APPENDIX A

HARBOURSIDE WATERFRONT

SEA LEVEL RISE FLOOD MANAGEMENT PLAN

Prepared for: Concert Properties

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November 15, 2012 - *Amended October 23, 2013*

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TABLE OF CONTENT

1.0 INTRODUCTION

1.1 FLOOD MANAGEMENT PLAN PURPOSE

This Flood Management Plan has been developed to support Concert and Knightsbridge Properties' rezoning application to the City of North Vancouver (the City) for Harbourside Waterfront. The rezoning application is being managed by the owners. Given Harbourside Waterfront's location and elevation, the growing risks associated with Sea Level Rise (SLR) and provincial government's draft adaptation guidelines, the owners were requested to develop a SLR management plan as a rezoning condition. This Flood Management Plan has several purposes:

- Define Flood Construction Level (FCL) Alternatives as a design basis of flood protection that responds to the Province's and Local Government guidelines for coastal flood hazard land use;
- Explain the site specific, technical review that supports the rezoning design basis; and
- Describe the Management Plan upon which the rezoning design basis is established.

1.2 SEA LEVEL RISE FLOOD MANAGEMENT PLAN OBJECTIVES

This is a high level Management Plan with several overriding objectives:

1. Define Year 2100 FCL alternatives incorporating SLR, regional and site level knowledge that conforms to provincial and local guidelines.
2. Develop a strategy framework using mutually reinforcing management approaches that avoid, accommodate or protect Harbourside Waterfront from flooding risk, support adaptive management options as risks change over time, and capitalize on design features that enhance the neighbourhood's character.
3. Create a solid foundation for City action in the broader Harbourside neighbourhood in the short term, and flexibility for adaptive management in the future.
4. Develop strategies upon key socio-economic and environmental principles:
   a. manage private and public risks while minimizing costs;
   b. contribute to the living and working experience of the neighbourhood in short and long term;
   c. use defensible science and engineering knowledge; and
   d. improve ecosystem health and function within site constraints.

1.3 SCOPE OF THE MANAGEMENT STRATEGIES

The Management Plan scope is focused on the buildings, and the owners' infrastructure within Harbourside Waterfront for the service life of representative buildings, i.e. up to 2100. Kings Mill Walk, and the proposed Spirit trail, are also vulnerable to SLR. Management strategies have been developed to reduce these vulnerabilities, and integrate the park into the broader flood management regime. An improved shoreline protection system has been developed from West of Burrard Yacht Club to a point perpendicular with the Western extent of Harbourside Place – this is also integral to the protection of Harbourside Waterfront. The Dog Park and the Western part of the Spirit trail are excluded from this rezoning application. The park and shoreline defence system would be designed in consideration of SLR for representative service lives. These assets would be maintained by the City.

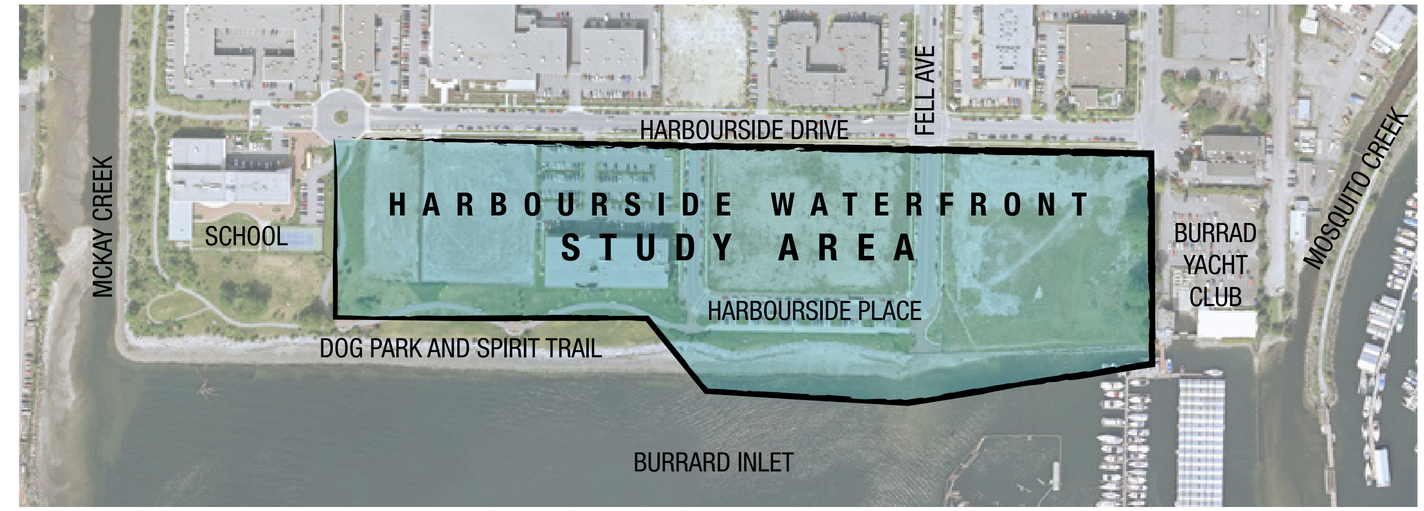


Figure 1. Scope of the Management Plan and site overview.

The City and broader Harbourside stakeholders will confront rising vulnerabilities to future SLR pertaining to broader Harbourside. As a neighbourly courtesy, key vulnerabilities and additional management considerations for the broader Harbourside area are discussed further in Section 4.4 of this plan.

1.3.1 COMMUNITY CO-BENEFITS

The owners' management strategy would provide significant co-benefits in the short term to many property owners, employees, and residents in the area and the City, including contributing to reduce the following:

- The flooding risk to city streets in the Harbourside Village for commercial, personal, public access, including emergencies;
- The park's vulnerability and increase its resiliency to flooding, enhancing recreational access and ecosystem benefits; and
- The vulnerability to storm surge to much of the broader Harbourside area, notably North of Harbourside Drive.

As outlined in the scope of work, the Study Area focuses on the owners' property, and the key area of the park and a portion of the shore line. Additional measures should be taken by others to ensure resiliency of the broader Harbourside neighbourhood. These additional management considerations are discussed in section 4.4.

The Management Plan, furthermore, provides a solid foundation for the City and senior governments to adapt to rising sea levels on a short, medium and long term basis, providing a precedent for a sustainable coastal development.

1.4 METHODOLOGY

1.4.1 DEFINING SITE WATER LEVELS

The following is an overview of the approach taken to define site water levels and develop the management strategies:

- Review relevant policy and planning literature, specifically BC standards and guidelines, and discuss policy and planning activity with the City of North Vancouver and City of Vancouver (Vancouver Harbour neighbour).
- Facilitate a meeting to solicit input from City staff on the general approach Concert is pursuing to manage flooding from SLR.
- Review literature of physical hydrologic and geologic conditions influencing SLR along BC and Vancouver coasts, including tides, storm surges, winds, waves, climate and tectonic stability.
- Collect and review available near-shore bathymetry and topography data.
- Visit site to characterize features of the existing coastal morphology and shoreline protection.
- Collect historical water level records from Canadian Hydrographic Service (CHS) station at Point Atkinson and performing a site specific analysis of water levels.
- Collect wind records from Environment Canada station at Vancouver Harbour CS and Vancouver International Airport, and conduct a wind statistic and return period analysis.
- Meet with the City and their SLR advisors to gain insight into their methodology, data, scope of work, granularity of analysis, and FCL results, and policy intentions to ground truth Golder's data and inform Concert's management strategy.
- Site specific analysis of wind-wave hindcast, wave run-up, overtopping and shoreline revetment stability.
- Develop basic design concepts and alternatives relevant to flood management for the shoreline, park, roads, sidewalks and building finished floor elevations.
- Define draft alternative site specific Flood Construction Reference plane (FCRP) and alternative FCL for 2020, 2070 and 2100 planning horizons.
- Meet with the City and their SLR advisor to share draft management strategies and FCRPs and FCLs for planning horizons.
- Update management strategies and FCRPs and FCLs to reflect input from City and their SLR consultants.

The depth of analysis in this plan reflects the current stage in planning and design specificity. Further risk analysis and management consideration will be required with detailed designs.

1.4.2 PLANNING HORIZONS

Given the dynamic risks associated with SLR, management strategies have been developed to protect the village at several milestones:

- 2020 represents the approximate period of initial construction.
- 2070 represents the approximate period:
  a. that would allow more scientifically informed decision making about any SLR interventions pre 2100; and
  b. where there would be significant life-cycle driven decisions about the site, building components and/or buildings.
- 2100 is the recommended SLR planning horizon for longer term development and land use.[1] It represents the approximate period of the end of life of much older than average buildings, and major adaptive management interventions, such as elevating a dike.

BUILDING LIFESPAN

The Coastal Flood Hazard Land Use Guidelines (BC MoE, 2011) recommend "allowances for SLR until the Year 2100 should be used in current planning and building approvals." The guidelines also recommend that "FCLs anticipate the age of the lifespan of the proposed building."

This Plan outlines major management strategies for flood protection for decades for sites and buildings, with priority given to flood protection of residential parts of the development. At the same time, the Plan allows for adaptive management to increase the elevation of a concrete band dike feature to a higher FCL. All buildings, it should be noted, are landward (North) of this dike feature. The Harbourside Place commercial FFE is below the FCL, but designed with extra height ceilings to accommodate elevating floor levels if necessary in the future.

The building lifespan used for planning was 80 years, i.e. to 2100. This duration was selected as a long building lifespan based on knowledge about building life.[2] A survey of large (non-residential) buildings found 57% are demolished within 50 years and 75% within 75 years. Only 8% of large buildings have 75-100 year lifespans. Only 7% have lifespans beyond 100 years.

2.0 CONTEXT

2.1 SEA LEVEL RISE SCIENCE AND RISKS

The balance of scientific evidence shows a well established SLR trend that can be attributed in part to human-induced climate change. Specific scientific references are provided in the sections that follow.

2.1.1 GLOBAL SEA LEVEL RISE PROJECTIONS AND FACTORS

Over the last century, the mean sea level rose 20 cm. [3] Projected changes over the 21st century range considerably from as low as approximately 30 cm to 150 cm or more.

Human-induced climate change is the consequence of elevated concentrations of heat trapping greenhouse gases (GHGs) in the atmosphere. The climate change-related mechanisms contributing to SLR are threefold:

- ***Thermal Expansion of the Oceans:*** Oceans have absorbed most of the atmospheric heating from increased GHG concentrations in the past 50 years. [4] As the temperature of seawater increases, it expands, causing the ocean's volume to increase. This was the primary contributor to global SLR in the last century.

- ***Melting Land-Based Glacial Ice:*** Most global ice loss in the last century was from smaller continental glaciers across the Americas, Europe, Asia and Africa, not the major ice sheets on Greenland and Antarctica. [5]

- ***Melting Greenland and Antarctic Ice Sheets:*** Relatively recent evidence has shown this ice sheet loss is the fastest growing contributor to SLR. [6 7 8]

Given recent GHG emission trends and projections, SLR is expected to rise at accelerated rates over this century and the next. Even with significant GHG reductions, the inertia of these climatic changes will continue to drive SLR over several millennia. [9]

Uncertainty about future SLR magnitude is due to relatively inadequate knowledge of the complex processes of these major contributors, future temperatures and atmospheric GHG concentrations.

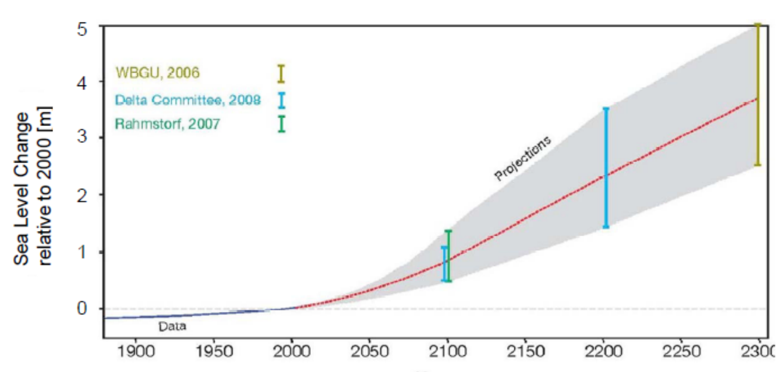


Figure 2. Range of recent peer reviewed projections of projected SLR [10]

2.1.2 LOCAL & REGIONAL FACTORS

Many local and regional factors influence SLR. Three principle ones relevant to this region are:

- ***Post Glacial Rebound:*** Most of coastal British Columbia is still rebounding from the loss of the immense weight of glaciers during the last ice age. This process is increasing the height of most of BC's coast at a rate of 0 – 3 millimetres per year, moderating global SLR. [11] This factor is relevant to the Harbourside area.

- ***Tectonic Forces:*** The Juan de Fuca tectonic plate off of BC's Coast continues to move under the North American plate. [12] This movement results in annual uplift of 2-3 millimetres off western Vancouver Island and Haida Gwaii and diminishes moving eastwards to a rate of zero near Vancouver. Vancouver Island's uplift would be partially reversed by a large earthquake.

- ***Sediment Compaction:*** Sediment compaction in the Fraser River Delta is resulting in subsidence rates of 1-2 millimetres per year, which includes the municipalities of Richmond and Delta. [13] Higher subsidence rates of greater than 3 millimetres per year are mostly connected to large construction projects such as BC Ferries Terminal in Tsawwassen, Deltaport, and the Vancouver International Airport.

2.1.3 SEA LEVEL RISE IMPACTS

SLR will result in significant land loss. Within the Lower Fraser River Basin and Southeastern Vancouver Island, a one metre SLR by 2100 would submerge 4,600 hectares of farmland and 2,675 hectares of combined residential, commercial and industrial areas. [14] This is roughly equivalent to losing the landmass of the municipalities of Hope and Port Moody respectively.
Without intervention, the impacts on coastal areas are significant, including [15]:

- More flooding of land, infrastructure, roads and buildings, with associated property damage, ecosystem changes, and risk to human well being;
- Increased coastal erosion from more land exposure to higher water levels and waves;
- Saltwater intrusion of coastal land and lands bordering rivers in coastal areas, altering agricultural potential and groundwater sources for drinking and agriculture;
- Changes in coastal sedimentation; and
- Elevating groundwater, impacting building foundations and underground parking, and compromising gravity based wastewater and stormwater systems with diverse implications including backups.

2.2 SEA LEVEL RISE POLICY CONTEXT

2.2.1 BRITISH COLUMBIA

SLR is compelling many governments to respond in a more systematic, proactive manner, the BC Government amongst them. In 2010, the BC Government embarked on a project focusing on strengthening land use planning and dikes in coastal areas. The project involved engineering analysis, scientific and policy review, and engagement. Three documents were generated out of this process to provide guidance for land use planning and dike design:

- BC Ministry of Environment: Climate Change Adaptation Guidelines for Sea Dikes and Coastal Flood Hazard Land Use – Draft Policy Discussion Paper, January 2011
- BC Ministry of Environment: Climate Change Adaptation Guidelines for Sea Dikes and Coastal Flood Hazard Land Use - Guidelines for Management of Coastal Flood Hazard Land Use, January 2011
- BC Ministry of Environment: Climate Change Adaptation Guidelines for Sea Dikes and Coastal Flood Hazard Land Use – Sea Dike Guidelines, January 2011

BC GOVERNMENT RECOMMENDED SLR PROJECTIONS

As a basis for planning, the BC Ministry of Environment (MoE) has recommended using the SLR curve presented in Figure 3. The guideline states that the recommended curve is slightly higher than the high projection for the years from the present up to year 2070, and moves below the current median projection with the recognition that in a planning framework, time remains to revise the recommended curve upwards.

Accordingly, SLR at time frames of 2020, 2070 and 2100 are projected to be increased by 0.2 m, 0.7 m, and 1.0 m respectively. BC SLR projections needs to be adjusted to account for local/regional geological conditions.

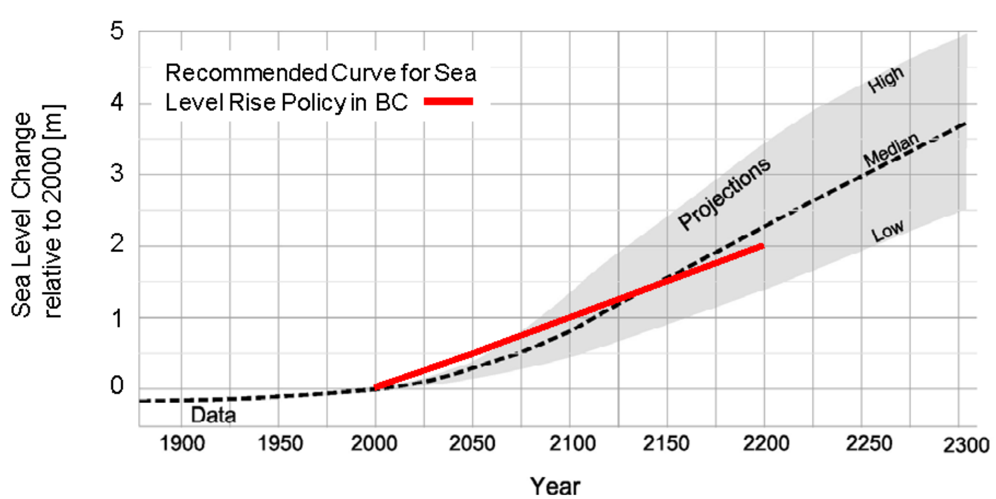


Figure 3. Recommended Global SLR Curve for Planning and Design in BC [22]

KEY LAND USE GUIDELINES

Key MoE land use guidelines relevant to this project are:

- ***Building Life Cycle Planning:*** "Given the prospect of rising sea level, it is necessary to establish a FCL that anticipates a level applicable to the end of the lifespan of the proposed building." [16]
- ***2100 FCL for Current Planning:*** "For land use management guidance in BC, allowances for SLR until the Year 2100 should be used in current planning and building approvals." [17]
- ***Adaptive Management:*** "Land use and building approvals based on FCL for 2100 should also include provisions for adaptive management of land uses to SLR to the Year 2200 and beyond." [18]
- ***Dynamic FCLs:*** "...FCL(s) will need to change over time as SLR continues. The pace of SLR and the arrival of more accurate projections will inform how often a revision to FCL needs to take place." [19]

2.2.2 FLOOD CONSTRUCTION LEVEL PARAMETERS

FCL is comprised of several parameters. These parameters are shown below:

Table 1. FCL parameters

High Tide	The highest tides at a site. Professionally, this is the Higher High Water Large Tide (HHWLT)
Storm Surge Allowance	A change in water level caused by the action of wind and atmospheric pressure variation on the sea surface. It can occur at any tide and can be positive or negative, though the effect of concern is a rise in water level above the astronomical tide level.
Wave Effect Allowance	The allowance for wave run up based on local winds, open water, and specific shoreline conditions
Regional Sea Level Rise	Sea level rise adjusted to regional/local conditions.
Freeboard	A margin of safety to account for uncertainty in climate impacts such as SLR height and storm intensity, and site conditions that may not be accounted for such as subsidence.

Several different flood levels are defined by MoE. The most important is the FCL.

Table 2. Definition of reference flood levels

Designated Flood Level (DFL)	= Future SLR Allowance + HHWLT + Total Storm Surge during Designated Storm
FCRP	= DFL + Estimated Wave Effect
FCL	= FCRP + Freeboard Allowance

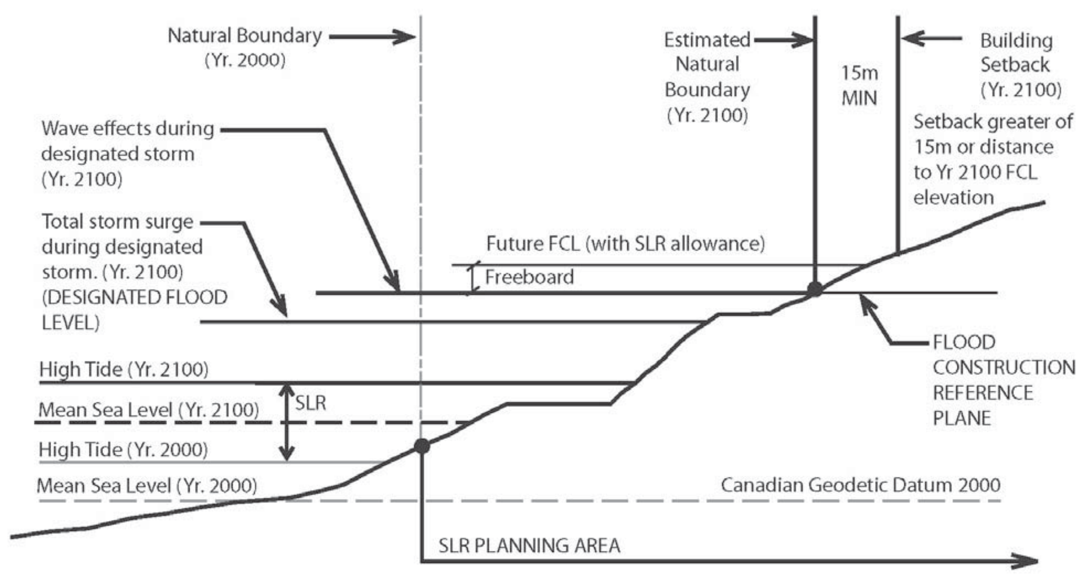


Figure 4. Definition for FCL and setback [22]

2.2.3 MUNICIPAL ACTION

Municipalities are only just now starting to respond to the BC Government's Guidelines. The following is an overview of the SLR flood management activity of the City of North Vancouver, and across the harbour at the City of Vancouver. [20]

CITY OF NORTH VANCOUVER

The following are key City activities related to flood management and SLR:

- The City's historic FCL established in the 1980's is 3.35 m (Bylaw 6746), and the City has recently updated this to an FCL of 4.5m.
- The City will be developing a community-wide SLR management strategy, but not before this rezoning application has been submitted.
- The City requested a SLR Flood Management plan from the Harbourside Waterfront owners as a rezoning condition, and indicated an interest in alternatives including phased adaptation to SLR.
- Until recently, the City indicated it would adopt the preliminary 2100 5.6 m FCL estimate for Vancouver Harbour referenced in the provincial guidelines. This was superceded by the 4.5m FCL established in the recent Bylaw update, consistent with City of Vancouver practice.
- The City commissioned a study to evaluate vulnerability of its waterfront and establish preliminary FCLs. This study is still reasonably coarse, evaluating seven reaches between Second Narrows and Lions Gate Bridge.
    - While this study has not been finalized, the preliminary, unpublished FCL being considering for the broader Harbourside site for 2100 is higher than the preliminary provincial FCL for Vancouver Harbour. The primary variable is wave effect allowance, which is variable with shoreline shaping.

CITY OF VANCOUVER

Sharing the same body of water, on the South shore of Burrard Inlet is the City of Vancouver. The following are key (City of) Vancouver activities related to flood management and SLR: [21]

- The City's historic FCL established in the 1980's is 3.35 m (Bylaw 6746). The City has recently been considering updating this FCL to 4.5 m in its Zoning Bylaw, but this change is not implemented at time of rezoning application.
    - New developments in Vancouver Harbour, e.g. North East False Creek, are conforming to the 4.5 m FCL
    - Some developers along the Fraser River have elevated large portions of their site by 1 m.
    - Until recently, the City indicated it would adopt the preliminary 2100 5.6 m FCL estimate for Vancouver Harbour referenced in the provincial guidelines. This may be superseded by the proposed 4.5 m FCL, which would be consistent with current City of Vancouver guidelines.
- SLR risk studies for City of Vancouver are ongoing, and it will be updating its flood proofing policies in the future as studies are completed.

3.0 DEFINING SITE WATER LEVELS

This section defines the constituent parameters of the site water levels, and alternative FCLs for future planning horizons.

3.1 TOPOGRAPHY AND BATHYMETRY

Figure 5 and 6 are topographic and bathymetric maps of the Concert site, generated from the following data sources:

- Topographic Map 092G06 for North Vancouver at 1:50,000 scale
- LiDAR data 2007, bare earth as provided by the City of North Vancouver
- Point survey conducted by Hobbs, Winter & MacDonald on September 4, 2012
- Nautical Chart # 3493 for Vancouver Harbour Western Portion at a scale of 1:10,000
- Partial Bathymetric survey data conducted by Underhill & Underhill on August 7, 2012

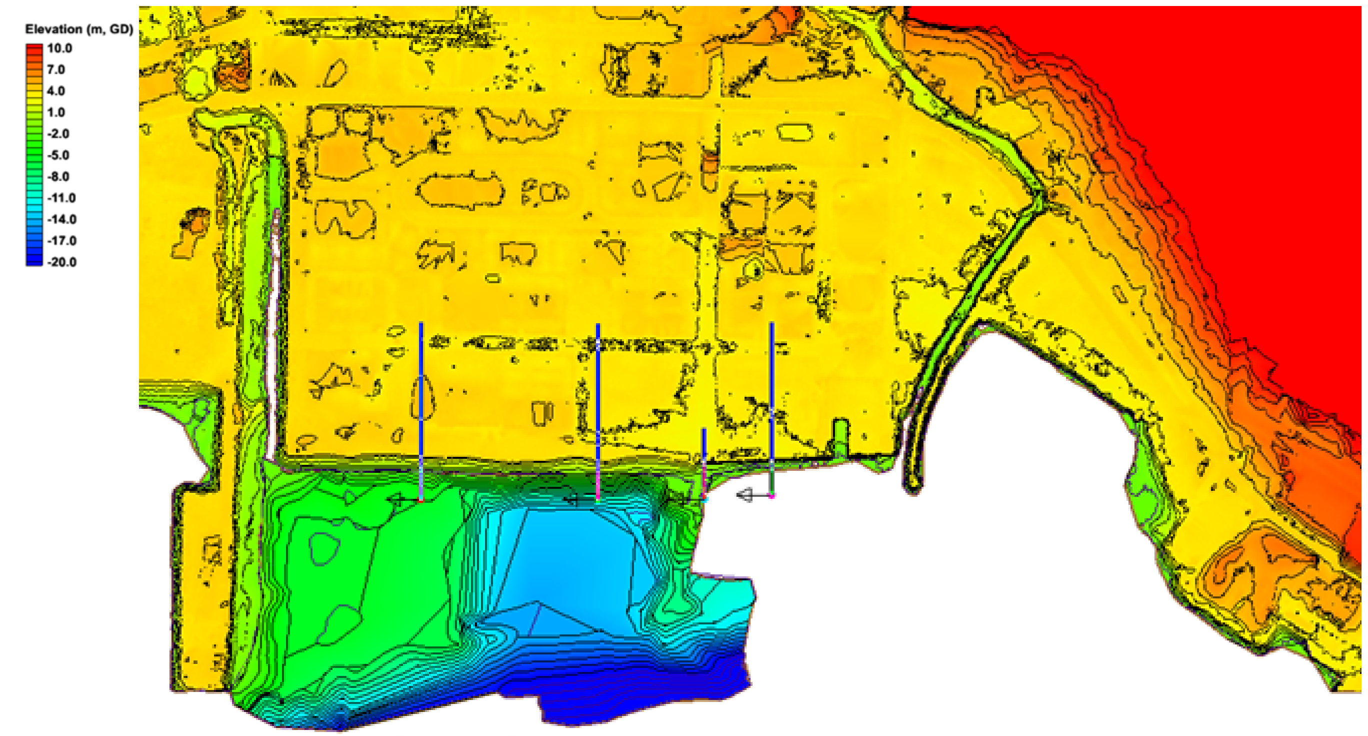


Figure 5. Topographic and bathymetric map showing the site and its large context

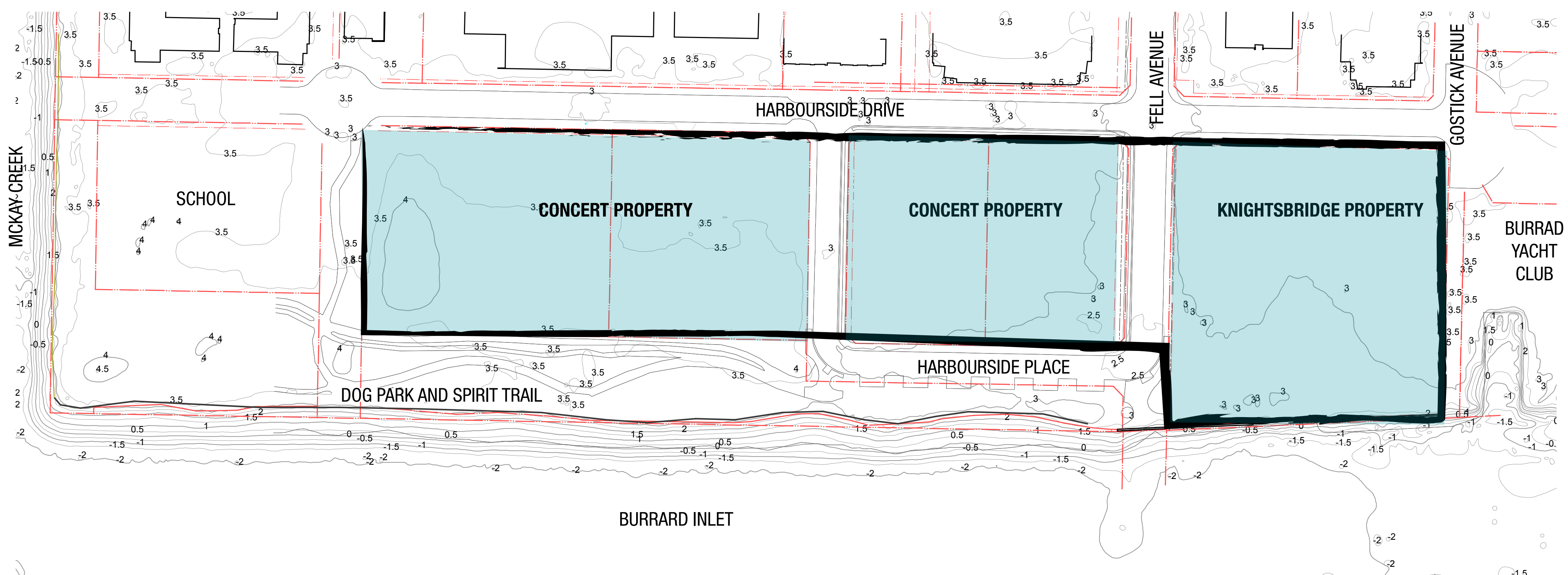


Figure 6. Topographic map of the site - Enlargement

3.2 PRESENT TIDAL LEVELS

Tidal levels due to astronomical tides are available from Canadian Hydrographic Service (CHS), Fisheries and Oceans Canada. The Canadian Tide and Current Table – Volume 5, published annually by CHS, provides the tidal ranges for Vancouver as summarized in Table 3. The tidal levels are referred to CGD which is defined as a Mean Water Level (MWL). Chart Datum (CD) is approximately -3.1 m CGD. The tidal level parameter used to define FCL is the Higher High Water Large Tide. The value of this parameter is the average of the highest high waters for a tide station over an established 19 year astronomical cycle.

Table 3. Tidal Levels for Georgia Strait at Point Atkinson, Vancouver *(Source: Canadian Tide and Current Tables, 2012 – Volume 5)*

Tides	Water Level (m, CGD)
HHWLT	1.9
Higher High Water, Mean Tide (HHWMT)	1.3
MWL	0.0
Lower Low Water, Mean Tide (LLWMT)	-2.0
Lower Low Water, Large Tide (LLWLT)	-3.2

3.3 STORM SURGES AND ANALYSIS OF EXTREME WATER LEVELS

3.3.1 PREVIOUS STUDIES

A preliminary analysis of extreme water levels was published in BC MoE Guidelines [22] based on several long term water level records measured at CHS tide stations along the British Columbia coast. The measured tidal data was considered as an integrated effect of the astronomical tide and the residual water level. The statistical Peaks Over Threshold (POT) analysis of the residual water level from the CHS station #8615 at Tofino (with records available for a 54 year period) provided a range of Annual Exceedance Probabilities (AEP) as shown in Table 4. It was assumed that the general characteristics of residual water levels calculated for the Tofino station could be applied to the West Coast Vancouver Island – Juan de Fuca Strait and Georgia Strait.

Following Table 4, a water level of 1.25 m was approximated and adopted as a 200-year AEP storm surge event in this project for Planning Horizons 2020, 2070 and 2100. However, additional literature review (e.g., BCMOE Sea Dike Guidelines (2011 [23]); Abeysirigunawardena et al. (2011 [26])) suggest that a 1.25 m 200-year AEP surge may be an over conservative (high) estimate for Point Atkinson (refer to section 3.3.2 for information supporting this).

Table 4. Expected Magnitude of Residual Water Levels for Georgia Strait at Point Atkinson, Vancouver *(Source: BCMOE Guidelines)*

AEP (%)	AEP (return year)	Water Level (m, CGD)
50	1	0.73
20	5	0.83
10	10	0.9
4	25	1.0
2	50	1.1
1	100	1.2
0.2	500	1.3
0.1	1000	1.4

3.3.2 WATER LEVEL ANALYSIS AT POINT ATKINSON STATION

Water level data from Point Atkinson Tide Station (Fisheries and Oceans Canada 2012 ([23]; Latitude: 49.337º N, Longitude: 123.253º W) at the mouth of Burrard Inlet northwest of First Narrows were downloaded and merged to provide water surface elevation information in support of the proposed SLR adaptation plan. The data were initially referenced to CD, but were adjusted to reference CGD for the purposes of this project. A 3.1 m adjustment was applied to elevations to convert from CD to CGD. Hourly data were available over a 98-year interval from 1914 through 2012; however, there were significant data gaps, sometimes spanning multiple years, in the early stages of the record. Because of the large gaps in the time series, the 51-year interval from 1961 through 2012 was selected as having limited time series gaps and was employed for the tidal statistics analysis. Figure 7 shows the complete water level record from 1914 through 2012. Note that there is a more continuous record in the latter half of the time series between 1961 and 2012.

An extremal analysis based on the annual maximum series was conducted on the 51-year sea level dataset to develop a table of return period water levels in support of the project. A total of 51 events (i.e., one maximum water surface elevation per year) were included in the extremal analysis. The results of the extremal analysis from the Total Water Level (TWL) record are presented in Figure 8, Table 5 and Table 6.

First, the annual maxima, record maximum, annual minima, record minimum, and record range were calculated. Two annual maximum values were within 3 cm and therefore both are presented as record maxima:

- Maximum recorded: 2.500 m (CGD) (5.600 m, CD) on 16 December 1982.
- Near maximum measurement: 2.470 m (CGD) (5.570 m, CD) on 05 December 1967.

The minimum water level recorded over the entire 98-year record was recorded in 1932 prior to the continuous data record.

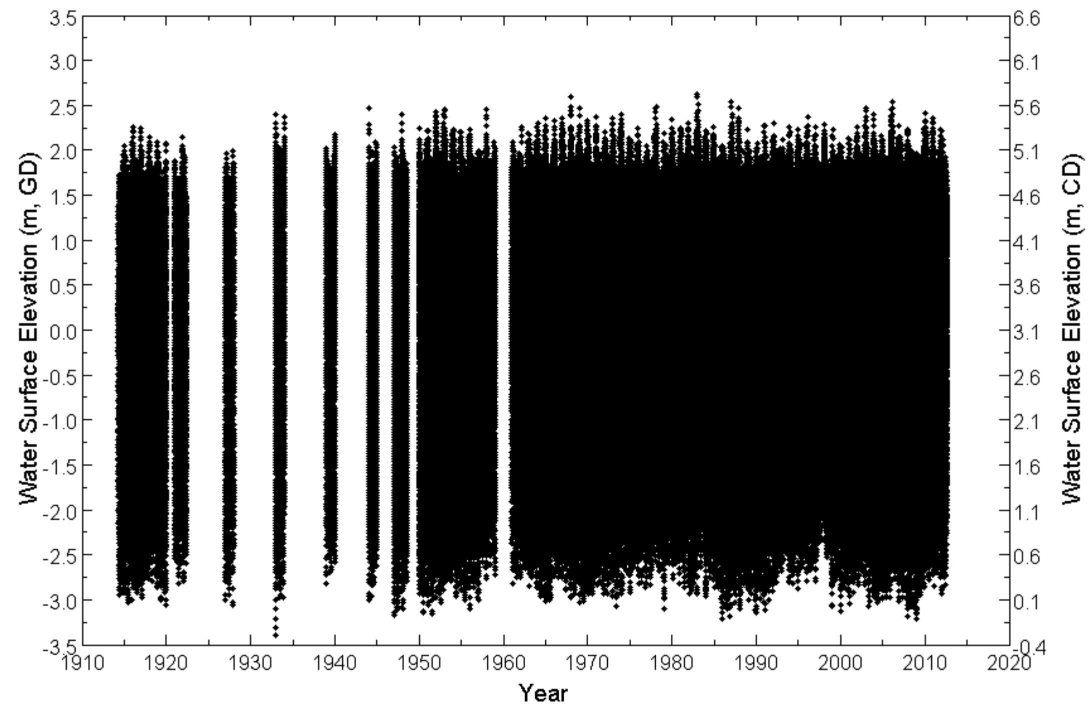


Figure 7. Hourly Sea Level Record at Point Atkinson, BC Tide Station (#7795) 1914-2012

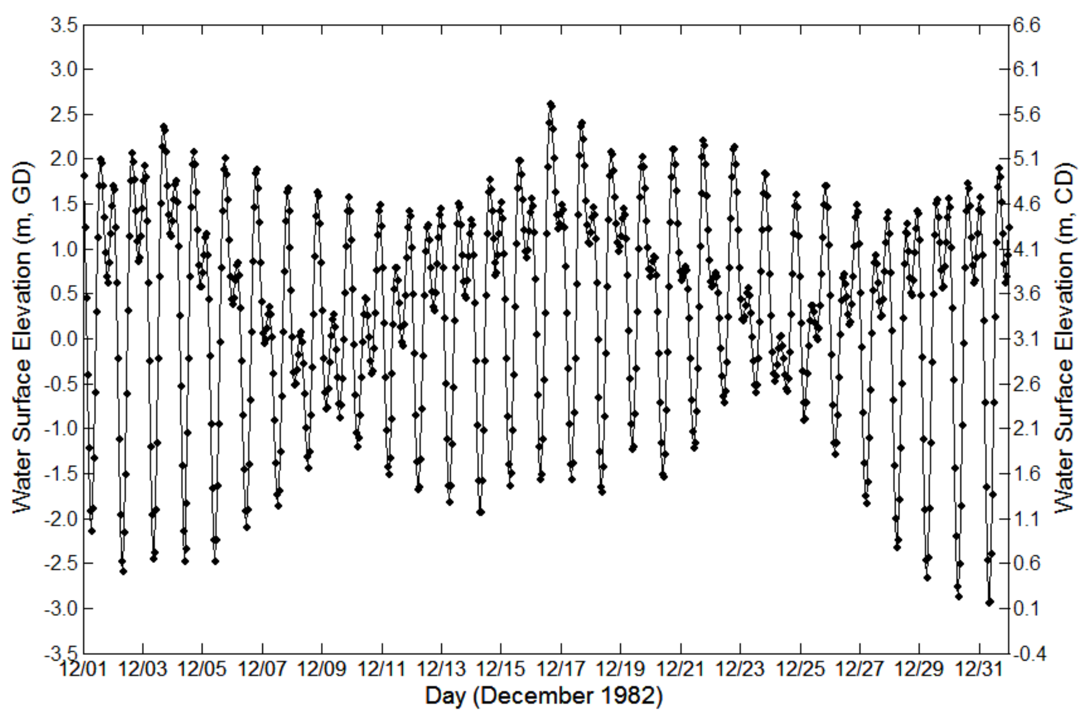


Figure 8. Hourly Sea Level Record at Point Atkinson, BC Tide Station (#7795) December 1982

Table 5. Extremal Analysis of Total Water Surface Elevation (in m, CGD) Measured at Point Atkinson

N= 51	Nu= 1.00					
NT= 51	K= 51	FT-I	Weibull	Weibull	Weibull	Weibull
Lambda=1.00			k= 0.75	k= 1.25	k= 1.55	k= 2.00
Correlation Coefficient		0.9778	0.8761	0.9591	0.9751	0.9845
Sum Square of Residuals		0.04	0.23	0.08	0.05	0.03
Return Period (years)		Water Surface Elevation (m, CGD)				
	5	2.28	2.23	2.27	2.28	2.29
	10	2.36	2.32	2.36	2.37	2.37
	25	2.46	2.46	2.47	2.46	2.45
	50	2.54	2.57	2.55	2.53	2.51
	100	2.62	2.69	2.63	2.59	2.56
	200	2.69	2.82	2.70	2.65	2.60
	500	2.79	3.00	2.79	2.73	2.66
	1000	2.87	3.13	2.86	2.78	2.70
	2000	2.94	3.28	2.93	2.83	2.74
	4000	3.02	3.42	3.00	2.88	2.78
Confidence Interval	Return Period (yrs)					
95 % C.I.	10	2.3 - 2.4	2.2 - 2.5	2.3 - 2.5	2.3 - 2.5	2.3 - 2.4
95 % C.I.	25	2.4 - 2.6	2.2 - 2.7	2.3 - 2.6	2.4 - 2.6	2.4 - 2.5
95 % C.I.	50	2.4 - 2.7	2.3 - 2.9	2.4 - 2.7	2.4 - 2.7	2.4 - 2.6
95 % C.I.	100	2.5 - 2.8	2.3 - 3.1	2.5 - 2.8	2.5 - 2.7	2.5 - 2.7
95 % C.I.	200	2.5 - 2.8	2.3 - 3.3	2.5 - 2.8	2.5 - 2.8	2.5 - 2.7
95 % C.I.	500	2.5 - 2.8	2.4 - 3.4	2.5 - 2.8	2.5 - 2.8	2.5 - 2.7
95 % C.I.	1000	2.5 - 2.8	2.4 - 3.4	2.5 - 2.8	2.5 - 2.8	2.5 - 2.7
95 % C.I.	2000	2.5 - 2.8	2.4 - 3.4	2.5 - 2.8	2.5 - 2.8	2.5 - 2.7
95 % C.I.	4000	2.5 - 2.8	2.4 - 3.4	2.5 - 2.8	2.5 - 2.8	2.5 - 2.7

Table 6. Extrapolated Return Periods from the Extremal Analysis

AEP Return Period (years)	Water Surface Elevation (m, CGD)	Water Surface Elevation (m, CD)
5	2.29	5.39
10	2.37	5.47
25	2.45	5.55
50	2.51	5.61
100	2.56	5.66
200	2.60	5.70
500	2.66	5.76
1000	2.70	5.80
2000	2.74	5.84
4000	2.78	5.88

This value is presented below along with two minima recorded after 1961 during the period of continuous water level measurement:

- Minimum recorded (occurs prior to continuous time series): -3.520 m (CGD) (-0.420 m, CD) on 29 December 1932.
- Minimum recorded after 1961: -3.341 m (CGD) (-0.241 m, CD) on 12 January 2009.
- Near minimum measurement: -3.240 m (CGD) (-0.240 m, CD) on 13 December 1985.

The maximum recorded water level on 16 December 1982 occurred during the peak of the strong 1982-1983 El Niño. Variations in the El Niño-Southern Oscillation (ENSO) index have been linked to elevated sea level 0.3 - 0.4 m above the mean (Bornhold 2008 [29]). Figure 8 presents a truncated time series for the month of December 1982 and including the 16 December 1982 maximum measured water level.

Table 5 presents the results from the TWL extremal analysis. Sensitivity was conducted on the Weibull k parameter to identify the value of k that produced the highest correlation coefficient. In this case, k = 2.00 resulted in the highest correlation coefficient of 0.9845 indicating that value as the most appropriate value to use and also indicating a high degree of statistical correlation. Table 6 summarizes the return period results from Table 5 into a concise form including conversion to other relevant vertical tide datum (CD). The water surface elevation is plotted in Figure 9 as a function of return period for the Weibull distribution (k=2.00) along with the 95% confidence intervals.
A probability of exceedance analysis was performed using the calculated probability of water level exceedance of each of the return periods for the three established planning horizons: 2020 (8 years from present), 2070 (58 years from present), and

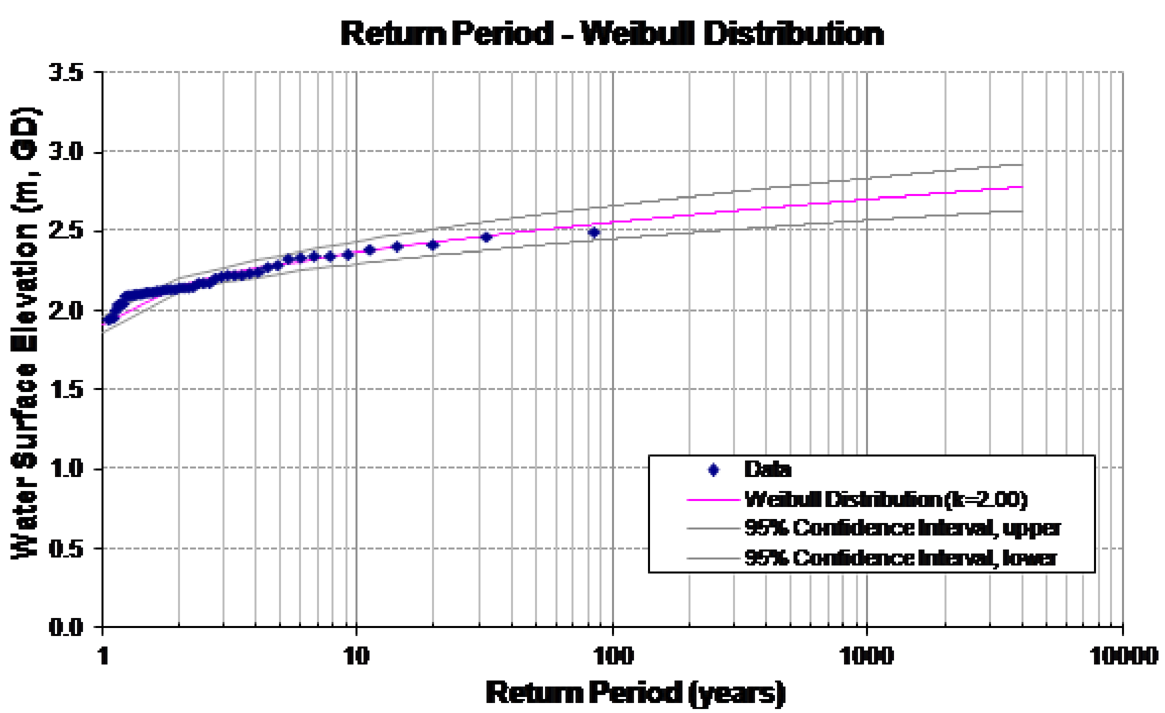


Figure 9. Sea Level as a Function of Return Period for Weibull Distribution (k=2.00)

2100 (88 years from present) as shown in Table 7. This analysis offers a statistical probability of various water level occurrences over the data collection period.

The TWL statistical analysis was performed on a 51-year water surface elevation time series at Point Atkinson Tide Station (Station ID: #7795). Despite a nearly 100-year record, significant data gaps required that the analysis be applied to a much shorter continuous record spanning 51 years. Due to the relatively short continuous record obtained from the time series there is significant uncertainty in the long-term estimates of sea level return periods. A typical rule of thumb suggests that extrapolations are valid out to a period three times the length of the time series (3 x 51 = 153 years). Therefore, this would suggest that the 100-year return period is statistically reliable, but return intervals beyond the 200-year return period should not be relied upon. However, the 2000 and 4000 AEP return TWLs are provided for comparison to the coupled AEP for the storm surge + HHWLT established in the BC MoE Guidelines. [23]

The HHWLT by definition is the average annual maximum astronomic tide calculated over a 19-year epoch, or as defined by

Table 7. Percent Chance of Value Equaling or Exceeding Return Period Sea Level by Each Planning Horizon: 2020 (8 years), 2070 (58 years), and 2100 (88 years)

AEP Return Period (years)	Planning Horizon		
	2020	**2070**	**2100**
5	83%	100%	100%
10	57%	100%	100%
25	28%	91%	97%
50	15%	69%	83%
100	8%	44%	59%
200	4%	25%	36%
500	2%	11%	16%
1000	1%	6%	8%
2000	0%	3%	4%
4000	0%	1%	2%

CHS: "the average of the highest high waters, one from each of 19 years of predictions." Therefore, it is the 1/19 AEP tide or approximately the 1/20 AEP return astronomic tide. The coupled AEP return would mean that the 1/200 storm surge occurs at the peak of the HHWLT, or at an AEP return of 1/4000 since the product of the 1/20 HHWLT and the 1/200 surge is 1/4000 TWL. Therefore, it is expected that the 1/4000 AEP TWL should be approximately 3.15 m, or: 1.9 (HHWLT from Table 2) + 1.25 (1/200 Storm Surge from Table 3). However, the calculated 1/4000 AEP TWL is 2.78 m (Table 6), a significant difference despite the uncertainties.

It is important to note that this analysis was conducted on the TWL record, which includes astronomical tides as well as storm surge and other meteorological and atmospheric forces contributing to water level variability. In order to separate the storm surge signal from the TWL record, an additional analysis of the astronomic tide would be required to determine the residual between the astronomic tide and the TWL. Abeysirigunawardena (2010 [25]) and Abeysirigunawardena et al. (2011 [26]) conducted an analysis of sea level and climatologic forcing for several tide stations in coastal British Columbia including Pt. Atkinson and Tofino. In addition to a TWL analysis that yielded similar results (well within the uncertainty) of the results provided in Table 5, Abeysirigunawardena (2010 [25]) and Abeysirigunawardena et al. (2011 [26]) also conducted a storm surge analysis of the residuals.

The surge was computed by subtracting the predicted astronomical constituent tidal level from the measured TWL at each tide gauge, then a POT analysis was conducted to develop the AEP return storm surges. The results of the surge analysis for various return periods covered in this document and for the maximum measured surge are presented in Table 8 (Abeysirigunawardena 2010 [25]; Abeysirigunawardena et al. 2011 [26]).

In these results, a 100-year AEP return was supplied as 0.998 m at Pt. Atkinson and 1.163 m at Tofino. Therefore, the estimated TWL for a 1/2000 AEP period (1/20 HHWLT x 1/100 surge = 1/2000) is calculated as 2.898 m, or: 1.9 (HHWLT from Table 2) + 0.998 (1/100 Storm Surge from Table 7). This is 16 cm greater than the 1/2000 AEP TWL value of 2.74 m (Table 6) but 36 cm less than the 1/2000 AEP estimated TWL at Tofino by Abeysirigunawardena (2010 [26] and the BC MoE Guidelines [23]. The BC MoE Guidelines reference an allowance for 200-year return period for storm surge, for which Abeysirigunawardena (2010 [26]) does not give a value.

Because of the significant differences between storm surge AEP return values at Pt. Atkinson and Tofino noted in the literature and this analysis, more detailed analysis is warranted to establish a regionally and locally relevant storm surge value for the City of North Vancouver. It is recommended that the City work with other governments, senior and local, and other private and public sector organizations with a stake in Vancouver Harbour (North America's third busiest port) to support this analysis.

3.3.3 RELATIVE SEA LEVEL RISE

Table 8. Extrapolated Storm Surge Return Periods and Measured Maximum Surge for Pt. Atkinson and Tofino Tide Stations (Abeysirigunawardena 2010 [25]; Abeysirigunawardena et al. 2011 [26])

Return Period (years)	Storm Surge (m) - Pt. Atkinson (7795)	Storm Surge (m) - Tofino (8615)
5	0.826	0.822
10	0.882	0.914
25	0.938	1.021
50	0.971	1.094
100	0.998	1.163
Maximum Measured	1.04	1.28

When the BC MoE's SLR projections are adjusted to the selected planning horizons of 2020, 2070 and 2100, sea level is projected to rise by 0.2 m, 0.7 m, and 1.0 m respectively.

The regional SLR for BC needs to be adjusted to account for regional crustal movements particular to the area under consideration. Thomson R., et.al [27] provided a description of crustal movements along the coast of British Columbia and a brief summary of uplift rates for selected locations, which were adopted in the BC MoE Guidelines [22]. An uplift rate was estimated at 1.2 mm/yr from 2010 at a reference location of Vancouver (19.287°N/123.110°W).
Considering both the effects of a predicted SLR due to climate change and an uplift rate of crustal movement at the specific project site, SLR at time frames of 2020, 2070 and 2100 are corrected to be 0.19 m, 0.63 m and 0.89 m respectively.

Geotechnical analysis will be necessary to ascertain long term site stability, notably subsidence risk. As buildings would be built on stone pilings placed on bedrock, the buildings are not vulnerable, however, the building sites, roads and park may be.

3.4 WIND CLIMATE

Table 9. Relative SLR to 2100

	2020	2070	2100
BC MoE SLR Projection 2010-2100	0.2 m	0.7 m	1.0 m
Adjusted for Regional Crustal Uplift	(1.2 mm/yr x 10 = .012 m)	(1.2 mm/yr x 60 = .072 m)	(1.2 mm/yr x 90 = .108 m)
Relative SLR	**0.19 m**	**0.63 m**	**0.89 m**

3.4.1 WINDS AT VANCOUVER HARBOUR

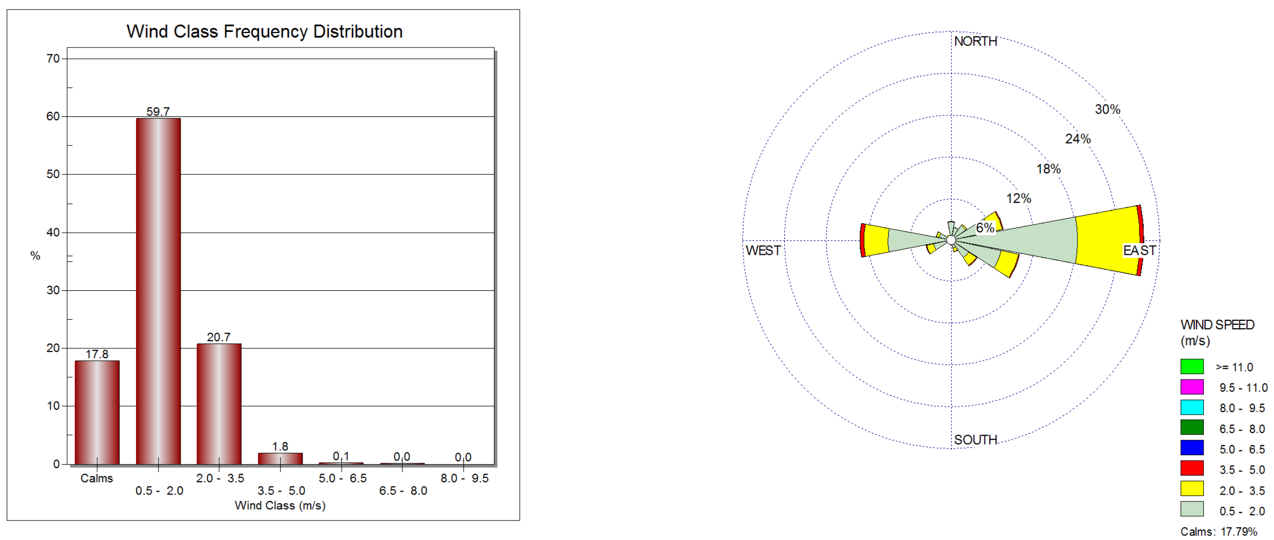
Wind data near the site is available from the National Climate Data and Information Archive, Environment Canada for Vancouver Harbour CS station (#1108446, 49°17'43.270" N/ 123°07'18.730" W), approximately 2.6 km southwest from the project site, as shown in Figure 10. This climate station consists of hourly climate records for the period from 1976 to present, with available wind data for 14 years for periods of January 1976 to December 1988, and January 1994 to December 1994.

Results of wind speed frequency and direction analysis for the climate station are summarized in Figure 11 a and b respectively. The data are summarized by the following observations:

- 98% of wind speeds are less than 3.5 m/s (or 12.6 km/hr), and 78% of wind of wind speed are less than 2.0 m/s (or 7.2 km/hr).
- Winds blow mostly from the East and the West, and the frequency from the East (approximately 27% of the time) is higher than from the west (approximately 13% of the time).
- Wind speeds greater than 12.6 km/hr occur mostly from the east, southeast, and west.
- The recorded maximum monthly wind speed at the station are summarized in Table 10. Wind maxima occurs mostly from the west in the winter and from the southeast and east in the summer. The maximum wind speed in the record is 46 km/h measured in 1978.

The probability and return period analysis based on the 14 year wind records is presented in Table 11 and Figure 12. The analysis indicates that the 200 year return period wind speed is estimated to be 48 km/hr.

Figure 10. Climate Station Vancouver Harbour CS and Wind Fetch at the Concert Project Site, North Vancouver, BC



a. Wind Frequency Distribution

b. Wind Rose Plot

Figure 11. Wind Frequency and Statistics at Climate Station Vancouver Harbour CS

Table 10. Recorded Maximum Winds at Climate Station of Vancouver Harbour CS (#1108446)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Maximum Hourly Speed (km/h)	41	43	44	**46**	44	41	33	30	33	43	44	44
Date (yyyy/dd)	1983/ 08	1977/ 21	1980/ 12	**1978/ 10**	1985/ 10	1985/ 15	1978/ 09	1978/ 24	1985/ 12	1980/ 08	1978/ 04	1979/ 09
Direction of Maximum Hourly Speed	W	SE	E	W	NW	NW	E	SE	SE	W	W	W

Table 11. Return Period and Probability of Climate Winds at Station Vancouver Harbour CS

Probability (%)	**Return Period** (return year)	**Wind Speed** (km/hr)
95.2	1.05	37
90.1	1.11	39
80	1.25	40
50	2	42
20	5	44
10	10	45
4	25	46
2	50	47
1	100	47.5
0.5	200	48

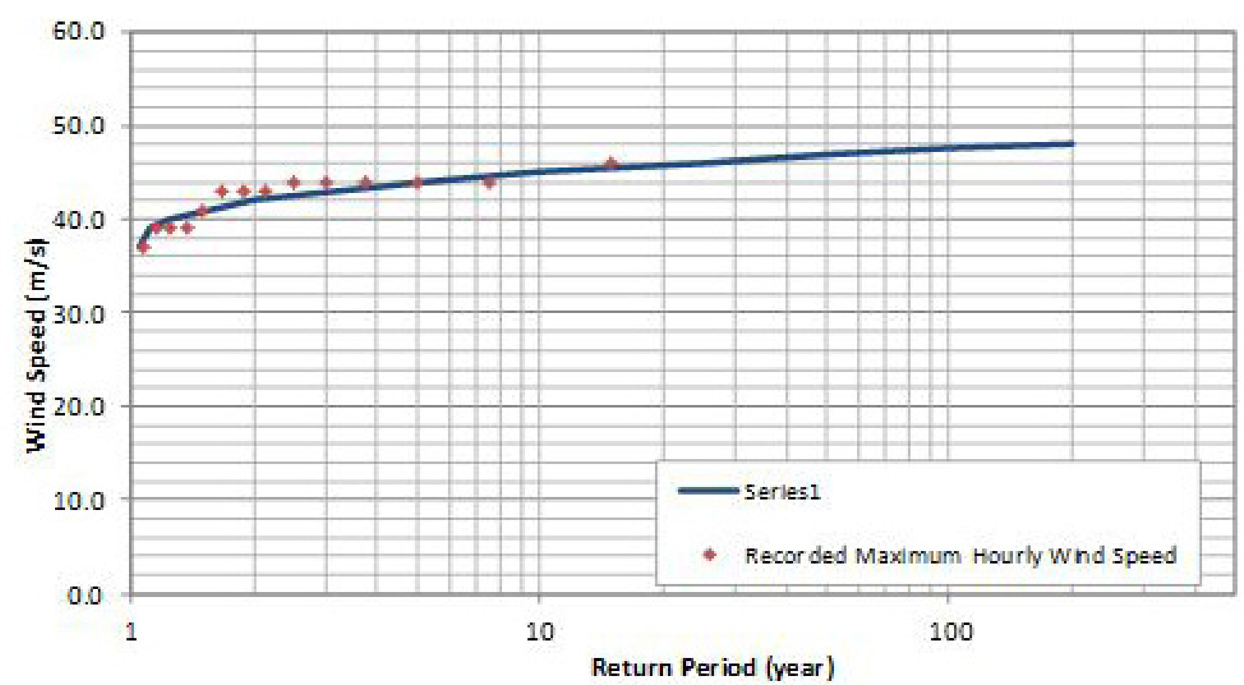


Figure 12. Wind Speed and Return Periods at Climate Station Vancouver Harbour CS

3.4.2 WINDS AT VANCOUVER INTERNATIONAL AIRPORT

Wind at Vancouver International Airport (YVR) (station number 1108447, 49° 11' 42.0" N / 123° 10' 55.0" W), approximately 14.5 km southwest from the project site, was assessed previously by SNC. A recent review of the wind data (conducted by SNCL, John Readshaw, personal communication, October 2012) at YVR and Vancouver Harbour (VH) showed the following conclusions:

- Winds during storm surges come predominantly from the Southeast and Southwest.
- An upper bound wind speed for the 1/200 year AEP event based on the wind record from YVR is 58 km/h (16 m/s).
- Wind records from the YVR anemometer are typically higher than those recorded at the VH anemometer by about 30%. Although slightly high, the YVR anemometer is a reasonable model for the over-water wind field within Vancouver Harbour.

3.5 WAVE ANALYSIS

BC MoE Guidelines [25] states that the crest elevation of a sea dike shall be established to provide the protection during designated storm as follows:

- DFL + Wave Run-up + Freeboard, or,
- DFL + Acceptable Crest Height

Similar to the second definition above, the crest elevation will be determined by the following definitions:

- DFL + Wave Effect Allowance; and,
- A 200 year return period for waves

The Wave Effect Allowance will be used in this study to define the crest elevation of a sea dike, and will be determined by an acceptable overtopping discharge under a 200 year return period waves.

3.5.1 WIND-WAVE HINDCAST

Representative two-dimensional sections *(one-dimensional wave model in oceanographic analysis)* are used as a basis to estimate the wave conditions at the site. The wave height and period were computed using the Automated Coastal Engineering System (ACES) interfaced within the Coastal Engineering Design and Analysis System (CEDAS) software package (CEDAS 4.03, 2006). This model uses a deep water wave growth approach to determine wave height and period for a given wind speed and fetch. The straight wind fetch, is approximately 3.2 km from Southwest and 4.3 km from Southeast as shown in Figure 10.

Predicted wind waves are summarized in Table 12 for the two scenarios, 200-year wind from Vancouver Harbour and 200-year wind from Vancouver International Airport, within an upper bound fetch of 4.3 km in the southeast direction.

Wave height and period were computed for the above winds for a SE fetch using the Automated Coastal Engineering System (ACES) interfaced within the Coastal Engineering Design and Analysis System software package (CEDAS 4.03, 2006). This model uses a deep water wave growth approach to determine wave height and period for a given wind speed and fetch. The results are summarized in Table 12.

Table 12. Wind Wave Prediction at the Concert Project Site, Using a 1-D Wave Model [A]

Scenario	Wind Field			Wave Hindcast	
	Speed	Direction	Fetch	Sign. Wave Height	Peak Wave Period
	(km/h)	(clockwise from North)	(km)	(m)	(sec)
200-Year Wind form Vancouver Harbour (VH)	48	SE	4.3	0.42	2.33
200-Year Wind from Vancouver International Airport (YVR)	58	SE	4.3	0.6	2.7

Note:
A. CEDAS, developed by U.S. Army Corps of Engineers

3.5.2 WAVE RUN-UP AND WAVE EFFECT ALLOWANCE

When wave run-up on a sea dike is higher than the designed freeboard, the wave overtops the crest of the sea dike. Overtopping discharge can cause damage to seawalls, infrastructure, or danger to pedestrians and vehicles. In this study, wave run-up analysis is based on an assumption of minimal overtopping. The wave run-up, R, is often defined and calculated as R2%, the vertical distance above mean water level exceeded by no more than 2% of the waves at the toe of the slope or structure.
For purposes of estimating Wave Effects, the wind speed with 200 year Return Periodfrom the Vancouver International Airport (YVR) long term record was used (see Table 12 above).

The analysis applies four different methods for estimation of the run-up R2% for testing sensitivity analysis: CEDAS-ACES; Coastal Engineering Manual (CEM, 2006); Hughes (2005); and TAW (2002). The TAW approach is consistent with the Eurotop (2008) and is considered a conservative upper limit estimate for run-up. Results among the different methods were found to be broadly consistent depending primarily on the assumption of roughness in the CEDAS and TAW approaches. The CEM and Hughes methods implicitly assume a rough impermeable slope. The Hughes method was subsequently adopted as a conservative approach based on this analysis. The parameters are defined by the sketch in Figure 13a for the 2020 planning horizon and in Figure 13b for the 2100 planning horizon.

The conditions for estimation of the R2% and parameter definitions are summarized in Table 13.

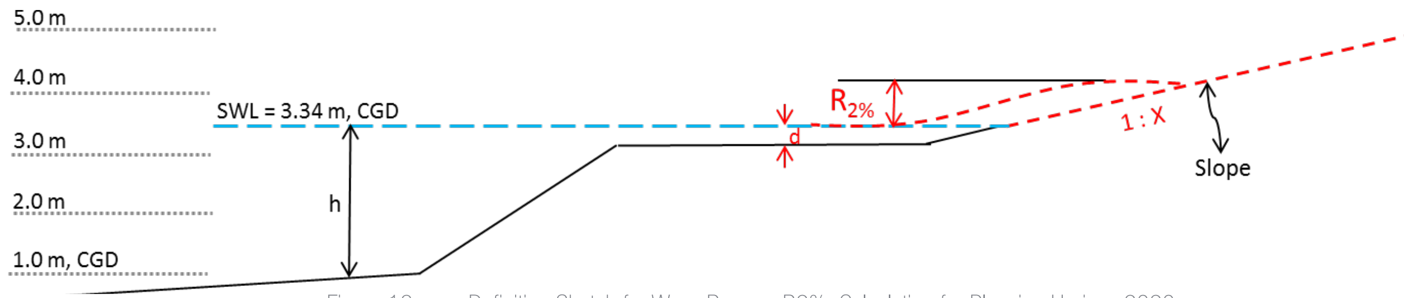


Figure 13. a Definition Sketch for Wave Run-up, R2%, Calculation for Planning Horizon 2020

Figure 13. b Definition Sketch for Wave Run-up, R2%, Calculation for Planning Horizon 2100

R2% estimates were calculated for several design concept cross sections and for the two planning horizons identified in Table 1 (years 2020 and 2100). The cross sections are coordinated with site plan concepts developed by PWL Partnership and Golder Associates in October 2012 (Alternative A) and September 2013 (Alternative B). Additional detail on elevations and slopes was added by Golder and the cross sections that are the basis of the wave effects assessment are illustrated here. Changes to these cross sections would result in changes to wave effect allowances.

Because the cross sections are complex terraced profiles with multiple slopes, R2% estimates were calculated for between two and three different representative slopes across each profile to identify a range of potential R2% values anticipated at each cross section depending on where along the cross section a given wave may break. The slopes used to calculate R2% are shown as red dashed lines in the cross sections shown.

The amount of wave effect is directly related to the parameters listed in Table 13. Changes in any of these parameters will change the resulting wave effect allowance. Changes to shoreline (planform) shaping, in particular, would change wave effects. Readers are therefore cautioned that allowances for wave effects are based on the cross sections shown herein, and changes to proposed cross section would necessitate recalculation of the wave effects, and potentially result in different (or higher) Flood Construction Elevations. A more thorough analysis of wave energy dissipation and run-up is required to refine the concept designs at a later stage in the project (e.g. development permit stage for each phase).

Recommended wave effect allowances are generalized based on the 'worst case cross section' to 0.6 m for Alternative A cross sections, and 1.2 m for Alternative B cross sections. In each case, these wave effect allowances represent the highest calculated wave effect from a review of several typical cross sections along the waterfront. The 1.2 m allowance for wave effects in Alternative B is based on the eastern (seawall) section of the waterfront and may be reduced by modified waterfront design detail and further wave analysis.'

Table 13. Assumptions for Prediction of Wave Run-up R2%, Using Hughes Method

Parameter	Value (Planning Horizon 2020)	Value (Planning Horizon 2100)	Unit
Design Sea Water Level: Designated Flood Level (DFL)	3.34	4.04	m, CGD
Design Wind: 200-Year Wind from Vancouver International Airport (YVR)	16	16	m/s
Design Wave: Significant Wave Height, H_s	0.6	0.6	m
Design Wave: Peak Wave Period, T_p	2.7	2.7	s
Water depth at the toe of primary slope, h	3.34	4.04	m
Water depth at the toe of secondary slope, *d*	0.2 to 3.3	0.6 to 4.0	m
Representative Slope, % Grade	2 to 55	2 to 55	%
Representative Slope, *X* 1V:*X*H	1.8 to 54.5	1.8 to 54.5	-
Roughness	rough surface	rough surface	-
Permeability	impermeable	impermeable	-

Note: A. Steven A. Hughes: Estimating Irregular Wave Runup on Rough, Impermeable Slopes, US Army Corps of Engineers, ERDC/CHL CHETN-III-70, July 2005

3.6 FREEBOARD ALLOWANCE

A freeboard is the vertical distance added to the FCRP to establish the FCL. A freeboard allowance of 0.6 m is recommended in BC Guidelines to cover uncertainties due to relative SLR and storm surges. [23] The uncertainties can be due to climate change (for example, the rate of SLR in correlation with climate change); or they can be induced by site-specific conditions (for example, surface and sub-surface conditions that may result in variable coastal erosion, sedimentation or subsidence). 0.6 m is the recommended freeboard as a precautionary approach for our analysis.

3.7 RECOMMENDED ALTERNATIVE FCL AT PLANNING HORIZONS 2020 AND 2100

Two Alternative sets of waterfront cross sections were analyzed: Alternative A in November 2012, and Alternative B in September 2013. Based on a site-specific analysis of tides, storm surges, winds and waves, combined with the site cross sections analyzed, the recommended Flood Construction Levels relative to Canadian Geodetic Datum (CGD) for the Concert Harbourside Development are summarized in Table 14.

Table 14. Estimated Future Water Level Elevations at the Concert Harbourside Site

Parameters	Alternative A			Alternative B		
	Year 2020	Year 2070	Year 2100	Year 2020	Year 2070	Year 2100
Flood Construction Level (FCL, m CGD) [1]	4.54	4.98	5.24	5.14	5.58	5.84
Freeboard (m)	0.6	0.6	0.6	0.6	0.6	0.6
Flood Construction Reference Plane (FCRP, m CGD)	3.94	4.38	4.64	4.54	4.98	5.24
Wave Effect Allowance (m) [2]	0.6	0.6	0.6	1.2	1.2	1.2
Designated Flood Level (DFL, m CGD)	3.34	3.78	4.04	3.34	3.78	4.04
Regional Sea Level Rise (SLR, m) [3]	0.19	0.63	0.89	0.19	0.63	0.89
Storm Surge Annual Exceedance Probability (AEP) 1/200 (m) [4]	1.25	1.25	1.25	1.25	1.25	1.25
High Tide (HHWLT, m CGD) [5]	1.9	1.9	1.9	1.9	1.9	1.9

Note:

1. Considering the effect of slr, regional crustal movement, tide, storm surge, wind and wave.

2. The allowance will require consideration of wave overtopping in the drainage system design.

3. Estimated according to the recommended global slr curve for planning and design in bc (bc ministry of environment - climate change adaptation guidelines for sea dikes and coastal flood hazard land use, January 2011) and the regional crustal movement of an uplift rate 1.2 mm/yf from 2010.

4. BCMOE guideline: climate change adaptation guidelines for sea dikes and coastal flood hazard land use, January 2011

5. Defined in Canadian hydrographic services (chS) - Canadian tide and current table - volume 5, 2011

3.7.1 LIMITATIONS TO THE FCL

- Wave analysis is based on a one-dimensional model *(corresponding to section drawing in design disciplines)* that provides coarse results of wave run-up for two typical slope profiles – a perched beach concept and a salt marsh concept. Those concepts are illustrated and further detailed in Section 4. To better understand the site hydrodynamics reflecting the hydrodynamic interactions of tide, storm surge, wind and waves with site specific topographic and bathymetric feature details, a two-dimensional model *(corresponding to 3-D model in design disciplines)* is required.
- Wave run-up is for the south shoreline dikes only. Any potential flood along the west or east shoreline (Mackay and Mosquito Creeks) has not been taken into account.

However, as identified through literature review and water level analysis in Section 3.3.2, there exist differences between storm surges at Point Atkinson and at Tofino. It should be noted that in the current BC MoE Guidelines, the storm surge value at Point Atkinson was defined referring to that at Tofino, which might have been overestimated.

4.0 MANAGEMENT STRATEGIES

4.1 SHORE LINE PROTECTION STRATEGIES

4.1.1 CHARACTERIZATION

For planning and design purposes, the site has been broken into three zones representing key coastal characteristics: the shore (West of Fell), the sea wall (East of Fell), and the transition in between the two (at the foot of Fell). A shore line composed of two major terraces, a lower and upper terrace, has been the base of the design assumptions for the wave-model analysis. This section refines the proposed scenario to further guide the design of Harbourside Waterfront.

The management strategies aim to enhance the natural and existing features of the site, while providing design adaptation to account for the estimated SLR to 2100. In the design concepts, the 2100 FCL is referenced to help illustrate the site's resilience within the planning horizons analyzed in this plan.

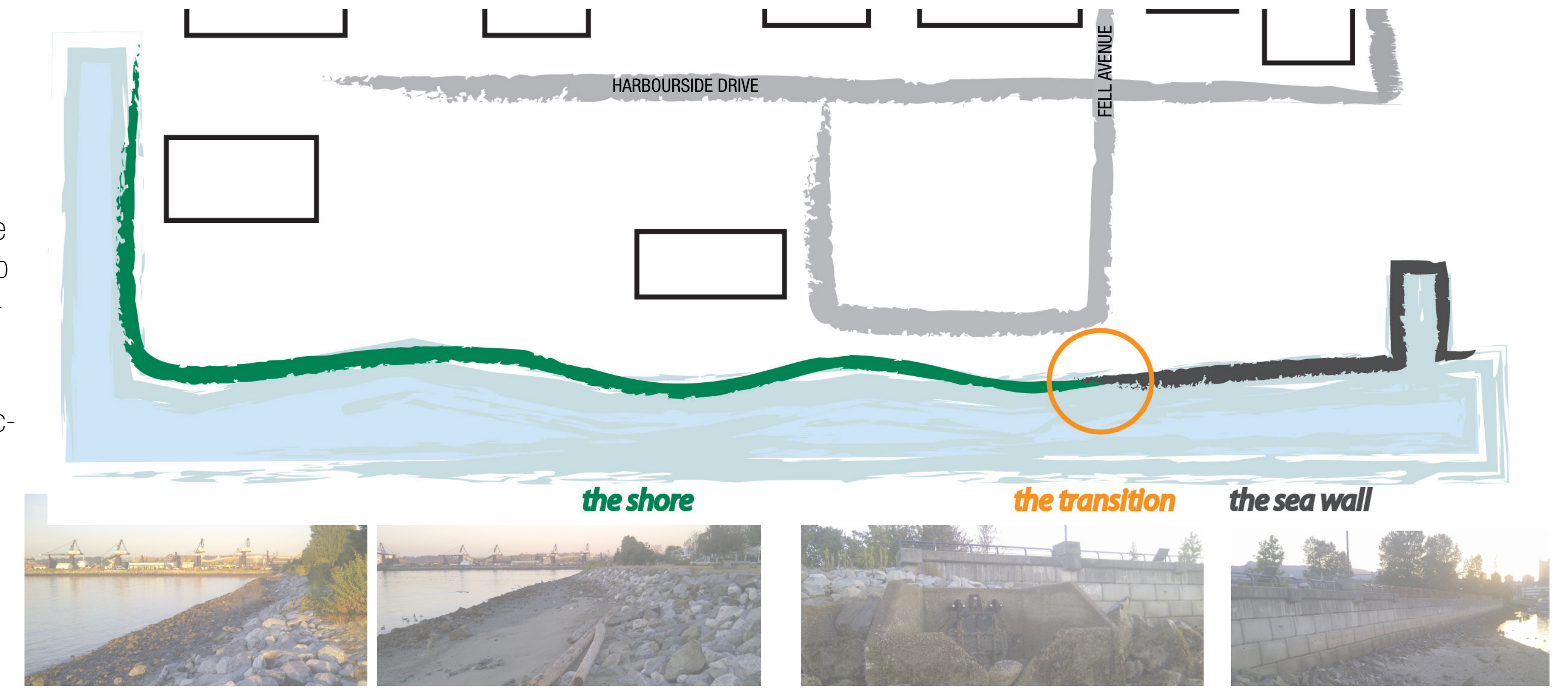


Figure 14. Characterization of the existing site conditions

4.1.2 BASIC DESIGN CONCEPT

In addition to site terracing and slopes in the building setback, a continuous concrete band (flood wall) throughout the site would act as a final dike / upper line of defense for the buildings and roads. The crest would be a protection measure that could be adapted for future planning horizons. The concrete band should be constructed with a consistent height, and any gaps at walkway crossings will need to be fitted with flood gates that would be closed at notice of a large storm surge event. At later stages of building life, if not replaced by new construction, the floodwall can be adapted by elevating the height of the concrete band into a higher retaining wall. The plan below shows a schematic outline of the proposed concrete band flood wall (shown in red on Figure 15). Its exact location should be integrated with the final design. In all cases, the concrete band should be on the landward and upper side of the slopes and terraces shown on the typical cross sections in this document. To avoid floods flanking around the west, east and north sides of the subject properties, similar terraces and floodwall would need to be extended up Mosquito or MacKay Creeks by others in the future. Elevations of flood protection along the creeks would need to consider the interface of creek hydrology with marine storm surge.

The design concepts involve terracing to transition between the existing conditions and elevated building sites and reducing run-up and a lower wave effect allowance than would be accomplished with a conventional rip-rap slope or vertical seawall. Thus, a certain amount of open space can be subject to flooding in rare circumstances. Parks and open space are recommended uses for areas at increased risk of flooding in the BC MoE Guidelines. The most important park infrastructure is the greenway (Spirit Trail) which should be situated on one of the higher terraces. As part of the broader amenity package, the owners are proposing to complete the Spirit trail seaward of their sites.

A drainage system is required to account for wave overtopping over the proposed protection regime. This drainage system would follow the concrete band. Its form should be adapted to the design; it can be designed as a bioswale or a trench drain with grate, and be sufficient in scale to accommodate overtopping within the accepted normal ranges for the location.

TWO ALTERNATIVES

The pages that follow illustrate two alternatives for treatment of the waterfront at Concert Harbourside.

Alternative A was an original concept, developed in November 2012, with a general upper terrace in the range of 5.0 m geodetic, and a significant headland at the foot of Fell Ave. In concept, Alternative A would construct the new development at levels in anticipation of a conservative FCL estimated for the Year 2100.

Alternative B was developed in the summer of 2013, and investigates a general upper terrace elevation of 4.5 m geodetic, with less headland volume at the foot of Fell Ave. Alternative B assumes that adaptation to SLR would be phased. The initial construction would protect for today's risks or better, and the design would allow for gradual additional construction or other adaptation to protect against SLR as it evolves over the decades ahead.

The concepts of shoreline terraces and slopes, backed up by an upper concrete and (floodwall), are common to all alternatives. The details of elevations of the elements vary among the alternatives, and these have an influence on wave effect allowances.
The intention of Alternative B is to construct to the elevations shown on the cross sections, with allowances to increase the height of the concrete band at points in the future when additional freeboard may be needed. As shown, the initial top of the concrete band floodwall would be 5.24 m CGD, which corresponds to the estimated Year 2100 Flood Construction Reference Plane (FCRP – see Table 14). Additional construction to add height to the wall would be necessary in latter parts of the century to add freeboard between the FCRP and the FCL.
Table A1 and A2 in the Appendices show a range of wave run-up calculations for each Alternative B cross section reviewed.

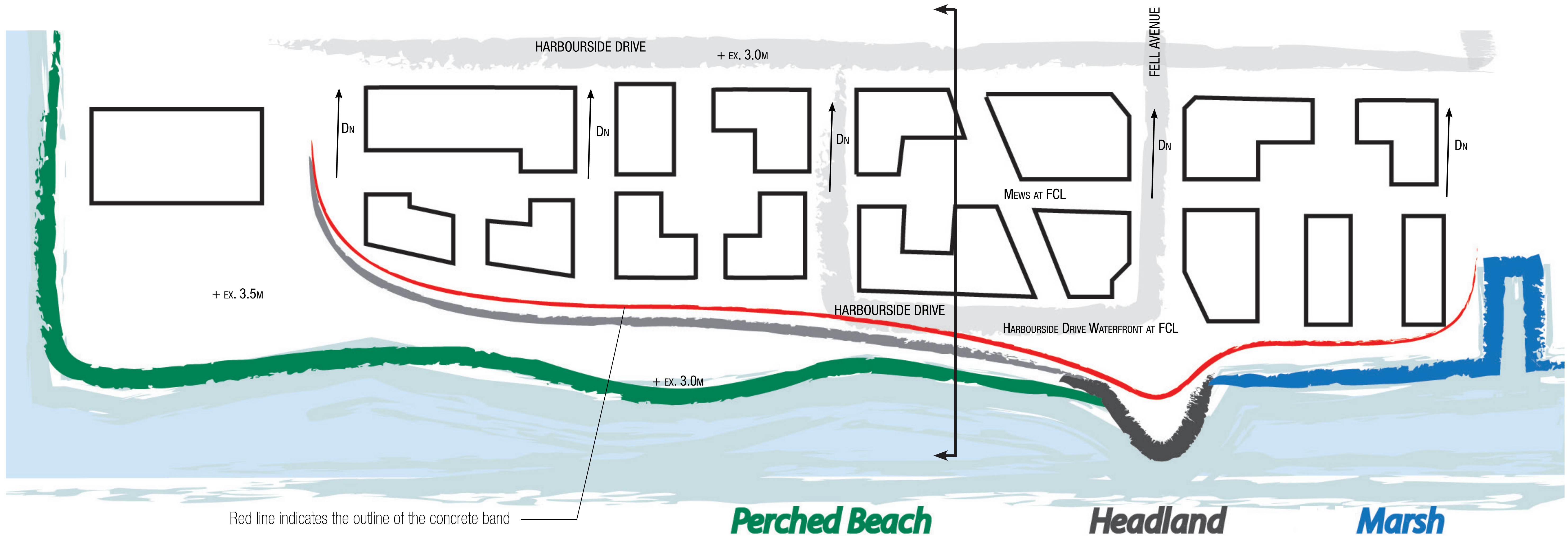


Figure 15. Overall Design Concept Plan (Note: these drawings may not reflect the most up-to-date building foot prints and site designs. Building setbacks and various site elevations are, nevertheless, sufficiently accurate to maintain the integrity of FCLs.)

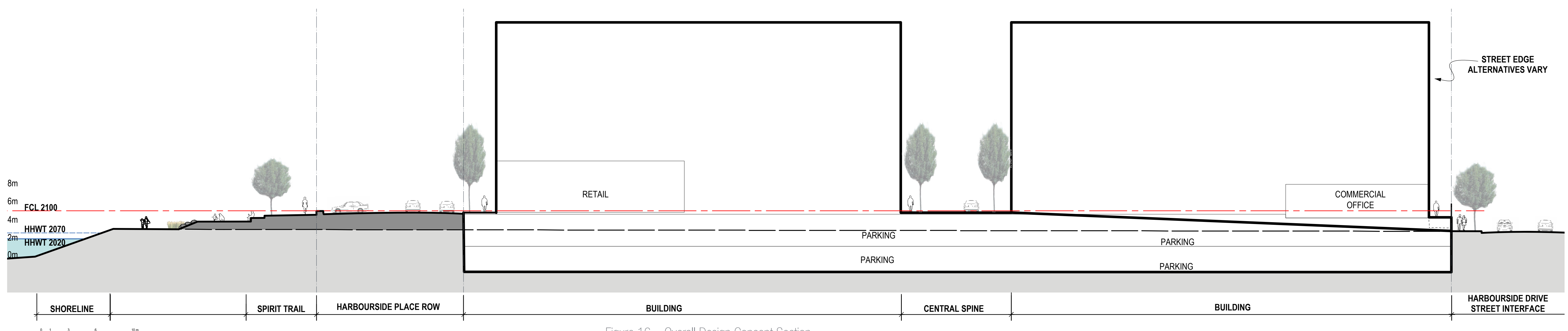
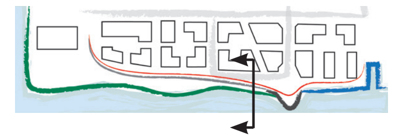


Figure 16. Overall Design Concept Section

4.1.3 THE SHORE



DESIGN CONCEPT: PERCHED BEACHES

The design concept of the perched beach has many beneficial aspects for the site. A perched beach is a terraced, or stepped, profile that provides a more gradual transition from the elevated FCL to the existing shore. It facilitates beach access, and also provides effective dissipation of incident wave energy, including run-up reduction. Perched terraces retain beach/park functions at lower/higher elevations respectively and can be used at high and low tide, depending on the time of day. Over time, perched beaches can become valuable tidal environment. The width of the perched beaches can vary to suit site conditions and overall design.

Aesthetics, environmental quality, and erosion resistance can be further improved by incorporating erosion resistant vegetation in the design of the shore protection. Care should be brought to select adaptive and native species in the vegetation beds. The perched beach concept can add habitat value to the existing shore line.

The design provides an option for along shore transition from low sloping beach to near vertical wall, in this case, adjacent to an artificial headland. It will also facilitate transition with the adjacent retained park conditions (Dog Park).

ALTERNATIVE A

Alternative A1 at the Harbourside Place parking, shows the proposed elevated parking area, with slopes and terraces to the waterfront with an average grade of 4.3:1 below the proposed Spirit Trail. The proposed concrete band at elevation 5.24 m CGD is located between the Spirit Trail and elevated parking at 5.0 m CGD at Harbourside Place. An estimated wave effect allowance of 0.6 m based on this section would lead to a Year 2100 FCL of 5.24 m at this location.

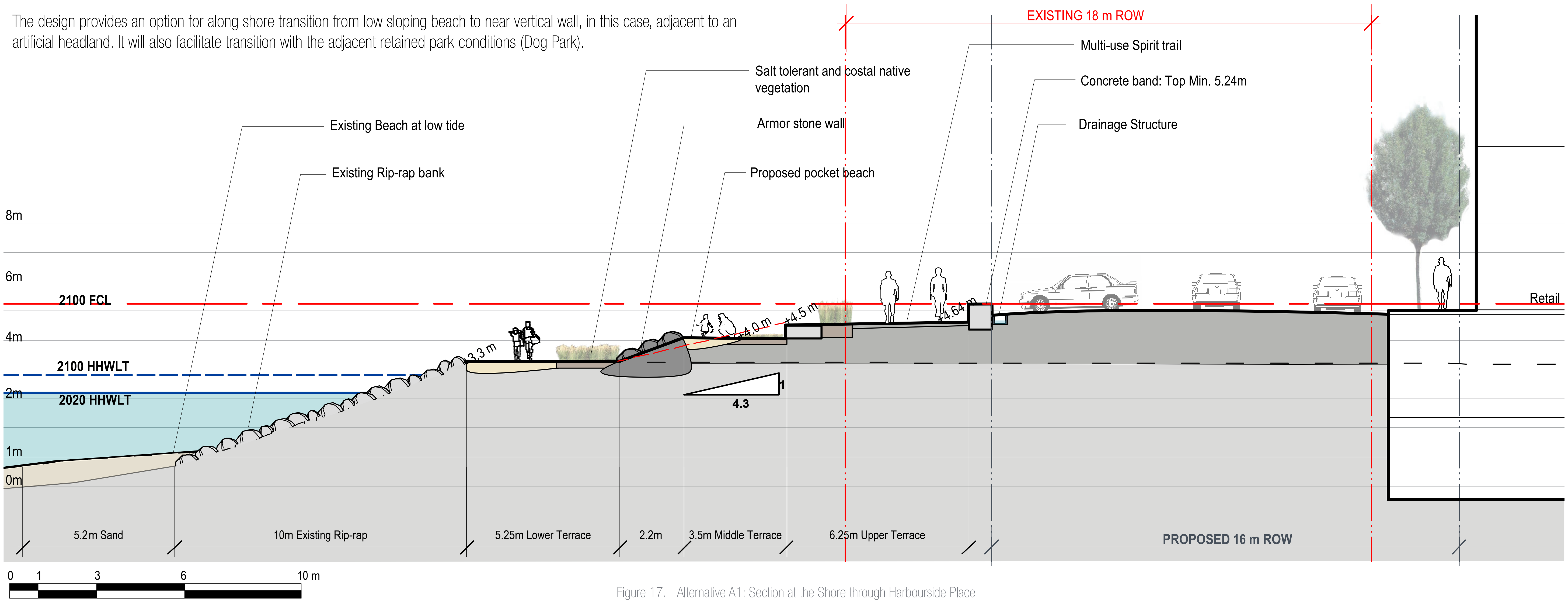
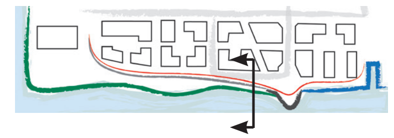


Figure 17. Alternative A1: Section at the Shore through Harbourside Place

ALTERNATIVE B



Alternative B1 is similar in concept to Alternative A1. Differences in slopes and terrace width lead to an estimated wave effect allowance of 0.7 m (see Table A2), and a Year 2100 FCL of 5.34 m based on this section.

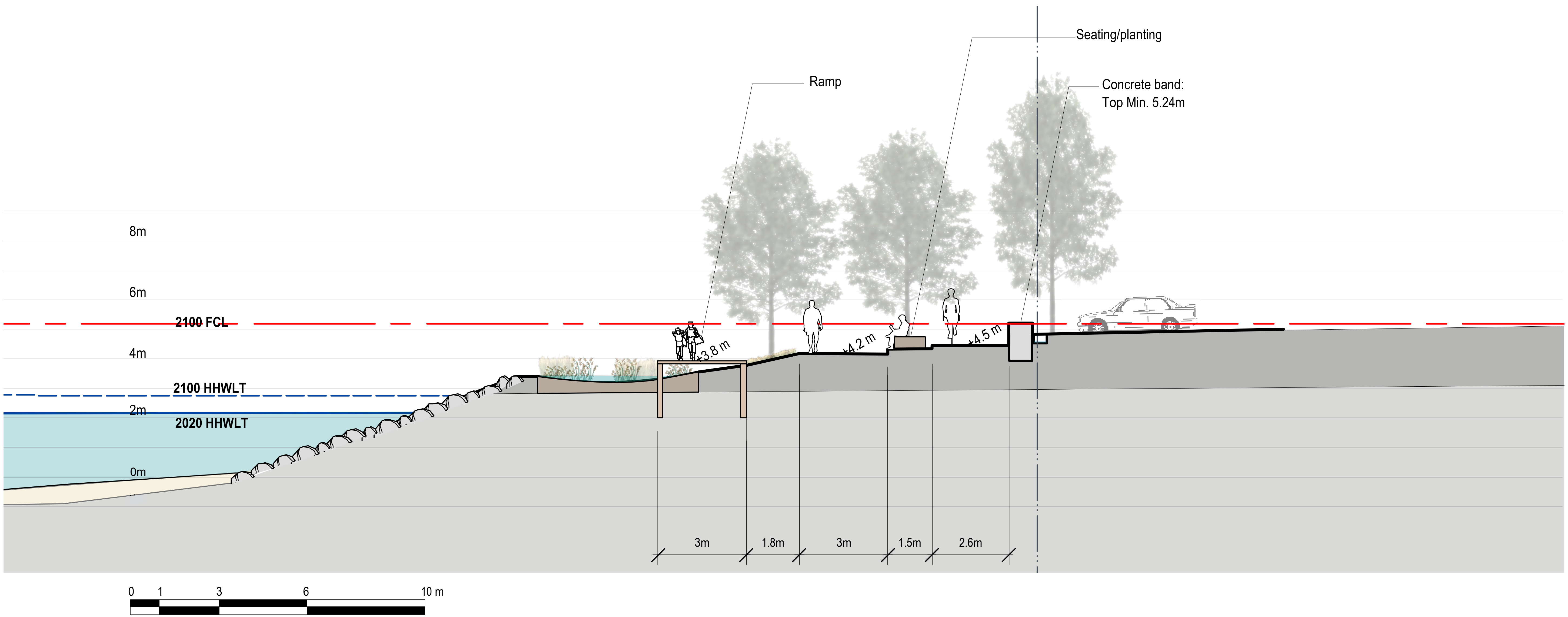
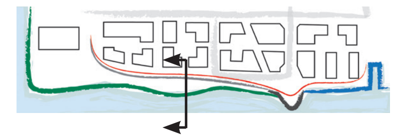


Figure 18.   Alternative B1: Section at the Shore through Harbourside Place

ALTERNATIVE B



In cross section B1a, the section is shown at the existing Dog Park, and reflects the desire to leave the this area largely in its existing condition and elevation. The Spirit Trail is shown elevated in the 4.3 to 4.5 m range, with the concrete band at 5.24 m above the trail. An estimated wave effect allowance of 0.8 m (see Table A2) based on this section would lead to a Year 2100 FCL of 5.446 at this location, which could be achieved by elevating the floodwall in the future.

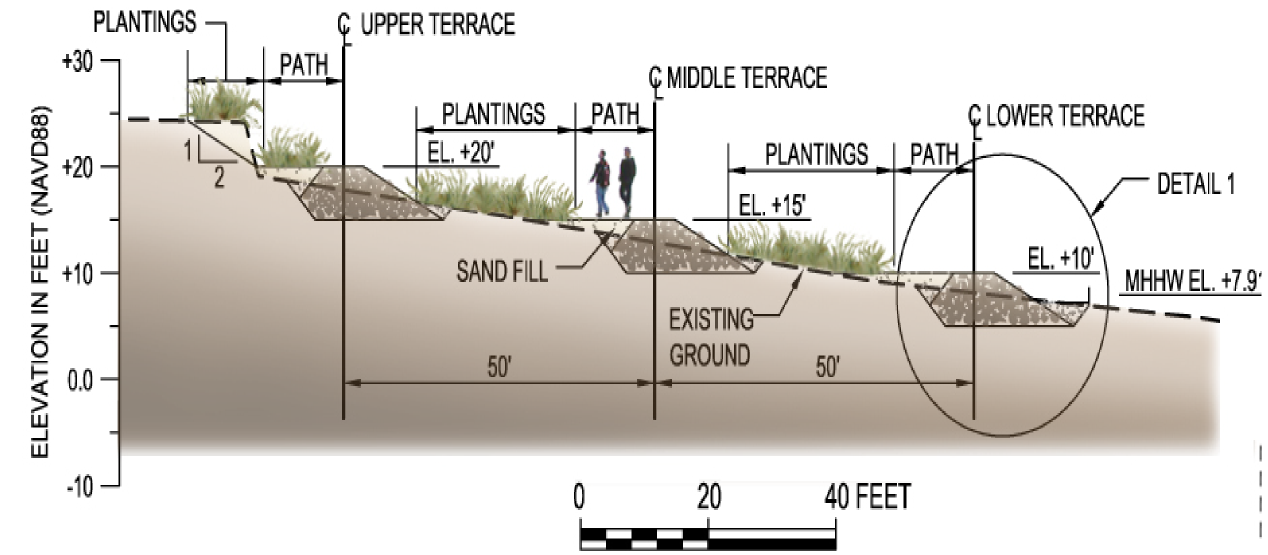


Figure 19. Study for Beach Half-Moon Bay, Westport, WA

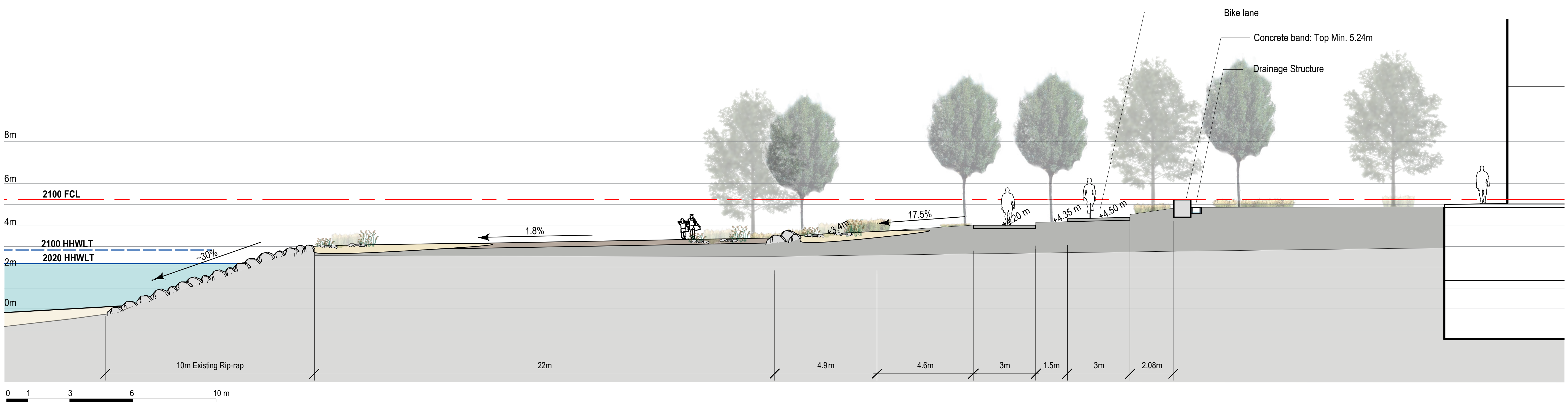
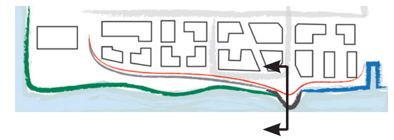


Figure 20 Alternative B1a: Section at the Shore through Harbourside Place

4.1.4 THE TRANSITION



DESIGN CONCEPT: ARTIFICIAL HEADLAND

An artificial headland is a hard point to focus and dissipate wave energy. It can be made out of concrete rubble, and sculptural and coloured shot-crete and with a more porous material, rip-rap. A combination of both would allow structural integrity, environmental and aesthetic value as a replacement of the existing lock-block wall and ending of stormwater outfall. The headland can be designed to include intertidal pools and display daily tidal fluctuation, and as long as it is designed to adapt to the SLR through the years.

The implementation of a headland provides a transition from a more organic shoreline (perched beach) to a steeper condition (rip-rap bank). It is also an alongshore sediment transport control point. It reduces wave energy and run-up on adjacent beach and seawall.

It can also serve as a conduit for surface water out-fall (flood water management). Stormwater run-off can be directed, collected and treated in a feature rain garden, before being transferred to the existing storm out-fall. Tidal gates or other backflow prevention structures can be incorporated into recesses in the headland

Note: More detailed/advanced design and, in turn, more robust modeling would be needed to conduct wave analysis of the headland. Wave effect analysis of cross sections West and East of the headland were considered sufficient at this stage in planning.

ALTERNATIVE A

Alternative A2 at the foot of Fell Ave shows a proposed extension of the existing small headland, with small stepping terraces down the slope. The existing storm sewer outfall would be extended. The proposed concrete band at elevation 5.24 m CGD is located between the Spirit Trail and elevated parking at 5.0 m CGD at Harbourside Place. An estimated wave effect allowance of 0.6 m based on this section would lead to a Year 2100 FCL of 5.24 at this location.

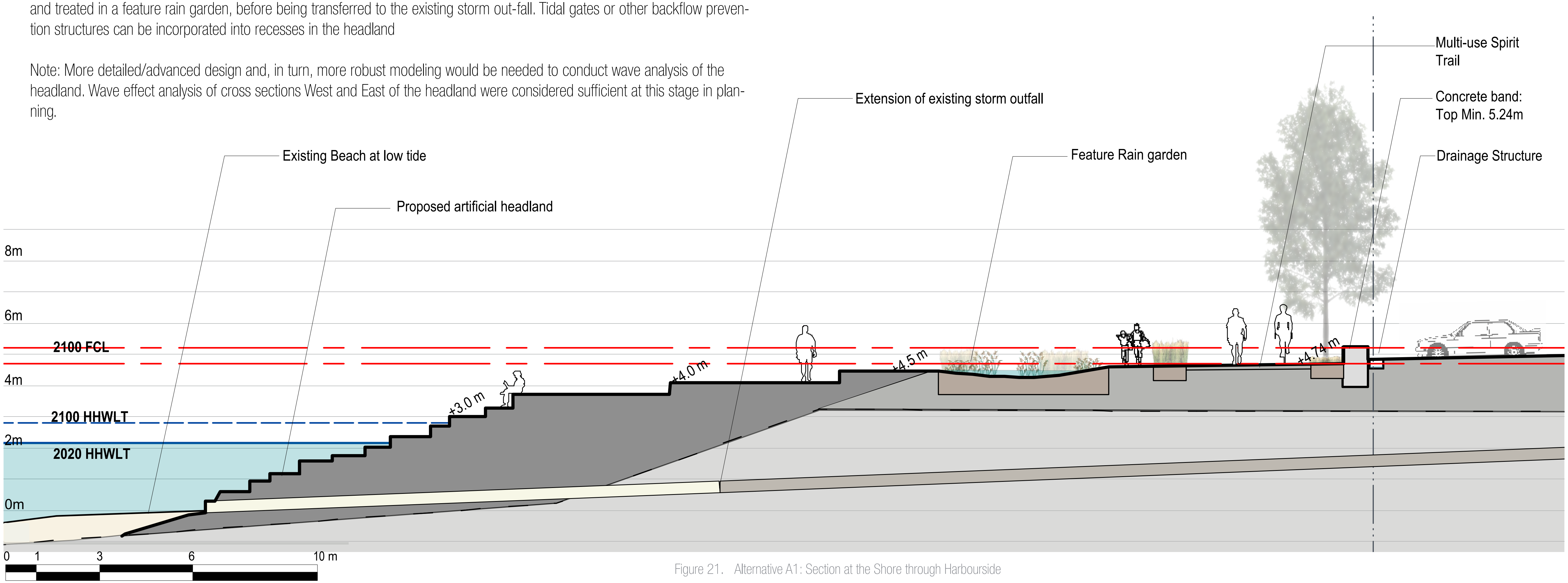
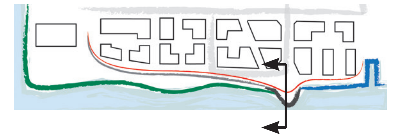


Figure 21. Alternative A1: Section at the Shore through Harbourside



ALTERNATIVE B

Alternative B2 reduces the proposed headland to be closer to the existing condition. A proposed riprap bank is topped by a small outlook pier. The Spirit Trail would have the concrete band (floodwall) at its upper side (not shown at this section line which is at an opening), to protect a plaza area at 4.5 m CGD. The estimated wave effect allowance for this cross-section is 0.7 m (see Table A2) resulting in a FCL of 5.34 m for the year 2100 based on this section. The latter could be achieved by elevating the floodwall in the future.

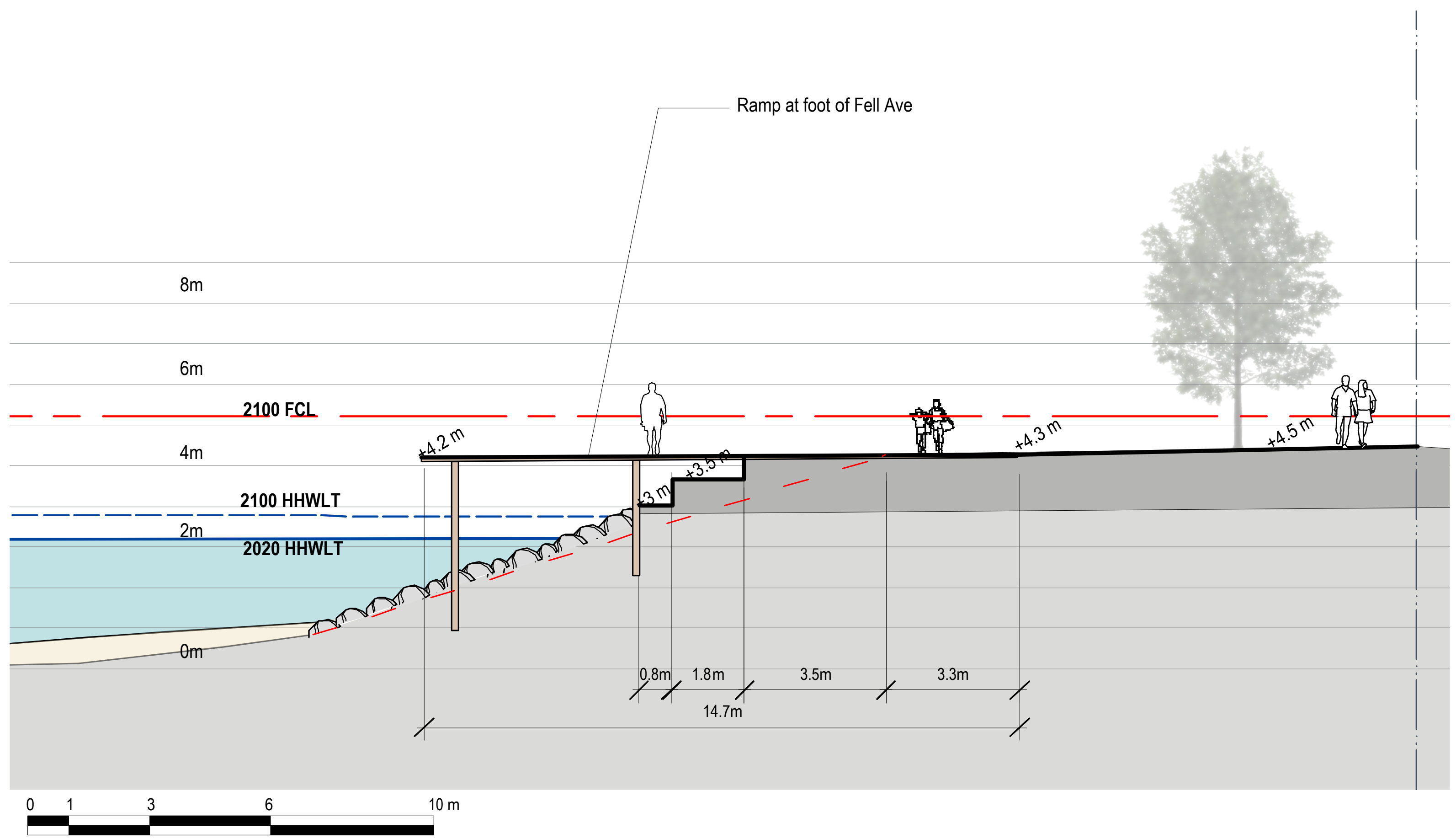
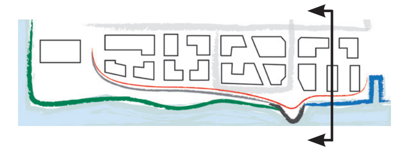


Figure 22. Alternative B2: Section at the ramp at end of Fell Avenue

4.1.5 THE SEA WALL



DESIGN CONCEPT: MARSH

On the Eastern portion of the site, a new rip-rap bank placed in front of the existing lock-block wall would help dissipate wave energy and provide habitat value to the existing shore line. Reducing the height of the existing wall will allow the creation of a marsh and display the influence of the tide on the natural environment.

The design concept of the perched beach can be integrated into the design. It can be adapted to a more urban environment with the use of large cut stone slabs.

An optional grade change above the FCL provides separation with the residential units and the public realm.

ALTERNATIVE A

Alternative A3 removes part of the existing lock-block seawall down to elevation 2.4 m CGD, which is at the upper level of current recorded high water in storm events. Short vertical stone steps would take up grade to an elevated Spirit Trail. The concrete band with top of 5.24 m provides separation and protection to semi-private landscape areas and buildings to the north. An estimated wave effect allowance of 0.6 m based on this section would lead to a Year 2100 FCL of 5.24 m at this location.

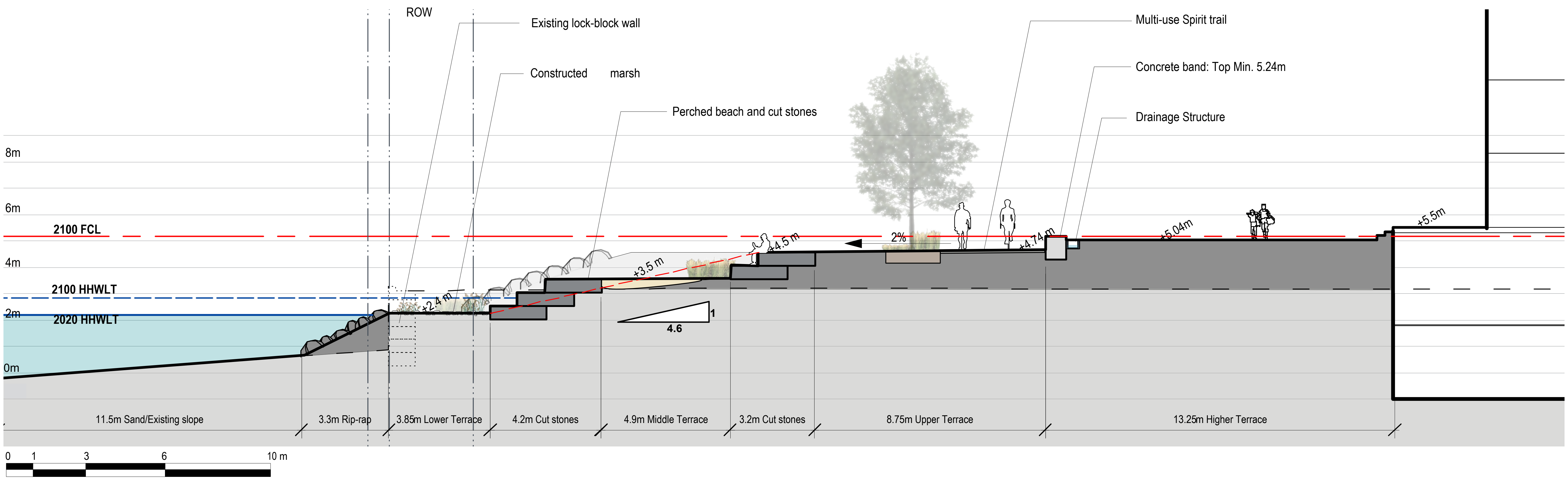


Figure 23. Alternative A3: Section at the march

ALTERNATIVE B

Alternative B3 increases the height of the lockblock wall to 3.07m CGD. Below the lockblock wall, proposed riprap is shown at a steep slope of 55%. A wetland behind this wall may have a base at 2.64 m. Stepped blocks and a perched beach rise to an elevated Spirit Trail. The concrete band with top elevation of 5.24 m is continuous on the north edge of the public property. This cross section creates an estimated wave effect allowance of 1.2 m (see Table A2) and a Year 2100 FCL of 5.84 m based on this section, which could be achieved by elevating the floodwall in the future. This section has the highest wave effects of those considered, and consideration should be given to reducing wave effects at the detail design stage, eg. by reducing further the height of the lock block wall, and reducing the steepness of the proposed shoreline riprap.

Kohimarama Beach, Auckland NZ & Evergreen Park Pocket

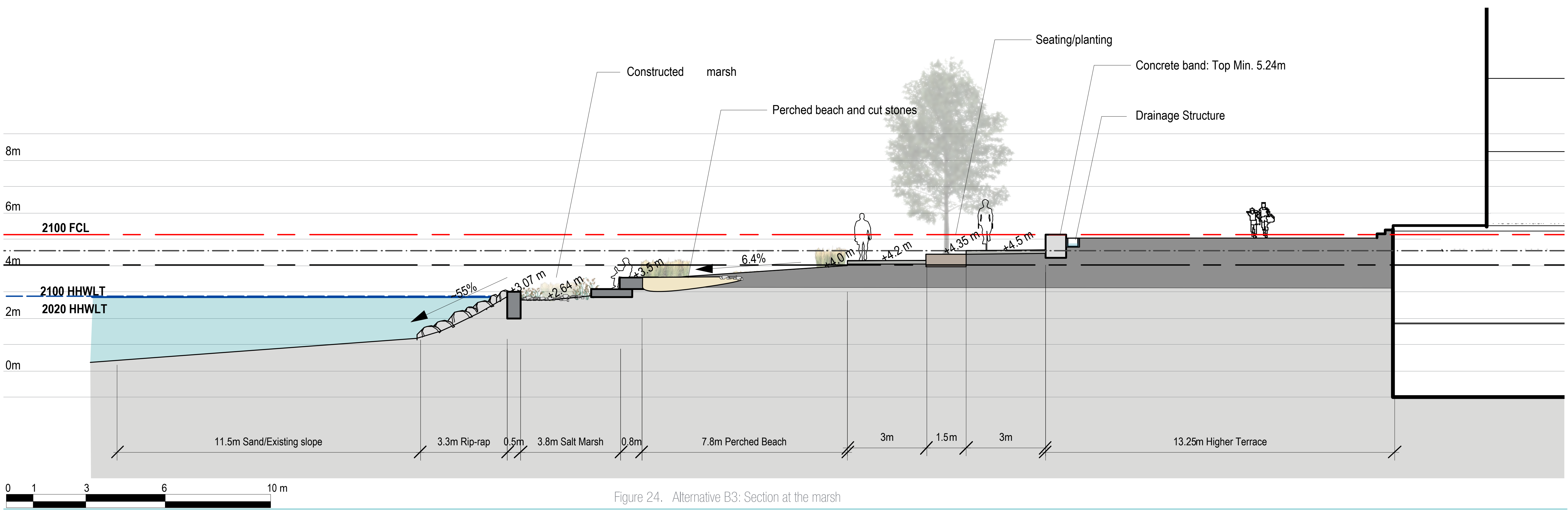


Figure 24. Alternative B3: Section at the marsh

4.1.6 HARBOURSIDE DRIVE AND SIDEWALK INTERFACE

TWO ALTERNATIVES AT HARBOURSIDE DRIVE

Two Alternatives were also considered for the south edge of Harbourside Drive, where the existing road will interface with the proposed buildings.

Both alternatives consider an initial floor elevation in the office/commercial uses along this frontage that is slightly lower than the level of residential and other uses on the waterfront side of the development. This is for three reasons:

- The need to avoid excessively high grade differentials between the existing road and proposed building floor levels,
- The lowered risk of wave effects on this sheltered side, and
- The lower risk to life associated with flooding of non-sleeping quarters.

Alternative A: A duplicate elevated sidewalk would provide access to commercial office. The sidewalk would be situated at shoulder height, at elevation 4.5 m CGD. The FFE would be at 4.5 m CGD.

Alternative B: The duplicate elevated sidewalk is eliminated, and roadside grades are feathered up with grades up to 5% to provide an initial floor elevation of 3.5 m, which approximates the elevation of most existing buildings in Harbourside. The ceiling height of these offices would allow the floor to be raised in future decades, eventually as high as 4.50 m CGD.

The general intent of all Alternatives is to accommodate future adjustments by others to the elevation of Harbourside Drive and neighbouring roads and adjacent properties to reduce vulnerability to flooding over the decades ahead.

At the same time, Alternative B avoids significant issues:

- Urban design and aesthetic impairment that would be presented by shoulder height walls along Harbourside Drive,
- Related accessibility and ramping challenges for persons with reduced mobility,
- Potential impacts on business viability along this frontage,
- Significant capital cost issues, and related effect on funds available for competing objectives.

By delaying investment in higher floor elevations until a future date, Alternative B provide adaptability to a wider range of future SLR solutions that may come forward for Vancouver Harbour or the Harbourside Neighbourhood, and is recommended as an initial phase towards future adaptability.

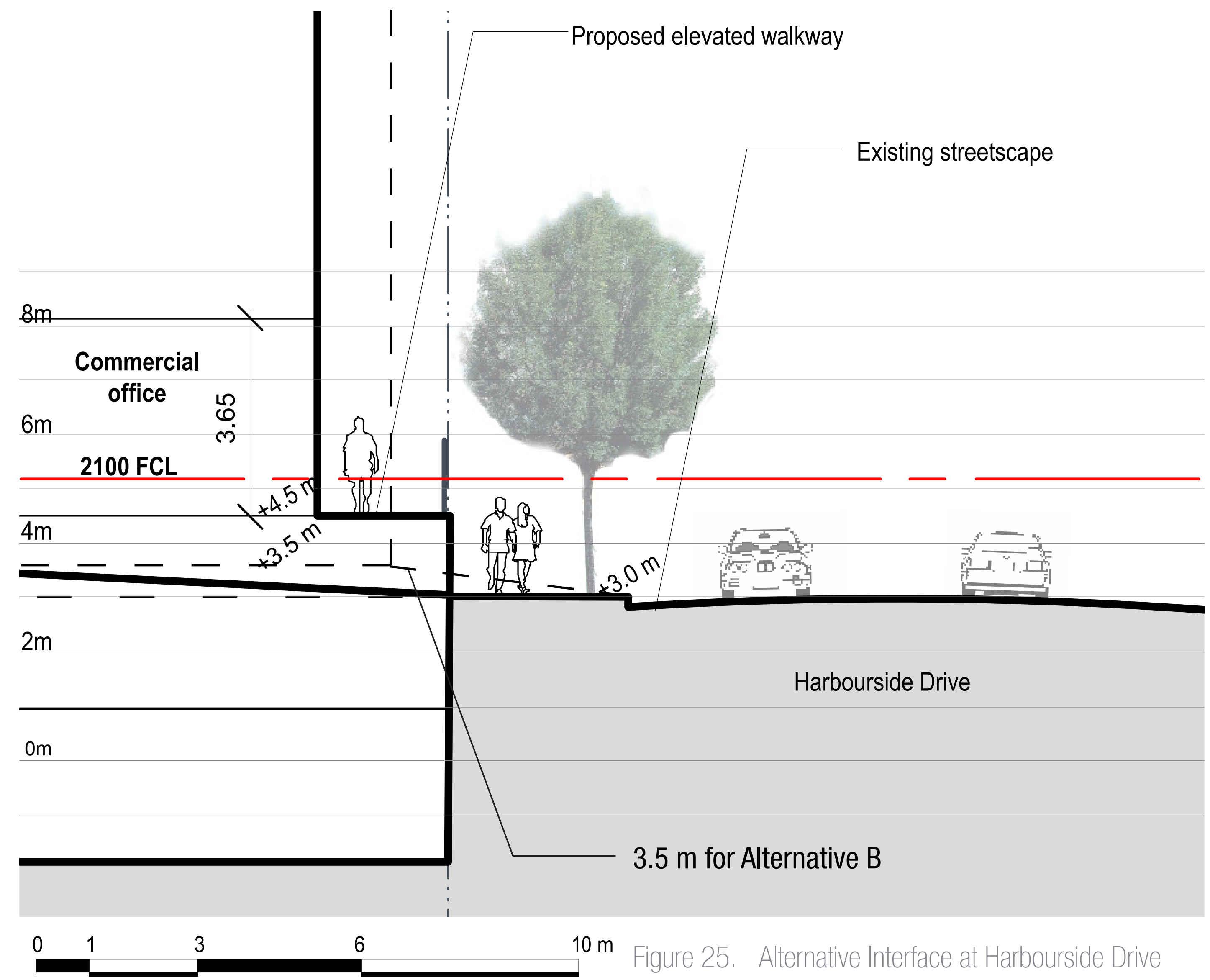


Figure 25. Alternative Interface at Harbourside Drive

4.2 RELATIONSHIP OF HARBOURSIDE DRIVE ALTERNATIVES TO PHASED ADAPTATION

In considering the treatment of the Harbourside development interface with Harbourside Drive, there are several Sea Level Rise and Coastal Flooding refinements that are currently under discussion by the City of North Vancouver and/or various stakeholders around Burrard Inlet:

1. Allowance in Designated Flood Level for design storm surge for Vancouver Harbour (Pt. Atkinson). The historical data record and its analysis are not complete, with questions about whether storm surge allowances should be slightly lower. A joint study on this topic by Burrard Inlet stakeholders has been recommended.
2. Applicability of wave effects in modelling the interface of coastal flooding with stream flooding – current modelling by CNV consultants includes wave effects, but that assumption may exaggerate the estuary backup of stream flood waters. The CNV consultants have recommended a review of current modelling to gain better certainty on this question.
3. Applicability of wave effects on the north side of the Harbourside study area – a very sheltered location, where wave effects are likely to be small to negligible. However, specific modelling of whether wave effects would significantly impact flood levels in this north side after completion of Harbourside, or as development phases take place in phases, has not been completed. The main issue would be potential flanking of waters around raised areas or over the creek banks to flood Harbourside Drive, with water accumulations in the low areas, including near the intersection with Fell Ave.
4. Cumulative vs. joint probability of the highest storm surge coinciding with the highest tides (HHWLT) vs. mean tides (HHWMT). City of Vancouver is currently investigating this issue.
5. Local Government policy on consequences of flooding of various land use types – the highest priority is for protection of life, and therefore residential quarters. Provincial guidelines encourage protection of all habitable (occupied) uses, but there is a case for slightly higher risk tolerance for non-residential uses (e.g. office, other commercial/industrial uses). Provincial policy does not suggest flood-proofing of non-habitable uses (e.g. underground parking, parks, agriculture).
6. Policy on floodproofing of buildings behind dikes (e.g. Richmond, Delta).
7. Role of flood resilient construction materials and methods in floodplains (under investigation in Vancouver, in particular for existing or infill buildings).
8. Actual rate of Sea Level Rise (faster or slower) as more climate data and analysis are available.
9. Timing and cost-sharing of Federal and Provincial funding programs to address the risks and issues (e.g. cost sharing for dike protection, raising roads or other public infrastructure adaptation). The timing of such programs may have an influence on the timing of diking or other adaptation programs to protect.
10. Pacing, roles and staging of investment in adaptation to Sea Level Rise in new developments and redevelopments, by both public sector and private sector. More work is needed on significant urban design issues about adaptation of higher building floor elevations in relation to existing much lower adjacent buildings and streets.

Both this Golder report and the CNV Consultants reports are consistent with current Province of BC Guidelines, which are based on cautious estimates of storm surge and tidal conditions for the design storm.

It is possible that more detailed scientific research will slightly reduce the short term Flood Construction Levels. However, gradual Sea Level Rise will continue for the foreseeable future. In essence, higher building floor levels now would provide more lead time before the effects of Sea Level Rise create unacceptable risk.

At the same time there is a need to not over-react, or act prematurely – tying up financial resources for decades, or making projects uneconomic or with a poor private/public/street interface unnecessarily.

The main need is to recognize the risks, uncertainties and be adaptable in the face of significant unknowns both in Sea Level Rise science across the Salish Sea, as well as the local context of adaptation actions in the Harbourside peninsula.

4.2.1 PROPOSED POLICY PROVISIONS RELATED TO HARBOURSIDE DRIVE AND SIDEWALK INTERFACE

More certainty will be gained in the policy and science context of Sea Level Rise in the next few months and years. The Harbourside rezoning can include provisions that respect the public interest, but allow limited adaptability to new information that becomes available. Therefore we recommend the following rezoning and flood bylaw provisions and process to determine urban design, flood management, and financial investment solutions for the Harbourside Drive interface with the proposed development.

Harbourside Drive Interface Land Use Policy (Zoning Bylaw)

The ground floor of Building Units fronting onto Harbourside Drive in the Harbourside study area shall include a 'flex-zone' designation, which would permit a choice of residential, office or commercial uses in clustered frontages within the general building envelope.

Interim Flood Construction Level Policy (Flood Clauses in Sewer or Zoning Bylaw)

Buildings in the coastal floodplain should meet a minimum Flood Construction Level set by the Local Government (e.g. 4.5 m CGD) for habitable uses, with design and construction provision for adaptation to higher levels that are predicted for the lifespan of the structure.

Phased adaptation to the minimum Flood Construction Level may be considered on a case-by-case basis, with prerequisites including:

- Residential quarters, and the lobby or emergency exit to all residential quarters, must be above the minimum Flood Construction Level.

- Office or other low-intensity commercial uses may be considered on a temporary basis at lower levels, but above the Designated Flood Level (Flood Construction Level minus Wave Effects and Freeboard) for the Design Year, subject to provision of emergency exits to suitable outdoor refuge areas that are at or above the Flood Construction Level, and only in areas where combined coastal and creek/river and storm sewer back-up flooding effects have been calculated by a qualified professional to confirm that the finish floor elevation would be above the Designated Flood Level plus any site-specific requirement for ponding and wave effects at the time of construction.
- Design and construction must incorporate provisions to allow adjustment to at or above the minimum Flood Construction Level when specified conditions (e.g. raising adjacent streets or sidewalks) are present.

Recommended steps, after the rezoning, to finalize Finish Floor Elevations along Harbourside Drive at the Development Permit stage include:

1. Monitor results of current inter-municipal discussions and studies on the ten issues presented above.
2. For each phase, complete a review by a qualified professional of waterfront wave effects, potential flanking or over-topping of phased flood protection with consideration of coastal and creek flooding and ponding, and determine a site-specific Designated Flood Level and Wave Effect Allowance for all sides of the proposed development, including Harbourside Drive.
3. Determine the proposed use or mix of uses in buildings fronting Harbourside Drive.
4. Finalize the proposed FFE along Harbourside Drive, based on the site-specific land use cluster and the Interim Flood Construction Level Policy.
5. Complete architectural and site design, including fronting works and services on Harbourside Drive, and associated staff and design panel review to meet urban design, accessibility and other established design guidelines. Include provisions for adaptation of the design if the adjacent Harbourfront Drive were raised by others in the future by up to 1.0 m.

CURRENT (DRAFT) INFORMATION ON INTERFACE OF COASTAL AND STREAM FLOODING

KWL Associates, in a separate assignment for the City of North Vancouver and District of North Vancouver, have been reviewing the hydrology and hydraulics of the main creek and river watersheds from Mackay Creek to Seymour River, as it relates to existing structures and floodplains. Their work (currently at the draft report stage, and un-released) also includes high level modelling of the interplay between Coastal Sea Level Rise and Stream Flooding, in design years of 2012, 2100, and 2200. Their draft results for Year 2012 and 2100 are summarized below.

Summary of Coastal Flood Levels in Geodetic Datum from KWL Associates (DRAFT)	Year 2012	Year 2100
Designated Flood Level	3.32	4.08
Allowance for Wave Effects	0.48	1.32
Flood Construction Reference Plane	3.80	5.40
Freeboard Allowance	0.60	0.60
Flood Construction Level	4.40	6.00

The current KWL Designated Flood Level (3.32 m CGD) for Year 2012 compares closely with Golder Designated Flood Level (3.34 m CGD) for Year 2020. The two reports vary on allowances for future wave effects, which are specific to the location and shoreline morphology of a given site.

4.3 HARBOURSIDE WATERFRONT 2020 MANAGEMENT STRATEGIES

There are a number of key additional management strategies that should be considered at the time of construction that are not illustrated in this plan.

Sector	Vulnerability	Management Strategy Recommendation
Sub-Surface Parking engineering	SLR will elevate the water table, increasing the displacement necessary for tanked parking lots.	Sub-surface parking would need to be engineered in consideration of SLR.
Sub-Surface Parking access	Depending on access, sub-surface parking could be vulnerable to flooding from behind the concrete band/dike and Mackay and Mosquito creeks.	To reduce vulnerability, access to subsurface parking should be adaptable to stay above the Designated Flood Level as SLR advances.
Utility Rooms	Sub-surface, parking-lot-located utility rooms may be vulnerable to flooding depending on parking lot design and utility room location.	Mechanical, electrical and other utility rooms (e.g. District Energy) would need to be engineered to effectively manage these risks due to a rising water table or marine flooding. There are a variety of management strategies including location optimization and watertight design.

4.3.1 ON SITE ADAPTIVE MANAGEMENT PROVISIONS

The site and buildings will be designed to accommodate key adaptive management options by other parties as vulnerabilities to the buildings, sites and the broader Harbourside neighbourhood, specifically:

Elevating Harbourside Waterfront Park concrete band	The concrete band seaward of Harbourside Place buildings has been designed to support to the future addition of a higher sea wall to replenish freeboard as SLR continues. It would be appropriate in the latter part of this century to evaluate vulnerability based on observation and increased knowledge of SLR about its appropriateness or the necessary height.
Accommodating changing Harbourside Drive Street-Building Interface	To reduce vulnerability of flooding to other residential and commercial buildings on the broader Harbourside neighbourhood, specifically Northward of Harbourside Drive, the City may at some point decide to elevate Harbourside Drive and these sites. The owners have designed the North Facing buildings and the building-street interface on Harbourside Drive to accommodate the street being elevated in the order of 1 m.
Elevating finished floor elevation in Harbourside Drive Units	The floors in the North Facing units, commercial office, along Harbourside Drive are designed to accommodate being elevated to a 4.5 m CGD FCL .

4.4 BROADER HARBOURSIDE VULNERABILITIES & MANAGEMENT CONSIDERATIONS

Some noteworthy vulnerabilities to the broader Harbourside neighbourhood may be of interest to other property owners and governments,

Location	Vulnerability	Management Considerations
Broader Harbourside Residents, Businesses and City infrastructure beyond Harbourside Waterfront	The owners's coastal defence system and the elevated site will reduce the vulnerability of the broader Harbourside neighbourhood to storm surges. Nevertheless, over time, a significant and growing portion of residents, businesses and infrastructure outside of Harbourside Waterfront will be below the FCL and vulnerable to flooding from Mackay and Mosquito Creeks.	The following management options or a combination thereof may be appropriate: • Other sites within the broader Harbourside neighbourhood could be elevated, along with roads and infrastructure with redevelopment. • The City could continue the concrete band into a dike feature the owners establish in the Park at construction out to, and along Mackay and Mosquito Creeks northward. • This diking system could be extended horizontally and vertically in stages, as vulnerabilities grow. • Senior governments, Metro Vancouver and Burrard Inlet municipalities may be interested in eventually establishing a storm surge gate at First Narrows or other harbour-wide strategies
Stormwater	Storm sewers may be vulnerable to flooding of outfalls due to higher sea levels leading to localized flooding in very low-lying areas. This vulnerability will be exacerbated during heavy rainfall when runoff would be obstructed by inundated outfalls, resulting in backwater.	The City may wish to enhance the resilience of the broader Harbourside's storm water management approach. This may include: • seal new sewer mains/manholes against floodwater/ groundwater inflow; • install one way valves in building sewers to prevent back up (existing/new); and • encourage low impact development upstream to reduce stormwater.

These management considerations should be initially evaluated within a more comprehensive assessment. They are not recommendations.

5.0 CONCLUSIONS AND RECOMMENDATIONS

1. In addition to proposed waterfront setback, slopes and terraces, establish a sea flood control line using a concrete band or equivalent form of dike protection at an elevation and in a design that can be adapted to the year 2100 Flood Construction Level, in a location seaward of all buildings and roads. This elevation should be incorporated into the development design and should be:
   Continuous, or if broken in any case by narrow walkways or other penetrations, there must be emergency procedures in place to close these gaps when extreme storm surge events are projected.
   On public property or easement, so that maintenance of this flood control is under public jurisdiction.

2. The Concrete Band Floodwall, or equivalent, should provide protection to the Flood Construction Levels established in this report, and refined by more detailed analysis in the design stage. The required height will vary over time as SLR progresses, and as more data becomes available. Minimum Finish Floor Elevation (FFE) behind the floodwall is recommended at 4.5 m CGD for residential.

3. Some FFE's below the FCL may be permitted, because they are located behind the concrete band dike feature. For example, non-habitable uses (e.g. retail and office) could be constructed to lower FFE. The City of North Vancouver proposed minimum floor elevation of 4.5 m CGD, which is above the Designated Flood Level, should be respected or exceeded for all habitable building floor construction on the waterfront half of the property, Non-residential uses located on northern, sloping portions of the site (between Harbourside Drive and the central spine) should use a phased adaptation approach guided by policies in Section 4.2.1 of this report. The risk of setting this lower Harbourside commercial office elevation should be evaluated more closely with detailed design.

4. Additionally, the edge conditions on Harbourside Drive should be designed to accommodate the possibility of elevating the street in the event that the City and property owners elevate adjacent properties.

5. Sub-surface parking should be designed for flood control and to accommodate a potential increase in the height of the water table (not calculated in this report). Given their setback and position behind the concrete band dike feature, parking lot entrances should be set minimally at the Flood Construction Reference Plane. Various management strategies can be used to manage flooding in sub-surface parking. The more freeboard above the Flood Construction Reference Plane, the less extensive these alternative management strategies would be. The final parking lot entrance elevation should be evaluated with more detailed design in consideration of the other management strategies, and the nature of the property being protected.
   Mechanical, electrical and other utility rooms utility rooms situated below the 4.5 m FCL – typically in sub-surface parking lots – should be engineered to effectively manage flooding risks from a rising water table or marine flooding.

6. Design of the public park, greenway and associated utilities seaward of the proposed sea flood control line (concrete band) should be designed to minimize flooding damage and to mitigate wave effects.
   The Spirit Trail, as the most critical community infrastructure in the Park, should be located on an upper terrace.
   Design should take into account the 'often dry, rarely wet' pattern, and also the changes in the elevation / location of this shoreline pattern as SLR progresses.
   At the detail design stage for each phase, a coastal engineering review of all proposed waterfront design slopes, walls, and terraces should be completed – varied slopes, materials and setbacks may lead to different wave effects and changes to resulting Flood Construction Elevations.
   Flooding of the area seaward of the concrete band due to extreme storm surge events combined with an extreme high tide are low probability events but could occur at any time, and the risk may increase in the latter part of the Century if SLR follows current trends and projections. The upper terraces will be above the DFL, and above the normal daily tide ranges. There may be a need to anticipate cleanup and repair of the landscape areas when the design flood combination of extreme storm surge, high tide and wave effects occur.

7. Projects within 30 m of the high water mark need to be reviewed by BERC or its descendant agencies. The proponent should advise BERC and be prepared to make an application which would include inventory of the shoreline and demonstrate the quality of the habitat enhancement and compensation of the proposed design. The extent of analysis needed depends on the alternative chosen.

8. Conceptual and detail designs of building and site development within the Harbourside Waterfront should undergo a SLR Engineering Review to ensure that the recommendations of this plan are understood and implemented. Such a Review should include geotechnical analysis of the study area's vulnerability to subsidence, and make any necessary adjustments to local relative SLR, and, in turn, FCLs.

9. Harbourside Waterfront should be designed to accommodate the option of extending the shoreline treatments and concrete band East and West and back along Mackay and Mosquito Creeks, to increase protection to the broader Harbourside neighbourhood, and also the option of elevating this dike at approximately 2100, as part of a broader City initiative. Design of shoreline treatment and diking in areas influenced by the creeks will need to consider the interface between stream hydrology and marine flood risks.

10. Given the scale for which there is reliable storm surge and wind data for Vancouver Harbour and the importance of this harbour (North America's 3rd busiest port), the City should work with other governments, senior and local, and large organizations with a stake in Vancouver Harbour (e.g. Port Metro Vancouver) to support more locally specific data.
    These strategies may not reflect the most up-to-date building sites and site designs. Building setbacks and various site elevations are, nevertheless, sufficiently accurate to maintain the integrity of the FCL.

These recommendations are based on current knowledge, published science, and accepted data in the public domain. There will be a need to periodically review these recommendations as new scientific data become available, in particular regarding appropriate storm surge allowances for the Pt Atkinson / Vancouver Harbour area, and regarding the rate of SLR due to global changes. An adaptive management approach is warranted.

GOLDER ASSOCIATES LTD.

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APPENDICES

1.0 ALTERNATIVE B WAVE RUN-UP PLANNING HORIZON 2020

Table A1 presents the calculated run-up values for planning horizon 2020. The range of run-up values shown reflects the range of slopes and depths over which the R2% calculations were applied. The design wave was held constant for each of the calculations. The design wave employed was calculated as part of the sea level rise flood management plan report prepared by Golder as forced by a 200-year design wind from Vancouver Airport (YVR) having a significant wave height of 0.6 m and a peak wave period of 2.7 seconds. The design water level was assumed to be the Designated Flood Level for the 2020 planning horizon: 3.34 m.

Table A1: Prediction of Wave Run-up (R2%) on Two Slopes for Planning Horizon 2020, Using Hughes Method.

Profile	Wave Hindcast		Structure Parameters		Wave Run-up
	Sign. Wave Height	Peak Wave Period	Slope	Depth	$R_{2\%}$
	(m)	(sec)	(%)	(m)	(m)
Section B1a (Parcel B Section 1 from PWL)	0.6	2.7	2 – 30	0.3 – 3.3	0.1 – 0.8
Section B1 (Parcel C Section 1 from PWL)	0.6	2.7	2 – 28	0.2 – 3.3	0.1 – 0.7
Section B2 (Parcel C from PWL)	0.6	2.7	23 – 25	0.3 – 3.3	0.6 – 0.7
Section B3 (Parcel D Section 2 from PWL)	0.6	2.7	34 – 55	0.7 – 3.3	0.8 – 1.2

2.0 ALTERNATIVE B WAVE RUN-UP PLANNING HORIZON 2100

Table A2 presents the calculated run-up values for planning horizon 2100. The range of run-up values shown reflects the range of slopes and depths over which the R2% calculations were applied. The design wave was held constant for each of the calculations. The design wave employed was calculated as part of the sea level rise flood management plan report prepared by Golder as forced by a 200-year design wind from Vancouver Airport (YVR) having a significant wave height of 0.6 m and a peak wave period of 2.7 seconds. The design water level was assumed to be the Designated Flood Level for the 2100 planning horizon: 4.04 m.

Table A2: Prediction of Wave Run-up (R2%) on Two Slopes for Planning Horizon 2100, Using Hughes Method.

Profile	Wave Hindcast		Structure Parameters		Wave Run-up
	Sign. Wave Height	Peak Wave Period	Slope	Depth	$R_{2\%}$
	(m)	(sec)	(%)	(m)	(m)
Section B1a (Parcel B Section 1 from PWL)	0.6	2.7	2 – 30	0.6 – 4.0	0.1 – 0.8
Section B1 (Parcel C Section 1 from PWL)	0.6	2.7	2 – 28	0.9 – 4.0	0.1 – 0.7
Section B2 (Parcel C from PWL)	0.6	2.7	23 – 25	1.0 – 4.0	0.6 – 0.7
Section B3 (Parcel D Section 2 from PWL)	0.6	2.7	6 – 55	0.6 – 4.0	0.2 – 1.2

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21. This information has been received from the City in several meetings (August 8th , 15th) and email correspondence (August 1st) with City staff, and a meeting with a City staffer and its SLR Consultant (September 6th).
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GLOSSARY AND ACRONYMS

BERC: **Burrard Environmental Review Committee**

DFL: **Designated Flood Level**

FCL: **Flood Construction Level**

FCRP: **Flood Construction Reference Plane**: The vertical elevation of an estimated future Natural Boundary from which the FCL is determined.

HHWLT: **Higher High Water Large Tide**

LiDAR: **Light Detection and Ranging:** A system, typically airborne, made up of three core components: (1) a scanning laser, (2) a positioning system, and (3) an attitude system. The system collects a set of measurements (laser range, x, y, and z coordinates of the airframe, and roll, pitch, and heading), which together allow three dimensional coordinates of the ground point to be computed.

BC MoE: **BC Ministry of Environment**

SLR: **Sea Level Rise**: An allowance for increases in the mean elevation of the ocean associated with the future climate change, including any regional effects such as crustal subsidence or uplift.

Storm surge: A change in water level caused by the action of wind and atmospheric pressure variation on the sea surface. The typical effect is to raise the level of the sea above the predicted astronomical tide level, although in some situations, such as when winds blows offshore, the actual water level may be lower than that predicted. The magnitude of a storm surge on the BC coast will be dependent on the severity and duration of the storm event in the North Pacific, its track relative to the BC coast and the seabed bathymetry at the site.