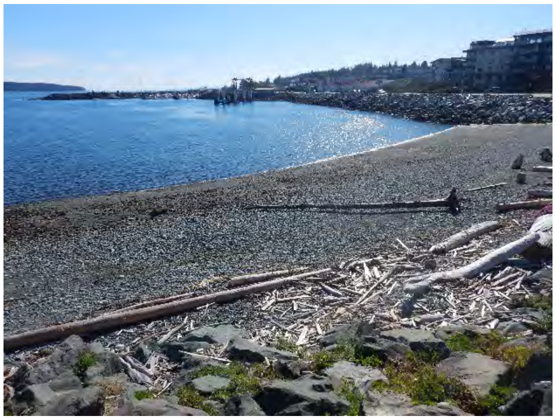
CITY OF CAMPBELL RIVER
SEA LEVEL RISE STUDY
TECHNICAL STUDY – 4 SITES

FINAL REPORT – REVISION 1



Prepared for:

City of Campbell River

Nov 4, 2019

NHC Ref. No. 3003835

northwest hydraulic consultants
water resource specialists

Cover page photos: credit G. Lamont (2018)

Photos taken April 2018, Campbell River

CITY OF CAMPBELL RIVER SEA LEVEL RISE STUDY
TECHNICAL STUDY – 4 SITES

FINAL REPORT

Revision 1

Prepared for:

City of Campbell River

City of Campbell River, BC

Prepared by:

Northwest Hydraulic Consultants Ltd.

North Vancouver, BC

4 November 2019

NHC Ref No. 3003835

nhc

Report Prepared by:

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
E. C. WANG
29366

Nov 4, 2019

Edwin Wang, P.Eng.
Coastal Engineer

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
G. F. LAMONT
32357

4 Nov 2019

Grant Lamont, P.Eng.
Sr. Coastal Engineer

Report Reviewed by:

PROVINCE OF BRITISH COLUMBIA
D. A. RAY
GEOSCIENTIST

4 Nov, 2019

Derek Ray, P.Geo.
Senior Coastal Geomorphologist

DISCLAIMER

This report has been prepared by **Northwest Hydraulic Consultants Ltd.** for the benefit of **City of Campbell River** for specific application to the **Campbell River Sea Level Rise Study.** The information and data contained herein represent **Northwest Hydraulic Consultants Ltd.** best professional judgment in light of the knowledge and information available to **Northwest Hydraulic Consultants Ltd.** at the time of preparation, and was prepared in accordance with generally accepted engineering practices.

Except as required by law, this report and the information and data contained herein are to be treated as confidential and may be used and relied upon only by **City of Campbell River**, its officers and employees. **Northwest Hydraulic Consultants Ltd.** denies any liability whatsoever to other parties who may obtain access to this report for any injury, loss or damage suffered by such parties arising from their use of, or reliance upon, this report or any of its contents.

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 downtown and waterfront development in the city is concentrated in lands that are generally about 4 m above mean sea level. The community has previously experienced flood and erosion hazards both along its riverfront from high river flows and oceanfront from king tides and storm surge events.

This report is a summary of technical work undertaken by Northwest Hydraulic Consultants Ltd. (NHC) to explore impacts of sea level rise on the Campbell River shoreline with respect to coastal flooding and erosion. The report includes a summary of previous phases of work that addressed the preliminary design criteria for a 200-year coastal flood event and empirical calculations of flood construction levels for various segments of shoreline within the City's jurisdiction. Following from this, the technical work focused on four key areas:

- Painter-Barclay escarpment: a neighbourhood north of the Campbell River Estuary;
- Downtown: the downtown area, which was largely built on land reclaimed from the ocean;
- MHC-Evergreen: a section of land between the Maritime Heritage Centre (MHC) and Evergreen Road; and
- Willow Point: a neighbourhood on a low lying promontory at the south end of the city that has a relatively higher exposure to waves from the Strait of Georgia than other neighbourhoods.

Specific to this study, NHC has worked with Lanarc Consultants (Lanarc) to develop conceptual-level protection and adaptation approaches that are reasonable within the context of constraints upon future use and development. Ryzuk Geotechnical were also engaged on the team to undertake an overview level geotechnical assessment of the Painter-Barclay escarpment.

The report presents results of numerical modelling work to examine specific wave-structure interactions within the four key areas, and also modelling undertaken to examine the feasibility of options for adaptation to sea level rise within these areas. It is found that the empirical estimates of wave runup that contribute to the flood construction level are sometimes conservative, but that for lower areas of shoreline the volume of overtopping and velocity of water at the top of the shoreline (along with any floating debris) become important hazards for design.

The geotechnicl analysis of bluff recession at the Painter-Barclay escarpment concluded that a median expected value of horizontal recession that would occur in response to 1 m of sea level rise is on the order of 12 m. It is noted that this value is based upon an assumption of no mitigation and allowing for natural erosion of the bluff.

Select areas of the City's shoreline, as well as the downtown area, experience joint flooding hazards from coastal and stream sources. Specifically, the downtown area was examined for river flood risk within the context of future sea level rise (NHC, 2018b). This previous study focused on examining the correlation between wave events (that lead to coastal flooding) and intense precipitation events and found a moderate correlation as is expected given that major storm events are often accompanied with precipitation. However, a lack of hourly precipitation data in the study area prevents a more detailed assessment of correlations on the timing of peak wave conditions with the timing of precipitation. Small streams such as Willow Creek and Simms Creek are expected to respond relatively quickly to precipitation events given the urban development and the small nature of their watersheds.

The results of this technical work have been used to inform the preparation of the Primers that have been prepared by Lanarc and NHC as deliverables to this study, and for this reason the more detailed discussion of the development of ideas for adaptation to sea level rise, and the cost and trade-off aspects of this work, are presented elsewhere and the reader is referred to those documents.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

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 downtown and waterfront development in the city is concentrated in lands that are generally about 4 m above mean sea level. The community has previously experienced flood and erosion hazards both along its riverfront from high river flows and oceanfront from king tides and storm surge events.

The City recognizes that being a coastal city with limited flood protection infrastructure, the negative consequences that could result from future seal level rise 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 is seeking to better understand the potential risks[1] posed by vulnerability to future floods and anticipated consequences.

Specifically, this study consolidates the findings of previous analysis work by Northwest Hydraulic Consultants Ltd. (NHC) for modelling of ocean currents and waves in Discovery Passage, along with a regional analysis of extreme weather events (wind storms and the timing of associated storm surge) to inform the specific analysis of local flood hazards for four component areas:

- Painter-Barclay escarpment: a neighbourhood north of the Campbell River Estuary;
- Downtown: the downtown area, which was largely built on land reclaimed from the ocean;
- MHC-Evergreen: a section of land between the Maritime Heritage Centre (MHC) and Evergreen Road; and
- Willow Point: a neighbourhood on a low lying promontory at the south end of the city that has a relatively higher exposure to waves from the Strait of Georgia than other neighbourhoods.

Detailed wave modelling of representative shorelines in the study areas were undertaken to refine earlier estimates of wave effects and to examine the effectiveness of select proposed conceptual-level mitigation measures. For the Painter-Barclay escarpment, a geotechnical assessment is incorporated into an estimate of future lateral recession of the bluffs that is expected to occur in response to sea level rise. An analysis of the correlation between rainfall events and wind storm events (as a proxy for wave events) is also provided to inform decision making on specific areas where stream flooding is an additional hazard to coastal flooding.

[1] Risk is defined as the probability of adverse consequences due to wave effects and flooding.

1.2 Study Objectives

This report summarizes the results of a detailed Metocean analysis of the four component sites of this assessment. The objectives for this work are to provide technical guidance to the development of shoreline mititgation options that are developed by the consulting team. As such, the technical work covers a broad range of site specific analysis at the four component sites in response to mitigation concepts developed during the study work.

The detailed analysis of the four component sites builds upon previous work for the City of Campbell River undertaken by NHC that included modeling of ocean currents and waves in Discovery Passage, along with a regional analysis of extreme weather events (wind storms and the timing of storm surge) and detailed joint probability statistical analysis for the timing of storm surges and wind generated waves at Campbell River waterfront.

Detailed wave runup modeling is undertaken using the phase resolving wave model software SWASH to ascertain specific statistics on wave runup and overtopping for shoreline geometries at the four component locations. Where necessary, multiple sections are examined such as at Willow Point where the exposure near the Ken Forde Boat Ramp Park is notably different than that at Adams Park. SWASH is also utilized for analysis of proposed mitigations to get quantifiable statistics on expected performance of each mitigation option in reducing the wave component of the Flood Construction Level (or FCL) calculation..

Additionally, precipitation projections for the region have been used to inform the corresponding predictions for probabilities of extreme levels of precipitation at Campbell River. While the Campbell River drainage is both large and controlled, there are a number of small streams near to developed lands in Campbell River. This study specifically examined the lower Willow Creek drainage and estuary. A high level statistical analysis is undertaken to determine the joint probability of rainfall events with extreme wind storm (wave) events. This analysis provides appropriate input of upland precipitation which may cause river and upland flooding coincident to the occurrence of coastal flooding.

2 METEOROLOGICAL AND OCEANOGRAPHIC CONDITIONS SUMMARY

The water level along the Campbell River foreshore 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. Detailed analysis of these effects have been previously studied by NHC for the City in the following reports:

1. City of Campbell River Sea Level Rise Study, Phase 1 – Downtown Waterfront Site. Final Report May 2018, Prepared by Northwest Hydraulic Consultants. (NHC, 2018a)

2. City of Campbell River Sea Level Rise Study, Phase 2 – Estuary Assessment. Final Report December 2018, Prepared by Northwest Hydraulic Consultants. (NHC, 2018b)

3. City of Campbell River Sea Level Rise Study, Phase 3 – Additional FCL Assessment. Final Report December 2018, Prepared by Northwest Hydraulic Consultants. (NHC, 2018c)

The following sections include a summary of key findings from these previous reports as they relate to the present study.

2.1 Tides

Tides near Campbell River are mixed semi-diurnal with annual mean tidal range of 2.7 m and large tidal range at 4.3 m. **Table 2-1** presents local tidal water levels based on values obtained from Campbell River from 2018 Canadian Tide and Current Tables Volume 6. Elevations are referenced to Geodetic Datum (GD, measured relative to Mean Sea Level) and Chart Datum (CD, measured relative to Mean Lower Low Water).

Table 2-1: Summary of Campbell River tide elevations

Sea State	Tide Elevation (m Geodetic Datum)	Tide Elevation (m Chart Datum)
Higher High Water, Large Tide (HHWLT)	1.7	4.6
Higher High Water, Mean Tide (HHWMT)	1.2	4.1
Mean Water Level (MWL)	0.0	2.9
Lower Low Water, Mean Tide (LLWMT)	-1.5	1.4
Lower Low Water, Large Tide (LLWLT)	-2.5	0.4

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 Fisheries and Oceans Canada tide station (8074 - Campbell River) water level data (from 1972 to 2016) by first removing the tidal component from the measured water level to obtain the tidal residual. Extreme value analysis was then conducted using the Peak-Over-Threshold method by considering tidal residual[2] values occurring when tides were greater than HHWLT. The results are summarised in **Table 2-2** below.

[2] 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.

Table 2-2: Summary of design storm surges

Return Period (yr)	AEP (%)	Storm Surge (m)
5	20	0.66
10	10	0.72
20	5	0.78
50	2	0.86
100	1	0.91
200	0.5	0.97

The takeaway from the above analysis is that the value for frequent occurring storm surge events (20% AEP [3]) is about 0.7 m. The difference between the 20% AEP storm surge and less frequent occurring 0.5% AEP storm surge is only 0.3 m. The maximum observed water level over the 45 years of record was 2.35 m GD on January 15th, 1974. The predicted tide level corresponding to this peak observation water level was 1.64 m GD which is just 0.07 m below HHWLT at Campbell River. The storm surge was 0.71 m which is close to the 10% AEP value.

2.3 Global Sea Level Rise

The sea level rise (SLR) policy for BC (BC Ministry of Environment, 2011b) recommends assuming a 1.0 m rise in global mean sea level between the year 2000 and 2100 as show in **Figure 2-1**.

[3] Annual Exceedance Probability, or the probability of an event of equal or greater magnitude occurring in a given year.

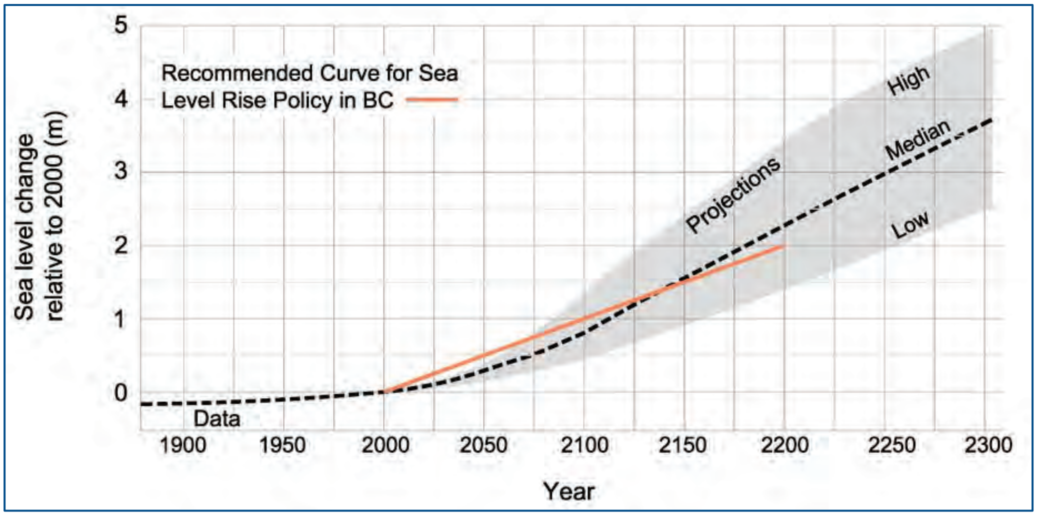


Figure 2-1: Projections of global sea level rise (BC Ministry of Environment, 2011b).

There is uncertainty in sea level rise projections presented in the draft provincial sea level rise policy, as shown in **Figure 2-1**, ranging from about 0.5 m to 1.3 m by 2100 and 1.4 m to 3.4 m by 2200. At the time of the preparation of the sea level rise policy, it was considered that a 1.0 m sea level rise estimate by 2100, which is in the upper limit of the range of estimates, would allow planners to be ahead of the curve. However, in recent years scientific papers have been concluding based on emerging information that SLR may occur faster than previously thought, although there is much uncertainty due to the complex interactions of potential feedback loops in nature as well as future human behaviour. A federal government study for Canadian coastal waters (Han et al., 2016) noted SLR of up to 1.63 m by year 2100 for a business as usual scenario, while the Canada Climate Change Report (Bush and Lemmen, 2019) noted sea level rise of 1.39 m or higher are possible by the year 2100.

Given the increasing certainty that SLR will occur, but the continued uncertainty about the rate of future SLR, it is worthwhile to consider 1.0 m of SLR for planning purposes but acknowledge that the timing of this event could vary considerably depending upon many factors including future human behaviours. Given these uncertainties, it is considered to be a reasonable and an appropriate approach for intermediate and long-range planning purposes to consider both 1.0 m and 2.0 m of SLR respectively, while acknowledging uncertainty on the timing. It is recommended that the City monitor predictions for sea level rise going forward to ensure City planning is in-line with updated science.

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). To the year 2100, this translates to a rise in land elevation relative to sea level of 0.41 m, which would partially counteract the effects that global SLR will have on local water levels.

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 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**. 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

*this station measured data during daytime hours only

Analysis finds 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 potentially does not adequately represent wind conditions that will generate waves in open water. This station is excluded from the analysis for predicting wave height in the northern Strait of Georgia.

Due to the shoreline orientation, the Campbell River foreshore is subjected to the greatest wave effects from southeasterly storms. Frequency analysis was conducted on the Sentry Shoal and Comox Airport hourly wind data for the period of record to obtain the design wind speed[4] for the southeasterly events. The results are summarised in **Table 2-4**.

[4] The design wind speed is based on observation data that 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.

	Sentry Shoal – Southeasterly		Comox Airport – Southeasterly	
Return Period (yr)	Speed (m/s)	Speed (km/hr)	Speed (m/s)	Speed (km/hr)
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 the values are less likely influenced by topographic (land) features.

2.6 Regional Wave Climate

Wave heights in the Strait of Georgia are limited by fetch[5] instead of by wind strength and duration. Looking at 26-years of wave data (1992 to 2017) collected at the Environment and Climate Change Canada (ECCC) Sentry Shoal buoy located 22 km southeast of Campbell River, there were a total of 51 events in the record for Sentry Shoal with significant wave height (Hs)[6] greater than 3 m. However, the data between 1998 and 1999 are doubtful as there were 35 storms with waves over 3 m in these two years, as well as waves over 9 m in the record, which is physically unlikely based upon the location and fetch lengths. NHC removed these two years from the historical analysis. In the truncated data, 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).

2.7 Local Wave Climate

Wave-current interaction is important in Discovery Passage due to the strong tidally-induced currents, that can reach up to 9 knots (4.6 m/s), and can result in a large localized increase to wave heights. The most noticeable effect occurs when waves propagate against the direction of the current.

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

[5] Fetch is the distance across a body of water over which the wind can blow and generate waves.

[6] The significant wave height is defined as the mean wave height of the highest third of the waves.

(20 m) along the Campbell River foreshore (**Figure 2-2**). A detailed description of the model development is provided in the Phase 1 report (NHC, 2018a).

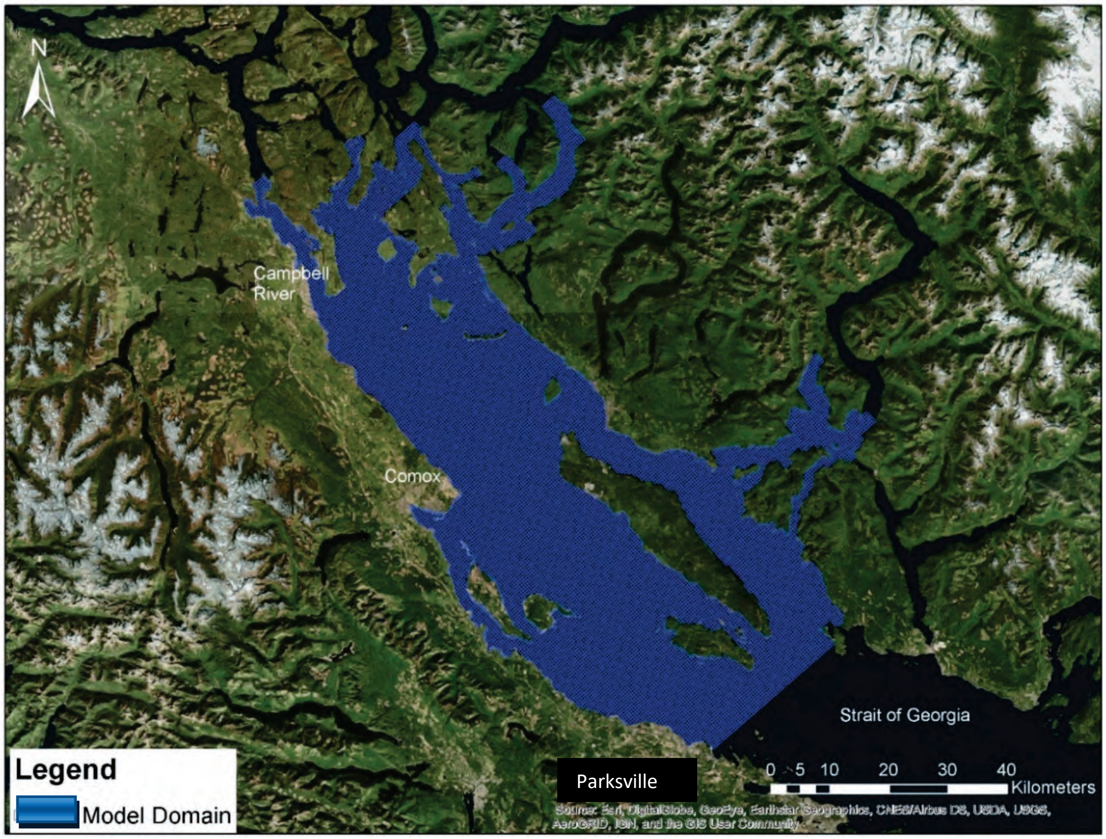


Figure 2-2: Campbell River hydrodynamic and wave model grid domain extents.

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"[7] (not to be confused with rip currents) 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 an ebb-slack to a flood tide in Discovery Passage, the opposing currents along the leading edge of the flood tide generate the most severe rips.

2.8 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

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

examine the probability of this at Campbell River, a joint probability analysis was conducted utilizing the historical water level record at Campbell River (that includes both astronomical tides and storm surge) and a coincident wave record from the Sentry Shoal buoy. Details of the analysis are provided in the Phase 1 report (NHC, 2018a)

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

For the wave effect assessment study, six combinations of wave height and water level (**Table 2-5**) were selected from the 200-year recurrence interval curve (blue line) from **Figure 2-3** in order to determine the tide and storm conditions that would result in the highest flood level along the shoreline.

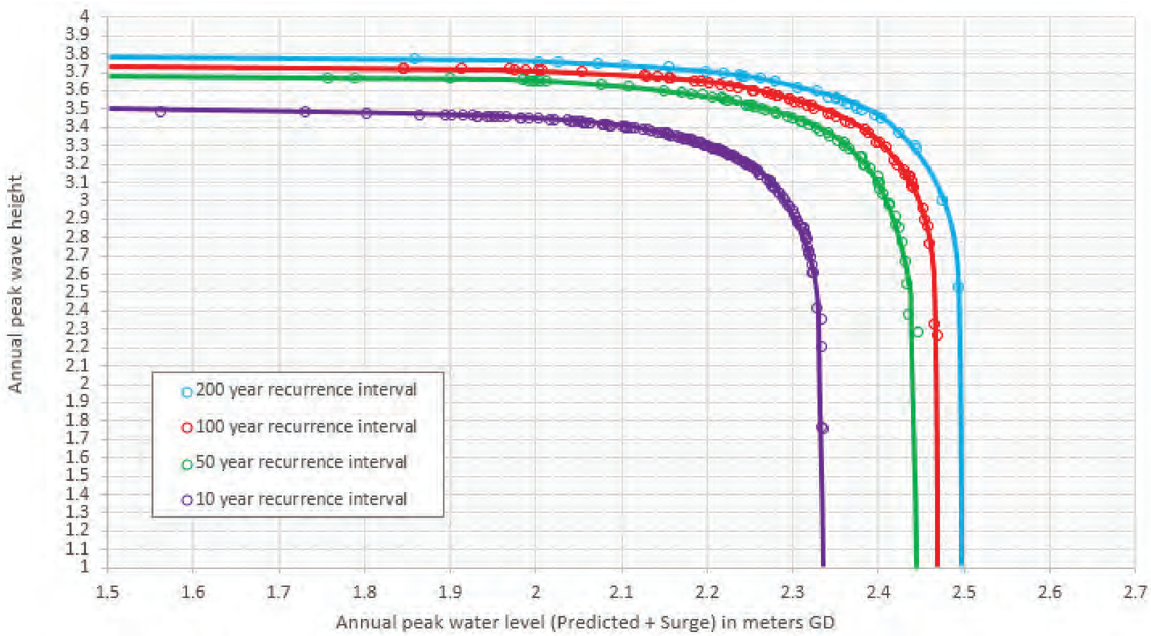


Figure 2-3: Wave height – Water Level curves of equal joint probability.

Table 2-5: 200-year joint probability recurrence for water level and offshore wave height values

Simulation	Water Level (m GD)	Wave height at Sentry Shoal (m)
1	2.50	2.50
2	2.45	3.20
3	2.35	3.55
4	2.30	3.65
5	2.20	3.70
6	2.10	3.75

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. The most severe wave conditions coincident with high tide levels in Discovery Passage are expected to occur towards the end of the flood tide when currents flow towards the southeast against the prevailing wind direction for southeasterly storm conditions. The results of the modelling also indicate that wave heights near to the shorelines are dependent on water depth (in depths below 5 m the larger waves begin to shoal and break) so the relationship between current speed and wave height is not direct. For the concerns of coastal flooding, the wave heights occurring close to periods of maximum water levels (i.e. towards the end of the flood tide) are examined closely. Tides of larger ranges (spring tides as compared with neap tides) generate stronger currents and higher high tides.

For the wave effect assessment analysis, four tidal conditions (**Table 2-6**) were examined for each of the six 200-year recurrence water level-wave height conditions (**Table 2-5**) for a total of 24 simulations. The peak water level over the course of each tidal cycle was adjusted to match the design water level value for each selected 200-year recurrence scenarios (**Figure 2-3**). The wind speed required to achieve the offshore design wave height at Sentry Shoal for each scenario was applied to the model. To illustrate the the effects of local currents and partial sheltering from Quadra Island, a snapshot of the model's output for Simulation 4 is shown in **Figure 2-4**. Wave height distributions are shown by colour and wave direction are shown by vectors[8].

Model simulations showed that the highest flood levels (including wave effects) along the shoreline occurred under flow conditions associated with Tide B and waves and waterlevels from Simulation 2. Thus, the simulations were run with a 2.0 m range tide to generate currents and a combined peak tide and storm surge elevation of 2.45 m GD. The corresponding offshore significant wave height (i.e. the deepwater wave height south of Quardra Island) for the design event is 3.4 m.

Table 2-6: Summary of modelled tide scenarios.

Simulation	Tidal range	Tidal condition
Tide A	+1.0 m	December 14th, 2016
Tide B	+2.0 m	November 12th, 2016
Tide C	+3.0 m	November 29th, 2016
Tide D	+4.0 m	December 16th, 2016

For the purpose of the large area Flood Construction Level (FCL) study (NHC, 2018c) the Campbell River shoreline was divided into 14 representative sections based on shoreline characteristics that are summarized in **Table 2-7** and exposure to shoreline wave conditions as obtained from the above

[8] Vectors have only been shown for every 15 grid cells in the fine grid model

analysis. The extent of the representative sections is shown in **Figure 2-5**. Maximum significant wave heights in the nearshore region at each section are summarised in **Table 2-8.**

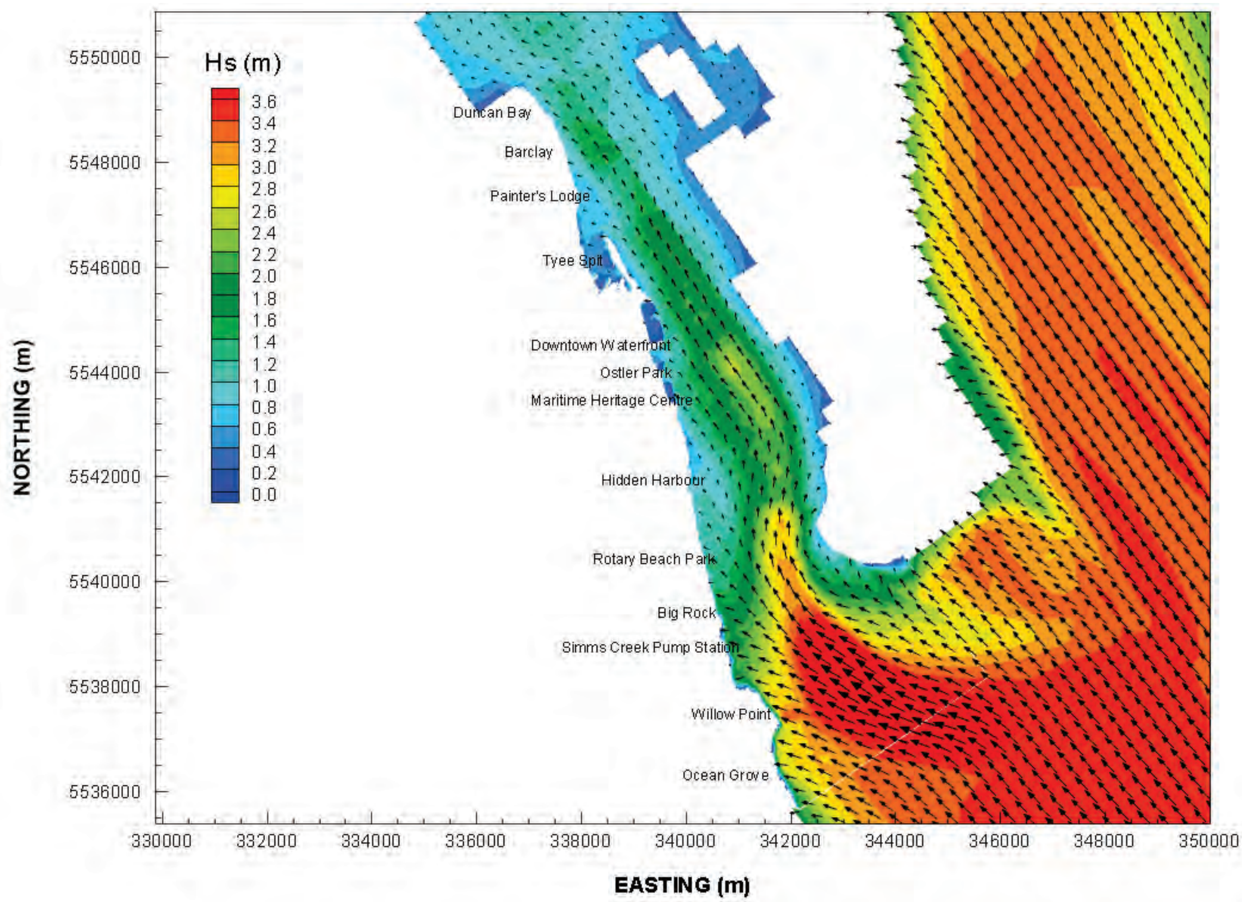


Figure 2-4: Wave height distribution map – Simulation 4.

Table 2-7: Description of representative section descriptions

Section #	Description
1	Ocean Grove
2	Willow Point
3	Frank James Park to Simms Creek Pump Station
4	Simms Creek Pump Station to Big Rock
5	Big Rock to Rotary Beach Park
6	Rotary Beach Park to Hidden Harbour
7	Hidden Harbour to Anchor Inn
8	Anchor Inn to Maritime Heritage Centre
9	Ostler Park
10	Downtown Waterfront
11	Tyee Spit
12	River mouth to McDonald Road
13	McDonald Road to Barclay Road
14	Duncan Bay

In contrast to the other representative sections of the Campbell River shoreline that were investigated in this study. the shoreline orientation at Duncan Bayis more exposed to northerly winds rather than winds from the south and southeast so a separate northerly designated storm simulation was conducted to determine the corresponding design wave height for this site.

nhc

Figure 2-5: Campbell River foreshore sections.

Table 2-8: Nearshore significant wave height at each representative section

Section	Description	Hs (m)
1	Ocean Grove	1.2
2	Willow Point	1.4
3	Frank James Park to Simms Creek Pump Station	1.4
4	Simms Creek Pump Station to Big Rock	1.4
5	Big Rock to Rotary Beach Park	1.1
6	Rotary Beach Park to Hidden Harbour	1.1
7	Hidden Harbour to Anchor Inn	0.9
8	Anchor Inn to Maritime Heritage Centre	1.1
9	Ostler Park	1.0
10	Downtown Waterfront	1.0
11	Tyee Spit	0.7
12	River mouth to McDonald Road	0.8
13	McDonald Road to Barclay Road	1.0
14	Duncan Bay	1.1

3 SUMMARY OF FLOOD CONSTRUCTION LEVEL ANALYSIS

The previously completed analysis is summarized here for the four component areas of this study. The four areas were provided in the request for proposal for this project and delineated in **Figure 3-1**.

Figure 3-1: Map showing the four (4) study areas for this report. (Note that the numbering of the four study areas is independent of the numbering of the shoreline sections for the FCL study.)

3.1 Flood Construction Level - Background

The water level along the Campbell River foreshore is primarily governed by the sea level and incorporates the combined effects of astronomical tide, storm surge, wave effect, future sea level rise and local uplift/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 FCL as follows and illustrated in **Figure 3-2**:

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

+ estimated wave effects from designated storm

+ freeboard

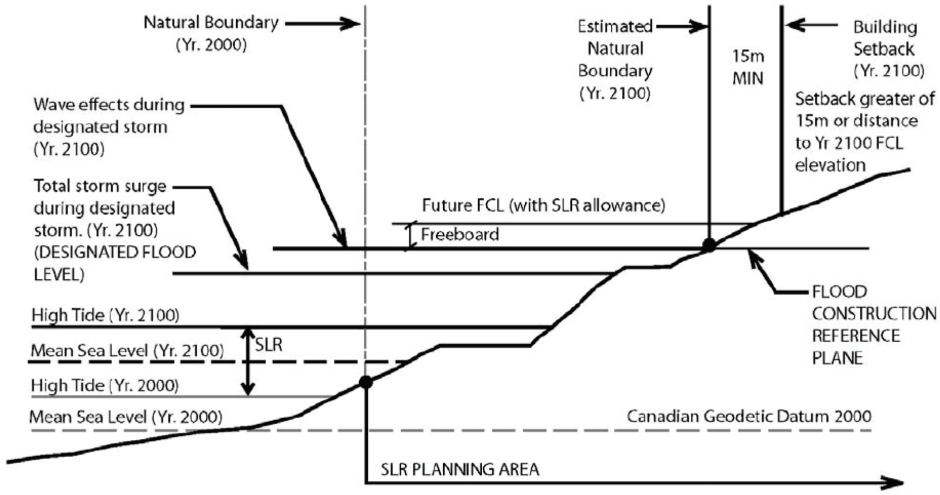


Figure 3-2: Definitions for updated FCL and setback (BC Ministry of Environment, 2011b) .

Although much of the underlying design storm events used by the method presented by the provincial guidelines are 200-year events, the probability of simultaneous occurrence of the suite of associated events is vaguely, if at all, defined. The Guidelines for Sea Dikes and Coastal Flood Hazard Land Use – Draft Policy Discussion Paper (2011a) suggests a probability of 200-year storm surge co-occurring with HHWLT of near 0.025% (4,000-year return period). Annual exceedance probability of this magnitude is stated by the accompanying policy document (BC Ministry of Environment, 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 Campbell River foreshore consists primarily of residential and commercial developments; it is not a continuous sea dike and does not present the same consequence if the design water level is exceeded. The design life of the typical residential and commercial development is likely on the order of 50-years to 100-years and this consideration is incorporated into the present analysis.

As discussed in earlier reports (NHC, 2018a, 2018c) 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 results in conservative values as it does not account for the probability of simultaneous occurrence of the events. A statistical simulation method using the joint probability method combined with a wave-current numerical model was used to reduce the excessive conservatism inherent in the additive approach.

To reduce the likelihood of damage from coastal flood inundation, the coastal flood level was assessed and used to derive a minimum FCL. The coastal FCL using the probabilistic approach is the sum of:

- 0.5% AEP total water level and deepwater wave conditions as determined by probabilistic analyses of tides, storm surge and designated storms;
- Estimated wave effects associated with the designated storm;

- Allowances for future SLR to the year 2050 and year 2100;
- Allowance for regional uplift or subsidence to the year 2050 and year 2100; and
- Freeboard

Each of these components are described in the following section for the four component areas for this study.

3.2 Joint Probability of Water Levels and Deep-water Wave Conditions

As discussed in **Section 2.8**, the 0.5% AEP total water level under present day conditions for the Campbell Rive shoreline adopted for the study is 2.45 m GD, which is applied to each of the four component areas.

3.3 Wave Effect

Wave run-up is the maximum vertical extent of wave uprush on a beach or structure above the still water level. For design purpose, wave run-up is calculated in terms of the two percent exceedance value of wave run-up ($R_{2\%}$) [9].

The wave run-up for each section was estimated using the method described in European Overtopping Manual (EurOtop, 2016) and the results are summarized in **Table 3-1**. The results were applied to determine the FCL values for existing conditions and future conditions. It is assumed in this analysis that the future foreshore slope and beach materials will be the same as that of the existing foreshore, and changes to the foreshore slopes would change the FCL.

It is noted that wave effects are limited to the area immediately adjacent to the shoreline and the wave runup effect does not in general extend large distances inland (i.e. typically less than 15 m to 20 m landward from the top of the shoreline bank upon which wave effects occur). Thus, a shoreline FCL that includes the wave effect is not necessarily an appropriate FCL for properties that are at similar elevations but are 30 m (to be conservative) or more setback from the shoreline. In those cases, the depth of flooding is governed by the volume of water from wave overtopping at the shoreline, upland and precipitation sources of flood waters, and the site drainage. NHC recommends that the coastal flood hazard FCL for properties more than 30 m landward of the shoreline be set to include all components of the coastal FCL (i.e.: total water level (tides and surge), SLR, vertical land movement, and freeboard) and 1/2 of the height of the wave effect to account for potential localized flooding from wave runup and

[9] $R_{2\%}$ is defined as the elevation that only two percent of the waves (i.e. 1 wave in 50) that are observed on a shoreline during the peak of a storm will reach or exceed.

It is also noted that the definition of wave run-up (BC Ministry of Environment, 2011b) for "coastal flooding hazard management [is] … taken as 50 per cent of the calculated run-up elevation on the natural shoreline." This reference is for natural zones (such as park land or bike paths) where temporary flooding could be an acceptable risk, and the 50% wave effect is recommended for land planning purposes to estimate the future position of the natural boundary.

spray in low lying coastal areas. (The use of 1/2 of the wave effect is adopted as this corresponds with the Provincial Guidance for the estimation of the future location of the natural boundary.)

Table 3-1: Summary of estimated $R_{2\%}$ wave run-up along Campbell River foreshore (year 2018)

Area	Section	Description	Hs (m)	Foreshore slope	Surface roughness	Wave run-up $R_{2\%}$(m)
Willow Point	2a	Willow Point South	1.2	Vertical Wall	Gravel	3.7
	2b	Willow Point Central	1.3	2H:1V	Riprap	2.6
	2c	Willow Point North	1.2	3H:1V	Riprap	2.0
	3	Frank James Park to Simms Creek PS	1.1	6H:1V	Gravel	2.1
MHC-Evergreen	7	Hidden Harbour to Anchor Inn	1.0	3H:1V	Small Riprap	1.8
	8	Anchor Inn to Maritime Heritage Centre	0.9	2H:1V	Riprap	2.1
Downtown	9	Ostler Park	0.9	3H:1V	Riprap	1.8
	10	Downtown Waterfront	1.1	2H:1V	Riprap	1.9
Painter-Barclay Escarpment	12	Campbell River to McDonald Road	0.8	Vertical wall	Smooth	2.4
	13	McDonald Road to Barclay Road	1.2	Vertical wall	Smooth	3.0

* Note that section numbers are consistent with the FCL Study previously undertaken for the entire Campbell River Shoreline.

3.4 Sea Level Rise

As discussed in **Section 2.3**, the sea level rise policy for BC (BC Ministry of Environment, 2011b) recommends assuming a 1.0 m rise in global mean sea level between the year 2000 and year 2100. This value is applied to each component area.

3.5 Uplift/Subsidence

As discussed in **Section 2.4**, the provincial guidelines (BC Ministry of Environment, 2011c) suggest a local uplift rate of 4.1 mm/yr for Campbell River. To the year 2050 and year 2100, this translates to a rise in land elevation relative to sea level of 0.21 and 0.41 m, respectively, and a partial offset of the expected sea level rise.

3.6 Freeboard

It is common practice to include provision for uncertainties by incorporating a minimum freeboard. Guidelines for Management of Coastal Flood Hazard Land Use (BC Ministry of Environment, 2011b) defines freeboard allowance to be the greater of:

- 0.6 m, or:
- For flood proofing fill – crest elevation of equivalent sea dike
- For exposed vertical building foundations – the wave-structure interaction
- For tsunami areas – the run-up elevation of the appropriate tsunami hazard

A freeboard allowance value of 0.6 m was adopted for the present analysis.

3.7 Summary of Flood Construction Levels

Table 3-2, **Table 3-3** and **Table 3-4** summarize the FCL for 2018, 2050 and 2100 respectively. It should be noted that the Painter-Barclay component area (Sections 12 and 13) consists of coastal bluff shoreline features and most of the infrastructure (residences) are located on top of the bluff. The top of bluff is at an elevation well above the estimated FCL; however, there is a sewer line along the foreshore that could be at high risk due to ongoing bluff erosion process.

Table 3-2: Flood Construction Levels – 2018

Section	Design Water Level (m)	SLR (m)	Uplift (m)	Wave Effect (m) *	Freeboard (m)	FCL (m GD)
2a – Willow Point (Vertical Seawalls)	2.45	0.18	-0.07	3.7	0.6	6.9
2b – Willow Point (Central Area)	2.45	0.18	-0.07	2.6	0.6	5.8
2c – Willow Point (North Area)	2.45	0.18	-0.07	2.0	0.6	5.2
3 - Frank James Park to Simms Creek PS	2.45	0.18	-0.07	2.1	0.6	5.3
7 - Hidden Harbour to Anchor Inn	2.45	0.18	-0.07	1.8	0.6	5.0
8 - Anchor Inn to Maritime Heritage Centre	2.45	0.18	-0.07	2.1	0.6	5.3
9 - Ostler Park	2.45	0.18	-0.07	1.8	0.6	5.0
10a - Downtown Waterfront (Hwy 19A)	2.45	0.18	-0.07	1.9	0.6	5.1
10b - Downtown Waterfront (behind breakwaters)	2.45	0.18	-0.07	0.6	0.6	3.8
12 – Campbell River to McDonald Road	2.45	0.18	-0.07	2.4	0.6	5.6
13 - McDonald Road to Barclay Road	2.45	0.18	-0.07	3.0	0.6	6.2

* Wave effect is calculated to be 100% of wave run-up for determination of FCL.

It is noted that there are a number of large breakwaters that protect sections of the downtown waterfront. There is no specific guidance for the determination of FCL along the shorelines in the protected areas behind the breakwaters. NHC recommends that a wave height of 0.6 m is adopted for shorelines in the lee of breakwaters along with a freeboard allowance to determine the FCL. Should these breakwaters not be maintained in the future or raised to accommodate SLR, then this FCL criteria would need to be re-evaluated accordingly.

Table 3-3: Flood Construction Levels – 0.5 m of SLR (year 2050)

Section	Design Water Level (m)	SLR (m)	Uplift (m)	Wave Effect (m)	Freeboard (m)	FCL (m GD)
2a – Willow Point (Vertical Seawalls)	2.45	0.50	-0.21	4.1	0.6	7.4
2b – Willow Point (Central Area)	2.45	0.50	-0.21	2.9	0.6	6.2
2c – Willow Point (North Area)	2.45	0.50	-0.21	2.1	0.6	5.4
3 - Frank James Park to Simms Creek PS	2.45	0.50	-0.21	2.1	0.6	5.4
7 - Hidden Harbour to Anchor Inn	2.45	0.50	-0.21	1.8	0.6	5.1
8 - Anchor Inn to Maritime Heritage Center	2.45	0.50	-0.21	2.1	0.6	5.4
9 - Ostler Park	2.45	0.50	-0.21	1.8	0.6	5.1
10a - Downtown Waterfront (Hwy 19A)	2.45	0.50	-0.21	1.9	0.6	5.2
10b - Downtown Waterfront (behind breakwaters)	2.45	0.50	-0.21	0.6	0.6	3.9
12 - River mouth to McDonald Road	2.45	0.50	-0.21	2.4	0.6	5.7
13 - McDonald Road to Barclay Road	2.45	0.50	-0.21	3.0	0.6	6.3

Note that due to vertical land movement, the local relative SLR is only 0.29 m for this case.

Table 3-4: Flood Construction Levels – 1.0 m of SLR (year 2100)

Section	Design Water Level (m)	SLR (m)	Uplift (m)	Wave Effect (m)	Freeboard (m)	FCL (m GD)
2a – Willow Point	2.45	1.00	-0.41	4.7	0.6	8.3
2b – Willow Point	2.45	1.00	-0.41	3.0	0.6	6.6
2c – Willow Point	2.45	1.00	-0.41	2.4	0.6	6.0
3 - Frank James Park to Simms Creek PS	2.45	1.00	-0.41	2.1	0.6	5.7
7 - Hidden Harbour to Anchor Inn	2.45	1.00	-0.41	1.8	0.6	5.4
8 - Anchor Inn to Maritime Heritage Center	2.45	1.00	-0.41	2.1	0.6	5.7
9 - Ostler Park	2.45	1.00	-0.41	1.8	0.6	5.4
10a - Downtown Waterfront (Hwy 19A)	2.45	1.00	-0.41	1.9	0.6	5.5
10b - Downtown Waterfront (behind breakwaters)	2.45	1.00	-0.41	0.6	0.6	4.2
12 - River mouth to McDonald Road	2.45	1.00	-0.41	2.4	0.6	6.0
13 - McDonald Road to Barclay Road	2.45	1.00	-0.41	3.0	0.6	6.6

Note that due to vertical land movement, the local relative SLR is only 0.59 m for this case.

4 DETAILED COASTAL ANALYSIS

NHC utilized a phase resolving wave model to examine more closely the potential wave effects of runup and overtopping. The purpose of this exercise is to improve confidence in the empirical estimates of wave effect determined in previous analysis, and also to utilize the wave models as tools to examine potential sensitivity of variations in shoreline geometry.

The shorelines of Campbell River are generally characterized by intertidal beaches of generally coarse materials with underlying bedrock at various depths. Upper beaches are generally steeper with gravel and cobbles dominating in most areas as indicative of exposure to waves and current levels that prevent deposition of fine sands. A significant proportion of the Campbell River shoreline has been armoured with rock revetments, seawalls, and breakwaters. The shoreline is exposed to both short period locally generated wind waves along with longer period waves generated in the northern Strait of Georgia.

The previous modelling (**Section 2.8**) utilized the SWAN (Simulating WAves Nearshore) wave model which is a phase averaged model for examining wave generation, propagation, and diffraction. The model was suitable to estimate the nearshore design seastate condition. However, a shortcoming of this model is its inability to resolve individual shoreline structures and features. Further, a phase averaged model is not capable of simulating the propagation of individual waves and their interactions (runup and reflection) with structures. To overcome these shortcomings, a phase resolving model was utilized to examine wave action at the representative shorelines.

The SWASH (acronym for Simulating WAves till SHore) wave model has been utilized in this work to examine non-hydrostatic free surface flows. A full description of the numerical model and its governing equations is given in Zijlema at al. (2011).

The analysis is applied to the four component areas, as described in the following sections.

4.1 Painter Barclay Escarpment

A study of the shoreline noted that there were two characteristic reaches in this component area. In one case, there is a relatively high beach with fine sediments that have been trapped in the area while in other areas the upper beach is lower with a steeper bluff at the top of the beach. The photos below show examples of two such areas (**Figure 4-1**). The two photo locations are not far apart geographically, but the beach on the left image is constrained between shore perpendicular groynes while the beach on the right is not.

Figure 4-1: Photos of the beach at two locations along the Painter-Barclay Escarpment (Source: Ryzuk). The photos are from the 4400 (left) and the 4500 (right) block of Discovery Drive.

The SWASH model was used to confirm the expected wave runup elevations for two sections considered representative of shoreline geometry along the Painter-Barclay escarpment. The model was run for a period of 40 minutes in order to ascertain statistics on the wave run up elevations. The location of the two transects is given in **Figure 4-2**. Transect PB01 is similar to the topography shown in **Figure 4-1** (left) while PB02 is similar to the topography shown in **Figure 4-1** (right).

Figure 4-2: Google Earth image showing the location and extent of the two SWASH model transects (yellow lines).

Maximium wave runup for transect PB01 was found to be 0.89 m above the still water line (**Figure 4-3**). The wave runup increased to 1.26 m for a SLR of 1.0 m for this shoreline due to the deeper water along the beach and increased wave energy reaching the bluff.

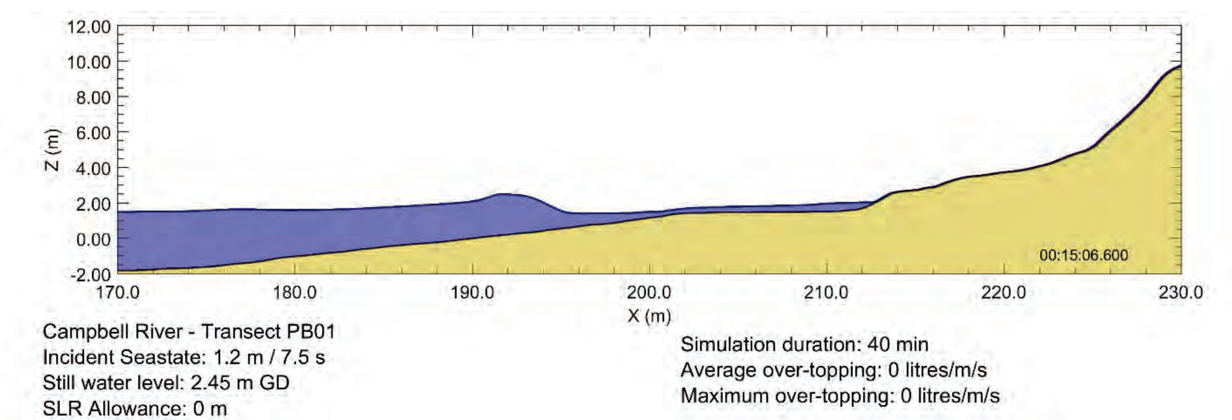


Figure 4-3: SWASH model simulation at PB01 where the beach crest is high.

Maximum wave runup was recorded as 3.4 m at PB02 with a relatively steep beach for a SLR of 0.0 m. With a SLR of 1.0 m, the maximum runup increases to 3.6 m for this model section. It is noted that no vegetation was included in the model, and the presence of vegetation would be expected to reduce the wave runup (**Figure 4-4**). The extent that runup is reduced by vegetation depends on the coverage and thickness of the vegetation.

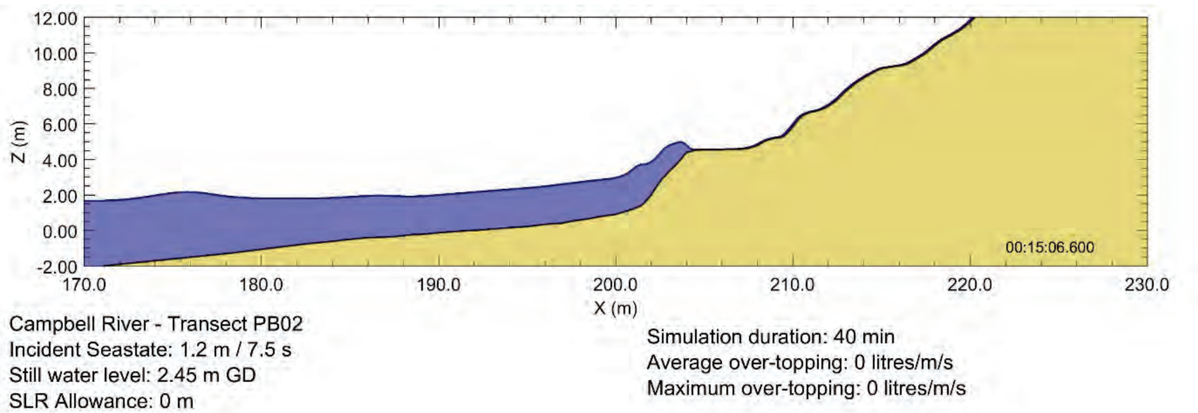


Figure 4-4: SWASH model simulation at PB02 where the beach elevation is lower and a near vertical face is exposed at the lower section of the bluff.

In summary, the wave modelling analysis found that the maximum wave runup varies from as low as 0.9 m to as high as 3.4 m depending on shoreline characteristics. The larger wave runup is observed on shorelines with a steep bluff at the top of the beach that acts similar to a seawall. Without adaptations, the seawall effect is amplified and wave runup (or wave effect) increases to 3.6 m in the future.

It is noted that in the prior report (NHC, 2018c) the general wave effects calculated for the area are between 2.4 and 3.6 m as averaged values without consideration of specific localized areas. Clearly, adaptations that can raise the beach (to induce wave breaking and reduce incident wave energy on the bluff toe) and also that reduce the slope of the toe of the bluff at the top of the beach will serve to significantly reduce wave runup (and hence wave effects) in this area.

That being said, reductions in the FCL are not a direct concern to the residential dwellings in this area as they are located at the top of the bluff, well above the direct effects of SLR. Of prime importance is wave interactions with the base of the bluff, as isconsidering steps to reduce the risk of erosion (or specifically the risk of increased rates of erosion) of the bluff in the future.

A range of bluff erosion mitigation options for this area are discussed in the Lanarc/NHC technical report (Lanarc and NHC, 2018). One mitigation considered is the installation of a steep cobble beach on the upper beach constrained within groynes similar to those that are already present. **Figure 4-5** shows results from a SWASH run with a cobble beach installed on the PB02 transect, with a reduction in wave runup from 3.4 m to 0.7 m for present day conditions.

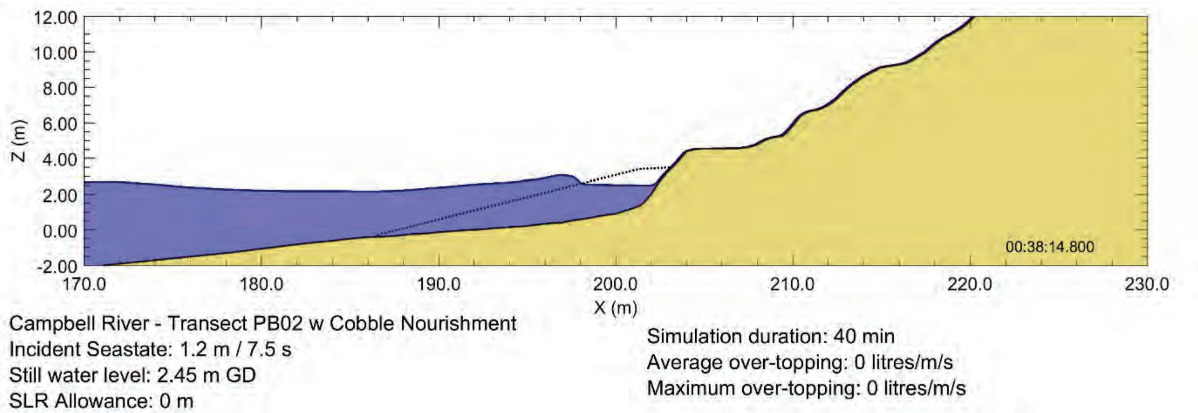


Figure 4-5: SWASH model simulation at PB02 with a cobble beach installed. Maximum wave run was reduced to below 1.0 m with this large cobble beach nourishment.

Based upon inspection of the shorelines, erosion from the escarpment does not contribute sediment to other beaches in the area as the sediments tend to be lost to adjacent sub-tidal zones. As such, measures to slow or prevent erosion in this area will not have negative consequences for other shorelines.

Stabilizing the bluff against future erosion from SLR is important for City infrastructure as well as private property owners. Rock armour revetments in select locations would be effective provided steps are taken to ensure adjacent sections of the bluff do not continue to erode.

The groyne system is shown to be effective in capturing and holding finer sediments on the beaches in this area, and it is recommended that these structures are maintained to continue to function to prevent the beach materials in this area from being lost and exposing the toe of the bluffs to increased erosion.

4.1.1 Bluff Recession at Painter-Barclay

Coastal bluffs are relatively high relief features that are generally steeper than 40°, cannot be overtopped by wave run up, and are formed in unconsolidated or poorly consolidated sediments (Davidson-Arnott, 2010) . Over time, erosional forces will break down the bluff material causing it to recede. It is important to recognize bluff erosion and bluff recession as two distinct but closely interrelated processes. Bluff erosion refers to the rate of loss of material from the bluff due to slope failures or coastal erosion of material from the bluff toe, whereas bluff recession can be defined as the horizontal movement of the overall bluff over time.

Under present conditions, waves typically break on the beach shore and run up the beach slope, only reaching the bluff toe during storm events that coincide with high tides. The shore platform is exposed to, and eroded by, breaking waves, whereas the bluff toe is exposed to erosion from sea water that hits the bluff during wave runup. **Figure 4-1** shows a typical view of the present condition of the bluff at two representative reach types. Logs and sediment deposits along the upper beach indicate that waves are capable of reaching the bluff toe. Fine sands and materials in this area are believed to be eroded sediments supplied by the bluffs.

A geotechnical assessment of the Painter-Barlcay escarpment was undertaken by Ryzuk Geotechnical engineers in 2018 and a complete copy of their report is available in **Appendix A** of this document. The assessment found localized areas of erosion, and areas of concern related to improper drainage control that is weakening the slope.

4.1.2 Bluff Erosion Models

Three bluff erosion models were used to investigeate the erosional response of the bluff to wave actions. These models are described in the following sections.

Wave Induced Bluff Erosion – Conceptual Model

A conceptual geomorphic model of wave induced bluff erosion at Painter-Barclay can be described in general terms as a function between the resistance of the bluff to erosion and the force of waves reaching the bluff toe. This model assumes the episodic and spatially dispersed subaerial erosion[10] of the bluffs that occurs under present conditions would continue, and is independent of wave induced erosion and slides triggered by the loss of bluff toe material.

Future SLR will increase the exposure of the bluff toe to wave impacts. For instance, a storm occurring at the present day higher high water large tide level will occur more frequently in the future than it does today. At present, most waves do not reach the bluff except during storms that coincide with relatively high tides; however, waves will reach the bluff with increasing frequency over time leading to increased erosion and undercutting of the bottom of the bluff.

Furthermore, with SLR there will be an increased probability that a significant storm will coincide with water levels that are capable of causing the upper beach profile to degrade until it is no longer able to naturally rebuild, further increasing the exposure of the bluff toe to wave attack and eventually causing erosion of the bluff that will replenish the beach materials. Modelling indicates an annually occurring storm event is capable of causing shoreline erosion and recession from the combined effects of wave attack and cross-shore sediment transport of the eroded sediment to deeper water. The model results indicate this type of erosion is often temporary (e.g. seasonal) for storms occurring at low to medium water elevation, and the beach profile is capable of rebuilding. However, alterations to the beach profile are more damaging when larger storms coincide with exceptionally high tides. In the future there will be a greater probability of coinciding high water levels and larger storm events, which could substantially change the profile of the upper beach near the bluff toe.

The rate and pattern of bluff erosion is complex and difficult to predict. The rate of bluff erosion will depend on the characteristics of the bluff sediments, such as the material hardness, thickness of bedding layers, presence of joints or fractures, relative strength of each sediment layer in the bluff, and the condition of the upper beach in front of the bluff.

---

[10] Subaerial erosion refers to erosion processes that occur outside of the influence of coastal processes.

The bluff and beach characteristics vary spatially, therefore the bluff's exposure to wave attack and resistance to erosion will similarly vary along the bluff and will determine the bluff recession rate. At present, the surface of the upper beach profile in front of the bluff toe is generally composed of a mixture of coarse sand and cobbles, with human made rock groynes and other non-natural features also present. With SLR, the upper beach will become inundated more often and therefore be more exposed to wave action that could potentially erode and transport this coarse surface layer of material exposing finer sediment that could be more susceptible to erosion. Similarly, erosion of the existing bluff slope surface will remove vegetation cover and could expose softer sediment in the bluffs, which could reduce the slope's resistance to erosion and increase the recession rate.

Bluff retreat patterns may also vary over time. One possible scenario is gradual erosion of the bluff toe until the bluff slope becomes unstable and eventually triggers an episodic mass failure that causes the bluff top to recede suddenly. An alternate scenario is that the bluff erodes during a rare and damaging storm event that causes sudden failure of the slope. The conceptual model conservatively assumes that sediment that is eroded from the bluff and deposited at the bluff toe would eventually be eroded further out of the beach system and therefore would not offer any substantial protection over the long term.

In order to estimate potential future bluff erosion rates and extents, two different analytical approaches were used; the Bruun Rule and the Soft Cliff and Platform Erosion (SCAPE) model. These empirical formulae were developed to estimate bluff recession and are utilized here to provide an estimate of the potential range of bluff recession at Painter Barclay.

Bruun Rule

The Bruun Rule (Ranasinghe et al., 2007) is a 2-dimensional theoretical relationship between shoreline position and SLR that is based on the conservation of mass. It demonstrates that an increased water depth from SLR will cause the shoreline profile to shift landward and upward, and sediment in the upper part of the profile will erode and deposit onto the lower part of the profile until the former equilibrium shoreline profile is re-established. There is little field data to verify that actual recession rates match those projected with the Bruun Rule, and this approach is considered to be more relevant for regional applications. Limitations include the following:

- Applies to soft-sediment coasts;
- Assumes the present-day shoreline is in equilibrium and will be in-equilibrium with SLR; and,
- Assumes all eroded sediment is redistributed to the lower part of the profile, and that the nearshore bed increases to the same degree as SLR (which is not unreasonable provided the groyne structures are maintained to trap sediments).

Soft Cliff And Platform Erosion Model

Soft Cliff And Platform Erosion (SCAPE) is a modelling tool for assessing erosional responses of bluffs. It was developed at the University of Bristol and is a freely available open source software. The model divides the bluff into a series of cross sections and includes several parameters including, tidal range, wave height, and rock strength. Cross section information from available LiDAR data was applied and the model was run over a period of 82 years (between 2018 and 2100) using an estimated bluff recession rate of 3 cm/year and 15 cm/year, which is the range of natural recession rates estimated based on natural recession rates in other bluffs in the region and engineering judgement on the condition of the bluff.

4.1.3 Future bluff recession scenarios

A precautionary approach to land development in coastal areas is important because of the potentially high consequences that can occur from flooding or erosion, and because of the high degree of uncertainty with future SLR and coastal erosion rates. The provincial government has integrated the precautionary principle into its coastal flooding guidelines (MoE 2011), which states that, "land use and building approvals based on FCL for 2100 should also include provisions for adaptive management of land uses to Sea Level Rise to the Year 2200 and beyond".

Two computational and conceptual methods have been used to assess future bluff recession rates to address uncertainties with any single approach, spatial variability in the sediment composition of the bluff and potential for sub-aerial slope failures that are independent of coastal processes, and to show the range in future projections for SLR rates of 0.5, 1.0 and 2.0 m. Given the substantial range in estimates between the two methods, no adjustment has been made for tectonic adjustment in the assessment of bluff recession rates. Both the Bruun Rule and the SCAPE Model have been applied to estimate the potential magnitude of bluff recession with SLR, and the results are summarized in **Table 4.1** for the following methods:

Method 1) Bruun Rule;

Method 2) Soft Cliff And Platform Erosion Model;

Table 4.1: Estimated potential magnitude of bluff recession with SLR, shown as a horizontal distance (m) from the present-day bluff position.

Method	0.5 m SLR	1 m SLR	2 m SLR
Lower Bound			
Method 1	4.3	8.3	17
Method 2	4	6	8
Upper Bound			
Method 1	7.4	14.8	30
Method 2	21	29	41
Median Value	5.9	11.6	23.5

The results above assume no mitigation or protection measures are in place to resist erosion of the coastal bluff at the Painter-Barclay escarpment area, and are thus conservative estimates of the potential range of future bluff recession. It is noted that future bluff recession likely will not be a linear process, but will accelerate with increasing rates of SLR. The results indicate that a suitable setback from the top of the bluff for development will reduce reliance upon shoreline protection measures to mitigate against future hazards and risks from slope failures. However, it is noted that there is City infrastructure (i.e. waterfront sewer system) that would be impacted by bluff recession and setbacks at the top of the bluff do not fully protect this neighbourhood from future impacts.

4.2 Downtown

The downtown shorelines of Campbell River are largely artificial, having been built to protect the seaward edge of reclaimed lands. **Figure 4-6** shows historical photos of the downtown area in which major changes over time are evident. In the 1950s the intertidal zone was wide with a natural shoreline significantly inland from the present day shorelines. From a coastal engineering perspective, this development has served to move the reclamined land shorelines into deeper waters and expose them to a higher level of wave energy as compared to a natural shoreline. Also, the present day shorelines are notably steeper than natural beaches, and thus there is little upper-intertidal shoreline available in the downtown area.

For the purpose of adaptations to SLR, the lack of existing natural shorelines necessitates the continuation of hard-armour type solutions of rock armour and seawalls. In this case coastal squeeze[11] considerations are minimal given that it has already occurred. Conversely, it is challenging given the historical land reclamation to begin to construct more natural beaches without either considering significant shoreline retreat, or significant volumes of beach nourishment coupled with installation of coastal structures to stabilize the beach nourishments.

Wave modelling was focused on confirming wave-shoreline interactions at two locations in the downtown area: 1) the north end of Ostler Park, and 2) a section of the shoreline adjacent to Highway 19A north of the ferry terminal. The marina harbours were not modelled as the harbour breakwaters can be readily adapted (at least from an engineering perspective, although not without costs) to increasing SLR by adding a layer or layers of additional rock armour. Seawalls and shorelines within the harbours need sufficient freeboard for small waves and the design water levels which is a relatively straightforward evaluation that does not warrant detailed wave modelling (see recommendations for FCL in **Section 3.7**). Adapatation options for the in-land downtown area are futher explored and discussed in the technical background document (Lanarc and NHC, 2019b) and summarized in the following sections.

[11] Coastal squeeze is a term that refers to the loss of the intertidal beach zone that will be increasingly submerged as sea level rise occurs, and will not be able to migrate landward due to the construction of 'hard' shoreline protection structures such as dikes. Hence the beach zone will be 'squeezed' between the rising sea levels and upland infrastructure.

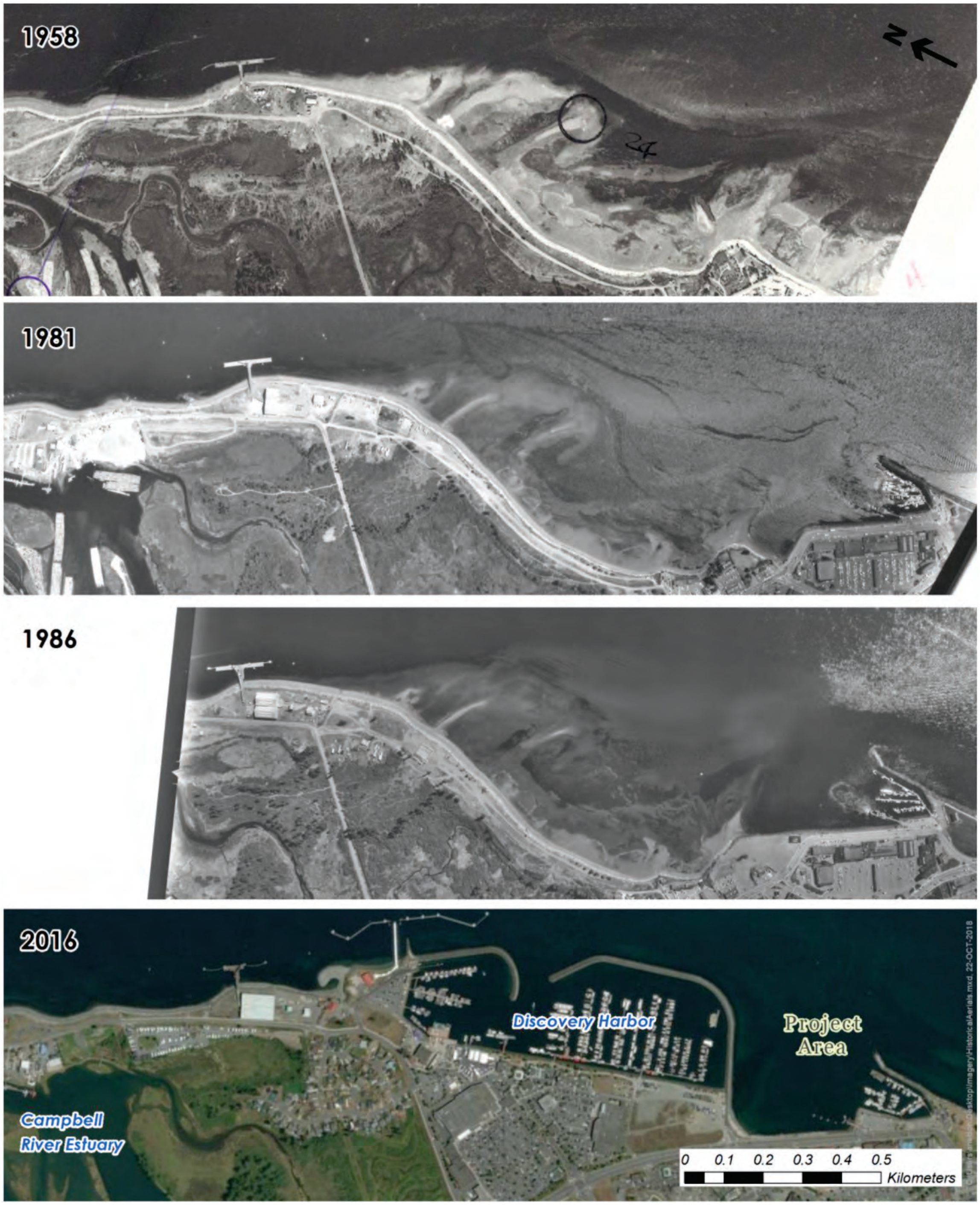


Figure 4-6: Aerial historical photos of the downtown area.

4.2.1 Ostler Park Shoreline Mitigations

The SWASH model was used to simulate wave-shoreline interactions including runup and overtopping in Ostler Park. Numerical modelling was undertaken for a cross section in Ostler Park as indicated in **Figure 4-7**. The section is at the north end of the Park, near to the location of the First Nations longhouse.

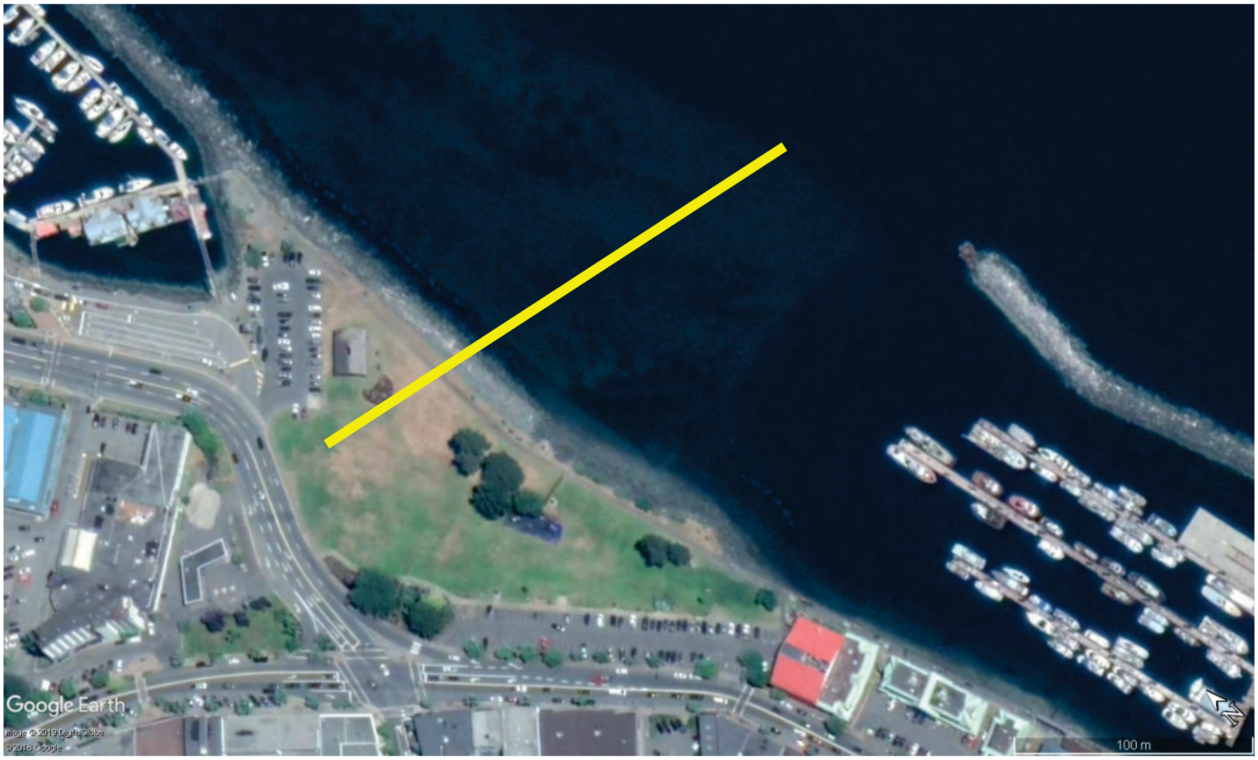


Figure 4-7: Aerial view of Ostler Park showing transect for wave model (yellow)

Mitigation options include raising the shoreline with a variety of treatments (Lanarc and NHC, 2019b). The model was run for a variety of scenarios including:

- the shoreline as it exists today (shoreline crest elevation of ~ 3.1 to 3.2 m);
- a raised shoreline walkway (raised by 0.4 m with a small wall raised by an additional 0.2 m);
- a raised shoreline and park with a crown wall at a crest elevation of 4.8 m; and
- a riprap slope as per existing but with the upper slope graded to a 1:8 natural beach above the riprap. The land in the park is raised by 1 m.

The modeling shows that the existing park is vulnerable to SLR, and that a design storm with 1.0 m of SLR will cause extensive flooding and even waves within the park. Waves are observed in the model to travel across the low areas of the park with relatively high velocity, creating a significant hazard (**Figure 4-8**).

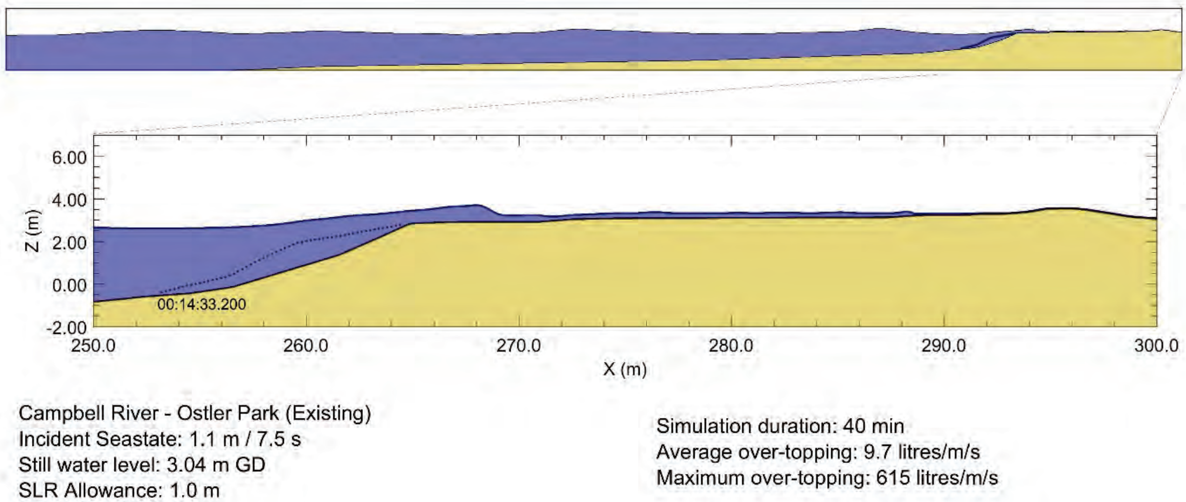


Figure 4-8: SWASH model run for existing Ostler Park shoreline with 1.0 m of SLR.

Mitigations considered for the north end of Ostler Park all built upon the existing riprap slope, but took steps to improve the rock armour and ensure proper design of filter layers and top of slope details. A small curb (**Figure 4-9**) and a larger crown wall (**Figure 4-10**) were considered in combination with a raised walking and bike path along the water. The crown wall can be set to an elevation to completely eliminate over-topping of water into the park with fill added to the park to raise the park sufficiently that ocean views are preserved for people using the park. The fill shown (**Figure 4-10**) is about 0.8 m to 1.2 m elevation of lift across the park area.

Allowing for some landward retreat it is possible to achieve a more gentle vegetated slope from the top of the existing riprap in combination with raising the ground elevations (by 0.8 to 1.0 m) to reduce the flood hazard (**Figure 4-11**). This allows for a more natural top of beach, but would still require rock armour on the shoreline slope to avoid erosion and scour damage to the shoreline during storms. Further south towards the marina, with the sheltering of the breakwater there is also the option to have a more natural slope with smaller sediments such as gravels and sands, but this would involve beach nourishment beyond the present natural boundary.

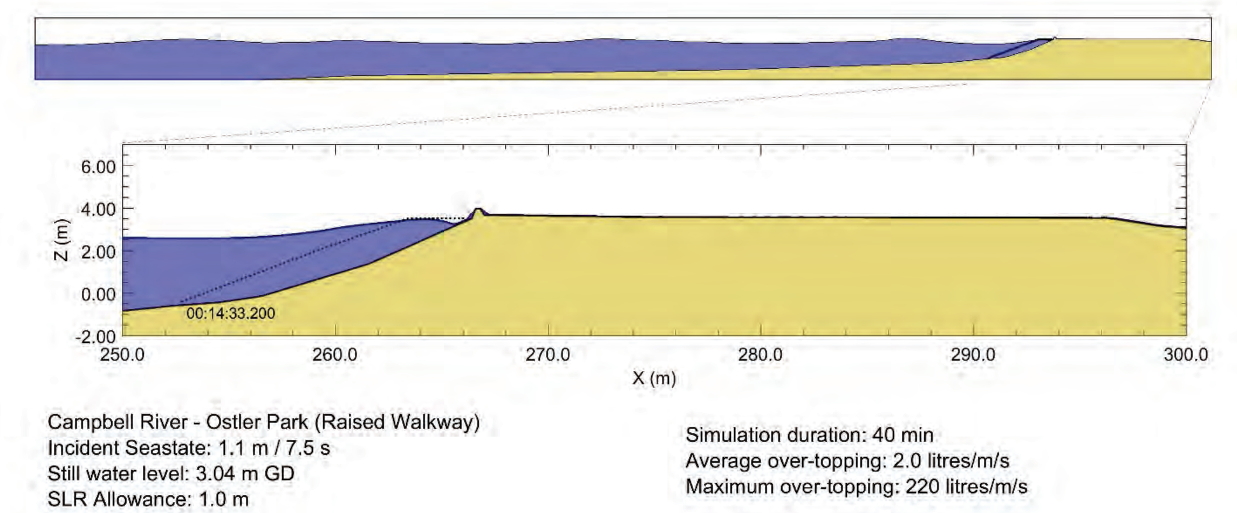


Figure 4-9: SWASH model run for Ostler Park with 1.0 m of SLR. The model geometry is for a modification with elevated bike/walk path and raising of the park land by ~0.4 m. There is a small 'curb' that is 0.2 m high to separate the riprap from the bike/walk path.

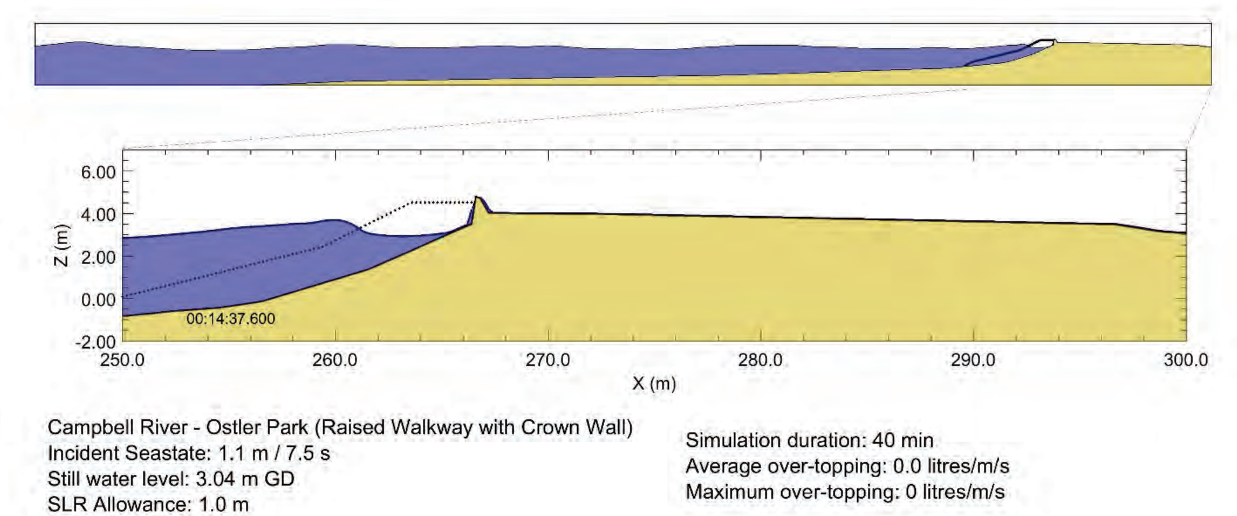


Figure 4-10: SWASH model run for Ostler Park with 1.0 m of SLR. The model geometry is for a raised path with a crown wall. The crest elevation of the crown wall is 4.8 m, and the pathway is at ~3.9 m elevation.

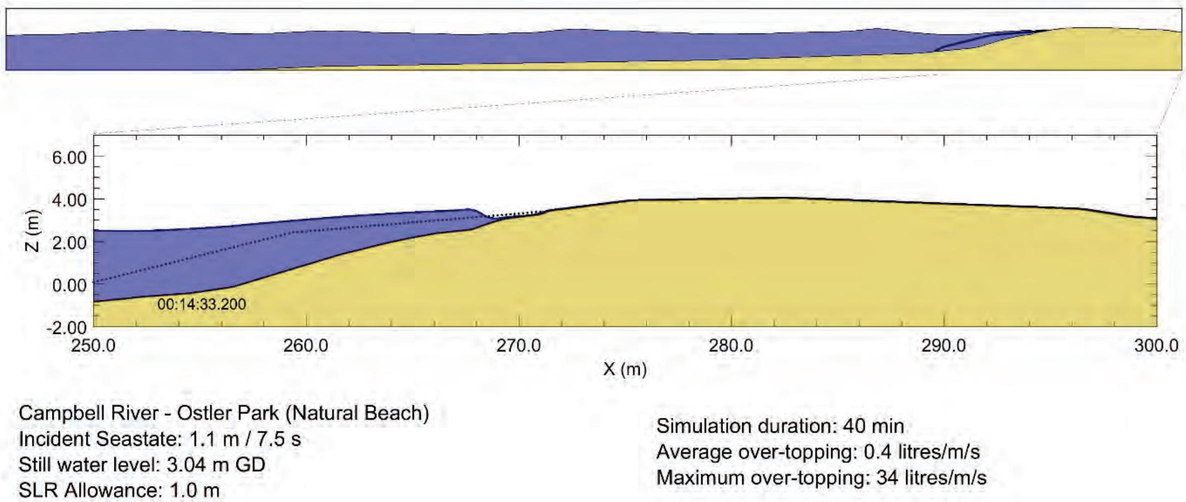


Figure 4-11: SWASH model run for Ostler Park with 1.0 m of SLR. The model geometry is for a raised park with a more natural beach and gently sloped top of rock armour transition. (Replacing the crown wall and bike/walk path with a 1:8 slope and vegetation)

Wave modeling using SWAN was also undertaken to examine the potential benefits of increasing the lengths of offshore breakwaters to better shelter the shorelines and reduce wave effects and thus reducing the FCL at the shoreline. **Figure 4-12** shows that a 100 m extension to the small craft harbour breakwater better shelters Ostler Park shoreline, as would be expected. The wave heights at the northern end of the park are reduced from ~1.4 m (Point 6 in the figure) to ~0.6 m.

All of the proposed mitigations for Ostler Park include raising the shoreline or the land in the park to create a barrier to drainage of flood waters from the downtown area into the ocean in the event of flooding in the area. Thus such mitigations would need to include provisions for upgrades to storm drainage and possibly the need for a pump station to prevent flooding in the downtown area during a storm event.

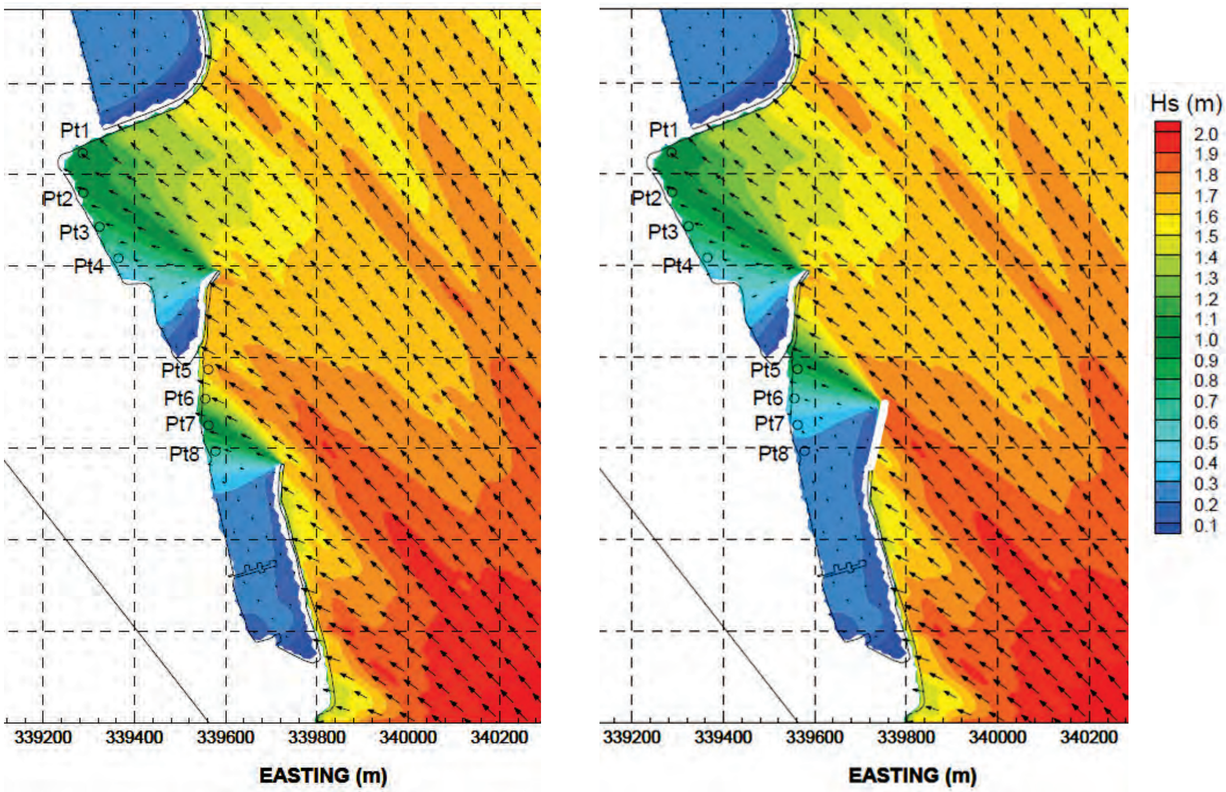


Figure 4-12: SWAN run showing design wave conditions at downtown site for the existing breakwater configurations (left) and with a 100 m extension to the federal Small Craft Harbour breakwater (right).

4.2.2 Downtown Shoreline – Highway 19A

The existing shoreline along Highway 19A just north of the ferry terminal is a relatively steep rock armour slope. There is a sidewalk immediately landward of the rock armour slope, with the road running adjacent to this. **Figure 4-13** shows the alignment selected for the wave modeling.

Figure 4-13: Google Earth aerial photo of the Highway 19A alignment north of the ferry terminal. Wave model transect shown in yellow.

The wave modeling examined the existing shoreline, and the potential hazard from future SLR. It also was used to determine what elevation the shoreline and highway need to be raised in order to have a safe level of runup and overtopping. **Figure 4-14** shows a section view of a proposed mitigation that includes lifting the roadway and raising the walkway.

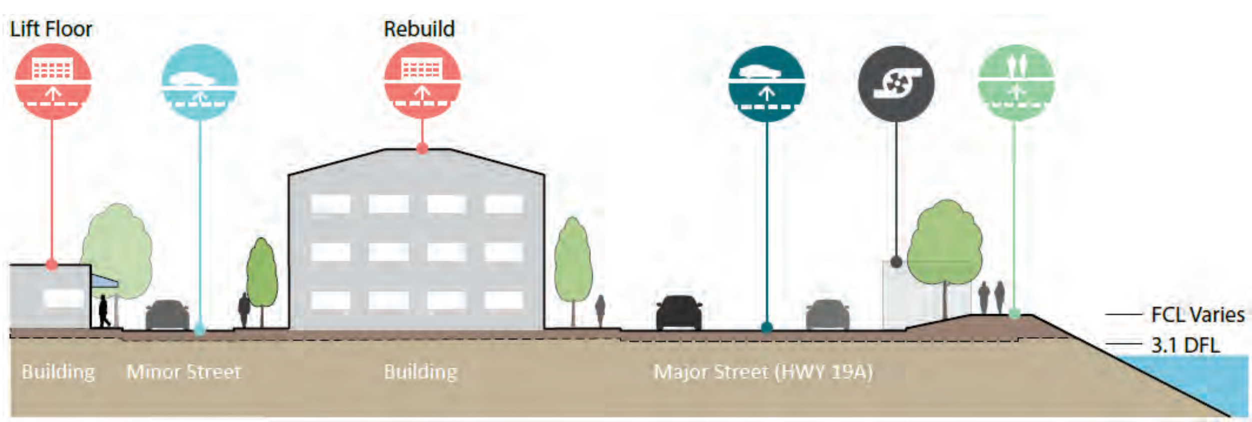


Figure 4-14: Option C of the Coastal Flood Management Area, Primer 3 (Lanarc and NHC, 2019a).

The existing shoreline as modeled was found to have moderate overtopping from a 200-year design event at present day water levels (**Figure 4-15**) and serious overtopping with 1 m of SLR (**Figure 4-16**).

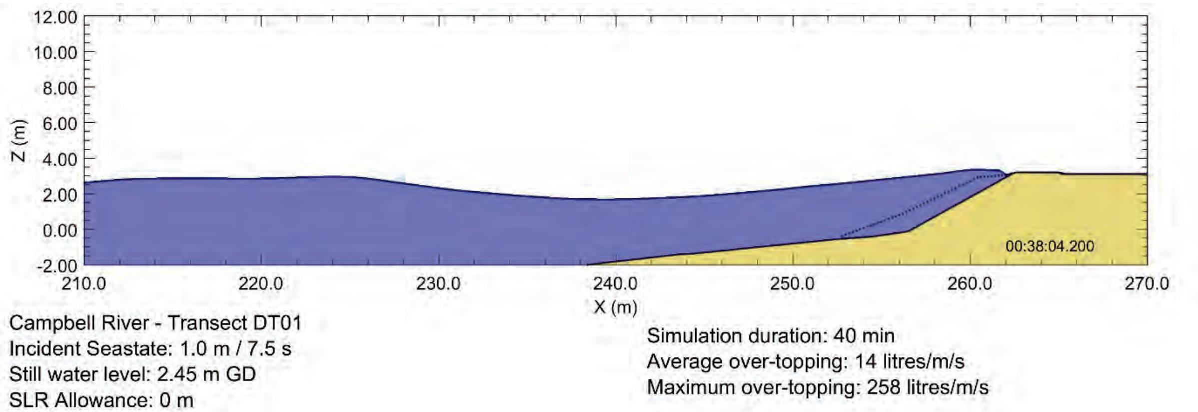


Figure 4-15: SWASH model run for the present day downtown shoreline adjacent to Highway 19A. The largest waves reach the elevation of the sidewalk on runup, with minor overtopping into the roadway is observed.

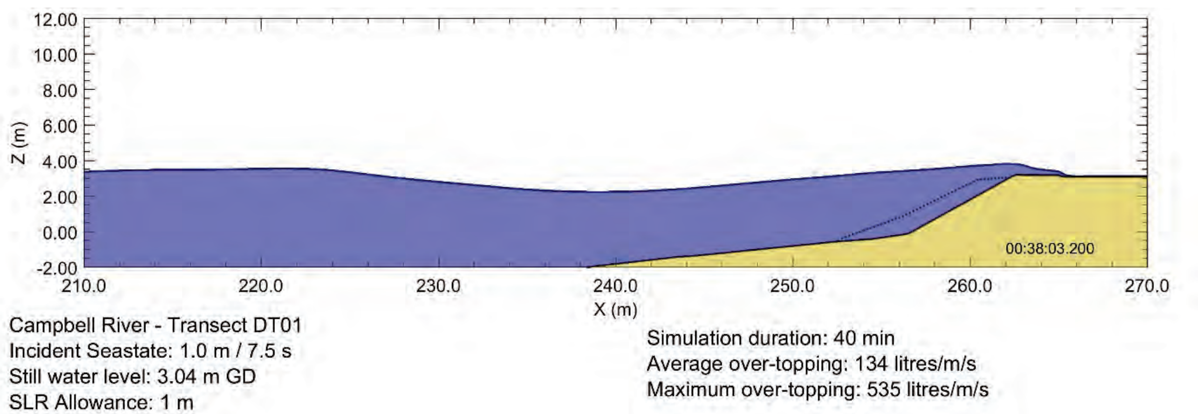


Figure 4-16: SWASH model run for the present day downtown shoreline adjacent to Highway 19A with 1 m of SLR. Significant overtopping is observed with high velocity wave runup reaching the sidewalk. Water depths in excess of 0.3 m cross into the roadway at high velocity.

The mitigation options were modelled, and it was noted that with a vertical lift of 0.75 m for the sidewalk with the rock armour slope extended similarly upwards, wave overtopping is returned to the present day hazard level. The present day hazard level is, for the design storm event, higher than recommended. Additional raising of the shoreline is recommended to create a safe level for the shoreline as noted in the background technical report (Lanarc and NHC, 2019b).

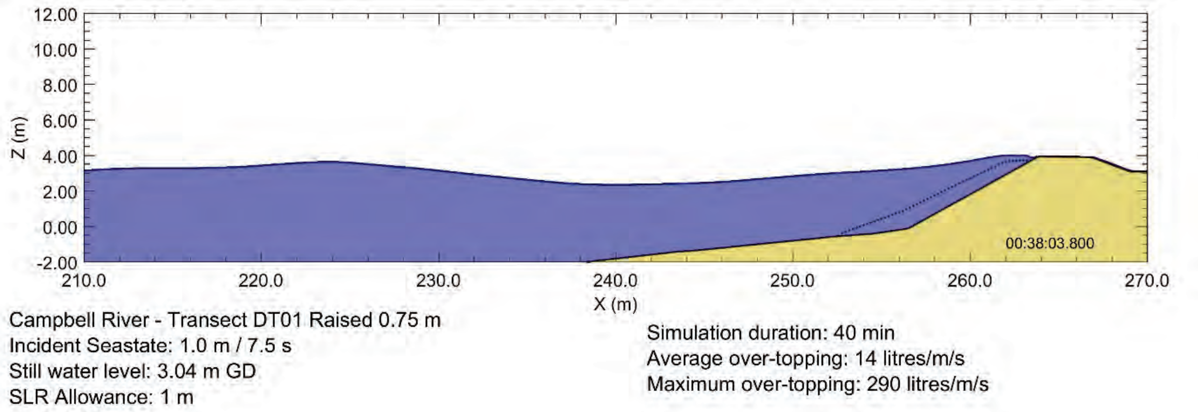


Figure 4-17: SWASH model run for raising the sidewalk by 0.75 m adjacent to Highway 19A. The average over-topping rate is similar to the present day condition for the 200 year storm.

Overall mitigation options for the downtown area are further explored in the City of Campbell River Sea Level Rise Primer 3 (Lanarc and NHC, 2019a) and the Technical Report (Lanarc and NHC, 2019b).

4.3 MHC to Evergreen

The shoreline between the Marine Heritage Centre (MHC) and the small craft harbour just south of the Anchor Inn has mostly been armoured to prevent erosion or to support land reclamation. Proposed mitigations focus on adaptations for the Sequoia Park reach taking into account the existing land use and condition of the shoreline. From a coastal engineering perspective, the risks posed by SLR to the area include increasing overtopping of existing armoured shorelines and erosion to existing shorelines. Also, the period of time each day available for public access to the existing beach will decrease under future SLR. Mitigations thus focused on approaches that would increase the resilience of the existing shoreline to hgiher water levels and storm damage. Consideration was also given to develop an option that would increase public access and utility of the beach in this area. The development of the options is more fully explained in the Technical Report that accompanies this report describing the coastal engineering analysis of the feasibility of these options.

The first option (A) was simply upgrading rock armour revetments to protect city infrastructure. Options B and C for this area include protection of parts of the shoreline using a headland and pocket beach mitigation approach as shown in **Figure 4-18**.

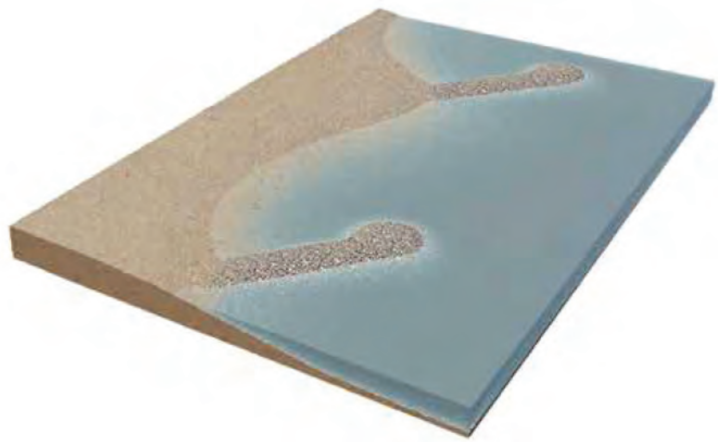


Figure 4-18: Sketch of a Pocket Beach with headland concept. (Lanarc and NHC, 2019a)

NHC undertook analysis to determine, at a preliminary level, the appropriate headland spacing and the expected natural beach positions for the predominate seastate climate. This work utilized the Static Equilibrium Bay model as developed by Hsu et al. (1989) and Silvester and Hsu (1993), the theory being that a pocket beach constrained within two headlands (non eroding and emergent structures) will form a natural equilibrium shape composed of a spiral and a tangent shoreline section. Through setting the position and spacing of the headlands, the range of potential shoreline positions can be controlled and thus erosion can be constrained.

The pocket beaches, as they are known, have the advantage of being constructed from finer materials and thus allow for a composite shoreline that is not completely comprised of heavy rock armour, opening opportunities for recreation and ecological enhancement of the park. **Figure 4-19** shows a conceptual layout for a headland and pocket beach system with five headlands.

Figure 4-19: Sketch of a Pocket Beach system at Sequoia Park beach. Beach shape and headland spacing designed using the equilibrium embayment methodology. Red arrow shows the incident wave direction for the pocket beach shape.

Figure 4-20 shows the same model but with the headland beach system shown over the LiDAR surface terrain for the area.

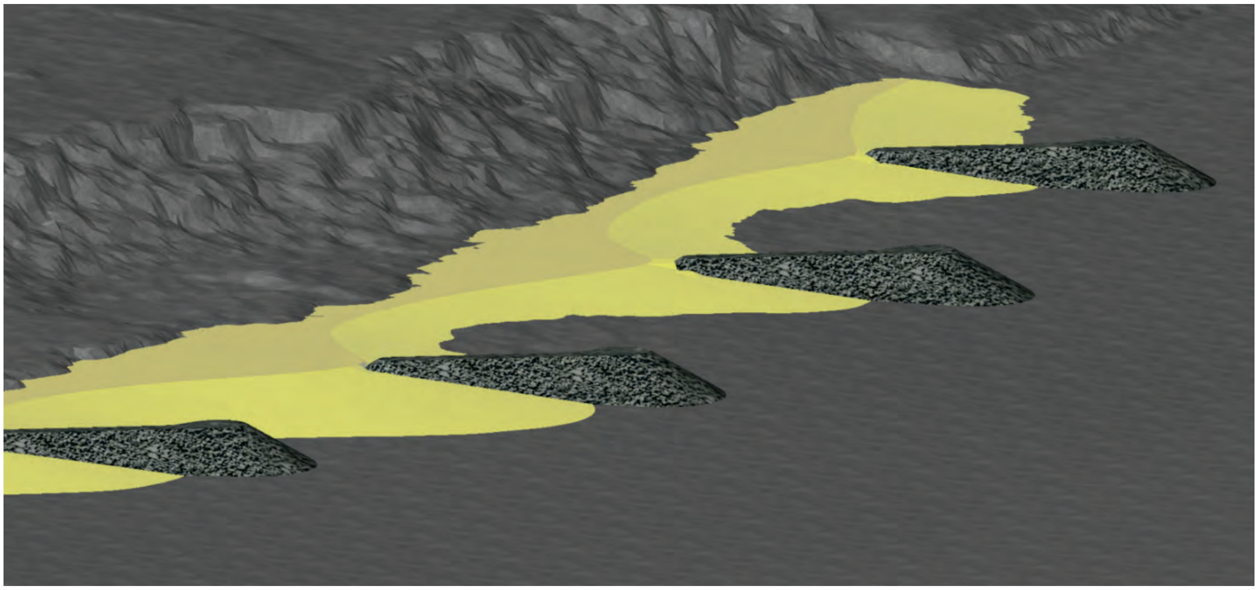


Figure 4-20: Oblique view from SE of headland beach system model at Sequoia Park. Proposed cobble beach fill shown in yellow to highlight this material as separate from the large rock armour headlands. (Lanarc and NHC, 2019b)

The wave runup is attenuated by a combination of the headlands and the cobble beach fill, thus reducing wave effects at the existing shoreline and accommodating a local level of SLR of up to 1.0 m. In time, the cobble beach and headland systems will need to be raised to adapt to increasing SLR above 1.0 m but overall this approach is adaptable through additional materials and increasing the footprint.

This mitigation is proposed as it provides shoreline protection against erosion, reduces the wave effects along the shoreline (thus negating a degree of wave runup), provides intertidal habitat and complexity along the rocky headlands, and increases the upper beach area for the park setting using cobbles that can be suitable for walking upon.

4.4 Willow Point

The Willow Point area has several distinct shoreline features. Immediately south of Willow Creek is a long gravel beach with the Ken Forde Boat Ramp at the north end of this beach cell. North of this is the Willow Creek estuary that is protected by a small breakwater, and immediately north of this is a vertical concrete seawall. Continuing north, the Willow Point shoreline is characterized by a wide and generally flat intertidal zone covered in small to medium rocks, with a small upper beach upon which shoreline protection in the form of rock revetments have been built. The rock revemtents are generally continuous until Adams Park, with additional rocky revetments protecting private properties along the shore stretch of shoreline between Adams Park and Frank James Park (**Figure 4-21**).

Figure 4-21: Aerial image of Willow Point (credit: Google Earth Image, 2018)

Key observations of this area are that the wave energy is largely dissipated on the wide intertidal flats between mean tide conditions and mean high tide water levels. However, with higher high tides and storm surges the situation quickly changes with significant wave energy reaching the upper beach areas. As a result, this areas is particularly vulnerable to SLR, as small increased in sea level will greatly increase the intensity of the wave energy reaching the upper beach during design storm events. The following sections describe various mitigation options.

4.4.1 Typical Rock Revetment – Existing Shoreline

The typical rock revetment along the central section of Willow Point (i.e. typical of shoreline near Jaycee Park) was modeled using SWASH. **Table 4-2** shows how increasing SLR results in negligible increases in runup elevation unless the shoreline is additionally raised. The wave, upon reaching the crest of the rock revetment, stops travelling upwards and instead spills over the revetment crest. For the present day conditions the maximum wave runup is 1.6 m. It is noted that $R_{2\%}$ [12] is expected to be slightly lower than the R_{max}.

Table 4-2: Summary of wave runup and overtopping from SWASH for Jaycee Park shoreline

Run	SLR	Water Level (m)	Wave run-up R_{max}(m)	Runup Elevation (m)	Mean OT (litre/s/m)	Max OT (litre/s/m)
1	0	2.45	1.6	4.05	0.9	137
2	1.0	3.04	1.0	4.07	26	658
3	1.6	3.64	0.4	4.08	198	1220

A key outcome of the above analysis is that the total runup elevation is largely tied to the shoreline geometry and if setbacks are adhered to, then site drainage and acceptable tolerances for over-topping volumes become equally important design parameters with runup and FCL. **Figure 4-22** shows that the land is slightly lower behind the shoreline, yet modeling indicates that waves will transform into overland bores[13] upon overtopping with heights up to 0.2 m. This highlights the importance of site drainage and setbacks for protection and consideration of mitigation of high velocity overland flows near the shoreline.

[12] $R_{2\%}$ is the runup elevation exceeded by only 2% of waves.

[13] An over-land bore is a wave front that travels across the land. It is similar in shape to a hydraulic jump, comprised of either a smooth wave front with a number of smaller trailing waves, or the face of the wave can be very turbulent with strong turbulence.

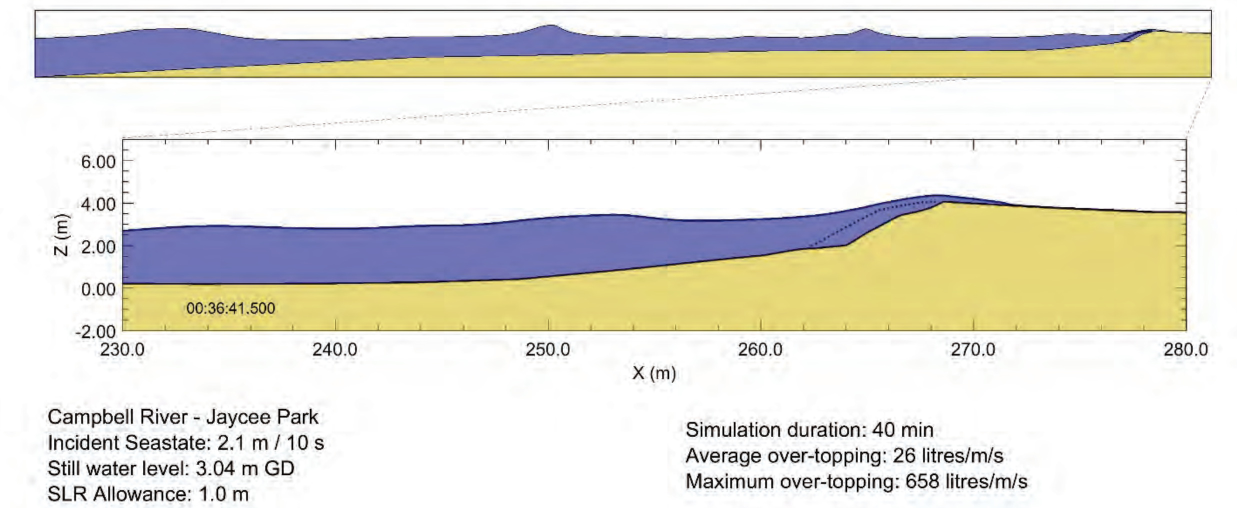


Figure 4-22: SWASH model run for existing shoreline at south end of Jaycee Park area. Rock revetment is critical to attenuating wave runup and overtopping under future conditions.

Modeling results also found that if the rock armour is over-steep or if the rock armour is poorly designed (too tightly packed or without proper filter layers) then wave runup can increase considerably. The recommended adaptations for the central park area of Willow Point include:

- reduce the slope of the existing rock armour;
- increase the armour size (to increase porosity and wave absorption) in order to reduce wave runup; and
- raise of the crest elevation of the rock armour by at least 0.6 m.

These are the recommended adapations that are presented for Option A for the central park shorelines (**Figure 4-23**).

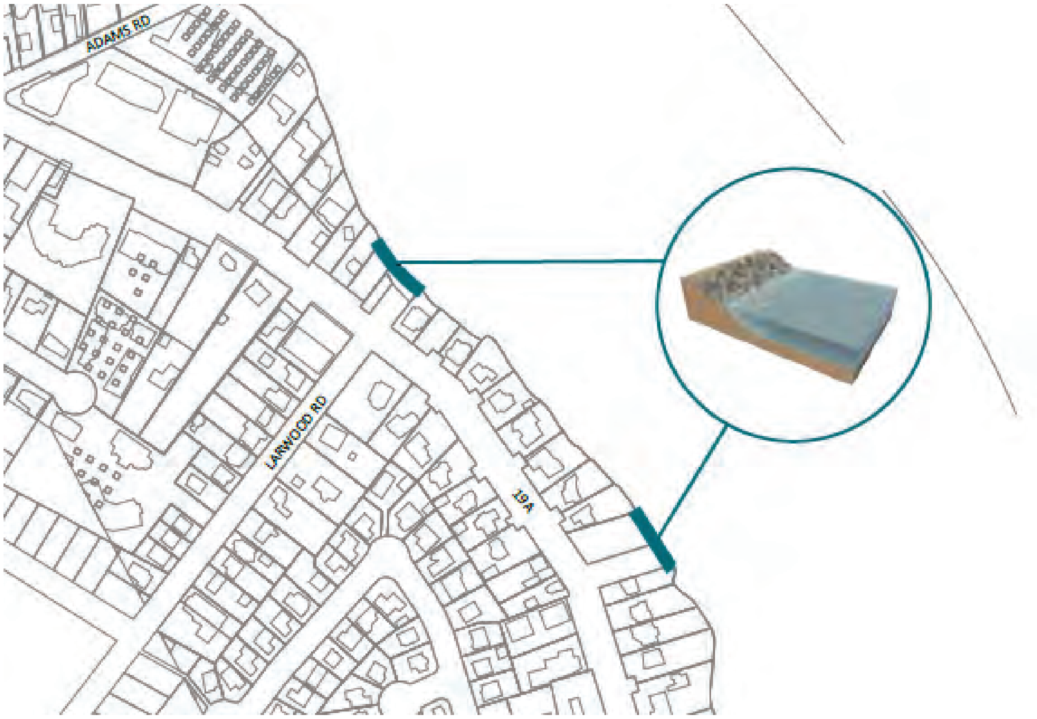


Figure 4-23: Option A of the Coastal Flood Management Area (Lanarc and NHC, 2019a) for Willow Point central area.

4.4.2 Offshore Breakwaters and Beach Nourishments

Several options were explored that would reduce wave energy at the shoreline that is fronted by the relatively flat intertidal bench that exists at Willow Point. One option is to construct offshore breakwaters on the intertidal bench and in select areas, undertake beach nourishments. This mitigation has the objective of keeping a more natural shoreline while creating structures that dissipate wave energy offshore. This mitigation involves no hard armouring of the shoreline nor raising of elevations along the beach face, thus allowing direct public access from the upper park areas onto the beach.

While this could be done at all locations of Willow Point north of Willow Creek, a conceptual layout of this mitigation option was investigated specifically for the shorleline at Frank James Park. Wave modelling using SWAN was conducted to confirm performance and estimate the required volumes of material for an offshore breakwater system. Preliminary modeling with SWAN was used to determine appropriate placement locations (**Figure 4-24**) in the vicinity of Frank James Park.

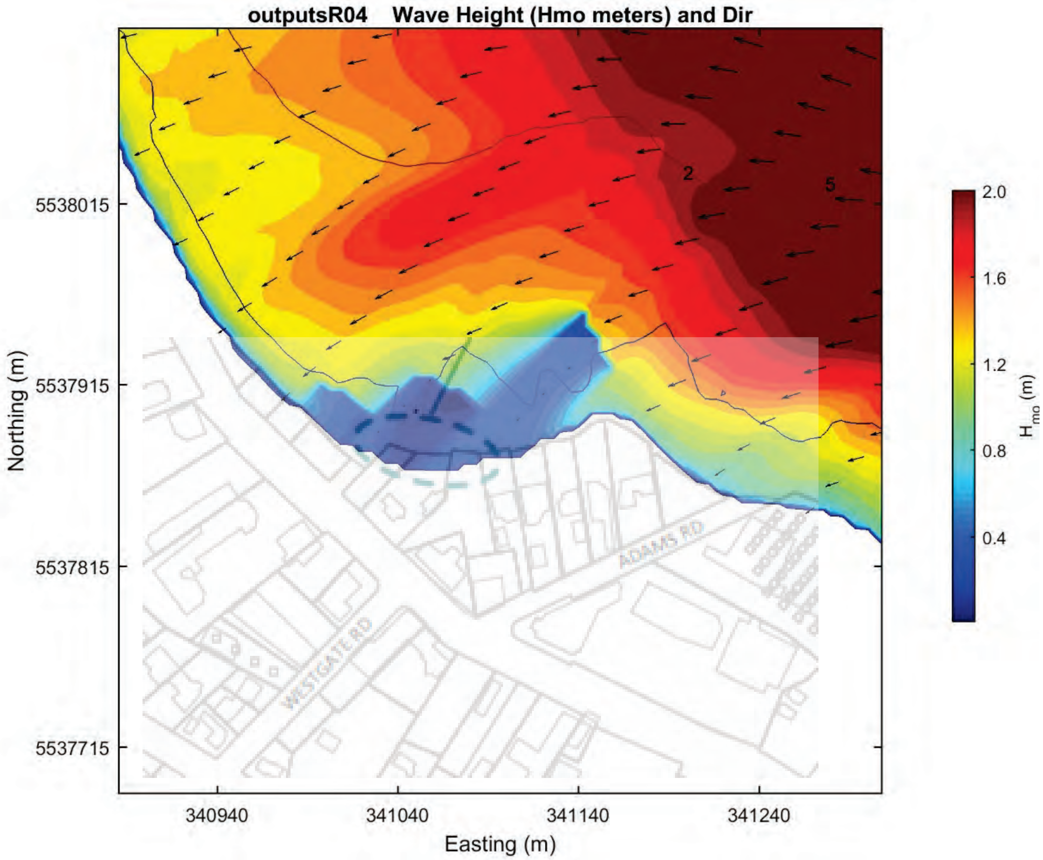


Figure 4-24: SWAN model run for a conceptual design of detached breakwaters to protect Frank James Park.

The SWAN modeling work found that it would be most effective to interrupt the waves along a shallow bench to the east of Frank James Park, with one or two additional low crested detached (or offshore) breakwaters closer to the park. Detailed 3D SWASH modeling was done to further examine the suitability of the concept and evaluate performance of the breakwaters.

SWASH modeling shows both a reduction in wave heights (**Figure 4-25**) and a reduction in shear stress at the seabed in the lee of the breakwaters. This will promote sediment accretion at Frank James Park and allow for raising of the beach with a beach nourishment of mixed sands and gravels.

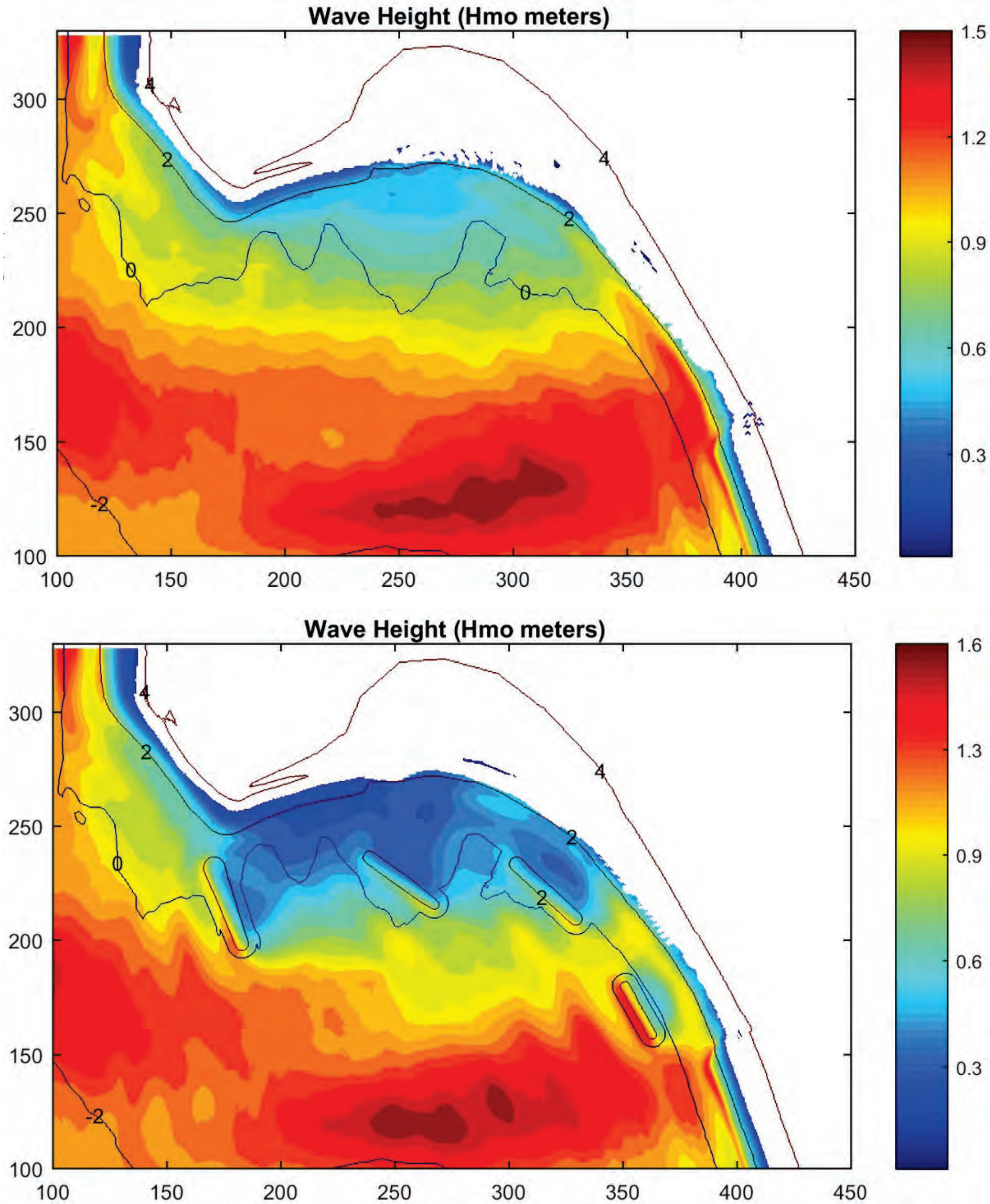


Figure 4-25: Maps of significant wave height for the present day shoreline (above) and with the detached breakwaters (below). There is a marked reduction in wave heights at the park shoreline. (Note that South is up in these views, as the model orientation was rotated to best align the grid with incoming waves.)

Based upon the modelling work, a conceptual layout for a mitigation using offshore breakwaters is presented in **Figure 4-26**. The concept includes four rock structures and one beach nourishment area.

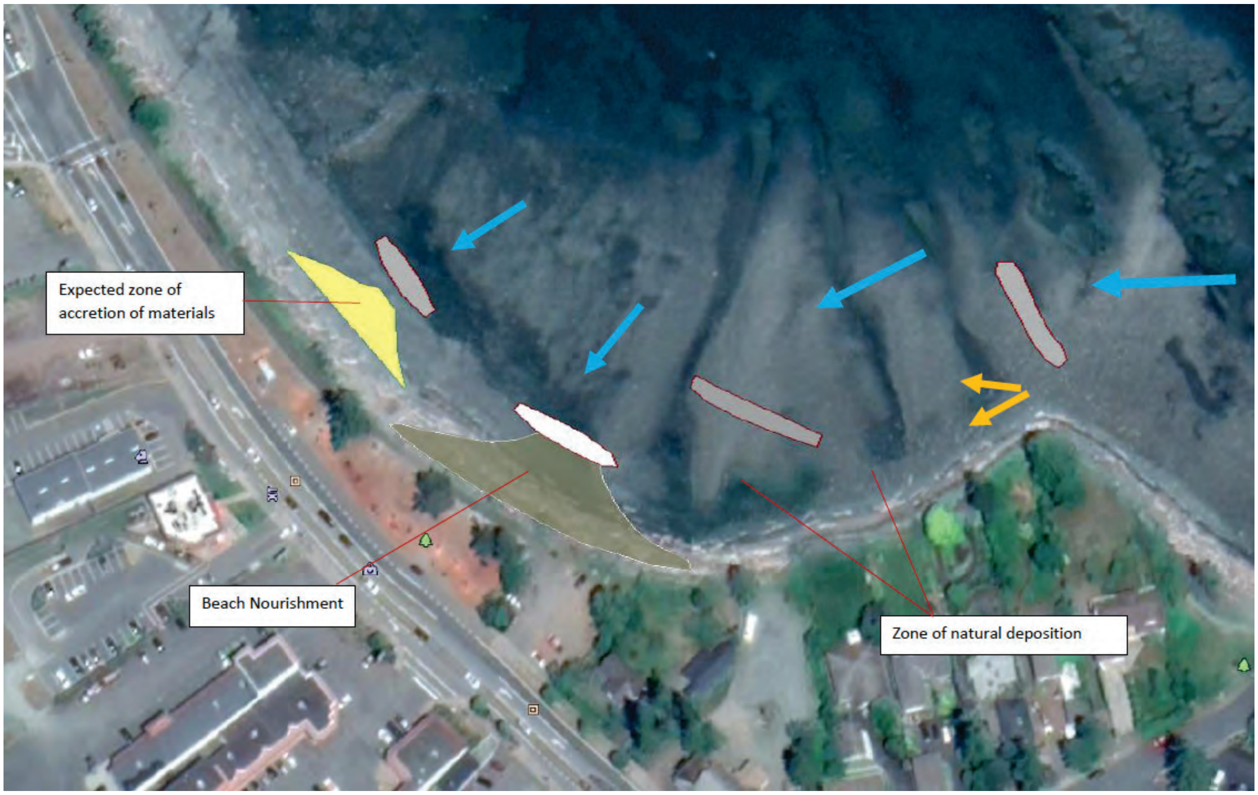


Figure 4-26: Plan of potential pocket beach and offshore headland mitigation for Frank James Park, as modeled with SWASH. Blue arrows show principal wave directions and orange arrows show area of expected wave diffraction. Four rock armour structures are shown. Grey structures have crest elevations of 3.5 m while the white shaded structure has a crest elevation of 2.8 m. A recommended area of a beach nourishment is shown in tan, while the yellow zone indicates an area where natural accretion is expected.

4.4.3 Window / Inland Beach Concept

As presented in Option C of Primer 3 (Lanarc and NHC, 2019a), one concept to create a more natural beach area is to create a break in the existing steep rock revetment, and build a mild slope embayment beach inland of the existing revetment alignment. The purpose is to improve connectivity between intertidal areas and upper park space, and increase the area of intertidal 'soft' beach in this region of Campbell River. This concept also allows a natural surface drainage pathway to the ocean for rainfall and localized flooding that might occur. This concept could be developed anywhere along the Willow Point shoreline where there is existing revetment and where the City could acquire three adjacent properties (or expand the existing Jaycee Park).

The shoreline at Willow Point is exposed to large waves and sheltering is needed to prevent erosion of fine sediments (such as sands and gravels). Wave runup is notably reduced in the pocket embayment such that the beach crest elevation in a packet beach is lower than the rock revetment, and allows for lower elevations to be kept within the inland park areas without risk of coastal flooding.

SWASH modeling was done to evaluate performance and check for resonance of waves in a proposed embayment (**Figure 4-27**). The modeling showed that a 40 m long beach and a 5 m opening in the revetment performed satisfactorily to attenuate wave effects in the pocket beach. A small opening also serves to reduce the potential for large woody debris to enter into the embayment, but could trap debris that does enter. Additionally, high water velocities at the entrance during storm conditions could be hazardous to the public if they were to enter the water during a major storm event. The precise alignment of the gap in the existing revetment will need further analysis to develop a detailed design sufficient for construction, but the existing concept study validates the overall approach to provide suitable conditions for a pocket beach.

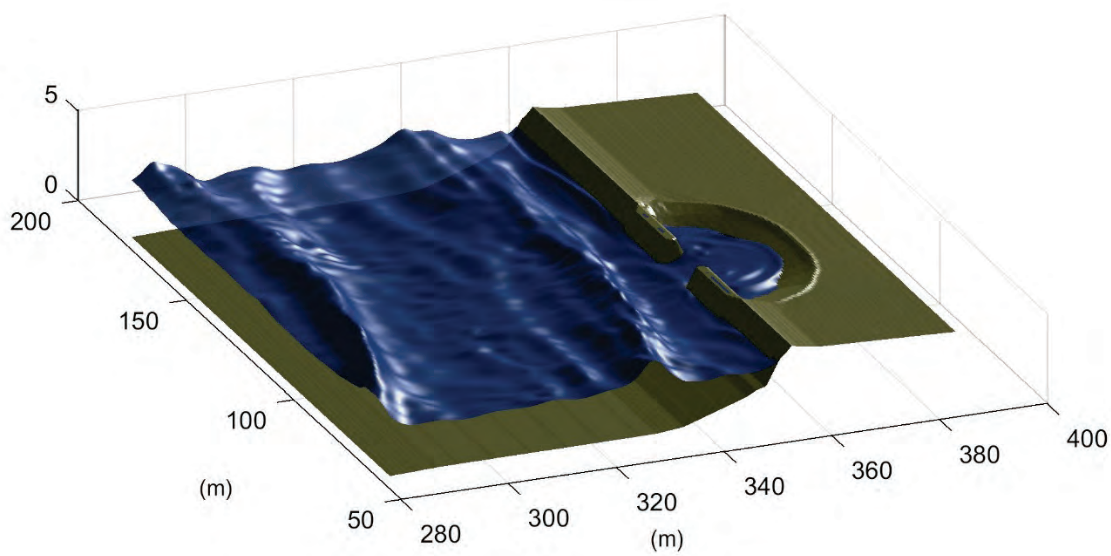


Figure 4-27: SWASH model run for a pocket beach concept for a location such as at Jaycee Park. Model included increasing the elevations of the shoreline revetment from 4 to 4.5 m.

4.4.4 Private Property – Naturalized Shoreline Option

A series of options were considered within the project that would allow private properties to adapt to new FCL levels (Lanarc and NHC, 2019a). One question explored was the potential viability of a non-hard armoured shoreline in conjunction with a raised property that could potentially fit within a typical lot in the area. The concept for the land was developed by Lanarc, and modelling to explore the wave effects for difference levels of SLR was undertaken by NHC. **Figure 4-28** shows the layout concept as proposed while **Figure 4-29** shows a snap-shot of wave runup during a model test. Maximum runup elevations reached the edge of a house that is located in a typical offset from the shoreline, but the wave velocity and volume was attenuated to levels that could be accommodated through other design features.

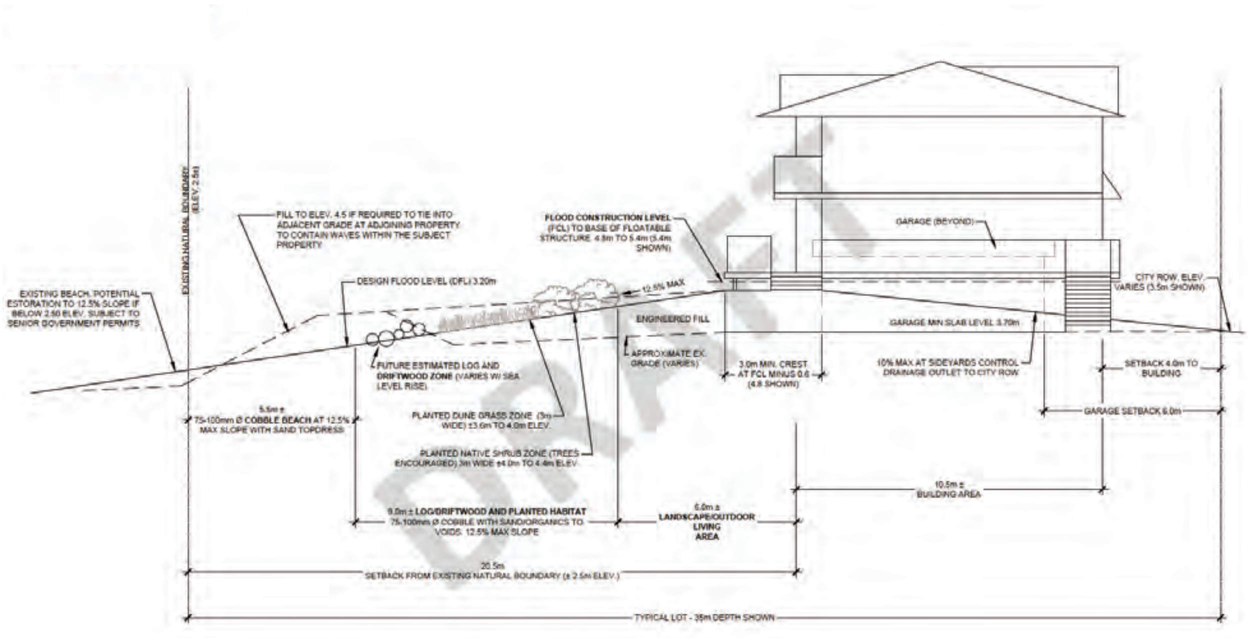


Figure 4-28: Lanarc Consultants concept design for an alternative shoreline to rock armour for a private lot. The re-graded beach includes a cobble beach face on the lower beach, with a vegetated slope above the natural boundary. Some woody debris is expected.

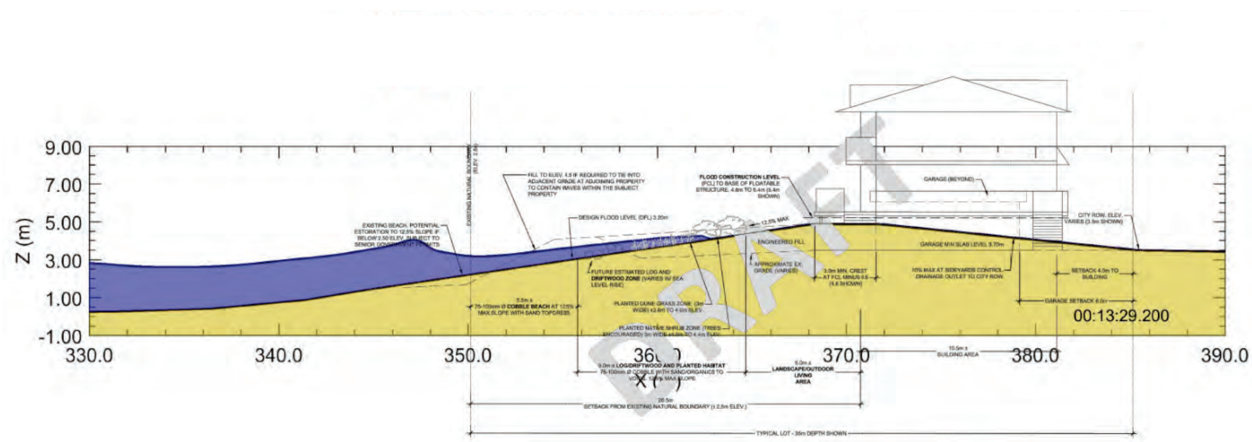


Figure 4-29: SWASH model run with 1.0 m of SLR for a naturalized beach face on a raised property. Wave runup under a 200 year design storm reaches the edge of the house, but this is notably reduced with additional vegetation on the slope. As design storms typically occur in winter months any vegetation will need to be perennial (i.e. not die off during winter) to provide this benefit.

5 COINCIDENT RAINFALL AND COASTAL FLOOD HAZARD

5.1 Correlation Analysis

NHC undertook analysis to examine the potential correlation between storm events that could cause coastal flooding (i.e. large wind-wave events) with rainfall timing. Wave height data has been taken from the Sentry Shoal buoy. As the rainfall data is only available as 'daily' accumulations from the Campbell River airport for long-term data, the comparison requires that the hourly wave data be converted to a daily record of maximum wave height.

The following assumptions were made:

- Rainfall is a reasonable proxy for high streamflows. This assumption is made because there is no flow data for small streams in the region. It should be reasonable because the analysis is focused on small, low elevation streams that are less likely to be affected by snow.

- A daily timestep is suitable for a first pass analysis. An event scale study with hourly data would improve the statistical correlation analysis but hourly streamflow or rainfall data for an extended period of time is not available in the small streams of interest (Willow Creek, Simms Creek, and Nunns Creek). The hourly data from the Campbell River gauge is not useful here as this river discharge is controlled.

- The data available is a representative sample of conditions.

As noted, the wave data time series was converted to a daily maximum, and two time series aligned. The following scatter plots show the comparison of data for each month of the year.

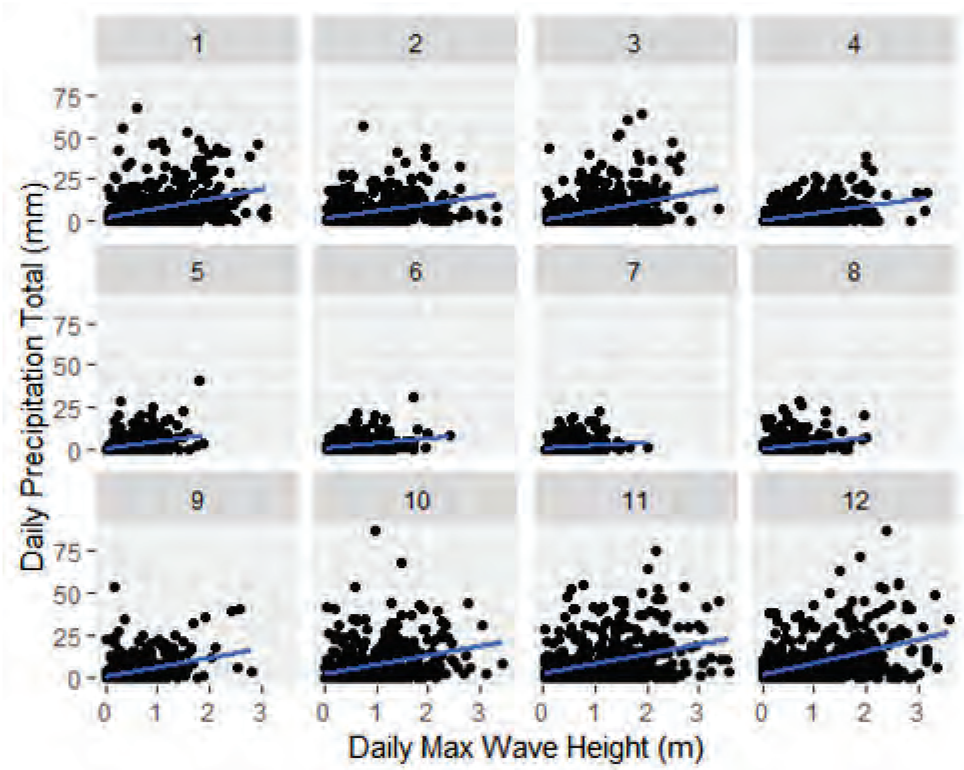


Figure 5-1: Scatter plots of precipitation and wave height for each month

This is not a clear direct correlation, so an analysis was undertaken using the 'texmex' package in R[14] to investigate the conditional probability (i.e. the likelihood of rainfall coinciding with high waves). The package was used to fit a conditional multivariate extreme value model (Heffernan and Tawn, 2004). Waves above 1.5 m in height and daily total precipitation above 10 mm per day were used in the extreme value analysis. The fitted model was then used to determine the conditional probabilities summarized in **Table 5-1**. For comparison, quantiles of the indepent polulations are given in **Table 5-2**.

In probability theory two events are independent if the occurrence of one event does not affect the probability of occurrence of the other event (i.e. the independence of the occurrence of high waves in the northern Strait of Georgia and the occurrence of rainfall). The conditional mean of a random variable is its expected value or the average expected value of the variable given a number of events.

[14] R is a statistical analysis software package

Table 5-1: Conditional Mean and Quantiles

	Wave Height (m) | Wave Height >Q98.5	PP (mm) | Wave Height >Q98.5
Mean	2.84	17.7
50%	2.78	12.9
75%	2.99	26.2
90%	3.22	43.8
95%	3.36	53.4
99%	3.62	74.9

* Conditioned on wave height being above its 98.5th percentile.

Table 5-2: Independant Mean and Quantiles

	Wave Height (m)	Precipitation (mm)
Mean	0.71	4.2
50%	0.52	0.4
75%	0.96	4.4
90%	1.59	13.3
95%	1.97	21.0
99%	2.66	40.4

* Conditioned on wave height being above its 98.5th percentile.

Figure 5-2 is a plot showing the data family used in the conditional multivariate model. The yellow curve is the line joining the equal quantiles of the two variables. The coloured dots are simulated data above the 2.5 m wave height (98.5th percentile) and the grey dots are actual data.

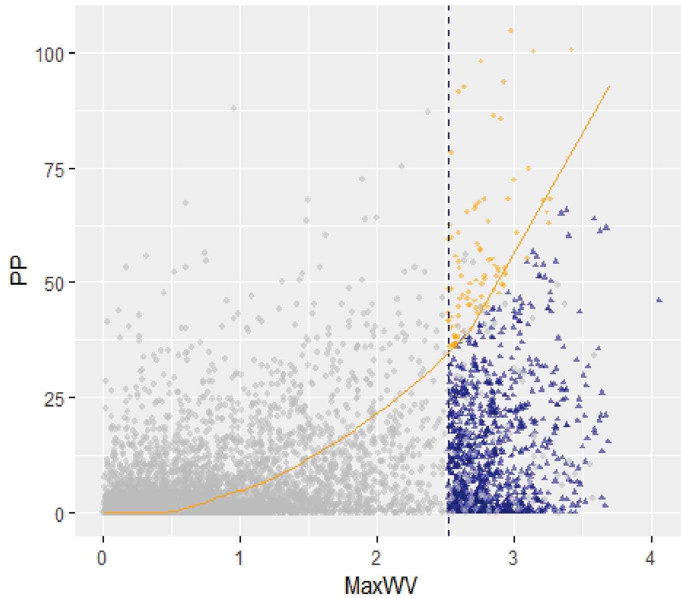


Figure 5-2: Plot showing line of equal quantiles of the two variables (max daily wave height in metres and precipitation in millimetres)

The key findings of the analysis are:

- When the wave height is above 2.5 m the <u>mean</u> daily expected rainfall is 18 mm.
- At the 90th percentile, precipitation is ~ 44 mm, thus there is a not insignificant (10%) chance of intense rainfall that could generate upland flooding or high runoff coincident with a period of high waves.
- All of the conditional quantiles for rainfall are far higher than the independent quantiles, indicating that there is a correlation between rainfall events and storm wave events.

The last finding is logical, in that major storms which predominantly occur in the fall and winter and generate storm waves in the Strait of Georgia also bring rainfall with them. As such, it is important that storm drainage systems be functional during periods when coastal storms occur (high water, surge, and waves) to avoid excessive flooding of vulnerable shoreline properties. Select areas with the four sites are exposed to both creek/river flooding (Willow Creek estuary, Simms Creek, Nunns Creek and the Campbell River Estuary) and coastal flooding and may require careful planning to mitigate both types of flood hazards going forward[15].

[15] Additional analysis of the potential hazard for river flooding and future SLR for the downtown area was undertaken by NHC for the City of Prince Rupert (ref: NHC (2018), City of Campbell River Sea Level Rise Study, Phase 2 Estuary Assessment),

Unfortunately, without detailed hourly data for precipitation and streamflow it is not possible to further examine correlations between the timing of rainfall and subsequent streamflow peaks and that of waves, tides, and storm surge. It is recommended that the City consider in the future obtaining hourly rainfall or streamflow data to allow for further analysis of this correlation.

Specifically, the data gathering is recommended to include:

- Hourly rainfall monitoring at a meteorological station within the Campbell River area. This could be set up at a public works yard, on the roof of City Hall, or at any suitable location where power could be provided for a data logger, inspection and maintenance would be easily done, and the rainfall gauge would be free from sheltering by trees or nearby buildings.

- Additionally, it would be beneficial to also directly measure streamflow and waterlevels within Willow Creek. A water level and streamflow gauge would be best installed in the lower reach of the Creek, but some distance upstream of the Highway 19A crossing. A suitable location would need to consider a variety of factors (steam bed conditions, vegetation, property access) that would require additional considerations beyond the scope of this report.

5.2 Willow Creek Flooding

Flood levels and extents in and around Willow Creek estuary with consideration of tides, storm surge, and sea level rise were evaluated using TELEMAC-2D model. TELEMAC-2D is a two-dimensional (2D) model that solves the Saint-Venant equations using the finite-element method and can perform transient simulations where conditions are changing over time. The model simulates free-surface flows in two dimensions of horizontal space. The equations and model descriptions are provided in detail in (Hervouet, 2007).

5.2.1 Hydrology

There is no ongoing record of flows in Willow Creek so a regional analysis was first conducted using nearby stations. The peak 20-year and 200-yr discharges are about 57 m^3/s and 73 m^3/s, respectively. There is a high level of uncertainty associated with these values due to the lack of streamflow data and hourly rainfall data. There exists one measurement of the 20-year flood value in Willow Creek (as referenced in **Figure 5-3**) to which the other stream data was scaled.

Synthetic hydrographs are shown below that were developed for the site. The shape of the hydrographs are based upon the Dove Creek data (BC Ministry of Environment) as this watershed is in the same geographical area (**Figure 5-3** and **Figure 5-4**).

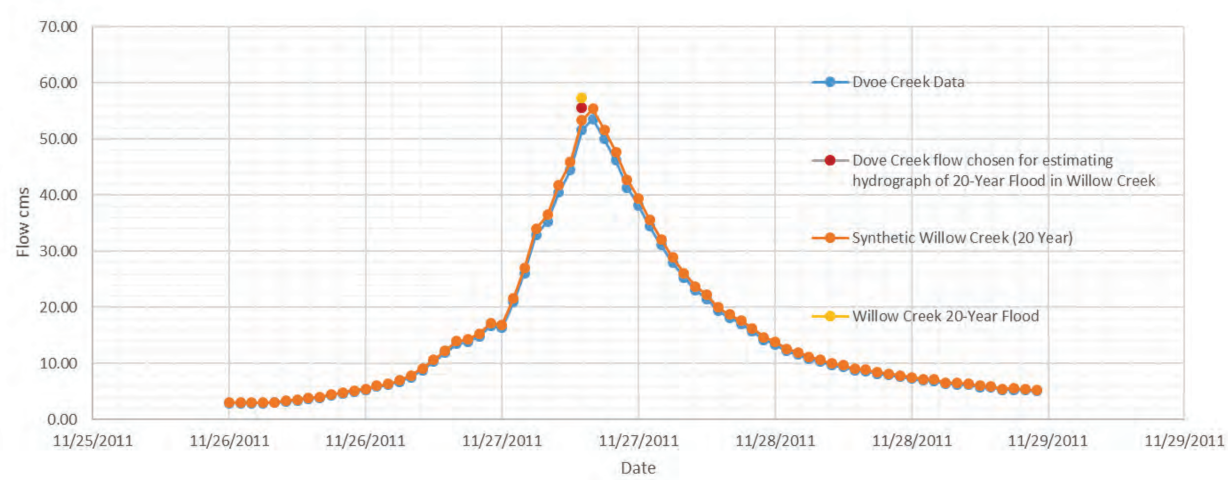


Figure 5-3: 20-year peak discharge synthetic hydrograph

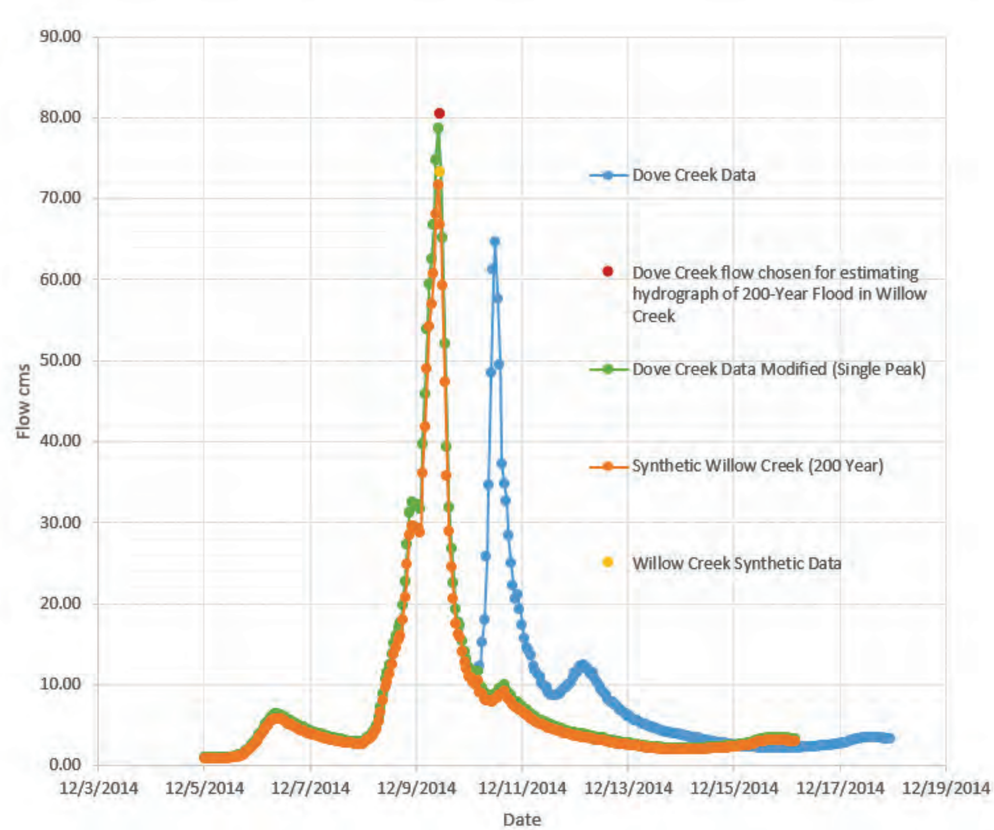


Figure 5-4: 200-year peak discharge synthetic hydrograph

5.2.2 Flood Simulations

Four flood simulations were conducted using TELEMAC2D with boundary conditions as noted in **Table 5-3**. Note that WSE is an acronym for Water Surface Elevation and refers to the combinations of tides, storm surge, and sea level rise on the ocean at the mouth of Willow Creek.

Table 5-3: Willow Creek estuary flood simulations

Run	Time frame	Surge AEP	Peak WSE (m GD)	Q AEP	Peak discharge (m^3/s)
1	2017	1-in-10	2.52	1-in-20	57
2	2017	1-in-1	2.29	1-in-200	73
3	2100	1-in-10	3.01	1-in-20	57
4	2100	1-in-1	2.78	1-in-200	73

The elevations in the model are based on a digital elevation model (DEM) based on LiDAR data provided by the City. Culvert function was implemented to represent the opening below the highway. Some areas of the creek are heavily forested which affects the accuracy of ground elevation assessment using LiDAR data. Without ground truthing, it is possible that the actual ground elevation may not be properly represented in the model.

5.2.3 Results

Maximum inundation maps for the four simulations are shown in **Figure 5-5** to **Figure 5-8**. The modeling shows that:

- In general, the affected areas do not vary much between simulations;
- The key area of concern is just upstream of the highway crossing;
- The overtopping flow on the right bank (looking downstream) mostly flows across the highway and drains into Discovery Passage through the Ken Forde boat ramp; and
- The overtopping waters on the north bank flow through the commercial lots, to the highway and to the waterfront properties.

5.2.4 Discussion

The model results show that some flooding will occur near the highway during a rainfall event with an annual exceedance probability (AEP) of 5% (1-in-20) with a 1/10 AEP storm surge (Scenario 01). Discussion with city staff suggested that flooding has occurred in Regions 1 and 2 but not to the extent modelled. Two possible reasons can lead to this:

- Uncertainty with the peak flow estimate; and
- Uncertainty with the DEM due to vegetation and standing water.

To improve the confidence in the flood hazard assessment, the following are recommended:

- Install a streamflow gauge in Willow Creek; and
- Conduct field survey to verify LiDAR elevation specifically in the heavily forested region and within the streambed.

Nonetheless, the modeling analysis highlights that there are three areas prone to flooding from combinations of high Willow Creek discharge and high tides with surge, and that flood depths will increase in the future due to SLR.

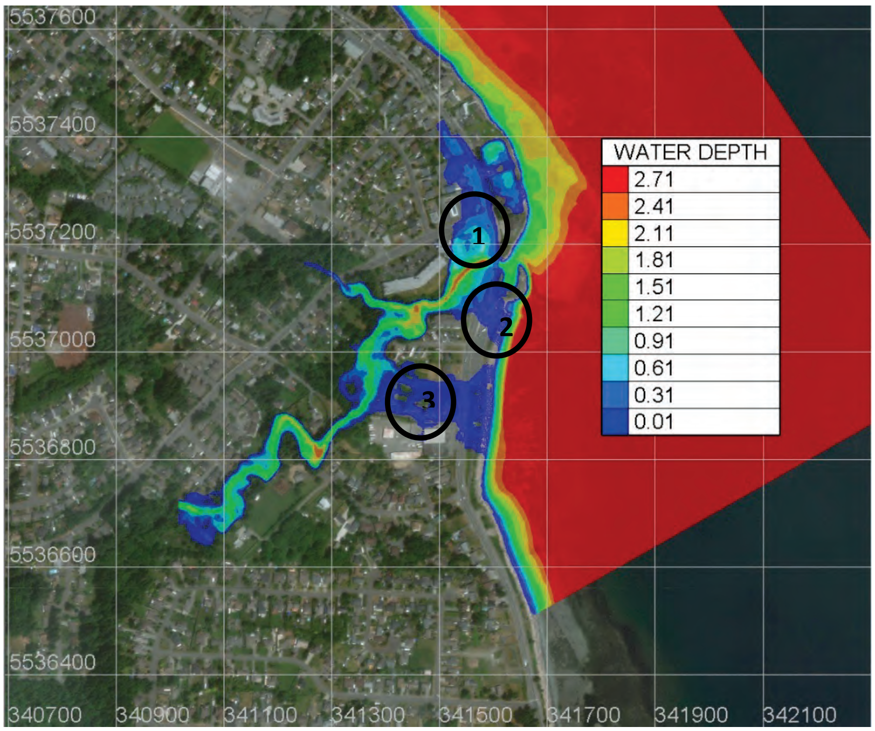


Figure 5-5: Scenario 01 - Maximum flood extent

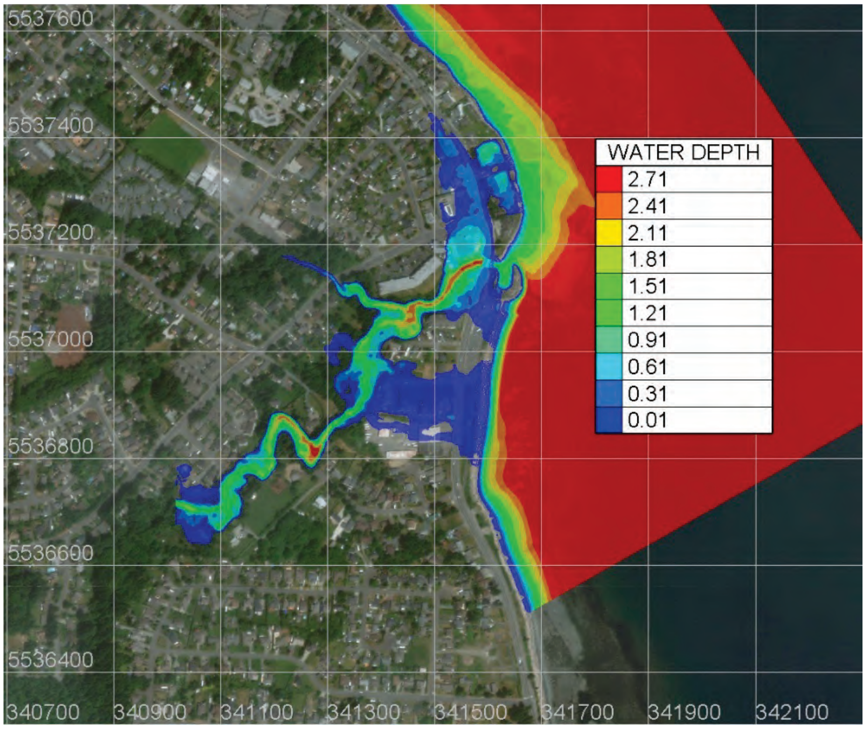


Figure 5-6: Scenario 02 - Maximum flood extent

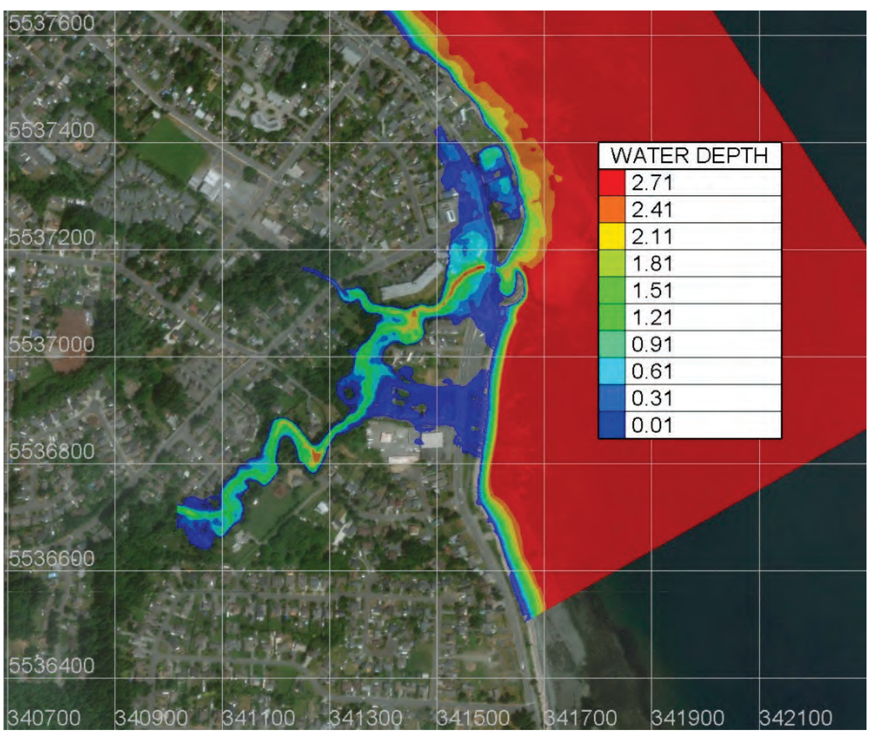


Figure 5-7: Scenario 03 - Maximum flood extent

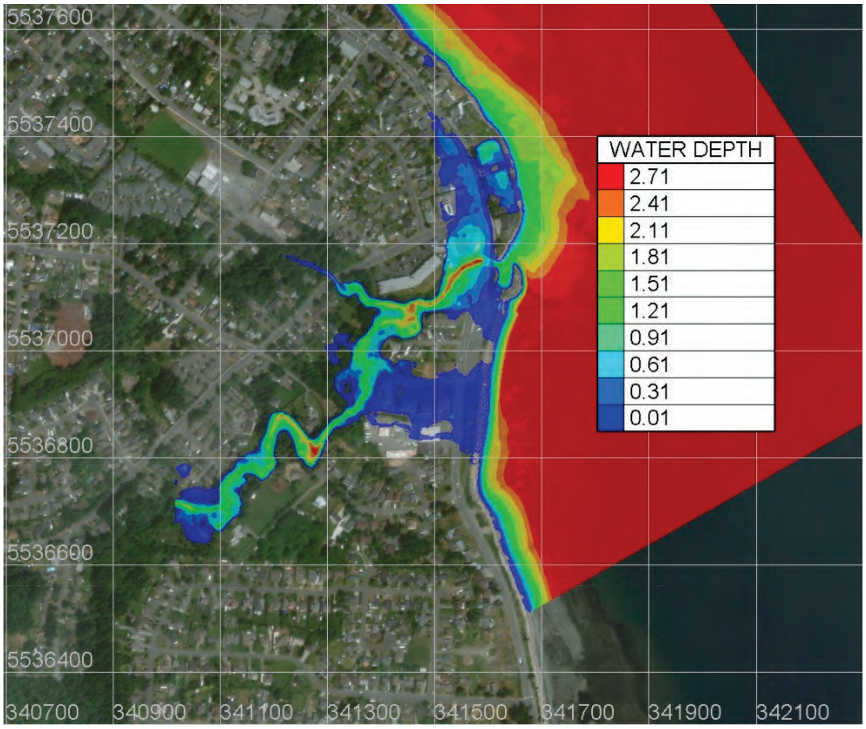


Figure 5-8: Scenario 04 - Maximum flood extent

5.3 Downtown Flooding

A detailed study of the potential effects of increased SLR on the hazard from river flooding in the Campbell River estuary and downtown areas has previously been completed (NHC, 2018b). The findings of this study will not be repeated here, but it is important to comment that the drainage from the downtown area into the ocean is important for limiting flood depths within the downtown area. Thus, SLR adaptations that involve raising the areas adjacent to the shoreline creates a potential 'bathtub' effect for the inland downtown areas, and would require careful consideration.

To examine the potential effect of not allowing drainage at the shoreline, the NHC Campbell River estuary model was run for a scenario in which the shoreline was simulated as a 'wall' that did not drain into the ocean (Figure 5-9) and also for a condition in which drainage could freely occur into the ocean (Figure 5-10). The results indicate that the river flood hazard for the downtown area is notably increased should the shoreline elevations be raised without allowances for drainage from upland sources. Close examination of the results shows increased flood depths near the Discovery Harbour Centre mall complex, as well as a larger and deeper flood extent in the downtown shopping areas and near Ostler Park.

For this reason, it is recommended that a combination of passive storm drain systems (with check valves to prevent downtown flooding from reverse flows during extreme high coastal water levels) in addition to an active pump station be incorporated into any plans to raise the land elevations along the coastal shoreline. Figure 5-11 shows a section view of a concept for a pump station with an underground flood water storage area that could be developed and implemented in a downtown location.

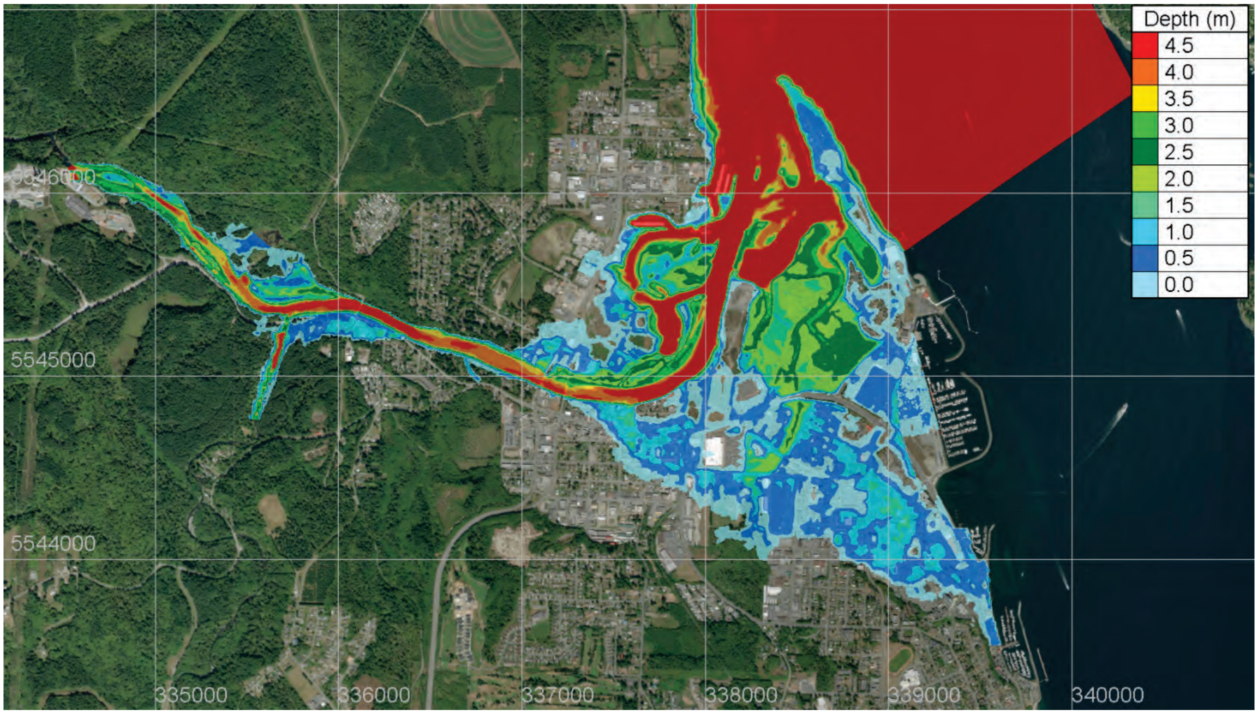


Figure 5-9: Maximum flood extent and depth – 1 m of SLR (year 2100), 1/20 AEP river discharge and 1/10 AEP storm surge without drainage into the ocean (NHC, 2018b)

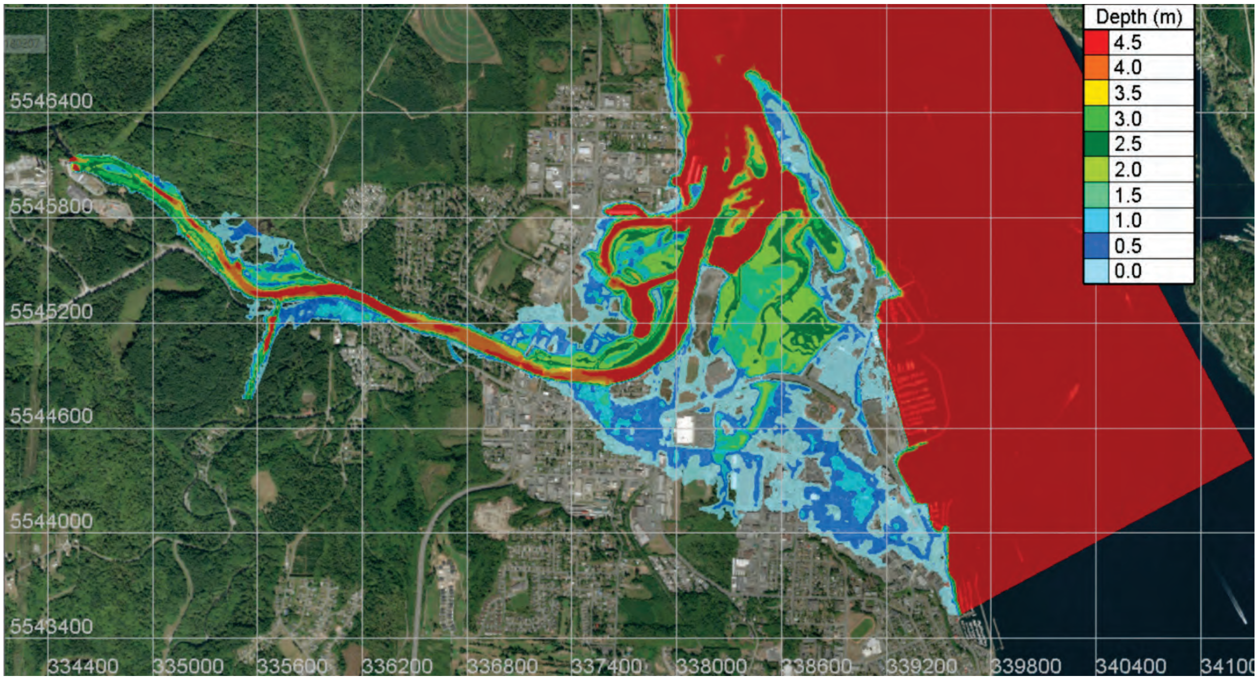


Figure 5-10: Maximum flood extent and depth – 1 m of SLR, 1/20 AEP river discharge and 1/10 AEP storm surge with free drainage of flood water into the ocean at the shoreline (NHC, 2018b)

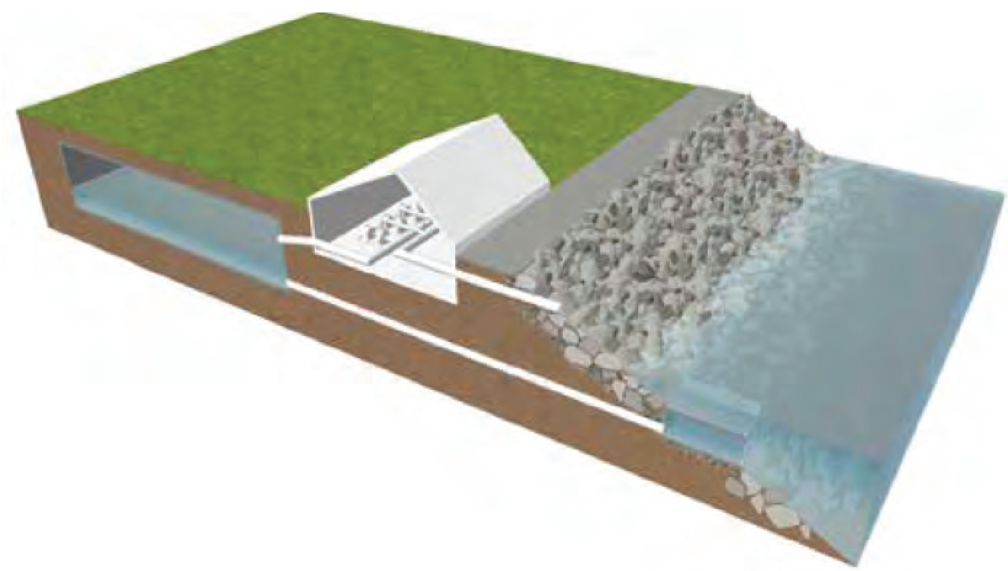


Figure 5-11: Concept drawing for a pump station that utilizes an undergound storage/capture area for flood waters to drain the downtown area (Lanarc and NHC, 2018).

In order to avoid creating a 'bathtub' effect through the development of coastal defense structures in the downtown areas the following are recommended:

- coastal defences should be raised sufficiently to prevent high volumes of water from wave overtopping to cause flooding in the downtown area;
- storm drainage capacity should be checked and, if needed, increased to allow sufficient capacity to fully drain any downtown areas of precipitation accumulations and localized flooding during the mid and low tide time windows; and
- a means of pumping flood waters from the downtown area should be provided that if not equal, can meet a proportion of the expected inflow rates of river flood waters sufficient to limit overall flooding depths for the two or three hour period around a peak tide.

The overall flood hazard for the downtown area could decrease in the future should adaptations for sea level rise be implemented both at the shoreline and also along the estuary and river banks as noted in the estuary assessment report (NHC, 2018b).

6 CONCLUSIONS AND RECOMMENDATIONS

Campbell River is fortunate that vertical land movements are, at present, minimizing the effects of global sea level rise (GSLR). Applying the provincial government's recommendation for 1 m of GSLR in year 2100, the vertical land movement over this time period serves to limit local SLR at Campbell River approximately 0.6 m. It is important to note that this favorable condition does not imply that Campbell River can avoid adaptation as the rate of SLR is expected to increase in the future.

Large sections of the Campbell River shoreline are already hardened (or armoured), and have relatively low crest elevations allowing wave runup and spray that are causing nuisance flooding during winter storm events. For example, extreme storms have caused notable flooding and deposition of debris in Ostler Park.

Detailed modeling was undertaken to establish coastal Flood Construction Levels (FCL) along sections of the Campbell River waterfront. As per standard methodology for such analysis, the FCL levels are somewhat conservative and detailed wave modeling indicated that for many existing shorelines the wave runup will not reach the theoretical limits. However, this is due to the relatively low crest elevations at many locations in Campbell River that result in large volumes of water overtopping the crests of the shorelines under future scenarios with sea level rise. As such, without raising the crest elevations at the existing shorelines there will be increased flooding that will put a focus on drainage of properties, and the relative elevation of structures to their immediately surrounding ground.

This technical document summarizes analysis of the four component areas, and the coastal engineering analysis undertaken to examine the feasibility of select mitigation options available to address sea level rise. This document should be read together with its companion document the Technical Background Report (Lanarc and NHC, 2019b) which further explores the potential mitigations for the four component areas.

The technical findings show that mitigating local sea level rise of less than 1.0 m is feasible and that there are various options to do so depending on the nature of the shoreline at Campbell River. However, it is important to note that the most recent scientific reports such as the Canada Climate Change Report (Bush and Lemmen, 2019) mention that there is the possibility of faster rates of sea level rise than presently allowed for in the provincial guidance and that 1.4 m of sea level rise or higher is considered possible before year 2100. As such, it is recommended that long term planning not only focus on meeting the provincial guidelines for 1.0 m of sea level rise, but also consider that higher rates of sea level rise remain a possibility that can not be ruled out at this time.

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*.

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.

Bush, E., and Lemmen, D. S. (2019). *Canada's Changing Climate Report, Government of Canada, ON, 444p*.

Davidson-Arnott, R. (2010). *An introduction to Coastal Processes and Geomorphology*. Cambridge University Press. New York.

EurOtop (2016). *Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application.* Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B. [online] Available from: www.overtopping-manual.com.

Han, G., Zhai, B., and Thomson, R. (2016). Twenty-first century mean sea level rise scenarios for Canada. *Canadian technical report of hydrography and ocean sciences*, (313), 1488–5417.

Heffernan, J., and Tawn, J. (2004). A conditional approach for multivariate extreme values. *Journal of the Royal Statistical Society*, 66(3).

Hervouet, J.-M. (2007). *Hydrodynamics of Free Surface Flows: Modelling with the Finite Element Method*.

Hsu, Silvester, R., and Xia, Y. M. (1989). Generalities on static equilibrium bays. *Coastal Engineering*, 12(4), 353–369.

Lanarc, and NHC (2018). *City of Campbell River Sea Level Rise Primer 2, Sea Level Rise Adaptation Best Practices*. Prepared by Lanarc 2015 Consultants Ltd. and Northwest Hydraulic Consultants Ltd. for the City of Campbell River.

Lanarc, and NHC (2019a). *City of Campbell River Sea Level Rise Primer 3, Local Adpatation Options and Evaluation Process*. Prepared by Lanarc 2015 Consultants Ltd. and Northwest Hydraulic Consultants Ltd. for the City of Campbell River.

Lanarc, and NHC (2019b). *City of Campbell River Sea Level Rise Study, Technical Background Report*. Prepared by Lanarc 2015 Consultants Ltd. and Northwest Hydraulic Consultants Ltd. for the City of Campbell River.

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.

NHC (2018a). *City of Campbell River Sea Level Rise Study Phase 1 - Downtown Waterfront Site*. Prepared by Northwest Hydraulic Consultants Ltd. for the City of Campbell River.

NHC (2018b). *City of Campbell River Sea Level Rise Study Phase 2 - Estuary Assessment*. Prepared by Northwest Hydraulic Consultants Ltd. for the City of Campbell River.

NHC (2018c). *City of Campbell River Sea Level Rise Study Phase 3 - Additional FCL Assessment*. Prepared by Northwest Hydraulic Consultants Ltd. for the City of Campbell River.

Ranasinghe, R., Watson, P., Lord, D., Hanslow, D., and Cowell, P. (2007). Sea Level, coastal recession and the Bruun Rule. *Proceedings of Coasts and Ports, Melbourne, Australia Conference paper*.

Silvester, R., and Hsu, J. (1993). *Coastal stabilization: innovative concepts*. PTR Prentice Hall, 1993.

Thomson, R. E. (1981). Oceanography of the Britsh Columbia Coast. *Canadian special publication of Fisheries and Aquatic Sciences*, 56, 25–41.

Zijlema, M., Stelling, G., and Smit, P. (2011). SWASH: An operationsl public domain code for simulating wave fields and rapidly varied flows in coastal waters. *Coastal Engeering*, 58, 992–1012.

8 CLOSURE

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

APPENDIX A

GEOTECHNICAL ASSESSMENT
PAINTER-BARCLAY ESCARPMENT – CAMPBELL RIVER, BC

RYZUK GEOTECHNICAL
Engineering & Materials Testing
28 Crease Avenue, Victoria, BC, V8Z 1S3 Tel: 250-475-3131 Fax: 250-475-3611 www.ryzuk.com

June 17, 2019
File No: 8804-1

Northwest Hydraulic Consultants
30 Gostick Place
North Vancouver, BC
V7M 3G3

Attn: Grant Lamont, P.Eng., Principal, Sr. Coastal Engineer
(by email: GLamont@nhcweb.com)

Re: Geotechnical Assessment
Painter-Barclay Escarpment – Campbell River, BC

As requested, and in accordance with our proposal, we attended site on June 8 and 9, 2018, to complete a geotechnical assessment of the Painter-Barclay Escarpment. Our observations and recommendations are summarized below.

SITE DESCRIPTION

The Painter-Barclay Escarpment is in North Campbell River and east of the North Island Highway. The location of our assessment was the approximately 2.3 km long shoreline area bordering the Discovery Passage channel roughly from the northern Orange Point Road cul-de-sac to Wildwood Lane.

In general, the Painter-Barclay Escarpment is a foreshore area occupied by single family residences throughout with a few larger resort buildings (Dolphins Resort and Painter's Lodge). The foreshore slope is almost entirely covered in thick mixed vegetation and periodic coniferous/deciduous trees. We noted numerous locations with exposed soils at the base of the foreshore, and erosional scarps near the mid height and crest of the slope.

INVESTIGATION OBSERVATIONS

Our investigation consisted of reviewing available geological and topographical mapping of the area and attending site to visually inspect the foreshore area. We photographed notable features on the slope which can be found in the attached Appendix A: Photo Log.

Foreshore Slope

The foreshore slope generally varies in height and geometry. We estimate the height to generally vary between 8 to 12 metres by vertical measure, transitioning to about 2 metres near Painter's Lodge. The slope of the foreshore is generally quite steep, with the majority ranging from about 3/4H:1V (horizontal : vertical) to 1H:1V. Sections of the foreshore that have experienced more significant erosion are generally steeper at about 1/4H:1V with some vertical or overhanging portions.

The foreshore slope has numerous erosional scarps that expose native soils. Based on our observations, the Painter-Barclay Escarpment appears to mostly be comprised of very dense grey/brown silty sandy glacial till with some gravel. We expect that closer to the mouth of Campbell River, near Painter's Lodge, the foreshore soils will transition to fluvial depositions of river channel soils such as silts, sands, and/or gravels.

We observed various types of foreshore slope erosion along the entire Painter-Barclay Escarpment. Predominantly the toe of the foreshore slope has been eroded by wave action at the base of the slope causing undercutting, oversteepening, and subsequent material loss. Photos 1 through 4 below show some of the more accessible areas of toe erosion not obscured by vegetation.

Photos 1, 2, 3, 4 – This set of photos shows general examples of toe erosion of the foreshore slope.

We also observed more substantial shallow translational or rotational failures which were around mid-height and near the crest of the foreshore slope. Most notable was the foreshore of 4565 Discovery Drive which appears to have had a recent failure of 5+ cubic metres of material as shown in photo 5 below. Most of the other failures are hidden from view as common horsetail vegetation quickly grows to obscure the exposed soils as shown in photo 6.


Photo 5 – Shows foreshore slope erosion at 4565 Discovery Drive.
Photo 6 – Shows foreshore slope erosion obscured by common horsetail vegetation.

Foreshore Revetment Structures

Some properties have foreshore revetment structures (seawalls) of varying composition and effectiveness to prevent erosion of the foreshore slope. We observed seawalls comprised of stacked boulder walls, bulk concrete pours, cast-in-place retaining walls, and sandbags. Some of the seawalls appear to have been well constructed and are adequately protecting the foreshore from erosion while some have completely failed and are providing minimal, if any, protection from erosion.

Property/City Drainage

We noted that a number of the residential properties discharge stormwater down the foreshore slope. We also observed that the City, in at least one location directs stormwater from Orange Point Road to the crest of the foreshore slope.

Most of the residential drainage pipes that were observed were poorly installed, with the exposed pipes leaking, discharging at the mid height or the toe of the foreshore slope, or a combination of these deficiencies.




Photo 7 – Two drainage pipes discharging near the toe of the foreshore slope.
Photo 8 – Leaking residential drainage pipe with signs of slope erosion.
Photo 9 – Residential drainage pipe leaking at roughly mid height of foreshore slope.
Photo 10 – Orange Point Road curb directs storm water to the crest of the foreshore slope.

GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

Based on our observations described above, we consider that the Painter-Barclay Escarpment foreshore slope will, for the most part, continue to exhibit erosion. The observed erosion is primarily a result of waves impacting the toe of the foreshore slope causing loss/transportation of the glacial till and translational and/or rotational failures. Erosion can eventually lead to the periodic overturning of trees/vegetation on the slope further accelerating slope erosion and regression of the slope. These erosional effects appear to be less severe near and to the south of McDonald Road. It is possible that the adjacent bathymetry, beach geometry, or other mechanisms are dissipating wave energy more effectively in this area.

While not the primary cause of erosion, the residential and City stormwater drainage improperly discharging on/near the foreshore slope will continue to negatively affect the slope by causing erosional channels. In addition, the sporadically installed seawalls can redirect wave energy causing flanking of seawalls and possibly worsening the observed erosion on neighbouring properties.

We expect that the foreshore slope will continue to exhibit erosion and regression. The rate at which slope regression progresses inland is challenging to determine as such will vary substantially from year to year based on the severity of storms and precipitation. We expect that based on the commonly accepted suggestion that more significant storms may be occurring more frequently associated with changing climatic conditions, as well as potential sea level rise that may occur over the next 50 to 100 years, the foreshore slope erosion will likely accelerate.

The foreshore slope erosion and associated regression will eventually impact the foundations of the nearby single-family residences and underground utilities in the area. Some of the single-family residences along Discovery Drive are within 3.5 to 5 metres horizontally from the crest of the foreshore slope. Based on utility locations from the City of Campbell River GIS mapping, storm and sanitary utilities are present within the foreshore slope area, and in some cases (4635 to 4685 Discovery Drive), sanitary utilities are located in near proximity to the foreshore slope. The foreshore slope erosion/regression will eventually cause loss of support to foundations causing settlement and eventual undermining of foundations. Similarly, when such erosion approaches underground utilities, such will undergo loss of trench support followed by settlement and possibly rupture of the utilities. It should be noted that shallow translational failures could cause unpredictable and instantaneous sections of the foreshore slope to fail, likely in the order of 3 metres horizontally inland from the crest of the slope. Such failures could cause immediate and severe damage to foundations and underground utilities.

Recommendations

We provide the following mitigation strategies associated with foreshore slope erosion.

- Protection against erosion of the foreshore slope by construction of revetment structures at or near the toe of the foreshore slope. The design of such would be based on information provided in the sea level rise assessment and wave energy/storm modelling. Ideally, such would consist of one continuous structure along the foreshore area.
- Creation/implementation of appropriate City bylaws that govern the capture and discharge of residential stormwater near the foreshore slope in a controlled manner.
- Capturing of City stormwater from streets near the foreshore slope and discharging by stormwater main or by suitable means at the toe of the slope.

CLOSURE

We trust the preceding is suitable for your purposes at present. If we can provide further information or clarification in this regard at this time, please contact us.

Regards,
Ryzuk Geotechnical

PROFESSIONAL
PROVINCE OF
JUNE 17, 2019
C. J. FLANAGAN
45682
BRITISH COLUMBIA
ENGINEER

Christian J. Flanagan, P.Eng.
Project Engineer

Shane W. Moore, P.Geo.
Senior Review Geoscientist
Managing Partner

Attachment: Appendix A – Photo Log

Practical. Innovative. Experienced.

Project: Geotechnical Assessment
Campbell River, BC

Client: Northwest Hydraulic Consultants

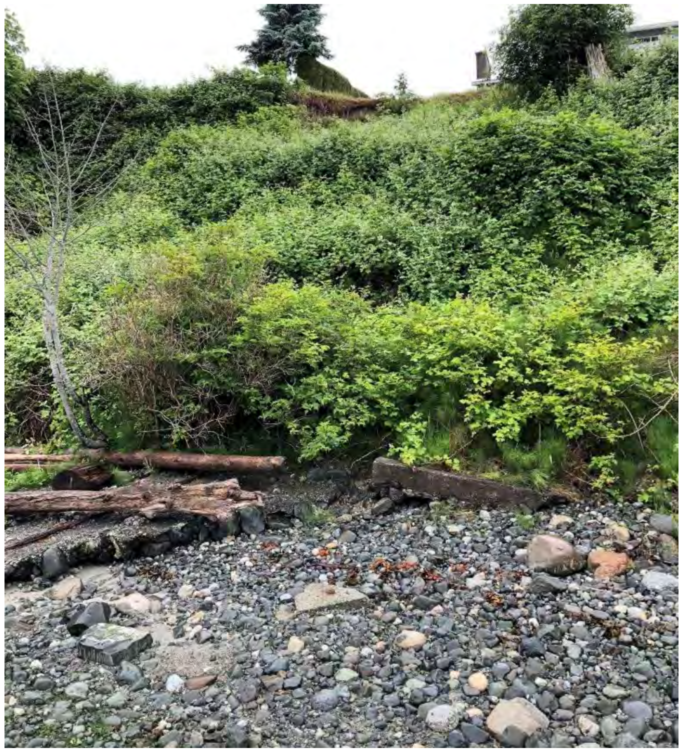
Investigation Date: June 8 and 9, 2018
Location: Painter-Barclay Escarpment

Inspector: CJF
Ryzuk Job Number: 8-8804-1

Appendix A: Photo Log

P01: Dilapidated boat ramp with toe erosion on both sides near 4785 Orange Point Rd.

P02: Toe erosion covered with common horsetail vegetation near 4685 Discovery Dr.



P03: Toe erosion and crest erosion near 4685 Discovery Dr.

P04: Close up of crest erosion near 4685 Discovery Dr.

P05: Toe erosion obscured by common horsetail vegetation near 4665 Discovery Dr.

P06: Undermined concrete pad caused by toe erosion near 4665 Discovery Dr.

P07: Toe erosion near 4665 Discovery Dr.

P08: Flexible drainage pipe near 4665 Discovery Dr.

P09: Tree showing signs of slope movement near 4665 Discovery Dr.

P10: Substantial erosional scarp obscured by common horsetail vegetation near 4655 Discovery Dr.

P11: Substantial erosional scarp obscured by common horsetail vegetation near 4655 Discovery Dr.

P12: Toe erosion near 4655 Discovery Dr.

P13: Loosely stacked boulder seawall near 4635 Discovery Dr.

P14: Loosely stacked boulder seawall near 4635 Discovery Dr.

P15: Drainage pipes discharging at toe near 4575 Discovery Dr.

P16: Drainage pipes discharging at toe near 4575 Discovery Dr.

P17: Substantial erosional scarp near 4565 Discovery Dr.

P18: Substantial erosional scarp near 4565 Discovery Dr.

P19: Substantial erosional scarp near 4565 Discovery Dr.

P20: Near 4565 Discovery Dr.

P21: Near 4565 Discovery Dr.

P22: Toe erosion near 4585 Discovery Dr.

P24: Toe erosion near 4535 Discovery Dr.

P24: Toe erosion exposing glacial till near 4535 Discovery Dr.

P25: Poorly installed drainage pipe and toe erosion near 4535 Discovery Dr.

P26: Toe erosion near 4535 Discovery Dr.

P27: Toe erosion near 4535 Discovery Dr.

P28: Rock groynes near 4515 Discovery Dr.

P29: Rock groynes near 4515 Discovery Dr.

P30: Near 4575 Discovery Dr.

P31: Near 4475 Discovery Dr.

P32: Near 4475 Discovery Dr.

P33: Near 4475 Discovery Dr.

P34: Near 4435 Discovery Dr.

P35: Near 4435 Discovery Dr.

P36: Near 4455 Discovery Dr.

P31: Near 4455 Discovery Dr.

P37: Near 4435 Discovery Dr.

P38: Near 4435 Discovery Dr.

P39: Near 4435 Discovery Dr.

P40: Near 4435 Discovery Dr.

P41: Toe/crest erosion near 4385 Discovery Dr.

P42: Crest erosion near 1800 Hughes Dr.

P43: Crest erosion near 4365 Discovery Dr.

P44: Close up of crest erosion near 4365 Discovery Dr.

P45: Close up of crest erosion near 4365 Discovery Dr.

P46: Near 4315 Discovery Dr.

P47: Toe erosion near 4315 Discovery Dr.

P48: Near 4315 Discovery Dr.

P49: Near 4315 Discovery Dr.

P50: Near 4235 Discovery Dr.

P51: Loosely stacked boulders near 4235 Discovery Dr.

P52: Near 1702 Wood Rd.

P53: Near 1702 Wood Rd.

P54: Near 4125 Discovery Dr.

P55: Toe erosion near 4125 Discovery Dr.

P56: Toe erosion near 4065 Discovery Dr.

P57: Toe erosion near 4065 Discovery Dr.

P58: Near 4065 Discovery Dr.

P59: Near 1665 Pengelley Rd.

P60: Seawall near 3847 McDougall Way

P61: Tiered slope and boulder stack walls near 3847 McDougall Way

P62: Loosely stacked boulder seawall near 3847 McDougall Way

P63: Loosely stacked boulder seawall near 3847 McDougall Way

P64: Loosely stacked boulder seawall near 3817 McDougall Way

P65: Near 3817 McDougall Way

P66: Storm water discharge to crest of foreshore slope near 4730 Orange Point Rd.

City of Campbell River

nhc

northwest hydraulic consultants

water resource specialists