CITY OF CAMPBELL RIVER SEA LEVEL RISE STUDY
PHASE 1 – DOWNTOWN WATERFRONT SITE

FINAL REPORT

Prepared for:

City of Campbell River
City of Campbell River, BC

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DISCLAIMER

EXECUTIVE SUMMARY

The City of Campbell River (the City) is located on the east coast of Vancouver Island on Discovery Passage at the northern end of the Strait of Georgia, and along the estuary of the Campbell River. Much of the development in the city is concentrated in lands that are only 4 m above mean water level. The community has faced flood and erosion hazards both along its riverfront from high river flows and oceanfront from king tides and storm surge.

A flood hazard assessment was conducted to determine the 2050 and 2100 flood construction levels (FCL) for the Downtown Waterfront Site by considering the impacts of Sea Level Rise in combination with extreme weather and tide events, investigate the implications of future flooding on existing infrastructure and development and future land use, and provide recommendations on mitigation strategies.

Two methods, the additive approach presented in the 2011 BC Ministry of Environment Climate Change Adaptation Guidelines for Sea Dikes and Coastal Flood Hazard Land Use and joint probability approach, were conducted to establish the FCL. The additive approach normally resulted in conservative values as they do not account for the probability of simultaneous occurrence of the events. A joint probability approach was conducted to reduce the conservatism inherent in the additive approach.

Due to the strong tidally-induced currents that develop in Discovery Passage, the wave field in the passage can change considerably due to wave-current interaction. The most noticeable effect occurs when waves propagate against the current. The analysis utilized combined wave-current models to develop the design wave heights at the project site.

The study shows that the FCL values derived using the joint probability approach is about 1.4 m lower than the values derived using the simplified additive approach. The recommended 2050 FCL values based on the joint probability approach for the armoured riprap shoreline and cobble beach shoreline are 5.3 m Geodetic Datum (GD) and 5.8 m GD, respectively. The recommended 2100 FCL values based on the joint probability approach for the armoured riprap shoreline and cobble beach shoreline are 5.7 m GD and 6.2 m GD, respectively. The existing top of bank is at elevation of about 4.2 m GD which is below the recommend FCL value for current and future conditions. The proposed development is expected to experience flooding during the 200-year recurrence design event. The estimated flood depth in the impacted area is 0.6 m and 1.8 m for 2050 and 2100, respectively.

There are several mitigation options for the project site to reduce the coastal flood hazard. These include the use of setbacks for primary buildings and the careful design of site drainage to allow wave over-topping water to be directed away from key infrastructure. Improvements to the rock revetment are also recommended to both decrease wave run-up elevations and the overtopping velocities of individual wave events.

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1 INTRODUCTION

1.1 Background

The City of Campbell River (the City) is located on the east coast of Vancouver Island on Discovery Passage at the northern end of the Strait of Georgia, and along the estuary of the Campbell River. Much of the development in the city is concentrated in lands that are only 4 m above sea level. The community has faced flood and erosion hazards both along its riverfront from high river flows and oceanfront from king tides and storm surge.

The Province of BC issued Guidelines in 2011 on flood hazard land use management that included direction related to sea level rise (SLR). In 2018 BC amended the Flood Hazard Area Land Use Management Guidelines to account for SLR for year 2100 and 2200 (2004, amended 2018). The Studies by BC Ministry of Environment (2011b) indicate that there will be a significant impact to coastal BC over the next century. Based on a review of scientific literature, global sea level rise from the year 2000 was estimated to be 1 m by the year 2100 and 2 m by 2200. The City recognizes that being a coastal city with limited flood protection infrastructure, the risks can be significant, and that the hazard and consequence posed by anticipated future coastal and river flooding may be better dealt with by using a combination of adaptation strategies, land-use changes and structural and non-structural approaches. Prior to developing such recommendations, the City sought to understand the potential hazard posed by future floods, vulnerabilities in these areas and anticipated consequences.

As part of a two-phase project, the present Phase 1 work focuses on defining the flood hazards for the potential development of the "Downtown Waterfront Site ", a 9.5 acre site located on the downtown foreshore of Campbell River. The site is part of lands claimed from the sea during the 1980's, and is currently being used as an informal parking lot. The development concept for the 9.5 acre site (Figure 1-1) includes:

- Salmon Centre of Excellence
- Aquarium
- Conference hall
- Restaurant and meeting rooms
- Incubator spaces and small food outlet

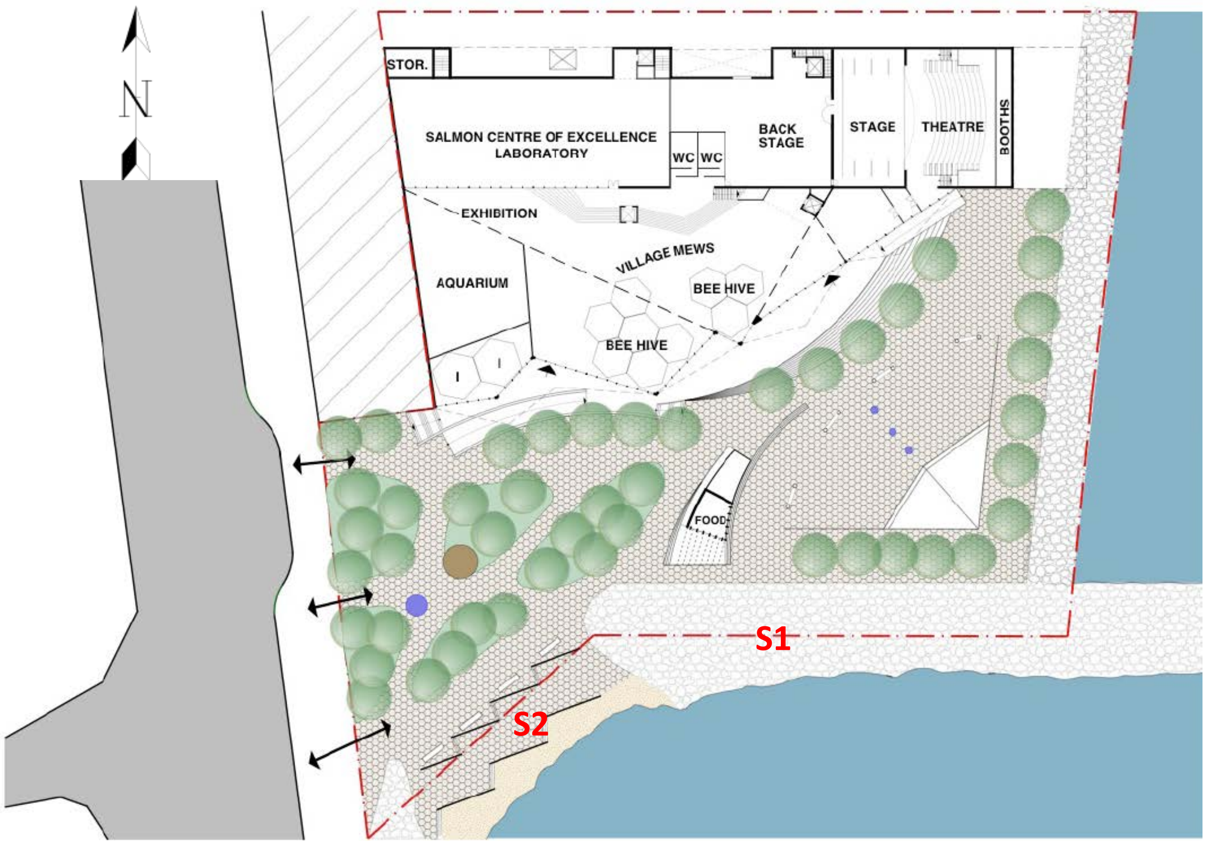

Figure 1-1: Proposed concept development for Downtown Waterfront Site by Cohlmeye Architecture (August 2017)

The City retained Northwest Hydraulic Consultants Ltd. (NHC) to conduct a flood hazard assessment to ensure that the new development is safe. The main objectives of the study are to:

- Determine the 2050 and 2100 flood construction levels (FCL) for the Downtown Waterfront Site by considering the impacts of Sea Level Rise in combination with extreme weather and tide events.
- Investigate the implications of future flooding on existing infrastructure and development and propose mitigations to reduce coastal flood hazard at the Downtown Waterfront Site.

1.2 Flood Construction Level

The water level at the Downtown Waterfront Site is primarily governed by the sea level and incorporates the combined effects of tide, storm surge, wave effect, future sea level rise and local subsidence. The 2011 BC Ministry of Environment Climate Change Adaptation Guidelines for Sea Dikes and Coastal Flood

Hazard Land Use (2011b) presents an approach for developing the flood construction level (FCL) as follows:

$$
\text{FCL} = \left.\begin{array}{l}
\text{Higher High Water Level Large Tide (HHWLT)} \\
+ \text{ storm surge during designated storm} \\
+ \text{ future sea level rise (SLR) allowance and local subsidence} \\
+ \text{ estimated wave effects from designated storm} \\
+ \text{ freeboard}
\end{array}\right\} \text{Designated Flood Level}
$$

Although much of the underlying design storm events used by the method presented by these documents are 200-year events, the probability of simultaneous occurrence of the events is vaguely if at all defined. The *Guidelines for Sea Dikes and Coastal Flood Hazard Land Use – Draft Policy Discuss Paper* (2011a) suggests a probability of 200-year storm surge co-occurring with HHWLT of near 0.025% (4000-year return period). Annual exceedance probability (AEP) of this magnitude are stated by the accompanying policy document (2011a) to be justified where the consequence of dike failure has moderate to high consequence, such as the Fraser River Delta where there is potential for several weeks of disruption, major financial losses for multiple owners, multiple people injured, and multiple loss of life. The proposed Downtown Waterfront Site consists primarily of commercial developments; it is not a continuous sea dike and does not present the same consequence if design water level is exceeded. The design life of the development is likely on the order of 50-years (typical building life) and this consideration is incorporated into the analysis.

Two methods were used to develop the FCL for the Downtown Waterfront Site:

1. Simple additive approach as presented in the Ministry of Environment Guidelines (2011b).

   This approach evaluates the potential water level by assuming that the design tide, storm surge, and wave were to occur at the same time.

2. A joint probability approach using Monte Carlo simulation.

   This approach evaluates the potential water level with consideration of the likelihood that any of design tide, storm surge and wave effect could occur at the same time. (Probabilistic Method, as per Ministry of Environment Guidelines (2011b).

The development concept incorporates two types of shoreline structure: 1) armoured riprap as shown in S1 in Figure 1-1 and 2) cobble beach as shown in S2 in Figure 1-1. Wave effects along each shoreline structure are different and are evaluated separately.

2 SITE CHARACTERIZATION

2.1 Physical Setting

The Downtown Waterfront Site is located on the west side of Discovery Passage, at the north end of the Strait of Georgia (Figure 2-1). At the project site, Discovery Passage is bounded by Vancouver Island to the west and Quadra Island to the east. Water depths in Discovery Passage reach a maximum of approximately 80 m.

Figure 2-1: Project site, wind stations and wave buoy locations.

2.2 Meteorological and Oceanographic Conditions

The water level at the Downtown Waterfront Site is primarily governed by the sea level and incorporates the combined effects of tide, storm surge, wave effect, future sea level rise and local subsidence. These processes are discussed in the following sections.

2.2.1 Tides

Tides near Campbell River are mixed with annual mean tidal range of 2.7 m and large tidal range at 4.9 m. Two months of predicted hourly tidal elevations at Campbell River are shown in Figure 2-2, illustrating the bi-weekly tidal variability.

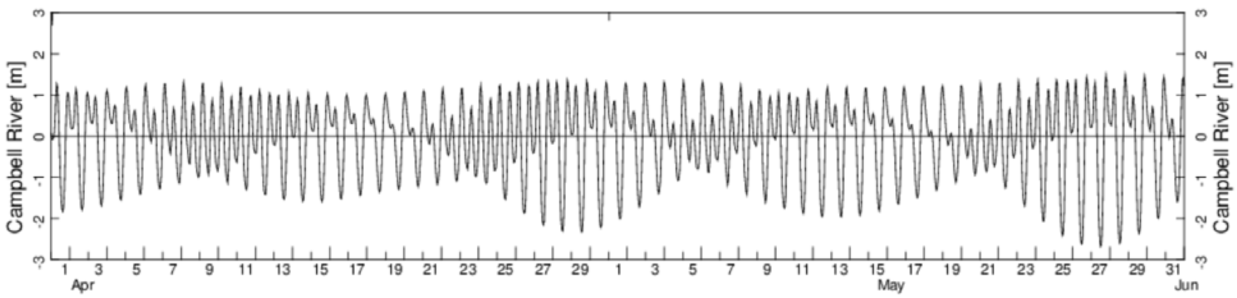

Figure 2-2: Predicted tides at Campbell River from April 1st to May 31st, 2017.

Table 2-1 presents local tidal water levels based on values obtained from Campbell River from 2017 Canadian Tide and Current Tables Volume 6.

Table 2-1: Summary of Campbell River Tide elevations

Sea State	Tide Elevation (m Geodetic Datum)
Higher High Water, Large Tide (HHWLT)	1.7
Higher High Water, Mean Tide (HHWMT)	1.2
Mean Water Level (MWL)	0.0
Lower Low Water, Mean Tide (LLWMT)	-1.5
Lower Low Water, Large Tide (LLWLT)	-2.5

2.2.2 Storm Surge

Storm surge is caused by weather effects (wind setup, wave setup, atmospheric pressure uplift) on the ocean. The design storm surge values were calculated from Department of Fishers and Oceans Station 8074 - Campbell River water level data (1972 to 2016) by first removing the tidal component from the measured water level to obtain the tidal residual. Extreme Value Analysis (EVA) was then conducted using the “Peak Over Threshold” method by considering tidal residual[1] values occurring when tides were greater than HHWLT. The results are summarized in the table below.

Table 2-2: Summary of design storm surges

Return Period (yr)	Storm Surge (m)
1	0.49
2	0.57
5	0.66
10	0.72
20	0.78
50	0.86
100	0.91
200	0.97

Note that the maximum observed water level over the 45 years record was 2.35 m Geodetic Datum (GD) which is 0.65 m above HHWLT. This event occurred on January 15th, 1974.

2.2.3 Sea Level Rise

The sea level rise policy for BC (2011b) recommends assuming a 1 m rise in global mean sea level between the year 2000 and 2100 as show in **Figure 2-3**.

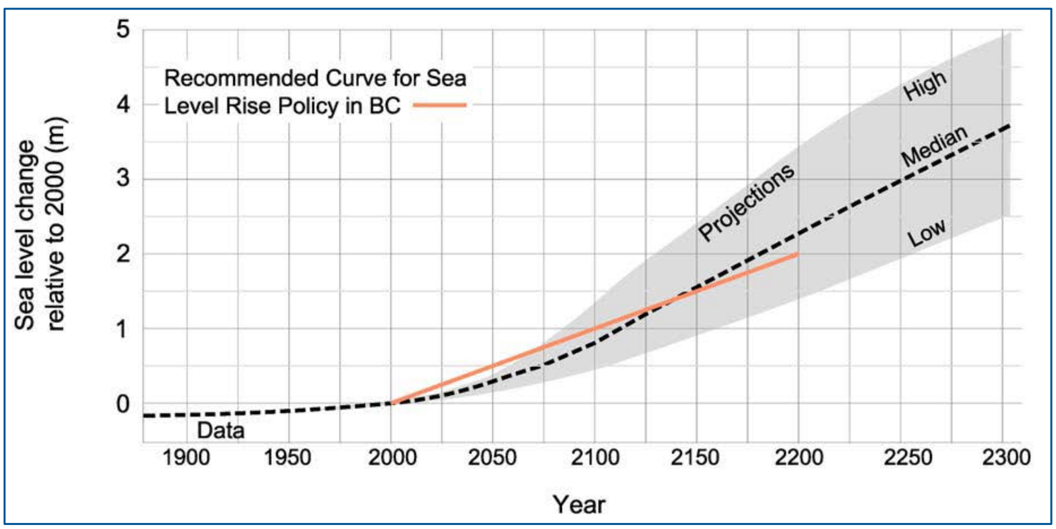

Figure 2-3: Projections of global sea level rise (2011b).

[1] Tidal residual (aka: storm surge) is the difference between the predicted astronomical tide and the actual observed tide levels. This difference is the result of many local, regional and sometimes global environmental factors. The most significant of these factors tend to be atmospheric conditions; specifically wind speed, wind direction and atmospheric pressure.

As part of this study, the impacts of sea level rise were assessed for the years 2050 and 2100. It should be recognized that there is significant uncertainty in sea level rise projections with a range in the rise presented in the draft provincial sea level rise policy and shown in **Figure 2-3**, from about 0.5 m to 1.3 m by 2100 and 1.4 m to 3.4 m by 2200. A 1.0 m sea level rise estimate by 2100 is in the upper range of projections and allows planners to be ahead of the curve. Whereas a 2.0 m rise estimate by 2200 is towards the low to mid-range of projections, it should be recognized that there is considerable uncertainty with estimates nearly two centuries away. Given these uncertainties, reliance on interpolation of simulation results, rather than detailed simulation of finer increments of sea level rise, is considered to be a reasonable and an appropriate approach for intermediate and long-range planning purposes. It is recommended that the City monitor changes in sea level rise estimates and adapt their flood management plans accordingly.

2.2.4 Land Uplift and Subsidence

Uplift refers to the vertical movement of land at a given location. Uplift may be positive or negative. Negative uplift is also known as subsidence. The rate of uplift/subsidence for Campbell River is reported to be at +4.1 mm per year (BC Ministry of Environment, 2011c).

2.2.5 Regional Wind Climate

Wind-generated waves are responsible for most of the waves experienced in the Strait of Georgia. The prevailing winds in the Strait of Georgia are predominantly from the northwest (in summer) and southeast (in winter), resulting in storm waves that align approximately with the main axis of Discovery Passage. Long-term wind data near the project site is available from Campbell River Airport, Sentry Shoal wave buoy, and Comox Airport. These climate stations provide hourly climate records as summarised in **Table 2-3** and the locations shown in Figure 2-1. The data were used to evaluate the frequency and direction distribution for wind in the northern part of the Strait of Georgia.

Table 2-3: Wind data source from Meteorological Service of Canada.

Station	Station ID	Station Location	Period
Campbell River Airport	1021261	Latitude: 49.95 Longitude: -125.27	1979 – 2013
Campbell River Airport	1021267	Latitude: 49.95 Longitude: -125.27	2013 – current
Sentry Shoal	C46131	Latitude: 49.91, Longitude: -124.99	1992 – current
Comox Airport	1021830	Latitude: 49.16, Longitude: -124.90	1953 – current

The local wind climate can be assessed by the use of a wind rose, a graphic presentation of winds for specified areas, utilizing arrows at the cardinal and inter-cardinal compass points to show the direction from which the winds blow and the magnitude and frequency for a given period of time. The wind rose derived from the observed data at Campbell River Airport, Sentry Shoal, and Comox Airport are shown in **Figure 2-4**, **Figure 2-5**, and **Figure 2-6** respectively.

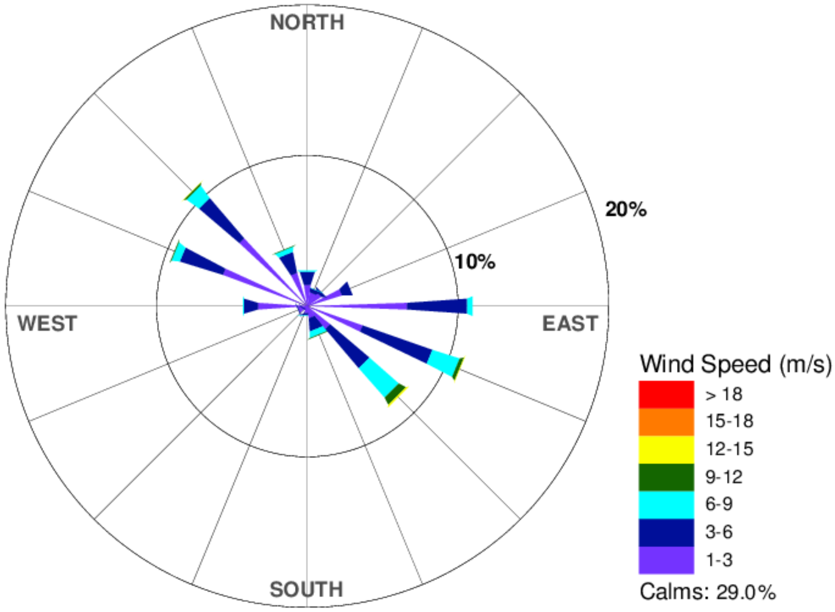

Figure 2-4: Wind distribution plot (wind rose) – Campbell River Airport[2].

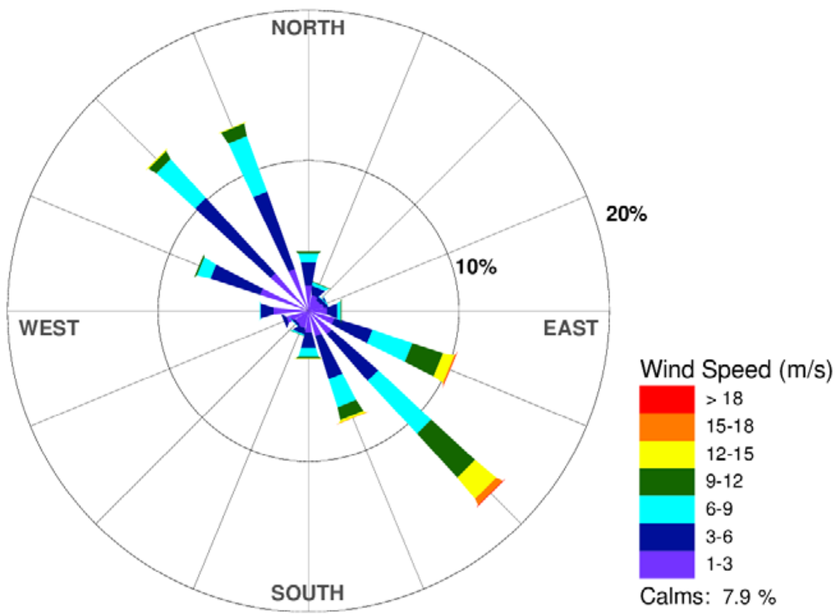

Figure 2-5: Wind distribution plot (wind rose) – Sentry Shoal.

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[2] Data from Station 1021261 and Station 1021267 are combined for the Campbell River Airport site.

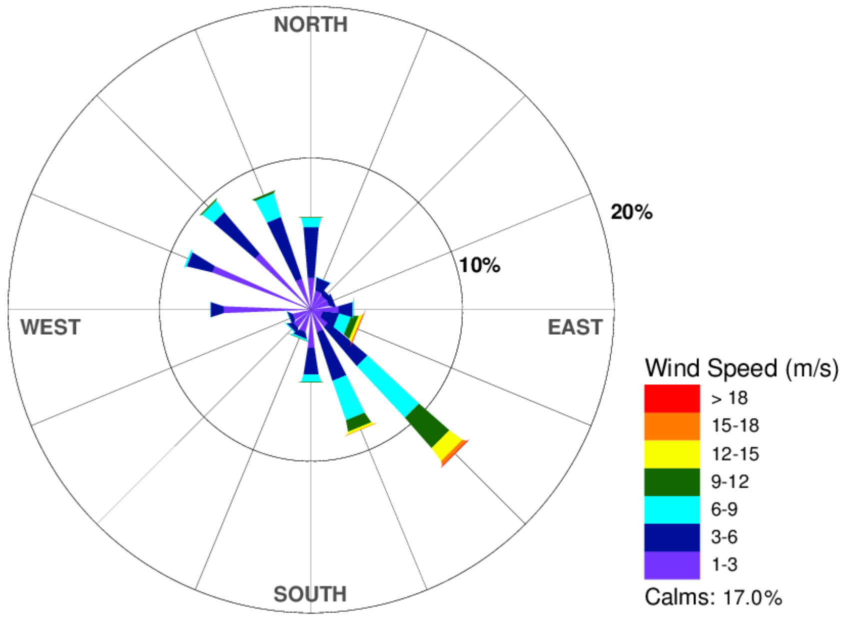


Figure 2-6: Wind distribution plots (wind rose) – Comox Airport

The results show that the strongest winds experienced in the northern part of the Strait of Georgia are from the southeast. Winds measured at Campbell River Airport are calmer than winds measured at Sentry Shoal and Comox Airport, likely because the Campbell River Airport station is located about 5 km inland and at an elevation of about 108 m above sea level and therefore doesn't adequately represent wind conditions that will generate waves in the Strait. This station is excluded from the analysis.

Frequency analysis is conducted on the Sentry Shoal and Comox Airport hourly wind data for the period of record to obtain the design wind speed[3] for the southeasterly events. The results are summarized in **Table 2-4**.

[3] The design wind speed is derived based on Sentry Shoal observation data which is the average wind speed for the most recent two-minute period prior to the observation time. This is also considered the "sustained wind" as used in wave hindcasting analysis.

Table 2-4: Summary of design wind speeds at Sentry Shoal and Comox Airport.

	Southeasterly Sentry Shoal		Southeasterly Comox Airport	
Return Period (yr)	Speed (m/s)	Speed (km/hr)	Speed (m/s)	Speed (km/hr)
1	19.0	68	20.1	72
2	20.2	72	20.8	75
5	21.8	79	21.7	78
10	23.0	83	22.5	81
20	24.2	87	23.3	84
50	25.8	93	24.4	88
100	27.0	98	25.1	90
200	28.2	102	25.8	93

The results show that the design winds calculated on the Sentry Shoal station predictions are slightly higher than the design winds for Comox Airport. Design winds from Sentry Shoal are used in the analysis because it is closer to the project site, it is in open water and its values are less likely influenced by land formations.

2.2.6 Regional Wave Climate

Wave heights in the Strait of Georgia are limited by fetch distance instead of by wind strength and duration. Looking at 26-years (1992 to 2017) of wave data collected at the Environment and Climate Change Canada (ECCC) Sentry Shoal buoy located 22 km southeast of Campbell River (Figure 2-1), there were a total of 51 events in the record for Sentry Shoal with significant wave height greater than 3 m. However, the data between 1998 and 1999 (**Figure 2-7**) are doubtful as there were 35 storms with waves over 3 m in these two years, as well as waves over 9 m, which is highly unlikely. Discounting these two years, there were 26 events in 24 years with waves greater than 3 m, with a maximum wave height of 3.6 m (December 12, 2006, November 11, 2007, November 24, 2016).

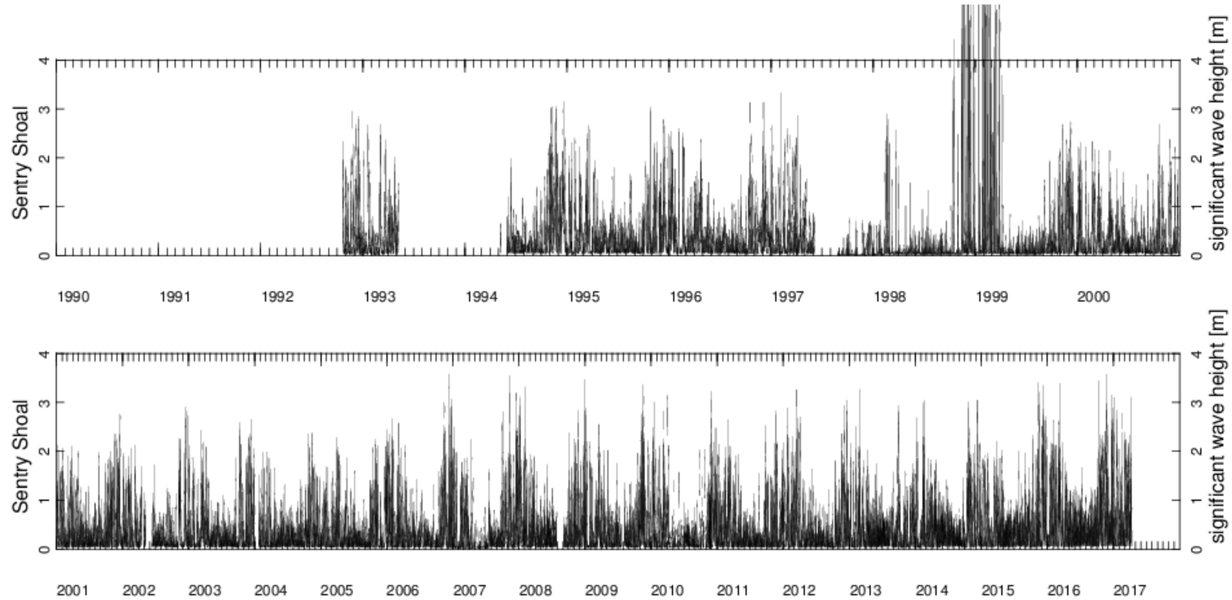


Figure 2-7: Sentry Shoal wave buoy measurements – 1992 to 2017[4].

In addition to the noticeable erroneous data in 1998 and 1999, an examination of the Sentry Shoal data shows that much of the reported data has been flagged as erroneous. Independent assessment was conducted by NHC on selected data and no noticeable difference was found between data that is considered to be good (Quality Code =1) and data that is considered to be erroneous (Quality Code =4). A request was sent to Meteorological Services of Canada for further information on the QA/QC procedure. No reply has been given at the time when this report was prepared.

2.2.7 Local Wave Climate

Wave-current interaction are important in Discovery Passage due to the strong currents, that can reach up to 9 knots (4.6 m/s), and can result in a large localised increase to wave heights. Due to the strong tidally-induced currents that develop in Discovery Passage, the wave field in the passage can change considerably due to wave-current interaction. The most noticeable effect occurs when waves propagate against the current.

A wave-current model was developed using commercial software Delft3D (Lesser et al., 2004) and SWAN (Booij et al., 1999) to evaluate the wave-current interaction process near the site. The model extends from Brown's Bay at the north boundary to Parksville at the south boundary. The model consists of four 2-way coupled model domains with progressively fining resolution, with the finest grid resolution in the

[4] No data available between May 1993 and June 1994, between June and August 1997, between May and June 2002, August 2008

vicinity of the basin where it is 12 m (**Figure 2-8**). Detailed model development is provided in **Appendix A**.

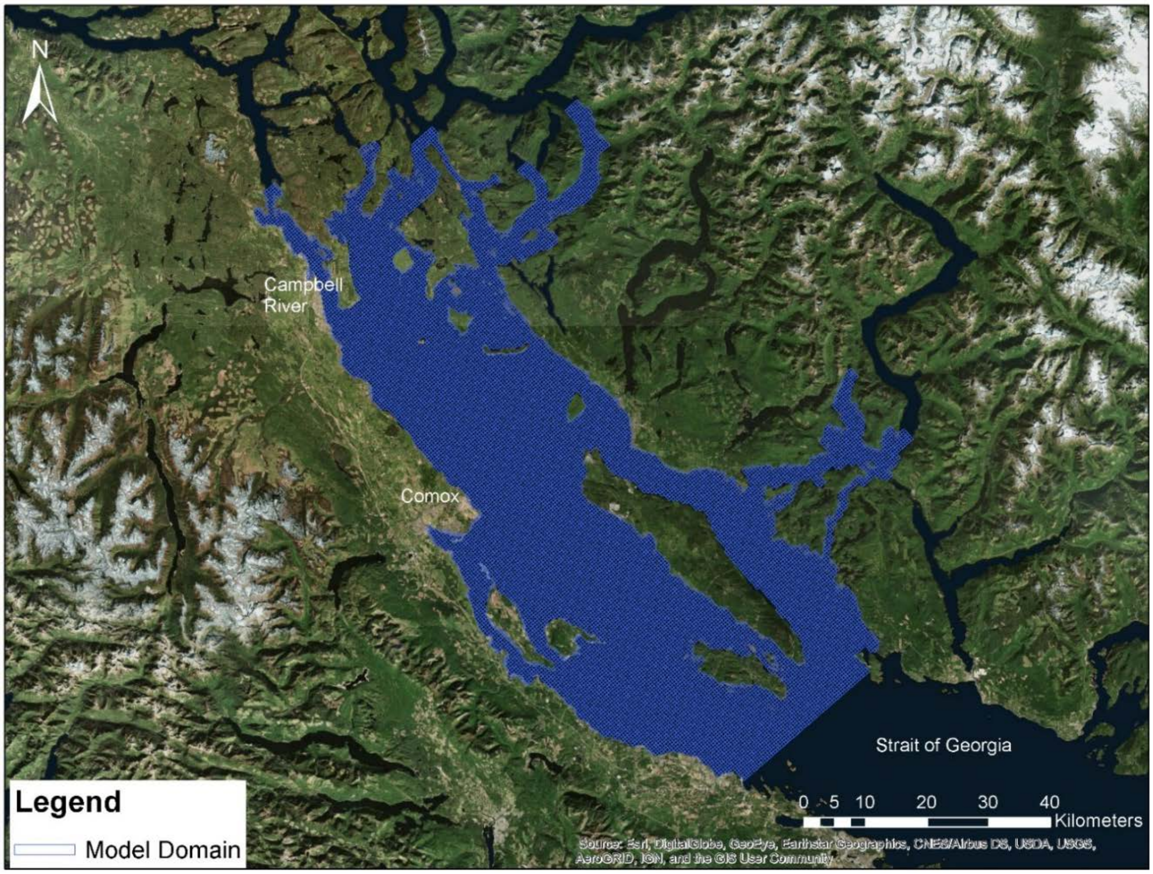


Figure 2-8: Campbell River Hydrodynamic and Wave Model grid domain extents.

High and steep waves can occur in conditions where strong currents oppose the direction of wind waves generated over long fetches. These areas of high and steep waves are known as "rips"[5] and are known to be most severe along the leading edge of the intruding opposing current into the wave field. For this reason, as the tide changes from a ebb-slack to a flood tide, the opposing currents along the leading edge of the flood tide generate the most severe rips.

Figure 2-9 and **Figure 2-10** illustrate the potential impact of tidal current on waves in Discovery Passage. The left panel in **Figure 2-9** shows the surface current vectors during flood tide condition in Discovery Passage. The middle panel shows the wave height distribution for a 65 km/hr southeasterly storm event in the absence of currents (i.e., no wave-current interaction). The colour-contoured field represents significant wave height. The right panel shows the wave height distribution for a 65 km/hr southeasterly event coinciding with the flood tide condition. The vector represents surface current speed and direction

[5] R.E. Thomson, Oceanography of the British Columbia Coast, Canadian Special Publication of Fisheries and Aquatic Sciences 56 (1981)

and the colour contoured field represents significant wave height. The figure shows that without wave-current interaction, the wave height in the channel is about 1.5 m. With opposing tidal current direction to the direction of waves, the wave heights in the channel are increased to about 3.0 m. The three panels in **Figure 2-10** present similar information as that in **Figure 2-9** but for ebb tide conditions in which the surface current and wind are heading in the same direction. The result shows that the wave height in the main channel is reduced slightly under this condition.

While the impact of wave-current interaction does not affect the wave climate near the project site as much as that in the channel, the wave-current interaction is a key process in understanding the overall wave climate in Discovery Passage.

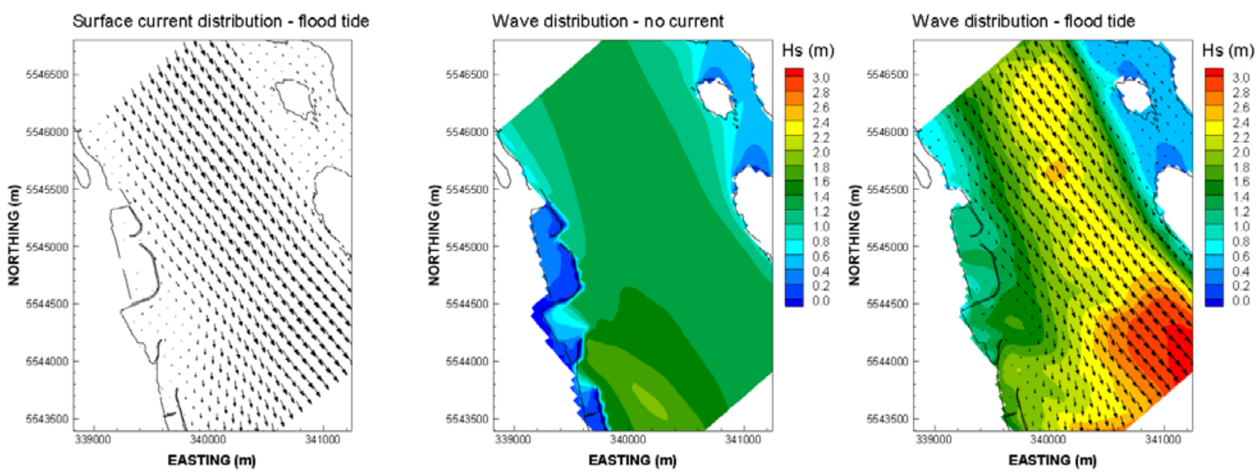


Figure 2-9: Current field, wave height distribution with no current, wave height distribution with flood tide.

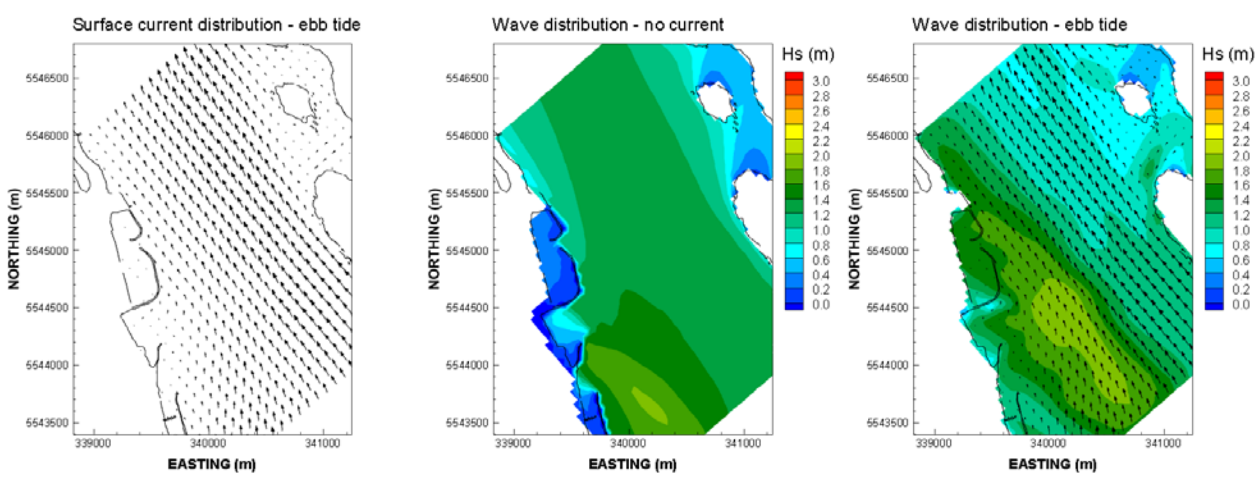


Figure 2-10: Current field, wave height distribution with no current, wave height distribution with ebb tide.

3 FLOOD CONSTRUCTION LEVEL ANALYSIS

Two methods, additive approach and joint probability approach, were applied to establish the FCL for the Downtown Waterfront Site.

3.1 Additive Approach

As discussed in **Section 1.2**, the additive approach presented in the 2011 BC Ministry of Environment Climate Change Adaptation Guidelines for Sea Dikes and Coastal Flood Hazard Land Use (2011b) defines FCL as a sum of Designated Flood Level (DFL), wave effect from a design storm and freeboard. **Table 3-4** summarizes the DFL derived based on information presented in **Section 2**.

Table 3-1: Summary of Designated Flood Levels.

FCL Components	2017	2050	2100
HHWLT (m GD)	1.70	1.70	1.70
200-year Surge (m)	0.97	0.97	0.97
Sea Level Rise (m)	0.17	0.50	1.00
Local Uplift/Subsidence (m)	-0.07	-0.21	-0.41
Designated Flood Levels (m GD)	2.77	2.96	3.26

3.1.1 Wave Effect

The proposed Downtown Waterfront Site consists primarily of commercial developments; it is not a continuous sea dike and does not present the same consequence if design water level is exceeded. The design life of the development is likely on the order of 50-years. The 50-yr Southeasterly storm event with wind speed of 93 km/hr was utilized to establish the wave effect for the study.

Due to the strong tidally-induced currents that develop in Discovery Passage, the wave field in the passage can change considerably because of wave-current interactions. Discussion in **Section 2.2.7** shows that the most noticeable effect occurs when waves propagate against the current direction. The most severe wave conditions in Discovery Passage are expected to occur during flood tide when currents flow towards the southeast, and under southeasterly storm conditions. The results of the modelling also indicate that wave heights are dependent on water depth so the relationship between current speed and wave height is not direct. **Figure 3-1** shows a histogram of tidal range at ECCC Campbell River station for the last 45 years (1972-2017)**.** Positive values represent flood tide conditions when the ocean changes from low tide to high tide while negative values represent ebb tide conditions when the ocean changes from high tide to low tide. The figure shows that flood tide occurred 50% of the time. A sensitivity analysis was first conducted to establish the design condition for the assessment. Four tidal condition scenarios were modelled, which are summarized in **Table 3-2**.

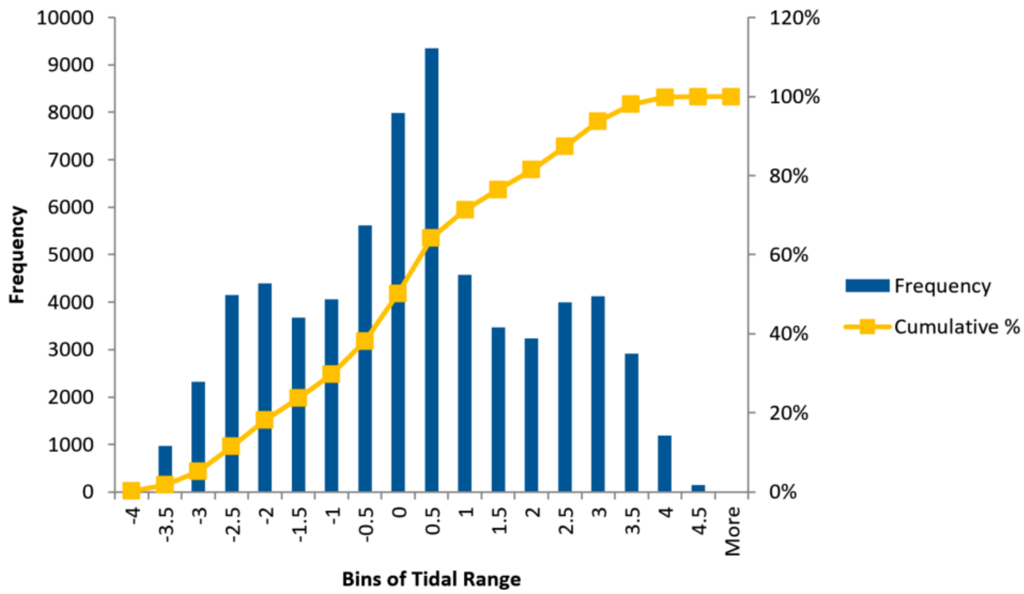


Figure 3-1: Campbell River Station Tidal range histogram.

Table 3-2: Summary of modelled scenarios for sensitivity analysis.

Simulation	Tidal range	Tidal condition	Wind condition
1	+1.0 m	December 14th, 2016	50-yr SE event
2	+2.0 m	November 12th, 2016	50-yr SE event
3	+3.0 m	November 29th, 2016	50-yr SE event
4	+4.0 m	December 16th, 2016	50-yr SE event

Modelled significant wave height distributions near the basin from each simulation are shown in **Figure 3-2**. The results show that the project site experiences slightly larger waves during smaller flood tide when the ocean level switches from Higher Low to Lower High. The modelled wave heights at the project site are summarized in **Table 3-3**. The differences in wave magnitude between +1.0 m tidal range and +4.0 m tidal range is about 0.2 m.

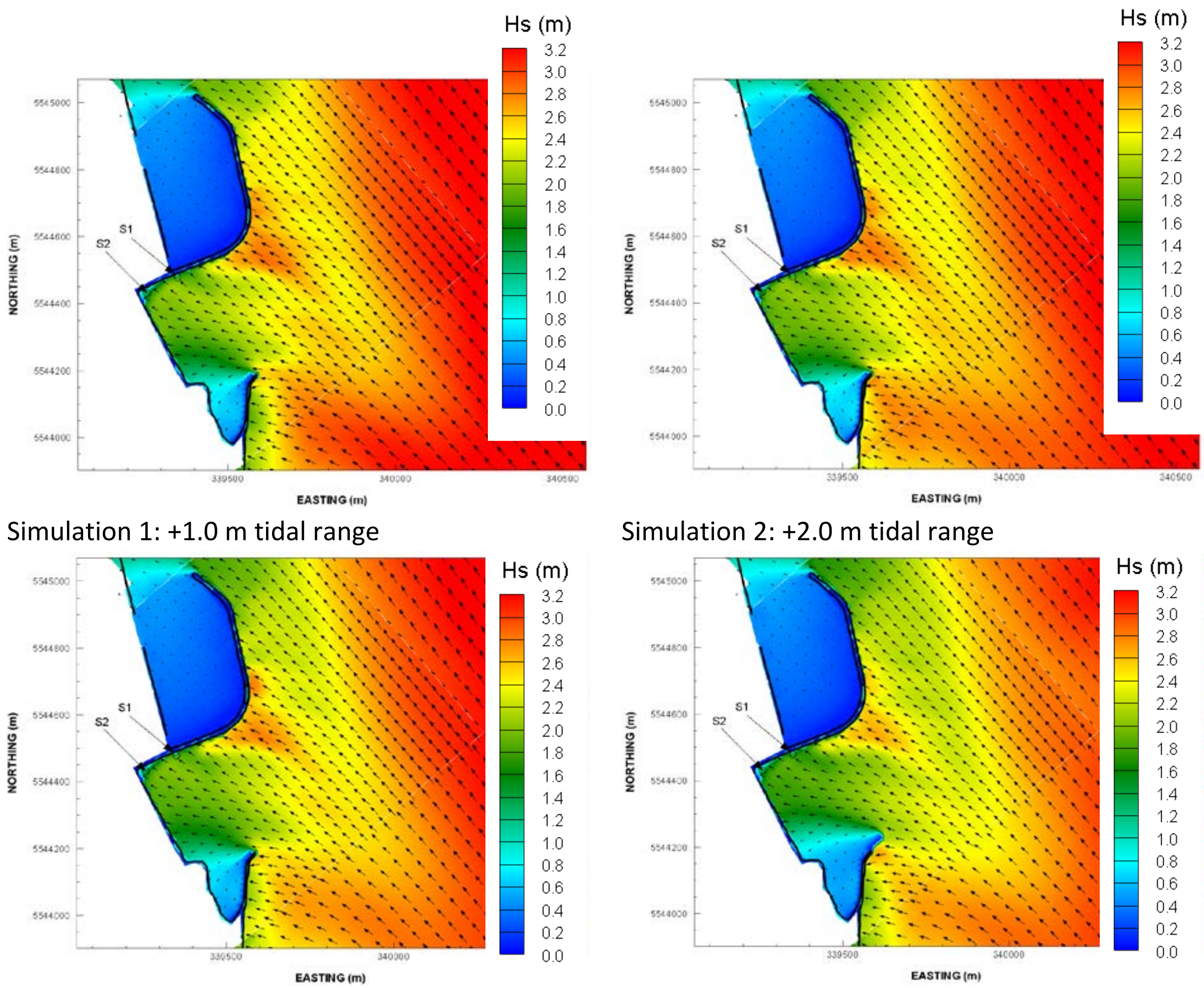
The December 14th, 2016 tidal cycle was selected for the assessment. Over this period the water level at Campbell River increased from 0.5 m GD to 1.5 m GD for a tidal range of +1.0 m, a condition that occurs frequently throughout the year. The design wave characteristics at S1 (armoured riprap) and S2 (cobble beach) shown on the figure are:

- S1: wave height of 1.9 m and period of 11.5 seconds.

- S2: wave height of 1.6 m and period of 10.1 seconds

Table 3-3: Summary of modelled wave heights.

Simulation	Tidal range	Wave height (m)
1	+1.0 m	2.1
2	+2.0 m	2.1
3	+3.0 m	2.0
4	+4.0 m	1.9



Simulation 1: +1.0 m tidal range

Simulation 2: +2.0 m tidal range

Simulation 3: + 3.0 m

Simulation 4: +4.0 m tidal range

Figure 3-2: Sensitivity analysis - wave height distribution maps.

Wave Run-up Analysis

The BC Provincial Sea Dike Guidelines (2011b) accept the use of a number of criteria for calculation of the wave run-up component for design elevation. For defining the sea dike crest elevation the wave run-up is taken to be the vertical distance exceeded by no more than 2% of the waves during the designed storm.

The wave run-up is estimated using the method described in European Overtopping Manual (2016). Required data includes wave heights and periods, angle of propagation, and structural design information such as the profile of the shoreline and bank, depth of the fronting slope, and roughness and porosity of the shoreline bank materials. It is assumed that the slope of the armoured riprap at S1 will be 2H:1V and the slope of the cobble beach at S2 will be 6H:1V. The estimated $R_{2\%}$ at S1 and S2 are 3.0 m and 3.5 m, respectively.

3.1.2 Freeboard

It is common practice to include provision for uncertainties by incorporating a minimum freeboard. The Sea Dike Guidelines (2011c) recommends that a freeboard value of 0.6 m be included in sea dike design.

3.1.3 Summary – Additive Approach Flood Construction Level

Table 3-4 and **Table 3-5** summarize the FCL derived using the additive approach FCL for current and future conditions.

Table 3-4: Flood Construction Levels at S1 – Additive Approach.

FCL Components	2017	2050	2100
HHWLT (m GD)	1.70	1.70	1.70
200-year Surge (m)	0.97	0.97	0.97
Sea Level Rise (m)	0.17	0.50	1.00
Local subsidence (m)	-0.07	-0.21	-0.41
Wave Effect (m)	3.03	3.09	3.17
Freeboard (m)	0.60	0.60	0.60
Flood Construction Level (m GD)	6.40	6.65	7.03

Table 3-5: Flood Construction Levels at S2 – Additive Approach.

FCL Components	2017	2050	2100
HHWLT (m GD)	1.70	1.70	1.70
200-year Surge (m)	0.97	0.97	0.97
Sea Level Rise (m)	0.17	0.50	1.00
Local subsidence (m)	-0.07	-0.21	-0.41
Wave Effect – 50yr event (m)	3.54	3.61	3.71
Freeboard (m)	0.60	0.60	0.60
Flood Construction Level (m GD)	6.91	7.17	7.57

3.2 Joint Probability Approach

The additive approach presented in the 2011 BC Ministry of Environment Climate Change Adaptation Guidelines for Sea Dikes and Coastal Flood Hazard Land Use normally resulted in conservative values as they do not account for the probability of simultaneous occurrence of the events. A joint probability approach was conducted to reduce the conservatism inherent in the additive approach.

3.2.1 Joint probability of water levels and deep-water wave conditions

Storm surge can sometimes coincide with high astronomical tides to produce unusually high water conditions along the British Columbia coast. Meanwhile, the exposure of a site to waves and the likelihood of peak waves for a given storm condition from a specific direction occurring coincident with storm surge and high tides is of paramount concern for determination of the coastal flood hazard. To examine the probability of this at the Campbell River waterfront, a joint probability analysis has been undertaken that utilized historical water level records (that include both astronomical tides and surge) and a coincident wave record for a deepwater buoy in the northern Strait of Georgia. Details of the analysis are provided in Appendix B, and a brief summary is provided here.

Figure 3-3 shows curves of equal probability for combinations of water levels and offshore wave heights at the Sentry Shoal area south of Quadra Island. It is observed that below a wave height of 2.5m there is little change in the probability of a given water level occurring. Similarly, below a water level of 2m (GD) there is little change in the probability of a wave event occurring.

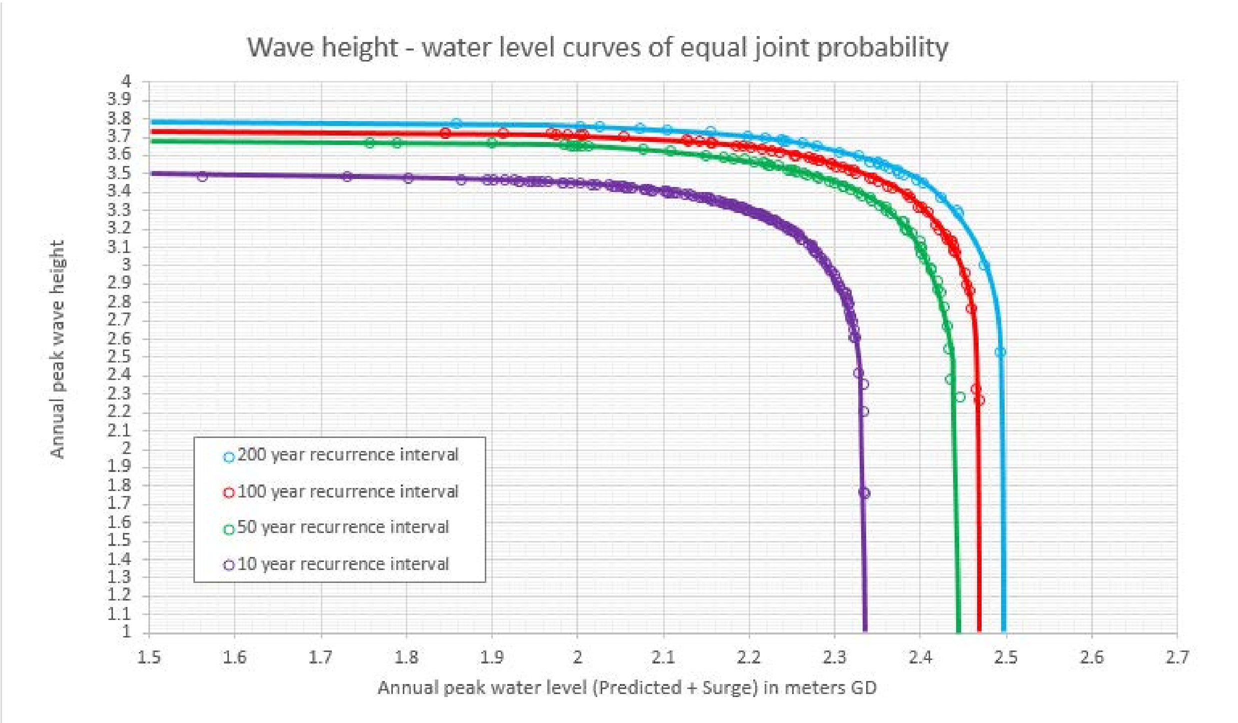


Figure 3-3: Wave height – Water Level Curves of Equal Joint Probability.

3.2.2 Wave Effect

Based on the findings from the additive approach assessment (**Section 3.1.1**), the project site experiences slightly larger waves during smaller flood tide when the ocean level switches from Higher Low to Lower High. The December 14th, 2016 tidal cycle with +1.0 m flood tide used in the additive approach assessment was used for the joint probability assessment. The peak water level over the course of this tidal cycle was adjusted to match the design water level value for each selected 200-year recurrence scenarios (**Figure 3-3**). The wind speed required to achieve the offshore design wave height at Sentry Shoal for each scenario was applied to the model. The results at S1 (armoured riprap) and S2 (cobble beach) are summarized in **Table 3-6.**

Due to the channel geometry of Discovery Passage, specifically at Yaculta Bank where the shallowest elevation is at about -5 m Chart Datum, large variations in the offshore wave heights at Sentry Shoal between 3.40 m and 3.75 m cause very little variation in the incident wave height at the project site.

Table 3-6: **Summary of 200-year recurrence wave heights**

Simulation	Water Level (m GD)	Wave height at Sentry Shoal (m)	Wave height at S1 (m)	Wave period at S1 (sec)	Wave height at S2 (m)	Wave period at S2 (sec)
1	2.50	2.50	0.82	8.5	0.67	7.9
2	2.45	3.40	1.07	9.4	0.88	9.4
3	2.35	3.55	1.08	9.8	0.89	9.6
4	2.30	3.60	1.09	10.1	0.89	9.8
5	2.20	3.70	1.07	10.1	0.89	9.8
6	2.10	3.75	1.06	10.1	0.88	9.8

Wave Run-up Analysis

The wave run-up is estimated using the method described in European Overtopping Manual (2016). **Table 3-7** and **Table 3-8** summarize the estimated wave runup for each scenario at S1 and S2 respectively. The results show that the worst condition occurs under Scenario 2 which consists of an offshore wave height of 3.4 m coinciding with a design water level of 2.45 m GD. The results from Scenario 2 are used to determine the FCL values.

Table 3-7: Summary of 200-year recurrence estimated $R_{2\%}$ wave runup at S1

Simulation	Water Level (m GD)	Wave height at S1 (m)	Wave runup at S1 (m)	Water level + wave runup (m GD)
1	2.50	0.82	1.60	4.10
2	2.45	1.07	1.95	4.40
3	2.35	1.08	1.95	4.30
4	2.30	1.09	1.95	4.25
5	2.20	1.07	1.91	4.11
6	2.10	1.06	1.87	3.97

Table 3-8: Summary of 200-year recurrence estimated $R_{2\%}$ wave runup at S2

Simulation	Water Level (m GD)	Wave height at S2 (m)	Wave runup at S2 (m)	Water level + wave runup (m GD)
1	2.50	0.67	1.91	4.41
2	2.45	0.88	2.42	4.87
3	2.35	0.89	2.42	4.77
4	2.30	0.89	2.42	4.72
5	2.20	0.89	2.39	4.59
6	2.10	0.88	2.35	4.45

3.2.3 Freeboard

It is common practice to include provision for uncertainties by incorporating a minimum freeboard. The Sea Dike Guidelines (2011c) recommends that a freeboard value of 0.6 m be included in sea dike design.

3.2.4 Summary Joint Probability Approach Flood Construction Level

Table 3-9 and summarizes the FCL derived using the joint probability approach FCL for current and future conditions.

Table 3-9: Flood Construction Levels at S1 – Joint Probability Approach

FCL Components	2017	2050	2100
Design Water Level (m)	2.45	2.45	2.45
Sea Level Rise (m)	0.17	0.50	1.00
Local subsidence (m)	-0.07	-0.21	-0.41
Wave Effect [1]	1.97	2.00	2.05
Freeboard (m)	0.60	0.60	0.60
Flood Construction Level (m GD)	5.12	5.34	5.69

[1] Wave effects are revised from Table 3-7 to account for variation in still water level from year 2000 to design year water levels.

Table 3-10: Flood Construction Levels at S2 – Joint Probability Approach

FCL Components	2017	2050	2100
Design Water Level (m)	2.45	2.45	2.45
Sea Level Rise (m)	0.17	0.50	1.00
Local subsidence (m)	-0.07	-0.21	-0.41
Wave Effect [1]	2.44	2.47	2.51
Freeboard (m)	0.60	0.60	0.60
Flood Construction Level (m GD)	5.59	5.81	6.15

[1] Wave effects are revised from Table 3-7 to account for variation in still water level from year 2000 to design year water levels.

3.3 Summary

Summary tables of the FCL results derived using the additive approach (**Section 3.1**) and joint probability approach (**Section 3.2**) are reproduced in **Table 3-11**, **Table 3-12**, **Table 3-13** and **Table 3-14**. The results show that the FCL values derived using the joint probability approach is about 1.4 m lower than the values derived using the simplified additive approach.

Table 3-11: Flood Construction Levels at S1 – Additive Approach.

FCL Components	2017	2050	2100
Design Water Level (m)	2.67	2.67	2.67
Sea Level Rise (m)	0.17	0.50	1.00
Local subsidence (m)	-0.07	-0.21	-0.41
Wave Effect (m)	3.03	3.09	3.17
Freeboard (m)	0.60	0.60	0.60
Flood Construction Level (m GD)	6.40	6.65	7.03

Table 3-12: Flood Construction Levels at S1 – Joint Probability Approach

FCL Components	2017	2050	2100
Design Water Level (m)	2.45	2.45	2.45
Sea Level Rise (m)	0.17	0.50	1.00
Local subsidence (m)	-0.07	-0.21	-0.41
Wave Effect	1.97	2.00	2.05
Freeboard (m)	0.60	0.60	0.60
Flood Construction Level (m GD)	5.12	5.34	5.69

Table 3-13: Flood Construction Levels at S2 – Additive Approach.

FCL Components	2017	2050	2100
Design Water Level (m)	2.67	2.67	2.67
Sea Level Rise (m)	0.17	0.50	1.00
Local subsidence (m)	-0.07	-0.21	-0.41
Wave Effect (m)	3.54	3.61	3.71
Freeboard (m)	0.60	0.60	0.60
Flood Construction Level (m GD)	6.91	7.17	7.57

Table 3-14: Flood Construction Levels at S2 – Joint Probability Approach

FCL Components	2017	2050	2100
Design Water Level (m)	2.45	2.45	2.45
Sea Level Rise (m)	0.17	0.50	1.00
Local subsidence (m)	-0.07	-0.21	-0.41
Wave Effect	2.44	2.47	2.51
Freeboard (m)	0.60	0.60	0.60
Flood Construction Level (m GD)	5.59	5.81	6.15

The difference FCL levels as calculated for the shoreline sections of S1 and S2 are entirely due to the wave effect component that come from the difference shoreline design. As these two shorelines are connected to a single property the lower of the two FCL criteria should govern for the overall site unless specific upland measures are taken to prevent coastal flooding from one area impacting on the adjacent area.

4 IMPLICATIONS FOR FUTURE DEVELOMENT

The existing top of bank is at elevation of about 4.2 m GD which is below the recommend FCL values for current and future conditions. The proposed development is expected to experience flooding during the design 200-year recurrence event.

Overtopping discharge is estimated using the method described in European Overtopping Manual (2016). Assumptions adopted for the calculation include the following:

- Top of bank at 4.2 m GD.
- Shoreline structure lengths for S1 and S2 are 90 m and 50 m, respectively (**Figure 4-1**).
- The impacted area behind the shoreline is 50 m x 125 m (as shown in the yellow polygon in **Figure 4-1**.
- The duration of the design high water and storm event is two hours. The tidal level typically drops by 0.3 to 0.5 m in two hours at Campbell River. As well, the sustained wind during the storm event at Sentry Shoal usually lasted for less than two hours.

The results are summarized in **Table 4-1** . It is important to note that section S2 has been calculated for the proposed development (see Figure 1-1) in which this area does not have a rock armour revetment.

Table 4-1: Summary of design overtopping discharge and flood depth

	2017		2050		2100	
FCL Components	S1	S2	S1	S2	S1	S2
Overtopping discharge rate (litre/s per m)	0.9	3.6	2.2	6.7	8.0	17.3
Section length (m)	90	50	90	50	90	50
Overtopping volume (m^3 per 2 hrs)	590	1290	1410	2420	5160	6220
Total overtopping volume (m^3 per 2 hrs)	1900		3800		11400	
Impacted area (m^2)	6250		6250		6250	
Flood depth (m)	0.30		0.61		1.82	

Figure 4-1: Estimated shoreline structure lengths and impacted area.

It is noted that an oceanfront boardwalk along the shoreline between the present day BC Ferries terminal and the Site would presumably align along the crest of the rock armour slope. For planning purposes the runup and overtopping as associated with S1 is indicative of what would be expected for a walkway at the top of the existing revetment. It is noted that moving south from the project site wave heights decrease in the lee of the BC Ferries Terminal and the associated runup will similarly decreased.

5 MITIGATIVE MEASURES

5.1 Mitigation strategies

There are several mitigative strategies possible for consideration to reduce the impacts of potential coastal flood hazard at the site. Some of these mitigation strategies can be used in combination. The effects of various mitigations is to reduce the volume of water that is causing flooding of coastal lands during storm events. Detailed analysis has not been undertaken within the existing scope, and the actual reduction of any flooding will require upon the details of the implementation.

Mitigative strategies include:

.1 Ensure all primary structures are well set-back (25-30 meters) from the shoreline. This is to allow a 'splash zone' for energy from any overtopping water to be dissipated before reaching structures, and to allow landscape measures to be utilized for drainage and attenuation of wave energy. Further, it ensures there is available land for future mitigations for sea level rise accommodation.

.2 Design of a flood resistant area landward of the shoreline revetment, with proper drainage design to allow high rates of coastal flooding to drain directly into the adjacent harbour basin.

.3 Upgrade to the existing rock armour shoreline with both a wider crest and a surface layer of larger rocks. This upgrade of larger rocks will help to decrease wave run-up velocities and provide a more porous structure to attenuate incoming energy. (Note that the rock armour requires a proper filter layer to prevent shoreline erosion.) Although rock revetments are not typically considered as a Green Shores approach, the additional void spaces have been noted to provide shoreline habitat suitable for juvenile salmon provided that the rocks are not tightly packed[6][7].

.4 Beach Nourishments using mixtures of sand, cobbles, and gravels to raise the elevations of the pocket beach along the shoreline at S2 and extending somewhat into S1. Raising the nearshore seabed elevation requires provincial and federal approvals, but as a mitigation strategy can be effective in reducing wave energy at the shoreline and help to retain a more natural and varied intertidal zone with recreational benefits. A potential downside may be increased potential for retention of woody debris, and also the potential need to re-supply the sediment supply in the future. It is noted that there is a reasonably stable beach in this location, and re-supply of the sediment is not expected to be a frequent or regular maintenance requirement.

.5 Construction of a re-curved parapet seawall[8] on the alignment of the existing revetment. Some analysis is required to determine if there is sufficient freeboard for a parapet to work in this location without obstruction of views from the existing property grades.

Concerns exist around parapet walls in that they mobilize a large volume of water into the air that can then be blown by wind landward and cause flooding concerns and negate some of the supposed benefit of the recurved wall (Figure 5-1). Such walls are also know for experiencing very high levels of impulse loads from the breaking waves and thus require suitably large

[6] J.T. Quigley, D.J. Harper, Streambank Protection with Rip-rap: An Evaluation of the Effects on Fish and Fish Habitat. Fisheries and Aquatic Sciences 2701 (2004). Although focused on streams, this report notes that rock structures with large void spaces are suitable for juvenile salmon.

[7] D.R. Haggarty (2001), An evaluation of fish Habitat in Burrard Inlet, BC. University of British Columbia, 2001. Study concluded that in Burrard Inlet (with similar shoreline developments as in Campbell River) juvenile chinook tended to use larger substrates such as bedrock and boulders over sand and mud, and that more chum were found over cobble substrates than mud.

[8] A seawall with a deflector built into the crest that re-directs the up-rushing wave jet back towards the ocean.

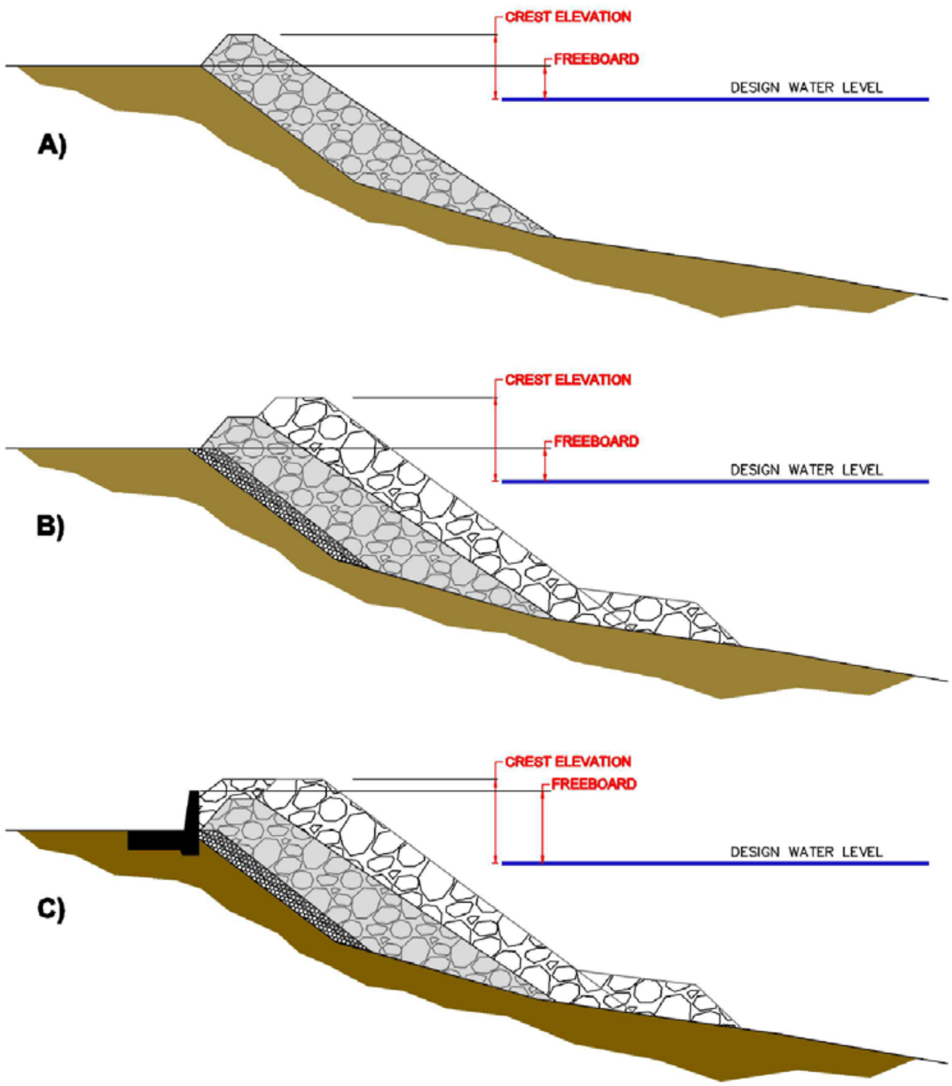
foundation structures and a high level of internal reinforcement which increases their costs for design and construction. In an area known for having woody debris in the water including full logs, the wall would also be susceptible to damage from impacts from this debris during wave events.

Figure 5-1: Wave over-topping from UK storm Eleanor, January 2018 (BBC).

.6 Addition of flood mitigation structures at the top of the proposed cobble beach to reduce wave over-topping in this location. Such mitigation measures could include large blocks placed to act as flow dissipation measures similar to those shown in **Figure 5-2**.

Figure 5-2: Example of blocks placed to dissipate wave runup (EurOtop II Manual).

Item 3 above notes a potential mitigation measure to reduce wave runup at the site as being possible through the addition of more rock armour to the face and crest of the existing rock armour. Several details are worth mentioning that can have a significant bearing on the effectiveness of the mitigation. Firstly, an effective and properly designed filter layer is necessary to prevent erosion of the slope behind the rock armour. To improve the filter layer would require the removal and replacement of the existing rock armour on the slope, but may be a required measure to ensure a robust design. Secondly, while additional rock armour on the face and crest will increase the void spaces to better dissipate wave energy, the actual freeboard of the slope is not increased and flood risk remains (although with lower wave runup velocities within the structure). **Figure 5-3** provides a scale sketch that shows how increased rock armour alone does not alter the freeboard[9] of the shoreline (compare **Figure 5-3** (A) and (B)) whereas a crown wall does (**Figure 5-3** (C)).



[9] Freeboard is here defined as the elevation of the impermeable land above the design high water still water level for the purpose of wave runup and overtopping. This is different from the notion of Freeboard used in the Provincial Guidelines in which it is used as a factor of safety to be included above the maximum wave runup elevation.

Figure 5-3: Sketch[10] showing effects of additional rock armour. The top sketch (A) is representative of the existing slope. (B) shows a section upon which additional large rock armour has been placed to raise the crest of the armour, and (C) shows the effect of a crown wall to raise shoreline resistance to flooding.

It is also worth noting that the larger seawall with a re-curved parapet would still likely, as a matter of best practice, require some amount of rock armour on the front face to prevent scour of the seabed. Schematic drawings of an upgraded revetment and a possible seawall option are shown in

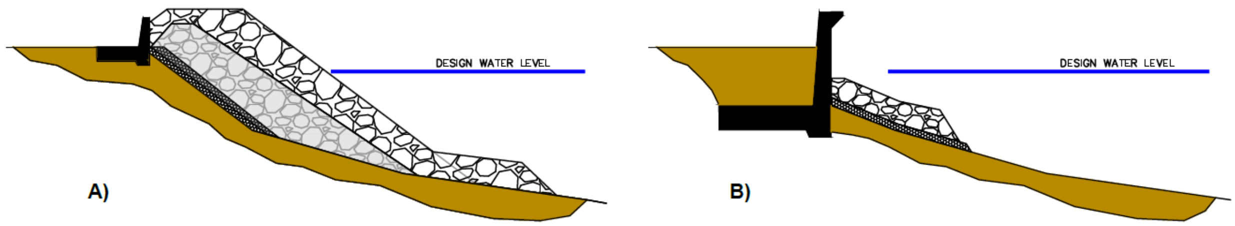


Figure 5-4: Sketch showing (A) a schematic of an upgraded rock armour revetment and (B) a schematic of a seawall.

5.2 Capital Cost Estimates

High level capital costing has been prepared for the possible mitigation options. It is important to note that some mitigations can be combined, while other mitigations are only practical for a select area of the shoreline but not everywhere. Where applicable, it is noted in the table if the treatment is suitable for S1, S2, or both areas of the shoreline.

[10] Note that the sketches in this report are only provided to show different crest arrangements, and have intentionally not shown details for the toe and filter layers below the beach grade as such items should be properly designed and not inferred from a sketch.

Table 5-1: Coastal Flood Mitigations Budget Bins

Mitigation Measure	Area	Budget Bin		
		< $ 500 k	$ 500k to $ 2M	> $2M
Setbacks from Shoreline [1]	Up-land	✓		
Enhanced Drainage [1]	Up-land	✓		
Rock Armour Upgrades [2]	S1 & S2		✓	
Beach Nourishment [3]	S2		✓	
Seawall with Parapet	S1 & S2			✓
Slope Blocks	S2	✓		

Notes:

1. The lost opportunity cost for reduced occupancy on the site has not been considered.
2. Depending on the details of the upgrade (with or without a crown wall for example) this mitigation could approach or exceed the $2M limit for the budget bin.
3. Assumed mixture of sand, gravels, and cobbles. Area and volume of nourishment required were only roughly estimated.

The cost of rock armour upgrades (including both design and construction) to the riprap as shown in **Figure 5-3** is assumed for budgeting purposes to be between $2500 and $3500 for each metre of shoreline, without allowance for a crown wall. A crown wall is assumed to be a similar order of magnitude cost to design and construct.

5.3 Mitigation summary matrix

Table 5-2: Summary of Mitigation Trade-offs

Concept	Advantages	Disadvantages	Permitting Implications[1]
Building Setbacks	▪ Buildings removed from zone of maximum potential damage near shoreline from wave over-topping and spray ▪ Allows space for other mitigations to be utilized effectively	▪ Reduction in available land for building usage on the site	▪ No impact to external agencies (Provincial / Federal) permits
Enhanced Drainage & landscaping	▪ Allows seawater a return path to ocean ▪ Greatly reduces risks of in-land flooding from coastal wave action ▪ Landscaping and civil works on-site to ensure coastal flood water does not flow from areas with high over-topping to other areas.	▪ Requires some maintenance to ensure functionality ▪ Landscaping requirements to reduce flooding extents could restrict development options.	▪ None foreseen, provided meets existing regulations
Rock Armour Upgrades	▪ Effectively reduces wave runup at shoreline ▪ Increases resiliency of shoreline for SLR	▪ Expensive to construct ▪ Reduces visibility of ocean from site	▪ Increased footprint on intertidal shoreline likely to require detailed assessment for DFO permit
Beach Nourishments	▪ Mix of natural intertidal sands, gravel and cobbles complements rocky shorelines ▪ Reduces wave effects on shoreline ▪ Provides recreational space	▪ Requires monitoring and more likely to require maintenance ▪ Larger footprint within intertidal leading to possibly more complex permitting requirements ▪ More susceptible to collection of woody debris	▪ Likely to require detailed assessment for DFO ▪ Likely to require sediment transport study
Seawall with Parapet	▪ Smaller footprint for construction (although wind blown spray could lead to a larger effective footprint for a hazard zone) ▪ Improved views of the water from seawall walkway	▪ Costly to construct. ▪ Low adaptability to future increases in SLR ▪ Negative impacts on foreshore and potential increase in wave reflections to ferry terminal.	▪ Likely to require detailed assessment for DFO permit for construction ▪ Likely to require sediment transport study
Slope Blocks for wave runup	▪ Considered for 'park' area in conjunction with a beach nourishment ▪ Can be effective as dissipating wave runup energy, while allowing public space usage at other times	▪ Relatively expensive to construct compared to more natural shorelines ▪ Potential 'log trap' for woody debris	▪ None foreseen as to be built above the natural boundary

6 CONCLUSIONS AND RECOMMENDATIONS

The study show that:

- The FCL values derived using the joint probability approach is about 1.4 m lower than the values derived using the simplified additive approach.

- The recommended 2050 FCL values for the armoured riprap shoreline and cobble beach shoreline are 5.3 m GD and 5.8 m GD, respectively.

- The recommended 2100 FCL values for the armoured riprap shoreline and cobble beach shoreline are 5.7 m GD and 6.2 m GD, respectively. It is noted that if proper drainage is allowed on site for any wave over-topping waters to flow back to the ocean, then there is a reasonable case for the FCL on the overall site to be reduced. Such a reduction would require upon the detailed design of the shoreline and providing engineering documentation of the adequacy of the site drainage.

- For the overall site, the higher of the two FCL values should be adopted unless specific upland measures are taken to prevent flood waters from migrating across the site. (For example, if the project site is of uniform elevation adjacent to the shoreline and there is no impediment to the flow of water, any flood water will migrate between S1 and S2 upland zones unimpeded.)

- The existing top of bank is at elevation of about 4.2 m GD which is below the recommend FCL value for current and future conditions. The proposed development is expected to experience flooding during the design event without suitable mitigations.

- It will be important to ensure any buildings are well set back from the shoreline and that habitable space is above the FCL as per applicable building codes. Flood tolerant structures and public spaces (such as outdoor park land) could be designed for areas of the property below the FCL.

  However, means of controlling public access will be needed to ensure that public safety is maintained if areas of the property are developed below the FCL. For example, a vehicle parking area built below the FCL level could pose a risk to public safety during a storm that might not be initially obvious due to how the environmental conditions can change in a short period of time from a combination of rising tides, storm surge, and increasing wave heights.

- There is concern that during storm events large woody debris could be thrust over the shoreline and onto the property. Improvements to the revetment are thus recommended regardless to reduce overtopping velocities of the water and lessen the risk of woody debris overtopping the shoreline at high velocities.

- A seawall is not a recommended mitigation as this option is costly to construction, more difficult to adapt upwards for future sea level rise, and will increase wave reflections towards the adjacent ferry terminal during storm events.

7 REFERENCES

BC Ministry of Environment (2011a). Climate Change Adaption Guidelines for Sea Dikes and Coastal Flood Hazard Land Use – Draft Policy Discussion Paper.

BC Ministry of Environment (2011b). Climate Change Adaption Guidelines for Sea Dikes and Coastal Flood Hazard Land Use – Guidelines for Management of Coastal Flood Hazard Land Use.

BC Ministry of Environment (2011c). Climate Change Adaption Guidelines for Sea Dikes and Coastal Flood Hazard Land Use – Sea Dike Guidelines.

BC Ministry of Water, Land and Air Protection (Amended 2018). Flood Hazard Area Land Use Management Guidelines. Amended by BC Ministry of Forests, Lands, Natural Resource Operations and Rural Development (January 2018).

Booij, N., Ris, R. C., and Holthuijsen, L. H. (1999). A third-generation wave model for coastal regions: 1. Model description and validation. Journal of Geophysical Research: Oceans, 104(C4), 7649–7666. doi:10.1029/98JC02622.

Cohlmeye Architecture. Campbell River Waterfront Development Task Force Review, (August, 2017).

http://www.overtopping-manual.com/ (2016). European Overtopping Manual.

Lesser, G., Roelvink, J., van Kester, J., and Stelling, G. (2004). Development and validation of a three-dimensional morphological model. Coastal Engeering, 51, 883–915.

R.E. Thomson, Oceanography of the British Columbia Coast, Canadian Special Publication of Fisheries and Aquatic Sciences 56 (1981)

8 CLOSURE

Please do not hesitate to contact Edwin Wang or Grant Lamont (ewang@nhcweb.com | glamont@nhcweb.com) should you wish to discuss of this analysis.