP4.5 scenario for the impacts of climate change, the climate atlas indicated a reduction of 43% in the number of FDD for the next 30 years. While current observations are more in line with RCP8.5, it is more conservative to use a more limited impact of climate change. The FDD with a return interval of once in 100 years under RCP4.5 is 106 °C.

Rivers tend to see less ice formation than lakes, and snow cover tends to limit the growth of ice. The tidal influences are also expected to break up newly formed ice, reducing the growth due to incoming frazil ice. A local condition coefficient of 1.5 is selected to reflect these conditions. This result in a once in 100-year ice thickness of 20 cm under current conditions and 15 cm based on the RCP4.5 scenario.

5.4.7.2 Dynamic Forces

The effective ice strength is based on large pieces of ice that are internally sound with a crushing strength of 1,100 kPa. Internally sound pieces of ice are expected in winter prior to the onset of the large freshet flows. The FBC indicated a maximum water level in winter of 2.8m CGVD13 for a 50-year streamflow event. Winter flows are typically much less, the lower water level where ice impact should be considered is -1.0m CGVD13.

5.4.7.3 Static Forces

While the river freezing over is uncommon, it has happened in the past. The piers under the trestle are subject to unbalanced freezing, but at temperatures close to the freezing point. The equation in CSA S6, Section 3.12.3 is only valid for ice temperatures significantly below the freezing point. Transport Quebec provides an alternative equation that can be used.

$$F = 245\, K\sqrt{h} \leq 184$$

F = kN/m, Static Ice Force

K = (1 + h/3b) < 2.57

h = m, ice thickness

b = m, width of contact zone

An ice thickness of 0.2m is recommended for design purposes. For design purposes, the top of ice can be considered at the water surface elevation. The static forces are expected to happen from low water level (-1.0m CGVD13) up to the seasonal high-water level of 2.4m CGVD13.

5.4.7.4 Lateral Thrust

Due to the planned spacing of the trestle piers, floating ice could get caught and form an ice dam. The expected thickness of the ice dam is 2m and pressure of 10 kPa should be considered to act against the piers.

5.4.7.5 Ice Jacking

The thickness for the ice around the pier is estimated to be 0.2m for design purposes. The resulting force can push up or pull down because of fluctuating water levels. The equation provided in CSA S6, Section 3.12.5, results in an ice mass around the pier with a diameter of near 10 meters. Given the configuration of the trestle foundation, this mass will be impacting more than a single pier.

Assuming an area of ice of 5m x 5m carried by a single pier, with a thickness of 0.2m, the total weight of the ice is 44 kN, assuming a unit weight of ice of 900 kg/m^3. During rising water levels, the ice can become submerged, resulting in a buoyant force. The resulting upward pressure is 5 kN per pier.

5.5 Loads During Construction

No additional hydrotechnical loads are expected during construction of the trestle.

6.0 SHORE PROTECTION

6.1 Hydrotechnical Results

The results of the hydraulic modeling (HEC-RAS) and the shipping induced waves and water movement is summarized in table 6.1. The HEC-RAS model results shown are maximum values during the model run. The design flow is the flow for the 100-year event with the assumption of moderate climate change impact and a 0.5m SLR.

Table 6.1: Shore Protection Hydraulic Modeling Results

Parameter	HEC-RAS	Shipping Induced
Design Flow	19,400 m^3/s	
Velocity at Embankment Toe	**1.2 m/s**	0.7 m/s
Depth at Embankment Toe	**8.0 m**	
Maximum Water Surface Elevation*	**4.1 m**	
Wave Height		**0.6 m**

* Elevations are reported on the CGVD2013 datum

Numbers shown in bold are the critical conditions and have been used in the analysis.

6.2 Design Guidance

CIRIA C683 provides recommendations for using riprap to protect the facility from wave action. The recommendations provided riprap sizing, gradation, design, and layout. In lieu of bed protection, the revetment slope was extended as indicated in section 6.3.4.

The extent of the revetment was defined using two vertical limits. For the upper limit, the design water elevation plus the considered freeboard was selected. The design event is the 100-year (1% AEP) discharge on the Fraser River with a SLR of 0.5 m and moderate climate change. No additional waves are considered during this scenario as the yard is not expected to be operational. With these considerations, the upper vertical limit stands at 5.4 m (CD).

For the lower limit, the wave run down during the low-low level was evaluated. The following figure (Figure 5.2 within the original reference) has been extracted from Chapter 5 of CIRIA C683 to describe the parameters.

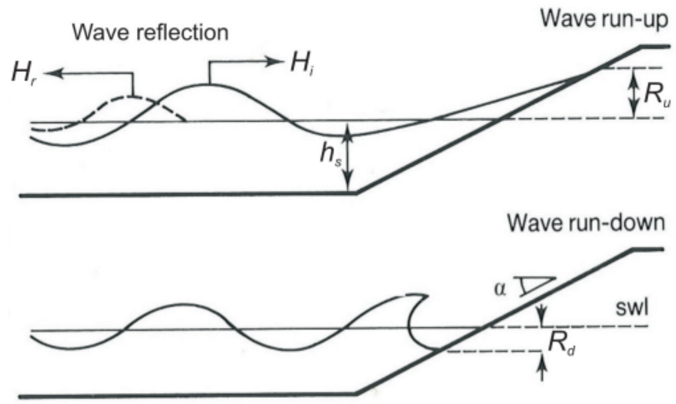

Exhibit 6.1: Hydraulic interactions related to waves and governing parameters (for wave run-up and wave-run down calculations).

With all preceding considerations, wave run down equation 5.25 from CIRIA C683 was used.

$$\frac{R_{d1\%}}{h_s} = 0.34\xi_p - 0.17$$

h_s = 4.0, Depth to SWL (m). In this case, measured as implied on Exhibit 6.1.

ξ_p = 3.06, Surf similarity parameter (-). ξ_p was obtained with the following equation:

$$\xi = \tan\alpha \,/\, \sqrt{H_i/L_i}$$

Where α is the slope angle of the structure and H_i & L_i are secondary ship wave height and length respectively.

$R_{d1\%}$ = Maximum run-down level (m)

With these values a lower limit of -3.5 m (CD) was determined. From these values it was considered that the riprap should extend from 5.4 m CD to -4m CD. See Figure 6.2.

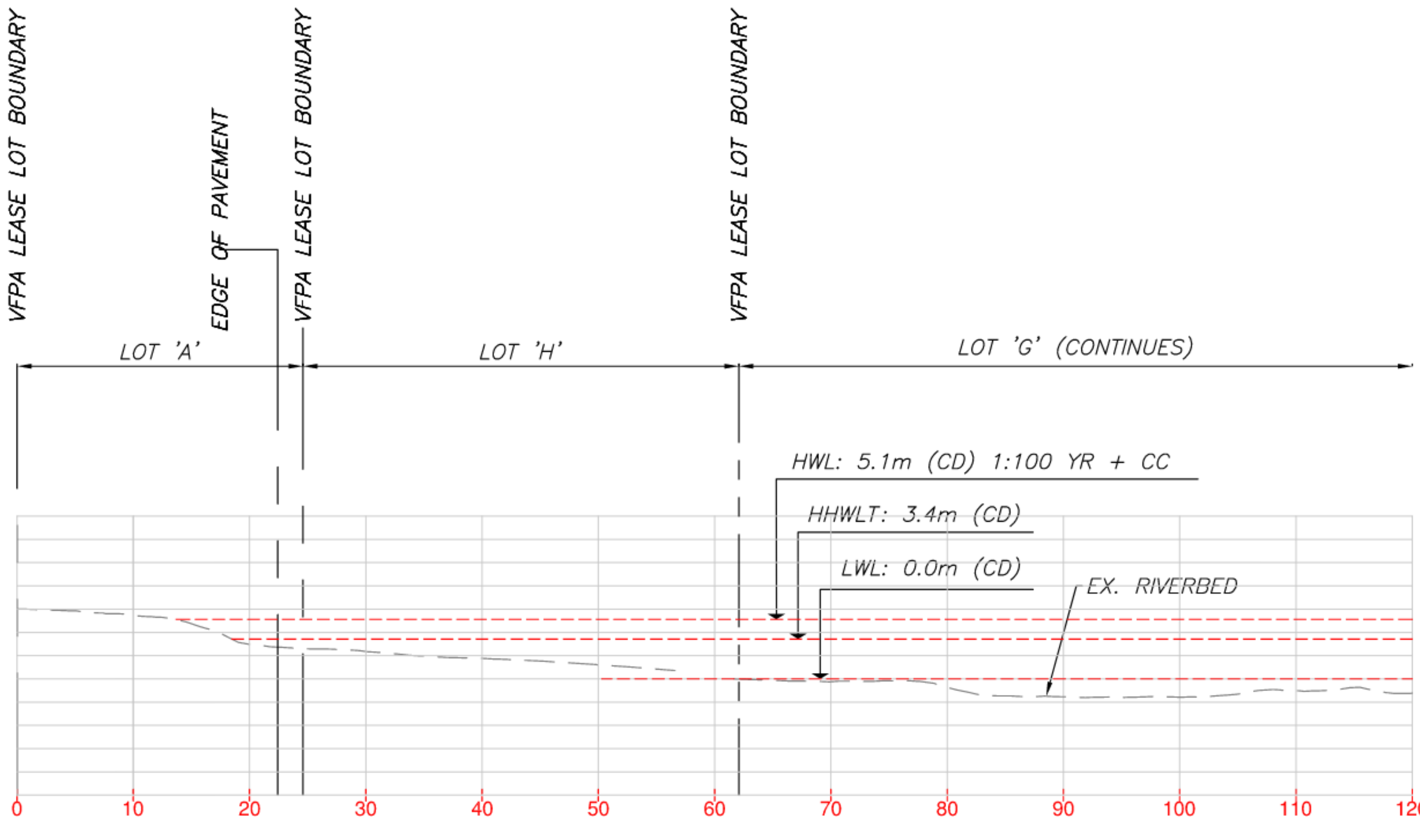

Exhibit 6.2: Upper and lower limits for design

6.2.1 Significant Wave Height Design

The significant wave height design was obtained by using the methodology described in CIRIA C683, chapter 4. In this analysis, the stern wave height produced the significant wave height design with a value of 0.6 m. For further details see chapter 4 of this report.

6.2.2 Relative Buoyant Density (Δ)

The density of an individual rock is assumed to be 2,500 kg/m^3 which is 2.5 times the density of water (specific gravity). Given this value, a relative buoyant density of 1.5 (dimensionless), was considered for calculations.

6.2.3 Stability Coefficient (K_D)

A stability coefficient of 2 was selected as a revised stability coefficient value for breaking waves on the foreshore.

6.2.4 Side Slope Cotangent (K1)

A side slope cotangent value of 2 is used to reflect a 2H:1V side slope. A flatter slope would increase this factor thus reduce the required riprap size. However, this slope has been deemed suitable for constructability purposes.

6.2.5 Rock Diameter

The result from the riprap sizing equation 5.134 from CIRIA C683 is the D_{n50}, which is the median nominal diameter of the riprap grading (m)

6.2.6 Water Depth (h)

This is the water depth in front of the toe, required to estimate if the toe is acting as a toe or a berm. By identifying the appropriate behavior, it is possible to select the appropriate toe sizing equation. Similarly, the depth of the toe below the water level ($h_{t,min}$) is also used for these calculations.

6.2.7 Damage Number (N_{od})

The damage number is equivalent to the number of displaced armor units within a strip of breakwater slope of width Dn, where Dn is the nominal diameter of armor unit, defined as the equivalent cube size of the unit concerned.

6.3 Riprap Sizing

6.3.1 Characteristic Size

CIRIA C683 provides recommendations for sizing armorstone on the slopes based on waves and velocities. Velocities have been disregarded given that port operations prevent ships from using full speed nearby the project area and natural riverine velocities on the Fraser River are considered small even during the design event.

The characteristic size was evaluated using equation 5.134 from CIRIA C683.

$$\frac{H_s}{\Delta D_{n50}} = \frac{(K_D cot\alpha)^{1/3}}{1.27}$$

H_s = 0.6, Significant wave design height (m)

Δ = 1.5, Relative buoyant density of the stone (-)

K_D = 2.0, Stability coefficient (-).

$\cot\alpha$ = 2, Side slope of 2:1 (-)

D_{n50} = Nominal diameter (m). Defined as the median equivalent cube size for design purposes.

With these values a nominal diameter of 320 mm was determined for the required riprap to protect the slopes.

A preliminary selection of toe height of 1 meter, allows to estimate if it is performing as a toe or a berm.

$$\frac{h_{t,min}}{h} \geq 0.5$$

h = 9.1, Water depth in front of the toe (m)

$h_{t,min}$ = 8.1, Depth of the toe below the water level (m)

Using these values, the ratio is calculated to be 0.89 which indicates that is performing as a toe and is in the acceptable range to use equation 5.188:

$$\frac{H_s}{\Delta D_{n50}} = (2 + 6.2(\frac{h_t}{h})^{2.7})N_{od}{}^{0.15}$$

H_s = 0.6, Significant wave design height (m)

Δ = 1.5, Relative buoyant density of the stone (-)

h = 9.1, Water depth in front of the toe (m)

$h_{t,min}$ = 8.1, Depth of the toe below the water level (m)

N_{od} = 2.0, Damage number (-)

D_{n50} = Nominal diameter (m)

With these values the nominal diameter required for the toe is smaller than the nominal diameter required for the slope. However, for ease of construction and replacement if needed, the same size will be used for protecting both the slope and the toe.

Considering the previous analysis, to protect the facility from wave action caused by passing vessels, rock with a nominal diameter equal or larger than 320mm should be used. A rock with a nominal size of 320 mm has weight of 82 kg. This would require a gradation equivalent to the MoTI 100 kg class.

6.3.2 Riprap Gradations

The MOTI standard riprap classes are widely graded, resulting in the need for very large rocks to meet the characteristic size (D_{50}) for a stable revetment. Alternative gradations would result in a reduction in large rocks, as well as a reduction in the required layer thickness.

A series of approximate average dimension of an angular rock for each specified rock class mass (s=2.640) can be found in the 2018 Design Build Standard Specifications for Highway Construction, an extract from Table 205-B from this report is shown below.

Table 6.2: Riprap Gradations (MoTI 100kg Class)

Class (kg)	Approximate average dimension (mm)		
	15%	50%	85%
MoTI 100kg	200	425	610
Minimum size	190	400	475
Maximum size	265	465	665

6.3.3 Revetment Design

The revetment is designed using an approved gradation with a toe deep enough to avoid being undercut by scour during the design event or shipping operations. The revetment is located outside the area impacted by the pier scour, so only the bedform scour is considered (see Section 5.3.3 from CIRIA C683). The scour depth along the toe of the embankment is 0.1m.

6.3.4 Revetment Layout

The riprap layer thickness was determined using the recommendations from Section 6.1.4.2 from CIRIA C683. Such recommendations include a thickness layer defined as:

$$Thickness = 2\ K_t D_{n50}$$

Where Kt is the layer coefficient which takes account of the layer-packing density. The layer was considered as single and the placement type was considered as dense, with these selections and considering irregular rock, a Kt of 0.82 was selected. Given the nominal diameter of 320 mm obtained in the previous section, the riprap thickness is equal to 525 mm. The required thickness is less than the nominal thickness for the MoTI 100 kg gradation which calls out for 700 mm. While narrower gradation could reduce the required thickness, 700 mm is used for design purposes.

Given the shape of the embankment, it is possible for waves to break on the toe structure, therefore the selected configuration corresponds to that shown on Figure 6.18 (d) from CIRIA C683. This configuration considers the breakwater toe to be made by extending the main armour layer.

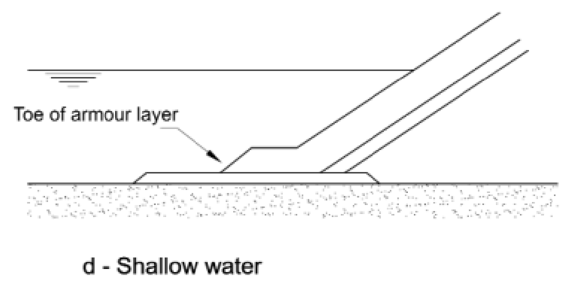

Exhibit 6.3: Schematic example of toe detail from CIRIA C683.

According to CIRIA C683, the toe width, Bt (m), should in general allow at least three stones to be placed. Therefore, the toe width (that in this case is the additional extension of the revetment) is 1 meter and it will provide bed protection.

Considering all previous criteria, the final revetment layout is shown below in yellow. Further detail is provided in Appendix A.

Along most of the marine yard, a bench at approximately 2 meters protects the embankment at higher elevations. Erosion along the marine yard will be a gradual process and the bench provides an opportunity to monitor the erosion over time. As an alternative to providing scour protection to the bottom of the dredging activities, the toe of the scour protection could end on the bench, supplemented by an annual inspection program. If an inspection indicates the erosion is progressing towards the yard, placing the additional riprap to the bottom of the dredged area might be warranted.

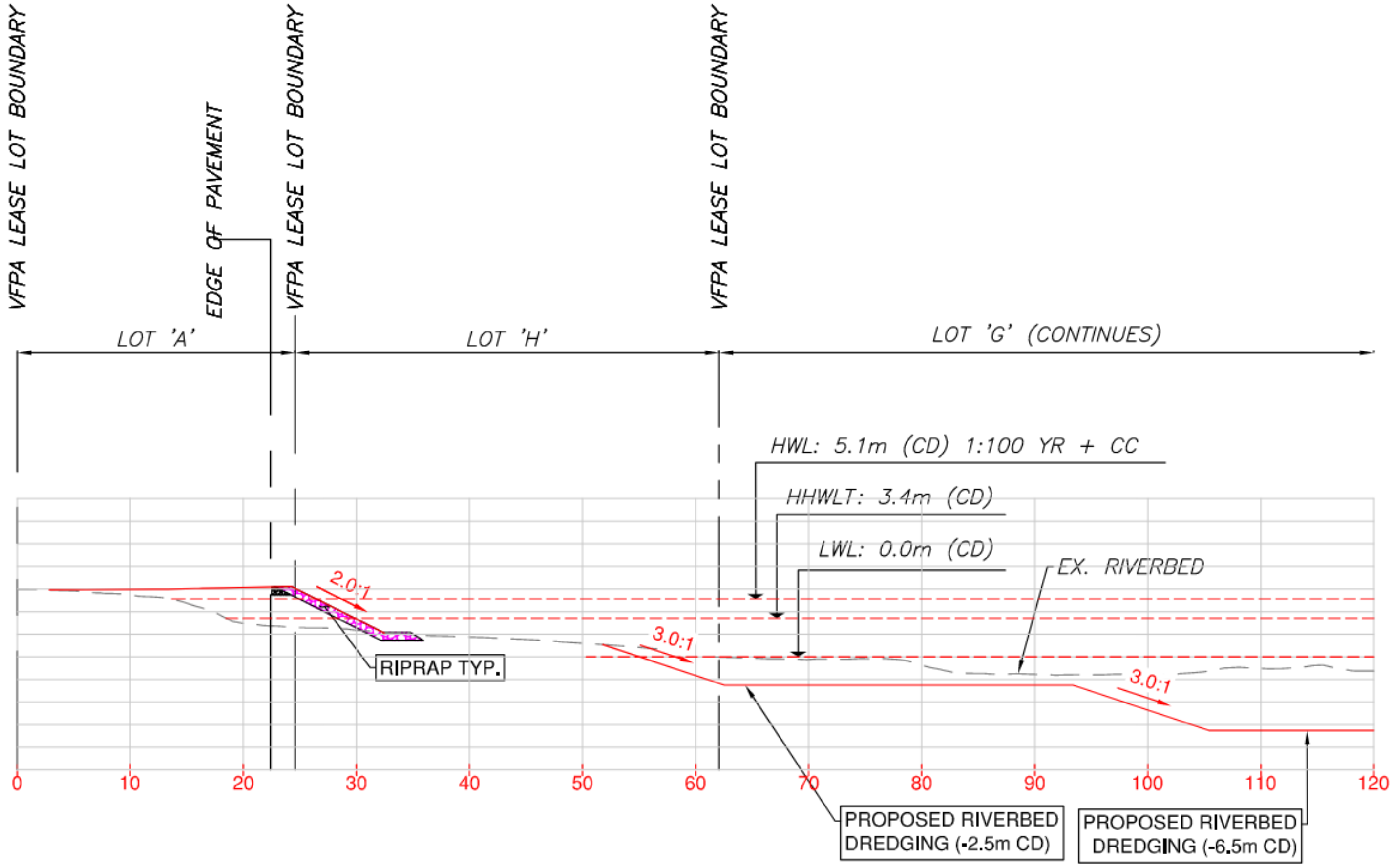

Exhibit 6.4: Revetment layout

6.3.5 Granular Filter Layer

The BC Ministry of Environment Riprap Design and Construction Guide indicates that a riprap layer with a D_{15} of less than 5 times the D_{85} of the native soil would not require a granular filter layer. The gradation of the native soil is pending the geotechnical investigation. Considering the low velocities during the design event and short duration impacts from shipping activities, it is not assumed that a granular filter layer will need to be installed underneath the revetment.

7.0 HABITAT COMPENSATION

Habitat compensation is required for any aquatic habitat destroyed below the seasonal high-water level. A successful habitat restoration has been implemented near the east side of the property. The proposed new habitat compensation for this project will be located at the very west extent of the VFPA water lot, as well as in an area adjacent to the existing habitat restoration (expanding the existing habitat). The design will mimic the successful existing habitat.

7.1 Seasonal High-Water Level

The seasonal high-water level determines the amount of aquatic habitat that needs to be compensated. The marine yard location is impacted by tidal influences and flows on the Fraser River.

7.1.1 Higher High Water Large Tide (HHWLT)

The Government of Canada Tide station (07654) in New Westminster is located about 3 kilometers downstream of the project location. The reported HHWLT is 3.25m relative to the New Westminster chart datum. As the tidal influences will be slightly different between the project and the gauge location, as well as the impact of the discharge in the Fraser River impacted the water surface slope, it is impossible to define a HHWLT conversion between New Westminster and the project location. However, given proximity of the locations, the New Westminster HHWLT provides an indication of what the HHWLT at the proposed marine yard would be.

7.1.2 HEC-RAS simulation

The Fraser Basin Council (FBC) HEC-RAS model was used to estimate the seasonal high-water elevations during high tide. The inflows into the FBC HEC-RAS models were scaled to obtain a peak flow of 8,500 m^3/s at Hope, representative for typical freshet flows. The model was run for a typical freshet season from the middle of May to early June. The highest reported water level at the project location was 2.4m CGVD13, which corresponds to 3.4m in the local chart datum (CD).

7.2 Design Seasonal High-Water Level

An elevation of 3.4 meters (CD) was used for the seasonal high-water level.

8.0 DEBRIS BOOM

To protect the newly created aquatic habitat from floating drift and debris flowing down along the river, a log and debris boom will be added upstream of the habitat offsetting area. The boom will be manufactured and designed by a supplier. This chapter provides the input required to design the boom.

8.1 Location

The boom will be permanently installed in the Fraser River, just upstream of the habitat offsetting area. Salt water is not a concern at this location and the water can be considered fresh water. The booms will be installed mostly perpendicular to the flow but could be angled to direct floating drift and debris away from the habitat areas.

8.2 Water Depth

The debris boom will be operating in an area impacted by tidal patterns and flows on the Fraser River. Water level can vary as much as 5.1m between low Fraser River flows during low tides and high Fraser River flows during high tide.

8.3 Water Velocity

The modeling provided a once in 100-year depth average water velocity of 1.6 m/s. The surface velocity is expected to be 20% higher at 1.9 m/s. While the design criteria for the boom are less than a 100-year event, much smaller events show only slightly reduced velocities and thus using the 100-year velocity is appropriate.

8.4 Wave Action

The Fraser River is an active shipping channel. While wind induced waves are not a concern, shipping induced waves can reach heights of 0.6m.

8.5 Ice Conditions

While it has happened in the past, freezing over of the Fraser River at the project location is uncommon. Static ice loading on the debris boom is not considered a design constraint for the boom. Ice floes coming down the Fraser River could get caught in the boom. However, any impact from ice floes is considered less than impacts from floating debris.

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APPENDIX A:

RIPRAP SPECIFICATION AND DESIGN

Riprap Specification
Source: 2020 Standard Specifications for Highway Construction

Table 205-B: Gradation of Rock by Class of Riprap

Class of Riprap (kg)	Rock Mass (kg)			
	Percentage Smaller Than Given Rock Mass			Max. Size
	15%	50%	85%	
10	1	10	30	50
25	2.5	25	75	125
50	5	50	150	250
100	10	100	300	500
250	25	250	750	1 250
500	50	500	1 500	2 500
1000	100	1 000	3 000	5 000
2000	200	2 000	6 000	10 000
4000	400	4 000	12 000	20 000

Table 205-C: Gradation and Intermediate Dimension of Rock by Class of Riprap

Class of Riprap (kg)	Intermediate Dimension (mm)			
	Percentage Smaller Than Intermediate Dimension			Max. Size
	15%	50%	85%	
10	90	200	285	350
25	125	270	385	450
50	155	340	485	600
100	200	425	610	750
250	270	575	830	1 000
500	340	725	1 050	1 250
1000	425	915	1 325	1 600
2000	535	1 150	1 650	2 000
4000	675	1 450	2 100	2 500

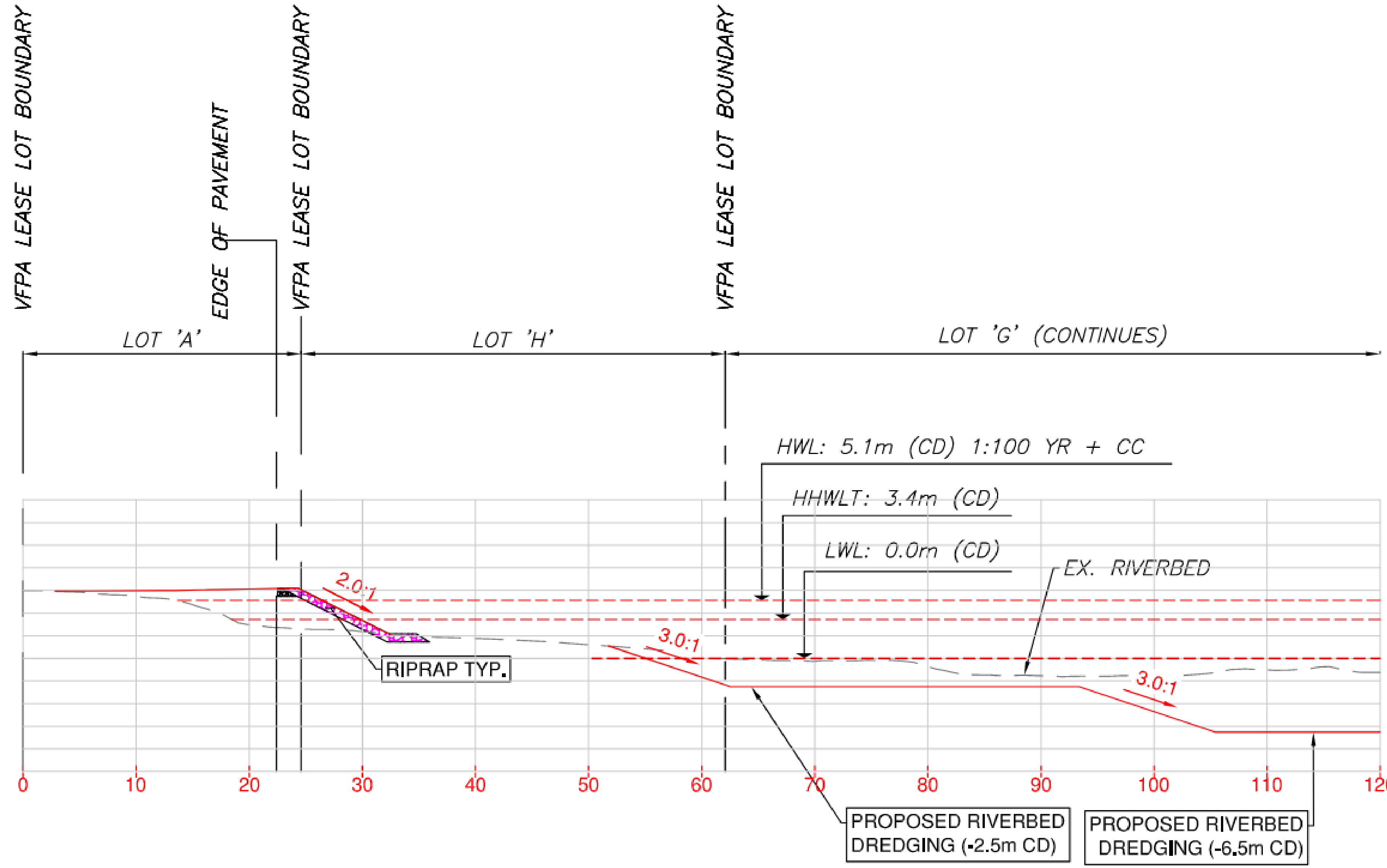
Note: Table 205-C shows the intermediate dimension as defined in the Wolman method as per FHWA FLH T 521 corresponding to the rock mass shown in Table 205-B, based on spherical volume, using Specific Gravity = 2.50. Regardless of actual source Specific Gravity, the dimensions indicated remain applicable (subject to the limits specified in Table 205-A*).*

Table 205-D: Placement Dimensions by Class of Riprap

Class of Riprap (kg)	Nominal Thickness of Riprap* (mm)	Surface Width, W* (mm)	
		2H:1V Slope	1.5H:1V Slope
10	350	783	631
25	450	1006	811
50	550	1230	992
100	700	1566	1262
250	1000	2236	1803
500	1200	2684	2163
1000	1500	3355	2704
2000	2000	4473	3606
4000	2500	5591	4507

* See SS Drawing SP205-1 for the description of the Nominal Thickness and Surface Width dimension "W".

Riprap Detail



APPENDIX B:
FISHERIES ACT AUTHORIZATION MEMO

Kiewit

MEMORANDUM

DATE: March 11, 2024

TO: Kelly Brignall, P.Eng.

FROM: Simon Draijer, P.E., P.Eng., CFM, Rebeca Hernandez

COPY:

SUBJECT: DFO Fisheries Act Authorization – Dredging Impact

DOC. NO: 105716-S00-KIE-HYD-MEM-0001

REVISION: 0

Introduction

Kiewit proposes to develop and expand Kiewit's barge loading facility and marine yard located on the Fraser River at 1950 Brigantine Drive, Coquitlam. This activity is planning to develop a terminal on the Fraser River in Coquitlam. A proposal submitted in October 2023 was reviewed by Fisheries and Oceans Canada (DFO) to determine the effects of the activity on fish and aquatic species at risk. DFO determined that insufficient information was provided to make this determination of both dredging impacts and creation of marsh habitat, and requested additional information. This memo provides the information requested in item 4 and 6 of the DFO letter, an analysis of the potential hydraulic effects of the proposed dredging.

HEC-RAS Model

To understand the hydraulic impacts of the proposed dredging, the project area was simulated in HEC-RAS, an industry-standard software to model flow in rivers. A HEC-RAS model of the Fraser River from Hope to the Salish Sea was obtained from the Fraser Basin Council (FBC). To reduce model run time, the FBC model extent was reduced to a section on the Fraser River, roughly between the Port Mann and Patullo bridges (short model). This short model was used to evaluate the impact of the proposed dredging on river flows, currents, and sediment transport, as well as potential impacts to the habitat just downstream of the site.

The boundary conditions for the short model were extracted from the full model. The model results for the 100-year flood, with moderate climate impact and 0.5m Sea Level Rise (SLR) were used to calibrate the short model.

The CAD design files for the proposed dredging activities, land infill and marsh bench habitat compensation were burned into the FBC terrain model. Breaklines were added to better reflect the proposed bathymetry in the HEC-RAS terrain. Due to the relatively small size of the proposed piers, these were not added to the terrain. The terrain roughness values (Manning's n) from the FBC model were incorporated in the short model and area boundaries adjusted as needed to meet the proposed boundaries.

Model Results

The existing condition short model showed velocities up to 1.6 m/s in the vicinity of the marine yard. The short model with the proposed conditions does not show an increase in maximum velocity near the project location. The proposed dredging, land infill, and marsh bench habitat compensation have limited impact on near-shore velocities, where velocities vary only slightly between existing and proposed conditions. In the picture below, existing velocity contours are shown in red while proposed velocity contours are shown in yellow.

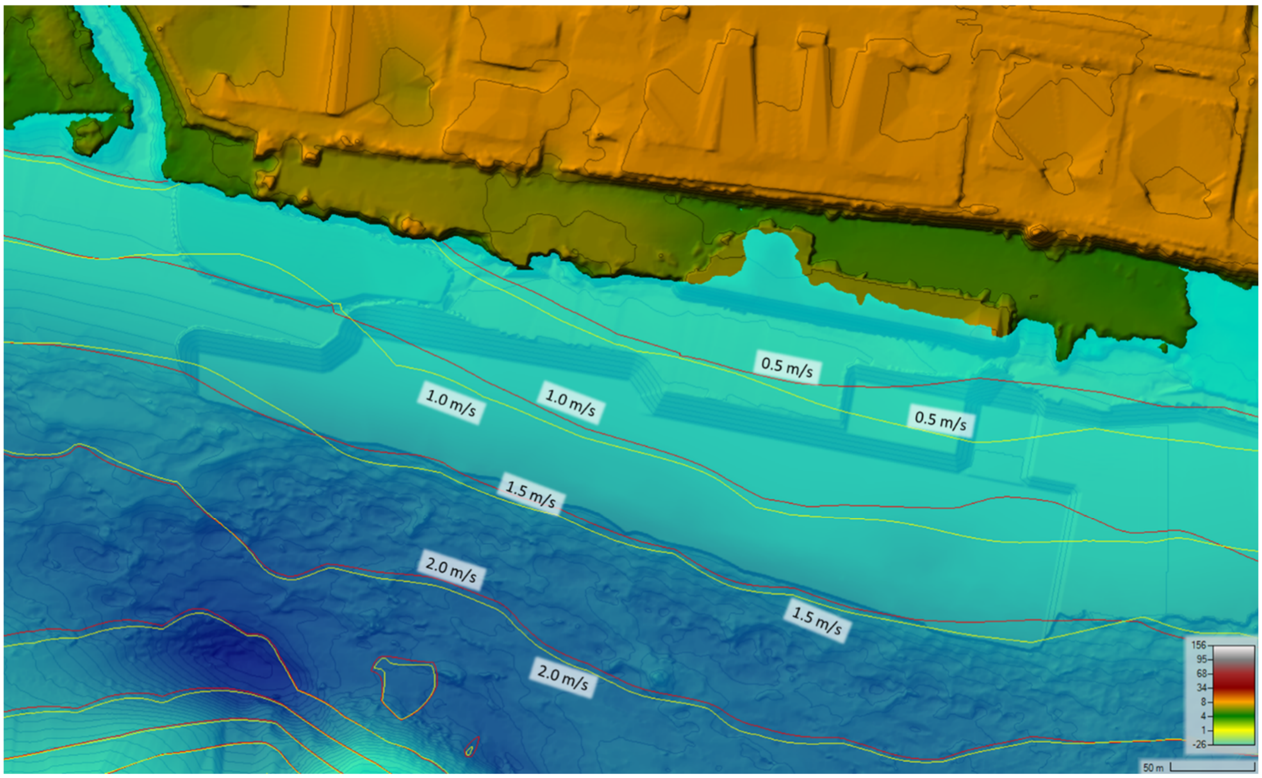


Figure 1. Velocity contour for existing condition shown in red, and for proposed condition shown in yellow. Underlying terrain models shows dredging outline and habitat compensation bench.

Variability in water surface elevation can be considered negligible between existing and proposed conditions as it can be seen in the next image.

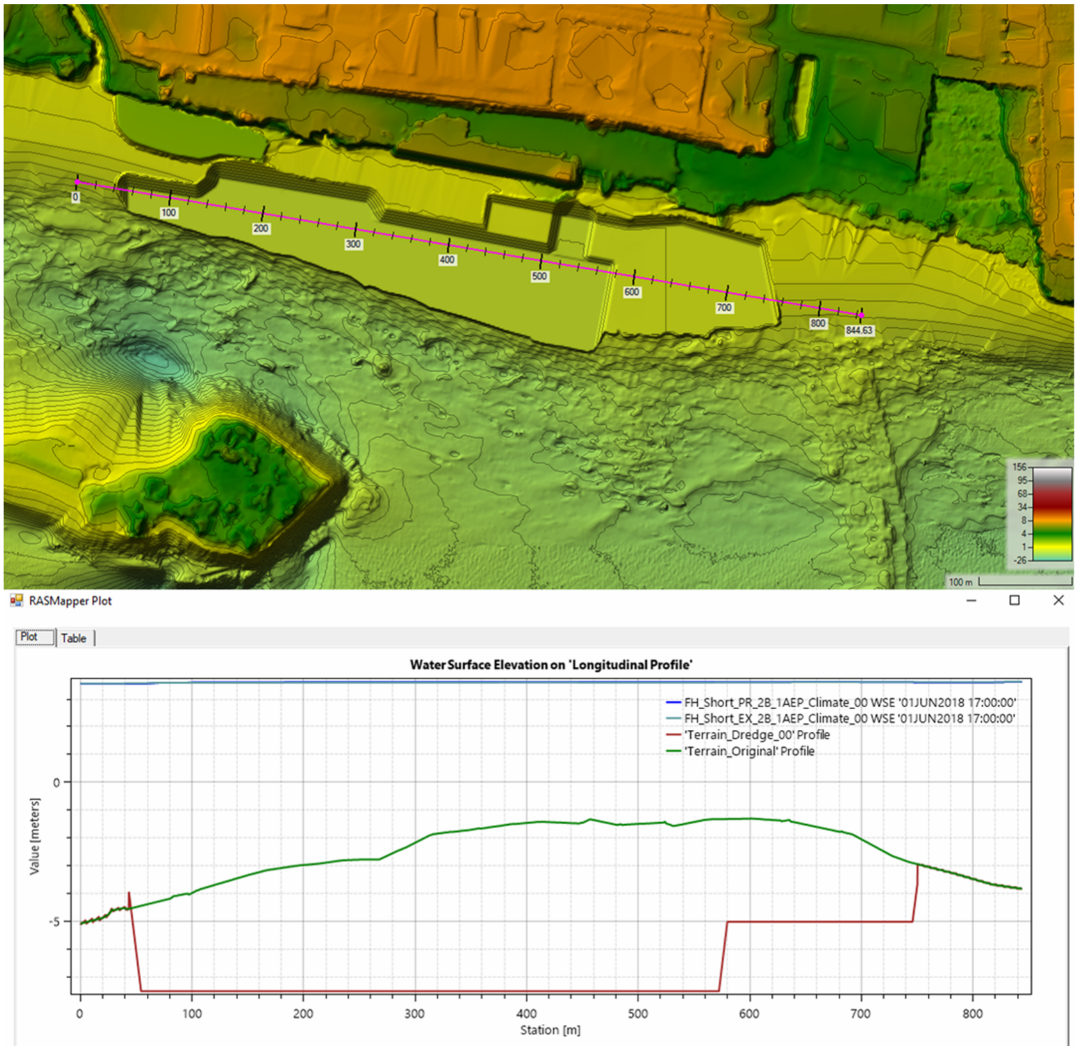


Figure 2. Profile line through dredging area (top). Existing ground (green line), proposed ground (red), and water surface (blue) along the profile line. Note that water surface lines for both existing and proposed conditions are on top of each other.

Between existing and proposed conditions, no significant change in flow pattern was observed, with most of the flow passing through the shipping channel on the south side of the river.

The Fraser River is a mobile bed with the bed elevation set by the equilibrium of the incoming sediment flux and water velocities. Neither the sediment load nor the water velocities are constant and will result in constant adaptation of the riverbed. Given the modest impact of the proposed activities on velocity and current patterns, any change in sediment transport will be insignificant compared to the natural processes. However, it is believed that the dredge area will act as a sediment sink and that maintenance dredging might be required from time to time to maintain the proposed water depth.

The habitat compensation area is near the shore, just downstream of the dredge area. Existing velocities in this area during a large event are approximately 0.5 m/s to 1.0 m/s. In this area, there is a slight decrease in the near-shore velocities between proposed and existing conditions and a slight increase at the riverside edge of the bench. The differences between existing and proposed velocities do not exceed 0.1 m/s during a 100-year event. For more typical events, the differences in velocities will be much less.

Conclusion

Due to the river width at the proposed marine yard, and the main flow path of the river being along the opposite site of the river, the result of the dredging and creation of marsh habitat does not have a significant impact on the flows, currents, and sediment transport in the Fraser River. The habitat created just downstream of the site is protected against river currents, shipping impacts, and debris. As such, it is not expected to experience any impact from flows and sediment transport that would otherwise be present from the equilibrium of natural incoming sediment flux and water velocities. The HEC-RAS models show that the proposed dredging activities do not negatively impact the created habitats or their proposed protection.