Eastern Bays Shared Path: Coastal Physical Processes Assessment

Prepared for Hutt City Council

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Executive summary

The Eastern Bays Shared Pathway is a key project in providing a safe and integrated network for commuting and recreational purposes in Hutt City and the wider Wellington region. The Hutt City Council (HCC) Project is to develop a safe and integrated walking and cycling facility to connect communities along 4.5 km of Marine Drive in Hutt City's Eastern Bays between Point Howard and the Eastbourne. Equally, the Project improves, and provides a basis for future opportunities for protecting the resilience of the road and underground services by upgrading the supporting seawalls and providing a robust platform that will assist in providing protection as sea levels rise.

This report is one of a suite of technical reports that has been prepared as appendices to the Assessment of Environmental Effects Report (AEE) for the Project.

The particular focus of this report is an assessment of the effects of the Project with respect to physical processes in the coastal and estuarine environment, both the temporary effects arising during construction and those effects that may accrue over the long-term operational life.

Summary

Overall the construction and operation of the Shared Pathway project will have no more than a minor effect on coastal physical processes – provided detailed design and construction elements mitigate the specific moderate effects outlined herein.

With regards to climate change effects on coastal processes (i.e. coastal hazards and sea level rise), the shared path is a first step in potential incremental upgrades that will assist in providing protection to the road (and underground services) in addressing the effects of sea level rise (SLR) along this section of the coast. The Project includes design elements will "buy some time" for HCC to develop a detailed dynamic (flexible) adaptive pathways plan for the Eastern Bays area to adapt to climate change and particularly ongoing sea-level rise over several centuries (along with any region-wide subsidence over this time). The Shared Path has been designed to enable additional protection to be added onto the top of it in the future if that is considered appropriate, it provides a wider foundation platform for any further structural adaptation options, and it does not compromise other future climate change adaptation options. However, future community expectations of its "shelf life" will need to be managed, pending a long-term adaptation planning and consultation process with the community and utility providers, as outlined in the 2017 Ministry for the Environment's (MfE) coastal guidance.

Existing environment

The existing environment along the Eastern Bays coastline of Wellington Harbour is characterised by a series of rocky headlands separating sand- and gravel-filled embayments. Marine Drive is constructed on a seismically uplifted wave-cut platform and the former backshore area of each beach.

Beach sediments within the Eastern Bays have arrived from local and distant sources over the Holocene period (12,000 years ago to present), interspersed with sediment pulses from past major earthquakes. However, human activities have interrupted the sediment supply (e.g. Seaview reclamation 1955, Seaview marina breakwaters 1980s, ongoing dredging at Hutt River mouth) hence present-day rates of sediment accumulation or erosion from Eastern Bays beaches are low and not anticipated to increase in the near future. The tidal range and tidal currents are small within the deep Harbour and most sediment in the coastal zone is transported through wave action, aided at times by wind-generated currents. Waves are relatively small (compared to the open ocean) due to the short inner-harbour distance for waves to develop and oceanic swell waves from Cook Strait are dissipated through the narrow harbour entrance.

Marine Drive has been widened several times through small seaward enlargements, with the coastal fringe supported by engineered concrete and rock defences, several of which are in poor condition or provide inadequate protection from overtopping by storms. These structures have encroached onto the upper Coastal Marine Area (CMA) over rock platforms and beaches. The existing seawalls have disrupted the natural sediment transport regime within each bay, particularly during storm events. Over time, the effect of the seawalls and reduced sediment supply (compared to the natural undeveloped state) on beaches has a slow loss of sand volume, reducing beach width, coarsening of beach material and subtly change the overall plan shape of the beach.

The low-lying Marine Drive and urban areas within the Eastern Bays are vulnerable during high water levels combined with waves and onshore winds. Storms regularly cause localised flooding in roads and property near the coast, with hazardous wave overtopping making Marine Drive unsafe at times for vehicles and pedestrians in several exposed locations (notably Lowry Bay).

In response to storms conditions, the beaches of the Project area show common morphological responses with short-term fluctuations of beach width and sediment distribution inside each bay (i.e., periods of erosion and accretion) on daily to seasonal timescales. There is no clear long-term trend of erosion or accretion in the embayments of the Project area, demonstrating that the sediment volume within each bay remains nearly stable in the long term and the embayments effectively function as isolated sediment compartments. However, some input of gravel and sand from southern shores is anticipated to the southern-most beach of the Project (Days Bay and south), but the future volumes are not expected to be substantial due to dwindling supply of sediment from south of Pencarrow Head and reduced wave energy within the harbour.

The proximity of the Project to active faults, expanse of soft seabed sediments and geological history of large seismic events suggests that the fill/reclamation structures will require careful design to maintain serviceability access following a seismic event. It also suggests that a major down-faulting event may submerge much of the Eastern Bays foreshore into the Harbour waters.

Climate change will have an unavoidable effect on the wider Eastern Bays Region. The principal effect on the Project comes from the rising sea level, which will reduce beach area (as no increase in sediment supply is anticipated) and increase coastal hazard risk. Beach size reduction of 50% less than present day is anticipated with recent SLR projections of 0.5 m by 2050 and will be worsened by secular subsidence of 2–5 mm/years. SLR will lead to an increased frequency of wave overtopping and coastal flooding on the low-lying foreshore, and an ongoing reduction in level of service along Marine Drive. Other climate change effects on coastal hazard drivers such as wave heights, wind speed and storm surges, are secondary to the effects of sea level rise (SLR) but are expected to further increase hazards in the Eastern Bays.

Effects Assessment: Operation of the Project

The overall assessment of operational and construction effects includes both the effects *of the* Project on the environment (such as beach erosion) and effects of the environment *on the Project* (such as extreme waves or climate change), which ultimately address public safety effects and management of significant risks [Resource Management Act (RMA); s6(h)]. This assessment is primarily documented at the wider scale across Eastern Bays Project area but supplemented on a bay-by-bay basis as necessary to highlight significant site-specific differences or effects.

Effects of the project on coastal physical processes during the operation of the project is outlined as follows. These assessments are caveated on the project following best practice construction techniques and detailed design, and with mitigation steps during design expected to further reduce the effect on coastal physical processes.

- Encroachment into Coastal Marine Area (CMA) and coastal zone. The loss of CMA and coastal zone area[1] (the area available for coastal physical processes to occur within) is unavoidable (on the basis that Marine Drive remains intact for this Project), but the effects of the relatively-small loss of area are negligible to no more than minor relative to the local scale of the total area of the Eastern Bays coastal zone. Note that this does not include assessment of loss of the area of beach available as a public amenity, which is addressed in a companion technical report (Greenaway 2018, Appendix K of the Project AEE).
- Beach Nourishment. The proposed beach nourishment to offset loss of beach amenity has no adverse effect on coastal processes such as erosion, wave reflections, wave overtopping or longshore drift over the lifetime of the project. In additional to maintaining beach amenity, the nourishment provides several minor benefits related to increasing the sediment volume, coarseness and longevity of beach sands which will benefit the Shared Path and Marine Drive as relative sea level rises.
- Change to nearshore hydrodynamics and sediment movement. Overall, the Project will have a minor effect on the changes to nearshore hydrodynamics (such as wave height, wave driven currents, wave reflections) and the coupled effect on nearshore sediment processes (sediment transport in the "coastal zone" leading to erosion or accumulation of sediment). However, some key features such as transition between wall types, transition to natural rock foreshore and proposed beach accessways could have a potentially moderate effect on nearshore hydrodynamics leading to accumulation of sediment or potential for erosion of sediment. This effect is somewhat unavoidable because of the need to maintain community access to the beach but has been mitigated to minor through ensuring "smooth" tapering of transitions between seawall/foreshore types and accessways over a 20-30 m length of shoreline.
- Interruption to longshore sediment transport. Overall, the Project will have a minor effect on local longshore sediment transport rates. However, for some features (beach accesses and wall transitions) there is a potentially moderate effect on longshore sediment transport within the confines of the relevant bay. This effect has been mitigated to be no more than minor by careful selection of accessway position within each bay, accessway orientation and type of beach access to prevent obstruction of longshore sediment movement where possible. Small local accumulations of sand or driftwood debris will still occur due to the natural wind/wave-driven surface currents. This accumulation of sand/debris is a short-term effect, depending on the wind conditions at the time, and is negligible in relation to bay-wide longshore sediment transport.

[1] Defined here to extend 200 m offshore from the existing seawall for the 4.4 km Project length.

- Edge effects at seawall transitions and tie-ins. These transitions have the potential to be problematic regarding coastal processes, particularly if poorly designed with abrupt transitions which cause edge effects (waves wrapping around and focussing waves on nearby structures) with associated changes to sediment transport patterns typically resulting in seabed and beach erosion. This is a potentially moderate effect on local sediment transport leading to erosion or (i.e. scour “holes”). This has been mitigated within the Preliminary Design Plans by including a gentle tapering of seawall types across the transitions, with the length of taper dependent on the level of wave exposure. Areas with small wave exposure, such as on the lee-side of rocky headlands, are transitioned over 0 m to 5 m in length. At the most wave exposed locations, where transitioning between rock revetment and double curve walls areas (e.g., northern Lowry Bay), the transitional taper has been lengthened to 20–40 m. These design modifications will ensure the potential effect is mitigated to a minor effect.

- Effect on adjacent seawalls The Project will have a minor detrimental effect on the structural integrity of adjacent older seawalls if appropriate construction phasing is programmed. Ongoing periodic review of seawall condition should continue.

- Fine sediment generation. The reworking of fine-grained sediments (e.g., silts) from beach sediments by the change to nearshore hydrodynamics will have a negligible effect on offshore sedimentation rates or suspended sediment concentrations within each bay and the wider Wellington Harbour and will be negligible relative to ambient turbidity that can be experienced during moderate-strong wave conditions or during Hutt River floods.

- Wave reflections. The change to wave reflection behaviour, caused by the replacement of existing rock revetments with new double-curved seawalls within the Eastern Bays, will likely have a negligible effect on other seawall sections and beaches.

- Wave overtopping. The proposed seawall replacements are all expected to reduce the overtopping hazard during minor to moderate storm events along all sections of Eastern Bays covered by the project (i.e., a minor positive effect). This is through structures that provide more effective deflection, dissipation and reflection of incident waves than the existing seawalls. However, there will be no change to overtopping hazard during large storms as there will be no change in crest elevation of the seawalls – albeit slightly lower to allow drainage from the seaward extension of the shared path. Wind-driven spray will continue cause some nuisance flooding during all storms, and temporary closure of the shared path and reduction in speed on Marine Drive will still be required during large storms. Several sections of coastline (e.g., the northern 200 m of Lowry Bay) are more susceptible to wave overtopping and road closures. The present design is more robust in this location and will more effectively reduce the overtopping. However, for larger storm events there is unlikely to be any change to the overtopping hazard as the unaltered crest height governs the overtopping discharge rates.

- Climate change. Climate change will have an unavoidable effect on the wider Eastern Bays Region. The principal effect on the Project comes from the rising relative sea level (including land subsidence in the Wellington region), which will increase the frequency of storm-tide events and reduce beach areas. This will lead to an increased frequency of wave overtopping and coastal flooding on the low-lying foreshore, and an ongoing reduction in level of service along Marine Drive. Other climate change effects on coastal hazard drivers such as wave heights, wind speed and storm surges, are secondary to the effects of sea level rise (SLR) but are expected to further increase hazards along Marine Drive.

- Climate change adaptation. The Project includes design elements which meet the dynamic adaptive pathways planning (DAPP) principles of iterative long-term management of uncertain sea level rise trends (which will continue for centuries) in the Ministry for the Environment (MfE) coastal guidance (MfE, 2017). However, essentially this is only the initial pathway that "buys some time" (a few decades) by improving the present-day resilience of the shoreline infrastructure and improving the foundation strength. This project therefore allows HCC time to develop long-term pathways to adapt infrastructure and communities to climate change, as well as enabling more structural adaptation options to utilise the improved foundations.

Effects Assessment: Construction of the Project

The assessment of construction-phase activities and works details the effects of the Project on the coastal environment. This assessment is primarily documented at the wider scale across Eastern Bays Project area but supplemented on a bay-by-bay basis as necessary to highlight significant site-specific differences or effects during construction. The assessment addresses the *additional* effects from the temporary occupation of formwork/staging - relative to the permanent occupation of the finished Project works which are covered in the Operational effects chapter (Section 5) in relation to permanent losses and effects within the CMA.

Note that the detailed construction methodology will be finalised on appointment of a contractor. However, the key features are included within indicative construction method in the Design Features Report and Beach Nourishment Report. The curved concrete walls are anticipated to be follow a similar construction methodology to the curved walls built in York Bay (between 2012 and 2015). A construction methodology for other key elements of the Project is yet to be fully developed, including the rock revetment, access paths, transitions between wall types and tie-ins to natural rocks. It is understood that these will follow the same principles as the Design Features Report and detailed in the Construction environmental management plan (CEMP).

- Encroachment into CMA and coastal zone: The temporary loss of CMA and coastal zone area by occupation of formwork/staging (the area available for coastal physical processes to occur within) is unavoidable (on the basis that Marine Drive remains intact for this Project), but the effects are negligible from the relatively-small and temporary loss of coastal area for the proposed phasing of construction sites (20-40 m under construction at any time), relative to the local scale of the Eastern Bays coastal zone. Note that this does not include assessment of loss of the area of beach available as a public amenity, which is addressed elsewhere (Greenaway 2018, Appendix K of the Project AEE).

- Beach nourishment. The proposed beach nourishment to offset loss of beach amenity has low risk to coastal processes during construction. With the proposed mitigation actions, the risk of turbidity exceeding the ambient turbidity that can be experienced during wave conditions or Hutt River floods is considered low. There is no risk to coastal processes such as erosion, wave reflections, wave overtopping or longshore drift during placement of nourished material as the beach will adjust to the natural profile over a period of weeks to months.

- Change to nearshore hydrodynamics and sediment movement. The temporary interruption to nearshore hydrodynamics and sediment transport by construction formwork/staging will be localised (a 20-m section at any one time) and short term (2-3 weeks per section) based on the incremental construction sequence. There is a negligible long-term effect arising from the temporary construction works.

- Fine sediment generation. Short-term reworking of fine-grained sediments (e.g., silts) from beach sediments by the change to nearshore hydrodynamic and wave processes will have a negligible effect on sedimentation rates offshore or suspended sediment concentrations within each bay and the wider Eastern Bays Region.

- Bulk sediment management. The potential effect on local sediment volumes (i.e. beach size) by the removal of sediment during Shared Path foundation excavation is minor to moderate (depending on size of the bay and depth of excavated foundation). Mitigation of these effect to minor throughout was accomplished within the proposed construction methodology by separating native from non-native beach material, disposing non-native beach material offsite, stockpiling beach native material nearby, and crushing rock removed from reef or headland platforms, with stockpiled beach material redeposited on the foreshore after construction of each wall section. Further, the beach nourishment programme will import beach-compatible fill into the bays, further compensating for any losses in beach volume (see Beach nourishment report, Appendix F of Project AEE).

- 3- to 6-month construction period within each bay. The period of construction within each bay will cause unavoidable localised alterations to beach hydrodynamic and sedimentary processes (i.e. scour or sediment accumulations) in the immediately vicinity of the construction works which will persist for the 3–6 month construction period. However, the overall effect is minor and confined to areas immediately adjacent to the construction works (within 5-10 m), and the beach will recover to a new state within days to weeks after construction has ended. The effect will be larger and longer lived if a large storm occurs while construction staging is in place, but the beach will still recover to a new state within days to weeks. If localised erosion looks to undermine the adjacent seawalls, an adaptive management approach to intervene with raking/scraping to fill scoured areas could be undertaken under the same consent with little long-term risk to the beach. There are also potential detrimental effects on the structural integrity of adjacent seawalls by wave reflection/focussing from rigid temporary construction staging (e.g., sheet piling, formwork). However, on completion of construction within each bay, the bay-wide beach processes will quickly readjust to the new beach state (months to seasons), with no long-term effect from the short-term construction activities.

- Multi-year construction phasing. The proposed multi-year phasing of the Project means that some of the wall replacements necessary to reduce the overtopping hazard may not be undertaken for several years. Sea-level rise in the intervening period will be very small (<15-20 mm) and have a negligible effect on the overtopping hazard during this time (notwithstanding the stochastic nature of extreme weather events).

- Seawall maintenance. There is a risk that maintenance of the existing seawalls will cease for up to 6 years because of the pending upgrades by the Project. This is a minor risk to the environment, and easily mitigated by ensuring continuation of the periodic condition assessments, and permission for emergency maintenance is required.

There are no cumulative effects of known external projects in the environs of Wellington Harbour or south Wellington coast which, when combined with the Project, will have a combined adverse effect on coastal physical processes of Eastern Bays that is more than minor. The effects of the external projects are generally local to that Project with negligible effects which could overlap with effects from this Project.

No further mitigation measures have been introduced in this report which have not been discussed by the Project team or community representatives and incorporated in the preliminary design Preliminary Design Plans and Design Features Report.

Several consent conditions have been suggested to document physical changes to the existing environment, ensure best-practice construction or post-construction house-keeping. We recommend HCC develop a management plan to undertake monitoring of beach volume via 6 monthly beach profiles (or equivalent elevation surveying techniques) for 5 years, with periodic expert assessment of results. This is to ensure the actual effect on beach sediment processes is in line with the expectations for generally minor redistribution of beach material with minor changes to beach volume and beach area compared to present day. In the unlikely event that the assessments indicate that unanticipated erosion is occurring (i.e. beach in disequilibrium), the beach nourishment consent will still be active (and other bays may still be under construction) and HCC may be able to easily top-up the beach with more fill to compensate for erosion losses.

1 Project Background

1.1 Introduction

The Hutt City Council (HCC) proposes to construct a 4.4 km Shared Path along Marine Drive (Figure 1-1 in two sections: between Point Howard and the northern end of Days Bay, and the southern end of Days Bay (Windy Point) to Eastbourne (Muritai Road / Marine Parade intersection)[2]. Approximately five thousand people live along the Eastern Bays, with Marine Drive providing the only road and infrastructure service connections.

Residents have identified[3] that the completion of the Eastern Bays Shared Path and concern about climate change are the two most important issues facing the Eastbourne Community. The Eastern Bays Shared Path Project ("the Project") presents HCC with an opportunity to integrate an efficient response to both of these issues for the next few decades.

The completion of an Eastern Bays Shared Path is a key component of the current HCC transport strategy '*Walk and Cycle the Hutt 2014–2019*'[4]. The Pathway is considered part of the Great Harbour Way/Te Aranui o Pōneke which is a walking and cycling route around Te Whanganui-a-tara, the harbour of Wellington, from Fitzroy Bay in the east to Sinclair Head in the west.

1.2 Key Drivers for the Project

The Project is to develop a safe and integrated walking and cycling facility to connect communities along Hutt City's Eastern Bays, and to provide links to other parts of the network for recreation and tourism purposes. Currently, pedestrians and cyclists connectedness and use along the Eastern Bays is low, due to few dedicated facilities and the tightly constrained nature of the road along Marine Drive. For the most part, cyclists and pedestrians must use the road shoulder, which is very narrow or non-existent in sections.

Equally, the Project improves, and provides a basis for future opportunities for protecting the resilience of the road and underground services by upgrading the supporting seawalls. Marine Drive provides the only road access to the Eastern Bay suburbs and is therefore a key transport route for the region. Key infrastructure services including the main outfall sewer pipeline (MOP) are located within the road corridor. The MOP is an 18 km long pipeline that conveys secondary treated wastewater from the Seaview Wastewater Treatment Plant (which services 146,000 residents and a large number of local industries) to the outfall at Bluff Point, near Pencarrow Head. The MOP is regionally significant infrastructure, and along with the road access and other services are important lifeline utilities for the wider community.

The road is currently experiencing closures, and/or reduced operations, in part due to wave overtopping because of the current state of the coastal edge. The existing seawall in places has a residual life of less than 5 years and, as it has been built on an ad hoc nature over time, is vulnerable to failure and does not provide consistent, nor effective, storm mitigation. Sea level is rising and the landmass subsiding in Wellington, aggravating the situation. MfE (2017) projections forecast a 13 to 27 cm sea level rise between 2030 and 2040 (depending on global emissions trajectories) plus for the Wellington region a subsidence of a further 2–5 mm/yr (or 2–10 cm by 2030-2040, Bell et al.

[2] Days Bay (1 km) is not included as part of the scope of this project as it currently provides a lower speed limit, some safe facilities for pedestrians and increased widths for on-road cyclists.

[3] Eastbourne Community Survey (2014) – see Appendix I of the Project AEE.

[4] http://www.huttcity.govt.nz/Your-Council/Projects/cycleways-and-shared-paths/eastern-bays-shared-path/

2018) should be factored in until improved information becomes available (MfE, 2017; Table 10). Further sea level rise will increase the frequency of all coastal flooding with sea level rise of 0.5 m forecast to be reached sometime between ~2060 and ~2110 and sea level rise of 1.0 m sometime after ~2100 (MfE, 2017, Table 11).

The Project recognises the ongoing processes of managing coastal values in the face of climate change and SLR and related pressures faced by Greater Wellington Regional Council and HCC. However, the Project is not a solution to all the effects of ongoing sea-level rise beyond the next few decades. The Project is a first step in incremental upgrades that will assist in providing protection to the road (and underground services) and is an initial adaptation option in addressing the effects of sea-level rise along this section of the coast. It does not preclude future options and has been designed to enable additional protection to be added onto the top of it in the future if that is considered appropriate.

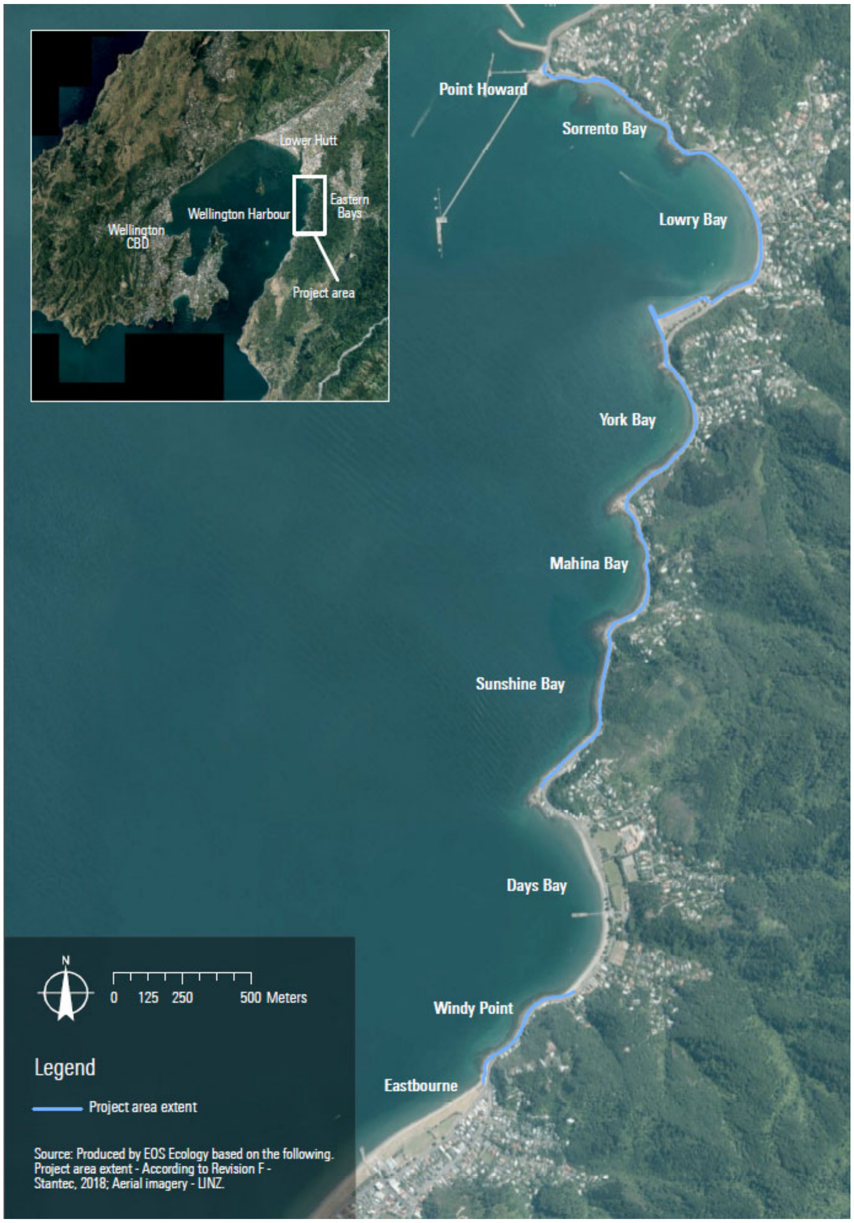


Figure 1-1: Project extents. [*Source*: Preliminary Design Plans (Revision J) and EOS Ecology (2018a)].

1.3 Scope

NIWA were contracted as technical experts to provide input to the Project as several stages of the Project. This included the MCDA for replacement seawall design options (Detailed Business Case stage, DBC) and the assessment of environmental effects (AEE) of the Project on coastal physical processes (Consenting stage – this report).

The multicriteria decision assessment (MCDA) process was used to evaluate design features of the Shared Path coastal frontage. The MCDA process had input from a range of technical experts including intertidal ecology, avifauna ecology, terrestrial ecology, coastal processes, landscape and visual, engineering design, planning and consenting, and community engagement. The assessment of seawall treatment options (Stantec 2018a: Alternatives Assessment) presents the MCDA findings for early phases of the Project and the options recommended are those are assessed in this report for their effects on coastal processes. A copy of NIWA's input to the MCDA process is included in Appendix A of this report.

1.4 Purpose of this report

This report is one of a suite of technical reports that has been prepared as appendices to the Assessment of Environmental Effects Report (AEE) for the Project. This report will included as be Appendix E of the Project AEE.

The particular focus of this report is an assessment of the effects of the Project with respect to physical processes in the coastal and estuarine environment, both temporary effects during construction and those effects that may accrue over the long-term operational life. Effects are assessed in line with requirements of the relevant statutory criteria.

In assessing coastal effects, the main elements of the Project that were considered are the replacement seawall design and performance, the shared pathway embankments that encroach into the coastal environment, the temporary staging for construction works, and mitigation measures to be incorporated into the design to minimise effects on coastal processes and the level of service for users.

The scope of this coastal processes report includes consideration and assessment of the effects of the Project on the existing coastal environment, mainly on:

- tidal processes (e.g., tidal currents, tidal elevations)
- natural coastal hazards (e.g., storm-surge and wave overtopping hazards)
- sedimentation and scour of the seabed, intertidal areas and beach (e.g., change to erosion and accretion patterns)
- suspended sediments arising from seabed disturbances (e.g., excavation for construction, import of beach material) and de-watering discharges, and
- effects of climate change, particularly sea-level rise.

These processes need to be factored into the overall design of the current Project along with future staging of seawall replacements in order satisfy the requirement under the RMA to consider the effects of climate change, and over a period of at least 100 years as stipulated in the NZCPS-2010 (Policy 24).

Specific matters addressed by other reports are: a) effects of operational stormwater discharges on the receiving environment; and b) ecological assessments including intertidal (EOS 2017, Appendix A of Project AEE) avifauna and vegetation (Overmars 2018, Appendix C of Project AEE), freshwater fish passage (EOS 2018, Appendix B of Project AEE), Beach nourishment design and assessment (T&T 2019, Appendix F of the Project AEE). This report does not cover erosion and sediment control measures that will be part of the Construction Environmental Management Plan (CEMP). Refer to the consent application document for a full list of reports supporting the Project AEE.

This assessment is based on drawing set Revision J and the design features report (Stantec 2018b).

The project data provided in this report was derived by EOS Ecology from GIS files that were provided by Stantec based on the Design Plans (Revision J) files. The data was provided to the project team as GIS shapefiles and as summary data in a MS Excel spreadsheet by EOS Ecology on 29 November 2018. Calculations from the derived GIS files are intended to provide a best estimate prior to detailed design. The data is referred to in this report as 'project data'.

2 Project Description

A full description of the Project, including its components and construction, is contained in the resource consent application for the Project. This assessment is based on the Preliminary Design Plans (Revision J, Appendix N of consent application), and the Design Features and Construction Methodology report (Stantec 2018b, Appendix J of the Resource Consent Application).

The various design features extracted from the Design Features Report are considered indicative of the final design, but it is acknowledged that various details will be refined through the detailed design stage and following consent submissions.

2.1 Overview

In brief, Hutt City Council (HCC) proposes to construct a shared walking and cycling pathway alongside Marine Drive, linking Eastbourne to Point Howard which are 5.5 km apart (Figure 1-1). The shared path construction has a secondary purpose of upgrading the coastal defences (e.g., seawalls or rock revetment) to improve resilience of the road network to hazardous wave overtopping along Marine Drive. The Project extents include 4.4 km of this 5.5 km length based on where the existing walking and cycling facilities are inadequate (see Figure 1-1).

The shared path connects several bays and headlands which have approximate extents shown in Figure 1-1 and Table 2-1. The Project chainage extents begin at Port Road, Seaview in the north (0 m) and extend to near Eastbourne in the south (5500 m). Note that Port Road to Point Howard (0 m to 530 m) and Days Bay (chainage 4110 to 4990 m) are not included in the project assessments.

Table 2-1: Project chainage descriptors. [Source: Project Drawings Revision J].

Name	Project chainage (m)		Length (m)
	Start	End	
Port Road*	0	530	530
Sorrento Bay (includes Point Howard Beach)	530	1150	610
Lowry Bay	1150	1960	810
York Bay	2190	2570	380
Mahina Bay	2910	3400	490
Sunshine Bay	3480	4000	520
Days Bay*	4110	4990	990
Rona Bay (incl. Windy Point)	4990	5500	510

*Excluded from Project scope

The width of the pathway will vary from 2.5 to 3.5 m wide, as rationalised in the assessment of alternatives (Stantec, 2018a, dated March 2018). A 3.5 m nominal width is anticipated except where specific obstacles (trees, bus shelters, beach amenity, etc.) restrict the width. Space for the enlarged pathway footprint is principally obtained by expanding seawards onto the beach and rocky foreshores, and to a lesser extent by optimising road shoulder widths and refinements to coastal defence designs. A key component of mitigating adverse effects is beach nourishment (import of beach compatible sand) to maintain the area of beach available for public amenity.

Not all the 4.4 km Project chainage requires a new coastal defence (Table 2-2), hence the Project will replace or upgrade seawalls along 70% (3.14 km) of the existing 4.4 km coastal frontage with no seawall works along 30% (1300 m). The existing seawalls are likely to either be demolished (and removed off site) to make way for the replacement seawall or will be buried by the replacement seawall.

The lengths of new seawall include several forms selected during the DBC phase (Stantec 2018a);

- a curved concrete seawall, or
- a rock revetment, or

Full descriptions and diagrams of the proposed seawalls is found in the Design Features Report (Stantec 2018b) and summarised here in Section 2.2.1.The location and extent of each type of coastal defence is summarised in Figure 2-1 from the Preliminary Design Plans (Revision J, see Appendix N of the Consent application), with the measurements of each type shown in Table 2-2. The Project also includes allowance for pedestrian access to the foreshore via the new seawalls and specific accessways which occupies a small proportion of the Project length (Table 2-2). Details of beach access are discussed in the Design Features Report (Stantec 2018b).

Table 2-2: Total length of proposed seawall types and beach access within Project extents. Grey shading highlights breakdown of seawall type, where relevant [*Source*: Preliminary Design Plans Revision J].

Seawall type	Seawall type - detailed	Total length (m)	Percentage of total project length (%)
Curved-stepped concrete wall	All	2647	59.6
	Single step	*190*	*4.3*
	Double step	*2128*	*47.9*
	Triple step	*225*	*5.1*
	Double/Triple	*104*	*2.3*
Revetment	-	434	9.7
Beach access	All	64	1.5
	Mini Steps	*17*	*0.4*
	Steps	*27*	*0.6*
	Ramp	*20*	*0.5*
No seawall works	-	1298	29.2
	Existing seawalls	*1004*	*22.6*
	No seawall	*294*	*6.6*
Total project length	-	4443	100

Figure 2-1: Proposed coastal defences along project area. [Source: Preliminary Design Plans (Revision J) and EOS Ecology (2018a)].

2.2 Design features

Key elements of the Project which could potentially affect coastal physical processes in the Coastal Marine Area (CMA), either during construction and/or during the operational lifetime, include defence type and design, extent of permanent occupation of the CMA and reclamation of the works, transitions between wall types, beach access points, beach nourishment, construction methodology and construction phasing.

Descriptions of the following design elements are taken from the Design Features Report (Stantec 2018b), with comments and justification added for clarity of the implication on coastal processes.

2.2.1 Coastal defence designs

The coastal defence types are shown in Figure 2-2 to Figure 2-3, with the location and extent of each type shown in Figure 2-1 for the project area.

Path elevation

A key coastal engineering constraint of the Project is that the proposed pathway has a smooth transition from the existing road surface to the shared path surface (i.e., no kerb channel for stormwater interception). This effectively constrains the elevation of the pathway to that of the existing road (at white edge line), minus some cross-fall allowance for stormwater drainage over the shared path. The alternatives assessment (Stantec 2018a) provides the assessment and rationale for this design limitation.

Rock Revetment

Where a revetment type seawall is proposed the shared path will be supported by a vertical reinforced concrete cantilever wall with rock protection on the coastal side (Figure 2-2). The revetment is proposed for rocky shore areas where it is desirable to maintain a non-concrete shoreline and in areas of existing rock revetment. Approximately 430 m (9.7% of project seawalls length) of the Project is proposed to have a revetment type coastal defence.

The cantilever wall behind the rock amour will be designed as a standalone element i.e., the wall will not be reliant on the seaward side rock armouring to retain the road pavement and shared path. The top of the concrete wall being flush with the shared path (i.e., no lip or kerb). The interface between the revetment and the shared path varies according to the structural requirements of the wall and the shore configuration.

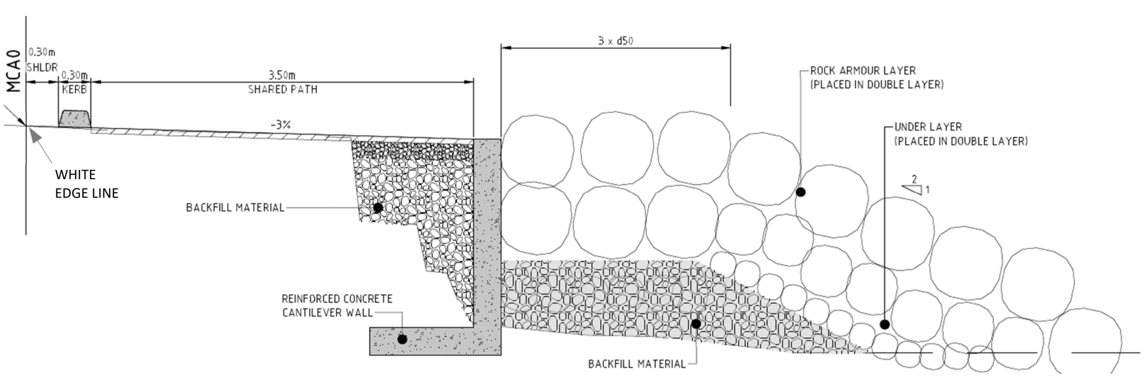


Figure 2-2: Revetment with reinforced concrete cantilever wall. Note a geotextile underlay (not shown) will be included between backfill and underlayer. [*Source*: Stantec 2017b].

The design of the revetment profile is consistent with international best practice guidance of the Coastal Engineering Manual (USACE, 2006) to optimise structural stability and minimise wave overtopping. As shown in Figure 2-2, the typical design of revetment seawall sections has an outer double layer of large rocks, average rock diameter (d_{50}) of 0.5 m, overlying smaller rocks which are placed on the foreshore and seabed. A geotextile underlayer will be placed between the large rocks and backfill material to prevent fines escaping and reduce slumping. The top of revetment rock is 0.3 m above the shared path elevation (approximately 0.5 × d_{50}) and is level for 1.5 m seaward (approximately 3 × d_{50}) before it slopes down to the seabed interface. The revetment typically slopes down towards the water at a gradient of 1V:2H.

To evaluate the stability of an average rock diameter of 0.5 m we used The Rock Manual (CIRIA 2006) with formulae from Van der Meer (1998) for randomly placed rock and Nurmohamed et al. (2006) for orderly placed rocks[5]. These preliminary calculations indicated that the wave height is the key parameter to determine rock size with no change to stability for a range of storm-tide elevations (see Section 4.4) or as mean sea-level elevation increases with future sea level rise (see Section 3.2.4). In Wellington Harbour, the largest waves expected at 1% Annual Exceedance Probability (AEP) are 1.5 m high (see Section 4.6). The calculated stability numbers for these wave conditions are 2.3 for orderly rock placement and 1.8 for randomly placed rocks. This indicates the revetment is statically stable with only limited damage (the transition away from statically stable occurs at stability number of 4) as small displacements of individual rock via rocks moving/rocking under wave attack, which may be easily repaired by re-placing displaced rocks after storms. Therefore, all storms smaller with waves smaller than the 1% AEP wave height are unlikely to cause material damage to the revetment and an 0.5 m average rock diameter is an appropriate conceptual design for consenting and the Preliminary Design Plans. Further, the revetment is not critical to the structural integrity of the Shared Path foundation, so the risk of revetment damage via under-sized rock is low and could be easily repaired after any damage.

The final design of the revetment areas will be addressed during detailed design, including secondary design aspects of rock sizing grading curve and any placement requirements (e.g. filter/bedding layer specifications).

Curved concrete seawall

A typical example of the curved concrete seawall is shown in Figure 2-3. The curved concrete wall has a flat top that forms the base of the shared path, and either a single, a double or triple curved face that acts as an oversized step, with a 0.9 m tread (0.6 m nose to nose) and an 0.8 m riser. It replicates the existing curved sea wall at the south end of York Bay. The total length of curved seawall is 2.65 km (59% of total project length, Table 2-2), with a double curved wall the most widespread type for 2130 m (47.9%), although variants include single (190 m or 4.3%), triple (230 m or 5.1%) or transitions between double and triple (104 m, 2.3%) stepped varieties.

The use of single-curve wall is only planned for southern Lowry Bay where the existing beach elevation is close to the shared path elevation (within 0.8 m). The benefits of the single curve in this location are the reduced excavation depth and smaller CMA encroachment when compared to the double curve design. The triple-curve wall is only planned for York Bay, where there is a large

[5] Calculated using slope of 1V:2H, rock density of 2.7 t/m^3, a permeable underlayer and core, safety factors for rock size of 1.2, mean wave period of 4 seconds, a 3 hour storm (over high-tide period) with 2400 individual waves.

elevation difference between the shared path and beach elevation (exceeding 1.8 m in places). the benefit of the triple-curve wall in this location is the consistent aesthetic form (i.e., the curved shape rather than the vertical foundation below the double-curve wall) when the beach is at lowest levels (i.e., when eroded after storm waves).

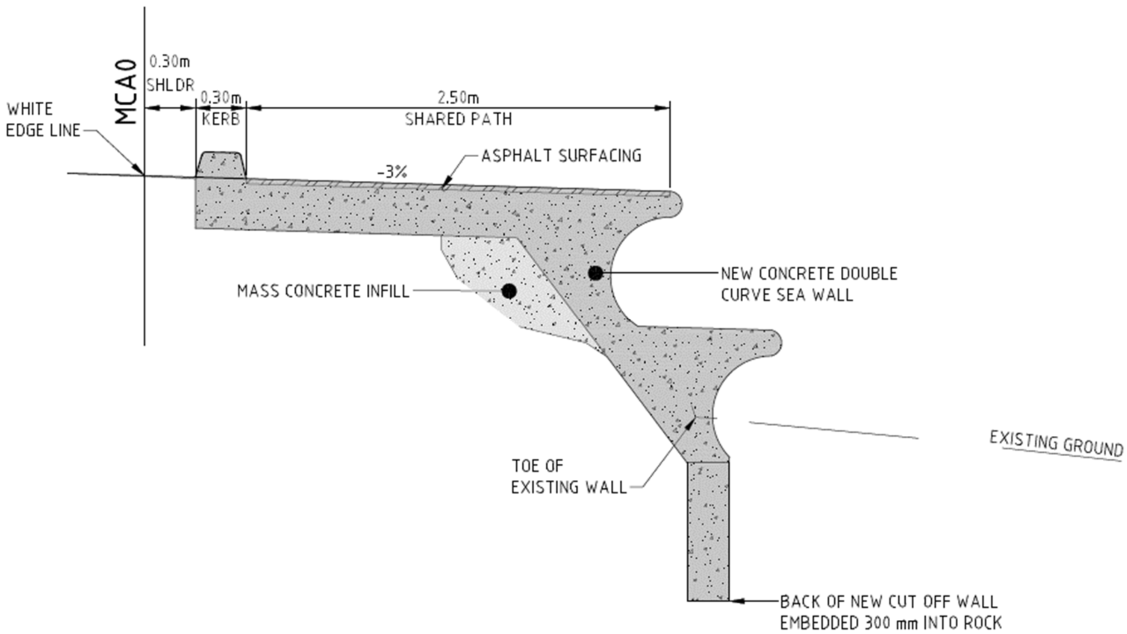


Figure 2-3: Double curved concrete seawall, variants include single and triple curves. [*Source*: Stantec 2018b].

The double curved wall will be cast in-situ to aid constructability given the irregularity in the coastal zone and founding bedrock. While there are environmental risks with this approach, they will be managed as outlined in the construction methodology.

Textures embedded into the concrete will be incorporated into the concrete surface of the seawalls to provide opportunities to establish biota habitat. These will be small features (less than 5 mm in depth) to avoid compromising the concrete performance.

2.2.2 Transitions between defence types

The proposed seawall designs vary along the coastline (e.g., Figure 2-1) as decided within the pre-consenting DBC phase and aligned to minimise environment effects (see Stantec 2018a). Where each wall type transitions to a different type (i.e., revetment to curved seawall, not from single-double curved), special treatment is anticipated to minimise the effects on coastal processes and marine ecology.

A generic wall transition between a double curve wall and a revetment structure is illustrated in the Figure 2-4. Variations to this transition design will incorporate pedestrian access steps into the transitions, either as concrete steps or via an informal 'unofficial' step and will require some contractor expertise and direction. This would be addressed in the CEMP.

These details will be refined through the detailed design stage.

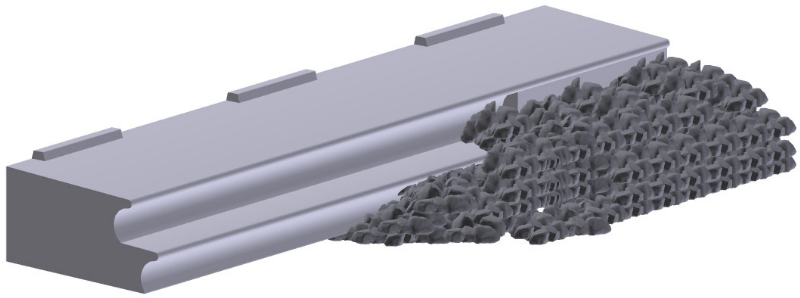


Figure 2-4: Typical design of transition between double curve and revetment seawall types. [Source: Design Features Report (Stantec 2018b)].

2.2.3 Beach nourishment

Beach nourishment is proposed at the priority beaches of Point Howard, Lowry Bay and York Bay as a strategy to mitigate loss of beach area available for beach amenity (Greenaway 2018, Appendix K of the Project AEE). Nourishing the beaches with imported beach-compatible fill, has a secondary benefit of improved coastal protection.

A conceptual schematic of beach nourishment is shown in Figure 2-5. Note that the beach width *W* is equal to the loss of beach width *W* by shared path encroachment. For this Project, the berm elevation *B* is equal to the tidal elevation where greatest loss of beach area is anticipated for which the nourishment must offset. No allowance is made for additional beach material to further widen or enhance the beach.

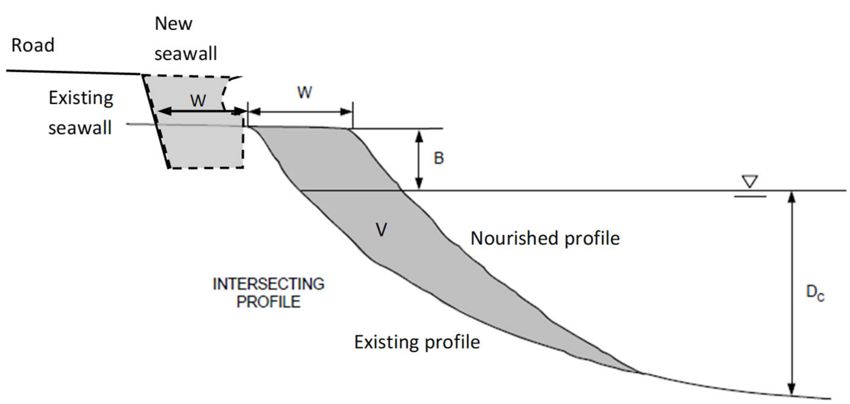


Figure 2-5 Schematic of beach nourishment as cross section through beach [not to scale]

Indicative volumes required for nourishment, proposed construction methodology, and suggested monitoring and consent conditions for the Eastern Bays Shared Path project are described in detail in the Beach Nourishment report (Appendix F of the Project AEE).

2.2.4 Beach access points

There is strong community support to continue to provide beach access to the beach area and the NZ Coastal Policy Statement (2010) has a requirement (Policy 19) to “maintain and enhance public walking access to, along and adjacent to the CMA". Access is primarily for pedestrians, but boat (non-vehicular) access is also anticipated for some areas, and to allow for beach maintenance.

Beach access options are shown in Figure 2-6 and described below. They include:

- **Standard steps**. Parallel to the shoreline to reduce encroachment with tapered platform to minimise localised beach transport effects. Also includes an option to integrate beach access and wall type transition.
- **Mini steps**. Proposed at intervals between the standard steps to achieve additional access to the beach without encroaching unnecessarily onto the coastal marine area.
- **Boat ramps**. Proposed only where existing boat ramps are provided. Maximum boat ramp grades have been set at 1V:4H, and parallel to the seawall.

A corrugated texture will be added to the concrete surfaces to shed sea water and reduce slipperiness.

Generally, the Preliminary Design Plans provide up to two accesses per beach.

Further details will be refined through the detailed design stage.

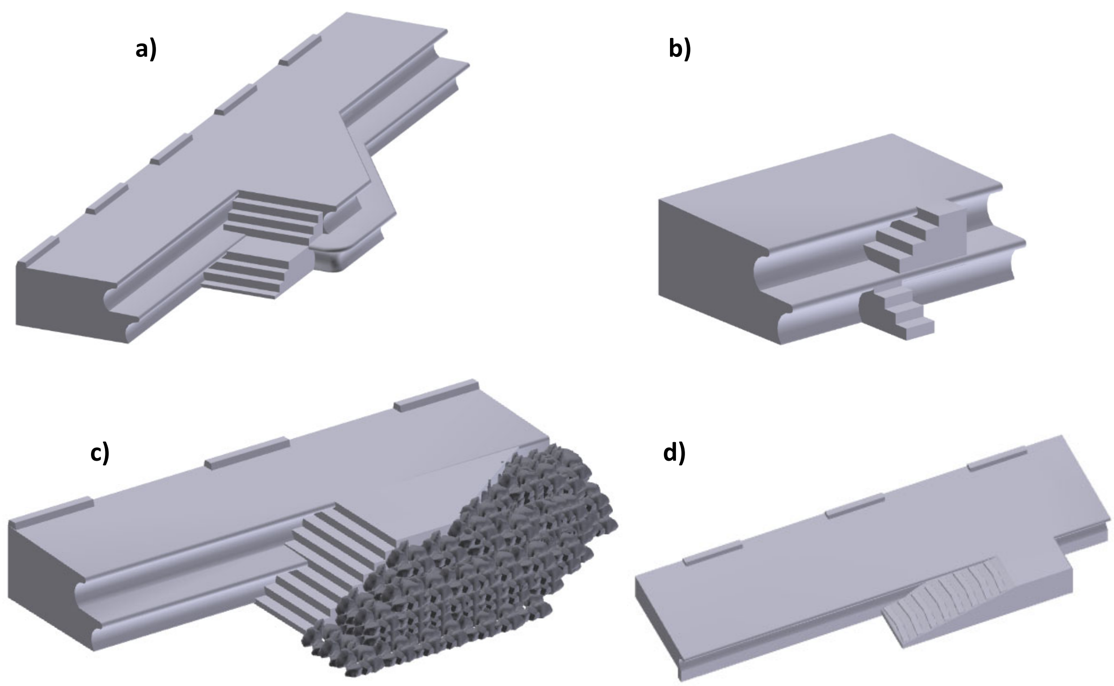


Figure 2-6: Beach access designs a) standard steps in double curved wall, b) mini steps in double curved wall, c) standard steps as transition between curved wall and revetment, and d) boat ramp access in single curved wall. [Source: Design Features Report (Stantec 2018b)].

2.2.5 Existing structures on the seaward side of Marine Drive

There are three existing bus shelter structures positioned on the existing road shoulder of the north-bound lane (Lowry Bay, York Bay, Mahina Bay) and two privately owned structures (boat sheds) above the CMA (Lowry Bay). All existing private structures are to be retained and unaffected by the

Project. The bus shelters are proposed to be relocated within each embayment to optimise shared path width and position.

The proposed seawalls intentionally integrate with the existing seawalls and coastal defences, especially beneath the Lowry Bay boat sheds. This is to ensure no detrimental effect on the existing defences or private structure.

These details will be refined through the detailed design stage.

2.3 Construction methodology

The final construction methodology will be developed by the Contractor once consent conditions are confirmed and further design has been undertaken.

At this stage, the construction methodology within the Design Features Report (Stantec, 2018b) includes guidance on the construction programme on a bay-by-bay basis, the activities necessary to construct the shared path (e.g., excavation, demolition, construction) and the management of the construction activities to minimise environmental effects.

The text below are points relevant to coastal processes that are extracted from the indicative construction methodology within the Design Features Report (Stantec 2018b).

2.3.1 Construction programme

HCC intends the shared path construction to be spread over six years, commencing in 2018.

The intention is to stage the construction of the shared path per bay, and that for each stage that a bay is completed in its entirety. Currently it is proposed to complete Windy Point first (south from Days Bay to Eastbourne, chainage 4990 to 5500), followed by Point Howard/Sorrento (chainage 530 to 1150), and then Lowry Bay (chainage 1150 to 1960), over 3 separate financial years. This will be followed by the other bays.

Each section is likely to take about 3-6 months to complete depending on the bay length, complexity, weather/wave windows and extent of the particular works per bay.

Works will generally be undertaken during daylight hours except where operations are being carried out at low tide or on the road that will require work to be undertaken during off-peak hours at night.

A full construction programme will be incorporated into the Construction and Environmental Management Plan (CEMP) to be prepared as a condition of this consent.

2.3.2 Construction activities

<u>Excavation and Demolition</u>

There are sections of the seawall where excavation will be necessary within the CMA e.g., where the toe of a seawall is to be embedded into the substrate. This will occur for the construction of the single, double and triple curve walls and the cantilever retaining walls supporting the road to the rear of the revetment sections. Excavation within the CMA will also be necessary to accommodate the foundations for the boat ramps and access steps and to toe-in the base of the revetment treatments. No excavation is anticipated for placement of the rock layers of revetment (outside of the toe) due to rock/gravel substrate.

The following tasks will be typically carried out:

- Demolishing the existing seawall as necessary to allow for construction of the new wall. In some instances, the complete removal of the existing seawall may be required to provide access to construct the new wall. The demolition will be undertaken within a silt-fence or behind the new seawall.
- Demolition and removal of the existing wall would be undertaken using an excavator and/or excavator mounted breaker. The use of the excavator on the beach would be minimised to limit damage to the beach area. The excavator would not be stored overnight nor maintained or refuelled in the CMA. Excavator access to the beach will be via existing accessways (i.e., boat ramps) with excavators tracking over steel plates to minimise disturbance to beach materials.
- The area of disturbance would be kept to the minimum to undertake the construction. Where there is adequate space, machinery would work from the road verge rather than from the beach/foreshore, meaning that there will be less area outside of the direct excavation zone that is subject to damage.
- Concrete materials are unlikely to be competent for recycling into the new seawalls. All waste materials will be removed from site to an appropriate landfill site.
- There will be no exposure of non-native backfill material to sea (to be enclosed by seawall and silt fences).

Figure 2-7: Shallow excavations and reinforcing at York Bay. [*Source*: Design Features Report (Stantec 2018b)].

- Whilst excavation will generally be shallow (<1 m, see Figure 2-7) for the majority of the Project, in some beach locations the site investigations (Stantec 2017) have indicated seawall foundations may need to extend down up to 5 m below current beach level in order to reach material of acceptable bearing capacity, whilst ensuring the design is not compromised (undermined) by the long-term effects of coastal erosion or short-term scour during prolonged storms. These areas of deep excavation include Sorrento Bay (50 m), Lowry Bay (585 m), York Bay (450 m), Mahina Bay (220 m), Sunshine Bay (250 m). These deeper foundations will utilise traditional deep foundation techniques such as reinforced concrete cut-off walls, sheet-piling, or bored or driven reinforced concrete piles as required, depending on depth and loading on the foundation. Details will be provided in the CEMP for the specific sections of seawall.

- Excavated beach material is to be stockpiled nearby and replaced on the beach after construction of each section of wall as appropriate. This will only relate to suitable natural weathered material and any non-native material will be removed to landfill.

Construction

Construction of the concrete components of the seawalls will be undertaken in situ. This has been adopted for this project as it is considered to provide a far better engineering solution from a constructability perspective than precast construction, in particular when considering the length of the project and the potentially difficult horizontal and vertical construction challenges associated with this site. This method of construction has also been proven to work well during the construction of the previously constructed York Bay section.

- The environmental challenges of in situ-concrete construction are mainly associated with the risks of releasing of cementitious products into the aquatic environment which is detrimental to intertidal ecology.
- In general, the in situ-founding material is competent weathered rock and the installation for the foundation base would be completed using an excavator and/or breaker either from the road verge or from the beach.
- The curved walls will be in situ-reinforced concrete with the same curve proportions as the double curve seawall installation in York Bay. The number of curves will be dictated by the height from beach level to cycleway level. The curves will be constructed using prefabricated formwork (e.g., Figure 2-8). The void to the rear of the curved façade will be backfilled with no-fines mass concrete. Walls will be formed in 'lifts' to aid construction and minimise time in the intertidal zone.

Figure 2-8: Example of in situconstruction methodology at York Bay. [*Source*: Design Features Report (Stantec 2018b)].

- The rock revetment comprises two components of construction; a reinforced concrete cantilever retaining wall supporting the cycleway/roadway, followed by placement of the rock protection material seaward of the cantilevered wall. The concrete element will likely to be poured concrete in situ as per the curved seawalls. Following this, the rocks will be placed in layers (bedding layer, secondary armour and primary armour) in front of this structure on a geofabric layer (refer to Figure 2-2). Geofabric may only be necessary when placed on unconsolidated sediment (beach, cobbles and gravels). Placement of geofabric over rock platforms may not be required as it will depend on the bedding material. If geofabric is placed, the edge and tails of the geotextile is to be hidden beneath the rock.
- Working within the tidal zone poses constraints on construction zones and concrete pours. Shoring will be required at some locations to enable construction to take place in a timely and environmentally acceptable manner. The location, type and depth of shoring to be used will be determined in the CEMP.
- Up to 20 m lengths of seawall would be under replacement at any one-time to retain small increments of potential discharge of sediments and reduce the risk of wave overtopping.
- Careful consideration and multi-day forecasting of tide, wind and wave conditions would be needed to prevent overwhelming of defense during construction. This will form part of CEMP. This is essential given the known sensitivity and frequency of overtopping, and the risk of cementitious materials being washed away during storm events.

<u>Sediment control</u>

The main risk to the intertidal environment during the construction phase is potential sediment-related effects as well as the accidental release of water contaminated with cementitious products. Sediment generation would be kept to a minimum through the use of crushed material that is clean of fines in the construction of the widened road. Sediments introduced as part of the construction would not incorporate fines smaller than sand particles, to promote quick settling of suspended particles.

Any excavation in areas where it is predominantly gravel/sand beach zone (as opposed to the rocky shore areas) would also be undertaken using methods that cause the least amount of sediment to be released from the construction area. This would include some form of bund that will effectively contain and isolate the construction area from the incoming tide until construction is completed. This bund could be built from clean beach gravel sourced from the excavated area, or could be achieved via the use of sheet piling. Such a structure would need to be large enough to allow construction to continue 'in the dry' while the tide is in and strong enough to withstand waves and the incoming tide.

In areas where the material in the construction footprint is of larger material (i.e., rocky shore habitats) or where seawall works occur close to the mid-tide mark, alternative sediment control devices will be provided. These may include sand filled geotextile containers or tubes (sand to be locally sourced) that can be easily removed following completion of the works.

Earthwork and construction activities would be sensitive to tide timing and tidal height to avoid movement of sediment containing fine sediment in a wet environment. Storm events will very likely overtop any sediment control mechanism used. During overtopping sediment will likely be shifted into the excavated seawall footing area. A site plan for removal of temporary sediment ingress would be developed. Site management would monitor weather conditions and forecasts to anticipate any weather, swell and high tide events that may lead to high seas and plan mitigation measures accordingly. These details will be addressed in the CEMP for the specific sections of seawall.

Dewatering of excavations

Excavations will need to be dewatered to enable foundations for the seawall and revetment to be constructed. All water from the excavations is to be treated for sediment and cementitious products before being discharged either to the CMA or through the stormwater system.

Dewatering is typically carried out by installing a pump system in the trench/excavation. The dewatered fluid would be pumped to a settlement tank or a large container (such as a shipping container) where it is retained for the length of time required for sediment to settle. This varies depending on factors such as dewatering rates and what type and how much sediment is present in the dewatered water. Ideally, the concentration of sediment in the water will be less than the permitted activity standards of 0.5 g/m^3 in the operative and proposed regional plans for a discharge to the stormwater system. The construction management and timing with weather condition is expected to reduce risk of uncontrolled sediment release during times of wave ingress to excavations.

2.3.3 Beach Nourishment

The construction methodology of beach nourishment is addressed in the Beach Nourishment Report (Appendix F of the Project AEE) while recognising that the construction planning will be done by the Contractor awarded to deliver the project.

In summary, the construction plan is to place imported beach compatible fill (size grading, colour, density, < 2% fines) on the beach by end-tipping into discrete location on the three priority beaches and spreading by hydraulic excavator. The fill will not be spread over the whole bay to minimize tracking and disturbances of the existing beach, rather it will be placed in discrete locations to form a wide bench and allow natural wave processes to sort, disperse and reshape into a new equilibrium. The initial placement area will be selected to avoid stormwater outlets as well as being as distant as possible from areas of sea grass.

2.3.4 Construction management

A Construction Environmental Management Plan (CEMP) will be prepared for the various stages of the Project. The CEMP will be a condition of the consent and will be updated and modified as appropriate once a contractor is appointed.

The CEMP, its subplans, and other site-specific environmental management plans (such as landscape and urban plan) for locations along the route, are to be consistent with and complement the AEE report, and will be developed in accordance with the proposed consent conditions.

Once the conditions have been confirmed through the consenting process, the CEMP will be prepared in conjunction with the contractor prior to works commencing. A separate CEMP will be prepared for each section of work (individual work package).

3 Assessment Methodology

This report describes the existing coastal marine environment within Wellington Harbour and the Project area, focusing on coastal physical processes, the effects of coastal hazards and climate change and the potential effects on coastal processes that may arise from construction and long-term operation of the Project.

There are no quantitative assessment criteria for assessing the degree of effects on hydrodynamic and sedimentation processes or beach geomorphology, so this assessment relies on expert appraisal (using a coastal expert and reviewer), supported by desktop research and field observations. Detailed numerical or physical modelling of the environment may also be used.

The investigations into assessment of effects on coastal physical processes of the Eastern Bays Shared Path seawall replacements were undertaken through a mix of:

1. desk-top reviews of previous reports, field data and aerial photography, along with discussions with long-term residents
2. walkover survey
3. assessment of potential effects of the Project based on expert appraisal, informed by the above analyses and information, including the analogue of long-term morphological change observed with the present seawalls and aligned with statutory requirements.

Relevant assessment criteria for discretionary activities related to coastal physical processes in the CMA (as examples):

- Cumulative effects on the coastal environment.
- Preserving natural character e.g., morphology, natural sediment substrate, and natural water and sediment movement patterns.
- Extent to which structures and embankments can cause erosion and/or siltation.
- Extent to which provision has been made in the design of the proposed embankments to minimise restrictions on natural movement of coastal water or floodwaters, including provision of culverts, floodgates, or bridges.
- Effect of climate change, particularly sea-level rise (SLR).

3.1 Desktop

The desktop analyses undertaken for HCC covering the existing coastal environment, included a review of the following information sources:

- Previous bathymetric surveys, along with geological surveys of the land and seabed.
- Past and recent field-instrument deployments and analysis, primarily for AEE analysis of nearby projects on the wave and sediment transport patterns within Wellington Harbour entrance, and the body of Wellington Harbour.

- Modelling of wave conditions in Wellington Harbour, including locally-generated wind waves and propagation of ocean swell into the harbour. Includes review of extreme conditions suitable for engineering design.
- Modelling of currents, sediment plumes and the deposition footprint arising from sediment discharges within the harbour.
- Research papers or reports on coastal physical processes, such as sediment transport along the Eastern Bays coastline, the formation of Point Webb (Eastbourne), inner-harbour seabed sediments, historic changes to Eastern Bays beach environment.
- Review of extreme coastal hazard conditions including storm-side elevation, wave conditions and wave overtopping occurrences.
- Long-term monitoring data for winds (MetService station at Wellington Airport) and waves (NIWA wave buoy at Baring Head operated on behalf of Greater Wellington Regional Council).
- Synthesis of climate change effects on the above environmental parameters.

3.2 Assessment criteria, national policies and regional plans

See the Project AEE Consenting Scope Report (Stantec 2018c) for full description and regulatory context of the Project, including revision relevant Resource Management Act (RMA) policy statements and plans. Here we outline the regulatory context for coastal physical process:

- the New Zealand Coastal Policy Statement 2010 (NZCPS)
- the Regional Policy Statement (RPS) for the Wellington Region
- the Regional Coastal Plan (RCP)
- the Proposed Natural Resources Plan (PNRP)
- the Hutt City District Plan
- the National Environmental Standard (NES) for assessing and Managing Contaminants in Soil to Protect Human Health.

The sections below are specific additions to the above regulatory criteria which concern the coastal physical processes AEE.

3.2.1 Resource Management Act (1991) requirements

The key relevant sections of the RMA in relation to coastal physical processes are:

- Part II, Section 6(h) — Shall recognise and provide for the following matters of national importance: - “the management of significant risks from natural hazards”.
- Part II, Section 7(i) — In achieving the purpose of the Act, “… shall have particular regard to … the effects of climate change”.
- Sections 5(2)(c) and 17— A duty to avoid, remedy, or mitigate any adverse effect on the environment arising from an activity ….

Under Section 3, actual or potential effects includes any positive or adverse effect, temporary or permanent effect, any past[6], present or future effect and any cumulative effects. In this Report, actual or potential effects on coastal physical processes alone have been assessed as:

- Negligible.
- Minor.
- Moderate (between minor and adverse).
- Major or adverse effect.

Central to the purpose of the Act (section 5) is sustainable management in managing the use, development and protection of natural and physical resources which enables communities to provide for social, economic and cultural wellbeing, while …. (c) avoiding, remedying or mitigating any adverse effects of activities on the environment.

3.2.2 NZ Coastal Policy Statement (2010)

The key relevant Objectives and Policies in the NZCPS-2010 which relate to either assessment of effects on <u>coastal physical processes</u> or management of the effects of <u>coastal hazards and climate change</u> on the Project are:

- Objective 1— Safeguard the integrity, form, functioning and resilience of the coastal environment …. By maintaining or enhancing natural biological and physical processes in the coastal environment and recognising their dynamic, complex and interdependent nature …
- Policy 10 (Reclamation and de-reclamation). Sub-section (1) Avoid reclamation … unless there are no practicable alternatives; and the reclamation will provide regional or national benefit. Sub-section (2) where reclamation is considered a suitable use … have particular regard to … the effects of climate change over 100 years, the use of appropriate materials (avoiding contaminated materials) …, the ability to remedy or mitigate adverse effects …, the ability to avoid consequential erosion and accretion. Sub-section (3) consider the extent to which the reclamation and intended purpose would provide for efficient operation of infrastructure …
- Policy 22 (Sedimentation)— Requires that … development will not result in a significant increase in sedimentation in the CMA …
- Policy 23 (Discharge of contaminants)— Sub-section (1) applies to the discharge of contaminants (which generally includes fine silts and muds) and the need to avoid significant adverse effects after allowing for reasonable mixing.
- Policies 24 and 27 (Coastal hazards, climate change and protection of significant existing development)— Requires assessment of the effect of coastal hazards and climate change over at least a 100-year timeframe, taking into account national guidance and the best available information. Policy 27 also recognises that hard protection structures may be the only practical means to protect existing

[6] Past and any continuing effects from the historic seawall works on the CMA are not considered in this Report, as reclamations are granted in perpetuity and become land.

infrastructure of national or regional importance, and to meet the foreseeable needs of future generations.

3.2.3 Regional Coastal Plan – Wellington

The Regional Coastal Plan (RCP), which has been operative from June 2000, provides objectives, policies and rules for managing activities in and above the CMA, which is the marine waters and intertidal land below Mean High Water Springs (MHWS).

The relevant CMA boundary for this Project is the landward extent of the CMA calculated by LINZ along the Eastern Bays foreshore. This is demarcation is generally positioned along the existing coastal defense structures supporting the road, or along the mid-upper beach area. Detailed maps of the MHWS boundary are shown in the Base Information plans (Appendix M of the consent application) with MHWS elevations shown on each of the Preliminary Design Plans (Appendix N of the consent application).

The RCP also specifies designated areas around the Wellington region of either significant or important conservation value. There no areas of important conservation value noted within the RCP for the project area.

3.2.4 Sea-level rise guidance

Under Policy 24 of the NZCPS, coastal hazards including climate change effects are to be assessed over at least 100 years, which for this Project effectively means out to 2120. Policy 24 also requires assessments to take into account national guidance and the best available information on the likely effects of climate change on the region.

The operative coastal guidance provided by the Ministry for the Environment (MfE) is the 2017 edition of *Coastal Hazards and Climate Change – A Guidance Manual for Local Government*.[7]

In its most recent report released in 2014, the Intergovernmental Panel on Climate Change (IPCC) projects that global sea-level rise by 2100 will be between around 0.3 m and 1.0 m above the 1995 level, depending on the amount of future greenhouse gas emissions. Over a shorter time-frame, up to 2060, there is less uncertainty, and the IPCC projects a narrower range of sea-level rise of 0.2 to 0.4 m. However, there is considerable uncertainty in the magnitude and rate of sea-level rise towards the end of this century and beyond, depending substantially on the actual carbon emissions trajectory in coming decades and the non-linear effect of instabilities occurring in the melting of polar ice sheets.

The 2017 MfE guidance provides four scenarios of sea-level rise to test land-use plans and projects against, to ensure sufficient flexibility is provided and to avoid locking in investment or path dependency that could occur trying to choose a "best estimate" (or potential over-design for a given timeframe if a "worst-case estimate" is adopted). A spread of sea-level rise scenarios for New Zealand are tabulated in Figure 3-1 from MfE (2017), based on projections for different representative concentration pathways (RCPs)[8] by IPCC.

[7] http://www.mfe.govt.nz/sites/default/files/media/Climate%20Change/coastal-hazards-guide-final.pdf

[8] IPCC assessed 4 RCPs, ranging from RCP2.6 (where global emissions are reduced to a net of zero by 2075) to RCP8.5 (which represents continued high carbon emissions with little effective reductions in global emissions).

NZ SLR scenario Year	NZ RCP2.6 M (median) [m]	NZ RCP4.5 M (median) [m]	NZ RCP8.5 M (median) [m]	NZ RCP8.5 H+ (83rd percentile) [m]
1986–2005	0	0	0	0
2020	0.08	0.08	0.09	0.11
2030	0.13	0.13	0.15	0.18
2040	0.18	0.19	0.21	0.27
2050	0.23	0.24	0.28	0.37
2060	0.27	0.30	0.36	0.48
2070	0.32	0.36	0.45	0.61
2080	0.37	0.42	0.55	0.75
2090	0.42	0.49	0.67	0.90
2100	0.46	0.55	0.79	1.05
2110	0.51	0.61	0.93	1.20
2120	0.55	0.67	1.06	1.36
2130	0.60*	0.74*	1.18*	1.52
2140	0.65*	0.81*	1.29*	1.69
2150	0.69*	0.88*	1.41*	1.88

* Extended set 2130–50 based on applying the same rate of rise of the relevant representative concentration pathway (RCP) median trajectories from Kopp et al, 2014 (K14) to the end values of the Intergovernmental Panel on Climate Change Fifth Assessment Report (IPCC AR5) projections. Columns 2, 3, 4: based on IPCC AR5 (Church et al, 2013a); and column 5: New Zealand RCP8.5 H^+ scenario (83rd percentile, from Kopp et al, 2014). Note: M = median; m = metres; NZ = New Zealand; SLR = sea-level rise. To determine the local SLR, a further component for persistent vertical land movement may need to be added (subsidence) or subtracted (uplift).

Figure 3-1: Decadal increments for projections of SLR for New Zealand. In metres above 1986–2005 baseline [Source: MfE 2017].

By 2070 (50 years from 2020), sea-level rise could range from 0.32 to 0.61 m (or higher), while by 2120, rises could range from 0.55 to 1.36 m (or higher), relative to MSL averaged over the period 1986–2005.

For this Project, sea-level rise values are used to evaluate the proposed seawalls by 2070 (50-year design life), and by 2120 (100-year assessment NZCPS-2010). The stress-test refers to assessing the effects of wave overtopping hazards and storm-tide (tidal elevation + storm surge) elevation over the long term.

The design life of the proposed coastal defence structures is 50 years, or to 2070. However, after 50 years have elapsed we understand there is currently no intention to relinquish the seawall and shared pathway to the rising sea-level with more frequent inundation events. Consequently, the longer-term assessment out to 2120 will be generalised to assume any replacement structures are similar in form to that currently proposed.

The use of 1.0 m SLR at 2120 is consistent with MfE (2017) guidance to use a transitional SLR allowance for planning purposes where a single value is required at a local or district scale while in transition to developing a dynamic adaptive pathways planning (DAPP) process that needs to be comprehensive across local communities of Eastern Bays, utilities and infrastructure for the longer term (given sea level will continue rising for several centuries). Figure 3-2 illustrates the transitional SLR allowances where this Project broadly fits into Category C, although elements such as the boat access or public seating may be category D.

Category	Description	Transitional response
A	Coastal subdividion, greenfield developments and major new infrastructure	Avoid hazard risk by using sea-level rise over more than 100 years and the H+ scenario
B	Changes in land use and redevelopment (intensification)	Adapt to hazards by conducting a risk assessment using the range of scenarios and using the pathways approach
C	Land-use planning controls for existing coastal development and assets planning. Use of single values at local/district scale transitional until dynamic adaptive pathways planning is undertaken	1.0 m SLR
D	Non-habitable short-lived assets with a functional need to be at the coast, and either low-consequences or readily adaptable (including services)	0.65 m SLR

Figure 3-2: Minimum transitional New Zealand-wide SLR allowances and scenarios for use in planning instruments where a single value is required at a local/district scale while in transition towards adaptative pathways planning using the New Zealand-wide SLR scenarios. [*source*: MfE (2017)].

The SLR values adopted for assessing climate-change effects on this Project are 0.5 m by 2070 (just above the RCP8.5 median projection), 1 m (just below the RCP8.5 median projection) and 1.35 m by 2120 for the RCP8.5 (83rd percentile of RCP8.5 range) projection. These values are relative to the mean sea level defined over the 1986-2005 period which is 0.164 m WVD-53 (see section 4.4.1) for Wellington.

Table 3-1: Sea-level rise values used in this report. Values relative to MSL averaged over the period 1986–2005.

Year	Sea-level rise (m)
2070	0.5
2120	1.0
2120	1.35

This sea-level rise will increase the frequency of wave overtopping and coastal inundation (Gorman et al. 2006). With only 16 cm of sea level rise (within 50 years) the frequency of the present day '100-year storm' in Wellington will have increased to once per year (Stephens 2015, PCE 2015).

Coastal Hazards

MfE (2017) states that "the other effects of climate change on coastal hazards will be secondary to ongoing sea-level rise, with the next most important effect being climate change sensitivity to wave heights for the exposed open coast, where wave runup is critical to hazard trigger or adaptation threshold levels for inundation or erosion".

MfE (2017) provide the following guidance to undertake sensitivity testing for coastal engineering projects and for defining coastal hazard exposure areas out to 2100, using:

- a range of possible future increases across New Zealand of 0–10 per cent for storm surge out to 2100
- a range of possible future increases across New Zealand of 0–10 per cent for extreme waves and swell out to 2100
- changes in 99th percentile wind speeds by 2100 and incorporating these for the relevant RCP scenario from Ministry for the Environment (2016) on climate change projections, to assess waves in limited-fetch situations, such as semi-enclosed harbours, sounds, fjords and estuaries.

These further assessments of climate change effects should be considered within sensitivity testing and detailed design.

4 Existing environment

This section provides a description of the existing coastal environment, relative to which the effects of introducing the proposed shared-path upgrade are assessed. It covers the geologic history, geomorphic and hydrodynamic setting, following by a description of coastal physical processes along the Eastern Bays. This description is based on previous studies supplemented by walkover inspections.

Note that while Days Bay is not included within the Project area (because there is adequate provision for walking and cycling) we have included it within the description of the existing environment because it is one of the few bays between Petone and Eastbourne that has been studied in the literature. Days Bay is also representative of the other embayments and provides background material for the remainder of the Project assessment.

In considering environmental effects of the Project, the existing environment is the baseline that is considered for assessing effects, including the existing seawalls and revetments. The impacts from the proposed structures within the adjoining CMA will be considered for the coastal permit applications.

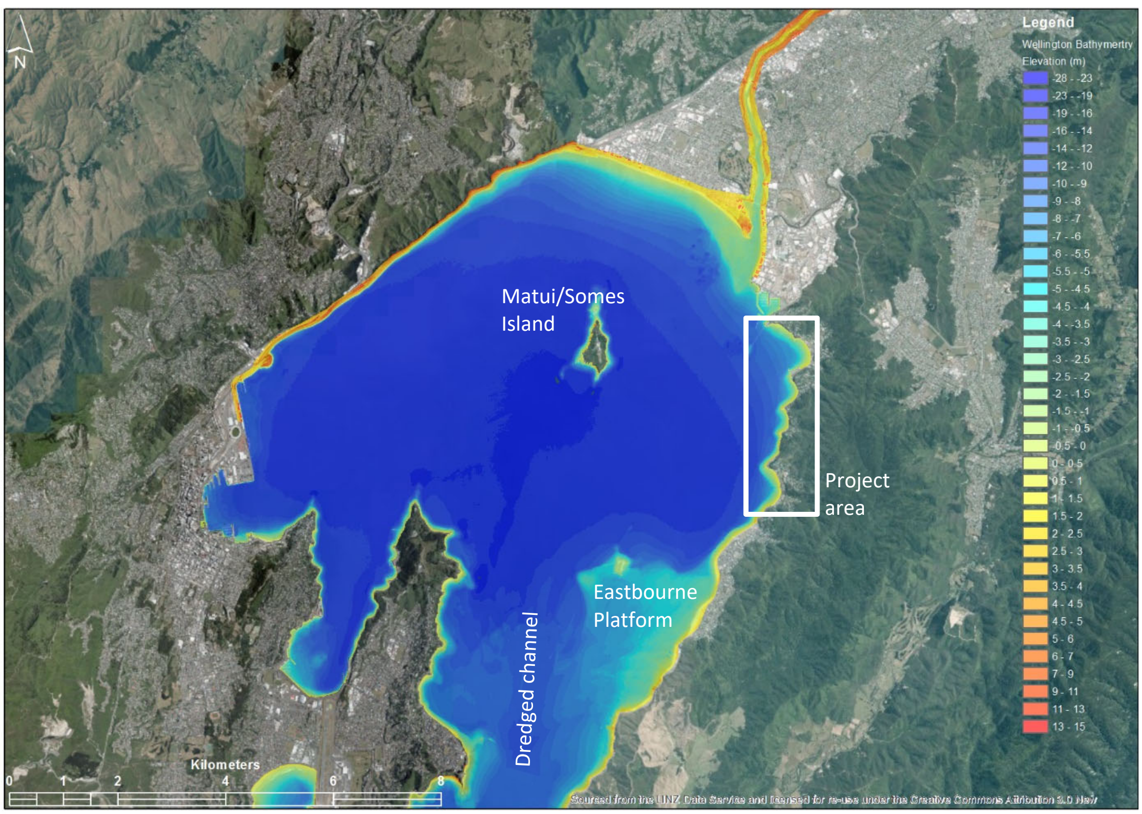


Figure 4-1: Digital terrain model of Wellington Harbour and near-shore areas used for modelling. Elevation relative to WVD-53. [*Source*: Terrestrial LiDAR - Wellington City Council, Hutt City Council, Greater Wellington Regional Council; Terrestrial survey - AECOM; and bathymetry - NIWA, Hutt City Council].

4.1 Geologic and topographic setting

The Eastern Bays project area occupies approximately 5 km of Wellington Harbour's eastern shoreline (Figure 1-1). Wellington Harbour is a roughly circular basin with a maximum width of 11 km, surface area of 85 km^2, maximum depth of 31 m and average depth of 14 m (Figure 4-1[9]). The Harbour is connected to Cook Strait via the narrow harbour entrance passage (1.7 km wide near Point Dorset) with maximum navigable depth actively dredged to 15 m[10] (Heath, 1977). The channel runs along the centre of the entrance with an irregular, pinnacle-studded western flank, and a shallow (<5 m deep) submerged platform on the eastern side that reaches a maximum width off Eastbourne (Lewis and Carter, 1995).

4.1.1 Seismic

The Wellington Region is bisected by several active faults which have fashioned the steep hillside topography, opened Wellington Harbour to Cook Strait, and continue to reshape the area.

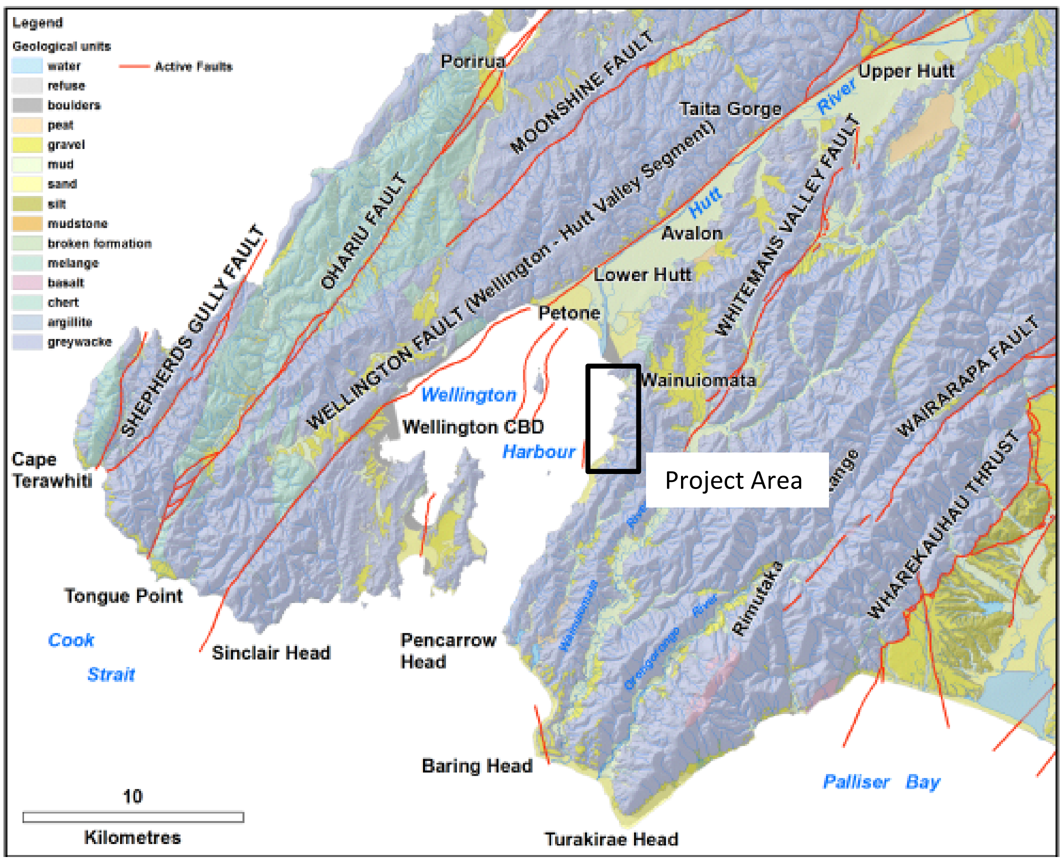


Figure 4-2: Prominent earthquake faults near Wellington. [*Source*: Townsend et al. (2015)].

The prominent Wellington Fault is one of the major faults close to the Project area (Figure 4-2). The fault's surface trace is positioned approximately 250-400 m offshore from the north-western edge of the Harbour (between Kaiwharawhara and Petone). The fault trace passes beneath Petone Beach about 400 m northwest from Petone Wharf and continues along the western side of the Hutt valley (GNS New Zealand Active Faults Database, http://data.gns.cri.nz/af/). This segment had a lateral slip-

[9] This information was combined from multiple survey sources for a nearby project (see Appendix A, Allis et al. 2017).

[10] Note Centreport is seeking resource consents to deepen the navigable channel to between 16.5 m and 17.2 m below Chart Datum.

rate of 6.0–7.6 mm per year for at least the last 140,000 years with the north-western side of the fault is uplifted (i.e., the Porirua side) (Van Dissen et al. 1992b). Following a Wellington Fault rupture, the low-lying Lower Hutt Valley floor and Eastern Bays will be approximately 1 m lower in elevation than present through co-seismic subsidence (Begg et al. 2002, Townsend et al. (2015), also [11]) as seen in Figure 4-3.

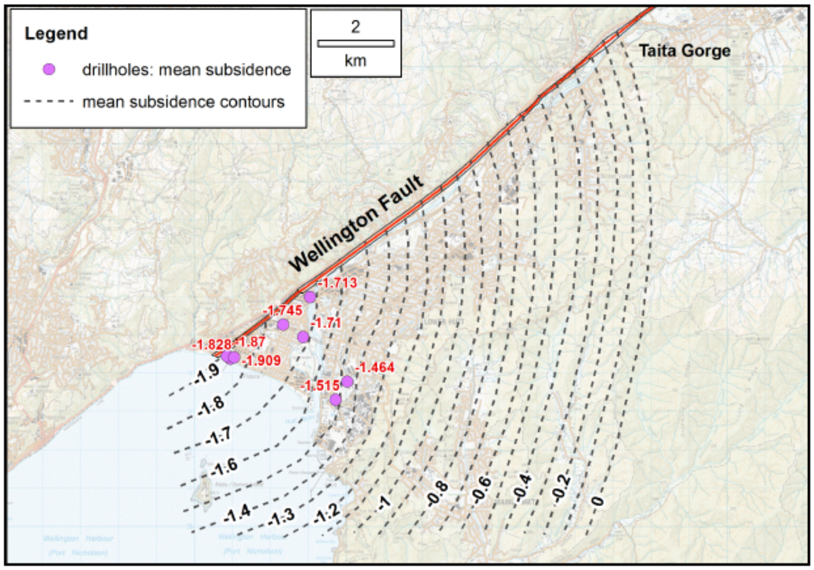


Figure 4-3: Drill hole locations and co-seismic subsidence (in metres) in the Hutt Valley and Eastern Bays resulting from rupture of the Wellington Fault. [*Source*: Townsend et al. (2015)].

Following a strong earthquake, either centred on the Wellington Fault, or elsewhere, significant liquefaction and/or ground shaking amplification is anticipated in the Eastern Bays area (Van Dissen et al. 1992a).

The existing Marine Drive corridor was built on a raised beach and wave-cut rocky platforms (Figure 4-4) which were uplifted and tilted by 1.2-1.5 m during the 1855 Wairarapa earthquake (see Wairarapa Fault, Figure 4-2). This earthquake is the most severe earthquake to have occurred in New Zealand since systematic European colonisation began in 1840 and was centred 25 km east of the Eastern Bays (McSaveney et al. 2006).

[11] https://www.eqc.govt.nz/research/research-papers/coseismic-subsidence-lower-hutt-valley-rupture-wellington-fault

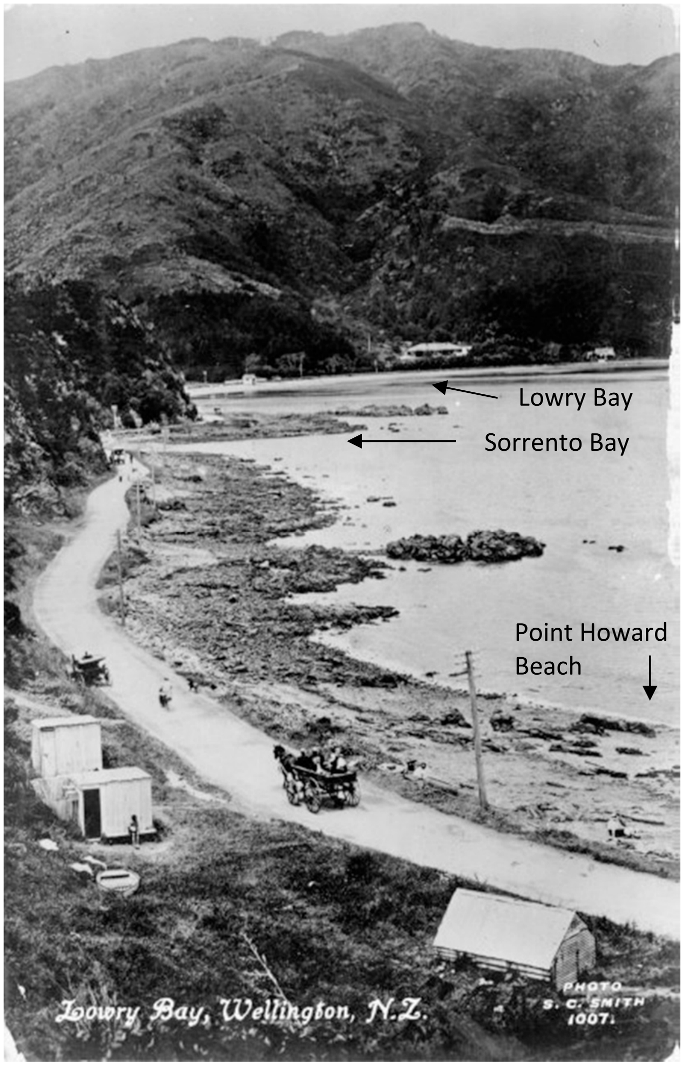


Figure 4-4: Point Howard Beach, Sorrento Bay and Lowry Bay, ca 1920. View from on Point Howard hillside. Annotations indicate present day beach names. Note broad rocky platforms on which the road is situated, which have since been partly covered by the widened road. [*Source*: 1/2-030788-F Alexander Turnbull Library. *Photographer*: Sydney Smith].

4.1.2 Long-term vertical land motion trends

It is difficult to provide a definitive long-term trend of vertical land motion for any site in the Wellington Region, largely due to the effects and ongoing influences on crustal movement of the recent earthquake events since 2013. Bell et al. (2018) analysed the vertical land motions and found that the effect of coseismic displacement was subsidence by up to 40 mm, while (ongoing) postseismic displacement is causing ongoing uplift of up to 50 mm to date. In addition, the region is affected by the occasional slow slip earthquake (SSE) that (typically) results in uplift, which has amounted to over 40 mm (Bell et al. 2018).

In the background to the complex coseismic/postseismic/SEE uplift/subsidence trends, the past decade has seen the land mass of the lower North Island subside at rates of 2 – 5 mm/yr at Wellington (Beavan and Litchfield 2012, Bell et al. 2018). If this secular subsidence continues it would need to be factored into sea-level rise rates that apply locally in Wellington Harbour (Section 5.3, MfE

Coastal Guidance, 2017). This relatively rapid subsidence is compounded by the coseismic subsidence associated with Wellington Fault rupture.

4.1.3 Geologic deposits

The hillside bedrock surrounding Wellington Harbour is comprised of Rakaia terrane greywacke sandstone-mudstone sequences and poorly bedded sandstones (Begg and Johnston, 2000). The bedrock has been uplifted in various seismic sequences, with rocky headlands now jutting out into the harbour and fringing the embayments of the Eastern Bays. The bedrock outcrops are "slightly to completely weathered, dark grey-brown, greywacke, siltstone and sandstone, weak to moderately strong, very closely spaced defects" (Stantec, 2017).

Sediments along the wider Harbour shoreline are derived predominantly from the fluvial and littoral sources; the Hutt River and other smaller catchments which deposit most of their sand and fine-sediment load into the harbour basin (van der Linden 1967; Carter and Gibb 1985), and littoral drift of sand and gravel spreading into the harbour entrance via southerly waves (Dahm 2009, Matthews 1980). Shoreline sediments and sediment processes in the Project area are discussed further in Section 4.2.

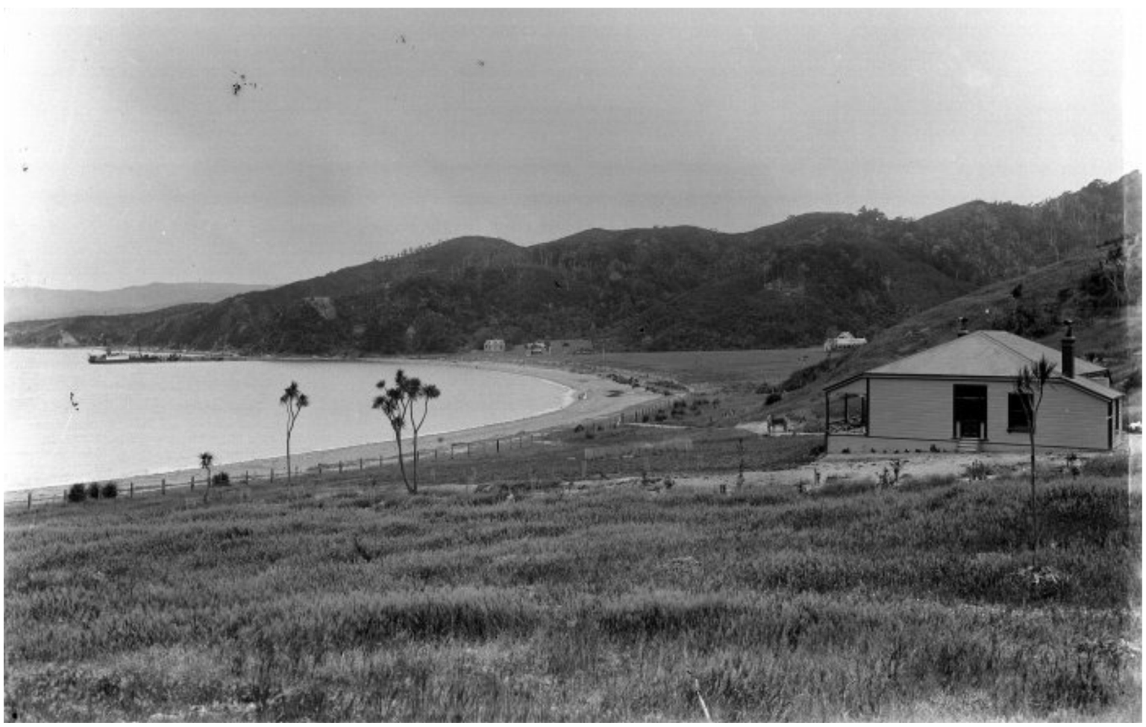


Figure 4-5: Lowry Bay ca 1890s, view North. Note beach backed by wide low backshore in the distance [*Source*: 1/1-020472-G. Alexander Turnbull Library].

The sandy deposits within each bay are Holocene beach deposits consisting of marine gravel with sand, mud and beach ridges (Begg and Johnston 2000). Prior to housing development, each embayment had a sandy and vegetated (tussock grass and flax) backshore behind the beach (e.g., Lowry Bay, Figure 4-5). The present-day beach deposits of the Eastern Bays overly bedrock at various cover depths within each embayment. Recent borehole logging and cone penetrometer tests (CPT) observed beach deposits up to 10 m thick overlying rock (Stantec 2017), with thickness varying from 3.8 metres below ground level (m below ground level – bgl) to 9.2 m bgl at Lowry Bay and from 1.4 m bgl to 8.0 m bgl at York Bay (GHD 2015b, Stantec 2017). The CPT records indicate Holocene beach deposits typically comprise alternating gravels, sands, silts and clays (Stantec 2017). This testing indicates beach deposits are generally thickest in the centre of the bay (and may exceed 10 m thickness in Lowry Bay and York Bay) as aligned with the sub-surface extension of bedrock gully topography, and reduce to a thin (<1 m) veneer where bedrock outcrops approach the surface. The CPT records show subsurface sediments are consistently distributed throughout each embayment (i.e., no prevalence for sub-surface sediment size at north or south) and are predominantly coarse grained (silts, sands and gravels) with a small clay proportion.

The sediments on the Wellington Harbour seabed show fairly uniform textural characteristics of silty-clays with medium diameters between 0.008 and 0.016 mm (van der Linden 1967). There is a gradual transition into the Harbour entrance through zones of sandy and very sandy deposits (Carter and Gibb 1985). Most sediment discharged into the harbour settles to the harbour floor and has contributed to the high sediment accumulation rates. For example, in the NE corner of the Harbour, the seabed has accumulated 4.2 m of sediment since the 1855 Wairarapa earthquake, at rates from 9 to 61 mm/year or an of average 26 mm/year (Goff 1997, Goff et al. 2012). The sedimentation rates are episodic and strongly correlated to seismic events and large Hutt River discharges (Goff et al. 2012). Recent field measurements and hydrodynamic modelling (Allis et al. 2017) observed background levels of suspended sediments strongly were correlated to Hutt River discharges or large waves events, with suspended sediments plumes (e.g., from the Hutt River) rapidly dispersed by wind and waves.

4.2 Existing coastal defences

The existing coastal defences within the project area are varied in design and age. The existing defences were constructed or upgraded at various times since the road was developed in the late 1800s (e.g., Figure 4-4, Figure 4-5), but generally the present walls have been in place since the 1960s with some sections older or newer.

Several descriptions of the existing seawall types have been made. The most recent condition assessment (by GHD in 2016, excluding the area south of Days Bay) reviewed 3150 m of coastal defence from an engineering viewpoint (Table 4-1), whereas EOS (2017) describe fewer categories of the various walls (Figure 4-7) from an ecological viewpoint.

The multitude of designs reflect the longstanding attempts to manage the shoreline and wave overtopping hazards along Marine Drive. The most recent designs are the double and single curved walls (modern) of which there are about 310 m in place (Table 4-1), and were installed between 2012 and 2015. The largest section of the modern double curved wall is at York Bay (Figure 4-6).

Table 4-1: Description of existing coastal defences. Excludes areas south of Days Bay. Summarised from 2016 condition assessment.

Existing coastal defence types (GHD 2016)	Approximate total length (m)
Revetment (riprap rock or concrete blocks)	240
Vertical wall (stone, concrete or combination)	110
Sloping (stone, concrete or combination)	350
Concrete wall (vertical, sloping or with stone insets)	945

Existing coastal defence types (GHD 2016)	Approximate total length (m)
Curved wall (early type, wide or sloped overlay)	550
Double and single curved wall (modern)	310
Gabion baskets	50
Headland (natural rock outcrop)	140
Various other walls	50
Timber boardwalk (Days Bay)	100

The effect of the modern double curved seawall on coastal processes is evident along the southern side of York Bay. These walls were constructed in 2012, and appear to allow continued sediment movement across the rocky platform with small accumulations of sediment showing ongoing sediment transport by waves (Figure 4-6). The absence of any noticeable effect on nearshore hydrodynamics and sediment movement on York Bay beach supports the continued use of this wall type for similar geomorphic embayments along Eastern Bays.

Figure 4-6: Modern double curved seawall at southern York Bay. [*Credit*: 25/12/2017, M. Allis (NIWA)].

Figure 4-7: Categories of existing coastal defences along the Eastern Bays shoreline. Locations as determined by EOS Ecology on 3 May 2016 and 8 June 2017 [Source: EOS Ecology (2017)].

The existing condition was assessed on a rating from excellent to very poor (Table 4-2) reflecting the visual appearance and structural competence as roadway foundation. The total length within each category shows most of the seawalls are at moderate or better condition, but nearly 1 km is poor or very poor condition.

Table 4-2: Existing seawall condition assessment description and total lengths. Lengths as per 2016 assessment by GHD [Source: HCC].

Condition Rating	Description	Total length (m)
Excellent or very good	Sound structure well designed. Well maintained.	370
Good	As previous condition showing wear and tear and minor deterioration of surfaces. Some spalling but with no corrosion staining; needs to be inspected in the medium term. Deterioration causing minimal influence on performance.	960
Moderate	Functionally sound structure but appearance affected by minor cracking, staining and vegetation growth. Deterioration beginning to be reflected in adjacent carriageway.	980
Poor	Structure functioning but with problems due to significant cracking, spalling, loss of stability, deformation and corrosion. Likely to cause a marked deterioration in performance in the medium term. Some asset rehabilitation needed within the medium term.	650
Very Poor	Structure has serious problems and has failed or is about to fail in the near future resulting in unacceptable performance. Minimal life expectancy, requiring urgent replacement or rehabilitation.	190

Note that the condition assessment does not include review of seawall performance regarding susceptibility to hazardous overtopping conditions. Some of the better-condition seawalls are poorly designed and exacerbate wave overtopping (e.g., northern Lowry Bay, see Figure 4-25).

Most of the Eastern Bays shoreline is protected by engineered structures in the form of concrete seawalls and rock revetments (see Section 4.2). These structures have encroached onto the upper beach, and have disrupted the natural sediment transport regime within each bay, particularly during storm events. Over time, the effect of the seawalls (compared to the natural undeveloped state) on beach sediments has been to lower the elevation of the beach (a common beach response with seawall placement), reduce the proportion of fines within the beach material (due to increased wave reflections that winnow out any fine sediment) and subtly change the overall plan shape of the beach altered hydrodynamics and sediment processes. For example, Lowry Bay was formerly a broad sandy beach (see Figure 4-5) and the loss of beach sediment, beach area and beach shape is evidence of these processes arising from seawall construction, reclamation at southern Lowry Bay, and coupled to a probable change to sediment delivery (see section 4.3.2).

Wave overtopping is a common hazard along Marine Drive (see Section 4.6.3). Several of the existing designs of gently-curved or planar concrete slopes are poorly designed to reduce overtopping. The smooth, sloped or vertical seawalls are the least effective at dissipating wave energy. Instead, the structures resist the wave energy by redirecting the water up and over the seawall, with a portion reflected offshore. Reflection creates turbulence, capable of suspending sediments (Bush et al. 2004), thus making them more susceptible to erosion. In a worst-case scenario, reflected energy can interact with incoming waves to set up a standing wave which causes intense scouring of the shoreline (French, 2001). This can cause beach lowering (reduction in elevation of beach surface), which reduces beach amenity value and increases wave loadings on the seawall by allowing larger waves closer to the shore which can lead to increased wave overtopping and frequent nuisance wave splash episodes at higher tides.

4.3 Beach sediment processes

4.3.1 Sediment sources

Figure 4-8 shows the general sediment supply sources into Wellington Harbour.

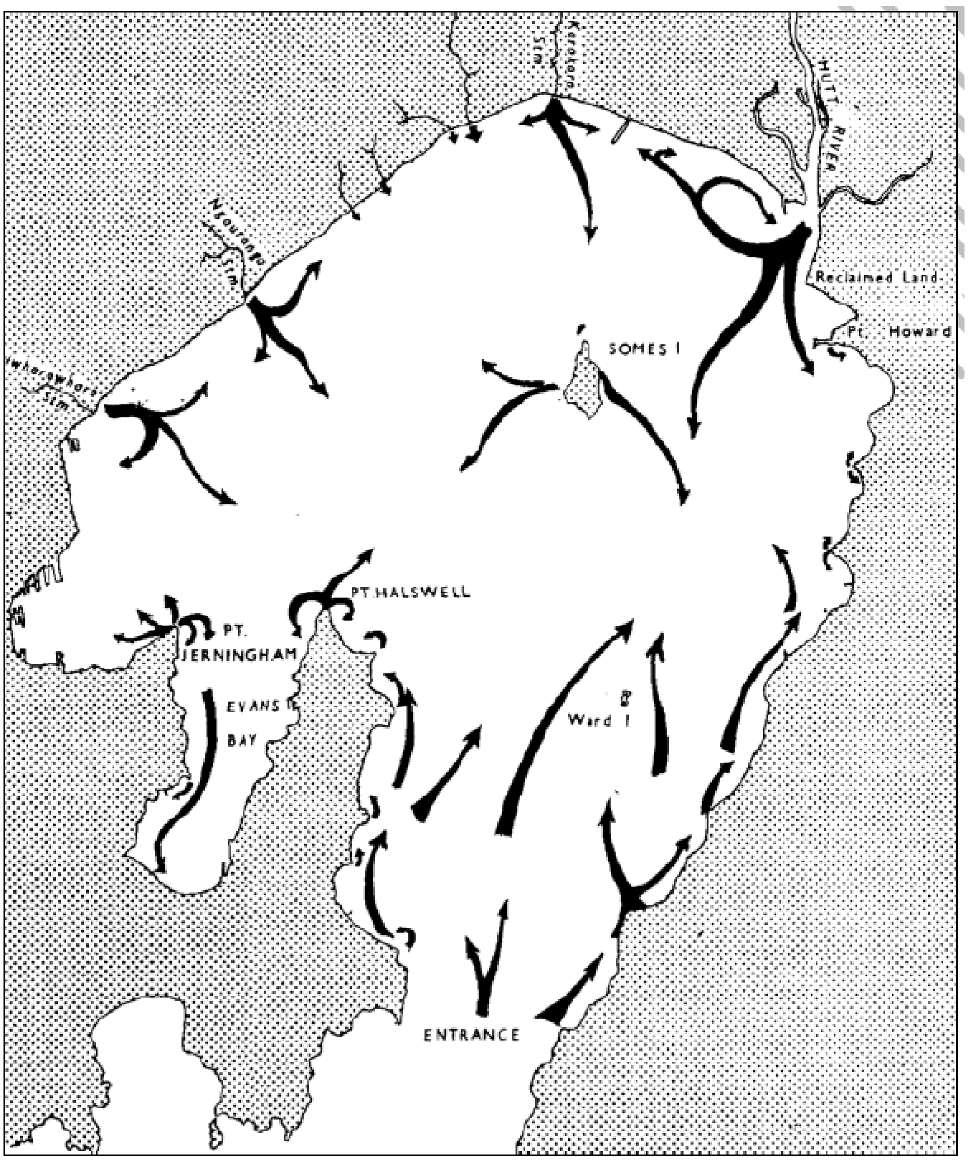


Figure 4-8: Supply and transport directions of sediment in Wellington Harbour. [*Source*: van der Linden (1967)].

The main historical (Holocene period, up to 12,000 years before present) source of sediment supply to the Eastern Bay's beaches would have been the Hutt River (see Figure 4-8) and from outside the harbour mouth. These natural sources were interrupted by human activities including a) the Seaview reclamation in 1955; b) sediment extraction at the river mouth (ongoing for river flood management purposes); and, c) construction of Seaview marina breakwaters in the 1990s (Dahm 2009), and d) construction of southern Lowry Bay reclamation 1969.

No specific studies into the origin of sediments within each embayment were available. However, the available literature suggests that the existing sediment within each bay is derived largely from distant sources with very slow input from local sources (Dahm 2009, Olson 2012) which has reduced over historic time by development and hardening of the hinterland.

- Local sources include local rivers and streams (sands and gravels), slow abrasion/erosion of rocky headlands, along with inner-harbour sources of sand and silt from infrequent uplift of nearshore sandy sediments following seismic events (Dahm 2009).

- Distant sources include the Orongorongo River and Wainuiomata River on Wellington's South Coast. These sands and gravels are transported into the harbour via subtidal wave-driven currents (e.g., sands waves on the seabed) and littoral transport by incoming waves and swell (predominantly gravels) (Carter and Lewis 1985, Matthews 1980).

Given the small contribution of local sediment, the arrival of distant-source sediment signals the importance of regional sediment transport patterns to the beaches of the project area.

4.3.2 Sediment transport

Longshore drift is the main influence on the sediment supply and morphology of the Eastern Bays.

Along the Eastbourne platform, Carter and Lewis (1985) suggest the predominant sediment transport in a northerly direction is a result of a combination of tides, swell, and southerly gales. Although, periodic transport to the south is anticipated in the entrance during ebb (outgoing) tides and northerly-wind wave events. However, these wind waves from the north are not large (due to the limited fetch) and their influence is confined to the shallow, northern reach of the Eastbourne platform (Carter and Lewis, 1995).

This general northerly drift means sediment supply to Project area is closely related to the sediment dynamics of the beaches south of Eastbourne.

South of the Project area, Eastbourne township is located on a coastal plain (cuspate foreland) of alternating sand and gravel sediments (Matthews 1980, Gibb 2005), and has experienced significant accretion along the shoreline between Pencarrow Head and Eastbourne over the last 100 to 150 years. This accretion has covered the original sandy beach environment with gravels (Olson et al, 2012) and is attributed to a pulse of gravels discharged from the Orongorongo River following the 1855 Wairarapa Earthquake. The gravel pulse was transported north with the prevailing littoral drift, and infilled the embayment's between the Orongorongo River and Pencarrow Head from around 1941 and arriving along the Eastbourne shoreline in around 1985/86 and Days Bay in around 2008 (Olson et al 2012, see Figure 4-9). Pebble tracer studies indicate that the deep-water swell waves drive the longshore sediment drift which is estimated at 1.66 km/yr along the Cook Strait shoreline (Matthews 1980a) but varies according to individual beach orientation.

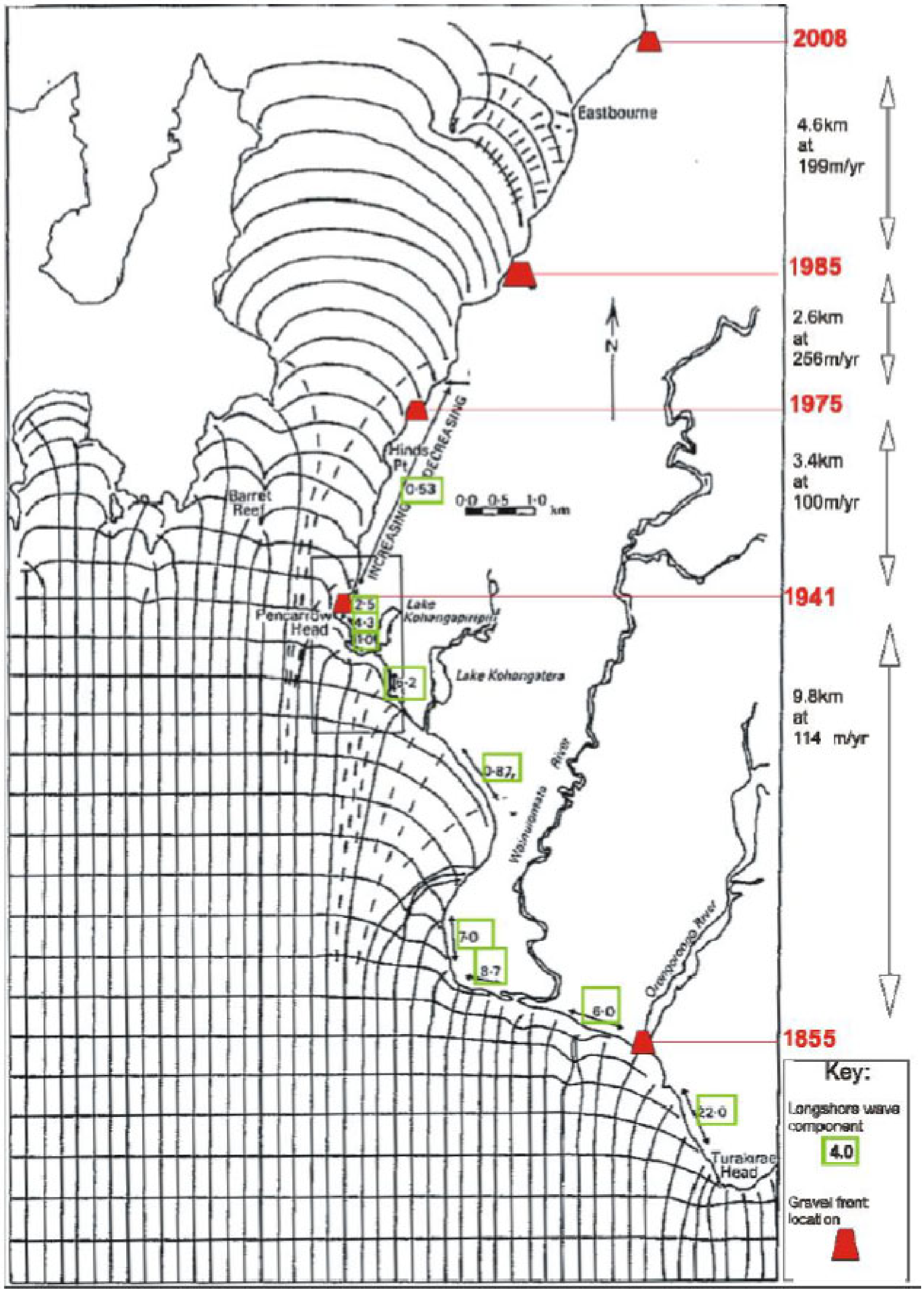


Figure 4-9: Location of the northern extend of the gravel pulse discharged from the Orongorongo River. Showing the relative migration rates of gravel in relation to wave refraction and the reduction in alongshore transport [*Source*: Olson 2009].

Gravels are transported primarily in the intertidal zone from wave swash processes, while sand can move sub-tidally, down to the limit of wave action on the sea bed. This means that sand-sized sediments from the Orongorongo River and the uplifted seabed were able to bypass the rocky headlands and therefore accumulate in the embayments, while gravel-size material had to infill the various embayments to the south before it could be transported into the next bay north. This phasing of sediment arrival between sand and gravel was observed in Robinsons Bay (southern flank of the Eastbourne foreland) and Rona Bay.

Olson et al. (2012) describes the process as: from 1863 to the 1920s Robinson Bay prograded approximately 80 m as a sandy beach; however, the early 1900s were marked by erosional events most likely relating to the depletion of the sediment supply from the sea bed (which had been uplifted by the 1855 Wairarapa Earthquake) (Olson et al, 2012). This led to the construction of seawalls in Robinson Bay which collapsed between 1934 and 1936 (Matthews, 1980; Gibb, 2005), with retreat rates of up to 0.5 m/yr subsequently occurring between 1941 and 1985 (Olson et al, 2012). The sediment eroded from Robinson Bay appears to have been transport alongshore and deposited in Rona Bay where the shoreline accreted from 1941–1969 (Figure 4-10a). The sediment source (sand from Robinsons Bay) was however soon depleted so by 1969–1985 there was little uniform direction of shoreline movement (Figure 4-10b). After 1985 Robinson Bay was reconnected to the longshore gravel drift system via the gravel pulse arriving from the south and resulted in beach progradation of up to 4 m/yr (Figure 4-10c). This influx of gravel marked the reconnection of Robinsons Bay with the gravel littoral drift system (Olson et al. 2012).

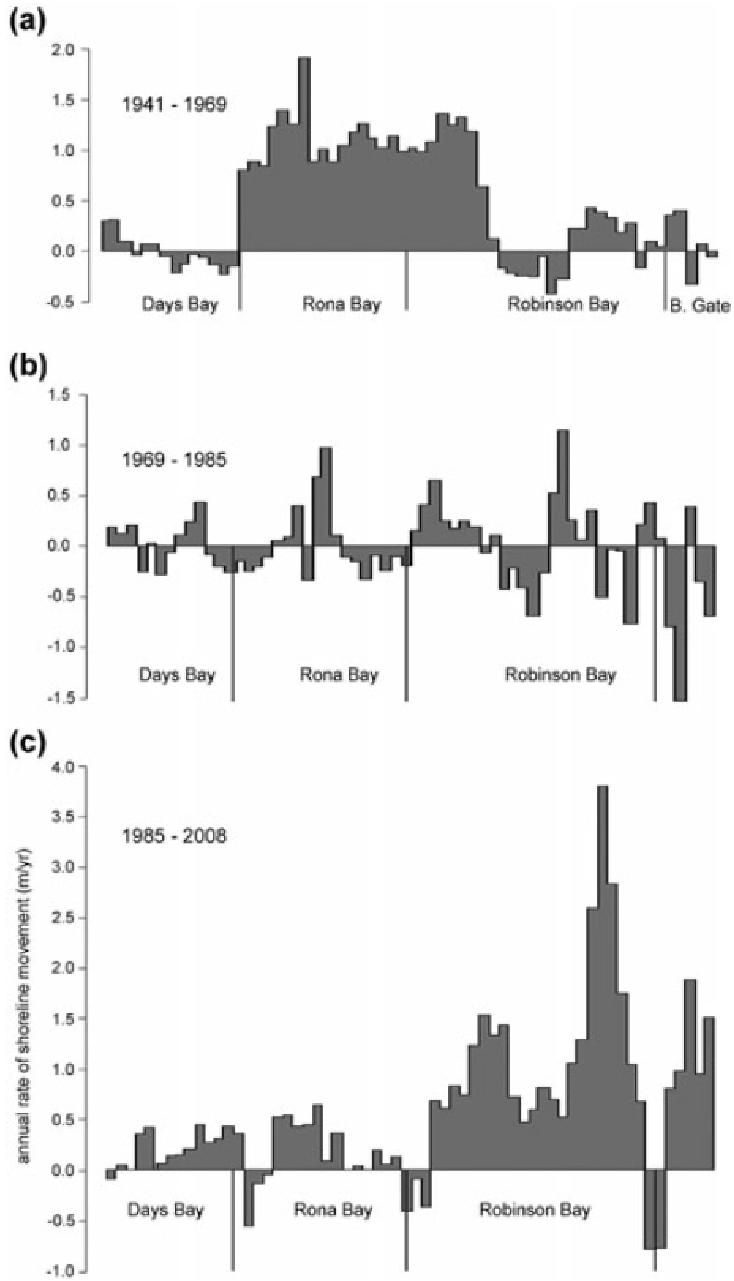


Figure 4-10: Average annual rates of shoreline movement for specific time periods for the shorelines between Days Bay and Burdens Gate. a) 1941–1969 b) 1969–1985 and c) 1985–2008. The x-axis represents individual profiles along the shoreline spaced at similar intervals [*Source*: Olson et al 2012, Figure 12].

In Rona Bay in the early 1900's, the shoreline was on the line of Marine Drive (see Figure 4-11), and in the 1941 shoreline occurs behind the current dune position with accretion of almost 60 m occurring by 2008 (Figure 4-11). The shoreline advance was greatest through 1941–1969 and is characterised by an accumulation of material immediately upstream of a wharf constructed in 1906 (see Figure 4-10, Olson et al. 2012). This indicates that Rona Bay accreted with sand which was eroded from Robinsons Bay. Rona Bay then stabilised as the sand supply was exhausted until gravel arrived more recently, and remains filled with sediment (Figure 4-11). Hence the littoral system and transport of gravel is still establishing at Rona Bay (Gibb, 2005) with some gravel arriving in Days Bay in 2008 (Dahm 2009, Olson et al. 2012). This phased arrival of sand and gravel explains the longer-term presence of sand in Days Bay which can be transported sub-tidally, down to the limit of wave action on the sea bed. The slow arrival of gravel is indicative of the reducing wave energy with distance north from Eastbourne (discussed in Section 4.6.1).

Figure 4-11: Shoreline changes at Rona Bay Top: Rona Bay circa 1930s (see also Robinsons Bay immediately beyond (south) of Eastbourne Township), bottom: Rona Bay in 2008 [*Source*: Top Alexander Turnbull Library Ref. 1/2-048183-G, bottom Dahm (2009)].

Figure 4-12: Photographs of Manaia Bay and Point Brown (between Rona Bay and Days Bay). [*Source*: Left, 1940s (Alexander Turnbull Library); Right 2008 [*Source*: J Butt, local resident].

The sandy beach present at Manaia Bay in the 1940s (Figure 4-12, between Rona Bay and Days Bay) demonstrates the northerly leakage of sands from Rona Bay during the period where sand filled Rona Bay (1941–1969). The sandy beach at Manaia in the 1940s contrasts with the narrow rocky (with some gravel) beach present in 2008 (Figure 4-12), but coincides with the intermediate period between Rona Bay and Days Bay accreting with sand (see Figure 4-10). This suggests the sandy beach of the 1940s resulted from overwhelming supply of sand migrating north from Rona Bay and Robinsons Bay. However, when that excess supply ceased, the beach returned to the present-day rocky environment which represents the natural beach state.

The northerly sediment transport of gravel, driven by waves from the south (wave decay discussed further in section 4.6), decays rapidly with distance north from Eastbourne, reducing the gravel sediment flux into the Eastern Bays embayments. Olson et al (2012) describes a terminus point of longshore transport of gravel in Days Bay, where the gravel pulse arrived in 2008. North of Days Bay, a prominent rocky headland separates Days Bay from Sunshine Bay, which is expected to prevent any further northerly gravel movement between embayments, especially when combined with the effect of reduced wave energy northwards along the Eastern Bays.

Geotechnical measurements from the beach surface (Olson 2009) and subsurface (Stantec 2017b) in each bay between Lowry Bay and Robinson Bay confirm the trend of finer sediments inside the northern embayments and coarser sediments in the southern embayments. i.e., a trend of finer sediment size with distance north from Eastbourne. This illustrates the effect on sediment sizes by the reduced northerly transport of coarse sediments by littoral wave action throughout the Project area. These recent geotechnical records also confirm the gravel terminus concept over the Holocene period, with accumulation of up to 10 m of beach sediment (sands and gravels) which now infills the Eastern Bays and forms the present-day beach.

Future gravel pulse movement

This littoral migration of gravel from Eastbourne northwards is locally thought of as "the saviour" of the Eastern Bays (in response to beach retreat), with some residents expecting the northern embayments to "fill up" with gravel as occurred at Robinsons Bay and Rona Bay, thus eliminating their present beach erosion and wave overtopping problems. This perception may have arisen from Carter and Gibb (1985) who recommend no intervention or management of the Days Bay shoreline as they expected the gravel sediment would eventually fill the embayment. More recent literature indicates this is not expected to occur as the transport of gravels does not continue northwards past Days Bay (Olson et al. 2012, Dahm 2009, and Olson & Kennedy 2009).

In contrast to the cessation of gravel transport, there are a local anecdotal records of gravel tracers (compressed double-baked brick roofing tiles) placed by Matthews (1980) reaching Mahina Bay in small amounts. This indicates that *some* of the Orongorongo River gravel pulse tracked by Matthews (1980) may persist in migrating northwards beyond Days Bay during large southerly storm events. However, the quantity of gravel supplied to the northern bays is expected to be minor, with the sediment transport capacity reducing further north - coupled to the reducing wave energy inside the harbour. In Days Bay and north, any gravels will only form an occasional veneer of gravel (combined with the small locally-derived sediment deposits) following storms, with the gravel redistributed around the bay under persistent northerly conditions. The few chips of roofing tiles may also be locally sourced (i.e., dumping of building waste) or possibly by subtidal transport (as per the sand transport) due to their different composition and density when compared to native greywacke gravels.

4.3.3 Beach sediment properties

The embayed beaches north of Eastbourne are largely mixed sand and gravel beaches. The gravel content of each beach decreases from south to north (coarsest at Eastbourne), while the beach profile grades from very steep to a flatter beach profile with change to finer sediment types. The reducing gravel content reflects the decreasing northerly transport of coarse sediments by littoral wave action. Each of the small gravel beaches or narrow gravel lenses present in all five bays are classified as an endangered and historically uncommon ecosystem (Overmars 2018, Appendix C Avifauna and Vegetation Assessment of this consent application).

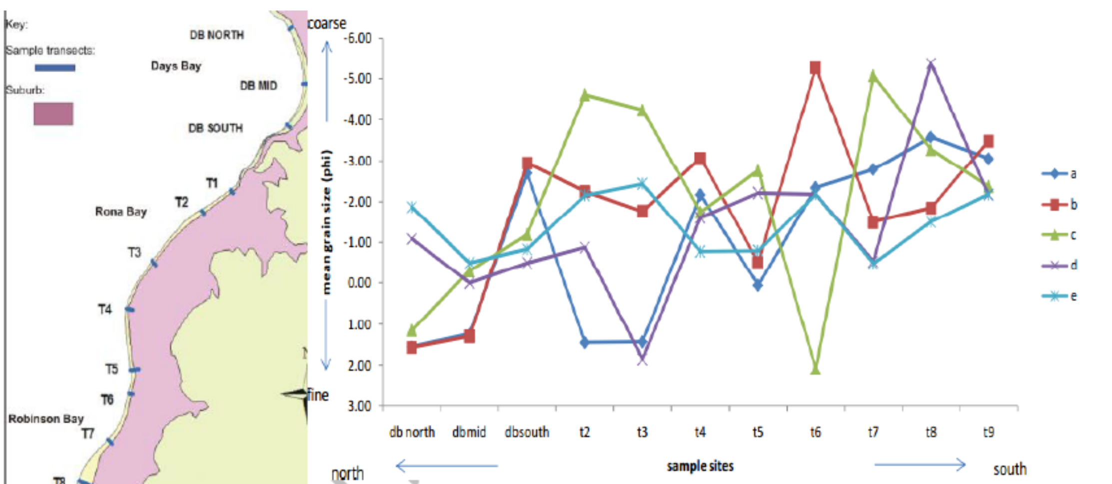
Beaches in this area tend to be reasonably steep, between 6 and 11 degrees which is equivalent to 5(H):1(V) to 10(H):1(V) (Matthews, 1980), however, beach steepness also varies within each embayment in response to wave exposure, sediment size/shape and sorting, nearby rock outcrops and concrete seawalls.

The measured longshore variation in sediment size on the beach surface between Muritai to Days Bay (Figure 2-3, from Olson 2009) shows a general trend of sediment size reduction from southern Robinsons Bay to Days Bay, but there is still a wide range of grain sizes recorded. GHD (2015b) continued the surface sediment samples and describe the general coarsening trend.

Table 4-3: Beach surface sediment sample description. [Summarised from GHD 2015b].

Beach name	Surface sediment description
Lowry Bay	The sediment characteristics at mid-embayment beach and upper beach profile are fine to medium sand with minor sub-rounded gravel content. The upper beach had a 100 mm thick layer of fine to coarse gravel overlaying sand. Minor shell content was also observed.
York Bay	The lower and upper beach are characterised by fine to coarse sub-rounded to rounded gravel with some grey sand content.
Mahina Bay	The beach sediment is grey well-graded sandy gravel and mostly fine to medium gravel at the back of the beach face.
Sunshine Bay	The lower beach is characterised by sub-rounded to rounded gravel with some sand and shell content. The upper beach profile has no sand content, dominated by gravel with trace shell fragments.
Days Bay	The low tide beach is characterised by gravelly dark grey sand, with sub-rounded to rounded gravel and poorly graded sand in the south and dark grey sandy gravel, overlain by 50 mm of sand in the middle of the embayment. The upper beach profile is characterised by sub-rounded to rounded gravel with minor shell fragments in the south and sand with minor shell and gravel in the mid-embayment. The gravel content decreases from south to north, while the beach profile grades from very steep to a flatter beach profile with change in sediment type.

The recent geotechnical CPT records (see Stantec, 2017) within the northern embayment's (Lowry, York, Mahina and Sunshine Bays) confirms the subsurface (up to 10 m below surface) beach sediment size distribution continues to grade from coarse to fine (for between 2 and 5 CPT locations per embayment).



Phi (Φ)	-6	-5	-4	-3	-2	-1	0	1	2
mm	64	32	16	8	4	2	1	0.5	0.25

Figure 4-13: Location of sediment sample transects and results of grading analysis. Sediments measured in phi (Φ), with "a" to "e" representing samples located from the most landward (a) to the most seaward (e) extent of the transect. Note: sediment size is presented in terms of phi (Φ) which is a scale based on the Udden-Wentworth system of universal size grading where size grades are separated by factors of two based on a grain size centre of 1.0 mm (to convert, mm = $2^{-\Phi}$); see table, inset. [Source: Olson (2009)]

Within each embayment wave action acts to preferentially concentrate sandy sediments in central and northern areas with a mixed sand gravel beach tending to characterise the southern end (Olson and Kennedy 2009).

As is typical with mixed sand and gravel beaches, the sand and gravel nature of the beach face can vary significantly over time within the embayments – sometimes gravels being concentrated in one area; other times the beach face being sandier. Storm waves also concentrate gravels and other coarser materials in storm ridges landward of the beach at these sites (which reflects wave-debris swept onto the road where the road has encroached into the backshore).

Note that most of the Eastern Bays shoreline is protected by engineered structures in the form of concrete seawalls and rock revetments (see Section 4.2). These structures have encroached onto the upper beach, and have disrupted the natural sediment transport regime within each bay, particularly during storm events. Over time, the effect of the seawalls (compared to the natural undeveloped state) on beach sediments has been to lower the elevation of the beach (a common beach response with seawall placement), reduce the proportion of fines within the beach material (due to increased wave reflections) and subtly change the overall plan shape of the beach altered hydrodynamics and sediment processes.

4.3.4 Long-term beach shoreline position

In contrast to the long-term shoreline changes around Eastbourne township (e.g., discussed in Section 4.3.2 for Robinsons Bay and Rona Bay), historic aerial photographs suggest there has been little to no long-term shoreline change at the beaches further north over the period since the early 1900s, (Dahm, 2009, GHD 2015a and HCC historic aerials website[12]). Similarly, public consultation interviews with long term residents and visitors to Eastern Bays (more than 30 years, and one for 60 years) noted that “the beaches in the study area north of Days Bay were generally quite stable, although they changed from day-to-day or week-to-week” (Recreation Assessment, Appendix K of Project Consent documents).

The historic aerial photo analysis of the Project area is shown in Figure 4-14 , as sourced from GHD (2015a). The exception to this stable trend is Days Bay, where the shoreline experienced net erosion during the 67-year aerial photographic record with significant periods of erosion 1941–1969, 1969–1988 and 1985–2001, although some accretion occurred between 2001 and 2008 (Olson et al. 2012). However, Days Bay is excluded from the Project area.

This long-term trend indicates that the embayments north of Days Bay have not received significant input of sediment from either alongshore or offshore sources over the last century (i.e., existing rates of net sediment supply appear to be balanced with rates of sediment losses), and suggests that the overall beach volumes (including beach face and sub-tidal areas) remain relatively unchanged in the long term. Hence each embayment is predominantly a “closed” sediment cell, maintaining a nearly stable total volume of sediment over time, with only small volumes of sediment exchanged between bays via headland bypassing during the largest wave/swell events.

[12] HCC historic aerial photograph website: https://maps.huttcity.govt.nz/HistoricAerials/index.html

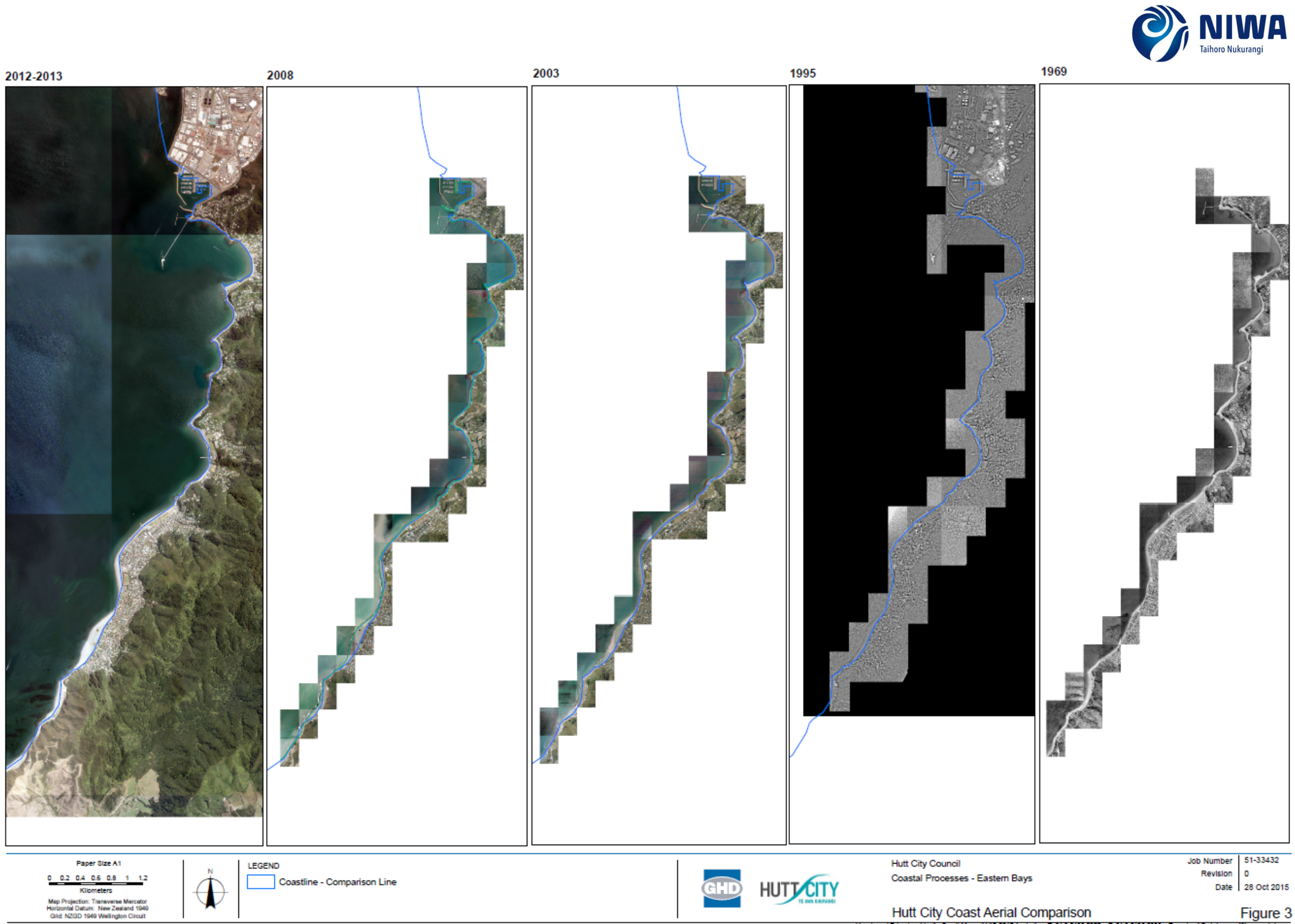


Figure 4-14: Eastern Bays aerial photograph record 1969-2013. [Source: GHD (2015a)].

4.3.5 Short-term beach shoreline position

The Eastern Bays experience short-term cyclical fluctuations in shoreline position (i.e., periods of erosion and accretion) and sediment distribution (i.e., concentration of sediment according to size) on daily, weekly, seasonal, annual and interannual timescales. This behaviour is evidenced in the historic aerial photograph sequence as beach width changes (see GHD 2015, Dahm 2009). These short-term changes reflect the redistribution of sediment within each embayment by wind and wave current action rather than a long-term trend.

In the short term, the beach will respond to the longest and most intense wave events (see Section 4.6), eroding and transporting sand from the beach face, shifting gravels up to cobble size (200 mm), and at times closing Marine Drive with water and debris cast over the road (Carter and Gibb 1985). Debris on the road typically comprises gravel (transported by waves) and sand (transported by wind and waves), however, concrete rubble from damage to poor-conditions seawalls can also be cast onto the road. This shift in sediment distribution on the beach face may be from large individual storms or seasonal changes to storminess.

Beach erosion during southerly storms is expected to be the most significant at the northern ends of the bays due to longer wave fetch (from the south) and exposed headlands, although bay-wide erosion caused by the offshore movement of sand is possible with high-tides combined with strong easterly winds.

Simplified numerical models of the potential erosion of the Eastern Bays beaches during present day 50-year ARI storm events show that overall the beaches will erode 0.2-0.5 m and up to 1 m at Sunshine Bay (GHD 2015b). Erosion will be concentrated at the beach area most exposed to wave attack, i.e., the northern end of each bay. GHD (2015a) also claim their modelling shows that the influence of the increased predicted sea level for 2040 with climate change does not lead to any significant increase in the storm erosion. NB this assessment did not assess sensitivity to wave height or storm surge increases due to climate change, however, the effects of these are anticipated to be secondary to sea-level rise.

During periods between storms, the prevailing northerly wind waves will move most sand size sediments southwards and redistribute sand around each embayment, assisting in retrieval of beach sediment from the lower-intertidal zone and rebuilding beach width between storm events.

Note that most of the Eastern Bays shoreline is protected by engineered structures in the form of concrete seawalls and rock revetments. Many of these structures were originally constructed over 100 years ago to protect Marine Drive and have been replaced and upgraded at various times since. These structures have encroached onto the upper beach, and have permanently influenced the natural sediment transport regime within each bay, particularly during storm events.

4.3.6 Rock platforms

The rock platforms which flank each beach are wave cut platforms comprised of fractured bedrock greywacke, with small to very small pockets of cobbles, gravel, sand, shell and beach detritus filling cracks, fissures and gaps. The wave-cut rocky platforms were uplifted and tilted by 1.2-1.5 m during the 1855 Wairarapa earthquake (McSaveney et al. 2006), with Marine Drive constructed on the exposed surface. Since this time, the gradual widening of the roadway and efforts to maintain access during storms, has encroached onto the platforms, the road now covers the upper 10-15 m of the uplifted platform.

Most of the rock platforms are fully submerged at high tide, except exposed rock pinnacles, with the sea at high tide reaching the Marine Drive seawalls in most areas. The rock platforms extend out from the shoreline, with platforms up to 20 m wide exposed at low tide.

The long-term erosion rate of the rocky outcrops has not been quantified, but is expected to be very slow (geological timescales) and only contributes a small volume of sediment to the adjacent beaches. Sediment delivery is anticipated to be closely linked to seismic events which fracture and release material from the body of rock.

Broad scale assessment of surficial sediment distribution and content for the assessment of ecological habitat is shown in EOS (2018, Appendix 1). These show a variety of classifications on and within the rocky outcrops, including boulder fields (also rock revetment), cobble field, cobble and bedrock, gravel field, along with sandy areas, each with individual intertidal habitat value.

Sediment transport across the rocky platforms is generally in the direction of the prevailing wave currents at high-tide, which spread sediment and detritus material around. Sediment is also transported sub-tidally along the sandy seabed offshore from the rock platforms. Although the overall rate of sediment transport along the Project area is low, the rocky platforms are the primary conduit for littoral sediment transport between bays.

4.4 Tides

4.4.1 Sea levels and tide elevations

Vertical or reduced levels in Wellington are usually defined in terms of Wellington Vertical Datum-1953 (WVD-53), which was established as the then mean sea level (MSL) at Queens Wharf from 14 years of sea level measurements collected between 1909 and 1946 (Hannah and Bell, 2012). The most recent published MSL[13] for Wellington was determined over the period 1998-2016 as 0.205 m WVD-53 (as per Figure 4-15). This is an increase of 0.041 m compared to MSL over the period 1986 to 2005 (0.164 m WVD-53) which is the defining period of IPCC sea-level rise projections (See section 3.2.4).

The tides in Wellington Harbour are semi-diurnal with a relatively small range of 1.25 m during mean spring tides (Figure 4-15) while during neap tides the tidal range is about 0.76 m. The value of MHWS range varies from year to year in a cycle of approximately 19 years in the order of up to 0.1–0.15 metres depending on tidal harmonic constituents[14].

The elevation of MHWS demarcates the CMA boundary separating land from sea. The present MHWS definition published by LINZ for surveyors is 1.77 m Chart Datum, or 0.855 m WVD-53. LINZ defines their MHWS as the average of the high-tide levels of all spring tides predicted to occur under average meteorological conditions during the 19-year period 1 Jan 2000 to 31 Dec 2018 using the designated harmonic constituent set [2 Jan 1996 for Wellington].[15]

[13] MSL is updated annually in the Nautical Almanac by LINZ. Where MSL over 19-year tidal epoch is 1.12 m CD which is 0.205 m WVD-53.
[14] https://www.linz.govt.nz/sea/tides/tide-predictions/standard-port-tidal-levels
[15] http://www.linz.govt.nz/data/geodetic-system/datums-projections-and-heights/vertical-datums/tidal-level-information-for-surveyors

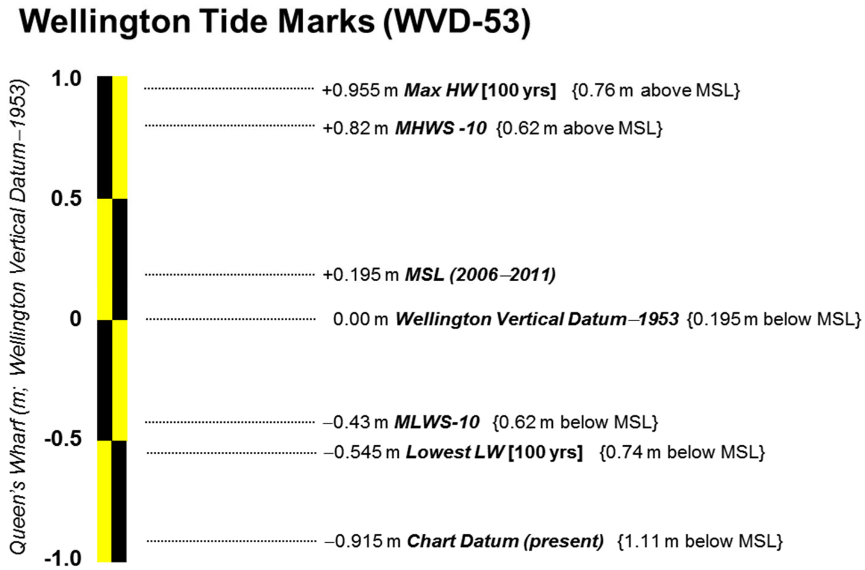


Figure 4-15: Present-day tide marks at Wellington relative to Wellington Vertical Datum-1953 (WVD-53). Note: MHWS-10 is the level exceeded by 10% of all high tides; MLWS-10, low tide mark at which 10% of all low tides descend below, and MSL is the present mean level of the sea for period 2006-2011 [*Source*: Hannah & Bell (2012)].

4.4.2 Tidal currents

Tidal currents are slow in the main part of the harbour, typically less than 0.03 m/s and at sheltered locations such as Lambton Quay they are 0.015 m/s. Currents are much higher through the harbour entrance, reaching a maximum of 0.45 m/s (Heath 1977). Current speeds within the Eastern Bays are expected to reduce even further due to the sheltering effects of rocky headlands and attendant reefs.

In calm weather (no wind or waves) the Harbour tidal currents are too weak to instigate seabed sand transport even within the faster-flowing harbour entrance (Carter 1977). The weak background tidal currents suggest the sedimentation regime within the wider harbour, away from sediment sources such as river/stream mouths or stormwater outlets, is strongly dependant on wind-driven circulation and wave processes.

The direction of current circulation during calm conditions (i.e., without wind) within the harbour is considered to be clockwise around Matiu (Somes) Island on the flood tide and counter-clockwise on the ebb tide (Brodie, 1958). However, this circulation is not always present and the circulation direction has been correlated to meteorological drivers (i.e., wind) and freshwater discharges from the Hutt River (Heath 1977, Allis et al. 2017).

The tidal prism (volume of water exchanged with Cook Strait over a tidal cycle) of Wellington Harbour is about 5% of the volume of the harbour (Heath 1977). This is 82×10^6 m^3 to 88×10^6 m^3 under neap and spring tides, respectively. This relatively small prism, exchanged with Cook Strait each tide, is a consequence of the small tidal range combined with the geologically constrained harbour entrance and contributes to the low tidal currents and high background sedimentation of fine sediment within the wider harbour.

4.4.3 Storm-tides

Storm-tide is the peak sea-level around high tide reached during a storm event, resulting from a combination of mean sea level variability + tide + storm-surge. Storm surges increase as barometric pressure drops combined with onshore winds.

Storm-tides in Wellington Harbour have been assessed by a number of recent investigations. The latest report in 2009 specifically addressing sea levels within the Harbour (as opposed to the open coast) estimates the storm-tide elevation occurrences as shown in Table 4-4 (Stephens et al. 2009). The recurrence interval of the sea levels is calculated from the results of a Generalised Extreme Value model fit to the measured annual maxima sea level from 1975 to 2008 measured at the Queens Wharf tide gauge and also includes a simulated storm-tide of 1.33 m for the Great Cyclone event on 2 February 1936. Extremes are generally described by exceedance events which are events which occur when some variable exceeds a given level or threshold. Two statistics are commonly used, the *average recurrence interval* (ARI) which is the average period between extreme events (that would occur over a very long period with many events), and the *annual exceedance probability* (AEP) which is probability of at least one exceedance event happening each year.

Table 4-4: Extreme storm-tide sea levels for Wellington Harbour. Results are in metres WVD-53, relative to a baseline MSL averaged from 1975-2008. A +0.041 m offset must be added to convert storm-tide elevations into present day (2006-2011) sea level defining period. C.I. =Confidence intervals of model fit.

ARI (years)	2	5	10	20	50	100	200
AEP	0.5	0.2	0.1	0.05	0.02	0.01	0.005
Median	1.13	1.20	1.23	1.26	1.30	1.32	1.33
5% C.I.	1.12	1.18	1.22	1.25	1.28	1.30	1.31
95% C.I.	1.15	1.21	1.25	1.28	1.32	1.35	1.38

Note that the MSL defining period differs between the MSL assessment (Hannah and Bell, 2012) and the storm-tide assessment (Stephens et al. 2009). The 33-year storm-tide MSL period encompasses the 20-year MSL period, and we assume negligible difference caused by the alternative defining periods. However, a +0.041 m offset must be added to the values of Table 4-4 to convert storm-tide elevations into present day sea level defining period (1998–2016).

During an extreme storm-tide event wave processes are expected to further increase the sea level at the shoreline through the process of wave setup. Stephens et al. (2011) and Lane et al. (2012) estimated the joint probability of extreme sea level including wave setup (i.e., storm tide + wave setup) for representative 1% AEP storm events as 1.48 to 1.49 m WVD-53. This suggests the wave setup may contribute an additional + 0.15 m to the total sea level during a 1% AEP joint probability event.

The 21 June 2013 storm event is a recent extreme event in Wellington that caused substantial disruption to coastal roads[16]. Sea levels reached 1.29 m WVD-53 at Queens Wharf corresponding to an ARI of 83 years (or 1.2% AEP)). This is the highest tide level recorded at Wellington since 1944 (PCE, 2015).

[16] https://hwe.niwa.co.nz/event/June_2013_New_Zealand_Storm

4.5 Winds

The long-term wind climate for Wellington Harbour has been derived from observations at Wellington Aero Club, covering a 28-year period from 1 January 1985 to 1 January 2013. The long-term wind rose is shown in Figure 4-16.

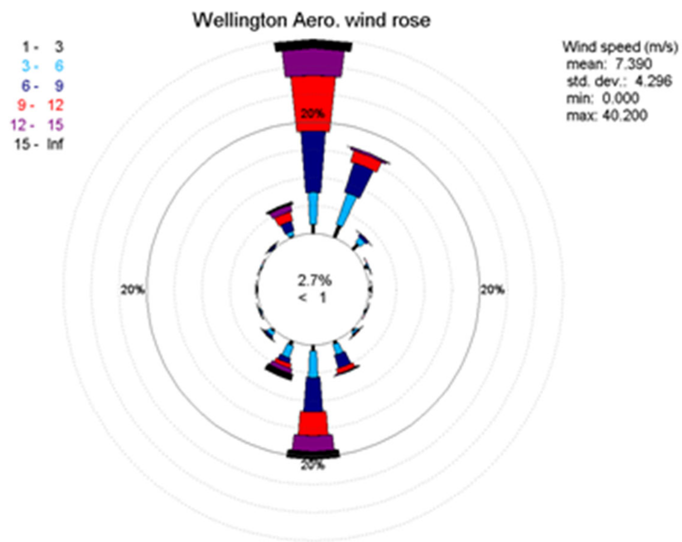


Figure 4-16: Wind rose for winds measured at Wellington Aero over a period of 28 years (January 1985 to December 2012). Meteorological convention is used in expressing the direction that the wind "blows from". [*Source*: Gorman et al. (2006)].

The mean wind speed (for 10-min averages) over the 28-year period to the end of 2012 was 7.2 m/s, with a maximum of nearly 30 m/s (108 km/hr) which was reached on 15 May 1985 when the wind blew from the SSW (220˚). Long-term wind directions exhibit a bi-modal distribution, dominated by winds from a northerly or southerly quadrant as the constrained topography means wind directly from the east and west are rare. Strong winds occur from both directions however the strongest winds tend to arrive from the southerly direction.

These winds are capable of generating substantial waves when blowing over the enclosed harbour waters. If directed onshore, the wind also propels wave splash and debris over coastal defences and onto the land behind, often creating hazards to people and vehicles directly adjacent to the shoreline.

4.6 Waves

Wave conditions within Wellington Harbour are the combination of locally generated wind waves from winds blow over the harbour and swell waves arising from distant storms which propagate through the harbour entrance. When these waves reach the coastline, they break at the beach/structure face causing wave runup and overtopping, often with hazardous consequences along the Eastern Bays foreshore when combined with high-tides and storm-tides (e.g., Figure 4-24).

4.6.1 Swell waves

A narrow window exists where southerly oceanic swell waves approaches directly through Wellington Heads from Cook Strait. Winds and waves from this direction essentially have an unlimited fetch to the south, producing large waves at the harbour entrance (Carter 1977; Pickerill and Mitchell 1979; Matthews 1980). Stephens et al. (2011) modelled the wave heights for a 45-year wave hind cast and calculated the extreme significant wave height for various annual exceedance probabilities for oceanic locations outside Wellington Heads (Table 4-5).

Table 4-5: Extreme significant wave heights at various AEP (0–100% scale) and ARI (years) for oceanic locations outside Wellington Heads. Site 4 corresponds to Baring Head and site 5 to Wellington south coast. [Source: Stephens et al. 2011, Table 2-2].

AEP	ARI	Site 4	Site 5
99%	0.2	4.74	4.78
86%	0.5	5.64	5.74
63%	1	6.26	6.39
39%	2	6.83	7.00
18%	5	7.50	7.72
10%	10	7.97	8.22
5%	20	8.39	8.68
2%	50	8.89	9.23
1%	100	9.24	9.61

These large offshore waves undergo substantial transformation as they enter the harbour, rapidly lose energy and diminish in size as they enter harbour through the narrow entrance (Palmer Head, Pencarrow Head and Point Dorset), shoal over the shallow harbour mouth (maximum 15 m deep), refract around shallow rocky outcrops and islands (Barretts Reef, Makaro/Ward Island, Matui/Somes Island and the Eastbourne Platform) and diffract by spreading into the body of the Harbour. These harbour features acts as a control on the swell propagating into the harbour, severely limiting swell penetration into the harbour.

Figure 4-17 and Figure 4-18 show a region of higher swell wave energy (compared to the rest of the harbour) above the Eastbourne platform, and extending from the harbour entrance up the eastern flank to Robinson Bay. This zone of higher wave energy drives the littoral drift of gravels into the harbour, contributing to the accretion of the Eastbourne foreland (Olson et al. 2012). However, the wave energy decays rapidly with distance further northwards, reducing the sediment transport rate past Eastbourne and into Days Bay and beyond.

This wave energy decay is evident in the modelled swell wave heights obtained by reprocessing the model results of Allis et al. (*in prep.*). The maximum <u>swell</u> height is not expected to exceed 0.5 m along Eastern Bays project area (Table 4-6), with the largest swell waves approximately 40% larger at Days Bay compared to Lowry Bay in all swell conditions.

Swell waves of this magnitude alone have little potential to cause damage to the coastal defences, however when combined with extreme locally generated wind-waves the total wave height is amplified. The effect of swell combined with storm-tides is to increase the potential for damaging waves which are able to reach further inland by the elevated sea levels, however the deeper water does not necessarily permit larger waves as wave heights are principally controlled by distance over which the wind blows.

Table 4-6: Modelled swell wave heights (no local wind waves) in Wellington Harbour for Eastern Bays project. Refer to Figure 4-19 for site locations. [*Source*: reprocessed model results from Allis et al. (*in prep.*)].

Location	Longitude	Latitude	Wave height (m) for Average Return Interval (years)				
			1	10	20	50	100
Lowry Bay	174.90191	-41.258519	0.32	0.32	0.32	0.33	0.34
York Bay	174.90510	-41.264425	0.37	0.37	0.37	0.38	0.39
Mahina Bay	174.90496	-41.269285	0.45	0.45	0.45	0.47	0.48
Sunshine Bay	174.90409	-41.272905	0.43	0.43	0.43	0.45	0.46
Days Bay	174.90256	-41.280072	0.47	0.47	0.48	0.49	0.51
Eastbourne/Makaro Island	174.88119	-41.293807	0.97	0.97	1.00	1.04	1.09

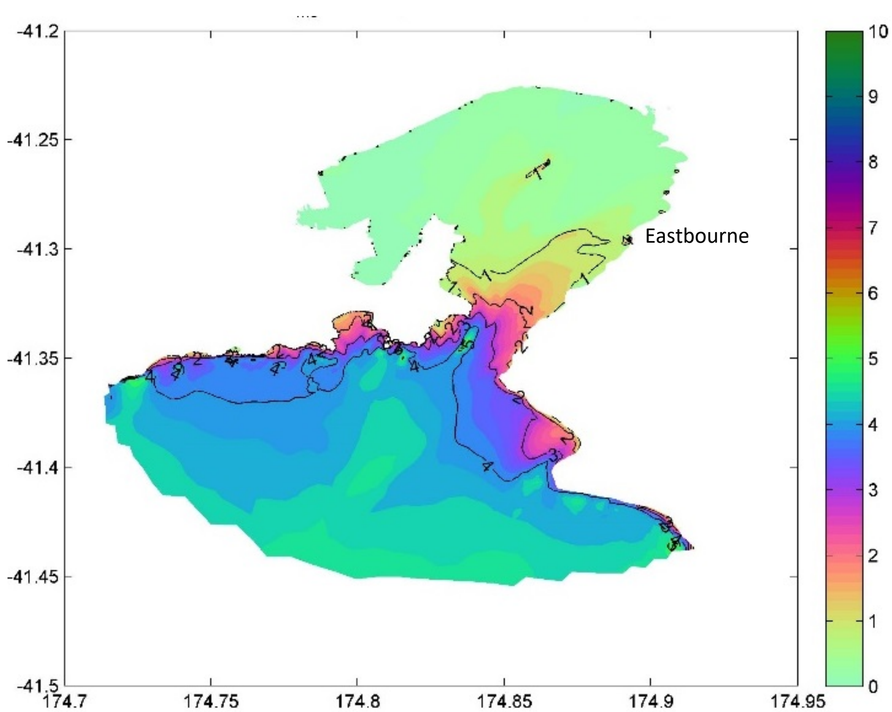


Figure 4-17: Modelled Wellington Harbour wave height for 1-year ARI storm excluding local wind waves (i.e., swell only). Note axis distortion for latitude/longitude is a plotting artefact. Offshore wave heights for this scenario =6.26 m [*Source*: Allis et al. 2017, Appendix D].

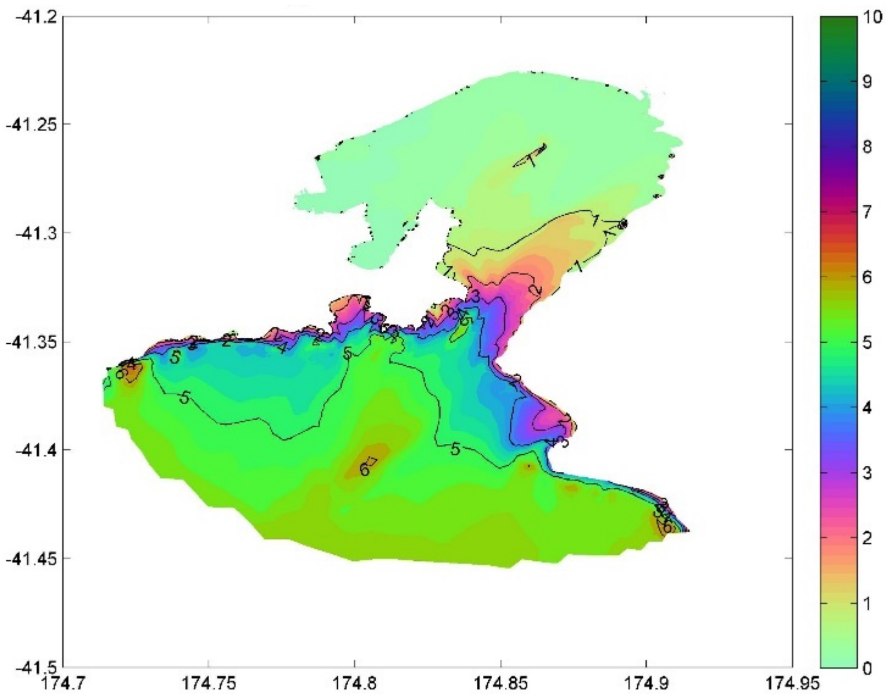


Figure 4-18: Modelled Wellington Harbour wave heights during 100-year ARI storm excluding local wind waves (i.e., swell only). Note axis distortion for latitude/longitude as a plotting artefact. Offshore wave heights for this scenario = 9.24 m [*Source*: Allis et al. 2017, Appendix D].

4.6.2 Wind waves

The largest waves impacting the Eastern Bays foreshore are mainly generated by local winds from within Wellington Harbour.

GHD (2015) estimated the wave heights for the Eastern Bays at five selected locations offshore of from each bay (Lowry, York, Mahina, Sunshine and Days Bay – see Figure 4-19) based on a 10-year wind and wave hindcast of Wellington Harbour. The extreme wave height was then characterised for the 1, 10, 50 and 100-year ARI storms (GHD 2015). These predicted the largest waves would reach 1.16 m (1 year-ARI) and 1.45 m (100 year-ARI) offshore from Lowry Bay during extreme conditions.

Figure 4-19: Location of wave condition outputs from Allis et al. (*in prep.*). Background image dated 11-5-2015. [*Source*: Google Earth].

The recent modelling undertaken by NIWA for the proposed Ngauranga to Petone shared pathway (Allis et al. 2017, Appendix D, reproduced in Figure 4-17 to Figure 4-21) confirm the 100 year ARI wave heights reach 1.50 m offshore from Lowry Bay site and the 1 year ARI wave conditions were consistent at about 1.0 m height (Table 4-7). These results represent the wave heights from combined extreme wind events combined with oceanic storm swells.

Table 4-7: Modelled wave heights (oceanic swell and local wind waves) in Wellington Harbour for Eastern Bays Project. Refer to Figure 4-19 for site locations. [*Source*: reprocessed model results from Allis et al. (2016)].

Location	Longitude	Latitude	Wave height (m) for Average Reccurrence Interval (years)				
			1	10	20	50	100
Lowry Bay	174.90191	-41.258519	0.95	1.18	1.34	1.42	1.50
York Bay	174.90510	-41.264425	0.88	1.09	1.23	1.31	1.37
Mahina Bay	174.90496	-41.269285	0.87	1.09	1.22	1.30	1.36
Sunshine Bay	174.90409	-41.272905	0.86	1.07	1.19	1.27	1.33
Days Bay	174.90256	-41.280072	0.82	1.03	1.17	1.23	1.29
Eastbourne/Makaro Island	174.88119	-41.293807	1.12	1.33	1.43	1.48	1.53

The reason for the slight underestimation of the largest extreme wave conditions by GHD (2015), is likely to be the length of wind record (10 years) used to simulate the wave conditions. 10 years is insufficient to capture the most extreme winds and characterise the extreme wave climate, as it excludes notable events the such as the 15 May 1985 gale which reached sustained (10-minute) speeds of 30 m/s [108 km/hr] from the SSW), or the 9–10 July 1954 southerly gale that gusted to nearly 46 m/s at Rongotai Airport causing waves to break on the Petone Beach and runup onto The Esplanade (Tait et al. 2002). In contrast, the newer NIWA modelling is based on a 45 year hindcast of offshore swell, wind and wave combined with 20 years of wave buoy measurements at Baring Head (Stephens et al. 2011, Allis et al. 2017).

Note that the wave heights of Table 4-7 are larger than the swell-only model results (Table 4-5) indicating the strong influence of local wind-generated waves to the Eastern Bays wave climate. The effect of local winds is more pronounced at the northern sites due to the increasing fetch for winds to blow and generate waves. Overall, within the Eastern Bays project area the wave heights are dominated by local wind speeds, and waves are largest at Lowry Bay and smallest at Days Bay.

These extreme wave heights correlate with wave heights of up to 2.2 m observed in the breaker zone off Petone Beach when allowing for wave shoaling[17] (Opus 2010), and the Seaview Marina was reportedly designed using an estimated significant wave height of 2.0–2.5 m which were expected about once every 20 years under south to south-westerly winds (Opus 2010).

GHD (2015) also mention that the average of their simulated storms occur over a 15 to 20 hours period. This suggests that the most extreme storms (high water + high waves) coincide with 1-2 tidal cycles, although most will only have one critical high tide phase.

[17] Increase in wave height and period caused by the decrease in wave speed as wave motion is obstructed by the seabed in shallow water.

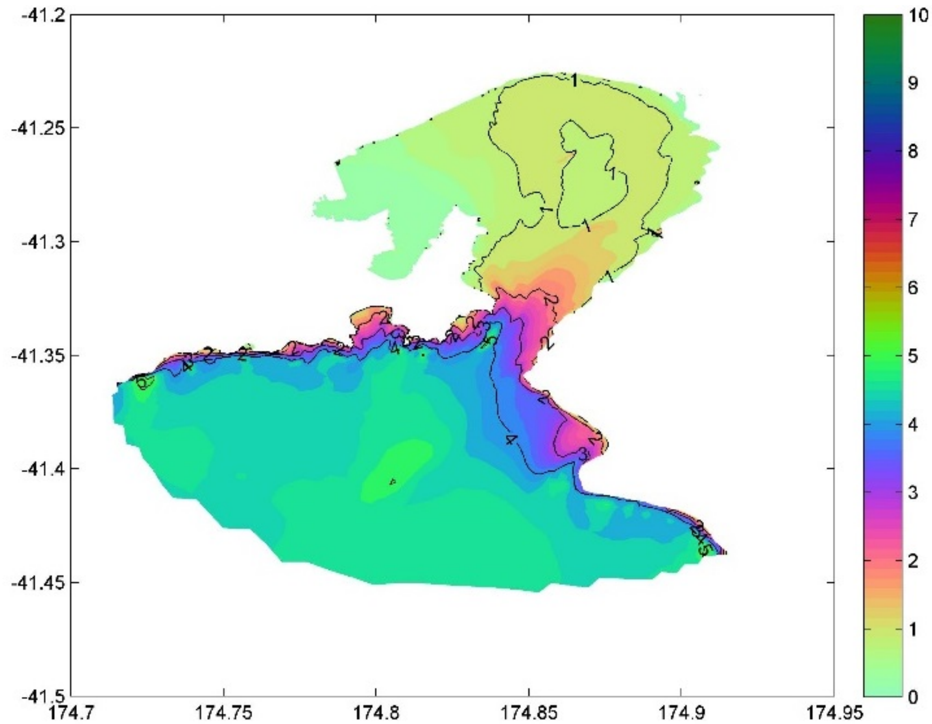


Figure 4-20: Modelled Wellington Harbour wave height for 1-year ARI storm including local wind waves. Note axis distortion for latitude/longitude as a plotting artefact. Offshore wave height scenario =6.26 m (1-year ARI) [*Source*: Allis et al. 2017, Appendix D].

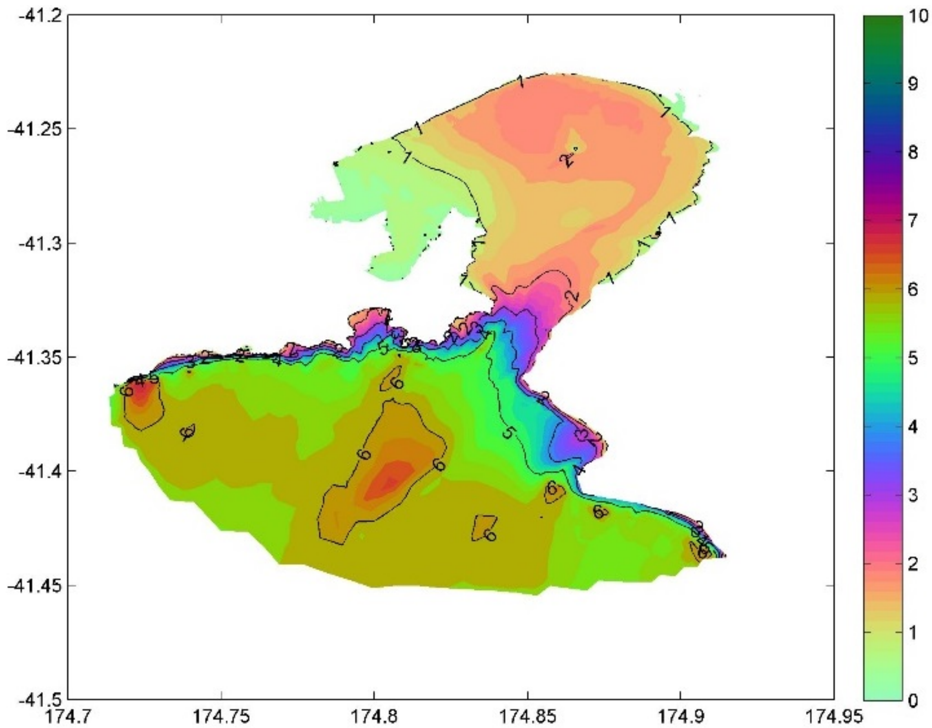


Figure 4-21: Modelled Wellington Harbour wave height for 100-year ARI storm including local wind waves. Note axis distortion for latitude/longitude as a plotting artefact. Offshore wave height scenario =9.24 m (100-year ARI) [*Source*: Allis et al. 2017, Appendix D].

4.6.3 Wave runup and overtopping

Along the Eastern Bays coastline, wave runup and wave overtopping on a beach or coastal structure is primarily dependent on three factors:

1. Crest freeboard (the difference in elevation between crest of beach structure and mean water level). This is the primary contributor to overtopping potential along the low-lying Marine Drive, and will become more critical with sea-level rise.

2. Coastal defence form, including profile (slope), offset from roadway, material composition (rock, concrete, gravels) and material roughness (i.e., rough rocks or smooth concrete), or gaps in the defence for pedestrian access. Seawalls along the Eastern Bays are generally steep, smooth and impermeable whereas seawalls which reduce overtopping are rough, permeable and flat-sloped.

3. Wave climate, as larger waves cause greater overtopping. The wave climate of Wellington Harbour is relatively small, compared to open coast beaches, hence the potential overtopping volumes are smaller, less hazardous and can be easily cleaned up once the storm passes.

The overtopping is usually white-water splash or wind-driven spray but can also be a more hazardous volume of “green” (surging) water during large storm conditions which can overwhelm stormwater drains and contribute to localised inundation, as occurs periodically along this stretch of coastline (e.g., 20-21 June 2013).

Wave overtopping of Marine Drive is well known, with rocks, sand and driftwood debris deposited onto the road and requiring road sweeping at an anecdotal frequency of 5-10 times per year. However, measurements of overtopping volume are limited to anecdotal records or photographs of resulting damage. Some examples include:

- 19–20 June 1947: High tides caused water to seep into basements of many waterfront buildings and waves were breaking over the Hutt to Eastbourne road around high tide (Tait et al. 2002).
- 8th May 1969: Waves come across Eastern Bays Marine Drive at Lowry Bay affecting traffic (see Figure 4-22)[18].
- 20th May 1983: strong winds and waves cause yacht to run aground at Lowry Bay (see Figure 4-23), with wave overtopping and debris spread across the Marine Drive[19].
- 11-13 October 2000: Weather bomb affects New Zealand. Marine Drive closed due to debris and waves over the road[20].
- 21st June 2013: Strong winds and waves caused wave overtopping in Lowry Bay. These waves also closed the Ngauranga to Petone railway for 1 week due to washout/subsidence of the rail foundations[21].
- 18 November 2016: Strong winds and waves caused wave overtopping in Lowry Bay (Figure 4-25). Local residents observed the waves beginning to overtop about 3 hours prior to high tide and at one point, every wave surge spread across the road to reach residents' private boundary walls in the centre of Lowry Bay[22].
- 30 April 2017: Marine Drive closed after water up to 70 cm deep forced several smaller cars to turn around[23].
- 22nd July 2017: Vehicles continue to drive through wave splashing and localised flooding (Figure 4-26) at the northern end of each bay[24].

[18] http://natlib.govt.nz/records/23128408?search%5Bi%5D%5Bsubject%5D=Eastern+Bays+Marine+Drive&search%5Bpath%5D=items
[19] https://natlib.govt.nz/records/23091580?search[path]=items&search[text]=eastbourne+waves
[20] https://hwe.niwa.co.nz/event/October_2000_New_Zealand_Weather_Bomb
[21] *Duff, Michelle; Sam Boyer; Shane Cowlishaw (22 June 2013). "Thousands remain powerless after storm". The Dominion Post. Retrieved 22 June 2013.*
[22] http://www.stuff.co.nz/dominion-post/news/wellington/86598545/drivers-advised-to-avoid-coastal-wellington-road-around-high-tide
[23] http://www.nzherald.co.nz/nz/news/article.cfm?c_id=1&objectid=11847953
[24] https://www.stuff.co.nz/dominion-post/news/wellington/95013382/waves-crash-over-cars-as-storm-hits-lower-north-island

Figure 4-22: Wave overtopping at southern Lowry Bay (8 May 1969). [Source: [18]].

Figure 4-23: Wave spray over yacht aground at Lowry Bay (20 May 1983). [Source:[19]].

Figure 4-24: Wave splash and overtopping at northern Lowry Bay (21 June 2013). [Credit: Sam Gorham].

Figure 4-25: Wave splash and overtopping at northern Lowry Bay (18 November 2016). [Source: [22]].

Figure 4-26: Wave overtopping at central Lowry Bay (22 July 2017). [Source: [24]]

Figure 4-27: Wave overtopping at Mahina Bay (22 June 2013). [Source: [25]].

[25] https://talltalestravelblog.files.wordpress.com/2013/06/photo-22-06-13-16-15-31.jpg

These overtopping events can be expensive to clean up. For example, HCC estimated a cost of $100,000 to remove debris washed up on the Petone foreshore by the 21 June 2013 storm, along with logs and driftwood which lay scattered around Marine Drive to Eastbourne[21]. Figure 4-24 shows white-water overtopping at Lowry Bay during this event. Many vehicles risk driving through these splashing flood waters[22,23,24,26] until the council closes the road.

The overtopping hazard along Marine Drive is strongly conditional on the event water level, the height of the crest of the foreshore profile and the associated crest freeboard, along with the structural form of the seawall. GHD (2016) determined that the predicted overtopping rates are not heavily influenced by the associated wave environment which is limited to 1.75 m during 100-year ARI storm conditions.

The safety of people, vehicles and structures may be assessed for various overtopping discharges, for example guidelines see the Coastal Engineering Manual (USACE 2006) or EurOtop II (2016). Determining these overtopping rates is important to inform design levels, serviceability and operational safety limits, and ascertaining expected cycleway closures and post-event maintenance.

Volumetric estimates of wave overtopping discharge were recently calculated for the present-day shoreline configuration and scenarios including upgrades to the coastal defences and sea-level rise (GHD 2015). Their modelling was stated to be conservative within the limitations of the overtopping calculator[27] (see also Eurotop, 2007) and, for example, simulated a vertical seawall rather than the curved wave-return wall, leading to an over estimation of the overtopping discharge.

GHD (2015) report that overtopping discharge rates could be kept below 2 litres/s perm length of wall (a threshold for safety to pedestrians and low-speed vehicles) with the upgraded seawall during the present-day 50-year ARI storm and SLR (0.2 m in 2040). Although they noted that an additional 0.8 m curved concrete unit was required to keep overtopping within this limit when SLR reached 0.45 m in 2065. In all cases the overtopping discharge was unlikely to cause damage to the structure or asphalt road surface.

In the event that hazardous wave overtopping is considered a critical issue for design of any future structures, GHD (2015) recommended, based on the limitations of previous assessments, that physical modelling and further testing of the coastal-defence profiles are considered in conjunction with the collection of further data to develop more accurate foreshore profiles.

4.7 Seiche waves

Seiches, or long-period standing waves, are known to occur in Wellington harbour with standing modes observed at periods of 159, 26, 17.3, 15.8, 14.7 and 8.3 minutes (Abraham 1997). Tidal records show evidence of these seiche waves achieving amplitudes[28] of 0.05 m to 0.1 m at Lambton Quay (e.g., Figure 4-28 below), and generating currents of 0.03 to 0.3 m/s (Heath 1977, Abraham 1997) with the same scale of activity expected in the Eastern Bays. Tsunami activity has also been observed to enhance seiching with large waves of amplitude of 0.8 m (period 160 minutes) measured following the May 1960 Chilean Tsunami (Heath 1977). While seiche waves do occur, the increase to still water level is generally small (compared to tides) and may be accounted for in a freeboard allowance within extreme water level design conditions.

[26] http://www.stuff.co.nz/dominion-post/news/84027375/Snow-hail-wind-thunder-what-happened-to-spring

[27] http://www.overtopping-manual.com/calculation_tool.html

[28] Amplitude is the half range of the wave height i.e., the height of the wave crest above the still water level at the time

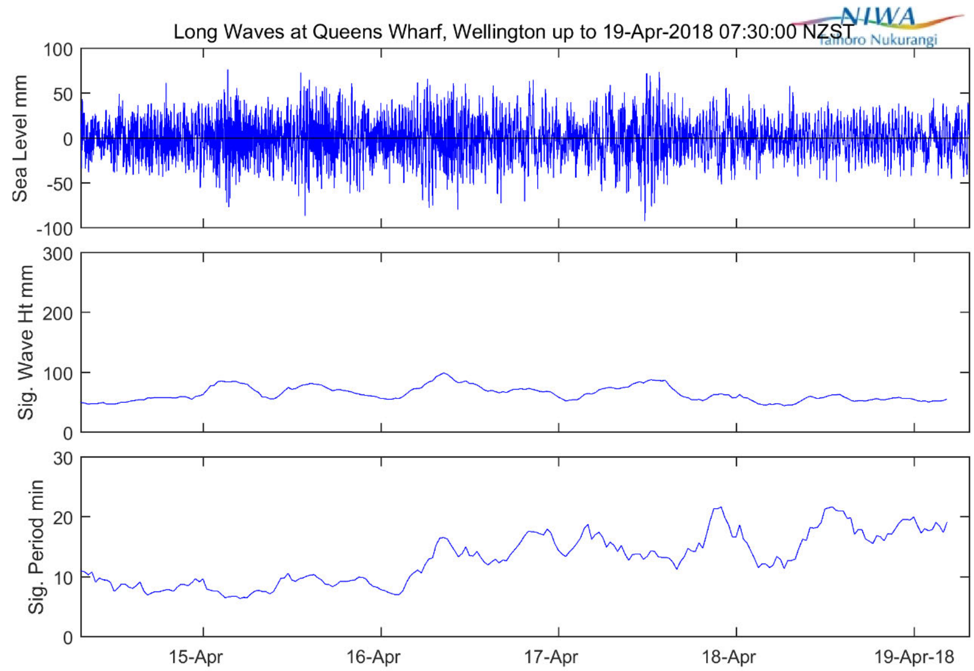


Figure 4-28: Example seiche waves at Queens Wharf, Wellington. Measured sea level elevation (top), seiche wave height (middle) and seiche wave period (bottom). Note seiche periods around 8 minutes on 14-15 April, rising in period to the main modes in range of 15 to17 minutes. Seiche amplitude generally less than 100 mm for the dates shown [Source: NIWA sea levels page[29]].

4.8 Tsunami

Tsunami have affected the Eastern Bays coastal areas in the past. For example, in the 1855 Wairarapa earthquake water levels rose at least 1.2 m above high-water level in the Wellington Harbour. The November 2016 Kaikōura earthquake created tsunami waves (trough to crest height) of 1.6 m near the Eastbourne foreshore[30].

The Wellington Regional Emergency Management Organisation (WREMO[31]) state that an earthquake on the nearby West Wairarapa Fault is the most likely event to cause significant tsunami effects in Lower Hutt City. A submarine rupture on the fault near Cape Turakirae (as occurred in 1855) could cause a 1.5 m rise in water level at Petone foreshore and the Eastbourne shoreline would experience a 1.3 m rise in water level. Seismic activity may also generate Tsunami waves from underwater landslides in Cook Strait (Lane et al. 2016). Within Wellington Harbour, wave activity would start approximately 20 minutes after the earthquake or underwater landslide. It is difficult to predict the frequency and distribution of distant deep or large earthquakes centred offshore, however the Tsunami hazards are documented by WREMO.

As required by NZCPS (Policy 24) it is noted that Marine Drive and many waterfront properties are well within the tsunami hazard zones. However, the extent that the Project could be affected by

[29] https://www.niwa.co.nz/our-science/coasts/tools-and-resources/sea-levels/queens-wharf-wellington
[30] http://www.metocean.co.nz/news/2017/1/8/kaikoura-tsunami-waves-measured-in-wellington-harbour
[31] https://wremo.nz/hazards/tsunami-zones/

tsunami is not addressed in this AEE because damaging tsunami are generally large and rarely economical for an engineering design to accommodate. In the relatively low-lying situation of Marine Drive the changes to the seawalls are minor compared to Tsunami wave size, and will have negligible effect at reducing the hazard from large tsunami waves. Further, the public safety effects are best dealt with through robust emergency-management arrangements including evacuation and link to road closures.

4.9 Climate change

4.9.1 Coastal hazards

The main effect of climate change on the existing coastal environment of the Eastern Bays is the increase the sea level leading to an increase in coastal hazards such as the frequency of wave overtopping and extent of coastal inundation (MfE 2017, Gorman et al. 2006). This is because areas with small tidal range, such as Wellington, are more sensitive to SLR (Stephens 2015), and because Marine Drive is low-lying with edge elevations typically 1.9-2.5 m WVD-53 for most bays and up to 3.5 m WVD in the Windy Point area.

In New Zealand the 1% AEP sea-level elevation is often adopted as a design "extreme sea-level" for coastal-hazard planning, being a high sea level that is exceeded infrequently when high tides and storm surges combine (i.e., a storm tide – see Section 4.4.3). A recent extreme event analogous to this 1% AEP scenario is the 21 June 2013 storm where sea levels reached 1.29 m[32] WVD-53 at Queens Wharf corresponding to an ARI of 83 years (or 1.2% AEP) (PCE 2015). This storm caused disruption to Marine Drive with wave overtopping (see Figure 4-24 and Figure 4-27) requiring multiple road closures and costly clean-ups.

Stephens (2015) shows that with only 16 cm of sea level rise the frequency of the present day 100-year ARI (or 1% AEP) event in Wellington will have increased to *once per year* on average (Stephens 2015, PCE 2015). Following MfE (2017) projections (Figure 3-1), this 16 cm SLR is expected to occur sometime between 2030 and 2040 (depending on global emissions trajectories).

As sea level rises beyond 16 cm within the next few decades the existing Marine Drive coastal route will be subject to more frequent high-water and wave overtopping events like the 21 June 2013 event, leading to more regular road closures and community disruption. For example, SLR of 1 m will create hundreds of occurrences per year of the present-day 1% AEP extreme sea level, with all high tides in Wellington exceeding this level (Stephens 2015).

Figure 4-29 shows an example of the existing seawalls at Point Howard beach along with the proposed designs[33] after periods of sea level rise. Also superimposed is the water level reached on 21 June 2013. The SLR values used for this assessment (Table 3-1) are shown in Figure 4-29.

[32] This is the highest tide level recorded at Wellington since 1944 (PCE, 2015)

[33] A full set of Revision J plans is appended to this report showing each bay with cross sections and MHWS as sea-level rises

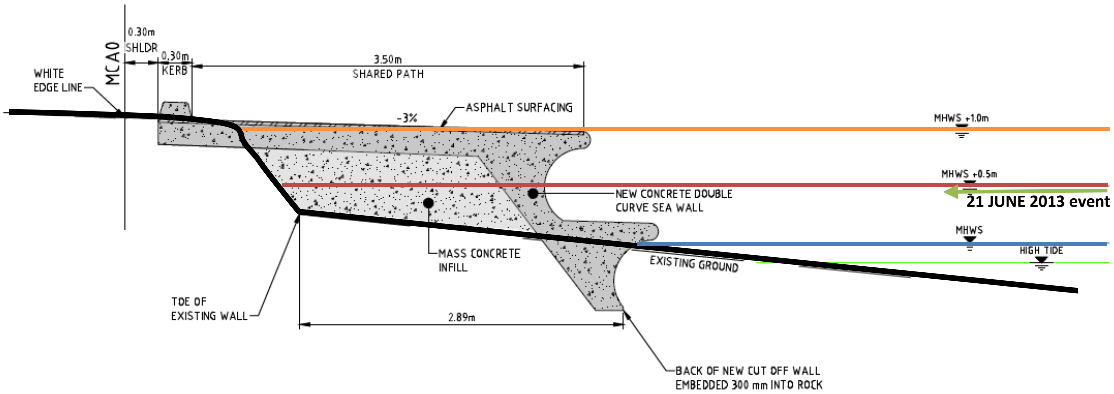


Figure 4-29: Schematic of existing seawall along with proposed seawall at Point Howard Beach showing MHWS elevation after periods of sea-level rise. Note that storm-surge and wave effects will be superimposed above the elevations shown. Cross section at Chainage 1060m.

There are other effects of climate change on coastal hazards which are secondary to ongoing sea-level rise, with the next most important effect being climate change sensitivity to wave heights for the exposed open coast, where wave runup is critical to hazard trigger or adaptation threshold levels for inundation or erosion.

4.9.2 Beach area

A further effect of rising sea levels on the Eastern Bays is the loss of beach area. At present, the beaches of the Eastern Bays are narrow bands between Marine Drive and the sea and bounded by rock outcrops at either end. The total beach area of the Eastern Bays (excluding Days Bay) is approximately 15,000 m^2 at low tide and 4,000 m^2 at high tide (Table 4-8) as exemplified by Figure 4-30 at Point Howard Beach.

Table 4-8: Estimated total Eastern Bays beach area (excluding Days Bay) at present day sea levels and with sea-level rise. Future beach areas assume no additional sediment supply to build beaches higher than present. No co-seismic uplift or subsidence is included in this estimate. Present-day values from Beach Nourishment Assessment (Appendix F of the Project AEE).

Tide stage	Present day tide elevation (m WVD-53)	Beach area at present day (m^2)	Beach area after 0.5 m SLR (m^2)	Beach area after 1.0 m SLR (m^2)
Low	-0.29	15,973	~8,000	~4,000
Mid	0.195	8,647	~4,000	<1000*
High	0.68	4,003	<1000*	<1000*

* assumes a narrow remnant tides beach <0.3 m wide with driftwood and flotsam deposits.

At a fundamental level, SLR will reduce the beach area by inundating a greater area (a linear response assuming a linear beach slope), this is exemplified in Table 4-8 where sea level rise reduces the beach area at each tidal stage. The inland limit of the Eastern Bays beaches are constrained by the road and seawall and hence SLR reduce their width (i.e. “coastal squeeze”) as opposed to a natural beach without seawalls on a sandy coast which would retreat inland. Therefore, after 0.5 m SLR there will be almost no beach at high-tide, and approximately half the current area of beach at mid and low tides. After a further 0.5 m SLR there would only be a small beach at low tide and almost no beach at higher tides (as for Sorrento Bay at present). Secondary to sea level rise are the unknown influences on sediment supply, storm-tides and wave conditions and human influences which will also change how the beaches respond to climate change.

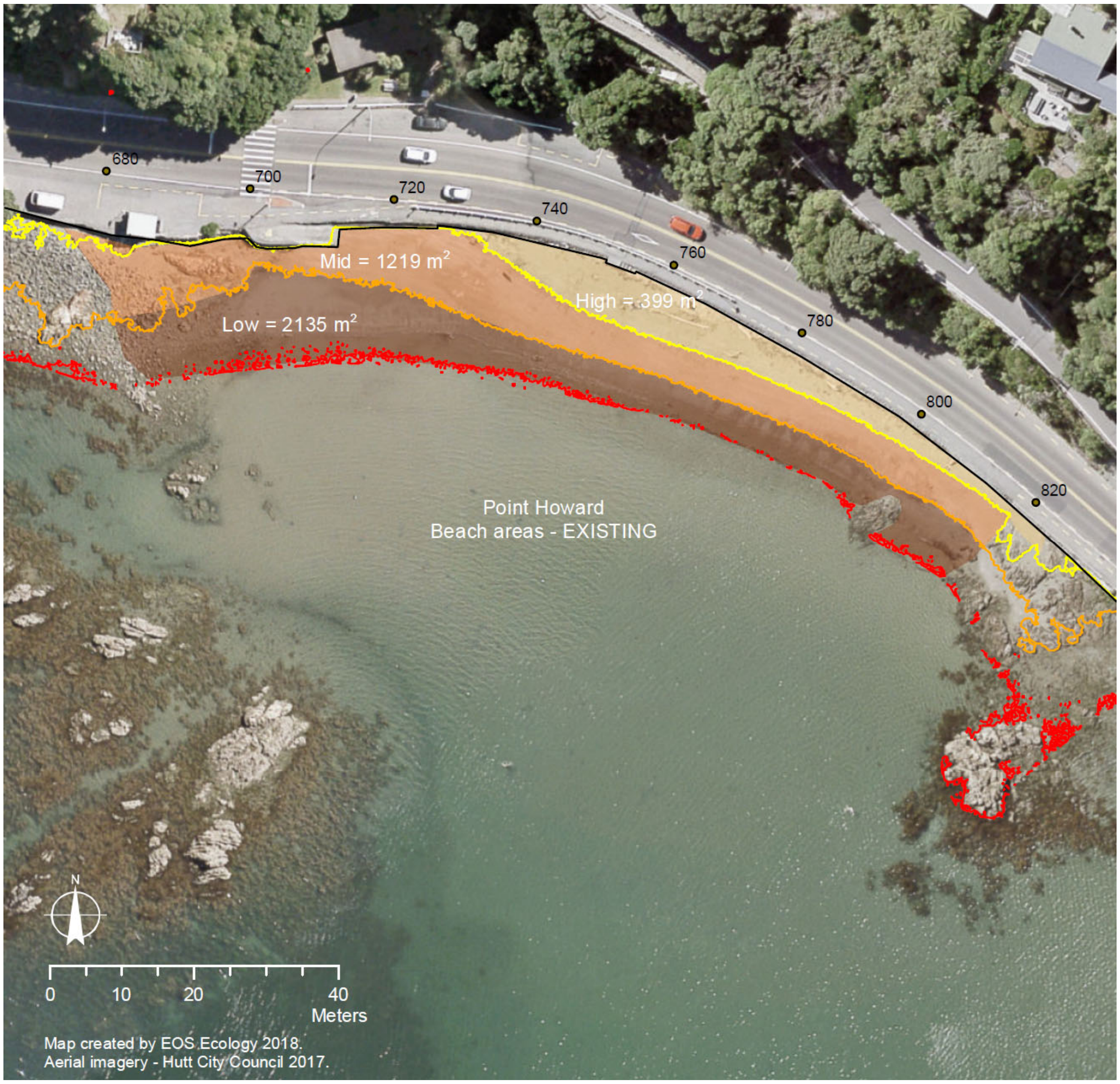


Figure 4-30: Example beach area calculation for Point Howard Beach. A full set of beach area calculations are contained within the Appendix C of this report which were used to inform the Beach Nourishment Report (Appendix F of the Project AEE).

4.10 Summary

The existing environment along the Eastern Bays coastline is characterised as a series of rocky headlands jutting out into Wellington harbour and fringing the sand-and gravel-filled embayments. Marine Drive is constructed on a seismically uplifted wave-cut platform and the former backshore area of each beach. The route has been widened several times through small seaward enlargements, with the coastal fringe supported by engineered concrete and rock defences, several of which are in poor condition or provide inadequate protection from overtopping during large waves.

Sediment on the beaches has arrived from local and distant sources over the Holocene period, but present-day sediment delivery and loss rates are low and not anticipated to increase in the near future. The tidal range and tidal currents are small within the Harbour and most sediment in the coastal zone is transported through wave action. Waves are relatively small (compared to the open ocean) due to the short inner-harbour distance for waves to develop and the shallow harbour entrance dissipating oceanic swell waves.

The low-lying Marine Drive and urban areas within the Eastern Bays currently experience flooding and road closures during high water levels combined with waves and onshore winds. Storms regularly cause localised flooding in roads and property near the coast, with hazardous wave overtopping making Marine Drive unsafe for vehicles and pedestrians in several locations (notably Lowry Bay). The existing seawalls are relatively ineffective at reducing the overtopping waves hazard.

In response to storms conditions, the beaches of the Project area show common morphological responses with short-term fluctuations of beach width and sediment distribution inside each bay (i.e., periods of erosion and accretion) on daily to seasonal timescales. There is no clear long-term trend of erosion or accretion in the embayments of the Project area, demonstrating that the sediment volume within each bay remains nearly stable in the long term and the embayment's are effectively isolated sediment compartments. Some input of gravel and sand is anticipated to the southern-most beach of the Project (Days Bay and south), but the future volumes are not expected to be substantial due to dwindling upstream supply and reduced wave energy within the harbour.

The proximity of the Project to active faults, expanse of soft seabed sediments and geological history of large seismic events suggests that the fill/reclamation structures will require careful design to maintain serviceability access following a significant seismic event. It also suggests that a major down-faulting event may submerge much of the Eastern Bays foreshore into the Harbour waters.

Ongoing climate change will unavoidably affect the existing environment primarily through rising sea levels which will increase the frequency and severity of coastal hazards and road closures along Marine Drive as well as reducing beach areas.

Climate change will have an unavoidable effect on the wider Eastern Bays Region. The principal effect on the Project comes from the rising sea level, which will reduce beach area (as no increase in sediment supply is anticipated) and increase coastal hazard risk. Beach size reduction of 50% less than present day is anticipated once SLR reaches 0.5 m (2050) and possibly sooner if the current secular subsidence of 2–5 mm/year continues. SLR will lead to an increased frequency of wave overtopping and coastal flooding on the low-lying foreshore, and an ongoing reduction in level of service along Marine Drive. Other climate change effects on coastal hazard drivers such as wave heights, wind speed and storm surges, are secondary to the effects of sea level rise (SLR) but are expected to further increase hazards in the Eastern Bays.

5 Effects Assessment: Lifetime operation of the Project

The overall assessment of operational effects over the design life of the works includes both the effects *of the Project* on the environment (such as beach erosion) and effects of the environment *on the Project* (such as extreme waves or climate change).

Assessment is mainly recorded at the scale of the Project area as a whole, supplemented on a bay-by-bay basis as necessary for any locally-specific effects.

The assessment includes the functioning of coastal physical process (from the previous chapter), the effects of the Project and an assessment of the degree of effect (negligible, minor, moderate, major and adverse). Mitigation options (including changes to the original design) are provided where effects would have been greater than moderate, along with the indicative reduction in effects after mitigation has been included.

Potential effects of the Project on coastal physical processes during the operational life cycle are outlined as follows.

- Encroachment into CMA and coastal zone[34].
- Beach nourishment to offset seawall encroachment
- Change to nearshore hydrodynamics and sediment transport
- Interruption to longshore sediment transport
- Edge effects at seawall transitions and tie-ins
- Effects on adjacent seawalls and rock revetments.
- Generation of additional turbidity or transport of fine sediment
- Wave reflections
- Wave overtopping hazard
- Climate change: sea-level rise
- Climate change: coastal hazards
- Climate change: adaptation

The calculations for encroachment of the proposed seawalls was based on the point of greatest encroachment as indicated in the design plans and as such is a slight overestimation of total encroachment. For example, revetment seawalls have the outer encroachment as the outward edge of the toe which following construction may be buried; whilst for curved seawalls the outer encroachment is based on the outward edge of the bottom-most curve which sits above the actual seafloor and overhangs the toe of the wall by approximately 0.2 m (Jamie Povall, Stantec, *pers. comm*.) Calculation of the proportion of length of proposed and existing seawalls was based on the project length as defined in the Design Features Report, and thus does not include the lineal length

[34] "Coastal zone" not a defined planning term. See Section 5.1 for definition used in this assessment.

of foreshore that is not specified within that plan (i.e., the majority of Days Bay, and the promontory north of Days Bay).

5.1 Encroachment into CMA and coastal zone

Seawall encroachment into the CMA is necessary for the project as the existing seawalls do not allow sufficient space between the road traffic lanes to accommodate the shared pathway on the seaward side. The landward boundary of the CMA is the jurisdictional boundary of the coastal marine area and land, with the "coastal environment" encompassing both the CMA and the coastal land margins. Environmental effects for both zones are required to be assessed under the RMA and the associated NZ Coastal Policy Statement 2010 (NZCPS). The CMA boundary is intersection of the land with mean high-water spring (MHWS) elevation, which has been defined for this Project as being 0.855 m WVD-53 as defined by Land Information NZ (see Section 4.4.1) for the present-day MSL. Along the Eastern Bays shoreline, this demarcation lies either on the beach, rocky outcrops or at the existing seawalls or revetments.

Table 5-1: Area measurements for proposed seawalls relative to CMA and coastal zone. Accuracy of area measurement from GIS is +/- 11 m^2 and rounded to 0.01 ha, hence the total areas do not all precisely match [*Source*: Project data, Preliminary Design Plans, Revision J].

Area defined	Measurement	Area (ha)
Total new seawall footprint		0.58
Coastal Marine Area (legal boundary only, below MHWS)	New seawall outside the CMA (above MHWS)	0.26
	New seawall within the CMA (below MHWS)	0.32
Coastal zone (effective coastal boundary including subtidal, intertidal and supratidal)	New seawall behind the existing seawall toe (i.e., gain of coastal zone area)	0.03
	Net loss of Eastern Bays coastal zone	0.55

The total footprint of the Project is 0.58 ha (Preliminary Design Plans, Revision J). This is generally shaped as a thin "sliver" with an average width of 1.3 m along 4.4 km project length between Eastbourne and Point Howard, or 1.8 m on average over the 3.14 km of new seawalls.

The shared pathway will encroach onto the CMA in places, but also onto the upper beach and upper rock platforms which are currently outside the present-day CMA. The total footprint of new works within the CMA is 0.32 ha (Table 5-1) with a further 2.6 ha being outside the CMA. Over the total length of the Project (4.4 km) the average width of CMA encroachment is 0.73 m with maximum width of CMA encroachment 6.6 m in the revetment section at Sunshine Bay where an existing revetment is to be upgraded.

In many locations above rock outcrops, the position of the CMA will change with the shared path advancing into the CMA, and should be updated with LINZ along with the seawall and revetment reclamations around the various pocket beaches.

The total CMA encroachment of 0.32 ha is a very small proportion of 8500 ha Wellington Harbour – albeit in the widely-used coastal fringe, which is where coastal processes are more active and dynamic.

Overall assessment: The loss of CMA area (that is no longer available for coastal physical processes to occur within) is very small relative to the local scale of each embayment and the regional scale of Wellington Harbour. The effect on coastal physical processes from this loss of CMA is likely to be negligible overall but may be no more than minor in localised areas.

However, the demarcation of the CMA does not represent any barrier to coastal physical processes and the effective "coastal zone" encompasses all areas of subtidal (below lowest low tide), intertidal and supratidal (above MHWS but within wave runup, splashing and wind-affected areas). These areas are fully interconnected with the dynamic processes of sediment transport and seawater movement occurring across all these areas (especially during more severe weather events and with rising sea levels). The extent of interconnected sediment transport within coastal zone varies according to the wave height and seabed slope. Along the Eastern Bays, we define the effective "coastal zone" by assuming an average width of 200 m (approximately to the 5 m depth contour at 1:40 bed slope), giving a coastal zone area of about 880,000 m^2 (88 ha).

Hereafter, the assessment of environmental effects of the Project considers the entire coastal zone seaward of the existing road seawall and approximately 200 m offshore.

Table 5-1 shows that not all of the new seawalls of the Project encroach into the Eastern Bays coastal zone. While the total footprint the Project area is 0.58 ha (Preliminary Design Plans, Revision J), approximately 5% (0.03 ha) is located inland from the existing seawall toe and represents a small gain in foreshore area (i.e., de-reclamation). In these areas the new seawall is positioned behind the existing seawall toe and a portion of the existing seawall/road area is being returned to the coastal zone. The de-reclaimed areas are generally shaped as a thin wedge between the existing and new seawall profiles, or positioned on existing rocky outcrops where the path location has been optimised to reduce encroachment (e.g. Figure 5-1).The location of the de-reclaimed areas is spread throughout the Eastern Bays with 101 m^2 in Mahina Bay, 89 m^2 in Sunshine Bay and 70 m^2 in York Bay. Less than 30 m^2 will be de-reclaimed in each of Lowry Bay, Sorrento Bay and Windy Point.

Figure 5-1: Example de-reclaimed area within Mahina Bay. Yellow outline shows area between existing seawall toe and new seawall toe to be returned to the coastal zone. Numbered annotations indicate Project chainage. [*Source:* Eos Ecology 2018 and project data].

The net loss of coastal zone from the new works is an area of 0.55 ha (Preliminary Design Plans, Revision J) as the new seawall extends beyond the existing seawall toe in most places. The overall shape of this encroachment is a thin sliver alongside Marine Drive with 1.23 m average encroachment over the 4.4 km project length. This total encroachment area is small (0.7%) compared to the total Eastern Bays coastal zone area (88 ha).

Overall assessment: The net loss of coastal zone area (the area available for coastal physical processes to occur within) is very small relative to the local scale of each embayment and the Eastern Bays coastal zone. The net effect is negligible, or in some localised areas could be no more than minor. Note that this does not include assessment of loss of the area of beach available as a public amenity which is addressed in a companion technical report (Greenaway 2018, Appendix K of the Project AEE).

Note that beach nourishment is not included into the permanent or temporary encroachment metrics on the basis that the deposition is for the purpose of managing or improving the amenity value of the foreshore in line with PNRP R207(c-e) and matters of control PNRP R207(1-7).

5.2 Beach nourishment

Beach nourishment is proposed to mitigate the loss of beach area caused by encroachment of the Shared Path onto the beach. The Beach Nourishment Report (Appendix F of the Project AEE) details the beach nourishment design and construction methodology.

In summary, the plan for beach nourishment is to nourish with sufficient capital (initial) placement to account for the capital requirement and losses over the consenting period. i.e. to setup the beach for a set time and then defer the long-term management of the beaches to the to-be-developed HCC long-term coastal strategy in accordance with the MfE guidance (MfE 2017). The net effect of beach nourishment will be to compensate for loss of beach area by translating the beach profile seaward but maintaining the present-day coastal processes. No on-going re-nourishment is proposed as part of this project. The nourishment volumes indicated the beach nourishment report (Appendix E of the Project AEE) provide a direct mitigation for the occupation of the shared path structure, but there is no enhancement, or betterment, of the existing beach area and no provision for the ongoing effect of sea level rise. This approach provides a balance with other values and concerns such as the potential risk to sea grass adjacent to the beach at Lowry Bay and the risk of increased stormwater blocking at the various outlets that discharge through the beach that may have potential effects on low flow flooding and migration of native fish species.

The material sourced for the nourishment is to be compatible (colour, size grading, density) to the existing beach material, with the native and imported material to mix by natural wave processes over time, and to slowly disperse around each bay.

The proposed beach nourishment has no negative effect on coastal processes over the lifetime of the project (such as erosion, wave reflections, wave overtopping or longshore drift). The nourishment provides several minor benefits related to increasing the sediment volume, coarseness and longevity of beach sands which will benefit the protection of Marine Drive as sea levels rise (see Section 5.10.2)

Assessment: There is no risk to coastal processes such as erosion, wave reflections, wave overtopping or longshore drift during placement of nourished material as the beach will adjust to the natural profile over a period of weeks to months. Dispersal and aggradation of sediment in deposition areas and long-term spread between bays will be at the rate of natural processes, with negligible change to beach slope (therefore reflection/overtopping) and sand composition.

5.3 Change to nearshore hydrodynamics and sediment movement

The entire length of the proposed 3.13 km seawall construction (total Project area length 4.4 km) is already protected by a seawall of varying type, age and condition (see section 4.2). The Project proposes to encroach typically 1-2 m (and up to 9 m to the seaward toe of rock revetments) from the existing seawall face, with a change to the structural design of the seawall (see section 2.2.1).

At beaches and on rock platforms the resulting effect of seawall replacement on coastal hydrodynamics and sediment movement will be a complex balance between several dynamic processes:

- More efficient and modern seawall designs (stepped curved seawall or rock revetment) have an improved ability to dissipate more wave energy as waves impact on the structures when compared to the existing structures.[35] This will lead to smaller wave reflections than at present (wave reflections will generally be directed back out to the harbour and of no further consequence). This reduction in reflected wave energy will reduce the offshore-directed sediment transport (compared to the existing situation), and hence should promote the net accumulation of sediment in front of the new structures at both beach and rock platform locations (leaving aside local erosion during extreme events).
- The proposed encroachment of protection structures onto the beach areas will cause the beach profile to steepen as the elevation of the beach surface is constrained by tidal elevation. The steeper beach is less efficient at dissipating wave energy and causes greater wave reflections from the beach face. The increased wave reflection promotes the movement of sediment offshore. This steepening effect will not occur at rock platforms, where loose sediments will continue to be swept alongshore and accumulate in small pockets.
- The effect of climate change on the upgraded seawalls and associated sediment supply and nearshore sediment transport is discussed in Section 5.10.1.
- The associated effect of more dissipative seawalls is a reduction to wave overtopping. This is discussed in Section 5.10.3.

If the proposed seawall replacements were along a natural beach, the net effect would be a pronounced loss of beach sediment leading to loss of beach area and exacerbation of wave overtopping (as has occurred at Lowry Bay since 1890 (see historic photograph – Figure 4-5 and description of existing defences in Section 4.2). However, along the Eastern Bays foreshore the consequential loss of beach sediment after historic seawall construction has already occurred (over the previous 100 years) leaving it in its present state. The net effect of this Project on the *existing environment* (baseline situation) will be a minor overall change to the nearshore hydrodynamics and sediment behaviour of the beach and rock platforms. Small accumulations of beach sediment may occur along the new seawalls as less sediment is transported away from the re-configured seawalls by the design features that reduce wave reflections.

The overall assessment is that there will be minor effects on coastal hydrodynamics and sediment movement in the upper coastal zone (i.e., nearest to the shared path in the upper intertidal area and beach face) with negligible effect in the lower coastal zone (i.e., lower intertidal area and subtidal area).

Specific design features were incorporated at the transition between wall types or beach access to ensure local effects are minimised. This is because of the diverging hydrodynamic and sediment processes, which occur at transitions, have the potential for moderate effects on nearby sediment accumulation or scour and wave overtopping. The transition types within the Project are from curved seawall to revetment, or seawall to natural rock outcrops. The Design Features Report specifically address transitions and present the preliminary transition design (see Figure 2-4, p. 25). The transitions include a gradual change in wall type over a 20-30 m length, without abrupt changes to shoreline shape. The precise details will be refined through detailed design by the contractor with

[35] Note: smooth vertical or sloping walls are the least effective to reducing wave runup and splash

coastal engineering input, but is anticipated to be consistent with the preliminary gradual transition plans with a minor potential environment effect.

This assessment of minor effects is caveated on the Project following best practice construction techniques during the detailed design. This is because poor construction techniques, poorly constructed seawalls and inattention to transitions in structures or inclusion of accessways, have the potential for moderate effects on sediment transport in the coastal zone leading to potential erosion or accumulation of sediment.

Overall assessment: The Project will have a minor effect on the changes to coastal zone hydrodynamics (such as wave height, wave driven currents, wave reflections) and sediment processes (sediment transport in the coastal zone leading to erosion or accumulation of sediment). This assessment of no more than minor effects is caveated on the Project following best-practice construction techniques and detailed design for coastal protection structures.

Specific assessment: At some locations, the transition between wall types, transition to natural rock foreshore and proposed beach accessways could have a moderate effect locally on nearshore hydrodynamics leading to local accumulations of sediment or potential for erosion of sediment. However, this effect has been mitigated through including a gradual change in wall type over 20-30 m length and will be refined further during the detailed design phase.

To validate this assessment, we recommend HCC develop a beach management plan with monitoring of the beach shape and volume via beach profiling within each bay before and after construction – discussed in Section 9.

5.4 Interruption to longshore sediment transport

The movement and distribution of beach sediment along the coastline changes according to the antecedent wind and wave conditions. Overall, the small width of encroachment combined with similarity of new and old seawall types and their alignment to the shoreline means the Project is likely to only have a minor effect on longshore sediment transport in the coastal zone.

However, longshore transport may be interrupted by some of the proposed seawalls transitions (e.g., revetment to double curve, see section 5.5) and beach access steps and ramps. Any structural features which extends out from the shoreline as a discontinuity from the smooth planform of the Marine Drive seawall have the potential to interfere with longshore sediment transport and could cause sediment build-up or erosion on either side. In some of these localised situations, there could be a moderate local (5-10 m alongshore) effect on coastal physical processes.

Regarding beach accessways, the position within each bay, orientation and type of beach access location are important to prevent obstruction of longshore sediment movement. With appropriate consideration of coastal processes, the effect has been mitigated to be no more than minor. The Preliminary Design Plans include beach accesses which are narrow and parallel to the shoreline (see Figure 2-6, p.26), and generally positioned at the northern or southern extremities of the beaches (at the change to natural rock outcrops) or integrated with wall type transitions.

Some effects of beach accessways are unavoidable, such as small accumulation of driftwood debris, and a localised reduction in the effectiveness of the coastal protection (due to the discontinuity of the coastal-defence face by the beach access opening). The accumulation of debris is a short-term effect, depending on the wind conditions at the time, and is negligible in relation to coastal physical processes.

Overall assessment: minor effect on overall longshore sediment transport rates.

Specific assessment: Potentially moderate effect on localised within-bay longshore sediment transport, but has been mitigated to a minor effect through careful selection of beach access locations to either end of the beach and careful design of the access configuration (refer to Design Features Report).

To validate this assessment, we recommend HCC develop a beach management plan with monitoring of the beach shape and volume via beach profiling within each bay before and after construction – discussed in Section 9.

5.5 Edge effects at seawall transitions and tie-ins

There are a number of transitions between the different seawall types (new and existing) and to the tie-ins with natural rocky headlands. These transitions have the potential to be problematic regarding coastal processes, particularly if poorly designed with abrupt transitions which cause edge effects (waves wrapping around and focussing waves on nearby structures) with associated changes to sediment transport patterns typically resulting in seabed and beach erosion. Transitions and tie-in also have the potential to be weak point between new defences where overtopping will occur at a higher rate. If poorly designed these transitions could have a moderate environmental effect.

The Preliminary Design Plans have accounted for these potential edge effects at transitions and tie-ins by including a gentle tapering of seawall types across the transitions, with the length of taper dependent on the level of wave exposure. Areas with small wave exposure, such as on the lee-side of rocky headlands, are transitioned over 0 m to 5 m in length. At the most wave exposed locations, where transitioning between rock revetment and double curve walls areas (e.g., northern Lowry Bay), the transitional taper has been lengthened to 20 - 40 m. These design modifications will ensure the potential effect is mitigated to a minor effect.

Detailed design is needed to determine final form, but it is expected to retain the key features to minimise the environment effects on coastal physical processes.

Overall assessment: Edge effects from coastal protection transitions and tie-in sections could potentially cause moderate effects on local sediment transport leading to erosion or accumulation of sediment and exacerbate wave overtopping. This has been mitigated to a minor effect with smooth 'tapering' of transitions within the Preliminary Design Plans and will be further mitigated with site-specific detailed design.

5.6 Effects on adjacent seawalls and rock revetments

The construction of the shared pathway means that at some locations, a new seawall will be adjacent to another older seawall in poorer condition and less efficient at reducing wave runup. There is a risk that the newly replaced seawall, being 1-3 m seaward and of different profile, will deflect waves to an adjacent seawall or another section of seawall within the bay. The change to wave deflection may cause a change to nearshore sediment movement and could lead to scouring/undermining and increased wave action on the older seawalls, hence could cause more rapid deterioration than would have occurred without the new wall. It is also likely that increased overtopping will occur in these locations.

This is an unavoidable effect, but is minor if careful phasing of seawall construction is programmed into the construction schedule. This is to ensure any existing seawalls in poor condition which are adjacent to the new seawalls are not left exposed for long periods. This will reduce the risk of unanticipated seawall failure arising from a change to wave action on the poorest-condition walls.

The indicative construction programme addresses this by recommending that construction is staged on a bay-by-bay basis, with each bay competed in its entirety. Therefore, there will be no new and old sections adjacent to one another exposed for a long duration.

This is also an effect of concern for the construction phasing.

Assessment: Minor effect on structural integrity of adjacent older seawalls if appropriate construction phasing is programmed. Ongoing periodic review of seawall condition should continue.

5.7 Fine sediment generation

There is the potential for higher than existing levels of suspended sediment concentration (SSC) to be generated by reworking of sediments within the coastal zone by the re-adjustment to nearshore hydrodynamics (waves and currents) from the replacement seawalls (i.e., higher winnowing rates by waves and generation of fine sediments from within existing beach deposits). The sediment reworking will primarily occur during combination high tides and wave events, and is part of the temporary adjustment to a new equilibrium beach composition and adjusted beach profile which will occur in the medium term (months to a few years).

Recent hydrodynamic and fine-sediment transport modelling was undertaken for the proposed Ngauranga to Petone shared path (see Allis et al. 2017, Appendix C), this modelling indicated that even when substantially larger (than the Eastern Bays shared path) fill volumes were deposited, the wind and wave climate would rapidly disperse the sediment around the harbour. Seabed sediment accumulation thicknesses were generally <20 microns outside a reasonable mixing zone (150 m) from the source, and maximum SSC remained lower than naturally occurring peak background levels during floods from the Hutt River.

The long-time scale and small volume of sediment redistribution arising from the Eastern Bays Project negates the potential for excessive sedimentation, with long-term generation of SSC expected to remain within natural background turbidity levels which are highest during Hutt River floods and storm wind/wave events. Additionally, suspended sediments arising from the presence of the additional Project works during storms will occur simultaneously with the naturally elevated levels during storm wind/wave events from winnowing of beach sediments along the entire coast. Along the Eastern Bays this is likely to be a negligible effect and is proportional to the expected natural suspended sediment concentrations which evolve over the medium-term timeframe.

Note this assessment does not include the discharge of fine sediments during construction (which is assessed in Section 6).

Assessment: The reworking of fine-grained beach sediments by the presence of the additional coastal protection works through changes to nearshore hydrodynamics is likely to have a negligible effect on nearshore sedimentation rates or suspended sediment concentrations within each bay and the wider Wellington Harbour.

5.8 Wave reflections

There may be some additional wave reflection where the double-curved wall replaces the existing rock embankments. This has the potential to increase wave reflection during high-water storm events. However, with the wave period generally short, wave energy reflection is not anticipated to change substantially. Waves which are reflected will generally be redirected offshore, and inconsequential to the performance of other seawall sections and beaches.

Assessment: The change to wave reflection behaviour, caused by the replacement of existing rock revetments with new double-curved seawalls within the Eastern Bays, will likely have a negligible effect on other seawall sections and beaches.

5.9 Wave overtopping

As discussed in section 4.2, the wave overtopping hazard along Marine Drive is strongly dependent on the crest freeboard (the difference in elevation between crest of beach structure and mean water level), the structural form of the seawall (slope, permeability and roughness), and the wave conditions.

Reducing existing rates of overtopping is therefore manageable through raising crest heights or revising seawall design, but is not reliant on adding more substantial armouring (e.g., rock size) to manage any risk of damage or erosion to surrounding infrastructure.

The existing wave overtopping hazard along Marine Drive is primarily governed by low freeboard. An increase to the crest height of the seawall would be an effective measure to reduce overtopping in the near future. However, the crest elevation of the Project is fixed to that of the existing road, minus some crossfall allowance for stormwater drainage and it is out of Project scope to change the crest elevation of the road carriageway (refer to Alternatives Assessment, Stantec 2018a).

The proposed replacement coastal defences are either a modern double curved concrete wall or rock revetment (see Coastal defence designs, Section 2.2.1). These new designs are better at dissipating and deflecting wave energy when compared to the structures they replace. Consequentially, the construction of the shared path is expected to generally reduce the rate of overtopping and wave splash onto Marine Drive. The additional width of the shared path also acts to reduce the overtopping which reaches the vehicle carriageways. During adverse storm/wave events, there will be reduced accessibility for pedestrians and cyclists on the shared path, which will be located further seaward, but some improvement from the effects of wind-driven spray and debris on vehicle movements and safety on Marine Drive.

The likely effect on the Project from combination high-tide and wave events in the next few decades (until the sea level rises further) is for a reduction to overtopping volumes during minor storm events (with moderate tide level and moderate waves conditions). For example, the approximately 10 overtopping instances at Lowry Bay are anticipated to be reduced to say, 5 per year (with the final prediction to be refined during detailed design) by preventing overtopping in small to moderate storms. However, the principal driver of overtopping along Marine Drive will remain the small crest freeboard, hence any storm-generated waves riding on an elevated storm-tide will cause overtopping irrespective of the seawall or revetment treatment.

A more accurate reduction in wave-overtopping hazard can be determined from detailed design and computational or physical modelling of the seawalls but is not undertaken for this environmental assessment. Detailed design of the seawalls will revise the crest elevation (where possible), coastal defence form, including profile (slope), offset from roadway, material composition (rock, concrete, gravels) and material roughness (i.e., rough rocks or smooth concrete), or gaps in the defence for public access to the beaches. However, the current design is suitable for the scope of the project at this consenting stage and represents a realistic design with future refinements only expected to be minor.

Note that the reduction to the overtopping hazard is only a short-term relief, as the effect of rising sea level will gradually increase again the overtopping frequency (discussed in section 5.10.1). Hence, the proposed works will only 'turn back the clock' for a short period of time (depending on the actual rate of SLR). The short-term benefit is the reason that detailed physical modelling of northern Lowry Bay is not required to refine the design for overtopping mitigation.

Overall assessment: The proposed seawall replacements would reduce the overtopping hazard during minor storm events along all sections of coast. This is through more effective deflection, dissipation and reflection of waves and the wider shared path which will reduce the frequency of impacts on traffic safety on Marine Drive. However, there will be no change to overtopping hazard during large storms combined with high storm-tide water levels as the seawall crest elevation will remain the same. Wind-driven spray and debris/sediment will continue cause some nuisance flooding impacts during storms (but less often for small-to-moderate storms), and temporary closure of the shared path and reduction in speed on Marine Drive may still be required. This is a minor positive effect on the coastal hazard of wave overtopping.

Specific assessment: Several sections of coastline (e.g., the northern 250 m of Lowry Bay) are more susceptible to wave overtopping and road closures. The present design will reduce the overtopping for small to moderate storm events. However, for less-frequent extreme events there is unlikely to be any discernible change to the overtopping hazard as the low crest elevation governs the overtopping discharge. Detailed design at each section will consider design improvements to mitigate overtopping where possible. This is a minor positive effect on the coastal hazard of wave overtopping.

5.10 Climate change

See section 3.2.4, p.36 for discussion on projected sea-level rise for Wellington and guidance for assessment methodology from MfE (2017).

5.10.1 Climate change: sea-level rise

The principal effect of a rising relative sea level (including ongoing subsidence trend) on the low-lying Marine Drive foreshore is an increase in frequency of wave overtopping and coastal inundation (as discussed in Section 4.9). This is exemplified in Figure 5-2 and Figure 5-3 which shows the elevation of MHWS following sea-level rise of 0.5 m and 1.0 m are superimposed onto the existing and proposed profiles.

The effect of storms will be more apparent as with only 30 cm of sea level rise (i.e., within 50 years - Table 3-1) the frequency of the present day "100-year storm" for coastal flooding in Wellington will have increased to once per year on average (NIWA 2015, PCE 2015). Given sections of the road are presently flooded a few times per year, this progression in hazard frequency clearly demonstrates that the present coastal road will have an increasingly marginal level of service into the future. A full set of Revision J plans is appended to this report showing each bay with cross sections and MHWS as sea-level rises.

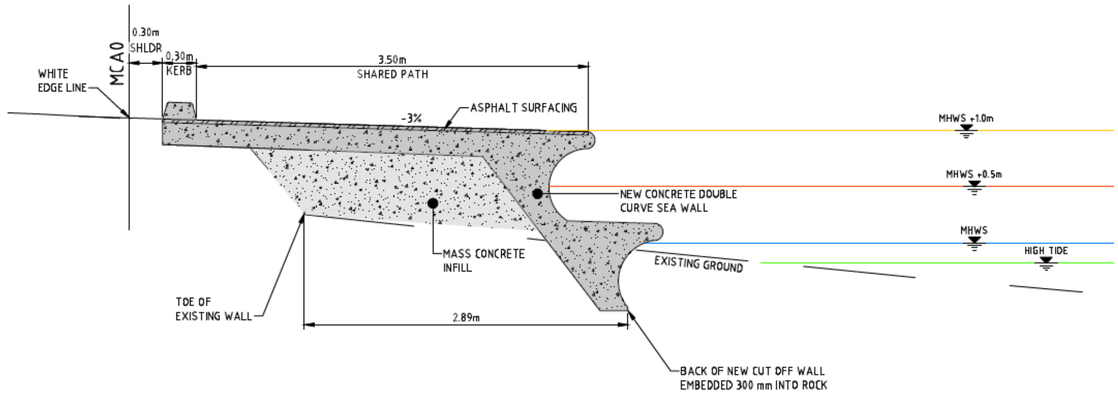


Figure 5-2: Schematic of proposed seawall at Point Howard Beach showing MHWS elevation after periods of sea-level rise. Cross section at Chainage 1060m.

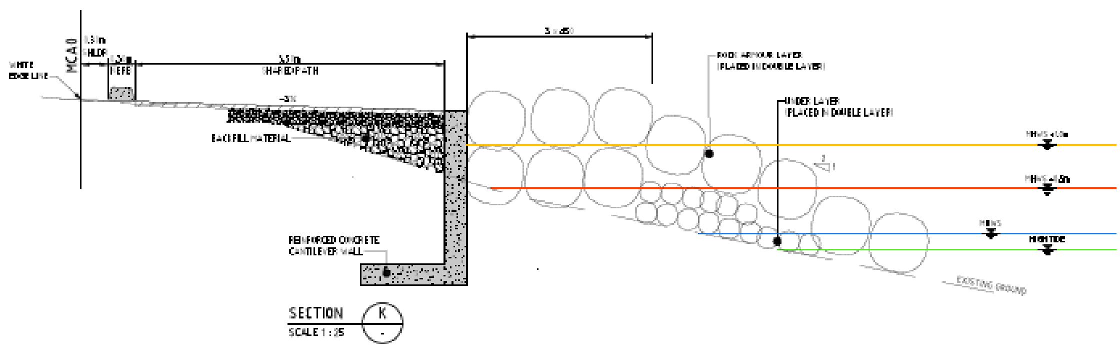


Figure 5-3: Schematic of proposed seawall at Sunshine Bay showing MHWS elevation after periods of sea-level rise. Cross section at Chainage 3960.

The proposed new coastal defence will perform better (i.e. reduced overtopping) under storm conditions than the existing profile (refer to Section 5.9).

However, the effect of ongoing sea-level rise means that any improvement to the level of service (with respect to wave overtopping hazard) along Marine Drive will only be short-term, as the rising sea level (and land subsidence[36]) will continue to reduce the level of protection provided and increase the number of road closures. Essentially, the improvements to the seawall design, the seaward extension for the shared pathway, and beach nourishment only delay the inevitable, or "buy some time" in terms of impacts on Marine Drive. In the time gained, HCC need to consider long-term options for managing the road access to Eastbourne, specifically allowing for adaptation to ongoing SLR, which will continue for several centuries.

Climate change is unlikely to alter beach erosion rates during storms. GHD (2015a) performed simplified numerical modelling of the potential erosion of the Eastern Bays beaches at the present day and with 0.5 m SLR (2040), and no change to wave conditions. The 50-year ARI storm events caused overall erosion between 0.2 to 0.5 m and up to 1 m at Sunshine Bay (GHD 2015a), and their modelling shows that SLR does not lead to much increase in this erosion. Secondly, the shoreline is

[36] Excluding future earthquake rupturing

protected with seawalls so storms under SLR scenarios are unlikely to cause much erosion inland of the seawall unless the seawalls were breached.

Climate change is also anticipated to deliver higher rainfall rates, particularly for short-duration events, but with potentially longer dry spells between rain events[37]. The net effect of climate change on sediment inputs from rivers and streams is uncertain, but sediment delivery to the Eastern Bays coastline is not anticipated to increase appreciably due to the small catchment sizes, improving land-use in the upper catchment and better urban runoff management.

Assessment: Climate change, particularly sea-level rise, will have an unavoidable increasing impact on the wider Eastern Bays Region. The primary effect is the increased frequency of wave overtopping events, and eventually more direct coastal-flooding events, on the back of rising sea level and land subsidence.

5.10.2 Climate change: beach area

A fundamental effect of SLR is to reduce beach area by "squeezing" the area available for a beach to form between the sea and Shared Path/ Marine Drive. Without any nourishment the beach area at each tide stage will reduce by approximately half with each 0.5 m of SLR above present day (refer Section 4.9.2).

The proposed beach nourishment has a minor benefit to delaying the negative effects of SLR on beach areas (i.e. the beach will last slightly longer than without nourishment) principally because the imported nourishment only recreates the present-day beach rather than intentionally increasing it. However, through recreating the beach the volume of each beach increases therefore providing a small increase in material for each beach to respond to rising sea levels, further the imported sediment will be slightly coarser than existing and not lost offshore as quickly so enduring within each bay for longer and providing a larger buffer than the existing beach alone.

Assessment: A minor benefit of delaying the negative effects of sea level rise by increasing the sediment volume within the nourished bays. This furthers "buy some time" for HCC to develop and implement a long-term climate change strategy.

5.10.3 Climate change: coastal hazards

Besides sea-level rise, climate change will affect coastal and estuarine environments by changes in weather related coastal hazard drivers, such as storm surges, waves, winds, and the frequency and intensity of storms. Any changes in impacts from these drivers will have implications for coastal erosion, coastal storm flooding and groundwater and drainage levels.

MfE (2017) states that "the other effects of climate change on coastal hazards will be secondary to ongoing sea-level rise, with the next most important effect being climate change sensitivity to wave heights for the exposed open coast, where wave runup is critical to hazard trigger or adaptation threshold levels for inundation or erosion".

As discussed in section 3.2.4, MfE (2017) provide guidance to undertake sensitivity testing for coastal engineering projects and for defining coastal hazard exposure areas out to 2100, using:

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[37] http://www.mfe.govt.nz/sites/default/files/media/Climate%20Change/climate-projections-snapshot.pdf

1. a range of possible future increases across New Zealand of 0–10 per cent for storm surge out to 2100
2. a range of possible future increases across New Zealand of 0–10 per cent for extreme waves and swell out to 2100
3. changes in 99th percentile wind speeds by 2100 and incorporating these for the relevant RCP scenario from Ministry for the Environment (2016) on climate change projections, to assess waves in limited-fetch situations, such as semi-enclosed harbours, sounds, fjords and estuaries.

Regarding detailed design for the Project, 2) and 3) are not anticipated to be important within the enclosed waters of Wellington Harbour. The storm surges within Wellington Harbour already reach 1.13 m to 1.33 m for 2-year ARI and 100-year ARI events, respectively. The potential increase of up to 13 cm above these existing values will have the effect of further minor increases (beyond that of SLR) to wave overtopping, and further compromise Marine Drive.

Essentially, the improvements to the seawalls only delay the inevitable, or "buy some time". In the time gained, HCC need to consider long-term options for managing the road access to Eastbourne, specifically allowing for adaptation to ongoing SLR.

Assessment: Climate change will have an unavoidable effect of increasing coastal hazards within the wider Eastern Bays Region. Beyond the direct effects of SLR discussed in Section 5.10.1, climate change will alter the coastal hazard drivers of storm surge, with a lesser effect on waves conditions and wind speeds. These changes contribute to the elevated risk for Marine Drive and shared path users into the future.

5.10.4 Climate change: future adaptation options

MfE (2017) supports dynamic adaptive pathways planning (DAPP) approach when considering long-term planning. DAPP supports the incremental upgrade of infrastructure thereby taking an iterative or sequenced risk management approach to dealing with coastal hazards.

The rebuilding (and upgrading) of existing seawalls and the construction of new seawalls for the accommodation of the shared path is a first step in incremental upgrades or alternative adaptation options, with the acknowledgement that it will not be the final solution to addressing the problem of SLR exacerbating coastal hazards along Marine Drive.

The seawalls of this Project, while not a long-term solution, add 2.5 m or 3.5 m of width to the road/path footprint along with strengthening the seawall foundations. The additional width provides a larger and stronger (than present) foundation platform if HCC decides that future structural upgrades of coastal defences are required after a thorough investigation and consultation strategy for the long-term future of the road. The present designs have adequate structural competence to support the additional loads from raising the defences in the future. Hence, the proposed seawalls do not *preclude* future adaptation options through short-sighted design, nor do they lock-in a future pathway beyond that of the present situation and Marine Drive alignment. However, any future intention to relinquish Marine Drive will need to contend with increased community expectation for further upgrades in time to protect access and private property.

Assessment: The project includes design elements which meet the dynamic adaptive planning principles of "buying some time" with this initial adaptation option ("pathway") with the ability for some incremental upgrades, while monitoring SLR and extreme event impacts and their changing frequency. In the future, HCC needs to consider a long-term suite of planning pathways (DAPP) to adapt to ongoing sea-level rise effects of climate change along Marine Drive and adjacent developments.

6 Effects Assessment: Construction phase

The overall assessment of construction-phase effects details the effects on the coastal environment as the Project is implemented. Assessment is recorded at an overall level across Eastern Bays, supplemented on a bay-by-bay basis as necessary where localised effects may occur during construction.

Note that the construction methodology will be finalised on appointment of a contractor. However, the key features are included within indicative construction method in the Design Features Report. The curved concrete walls are anticipated to be follow a similar construction methodology to the curved walls built in York Bay (between 2012 to 2015). A construction methodology for other key elements of the Project is yet to be fully developed, including the rock revetment, access paths, transitions between wall types and tie-ins to natural rocks. It is understood that these will follow the same principles as the Design Features Report and detailed in the CEMP.

The assessment includes description of the physical process, the effects of the project and assessment of degree of effect (negligible, minor, moderate, major and adverse). Mitigation options are provided for potential effects that would be moderate or greater, along with the indicative effect after mitigation options have been included.

The key aspects of Project construction are:

- construction over 6-year period, with 3-6 months of construction per year
- construction to be completed on a bay-by-bay basis, with each section completed before progressing
- temporary occupation of the CMA and coastal zone
- slow construction progress with only a small length of seawall under replacement at any one-time, hence small increments of potential discharges or seabed disturbance
- construction works predominantly in the upper-intertidal and supra-tidal coastal zone, and not under sustained wave attack
- construction works predominantly concrete formwork and pouring, with some rock revetment placement
- all demolition to be contained within silt-fence or behind new seawall, with waste material disposed off-site
- no dewatering/infill works with continual sediment discharges
- small overall volume of new fill, and
- no exposure of non-native backfill material to sea (enclosed by seawall and silt fences).
- Beach nourishment after completion of seawall construction within each bay.

6.1 Temporary occupation of the coastal zone

Seawall encroachment into the CMA and the wider coastal zone (as defined in Section 5.1) is necessary for the construction of the shared path. The temporary encroachment into the coastal zone described here is additional to the 0.58 ha area of permanent occupation of the Project.

The temporary footprint of the Project construction is 1.52 ha (Preliminary Design Plans, Revision J) and occupies areas outside the CMA (i.e., above MHWS) and within the CMA (i.e., below MHWS) as shown in Table 6-1. The temporary footprint area is calculated by factoring an additional width of seaward encroachment into the coastal zone for temporary construction activities to take place within. The additional width beyond the toe of the new seawalls is assumed to be 3 m for concrete walls and 5 m for rock revetments.

Table 6-1: Temporary occupation of the CMA for construction activities. Excludes permanent occupation area [Preliminary Design Plans, Revision J].

Construction footprint component	Area (ha)
Area within intertidal CMA	1.5
Area within sub-tidal CMA (i.e., below low-tide elevation)	0.02
Total construction footprint	1.52

When combined with the area of permanent occupation (0.58 m^2), the total area of temporary and permanent encroachment into the coastal zone is 2.1 ha which is a small percentage (2.4%) of the 88 ha Eastern Bays coastal zone.

However, the 1.52 ha encroachment for construction activities will not be simultaneous, with approximately 20 m of seawall under construction for approximately 2 weeks at any one time, with construction anticipated for 3-6 months per year and spread over 6 years.

This means the true occupation of the coastal zone for construction activities at any one time is limited to an area of 60 m^2 to 100 m^2, depending on the type of seawall under construction. The majority of the new seawalls are curved concrete (60% of the total 4.4 km length), so the temporary occupation locally will typically be nearest to 60 m^2.

The percentage occupied at any one time for construction activities is therefore in the order of 0.1% of the Eastern Bays coastal zone.

On a bay-by-bay basis, the temporary encroachment occupies a larger proportion for the smallest bays. By ensuring only 20 m of seawall is under construction at one time, the proportion of each bay with construction activities is less than 6% of the length of each bay (maximum of 5.25% of 380 m York Bay length – refer Table 2-2 for bay lengths).

Overall assessment: During the construction phase, the temporary occupation of the CMA and coastal zone by formwork/staging (that is no longer available for coastal physical processes to occur within) is very small relative to the local scale of each embayment and the Eastern Bays coastal zone. The direct effect of occupation of these relatively small areas would be mostly negligible – but no more than minor.

Note that this does not include assessment of temporary loss of the area of beach available as a public amenity, which is addressed in a companion technical report (Greenaway 2018, Appendix K of the Project AEE).

6.2 Change to nearshore hydrodynamics and sediment movement

During construction, the staging works will need to enable construction at all tides (i.e., sheet piling around construction sites, wave bunds), but have the potential to interrupt natural hydrodynamic transport and altering the sediment transport within each bay.

The potential effects include scour in front of temporary defence structures, beach lowering adjacent to temporary structures, increased wave overtopping adjacent to these structures. These effects are unavoidable and arise from the need to contain construction within a barrier to minimise the ingress of seawater and prevent accidental discharge of sediments from the construction site.

Overall, with only 20 m under construction at any time for a 2-week period, and with construction for 3–6 months per year, these potential effects will be localised and temporary. On removal of construction staging the beach materials will return to a similar distribution and arrangement as pre-construction, and this recovery should occur over a timespan of days to months depending on climate and wave conditions during and following construction. The effect overall is negligible, with local effects short-lived.

Assessment: Negligible long-term effect on nearshore hydrodynamics and sediment movement by temporary construction staging structures.

To validate this assessment, we recommend a consent condition to monitor the beach shape and volume via beach profiling within each bay before and after construction, with expert assessment to compare the new beach shape and volume to the existing situation. This condition is discussed in Section 9.

6.3 Fine sediment generation

There is the potential for higher than existing levels of suspended sediment concentration (SSC) to be generated by reworking of sediments within the coastal construction area by the temporary alteration of nearshore hydrodynamic processes (waves and currents) during construction of the replacement seawalls. Some examples are: currents and waves around sheet piling; reflections from wave bunds; scour of stockpiled beach material alongside excavated areas. The sediment reworking will primarily occur during combinations of high tides and wave events.

The background sedimentation regime within the wider harbour, away from sediment sources such as river/stream mouths or stormwater outlets, is strongly dependent on wind-driven circulation processes. Consequently, the critical timing for the advection and dispersal of construction discharges is during calm periods when in-situ turbulence from winds and waves is not present to mix and disperse the suspended sediments. If dispersal processes are weak, a more constrained turbid plume may slowly move depending on the circulation pattern at the time, and more localised settling of particulates will be enhanced.

However, for this Project there will only be small lengths (20 m) under construction any one time and the construction works being predominately above high water mean fewer opportunities (e.g., storm events) to generate, collect and discharge substantial volumes of fine sediments. The construction method also anticipates collecting and treating excess water (a mix of seawater and freshwater runoff) from excavations before discharge to the harbour – as detailed in the Construction Features Report (Stantec 2018a). The additional suspended sediments arising from the Project during storms will occur simultaneously with the naturally elevated levels during stormy conditions and are expected to remain, after reasonable mixing, within the range of natural harbour turbidity levels which are highest during Hutt River floods and storm wind/wave events.

Along the Eastern Bays this will be a minor effect and is proportional to the expected natural suspended sediment concentrations which evolve over the medium-term timeframe.

Fine sediments may also be winnowed from surface deposits on revetment rocks. This turbidity is likely to occur only on the first high-tide after rock placement. If reasonably clean-fill is to be used (a consent condition) the discharged volume will be small and will have a negligible temporary effect relative to natural sources.

Assessment: short-term reworking of fine-grained sediments (e.g., silts) from beach sediments by the alteration to nearshore hydrodynamics from construction staging will have a negligible effect on sedimentation rates or suspended sediment concentrations within each bay and the wider Eastern Bays Region.

Regular monitoring of suspended sediment plumes arising from construction works is not recommended, due to the small scale of the works during each construction phase and the risk can be better managed through source control and treatment.

6.4 Beach Nourishment

Assessment of the temporary (construction) effects of beach nourishment are addressed in the Beach Nourishment Report (Appendix F of the Project AEE).

In summary, the release of fines from the imported beach nourishment is a potential risk to the flora and fauna present on the seabed adjacent to the work area and also may be visually unattractive. For this project the potential risk of the generation of suspended sediment clouds that might add to the existing turbidity within the nearshore area is mitigated by:

- Selecting sand/gravel from a marine source that limits the potential release of minerals and fines typical of land based sources,
- Selecting sand/gravel gradings that match or are coarser than the in situ sediment and restrict the proportion of finer material,
- Forming the high tide construction bench with a slightly over-steepened profile so that the existing beach sediment are more exposed to typical wind and wave action,
- Only transferring and shaping the beach profile during lower tide levels.

Assessment: With these proposed actions, the risk of turbidity in excess of the ambient turbidity that can be experienced during wave conditions is considered low. There is no risk to coastal processes such as erosion, wave reflections, wave overtopping or longshore drift during placement of nourished material as the beach will adjust to the natural profile over a period of weeks to months.

6.5 Bulk sediment management

Proactive management of material excavated for seawall foundations is recommended. There are low rates of sediment delivery and loss from the Eastern Bays shoreline and any removal of natural materials offsite (to landfill) may never be recovered.

The net sediment transport capacity (volume of sediment moved by waves *if* sufficient sediment volumes are available for transport) is estimated to be 10,000 m^3/year at Eastbourne, reducing to 7,000 m^3/year at Days Bay and less for the northern embayments (T&T 2016). The cumulative loss of only 1 m^3/m of seawall along the 3.3 km length of construction would lead to a total deficit of 3300 m^3 of sediment arising from the Project. This is the nearly half of the potential annual sediment delivery rate for one year. The loss of this sediment volume from the Eastern Bays would potentially have a minor to moderate effect with the final assessment depending on bay size and volume of sediment removed from the coastal zone at each site.

The simplest mitigation option is for excavated beach material to be stockpiled nearby and replaced on the beach after construction of each section of wall. For rocky material that is cut or excavated from the rock platforms, mitigation by crushing/breaking down to gravel/cobble size and placement on the adjacent rock platforms is recommended. In both situations, waves during the next high tides will redistribute the material naturally along the foreshore.

Note the stockpiling and placement only refers to *natural* beach materials. Discarding demolition waste (concrete, historic non-native bulk-fill, reinforcing) is expected to be to landfill only.

Overall Assessment: The potential effect on bay-wide sediment volumes by the removal of sediment from foundation excavation in the coastal zone is minor to moderate (depending on bay size, excavated volume and construction methodology). Mitigation of this effect to minor is possible through separating native from non-native material, stockpiling native material nearby, and crushing rock removed from reef or headland platforms, redepositing on the beach or adjacent rock platforms after construction of each wall section.

6.6 3 to 6-month construction period

Each section is likely to take about 3-6 months to complete depending on the bay length, complexity, and extent of the construction activities per bay.

6.6.1 Effect on local hydrodynamic and sedimentary processes

The multi-year phased construction has the potential to result a beach backed by a combination of new and old seawalls for a period of 3-6 months. Each wall having different wave reflectivity, crest height and encroachment into the coastal zone.

This potentially could result in temporary abrupt changes to seawall position and shape at the construction site (depending on the abutting seawall types and construction staging activities), altering wave processes and may scour loose sediment away the beach or rock platform, and/or could locally exacerbate wave overtopping. The effect is most likely to occur on sandy beaches and is a potentially moderate temporary effect for small bays with small volumes of beach material (e.g., Point Howard Beach, Sorrento Beach, Mahina Beach), and a potentially minor effect for the larger bays (e.g., Lowry Bay, York Bay). The effect is negligible for rock platforms.

However, the loss of sediment will be localised to within 20 m of the construction site, and only persist while construction is underway – which will be a shorter duration for the smaller bays. If the resulting erosion is unacceptable, it may be possible to intervene and rake/scrape the beach to reprofile if too much scour occurs at junctions between types of walls – this is and adaptive management approach. After construction has completed within a bay, the sediment movement patterns will recover to the new state with the new seawalls (refer to Operational Assessment, Section 5), with no long-term effect from the short-term construction activities

These effects are unavoidable and arise from Project construction and the need to contain construction activities within a barrier to prevent ingress of tidal water and prevent accidental discharge of sediments from the construction site.

Assessment: There will be an unavoidable localised alterations to beach hydrodynamic and sedimentary processes in the immediately vicinity of the construction zone which will persist for the 3–6 month construction period. However, the effect is confined to areas immediately adjacent to the construction zone, and the beach will recover to a new state after construction has ended. The overall effect is minor, but could be more than minor in some locations following storms. If localised erosion is unacceptable, and adaptive management approach to intervene with raking/scraping to fill scoured areas could be undertaken with little long-term risk to the beach.

To validate this assessment, we recommend a consent condition to monitor the beach shape and volume via beach profiling within each bay before and after construction – discussed in Section 9.

6.6.2 Effects on adjacent seawall condition

The phased construction of the shared pathway means that at some point one seawall will be new, and immediately adjacent to another in poorer condition. There is a risk that the newly replaced seawall, being more reflective of wave energy, could deflect waves to another section of seawall within the bay. Such increased wave action on the poorer condition seawall may cause more rapid deterioration than would have occurred without the new wall.

This is a similar effect to the potential for sediment redistribution due to changed wave conditions on the upper beach face through phased seawall replacement.

This is an unavoidable consequence, but only a minor effect if careful phasing of seawall construction is programmed into the construction schedule. This is to ensure any existing seawalls in poor condition which are adjacent to the new seawalls are not left exposed for any period more than 2 months.

Assessment: minor effect on adjacent seawalls if appropriate construction phasing is programmed. Ongoing periodic review of seawall condition should continue.

6.7 Multi-year construction phasing

HCC intends the shared path construction to be spread over 6 years, commencing in 2018.

The intention is to stage the construction of the shared path per bay, and that for each stage that a bay is completed in its entirety. Currently it is proposed to complete Windy Point first (south from Days Bay to Eastbourne, chainage 4990–5500 m), followed by Point Howard/Sorrento (chainage 530–1150 m), and then Lowry Bay (chainage 1150–1960 m), over 3 separate financial years. This will be followed by the other bays.

6.7.1 Overtopping susceptibility due to climate change during construction

The proposed phasing of the Project means that some of the wall replacements necessary to reduce the overtopping hazard may not be undertaken for several years. For example, the northern end of Lowry Bay where overtopping is common and hazardous (see Figure 4-23 to Figure 4-26, page 76).

The effect of the proposed construction phasing is that the overtopping hazard in some areas will not be reduced until the replacements are constructed, which could be several years. Reducing the overtopping hazard and number of road closures are very important to the wider community south of Lowry Bay.

However, while beginning the Project at Lowry Bay is advisable to mitigate the immediate overtopping hazard, the delay of 3 years until the Lowry Bay section is constructed will not see a noticeable increase in overtopping events due to sea level rise due to the small increment of SLR (<15–20 mm) over this time.

We understand the decision to prioritise Windy Point is to link Eastbourne with the existing beach and cycleway facilities at Days Bay, hence providing a continuous shared path south from Days Bay to Eastbourne. The second priority of Point Howard/Sorrento Bay is to continue the gradual infilling of the "gap" between cycleways by beginning at the northern end (Seaview). Further, these two sections are simpler to construct than Lowry Bay, and allow for more detailed design/investigations of Lowry Bay to result in an improved design.

Assessment: the increase to overtopping hazard caused by sea level rise during the multi-year construction programme will be imperceptible from the present-day situation. The wider community and design benefits from delaying construction at Lowry Bay further justify the continued low level of service during the delay.

6.7.2 Seawall condition

There is a risk that maintenance of the existing seawalls will cease for up to 6 years because of the pending upgrades of the seawalls by the Project. Some of the existing seawalls are in poor to very-poor condition (Table 4-2) and there is a risk of failure during this time unless properly monitored. Failure of a seawall section during a storm would cause increased wave overtopping and traffic disruption, additional costs for temporary repairs and interruption to the Project construction (if equipment is moved to repair the damaged section).

This is a minor risk to the environment, and easily mitigated by ensuring continuation of the periodic condition assessments, and permission for emergency maintenance is required. If this monitoring and repair takes place, then there is no need to alter the construction phasing of the 6 years to address seawalls in poor condition.

Assessment: Minor effect on adjacent seawalls if appropriate construction phasing is programmed. Ongoing periodic review of seawall condition and emergency maintenance should continue.

7 Effects Assessment: Cumulative effects

7.1 This project

In the regional-scale context of Wellington Harbour, which covers an approximate surface area of 8500 ha, the total area lost when the entire shared path is complete is about 0.48 ha (4877 m^2). This disturbance is a very small proportion (<0.01%) and will have a negligible effect on the regional coastal zone, tidal prism, tidal flows and tidal range.

At the local scale the cumulative effect of encroaching onto the upper intertidal zone on the beaches and rocky platforms result in the permanent loss of 0.69 ha (6909 m^2) of the 88 ha Eastern Bays coastal zone (assumed to be 200 m wide for the entire project length). This loss is a small (<1%) proportion of the Eastern Bays coastal zone and will have a negligible effect on tidal prism, tidal flows and tidal range.

Assessment: the cumulative effect of the proposed works on the local and regional coastal hydrodynamic processes is negligible.

Based on the necessary construction works using best practice sediment and erosion management, the cumulative volume of sediment discharged to the harbour receiving waters (via fill deposition, dewatering discharges or reworking of existing sediments) is anticipated to be negligible relative to other background sources (such as Hutt River floods or natural wave-reworking of fine-grained seabed sediments during storms). Similarly, the additional contribution to turbidity of coastal waters from this Project is negligible compared to the high background sedimentation rates – given the phased construction sequence.

Assessment: Negligible cumulative effect of all likely fine-sediment discharges throughout construction on the Wellington Harbour sedimentation rates or suspended sediment concentrations.

7.2 External Projects

Several other large projects are currently being considered in the Wellington Region which have potentially overlapping and cumulative effects on local and regional coastal physical processes of Wellington Harbour.

Projects and their stated effect which may overlap with the present Project include:

- Centreport dredging of Wellington Harbour entrance to facilitate larger vessel navigation and entrance. MSL (2016) state that the project will cause negligible change to wave climate along the Eastern Bays, while T&T (2016) assess there will be little change to the existing coastal processes north of Eastbourne, including sediment transport rates. Dredge material disposal will be offshore from Baring Head, with sediments expected to disperse and settle to the seabed within 24 hours of disposal. Turbid plumes are not anticipated to enter Wellington Harbour. Dredging dates are unknown, but estimated to be over a period of 1-2 years.

- Ngauranga to Petone shared path and resilience project. Proposed “sliver” reclamation 4.5 km long and 5-20 m wide between Ngauranga and Petone on the NW shoreline of Wellington Harbour (see project description in Allis et al, 2017). Includes upgrades to coastal defences to better withstand extreme coastal hazards. The earliest construction start date is 2021 and anticipated to be over a period of 5 years (*pers. comm.* Michael Siazon, NZTA project manager*)*.

- Wellington Airport runway extension. No effect in Wellington Harbour as the present proposal is to extend into Lyall Bay.
- Wellington Cross-Harbour Pipeline[38] is a submarine water supply pipeline that will cross Wellington Harbour and carry water from the Waiwhetu Aquifer, in Lower Hutt, to Evans Bay. The pipeline will be a high-density polyethylene pipe nestled into the harbour floor. Investigations are currently underway to determine the best locations for the supporting infrastructure and pipeline itself. The project is currently funded for $116 million in the regional council's 10-year plan but has not yet submitted a resource consent.

While these projects have potentially overlapping construction timeframes and potential effects on coastal physical processes in Wellington Harbour, such as suspended sediment discharges and changes to wave patterns, the slow construction timeframes (decadal) and minor effects of the Eastern Bays shared path are such that the cumulative contribution to regional effects on coastal processes is negligible, as are the potential effects on these other projects. Further, the Shared Path is the most advanced of these projects (in terms of consent submission timeframe) and likely to be the first to begin constructed.

Assessment: the cumulative effect of the other large proposed projects combined with the full extent of the shared pathway on the local and regional coastal physical processes is negligible.

[38] https://www.wellingtonwater.co.nz/work-in-your-area/cross-harbour-pipeline/

8 Proposed mitigation measures

Throughout the pre-consenting phases of this Project, we have been able to modify key design features and the preliminary design plans. Many of these discussions have included important concepts to mitigate potential environment effects on coastal physical processes, with the Preliminary Design Plans and Design Features Report containing the outcome.

From a coastal processes perspective, there are no new specific design or construction mitigations in this report which have not been discussed or included within the preliminary design plans. Design refinements are expected within the detailed design when the contractor is engaged, but these are expected to further reduce any environmental effects from this assessment.

9 Suggested consent conditions

The following coastal monitoring conditions are recommended to either:

- Gather or document physical changes to the existing environment.
- Best-practice construction or post-construction house-keeping.
- Compliance monitoring as part of an adaptive management approach e.g., to ensure sedimentation is within acceptable limits for no adverse effects on benthic habitats.
- Monitoring to ensure the anticipated minor effects on beach properties (slope, width, volume, plan shape) are achieved.

Note that no requirement for compliance monitoring conditions such as field deployment of sensors is recommended for the sequenced construction phase provided that the turbidity and sedimentation effects are mitigated through the CEMP and construction design revisions. However, in accordance with best-practice environmental management, it is recommended that the CEMP include provision for visual observations of turbidity and suspended sediment which trigger an action to review sediment control features and records.

C.1 – HCC shall provide to the Greater Wellington Regional Council, plans and drawings (including dimensioned, cross sections, elevations and site plans) of all areas of proposed structures (including associated permanent and temporary coastal zone occupations, reclamations and de-reclamations), at least 20 working days before the proposed date of commencement of the construction of the Project.

C.2 – HCC shall supply to the Greater Wellington Regional Council and the LINZ Hydrographic Services Office and LINZ Topographic Services Office (Chief Hydrographer, National Topo/Hydro Authority, Land Information New Zealand, Private Box PO Box 5501, Wellington 6145), a set of "as built" plans, final topographic and bathymetric data covering the finished works, and appropriate certification confirming that the new structures and structures have been built in accordance with sound engineering practice, within 60 working days of the completion of the works.

C.3 – HCC shall maintain the site in good order and shall, as far as practicable, remedy all damage and disturbance caused by vehicle traffic, plant and equipment to the foreshore during construction, to the satisfaction of the Greater Wellington Regional Council.

C.4 – Cleanfill**:** All imported fill/rock material to be used in the reclamations, revetments and associated toe aprons and wave/tide bunds shall be in accordance with the Ministry for the Environment "cleanfill" definition, as detailed in Publication ME418 "A Guide to the Management of Cleanfills, 2002" or subsequent updates.

C.5 – HCC shall maintain a log recording the source of fill/rock material imported onto each reclamation or temporary and permanent occupation site. This log shall be made available to the Greater Wellington Regional Council for inspection on request.

C.6 - HCC shall develop a beach management plan which includes monitoring of beach volume via 6 monthly beach profiles (or equivalent elevation surveying techniques) for 5 years in each bay. This is to ensure the actual effect on beach sediment processes are in line with the expectations for generally minor redistribution of beach material and minor changes to beach volume, as well as confirm whether the beach nourishment has been successful in maintaining the same beach area as at present day. The surveying shall commence before construction begins and continue for 5 years after construction ends in each bay. The surveys shall include cross-shore transects from Marine Drive to 3 m below Chart Datum, and at 50 m spacings along each beach. The survey resolution should be of sufficient detail to identify significant changes in grade and the presence of key features such as rocky reefs, stormwater outlets, stairs and accessways, as well as determining a MSL shoreline contour. This survey information shall be interpreted after year 2 and year 5 by an experience coastal scientist to assess the changes to see whether the beaches are approaching a new equilibrium in line with expectations, and make recommendations on the requirement for ongoing monitoring, or if the monitoring could cease. However, in the unlikely event that the 2nd year assessment indicates that unanticipated erosion is occurring (i.e. beach in disequilibrium), the beach nourishment consent will still be active (and other bays may be still under construction) and HCC may be able to easily top-up the beach with more fill to compensate for erosion losses. These assessment reports shall be provided to the Greater Wellington Regional Council within 2 months of each survey.

C.7 – HCC shall develop a beach nourishment plan to be submitted to GWRC for approval that includes: The name and location of the sediment source and evidence of approvals and consents for taking the material, a specification of the borrow material including median grain size, grading envelope and colour; and a construction methodology from the contractor and measures to limit potential adverse effects. For more details refer to the Beach Nourishment Report (Appendix F of the Project AEE).

10 Summary

This report is one of a suite of technical reports that has been prepared as appendices to the Assessment of Environmental Effects Report for the Eastern Bays Shared Path Project.

Overall, the construction and operation of the Shared Pathway project will have no more than a minor effect on coastal physical processes - provided detailed design mitigates the moderate effects outlined above and in the companion technical reports.

It is important to note that the Shared Path should not be considered a long-term solution to the increasing level of coastal hazard exposure due to climate change and particularly ongoing sea-level rise. However, the Project includes design elements that forms an initial adaptation pathway which will “buy some time” for HCC to develop and implement a dynamic adaptive pathway planning approach for the Eastern Bays to adapt to climate change.

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12 Acknowledgements

The author acknowledges the discussions regarding the coastal process of the Eastern Bays with members of the local community John Butt, Judy Lawrence and Roger Lawrence, along with the collaborative discussions with technical experts and design team at project workshops.

13 Glossary of abbreviations and terms

Annual exceedance probability (AEP)	The probability of a given (usually high) sea level being equalled or exceeded in elevation, in any given calendar year. AEP can be specified as a fraction (e.g., 0.01) or a percentage (e.g., 1%).
Average recurrence interval (ARI)	The average time interval (averaged over a long time period and many “events”) that is expected to elapse between recurrences of an infrequent event of a given large magnitude (or larger). A large infrequent event would be expected to be equalled or exceeded in elevation, once, on average, every “ARI” years.
Joint probability	The probability of two separate processes occurring together (e.g., large waves and high storm-tide).
Seawall	Structure positioned on the shoreline designed to defend the land from wave attack. General definition includes all structures achieving this purpose such as rock revetment and a concrete seawall.
Seiches	Long-period standing oscillations of enclosed or semi-enclosed water bodies.
Storm-surge	The rise in sea level due to storm meteorological effects. Low-atmospheric pressure causes the sea-level to rise, and wind stress on the ocean surface pushes water down-wind and to the left up against any adjacent coast.
Storm-tide	Storm-tide is defined as the sea-level peak resulting from a combination of monthly MSL + tide + storm surge. (In New Zealand this generally occurs at high tide).
Wave overtopping	The discharge of water over top of a coastal defence structure. Includes spray driven by wind, and greenwater 'surging' overtopping.
Wave runup	The maximum vertical extent of wave “up-rush” on a beach or structure above the still water level and thus constitutes only a short-term upper-bound fluctuation in water level relative to wave setup.
Wave setup	The increase in mean still-water sea level at the coast, resulting from the release of wave energy landward of the surf zone as waves break.

Appendix A MCDA workshop memo

This appendix contains the memo from NIWA to discuss the assessment of coastal processes for each wall option within the multicriteria decision assessment (MCDA). A range of technical experts contributed to the MCDA process including intertidal ecology avifauna ecology, terrestrial ecology, coastal processes, landscape and visual, engineering design, planning and consenting, and community engagement.

28 June 2017

Jamie Povall

Stantec New Zealand Ltd
Level 11, 155 The Terrace
Wellington 6011

Dear Jamie

This letter addresses the coastal processes discipline for the detailed business case (DBC) phase of the Eastern Bays shared pathway project for Hutt City Council (HCC). Specifically, this letter outlines the assessment and scoring for the first Multi-Criteria Assessment (MCA) workshop, held 22nd June 2017 in Wellington.

The coastal processes topic area includes considerations related to sediment transfer to and from the shoreline, wave action and wave overtopping onto the shared pathway, and climate change considerations (including sea-level rise).

This letter outlines a brief background to coastal processes, baseline conditions and assumptions for the project shoreline, anticipated impacts on the wall types and MCA scoring rationale and values.

1. Background to coastal processes[1] [2]

The shoreline of the Project area is characterised by rocky outcrops/headlands interspersed with multiple small beach embayments between outcrops. Most of the project shoreline has been protected by engineered structures in the form of concrete seawalls and rock revetments.

Coastal processes within the project area are mainly driven by wave patterns with smaller driver of tidal currents. The wave climate is a combination of swell waves (smaller height but longer period) entering the harbour and locally generated wind waves (taller waves but short period). The largest waves expected within the harbour are in the order of 1.4-1.5 m, with smaller waves expected within bays sheltered behind rocky headlands and reefs. Large waves combined with high-tides are known to cause wave overtopping along much of the project foreshore, at a frequency requiring road sweeping of 10 or more times per year (Figure 1),

The beaches along the project area undergo natural periods of accretion and erosion responding to changes in sediment supply, wave conditions and man-made modifications to the shoreline. Longshore drift is the main influence on the sediment supply and general morphology of each bay. The project area is expected to have a small net longshore drift rate to the south driven by the prevailing northerly winds, but with sporadic periods of northerly drift stemming from large swells and winds during southerly storms. Beach erosion during storms is expected to be the most significant at the northern ends of the bays due to longer wave fetch, although bay-wide erosion is possible with high-tides combined with strong winds.

[1] BECA 2015. Review of Design Options to Manage Erosion – Eastern Bays, Marine Drive. Prepared for Hutt City Council, November 2015. 69 pp.

[2] NIWA 2016. Ngauranga to Petone cycleway: Coastal Physical processes. Draft baseline assessment. *NIWA Client Report* HAM2016-023. Prepared for New Zealand Transport Agency. 27 pp.

Figure 1 Lowry Bay during the June 21st 2013 storm that saw many roads flooded around Wellington Harbour. On this day the tide gauge at the port recorded the highest sea level since records began in 1944. [*Credit*: Sam Gorham]

Under Policy 24 of the NZCPS-2010[3], coastal hazards including climate change effects are to be assessed over at least 100 years, which for this Project effectively means out to 2120. Policy 24 also requires assessments to take into account national guidance and the best available information on the likely effects of climate change on the region. The operative coastal guidance provided by the Ministry for the Environment (MfE) is the 2008 edition of *Coastal Hazards and Climate Change – A Guidance Manual for Local Government*.[4] Essentially, a sea-level rise value of 0.8 m by the 2090s from the MfE 2008 guidance has been adopted by councils and previous Transport Agency roading projects (e.g., North-western Motorway, Auckland). Extending the rate of rise to 2115, produces an equivalent to a sea-level rise of around 1 m. However, there is considerable uncertainty in the magnitude and rate of sea-level rise by 2100 and beyond, especially considering the actual carbon emissions trajectory that transpires and the non-linear effect of instabilities occurring in the melting of polar ice sheets.

For this assessment, by 2070 (50 years from 2020), sea-level rise could range from 0.32 to 0.6 m (or higher), while by 2120 (100 years from 2020), rises could range from 0.55 to 1.35 m (or higher), relative to MSL averaged over the period 1986–2005.

This range sea-level rise will increase the frequency of wave overtopping and coastal inundation[5]. With only 30 cm of sea level rise (within 50 years) the frequency of the present day '100 year storm' will have increased to once per year.

Note that the small expected increases to wave height will have a secondary effect to sea-level rise, especially within the enclosed waters of Wellington Harbour.

Sources: Gorman et al. 2006[6], Lane et al. 2012[7].

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[3] http://www.doc.govt.nz/about-us/science-publications/conservation-publications/marine-and-coastal/new-zealand-coastal-policy-statement/new-zealand-coastal-policy-statement-2010/

[4] http://www.mfe.govt.nz/publications/climate-change/coastal-hazards-and-climate-change-guidance-manual-local-government-ne-0

[5] http://www.pce.parliament.nz/media/1390/preparing-nz-for-rising-seas-web-small.pdf

[6] Gorman, R., Mullan, B., Ramsay, D., Reid, S., Stephens, S., Thompson, C., Walsh, J., Walters, K., Wild, M. (2006) Impacts of long term climate change on weather and coastal hazards for Wellington City. NIWA Client Report HAM2006-036.

[7] Lane, E., Gorman, R., Plew, D., Stephens, S.A. (2012) Assessing the storm inundation hazard for coastal margins around the Wellington region. NIWA Client Report. Greater Wellington Regional Council, Kapiti Coast District Council and Wellington City Council.

2. Assessment conditions

The coastal processes assessment is internally relative between wall types, and considers:

- A generic 'beach' and 'non-beach' shoreline configuration with representative characteristics which are typical of the Eastern Bays location within Wellington Harbour,
  - Beach: Sand and gravel beach sediments of shallow depth (assumed to be <4 m) to underlying bedrock (deepest at bay mid-point). Beach composition predominantly medium sand to medium gravels. Beach is concave shape and located at the head of a bay, between rocky headlands/outcrops. The response of the beach assumed to be typical of a fine-sediment beach exposed to a low wave climate and small tidal range, with predominantly longshore sediment movement controlling beach position. Engineered defences form the inland limit of beach area.
  - Non-beach: Rocky outcrops with very small (<5 m wide) pocket-bays between rock pinnacles and protected shorelines. Sediment predominantly fine to coarse gravels, with shells and surficial driftwood detritus washed in by wind/waves on high tides. Slightly higher wave exposure than beach locations due to deeper water, but offset by dissipative effects of rocky outcrops. Sediment transport dominated by wave conditions in higher-energy environment with only large sediment typically remaining as fine sediment is flushed out.
- The existing or receiving environment is the current configuration of the harbour foreshore, coastal protection and transport corridor from Point Howard to Eastbourne. There are existing coastal processes hazards (namely overtopping and inunation) and known sensitivity of the beaches to erosion in the area.
- The provisions of Part 2 of the RMA[8] and Policy 10 of the New Zealand Coastal Policy Statement[9] as this project has some measure of reclamation proposed.

The assessment also assumes

- Assuming operational stormwater treatment is sufficiently addressed by other experts, and the temporary stormwater is addressed by the contractor's erosion management plan and construction method.
- The assessment assumes best practice construction and acceptable mitigation for potential adverse environment effects. i.e. construction discharges into the Harbour are mitigated using best environmental techniques such as material sources, construction practice and silt curtains.
- Assuming no effect on stream outfalls within each bay. While there is a coastal processes element to these outfalls, these are assumed to be dealt with by others, or largely unchanged from present situation.

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[8] http://www.legislation.govt.nz/act/public/1991/0069/latest/whole.html#DLM231904

[9] http://www.doc.govt.nz/documents/conservation/marine-and-coastal/coastal-management/nz-coastal-policy-statement-2010.pdf

3. Coastal processes elements

The individual elements considered within the MCA scores are outlined Table 1 below, with rationale for their effect on the criterion score.

Table 1 Coastal processes attributes and rationale

Element	Rationale

Loss of CMA. i.e. reclamation extent.	• Unavoidable given project objective and site constraints, but NZCPS and RMA emphasis on minimising where possible.
	• Areal extent of CMA loss determined by components of:
	    o Width of pathway -> *equal effect between wall types*, and
	    o Structure slope -> *larger effect for flatter profiles*.

Effect on wider Harbour hydrodynamics (tidal range, tidal prism, tidal currents, wave processes)	• Near-equal and negligible potential effect for all options -> *no relative score difference*
Manages the risks of coastal erosion over time	• Near-equal design life -> *no relative score difference* • Near-equal and adequate performance as coastal defence to maintain shoreline position. -> *no relative score difference* [Long term adaptability to sea-level rise addressed below]

Effect on local shoreline form, wave processes, sediment processes, sediment distribution and beach characteristics.	• Near-equal effect on rocky headlands due to underlying geologic bedrock controls -> *no relative score difference between all non-beach wall options*.
	• New wave reflection patterns may change wave climate, bay-wide currents and sediment distribution. Greater potential for localised erosion within sandy bays than gravel/rock bays. Effect on shoreline determined by existing setting, but is influenced by
	    o Degree of change to wall type (i.e. rock revetment to curved seawall) - > *General replacement assumed to be like with like, therefore assume similar relative effects*.
	    o Degree of change to shoreline protection (i.e. beach to wall) around whole bay -> *Part 2 of workshop, but potential effects reduced as all shorelines are to be protected*.
	    o Structure slope -> *larger reflections and greater potential effects for structure slopes steeper than existing*.

Element	Rationale
Risk posed by overtopping waves	• Existing nuisance issue occurring at least 5 times per year caused by low elevations and high-tides with waves. *-> no increase in pathway elevation to reduce present-day overtopping (all crest heights equal), so near-equal effect between walls.* • Any cycleway width and all walls increase the setback of the roadway from the beach face *-> all wall upgrades should improve the status quo, reducing the present-day overtopping hazard on the road. No relative score difference.* • Wall shape/form increase or decreases potential overtopping risks *-> curved wave returns features and rocky surfaces perform better than vertical or linear faces, or small steps.*
Ability to respond to uncertainty (i.e. sea-level rises) over time and avoids inflexible outcomes	• Option is able to readily respond to uncertain climate outcomes *-> wider options provide greater footprint for future adaptation. Narrower footprint will either require loss of pathway or further reclamation to accommodate future adaptation options (raising the road elevation).* *-> mass-block and mass-fill options provide greater future foundation stability than timber or tied to existing walls.* *-> Same assessment for beach and non-beach locations.* • Wave climate changes not anticipated to be substantial over design lifetime *-> equal and negligible effect for all options* • Sea-level rise can change structure stability through ground-water level pressure, armour stability, and scour at toe. *-> porous structures and mass-fill structures perform slightly better, with little relative score difference between walls.* • End of design life (50 years) options *-> Concrete requires removal and replacement, rock revetments may require top-up but bulk material remains as coastal defence.*

Distilling these coastal processes factors into the key design elements gives the following key attributes:

1. Reclamation extent -> areal extent (structure width and slope)
2. Local shoreline effect - > profile shape (front slope, construction materials, structure width)
3. Overtopping hazard -> profile shape (front slope, front profile, construction materials)
4. Adaptation potential -> areal extent (structure width, structure slope, construction materials, reclamation volume)

These attributes are then scored for each wall type in a beach and non-beach environment.

4. Scoring system

Wall options for both beach and non-beach are assessed against the coastal processes attributes using the scoring system provided (Table 2).

Table 2 MCA scoring system

Score	Description
1	The option presents few difficulties on the basis of the criterion being evaluated and may provide significant benefits in terms of the attribute.
2	The option presents only minor aspects of difficulty on the basis of the criterion being evaluated, and may provide some benefits in terms of the attribute.
3	The option presents some aspects of reasonable difficulty in terms of the criterion being evaluated and problems cannot be completely avoided. There are few or no apparent benefits in terms of the criterion.
4	The option includes clear aspects of difficulty in terms of the criterion being evaluated, and very limited perceived benefits.
5	The option includes significant difficulties or problems in terms of the criterion being evaluated and no apparent benefits.

Comments on scoring table.

- Benefits difficult to establish as it needs to be compared to the existing situation, benefits also somewhat disguised between options,
- Wall success/failure in coastal environments is extremely site sensitive.
- The terminology isn't evenly balanced between positive and negative. Suggest negligible, minor, moderate, reasonable, significant for negative, and reversed for positive.

However, the scoring has been assessed according to the above table, and is internally consistent for the coastal processes MCA topic area.

Table 3 ID and name for wall options

ID	Wall name
A	Placed rock revetment
B	Vertical cantilever wall
C	Concrete Mass block wall
D	Dwarf mass concrete wall
E	Mass concrete wall
F	Double curve wall
G	Single curve wall
H	Boardwalk over wall
I	Boardwalk over revetment

5. Wall Scores

Refer to drawings "80509137-01-001-Wall Details" page SK801 to SK809 for the below wall options. These are summarised into options "A" to "I" in Table 3.

Note that during the workshop Option H and Option I were reviewed for consistency, with Option H now referring to a boardwalk positioned over a concrete seawall and Option I as a boardwalk positioned over a rock revetment. This change is reflected in the post-workshop scores.

Table 4 and Table 5 outline the scores for each attribute within the coastal processes assessment.

Table 4 Wall scores for coastal processes attributes: beach environment

		Wall option (Drawing SK801-809)								
		A	B	C	D	E	F	G	H	I
Reclamation extent		5	3	3	3	3	3	3	2	4
Local shoreline effect		2	3	3	2	2	3	3	3	2
Overtopping hazard		3	2	4	3	4	2	2	2	2
Adaptation potential		1	4	2	4	3	3	3	5	5
Scoring statistics	Sum	11	12	12	12	12	11	11	12	13
	Average	2.8	3.0	3.0	3.0	3.0	2.8	2.8	3.0	3.3
	Median	3	3	3	3	3	3	3	3	3
Selected scoring	Pre-workshop	3	3	3	3	3	3	3	3	2
	Post-workshop	3	3	3	3	3	3	3	3	3

Table 5 Wall scores for coastal processes attributes: non-beach environment

		Wall option (Drawing SK801-809)								
		A	B	C	D	E	F	G	H	I
Reclamation extent		4	3	3	3	3	3	3	2	3
Local shoreline effect		2	2	2	2	2	2	2	2	2
Overtopping hazard		3	2	4	4	4	2	2	3	3
Adaptation potential		1	4	2	4	3	3	3	5	5
Scoring statistics	Sum	10	11	11	13	12	10	10	12	14
	Average	2.5	2.8	2.8	3.3	3.0	2.5	2.5	3.0	3.5
	Median	3	3	3	4	3	3	3	3	4
Selected scoring	Pre-workshop	3	3	3	3	3	3	3	3	3
	Post-workshop	3	3	3	3	3	3	3	3	4

Changes to scoring at workshop

- Higher score (greater disadvantages) for board walk options (Option H and I) for adaptation potential. Reconsidered how difficult the construction and adaptation would be for the complicated structure.
- Higher score (greater disadvantages) to reclamation extent for Option I due to reclamation footing for revetment rock.

6. Summary

The wall options have been scored relative to one another for their general suitability (in regards to coastal processes) in both a beach and non-beach environment in the Eastern Bays project area. The coastal processes considered relate to sediment transfer to and from the shoreline, wave action and wave overtopping onto the shared pathway, and climate change considerations (including sea-level rise).

Note that the success or failure of a wall option (in regards to the coastal processes criteria) is strongly linked to the specific characteristics of each site. The key coastal process vary along the coast and sites less than 10 m apart may require different wall treatment. It is understood that all designs are to be refined with specific placement on the shoreline. The transition between wall types is a key design feature which will influence the overall performance of the coastal defence and shared pathway.

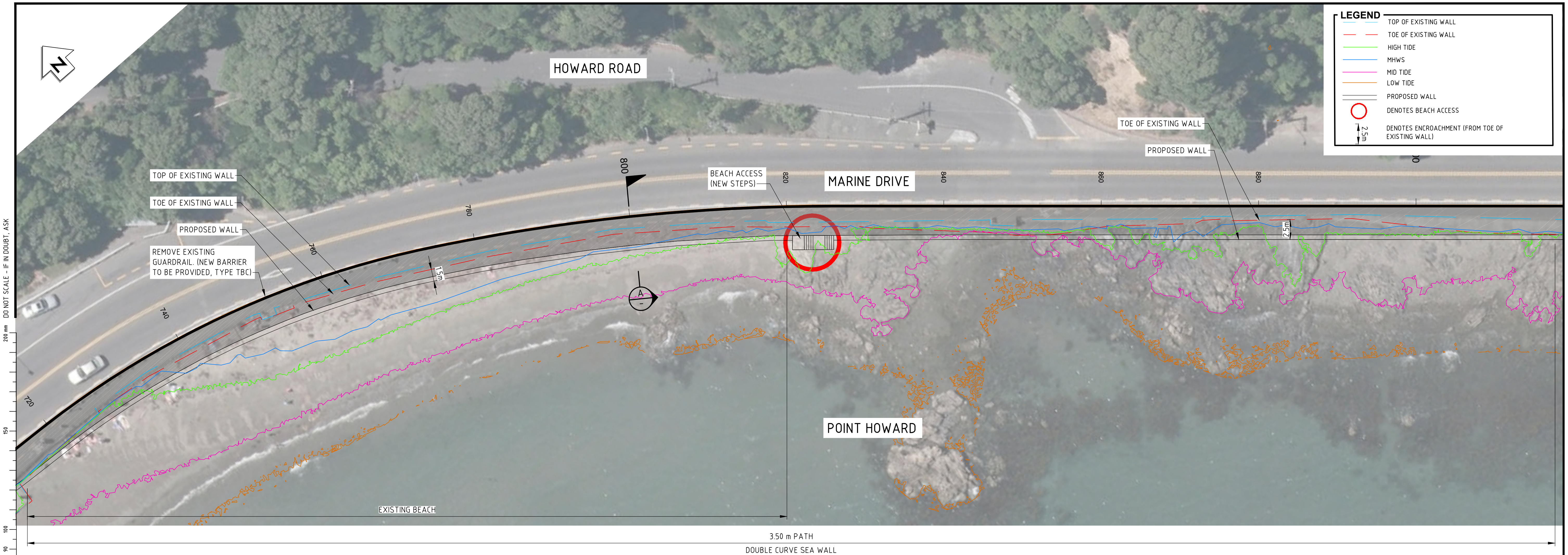
Please let me know if you have any queries on the above,

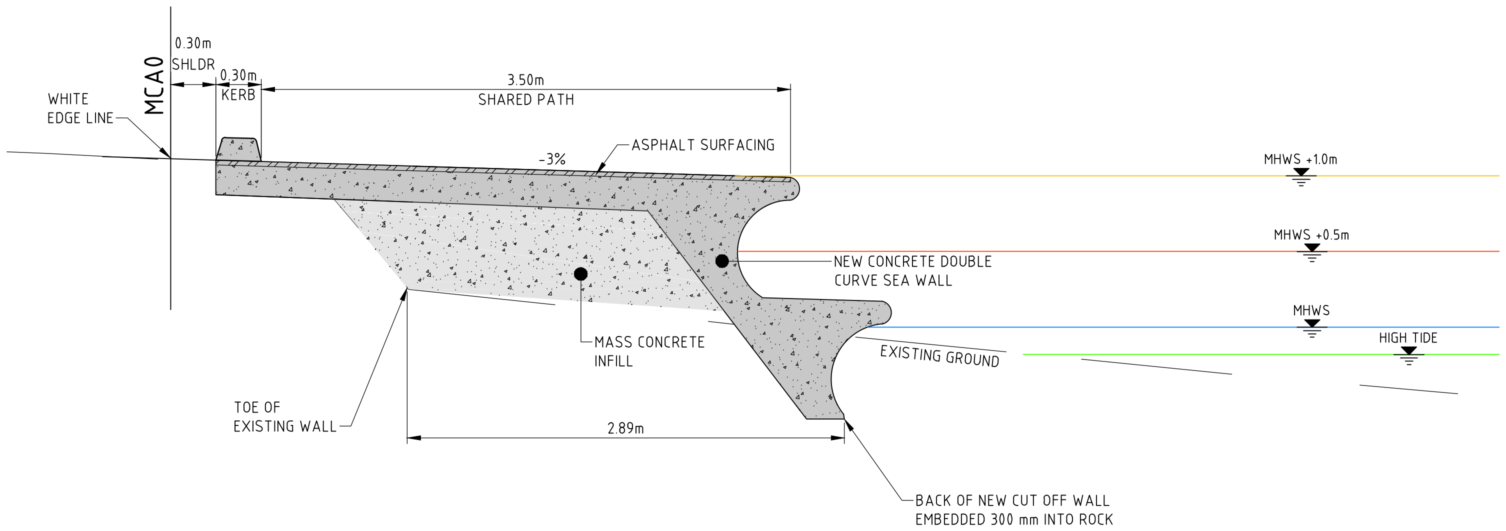
Yours sincerely

Michael Allis
Coastal Engineer

Appendix B Sea-level rise cross sections

The following pages contains Preliminary Design Plans (Revision J) which include MHWS elevations after a 0.5 m and 1.0 m of sea-level rise. [*Source*: pers. comm. Stantec (Caroline van Halderen), 23-11-2018]





SECTION A
SCALE 1 : 25

DO NOT SCALE - IF IN DOUBT, ASK

ORIGINAL SIZE A1

Last modified: Nov 23 2018 By Kevin Singh

NOT FOR CONSTRUCTION

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J	FOR CONSENT - ROCK REVETMENT REMOVED AND PATH WIDTH CHANGED	KVS	JP		08/18
H	FOR CONSENT - MINOR AMENDMENT	KVS	JP		05/18
G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

SURVEYED	
DESIGNED	
DRAWN	
CAD REVIEW	
DESIGN CHECK	
DESIGN REVIEW	NOT APPROVED
APPROVED	
PROF REGISTRATION:	

Stantec

Client: HUTT CITY TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
POINT HOWARD SORRENTO BAY STATION 720 - 900

Status Stamp: WORKING PLOT

Date Stamp:

Scales: 1 : 250 (A1)

Drawing No.: 80509137-01-001-C221

Rev.: J

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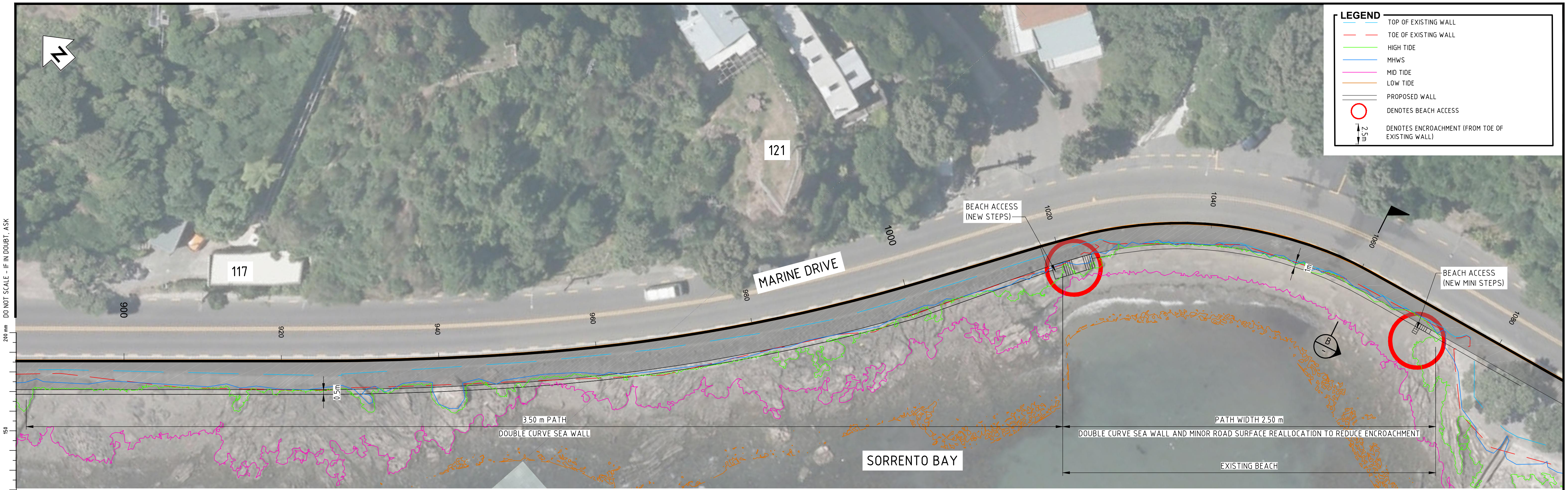
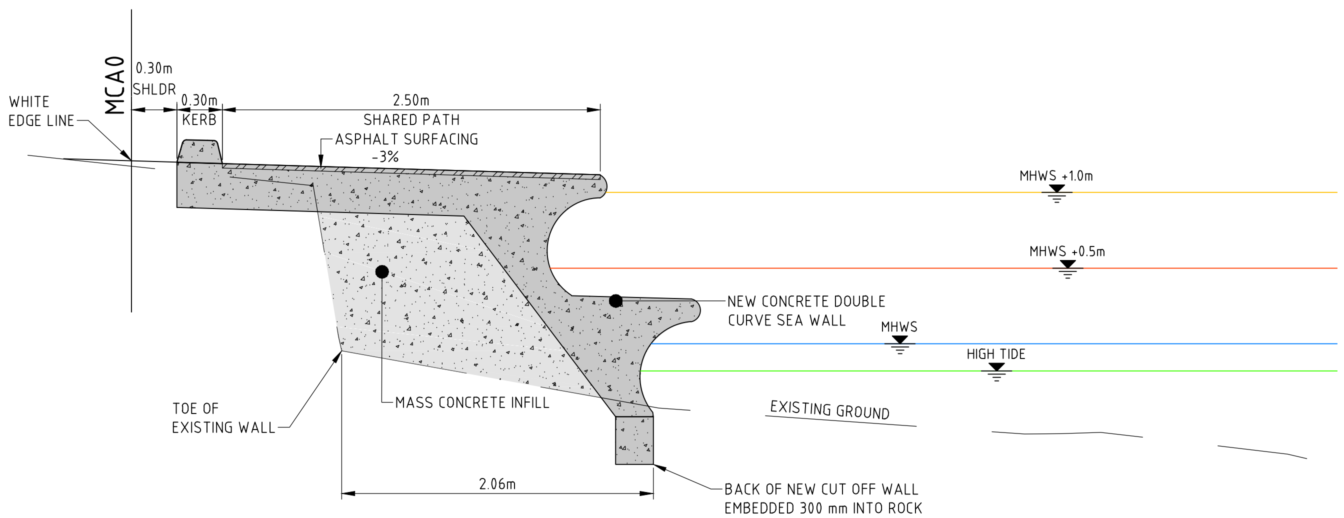
DO NOT SCALE - IF IN DOUBT, ASK





SECTION B -
SCALE 1 : 25

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G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

SURVEYED	
DESIGNED	
DRAWN	
CAD REVIEW	
DESIGN CHECK	
DESIGN REVIEW	NOT APPROVED
APPROVED	
PROF REGISTRATION:	

Stantec

Client: HUTT CITY TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
POINT HOWARD SORRENTO BAY STATION 900 - 1080

Status Stamp: WORKING PLOT
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Rev.: J

Last modified: Nov 23 2018 By Kevin Singh

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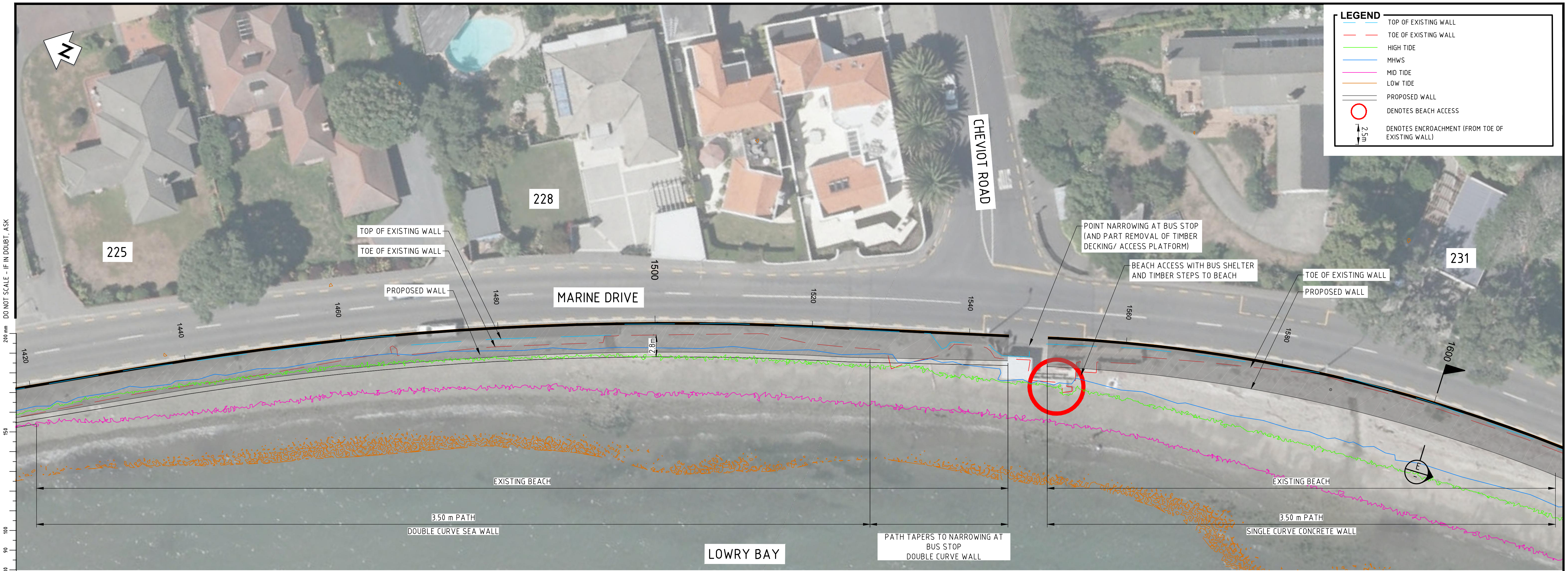

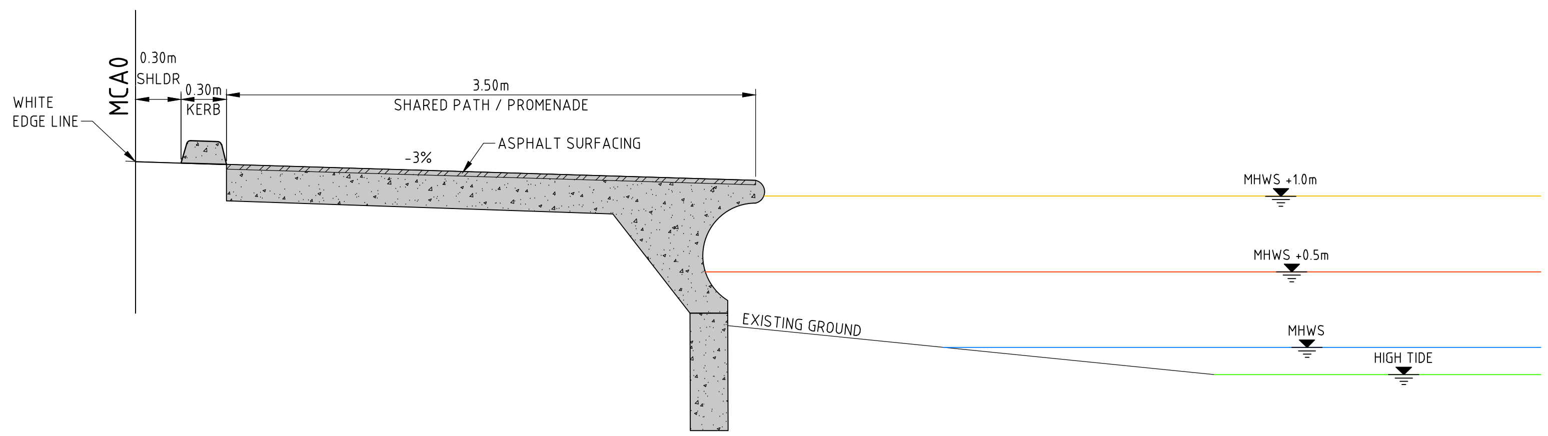

SECTION E
SCALE 1 : 25

NOT FOR CONSTRUCTION

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H	FOR CONSENT - MINOR AMENDMENT	KVS	JP		05/18
G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

SURVEYED
DESIGNED
DRAWN
CAD REVIEW
DESIGN CHECK
DESIGN REVIEW
APPROVED
PROF REGISTRATION:

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Stantec

Client: HUTT CITY TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
LOWRY BAY STATION 1420 - 1600

Status Stamp: WORKING PLOT
Date Stamp
Scales: 1 : 250
Drawing No. 80509137-01-001-C225
Rev. J

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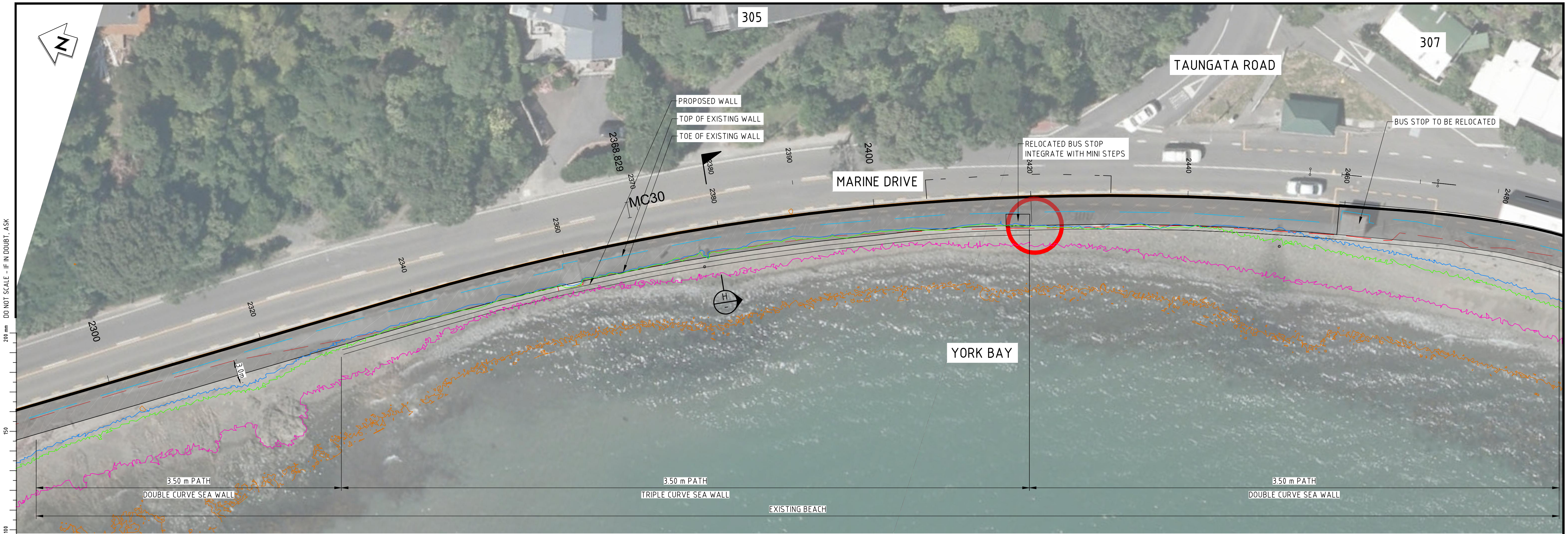

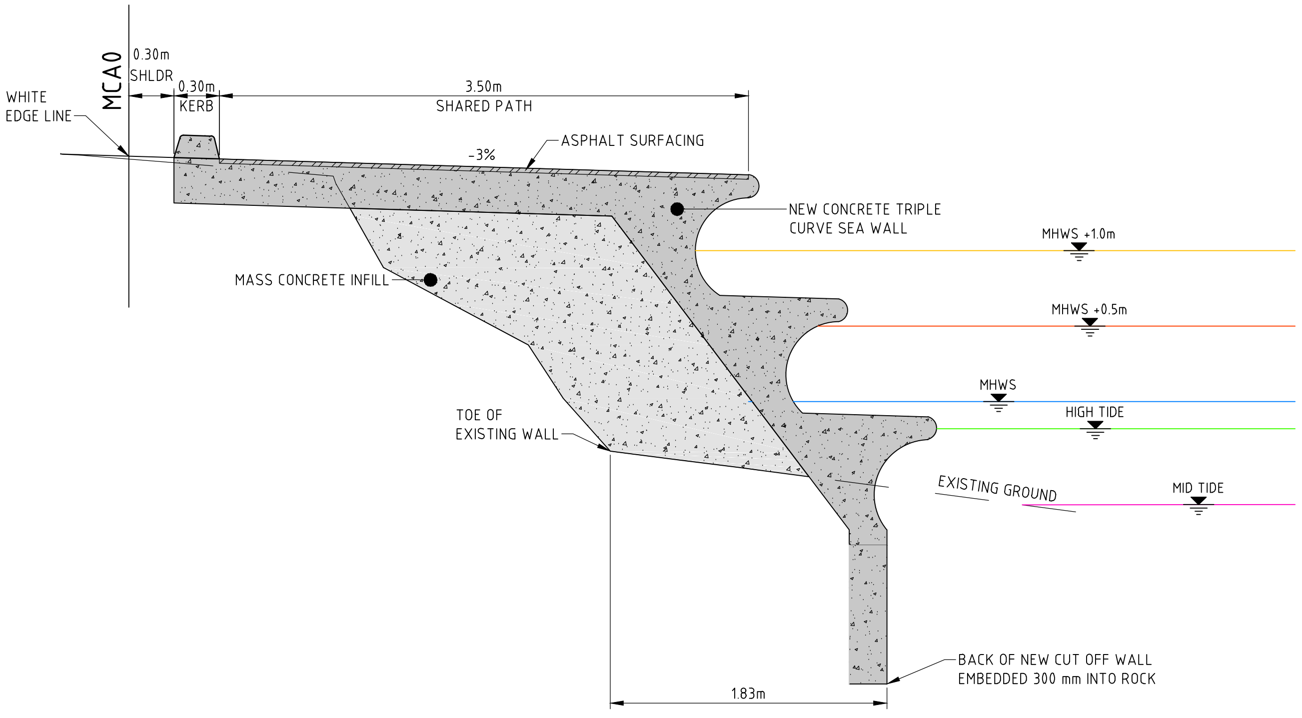

SECTION H
SCALE 1 : 25

LEGEND

- TOP OF EXISTING WALL
- TOE OF EXISTING WALL
- HIGH TIDE
- MHWS
- MID TIDE
- LOW TIDE
- PROPOSED WALL
- DENOTES BEACH ACCESS
- 2.5m DENOTES ENCROACHMENT (FROM TOE OF EXISTING WALL)

DO NOT SCALE - IF IN DOUBT, ASK

ORIGINAL SIZE A1

Last modified: Nov 23 2018 By Kevin Singh

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REV	REVISIONS	DRN	CHK	APP	DATE
J	FOR CONSENT - ROCK REVETMENT REMOVED AND PATH WIDTH CHANGED	KVS	JP		08/18
H	FOR CONSENT - MINOR AMENDMENT	KVS	JP		05/18
G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

SURVEYED
DESIGNED
DRAWN
CAD REVIEW
DESIGN CHECK
DESIGN REVIEW
APPROVED
PROF REGISTRATION:

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Stantec

Client: HUTT CITY TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
YORK BAY STATION 2300 - 2480

Status Stamp: WORKING PLOT
Date Stamp:
Scales: 1 : 250
Drawing No. 80509137-01-001-C230
Rev. J

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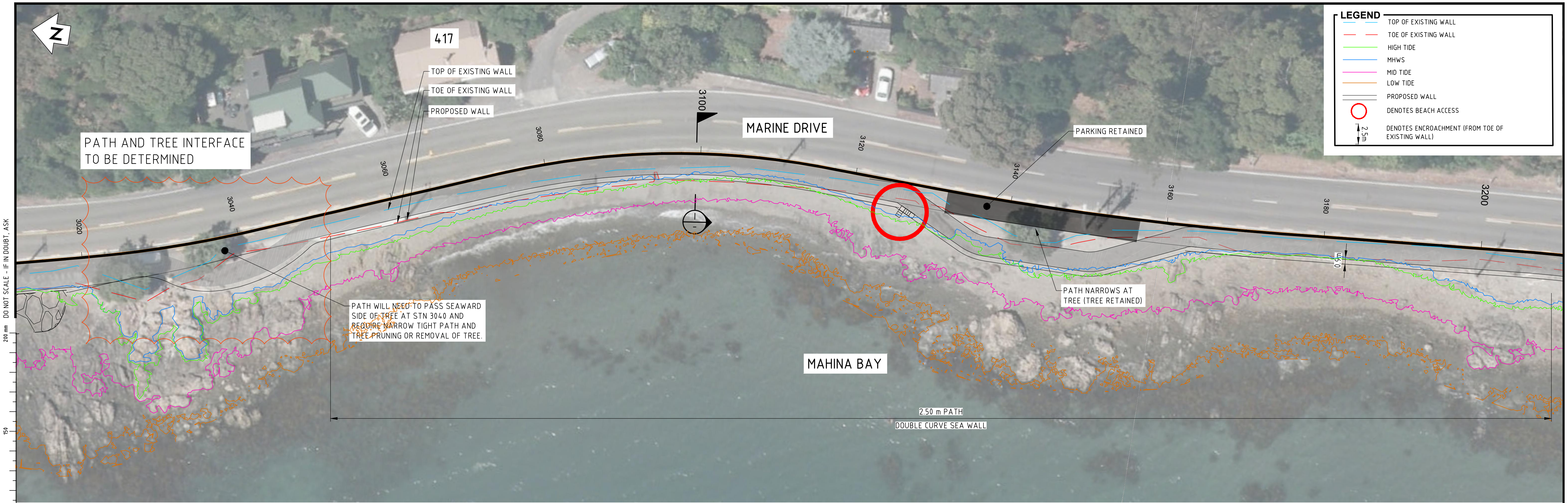

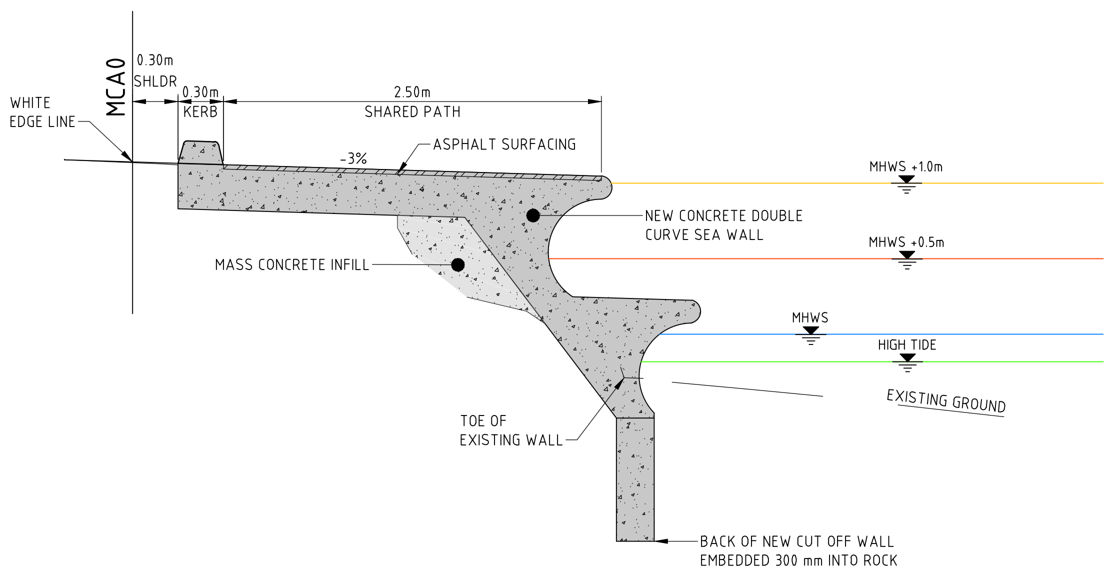

SECTION 1 / -
SCALE 1 : 25

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ORIGINAL SIZE A1

Last modified: Nov 23 2018 By Kevin Singh

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REV	REVISIONS	DRN	CHK	APP	DATE
J	FOR CONSENT - ROCK REVETMENT REMOVED AND PATH WIDTH CHANGED	KVS	JP		08/18
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G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

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DRAWN
CAD REVIEW
DESIGN CHECK
DESIGN REVIEW
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PROF REGISTRATION:

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Stantec

Client: HUTT CITY TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
MAHINA BAY STATION 3020 - 3200

Status Stamp: WORKING PLOT
Date Stamp
Scales: 1 : 250 (A1)
Drawing No.: 80509137-01-001-C234
Rev.: J

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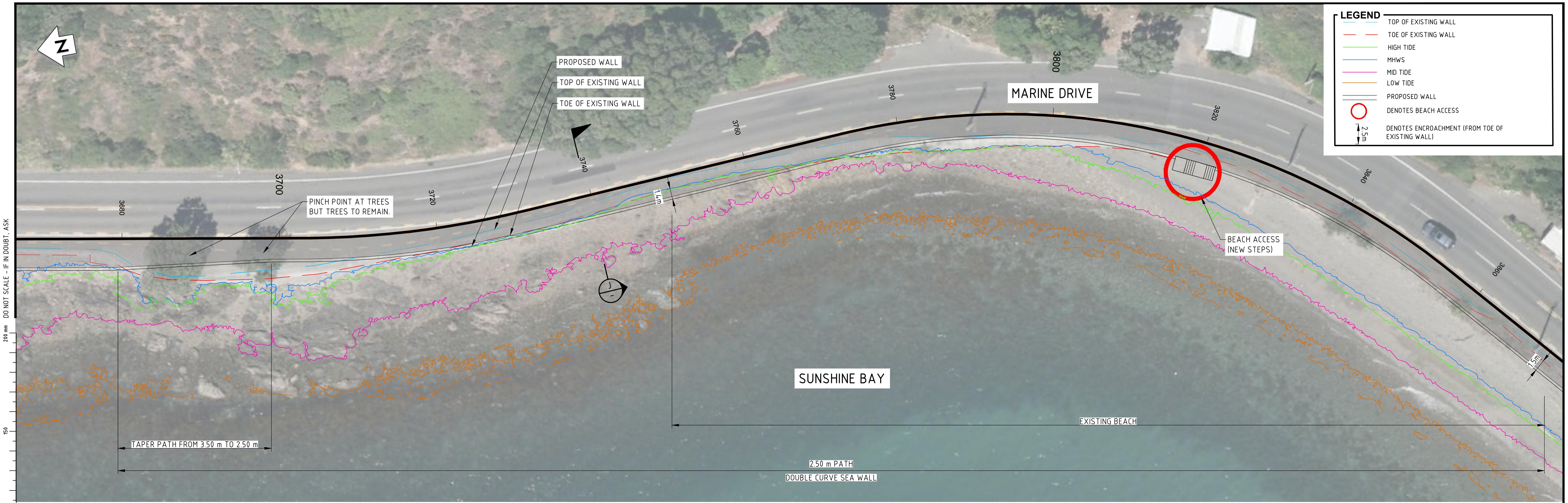

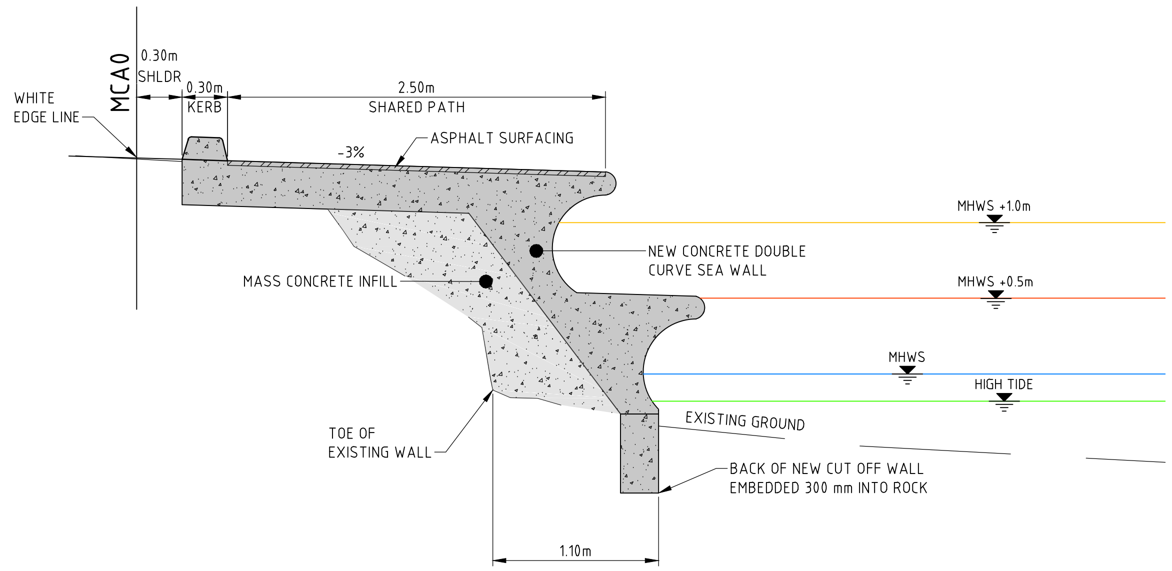

SECTION J / -
SCALE 1 : 25

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J	FOR CONSENT - ROCK REVETMENT REMOVED AND PATH WIDTH CHANGED	KVS	JP		08/18
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G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

SURVEYED
DESIGNED
DRAWN
CAD REVIEW
DESIGN CHECK
DESIGN REVIEW
APPROVED
PROF REGISTRATION:

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Stantec

Client: HUTT CITY TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
SUNSHINE BAY STATION 3680 - 3860

Status Stamp: WORKING PLOT
Date Stamp:
Scales: 1 : 250 (A1)
Drawing No. 80509137-01-001-C238
Rev. J

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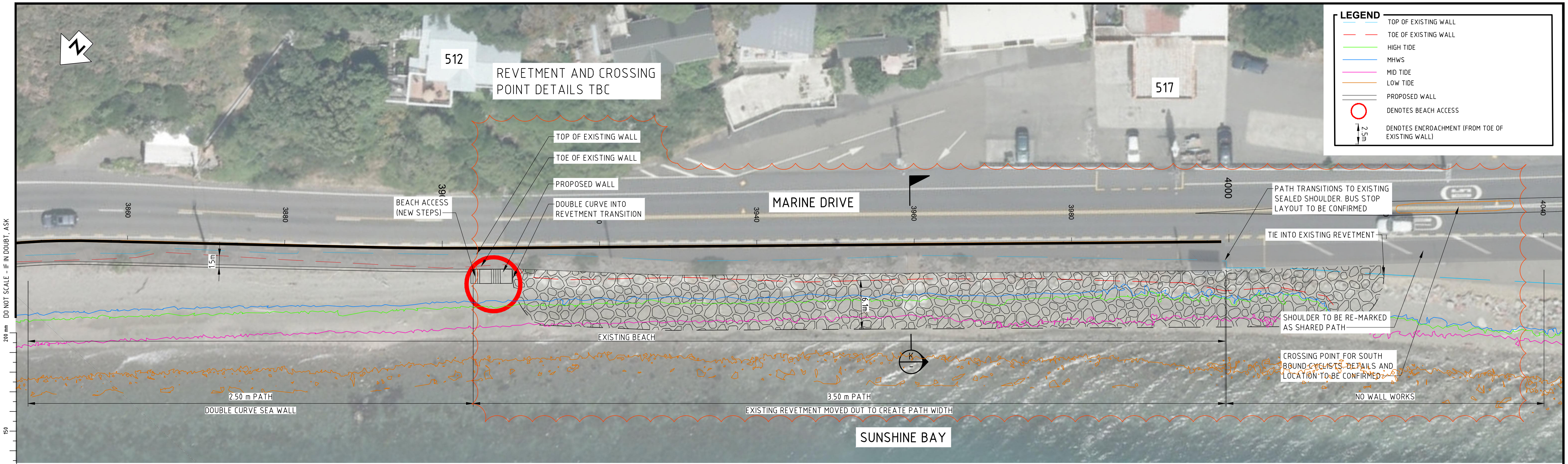

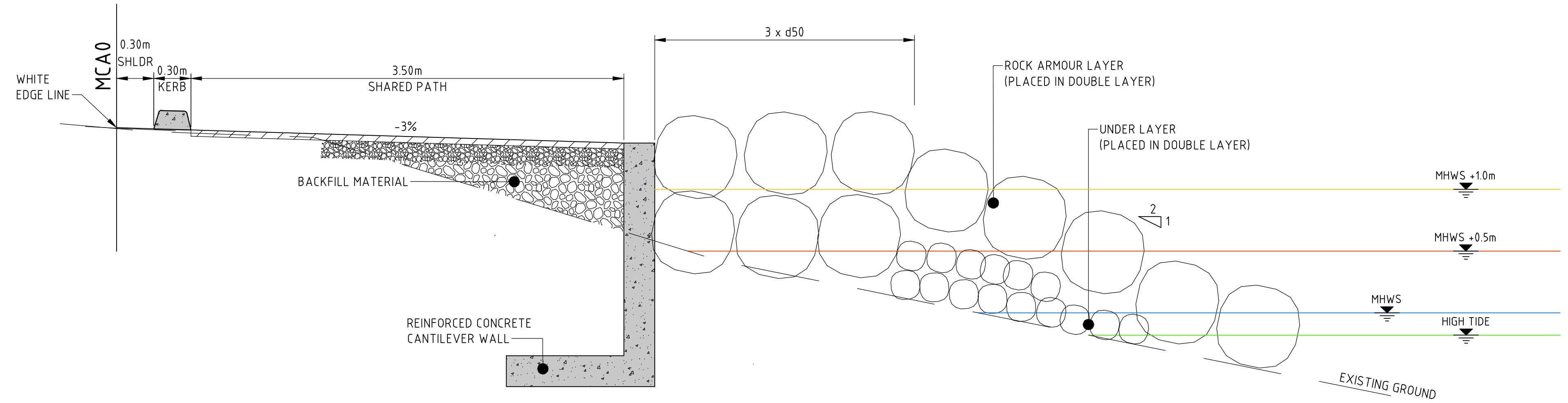

SECTION K
SCALE 1 : 25

DO NOT SCALE - IF IN DOUBT, ASK

ORIGINAL SIZE A1

Last modified: Nov 23 2018 By Kevin Singh

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REV	REVISIONS	DRN	CHK	APP	DATE
J	FOR CONSENT - ROCK REVETMENT REMOVED AND PATH WIDTH CHANGED	KVS	JP		08/18
H	FOR CONSENT - MINOR AMENDMENT	KVS	JP		05/18
G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

SURVEYED
DESIGNED
DRAWN
CAD REVIEW
DESIGN CHECK
DESIGN REVIEW
APPROVED
PROF REGISTRATION:

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Stantec

Client:
HUTT CITY TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
SUNSHINE BAY STATION 3860 - 4040

Status Stamp: **WORKING PLOT**
Date Stamp
Scales: 1 : 250 (A1)
Drawing No.: 80509137-01-001-C239
Rev.: J

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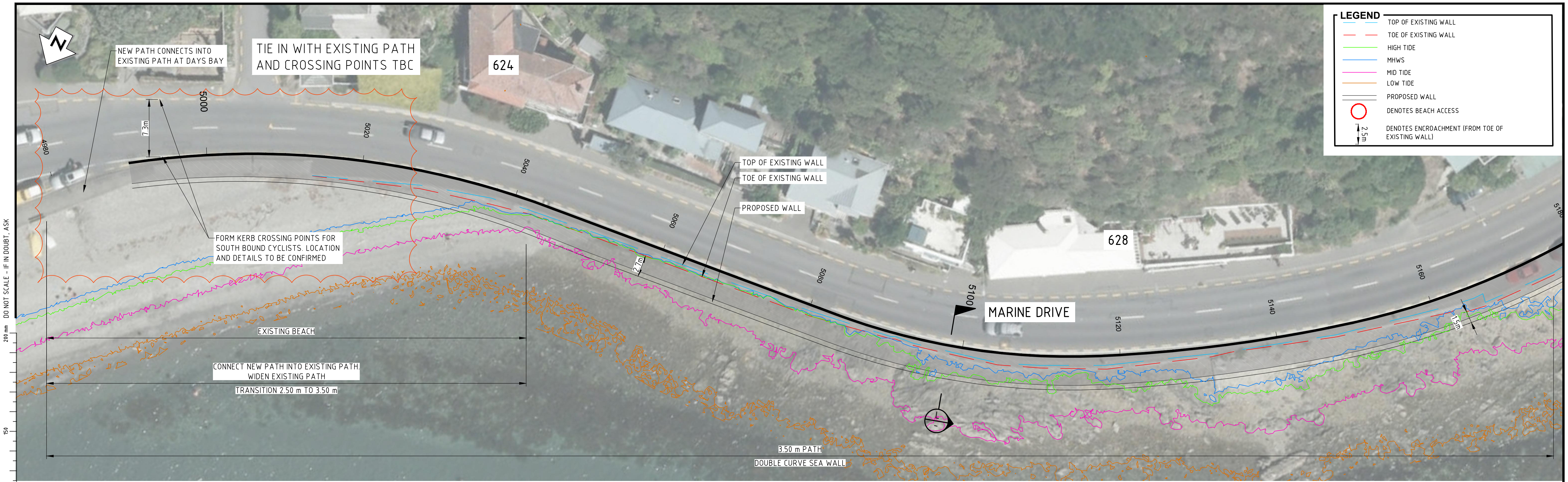

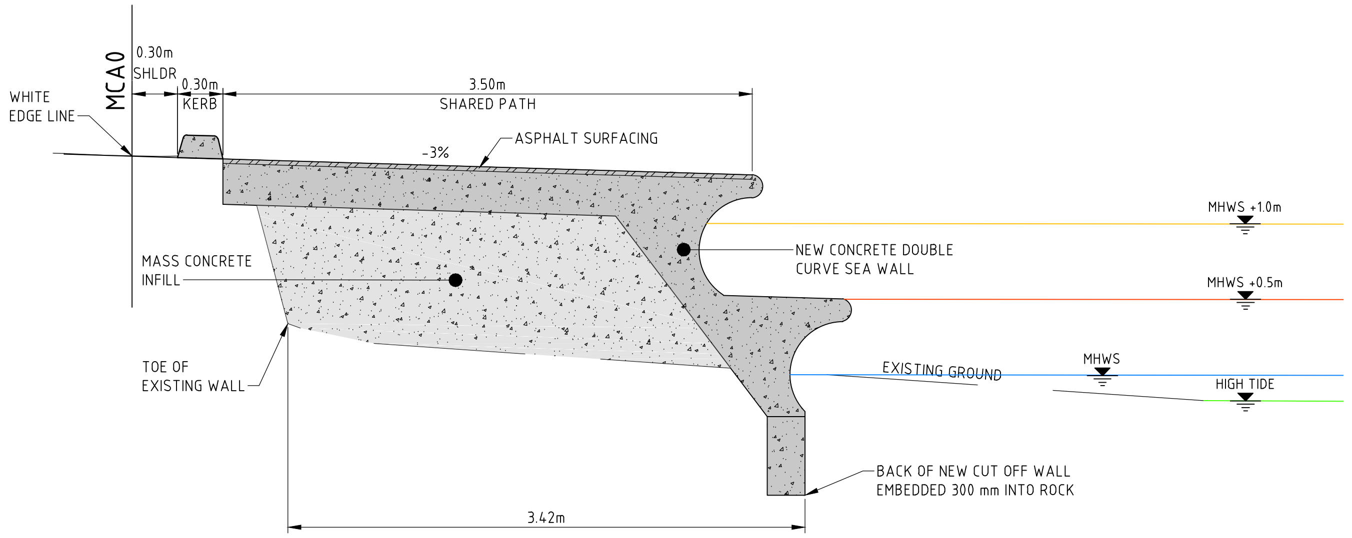

SECTION L -
SCALE 1 : 25

DO NOT SCALE - IF IN DOUBT, ASK

ORIGINAL SIZE A1

Last modified: Nov 23 2018 By Kevin Singh

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REV	REVISIONS	DRN	CHK	APP	DATE
J	FOR CONSENT - ROCK REVETMENT REMOVED AND PATH WIDTH CHANGED	KVS	JP		08/18
H	FOR CONSENT - MINOR AMENDMENT	KVS	JP		05/18
G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

SURVEYED
DESIGNED
DRAWN
CAD REVIEW
DESIGN CHECK
DESIGN REVIEW
APPROVED
PROF REGISTRATION:

NOT APPROVED

Stantec

Client: HUTT CITY TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
WINDY POINT STATION 4980 - 5180

Status Stamp: WORKING PLOT
Date Stamp
Scales: AS SHOWN
Drawing No. 80509137-01-001-C240
Rev. J

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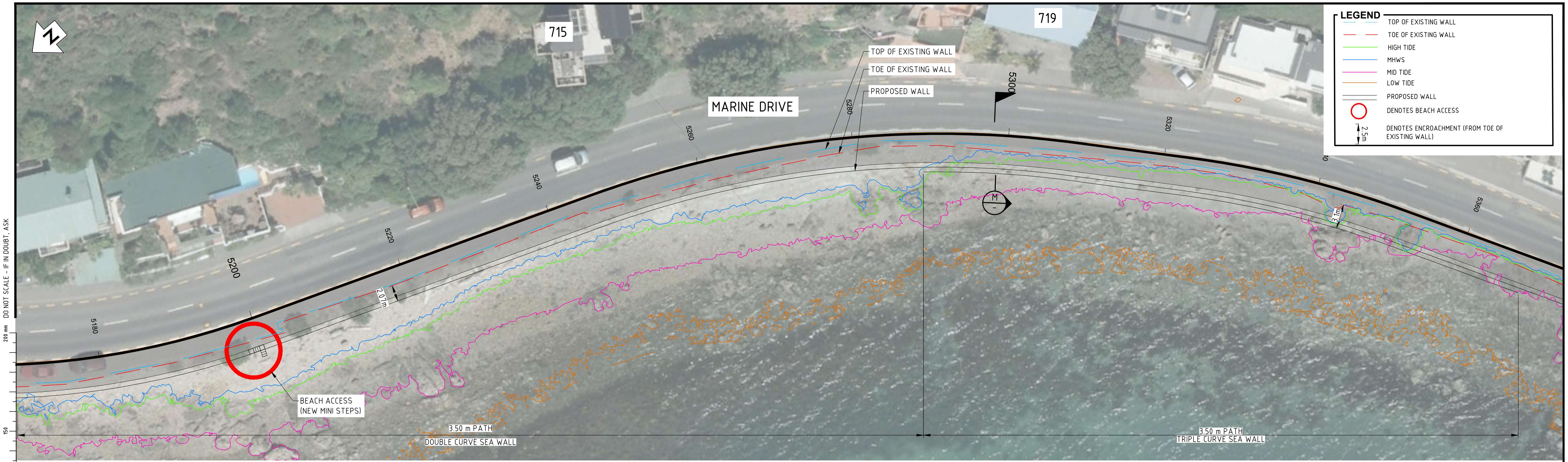

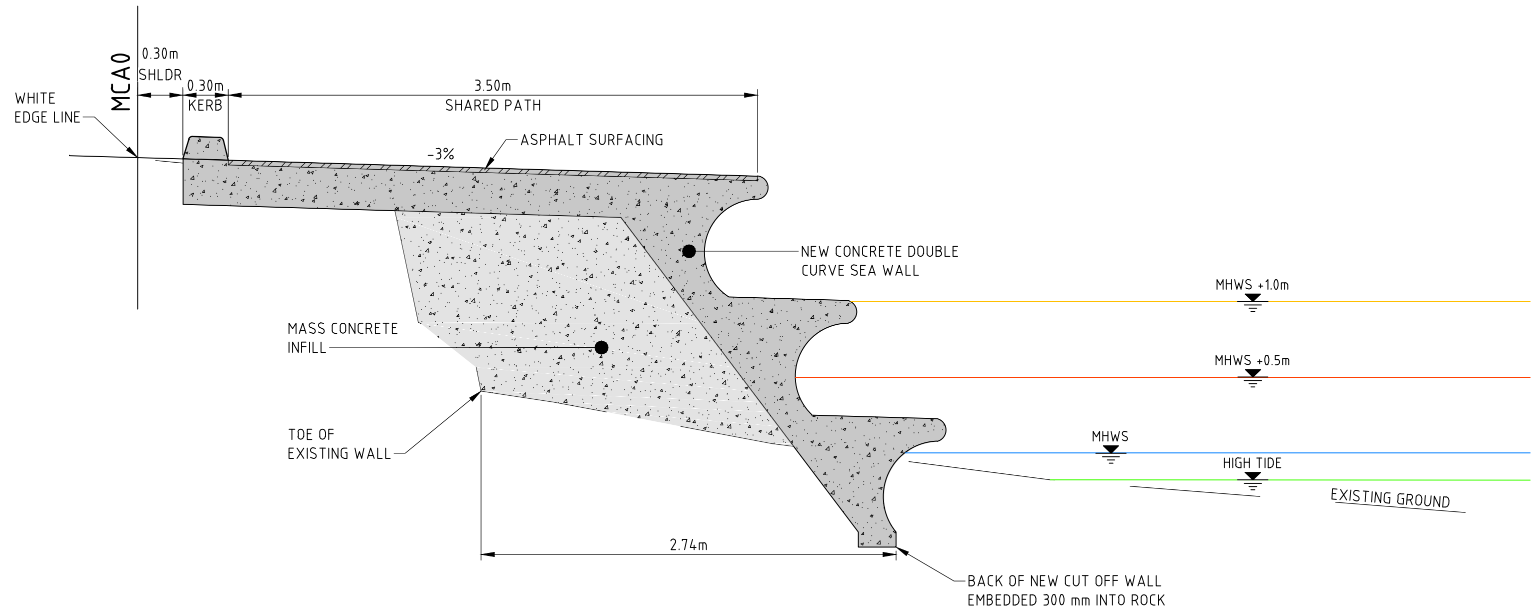

SECTION M
SCALE 1 : 25

DO NOT SCALE - IF IN DOUBT, ASK

ORIGINAL SIZE A1

Last modified: Nov 23 2018 By Kevin Singh

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REV	REVISIONS	DRN	CHK	APP	DATE
J	FOR CONSENT - ROCK REVETMENT REMOVED AND PATH WIDTH CHANGED	KVS	JP		08/18
H	FOR CONSENT - MINOR AMENDMENT	KVS	JP		05/18
G	FOR CONSENT	KVS	JP		04/18
F	MINOR AMENDMENT	KVS	JP		09.03.2018
E	MINOR AMENDMENT	KVS	JP		02.03.2018
D	PRELIMINARY DESIGN	GC	JP		19.09.17
C	FOR REVIEW - MANY SHEETS ADDED, MANY SHEETS RE-NUMBERED	PJ COOK	JP		08/17

SURVEYED
DESIGNED
DRAWN
CAD REVIEW
DESIGN CHECK
DESIGN REVIEW
APPROVED
PROF REGISTRATION:

NOT APPROVED

Stantec

Client:
HUTT CITY
TE AWA KAIRANGI

HUTT CITY COUNCIL
EASTERN BAYS SHARED PATH - DBC

PLAN - MCA0
WINDY POINT STATION 5180 - 5360

Status Stamp: WORKING PLOT
Date Stamp
Scales: AS SHOWN
Drawing No.: 80509137-01-001-C241
Rev.: J

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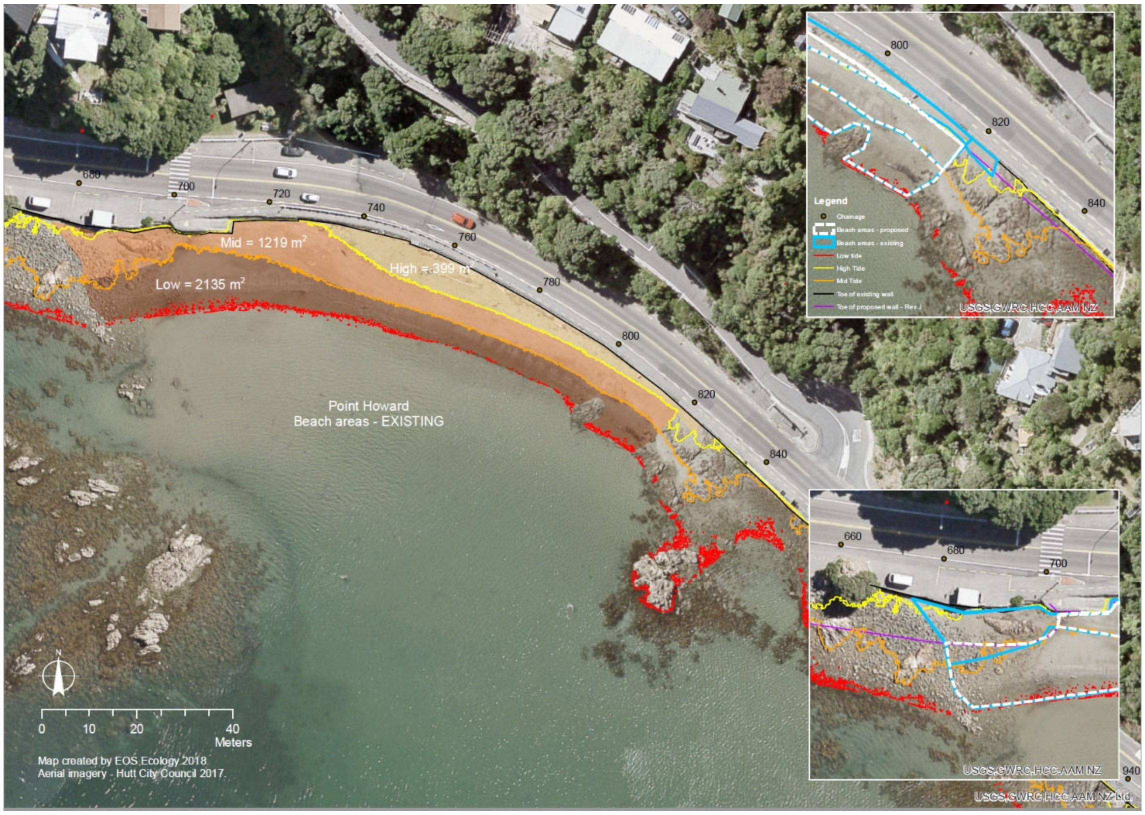
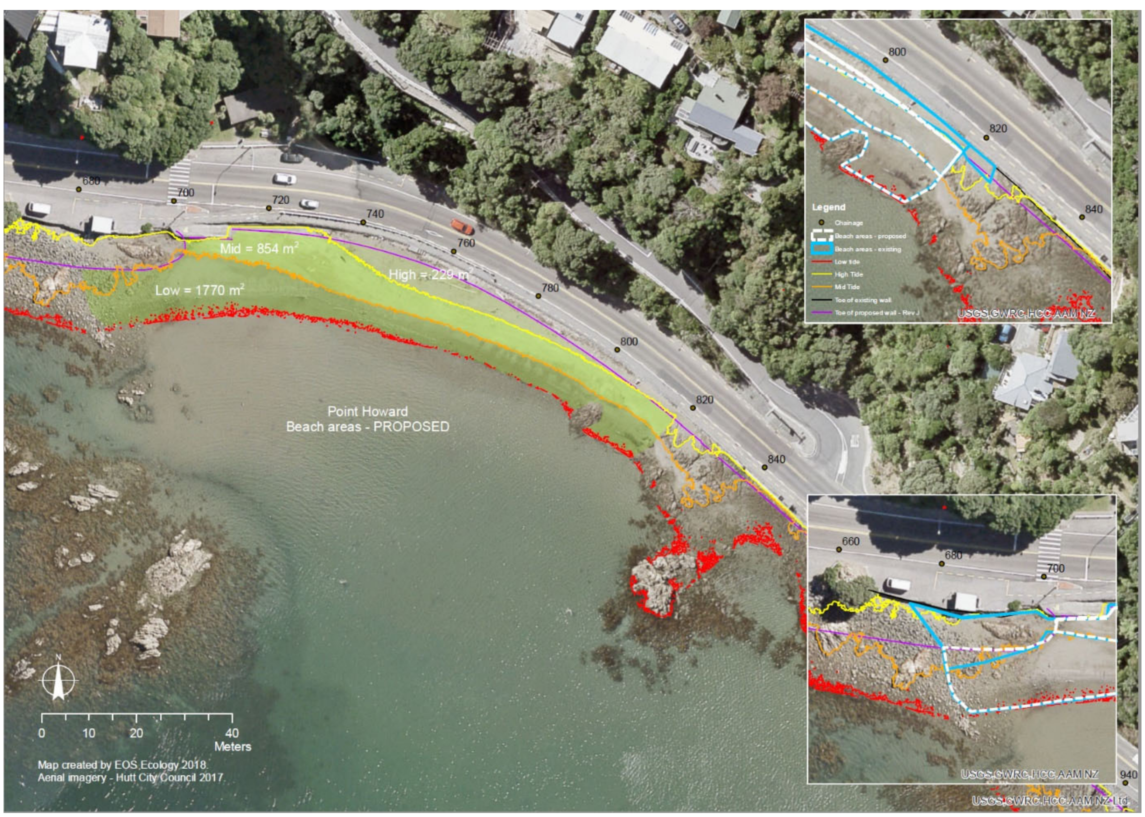
Appendix C Appendix C Beach area calculations

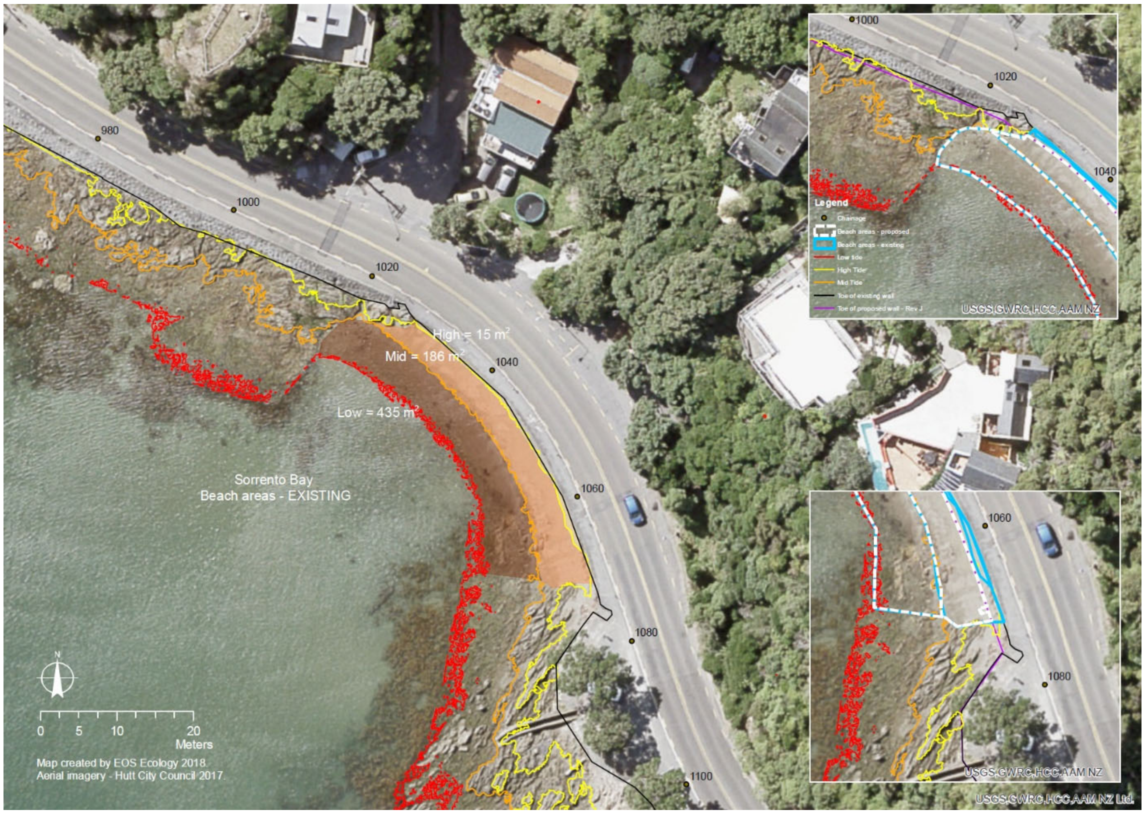
The following pages contain calculations of the beach areas available for public amenity in the existing environment and following proposed seawall construction. The calculations do not include the offsetting effects of beach nourishment but are the values underpinning the beach nourishment calculations.

Tidal levels referred to are nautical tides, with elevations of -0.29 m WVD-53 (low tide), 0.195 m WVD-53 (mid tide or MSL) and 0.68 m WVD-53 (high tide).

The proposed beach areas are for Preliminary Design Plans (Revision J). [*Source*: pers. comm. EOS Ecology (Kirsty Brennan), 21-12-2018]


Mid = 1219 m²
High = 399 m²
Low = 2135 m²
Point Howard
Beach areas - EXISTING
Legend
Meters
Map created by EOS Ecology 2018.
Aerial imagery - Hutt City Council 2017.

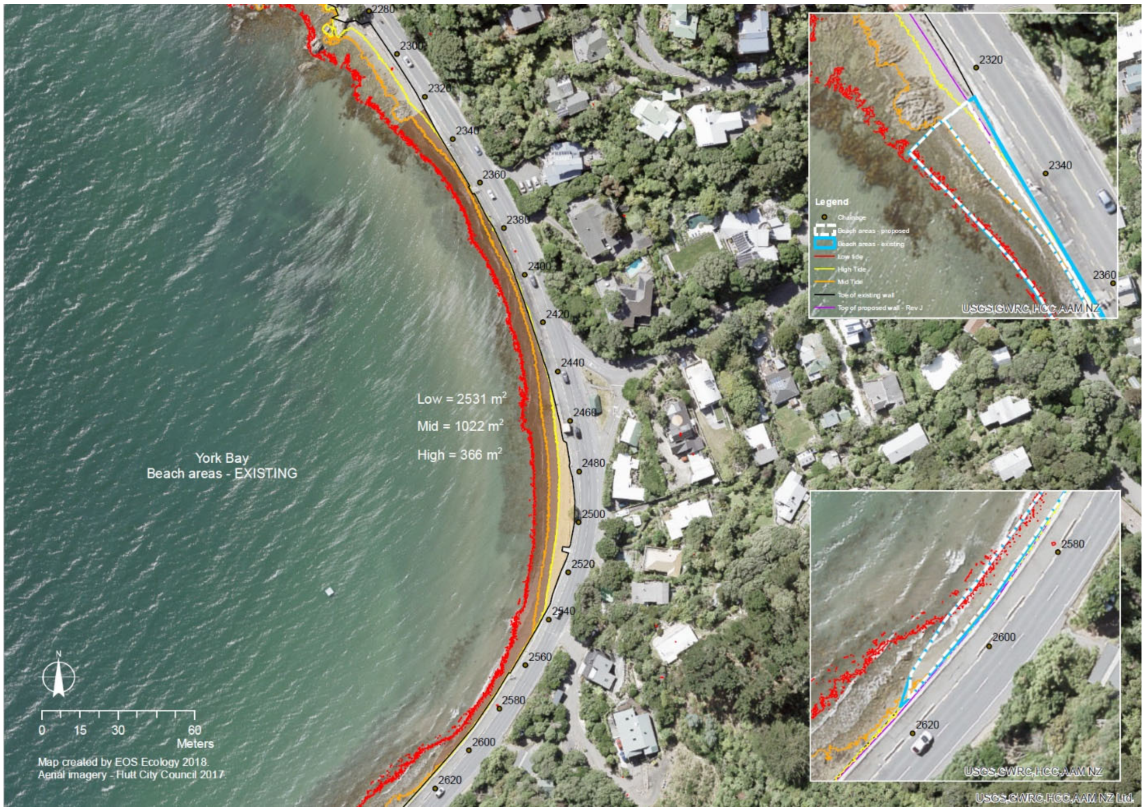

Mid = 854 m²
High = 229 m²
Low = 1770 m²
Point Howard
Beach areas - PROPOSED
Legend
Meters
Map created by EOS Ecology 2018.
Aerial imagery - Hutt City Council 2017.

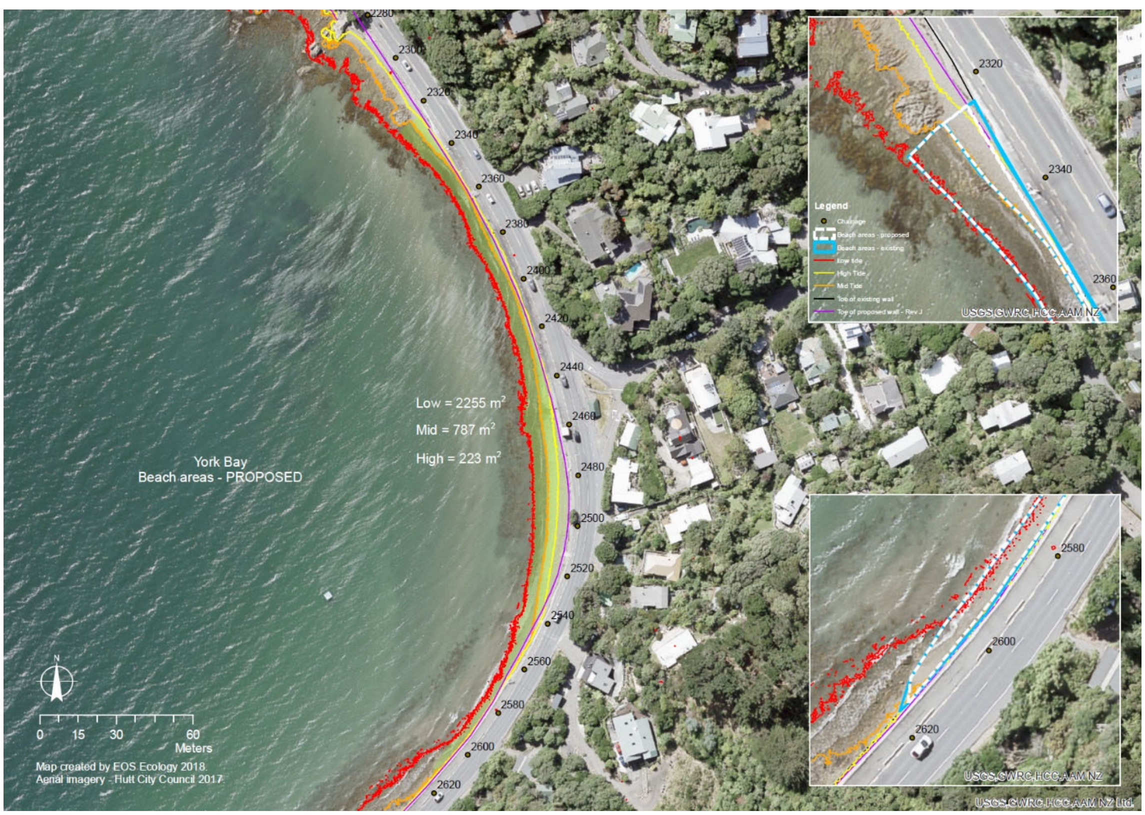


High = 15 m²
Mid = 186 m²
Low = 435 m²
Sorrento Bay
Beach areas - EXISTING
0 5 10 20
Meters
Map created by EOS Ecology 2018.
Aerial imagery - Hutt City Council 2017.
Legend
USGS,GWRC,HCC,AAM NZ Ltd.

High = 0 m²
Mid = 145 m²
Low = 394 m²
Sorrento Bay
Beach areas - PROPOSED
0 5 10 20
Meters
Map created by EOS Ecology 2018.
Aerial imagery - Hutt City Council 2017.
Legend
USGS,GWRC,HCC,AAM NZ Ltd.

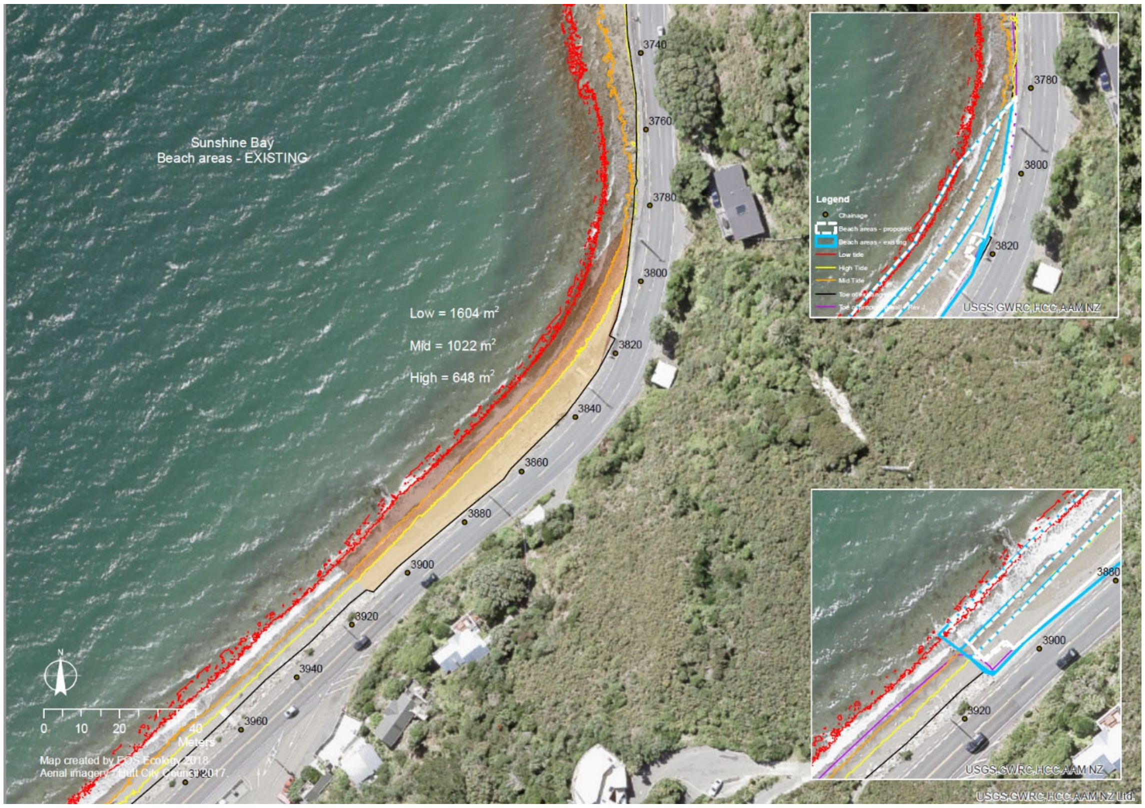
NIWA
Taihoro Nukurangi

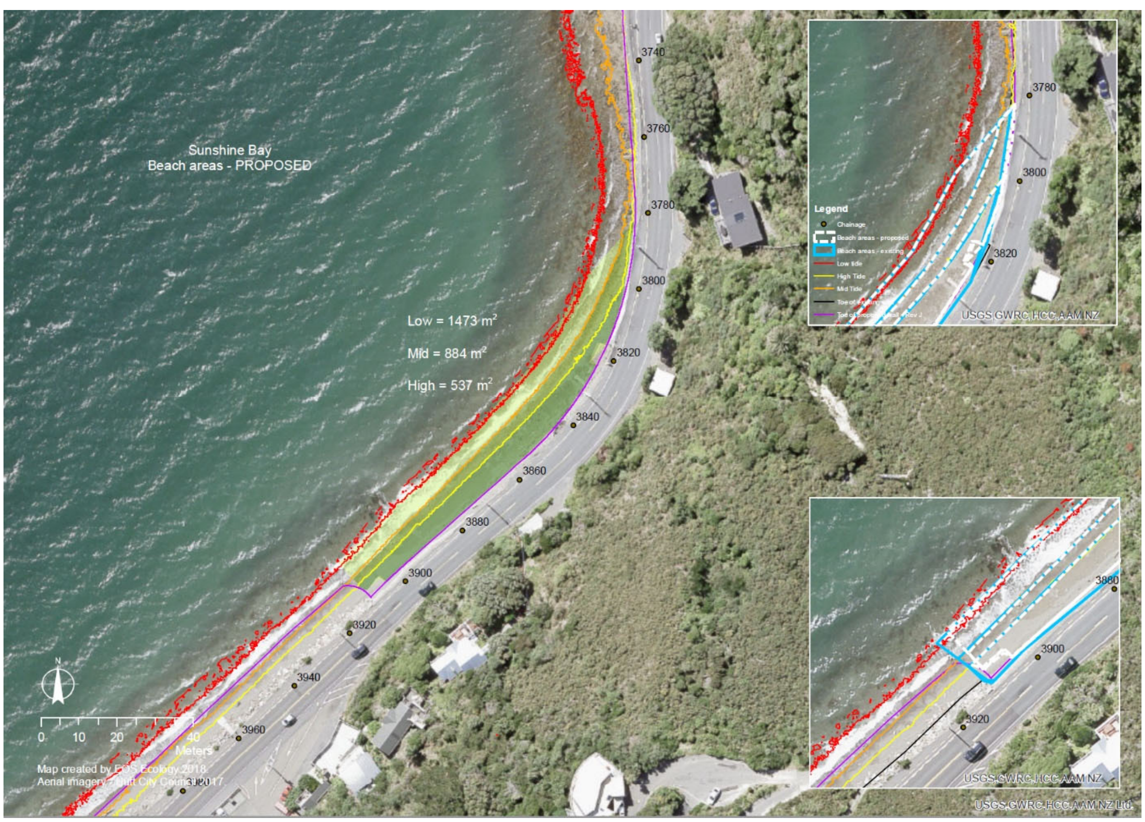

Low = 2531 m2
Mid = 1022 m2
High = 366 m2
York Bay
Beach areas - EXISTING
Map created by EOS Ecology 2018.
Aerial imagery - Hutt City Council 2017
Legend
USGS,GWRC,HCC,AAM NZ Ltd.


Low = 2255 m2
Mid = 787 m2
High = 223 m2
York Bay
Beach areas - PROPOSED
Map created by EOS Ecology 2018.
Aerial imagery - Hutt City Council 2017
Legend
USGS,GWRC,HCC,AAM NZ Ltd.


Mahina Bay
Beach areas - EXISTING
Low = 2775 m2
Mid = 1637 m2
High = 720 m2
Legend
Chainage
Beach areas - proposed
Beach areas - existing
Low tide
High Tide
Mid Tide
Toe of existing wall
Toe of proposed wall
Meters
Map created by EOS Ecology 2018
Aerial imagery - Hutt City Council 2017
USGS, GWRC, HCC, AAM NZ Ltd.


Mahina Bay
Beach areas - PROPOSED
Low = 2498 m2
Mid = 1277 m2
High = 351 m2
Legend
Chainage
Beach areas - proposed
Beach areas - existing
Low tide
High Tide
Mid Tide
Toe of existing wall
Toe of proposed wall
Meters
Map created by EOS Ecology 2018
Aerial imagery - Hutt City Council 2017
USGS, GWRC, HCC, AAM NZ Ltd.


Sunshine Bay
Beach areas - EXISTING
Low = 1604 m2
Mid = 1022 m2
High = 648 m2
Legend
Map created by EOS Ecology 2018
Aerial imagery - Hutt City Council 2017
USGS GWRC HCC AAM NZ Ltd


Sunshine Bay
Beach areas - PROPOSED
Low = 1473 m2
Mid = 884 m2
High = 537 m2
Legend
Map created by EOS Ecology 2018
Aerial imagery - Hutt City Council 2017
USGS GWRC HCC AAM NZ Ltd