Takutai Kāpiti – Coastal Science Engineering Services

Kāpiti Coast Coastal Hazard Susceptibility and Vulnerability Assessment
Volume 1: Methodology

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Takutai Kāpiti – Coastal Science and Engineering Services

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Glossary

Term	Definition
AEP	Annual Exceedance Probability. The probability expressed as a percentage that an event larger that the magnitude specified will occur in any one year.
ARI	Average Recurrence Interval. The average or expected time interval (e.g. years) between exceedances of a given event.
CED	Coastal Erosion Distance
Cell Boundary	The boundary between coastal erosion assessment cells used in this study.
CEPD	Coastal Erosion Prediction Distance. From Coastal Systems Limited (2008, 2012), being the distance that the open coast is predicted to erode in specified future timeframes.
CHMA	Coastal Hazard Management Area
DS	Dune Stability
Dynamic equilibrium	Long-term fluctuations in the reference shoreline position about a static position
ENSO	El Niño Southern Oscillation. An irregular periodic variation in winds and sea surface temperatures over the tropical eastern Pacific Ocean which effects sea levels.
EPR	End Point Rate. Average rate of change in shoreline position calculated by dividing the difference in shoreline position (distance) from the first observation/measurement (usually aerial photograph or survey) to the last by the observation/measurement by the time period between the first and last observations/measurements.
Fluvial Method	Method 4 of the Coastal Hydrosystem Decision Tree where future inlet migration has been based in inundation mapping of 0.4m, 0.6m and 1.65m SLR with a 1% AEP storm tide event.
GL	Ground Level
GWL	Ground Water Level
GWRC	Greater Wellington Regional Council
HAT	Highest Astronomical Tide
Historical Envelope Method	Method 3 of the Coastal Hydrosystem Decision Tree where no inlet migration line determined; maximum historical envelope is shown. This is a conservative approach.
Hs	Significant Wave Height. It is the height of the highest third of waves within a time series.
Hydrosystem	Includes river/stream mouths and estuaries where there is a coastal influence on the location and morphology of these environments.

Term	Definition
IEPD	Inlet Erosion Hazard Prediction Distance From Coastal Systems Limited (2008, 2012), being the erosion distance that river/stream mouths were predicted to predicted to erode in specified future timeframes.
IMC	Inlet Migration Curve From Coastal Systems Limited (2008, 2012), being the envelope of past change in shoreline position around the vicinity of each river/stream mouth based on historical aerial imagery.
IPCC	Intergovernmental Panel on Climate Change
IPO	Interdecadal Pacific Oscillation. A large-scale, long period oscillation that influences climate variability over the Pacific Basin with phases lasting around 20 to 30 years.
KCDC	Kāpiti Coast District Council
LiDAR	Light Detection and Ranging. A remote sensing method that uses light in the form of a pulsed laser to measure ranges (variable distances) to the Earth from measuring device, usually mouthed on a plane or drone.
Likely	>66% probability of occurrence
LINZ	Land Information New Zealand
Long-term accretion	Long-term beach growth which results in the reference shoreline moving in a seaward direction
Long-term retreat	Long-term erosion of the beach causing the reference shoreline to move landward
LRR	Linear Regression Rate. Average rate of change in shoreline position calculated by linear regression of all the observation/measurements of positions within the time period between the first and last observations/measurements.
MfE	Ministry for the Environment
MHWS	Mean High Water Springs
MHWPS	Mean High Water Perigean Springs. The high tides that occur 6-8 times a year when the moon is either new or full and closest to the Earth, hence a stronger gravitational pull and higher tides than normal spring tides.
Most Likely Shoreline Position	The most likely position that the shoreline will be over the proposed timeframe, calculated between the P33 and P66 probability of occurrence.
MSL	Mean Sea Level . Average level of the surface of a water body.
MSLA	Mean Sea Level Anomaly. The non-tidal variation in sea level at scales from monthly to decades due to climate variability.
NZCPS	New Zealand Coastal Policy Statement released by the Department of Conservation in 2010. This a mandatory national policy statement under the Resource Management Act 1991.

Term	Definition
PFSP	Projected Future Shoreline Position. General term used for the calculated shoreline position over the assessed time period.
Potential run-up overtopping areas	Areas where the beach dune or crest ridge is below the elevation of the estimated wave run-up in a 1% AEP coastal storm event should that beach overtopping could occur resulting in an increase in coastal inundation depth and/or extent.
Potential Shoreline Zone	Area between the P0 (present day shoreline) and the P90. It is considered that erosion beyond the landward limit of this zone is 'very unlikely' to occur.
'Present day' erosion susceptibility	The magnitude of erosion that could occur should a 1% AEP storm occur. Is calculated from the additional of the short-term and dune stability components of the CED formula to determine the PFSP
Reference shoreline position	The position on the shoreline taken as the reference point for all CED calculations. For unprotected shorelines, is taken as the beach vegetation line on sand beaches, and the back of the gravel beach from the Ōtaki River to Te Horo. For shorelines protected by seawalls, is taken to be the position of these structures. This is taken from 2017 aerial imagery.
RCP	Representative Concentration Pathway. A greenhouse gas concentration trajectory adopted by the IPCC to describe different climate futures, all of which are considered possible depending on the volume of greenhouse gases emitted in the years to come.
RMA	Resource Management Act 1991
RSLR	Relative Sea Level Rise. The total sea level rise from climate change and local vertical local movements relative to the level of the land.
SEE	Standard Error of Estimate
SLR	Sea Level Rise
SROCC	Special Report on the Ocean and Cryosphere in a Change Climate (IPCC, 2019).
SSE	Slow Slip Events. These are long lived shear slip events at subduction interfaces and the physical processes responsible for the generation of slow earthquakes.
Translation method	Method 2 of the Coastal Hydrosystem Decision Tree.
Translation/Historical Envelope Method	Approach used where over the 2050/2070 the historical envelope has been used to show the potential hydrosystem migration zone, however in 2070/2120 the PFSP open coast shorelines are landward of the historical shoreline extent of the hydrosystem. In this instance, the present day shoreline has been translated landward to meet the adjacent PFSP line. This is then incorporated with the historical envelope of the inlet where the historical envelope is more landward of the translated shape.
Tp	Peak Wave Period. Wave period is the time between waves hence the inverse of wave frequency, with the peak period being the wave period with the highest energy within a time series.
Unlikely	< 33% probability of occurrence
Very Unlikely	< 10% probability of occurrence

Term	Definition
VLM	Vertical Land Movement
WASP	Wave and Storm-surge Projections Project
WVD53	Wellington Vertical Datum 1953 (WVD53). This is the local Vertical Datum for land surveys established in 1953, in which zero elevation was set at MSL at the time. However, with sea level rise (SLR) since that time, the elevation of MSL for the 2005 - 2011 period was established to be 0.196 m WVD53 and increased to 0.25 m WVD53 for the 2012-2018 period (Bosserelle and Lane, 2019).

1. Introduction

1.1 Purpose

As part of their "*Takutai Kāpiti: Our community-led coastal adaptation project*", the Kāpiti Coast District Council (KCDC) have initiated a Coastal Hazard Susceptibility and Vulnerability Assessment for the whole 38 km of the Kāpiti Coast District coastline from Ōtaki in the north to Paekākāriki in the south (Figure 1.1). The purpose of this assessment is to update previous coastal hazard assessments undertaken along the Kāpiti Coast District shoreline involving the identification of areas susceptible to current and future coastal erosion and inundation hazards, and having consideration of how these will interact with other hazards in the coastal environment including high groundwater levels, pluvial, and fluvial flooding.

The outputs of the assessment include map overlays of coastal hazard susceptibility with various magnitudes of future sea level rise (SLR) and the vulnerability of key council infrastructure, community services and private property to those hazards, and a report detailing the methodology and limitations for undertaking the assessments. These outputs could be used by KCDC in a number of ways including; community awareness initiatives, for the development of future coastal asset planning and management strategies, as input into decision making of triggers and actions under dynamic adaptive planning pathways, and for the development of coastal hazard planning provisions in the Kāpiti Coast District Plan review.

This methodology report has been externally peer reviewed by Beca and Greater Wellington Regional Council, and statements of review are attached at the rear of this report.

1.2 Definitions

Coastal Hazard:
In the context of this assessment, coastal hazards include current and future erosion and inundation due to actions of the sea. This includes the combination of sea level and waves, and how these will interact with high groundwater levels, pluvial, and fluvial flooding to the degree required under the scope of the assessment.

Coastal Hazard Susceptibility:
Susceptibility is the likelihood of the hazard event occurring, covering both frequency of occurrence (incorporating consideration of duration and intensity of the event) and uncertainty of occurrence, and the spatial extent of the area affected by the erosion or inundation within specified timeframes. Therefore, SLR is a major driver of increasing susceptibility to hazard by increasing both the frequency of occurrence and/or the spatial extent affected.

Coastal Hazard Vulnerability:
Ministry for the Environment (MfE) (2017) defines vulnerability as the "*predisposition to be adversely affected*". It encompasses a variety of concepts and elements including exposure, sensitivity or susceptibility to harm or damage from natural hazards, and degree of adaptive capacity of the natural and human system to accommodate change in the hazard exposure with minimum disruption or additional cost.

In this assessment we quantitatively assess the *vulnerability* of key council infrastructure (e.g. critical roads and three waters), community services (e.g. schools, medical centres) to the current and future exposure to coastal hazards. The evaluation of *adaptive capacity,* being consideration of the ability to accommodate and/or adapt to the hazards form part of the community engagement on adaptive planning pathways, which is Phase Two of the *Takutai Kāpiti* project.

Figure 1.1: Overview of Kāpiti Coast District coastline covered in this coastal hazard susceptibility and vulnerability assessment.

It is noted that the original Scope of Works for the coastal hazard assessment referred to a ***Risk*** assessment. Risk is commonly defined to be *likelihood x consequence,* with the consequence component of the equation including the consideration of the full range of economic, social, cultural, and environmental consequences. Risk assessments also commonly include consideration of the above consequences on strategies and actions for dealing with the impacts of the hazards. However, consideration of the full range of these consequences and possible remediation/adaptation actions is both outside the scope of this assessment, and best considered in the Phase Two (community engagement) part of the *Takutai Kāpiti* project. Therefore, we have re-defined the assessment to be coastal hazard vulnerability rather than coastal hazard risk.

1.3 Statutory Framework

There is a strong statutory requirement for KCDC to assess the susceptibility and vulnerability of the district to coastal hazards, and to make decisions on how to manage these hazards in both current and future timeframes.

1.3.1 Resource Management Act, 1991

Under Section 31 of the Resource Management Act (RMA) (1991) the functions of district councils include: *(b) "the control of any actual or potential effects of the use, development, or protection of land, including for the purpose of avoidance or mitigation of natural hazards"*. The management of significant risks from natural hazards is a matter of national importance under Section 6(h) and particular regard to be paid to the effects of climate change under Section 7(i).

1.3.2 New Zealand Coastal Policy Statement, 2010

The mandatory *New Zealand Coastal Policy Statement (NZCPS)* (Department of Conservation, 2010), prepared under the RMA includes the following objectives and policies in regard to coastal hazards, which local authorities are required to give effect to through their district and regional plans and policy statements.

Objective 5: *"to ensure that coastal hazard risks, taking account of climate change, are managed by:*

- *locating new development away from areas prone to such risks;*
- *considering responses, including managed retreat, for existing development in this situation; and*
- *protecting or restoring natural defences to coastal hazards."*

Policy 24 '*'Identify areas in the coastal environment that are potentially affected by coastal hazards (including tsunami), giving priority to the identification of areas at high risk of being affected. Hazard risks, over at least 100 years, are to be assessed having regard to:*

a) physical drivers and processes that cause coastal change including sea level rise;
b) short-term and long-term natural dynamic fluctuations of erosion and accretion;
c) geomorphological character;
d) the potential for inundation of the coastal environment, taking into account potential sources, inundation pathways and overland extent;
e) cumulative effects of sea level rise, storm surge and wave height under storm conditions;
f) influences that humans have had or are having on the coast;
g) the extent and permanence of built development; and
h) the effects of climate change on:
    i. matters (a) to (g) above;
    ii. storm frequency, intensity, and surges; and

iii. coastal sediment dynamics;

taking into account national guidance and the best available information on the likely effects of climate change on the region or district."

Policy 25: "*In areas potentially affected by coastal hazards over at least the next 100 years:*

a) avoid increasing the risk of social, environmental, and economic harm from coastal hazards;

b) avoid redevelopment, or change in land use, that would increase the risk of adverse effects from coastal hazards;

c) encourage redevelopment, or change in land use, where that would reduce the risk of adverse effects from coastal hazards, including managed retreat by relocation or removal of existing structures or their abandonment in extreme circumstances, and designing for relocatability or recoverability from hazard events;

d) encourage the location of infrastructure away from areas of hazard risk where practicable;

e) discourage hard protection structures and promote the use of alternatives to them, including natural defences; and

f) consider the potential effects of tsunami and how to avoid or mitigate them."

Policy 26: "*Natural defences against coastal hazards:*

(1) Provide where appropriate for the protection, restoration or enhancement of natural defences that protect coastal land uses, or sites of significant biodiversity, cultural or historic heritage or geological value, from coastal hazards.

(2) Recognise that such natural defences include beaches, estuaries, wetlands, intertidal areas, coastal vegetation, dunes and barrier islands."

Policy 27: *"Strategies for protecting significant existing development from coastal hazards*

(1) In areas of significant existing development likely to be affected by coastal hazards, the range of options for reducing coastal hazard risk that should be assessed includes:

(a) promoting and identifying long-term sustainable risk reduction approaches including the relocation or removal of existing development or structures at risk;

(b) identifying the consequences of potential strategic options relative to the option of 'do-nothing';

(c) recognising that hard protection structures may be the only practical means to protect existing infrastructure of national or regional importance, to sustain the potential of built physical resources to meet the reasonably foreseeable needs of future generations;

(d) recognising and considering the environmental and social costs of permitting hard protection structures to protect private property; and

(e) identifying and planning for transition mechanisms and timeframes for moving to more sustainable approaches.

(2) In evaluating options under (1):

(a) focus on approaches to risk management that reduce the need for hard protection structures and similar engineering interventions;

(b) take into account the nature of the coastal hazard risk and how it might change over at least a 100-year timeframe, including the expected effects of climate change; and

(c) evaluate the likely costs and benefits of any proposed coastal hazard risk reduction options.

(3) Where hard protection structures are considered to be necessary, ensure that the form and location of any structures are designed to minimise adverse effects on the coastal environment.

(4) Hard protection structures, where considered necessary to protect private assets, should not be located on public land if there is no significant public or environmental benefit in doing so.

While the proposed RMA reform may change the over-arching legislation governing the operation of the NZCPS, it is assumed that the above policies or similar will continue to shape the direction of coastal hazard identification and management under one or more of the proposed three replacement Acts[1].

For all of these policies, there is a need to establish the spatial extent of the land susceptible to coastal hazards and the vulnerability of the assets, infrastructure, services, and values attached to the land susceptible to the hazards. As established by Policy 24, the Identification of coastal hazards needs to take into account national guidance and the best available information on the likely effects of climate change on the region or district. This guidance on the methodologies to be used is provided in the MfE (2017) *Coastal Hazard and Climate Change: Guidance for Local Government.*

1.3.3 Wellington Regional Policy Statement, 2013

The Wellington Regional Policy Statement (RPS) (2013) sets out how the Greater Wellington Regional Council (GWRC) will manage the land and water (among other parts of the environment) of the region in a way that meets the definition of sustainability. In relation to natural hazards, the RPS includes the following objectives and policies that KCDC must give effect to in their District Plan and other planning documents.

Objective 19: *The risks and consequences to people, communities, their businesses, property and infrastructure from natural hazards and climate change effects are reduced.*

Objective 20: *Hazard mitigation measures, structural works and other activities do not increase the risk and consequences of natural hazard events.*

Objective 21: *Communities are more resilient to natural hazards, including the impacts of climate change, and people are better prepared for the consequences of natural hazard events.*

Policy 29: *Avoiding inappropriate subdivision and development in areas at high risk from natural hazards – district and regional plans.*

Policy 51: *Minimising the risks and consequences of natural hazards – consideration.*

Policy 52: *Minimising adverse effects of hazard mitigation measures – consideration.*

1.4 Relationship with Ministry for the Environment (2017) Coastal Hazards and Climate Change: Guidance for Local Government

This guidance provides a step-by-step approach to local government for assessing, planning, and managing the increasing hazard risks facing coastal communities. The guidance is structured around an iterative 10-step approach to secure and implement a long-term strategic planning and decision-making framework for the management of coastal areas already, or potentially in the future, affected by coastal hazards with climate change and SLR. This 10-step approach is shown in Figure 1.2, with the preparation of hazard assessments featuring in step 2, and vulnerability and risk assessments at step 4. As can be seen in Figure 1.2, at the core of

[1] From Press release from Minister of Environment, Hon David Parker, 10 February 2021: RMA to be repealed and replaced by Natural and Built Environment Act (NBA), Strategic Planning Act (SPA) and Climate Change Adaptation Act (CAA).

the approach is community engagement, to be involved at each step of the decision-making cycle, which is the essence of the *Takutai Kāpiti* project.

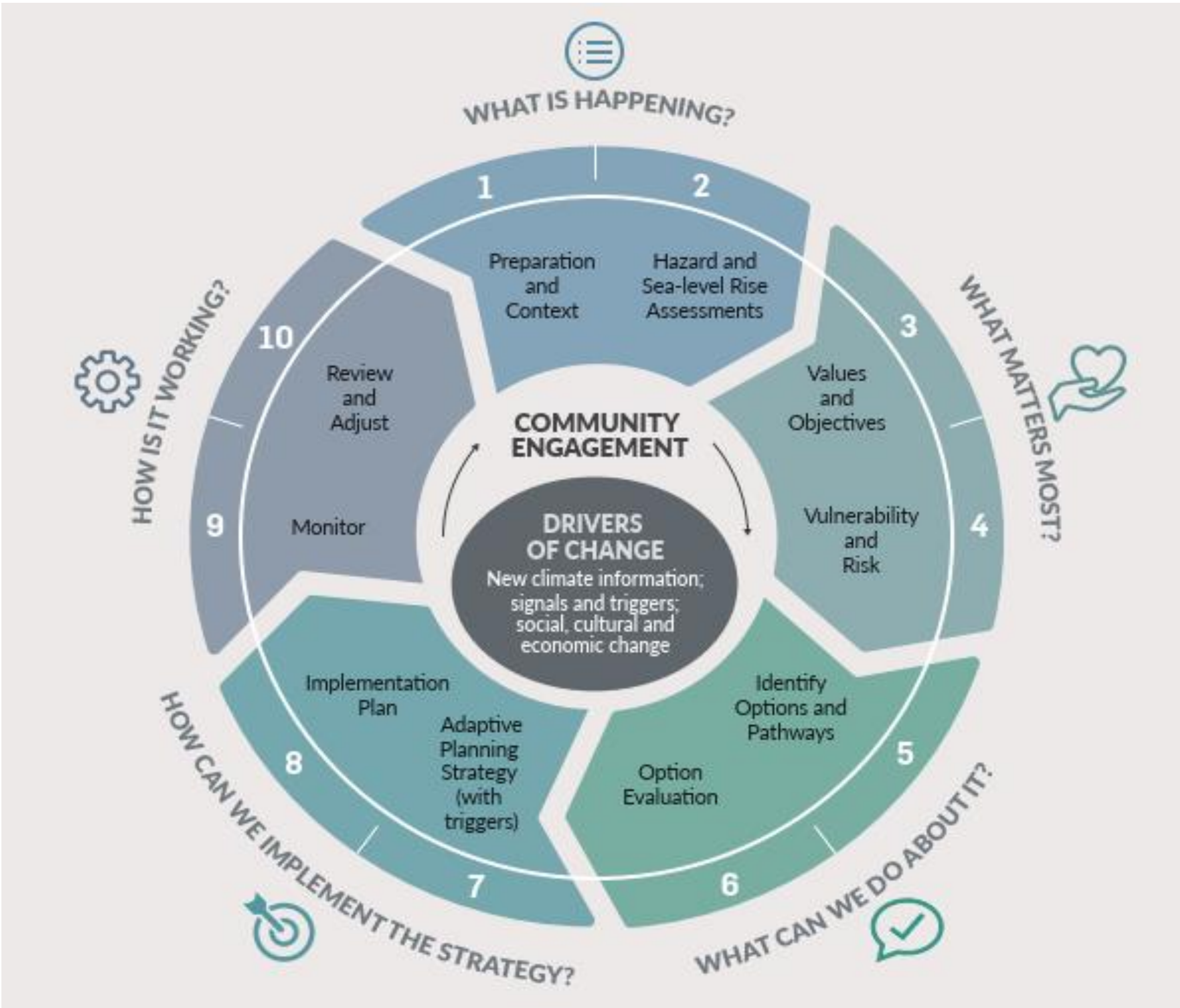

Figure 1.2: 10-step decision cycle for assessing, planning, and managing increasing hazard risks facing coastal communities. Source: MfE (2017).

The hazard identification methodologies employed in this assessment are consistent with those outlined in the guidance and are presented in Section 5 for coastal erosion hazards and Section 6 for coastal inundation hazards.

1.5 Reporting Structure

This susceptibility and vulnerability assessment is reported in two volumes:

(1) Volume 1: Methodology

(2) Volume 2: Results

The purpose of this Volume 1 report is to provide a summary of the coastal process environment and a detailed description of the methodology used to determine the coastal erosion and inundation susceptibility and vulnerability assessments. The structure of this Volume 1 report is as follows:

- **Section 2** provides background context of the coastal process environment of the Kāpiti Coast.
- **Section 3** provides information around to the projections of relative sea level rise (RSLR) on the Kāpiti Coast, with the inclusion of regional subsidence.
- **Section 4** identifies the extreme water levels at the Kāpiti Coast District shoreline used for the coastal inundation assessment.
- **Section 5** summarises the previous coastal erosion and inundation assessments which have been conducted on the Kāpiti Coast, and the methods applied.
- **Section 6** details the methods used for this coastal erosion assessment.
- **Section 7** details the methods used for this coastal inundation assessment.
- **Section 8** details the methods used for this vulnerability assessment.

2. Coastal Process Environment

This chapter provides a brief summary of the coastal geomorphology and processes operating along the Kāpiti coast which influence and shape the extent, magnitude and frequency of current coastal erosion and inundation hazards along the Kāpiti Coast District shoreline. The modelled or assumed impacts of future climate change on these coastal processes which influence the coastal hazards in this assessment are also summarised.

2.1 Geomorphologic Setting

The Kāpiti Coast District coastline comprises of approximately 38 km of sand beaches, with small 5 km long micro-morphology of composite sand and gravel beach type from the Ōtaki River mouth south to Te Horo. The majority of the shoreline is composed of fine to medium grained sand, with low sloped profiles (Lumsden, 2003) backed by large coastal plains (2-4 km wide) of sand dunes built up over the last 6000-6500 years (Gibb, 1978). These plains narrow from north to south, and are only 100-200m wide at Paekākāriki. The coastal plains were constructed by sediment transport drift material supplied from major river systems to the north (Whanganui, Whangaehu, Rangitikei and Manawatu), with some sources believed to be as far away as Cape Egmont (Fleming 1965, 1972). The mineralogy of the beaches suggest that 5-10% of sediment is derived from volcanic sources (Taranaki and Ruapehu), and 95% comes from greywacke sources (Tararua and Ruahine ranges) (Tonkin and Taylor, 2018).

The shoreline is orientated in a general north east to south west direction. However, the presence of the 9 km long Kāpiti Island located 5.5 km offshore from Paraparaumu (Figure 1.1) has a significant influence on the local wave climate, and therefore on the coastal morphology of the Kāpiti District shoreline. The island creates a wave shadow effect that reduces the wave energy of the prevailing northwest waves, and has resulted in the development of a substantial cuspate foreland at Paraparaumu from the deposition of the net southward sediment transport from the northern supply (Coddington, 1972). The presence of the wave shadow from the island and resulting cuspate foreland is one of the causes for the long-term trend of shoreline accretion north of Paraparaumu, and retreat south to Paekākāriki as the cuspate foreland acts like a natural groyne, reducing the long shore sediment supply to the south. Erosion in this southern part of the district is further enhanced by some of the sand that does pass southwards of the cuspate foreland being deflected offshore to form an offshore sand bank from Raumati and Paekākāriki as shown in the bathymetry map presented in Figure 2.1 (from Lumsden, 2003). Gibb (1978) considered that the development of the cuspate foreland has also steadily constricted the tidal flow in the Rauoterangi channel between Paraparaumu and Kāpiti Island, and also that the net longshore drift between Paekākāriki and Paraparaumu is to the north. While the first point is likely, the second point has not been confirmed and is counter intuitive as dominant wave direction is still from the north west. Possibly as a result of the consideration of a northerly transport, there is also a frequently repeated argument that erosion along the southern Kāpiti coast has been caused by the construction of SH1 in the late 1930's cutting off the supply of material from the escarpment behind the highway. However, we consider that this would have only supplied a small volume of gravel to the local shoreline at the southern limit of the district.

This study identifies ten notable breaks in the coastal dune system along the Kāpiti Coast District Coastline being the mouths of various sized streams and rivers discharging to the Tasman Sea. Two of these dune breaks form much larger coastal hydrosystem environments due to the inherent size of their water bodies flowing into the interface environment, these being (1) the Ōtaki River mouth, which is a gravel spit barrier (Hume et al, 2016) backed by a hapua lagoon, and (2) the Waikanae River mouth, which is a sand spit barrier backed by a tidal river mouth estuary. These two rivers are the main sources of coastal sediment from within the district, however their contribution to the sediment budget is much less that from the major rivers to the north of the district (see Section 2.2). The other eight dune breaks are from smaller hydrosystems where the alongshore fluctuations in mouth position are largely influenced by coastal processes and adjacent shoreline response. These hydrosystems have a mixture of natural and structured edges, which influence the dynamics of the river/stream mouth migration.

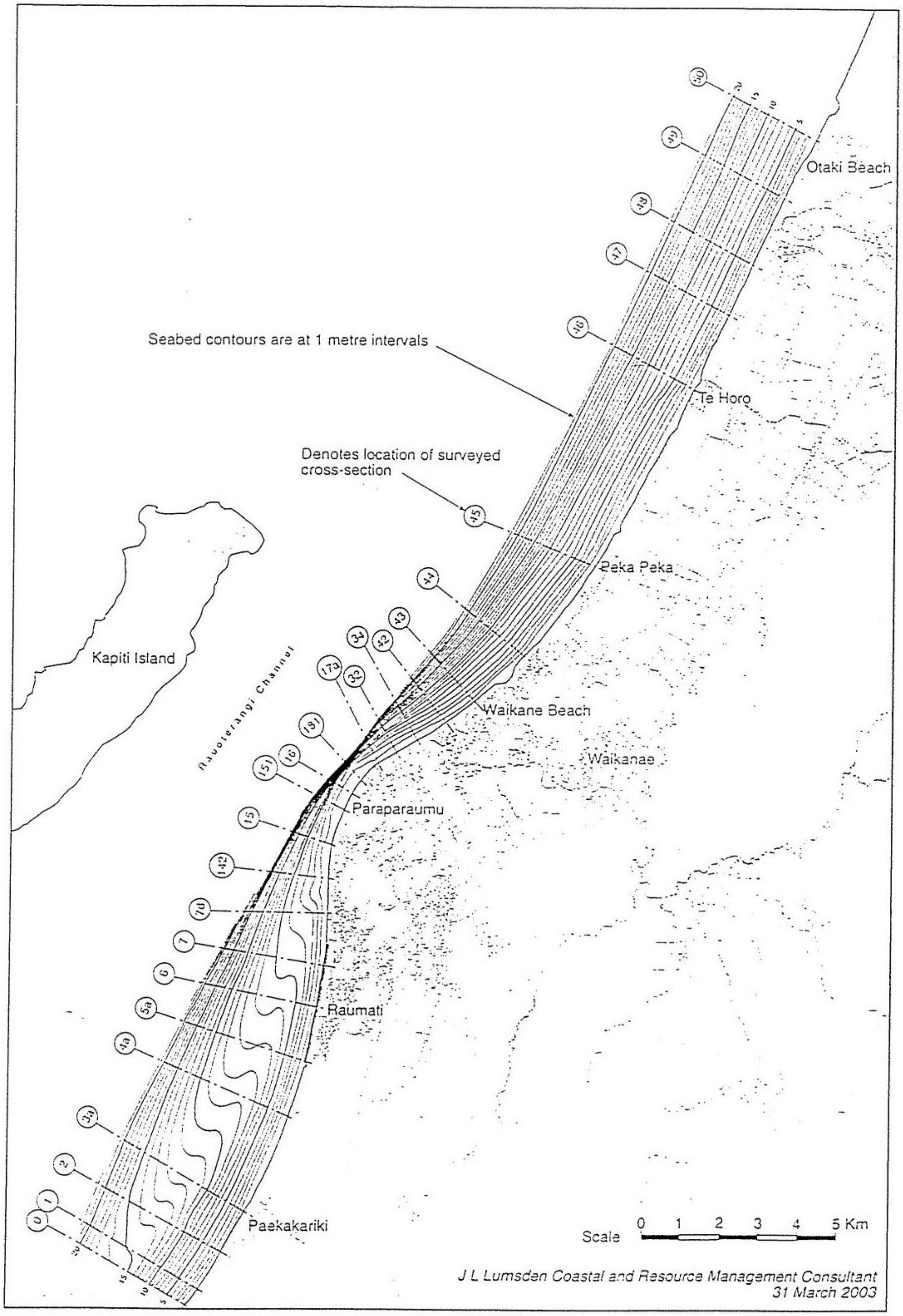

Figure 2.1: Bathymetry map of the Kāpiti Coast, data collected in 2000. (From Lumsden, 2003).

2.1.1 Shoreline movements and storms

Erosion along the southern Kāpiti Coast District shoreline from Raumati to Paekākāriki was considered by Gibb (1978) to have commenced between AD 150 and 1874, when 25% of the district coast was in net retreat, with 18-60 m of foredune retreat at Paekākāriki, and 24-37 m at Raumati. Over this same period, the remaining 75% of the Kāpiti Coast District shoreline was reported by Gibb (1978) to have advanced with 171-195 m of accretion on the cuspate foreland Paraparaumu, and 49-171 m of accretion at Waikanae. Between 1974 and 1978, Gibb noted that there was a reversal of this accretionary trend north of Paraparaumu, with only 7% of the shoreline advancing, 75% retreating, and 18% remaining static.

The long-term shoreline assessment in this study since this time showed that approximately 80% of the shore has accreted up to 2017, and 20% has eroded. The eroding sections largely occur in the southern part of the district (Paekākāriki, Queen Elizabeth Park, and Raumati), with some localised erosion measured around coastal hydrosystem environments in the northern end of the district where there has been large river mouth migration.

While the above district and regional wide coastal processes influence the long-term trends of the shoreline movement, large episodic storm events, especially those occurring close together, also play a role in the shoreline movements over shorter term periods. This was noted by Gibb (1978) as the reversal of the long-term accretion trend between 1974-1977 was due to the very significant storm event in September 1976.

Notable storms recorded in the literature (Donnelly 1959; Gibb 1978; Lane et al, 2012; Iain Dawe, GWRC, *Pers Com,* 2021) include:

- 10-11 July 1954
- 12-13 October 1957
- 12 September 1976
- 16 January 1980
- 6 September 1994
- 7 November 1994
- 29 March 2002
- 3 October 2003
- 2 January 2006
- 23 July 2008 (Figure 2.2)
- 24 July 2016
- 1 February 2018 (Ex Tropical Cyclone Fehi)
- 21 February 2018 (Ex tropical Cyclone Gita)

The largest event reported in Lane et al (2012) assessment of joint probability of storm tide and wave height was September 1976, with close to 0.5% annual exceedance probability (AEP[2]). The storm tide level and wave characteristics in this event are given as being:

- Storm tide level: 1.63 m relative to 2019 MSL (from Lane et al, 2012).
- Deep water significant wave heights (Hs) of 6.3 m (from Bosserelle and Lane, 2019).
- Peak Spectral wave period (Tp) of 13.8 seconds (from Bosserelle and Lane, 2019).

These are the most extreme waves of the four storm events presented by Lane et al (2012), but is noted that the storm tide was considerably higher in the September 1994 event (1.96 m) with a much less energetic waves (Hs = 2.3, Tp = 9.5 seconds) . It is further noted that wave and water level records from the events in 1954 and 1957 are not reported and that the events in 2016 and 2018 post-date the Lane et al (2012) report, so it is not known how

[2] AEP: The probability expressed as a percentage that an event larger that the magnitude specified will occur in any one year. If the inverse of the Annual Recurrence Interval (ARI), so a 0.5% AEP, has an ARI of 200 years.

their waves and water levels compare to the September 1976 event. However, the Ex Tropical Cyclone Fehi event is reported to have coincided with very high astronomical tides (MHWPS tides[3]).

An example of the storm tide and wave conditions on the beach at Raumati during the July 2008 storm event is shown in Figure 2.2. Storm tide and wave heights in this event are not reported.

It is noted that the above storm list infers a 10 – 20 year interval between damaging storms or groups of storms, and that all of the storms up to 2018 occurred during El Niño conditions when stronger westerly conditions are experienced. However, the Ex Tropical Cyclone events in 2018 occurred during La Niña conditions.

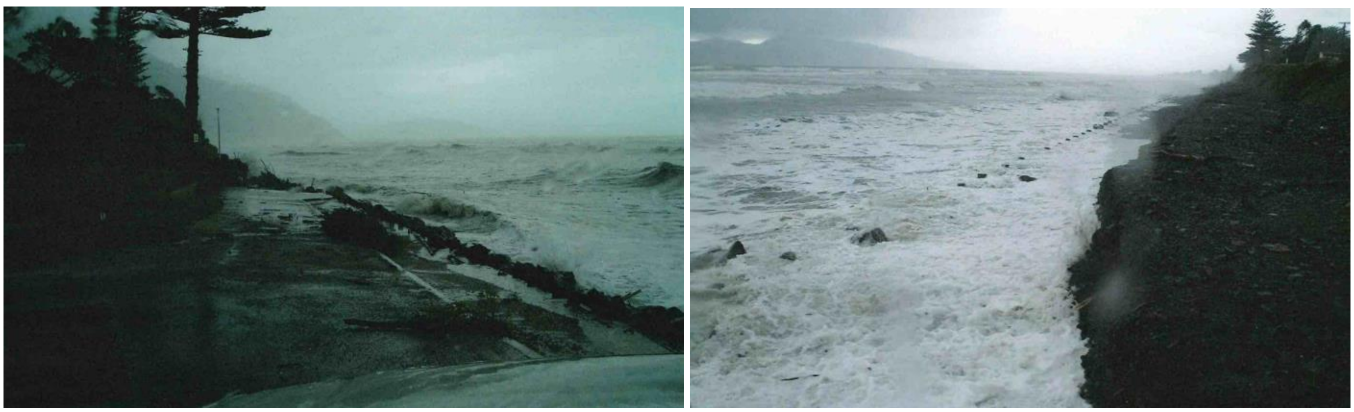

Figure 2.2: Overtopping of structures in storm event on 23 July 2008 at Tiromoana Road (left) and The Esplanade (Right) in Raumati.

2.1.2 Coastal protection structures

Coastal subdivision within the Kāpiti Coast District has been occurring since 1906. As a consequence of dwellings being placed close to the dynamic area of the shoreline, coastal protection structures have been present at Raumati and Paekākāriki since 1955 onwards. This early construction occurred where coastal erosion was becoming a major threat following large coastal storms in July 1954 and July 1957 which lead to 2-3 m of foredune erosion. Examples of the erosion experienced at Paekākāriki in the October 1957 storm are shown in Figure 2.3. As a result, the Raumati and Paekākāriki shorelines became largely protected following these events (Donnelley, 1959).

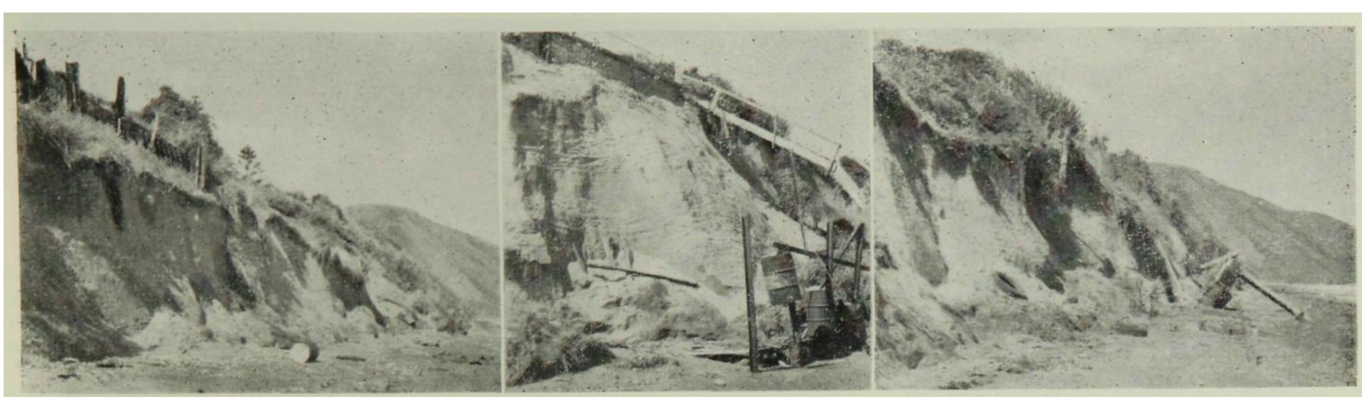

Figure 2.3: Erosion at Paekākāriki as a result of a storm event on 12-13 October 1957 (From Donnelley, 1959).

[3] MHWPS: Mean High Water Perigean Spring Tides.

The September 1976 event highlighted the consequences of protection structures failing during storm events. Whereas around 6 m of erosion occurred along sections of unprotected shoreline, however, where structures were present, beach lowering occurred allowing for waves to hit the structures at full force, destroying many and resulting in up to 15 m of foreshore erosion, as shown in Figure 2.4. Those destroyed have subsequently been rebuilt and further piecemeal private structures have continued to be constructed since this time along with a number of larger public structures being consented, such as the recently consented Paekākāriki seawall.

Figure 2.4: Coastal erosion behind failed coastal structures which fronted homes on Rosetta Road, Raumati (top), and the Esplanade, Raumati (bottom) following the September 1976 storm (Images provided by KCDC).

Today, approximately 20% of the district's shoreline is protected with an ad hoc mix of public and private structures, varying in length, type and age, which are concentrated along the Paekākāriki and Raumati shorelines. A 2016 coastal structures database compiled by Tonkin and Taylor recorded 11.2 km of shoreline structures, some of which were overlapping (e.g. a primary and secondary structure). The database records that almost 8 km of these structures are made of timber, sometimes in combination with toe riprap, rocks, concrete, and old tyres. A further 1.3 km is rock revetment and 670 m is made of concrete, with the remainder being gabion baskets, railway iron, old tyres, and vegetation. Information on the residual life of the structures contained in this database, along with information from a 2021 update for structures at Raumati, has been used in this assessment (See Section 6.1.1).

Figure 2.4: Coastal erosion behind failed coastal structures which fronted homes on Rosetta Road, Raumati (top), and the Esplanade, Raumati (bottom) following the September 1976 storm (Images provided by KCDC).

2.2 Sediment Budget

A sediment budget is a conceptual coastal management tool used to the balance between sediment added and removed from a coastal system. So it can be considered to be like a bank balance; when more material is added than removed there is a sediment *surplus* normally expressed as shoreline accretion, and when more is removed than added there is a sediment *deficit* expressed as shoreline erosion. As a conceptual tool, a sediment budget does not include consideration of the internal transfers within the coastal cell, or the processes involved in these transfers (e.g. longshore and cross-shore transport). Therefore, net long-term shoreline movement within a coastal cell is assumed to be the consequence of an imbalance between the sediment supply (known as *sources)* to the coastal cell, and the sediment losses (known as *sinks)* from the coastal cell. The calculation of a sediment budget requires being able to identify and estimate all the sediment sources and sinks within section of coastline, however, this is an extremely difficult task and as a result few sediment budgets have been accurately determined (Morton, 2003).

Based on previous work, Tonkin and Taylor (2018) produced a sediment budget for the Kāpiti Coast, as presented in Table 2.1 and visually presented in Figure 2.5. The budget quantifies the dominant sediment supply from the major rivers to the north of the district, and the local rivers within the district, however it does not provide volumes for shoreline accretion in the northern part of the district or for erosion losses from southern part.

Table 2.1: Coastal sediment budget from Tonkin and Taylor (2018).

Credits		Debits	
Item	**m3/yr**	**Item**	**m3/yr**
Longshore Drift from North	94,000 [1]	Longshore Drift to South	Unknown
Ōtaki River	11,000 [2]	Ōtaki extracted volume that might reach the coast	Unknown
Waikanae River	4000 [2]	Waikanae extracted volume that might reach the coast	60-240 [4]
Other Streams	Unknown - minimal	Extraction from streams	~0
Coastal Erosion	Unknown	Accretion	Unknown [5]
Local Inner Shelf	0-1000 [3]	Offshore Losses	Unknown

1 Griffith and Glasby (1985)
2 Brougham and McLennan (1983)
3 De Lange (2013)
4 Derived from CSL (2002)
5 Net accretion derived from Lumsden (2013) for period 2000-2011 was approximately 70,000m^3

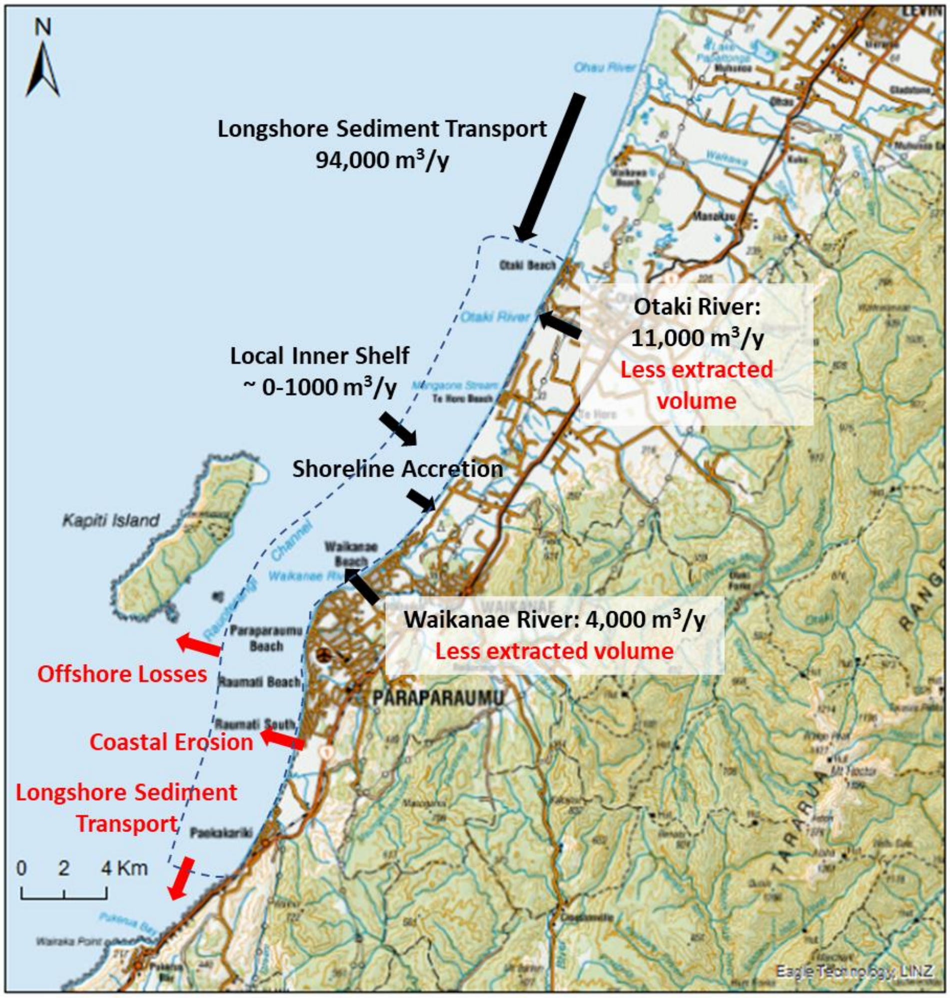

Figure 2.5: Visual representative of coastal sediment budget for Kāpiti Coast District coastline. Losses are shown in red and credits in black (Adapted from Tonkin and Taylor, 2018).

2.2.1 Impact of climate change

Climate change can potentially alter the sediment supply from rivers and longshore transport of that sediment by waves, which could have an impact on the coastal sediment budget. In discussing these potential changes, Tonkin and Taylor (2018) report made the following observations:

- *The potential for increased frequency and intensity of storms may cause an increase in supply of sediment to the rivers from their headwaters. While there may be some increase in the rivers ability to transport sediment, the current trend of riverbed aggradation is likely to continue as long as the rivers remain confined to their managed corridors, and extraction will need to continue to maintain flood conveyance at current thresholds. ……De Lange (2013) notes that based on evidence reviewed in his work, climate change is not a direct driver of sediment supply for the Kāpiti Coast.*

- *The sediment transport study Opus (2012) commented that the existing pattern of variability in flow in the Waikanae River contains greater variation from year to year than the overall shift predicted from climate* *change. Variability in stream flow and energy are therefore very much part of the normal, existing environment.*

- *De Lange (2013) notes that the development of the Kāpiti coastal plain occurred during a period of fluctuating sea levels, including intervals with higher sea levels than at present. He concludes that there is no clear relationship between regional sea level variations and the shoreline response along the Kāpiti Coast, and a trend of accretion has occurred regardless of whether sea level rose or fell.*

  However, it is considered that this statement needs to be qualified for current conditions as De Lange's assessment is based on geologic timescales. In the post-glacial, early Holocene period, there was a huge amount of sediment that was released out of the catchments and material was being swept up off the nearshore continental shelf and being bought onshore as sea levels rose. However, once the catchments and sea level stabilised there was much less sediment moving around in the system. In the present day, we now have a shoreline that has a large natural cuspate on it, has hugely modified dune systems full of stabilising marram, and many heavily developed areas are protected by engineered structures - truncating and fixing the shoreline in place and preventing natural shoreline response. A rise in sea level by the projected rates and magnitudes will have an influence on these processes; in potential sediment supply and transport, in the nearshore -beach - dune profiles on un-protected shoreline, and in effectiveness of existing structures on protected shorelines.

- *A change in the magnitude and direction of wave energy reaching the coast could have a significant impact on the direction of littoral drift and the sediment budget of the Kāpiti Coast.* However, as pointed out in Section 2.4.1, recent research presented in Bryan and Coco (2020) suggest that there is unlikely to be an increase in wave energy for the west coast of the Wellington region. A MetService analysis of wind records for Wellington over the past 50 years indicate that the wind, particularly from the North West, has become less strong, which may lead to lower wave energy and reduced longshore sediment transport (Iain Dawe GWRC, *pers com*, 2021).

As a result of these observations, it is concluded that any impact of climate change on the coastal sediment budget may be positive with increased supply, but may also be negative in terms of longshore transport. Therefore, due to the uncertainty, they are assumed to be neutral for the context of this assessment. However, on-going long-term monitoring of catchment sediment supply and the drivers of longshore sediment transport (e.g. waves) should be initiated to ensure the early detection of any changes in these processes that govern long-term shoreline movements.

2.3 Storm Tides

2.3.1 Background

Storm tides are extreme high sea levels at shore resulting from the combination of the following parameters:

- **Astronomical high tides:** Tides are caused by the gravitational pull of the sun and the moon on the earth, so can be predicted with great accuracy from the orbits of the planet and the rotation of the earth. The highest tides within a month, known as a perigean spring tide, occur when the moon is either new or full and closest to the earth, with the largest annual extreme tides are known as king tides. The largest tide predicted to occur is known as the Highest Astronomical Tide (HAT), which occurs once every 18.6 years over a lunar cycle.

  Tide heights vary from place to place, with the tidal range around the Greater Wellington region being relatively small compared to much of the New Zealand coastline. Along the west coast of the region Stephens et al (2011) modelled the height of the highest tide (e.g. HAT) to increase from around 0.6 m above mean sea level

(MSL) to the south of the Kāpiti Coast District to around 1.1 m above MSL at Waikanae. The high tide levels at Kāpiti Island calculated by MetOcean (2007) from the tidal constituents are presented in Table 2.2.

Table 2.2: High tide levels at Kāpiti Island (from MetOcean, 2007)

High tide Parameter	Acronym	Level above MSL (m)	% of high tides above
Highest Astronomical Tide	HAT	1.102	0.0
Mean High Water Perigean Springs	MHWPS	0.897	7.9
Mean High Water Springs	MHWS	0.813	19.7
Mean High Water	MHW	0.550	59.8

- **Mean Sea Level Anomaly (MSLA):** Is the non-tidal variation in sea level at scales from monthly to decades due to climate variability, including seasonal effects and the effects of El Niño–Southern Oscillation (ENSO) and the Interdecadal Pacific Oscillation (IPO) on sea level through changes or climate-regime shifts in wind patterns and sea temperatures.
- **Storm surge:** Is the short-term rise in sea level due to meteorological conditions in storm events. There are two components to storm surge:
    i. Barometric lift, being the rise in sea level when low-atmospheric pressure relaxes the pressure on the ocean surface causing a short term lift in MSL. The standard relationship is expressed as 1 cm of rise for every mb of pressure below 1013 mb.
    ii. Wind stress on the ocean surface pushing water down-wind or to the left of alongshore wind (for southern hemisphere) from a persistent wind field to pile up against any adjacent coast. The magnitude of wind stress is dependent on the wind speed, with gusty winds producing larger stresses than steady winds of the same average speed.

From 9 years of water level record from Kāpiti Island (1997-2006), MetOcean Solutions (2007) measured maximum storm surge to be 0.373 m (e.g. above predicted tidal level), of which barometric lift contributed around 80%. Using the general purpose finite element hydrodynamic modelling, Stephens et al (2011) calculated maximum storm surge for the mainland Kāpiti coast to be in the order of 0.45 m, with a slight decrease from north to south.

2.3.2 Kāpiti Coast Extreme Storm Tides

Extreme storm tides along the Kāpiti Coast occur when large tides coincide with moderate to large storm surge, with the resulting sea level being strongly influenced by the high tide level. The extreme levels are predicted to increase from south to north along the coastline. The modelled AEP results for the Kāpiti coast from Stephens et al (2011) for the joint probability of tides and storm surge are presented in Table 2.3. Note that the levels presented in Table 2.3 are relative to Wellington Vertical Datum 1953 (WVD53)[4] in 2005-2011. It is noted that when converted to this datum, a similar range of results was obtained by MetOcean Solutions (2007) using the 9 years of data from Kāpiti Island.

[4] Wellington Vertical Datum 1953 (WVD53). This is the local Vertical Datum for land surveys established in 1953, in which zero elevation was set at MSL at the time. However, with sea level rise (SLR) since that time, the elevation of MSL for the 2005 -2011 period was established to be 0.196 m WVD53 and increased to 0.25 m WVD53 for the 2012-2018 period (Bosserelle and Lane, 2019).

Table 2.3: Modelled extreme storm tides for various annual exceedance probabilities from Stephens et al (2011). Levels are relative to WVD53 2005-2011 base level.

AEP (Annual Exceedance Probability)	ARI (Average Recurrence Interval) in years	Modelled storm tide in Paekākāriki (m – WVD53 2005-2011)	Modelled storm tide in Waikanae (m – WVD53 2005-2011)
63%	1	1.29	1.47
39%	2	1.32	1.50
18%	5	1.35	1.53
10%	10	1.36	1.55
5%	20	1.38	1.57
2%	50	1.41	1.59
1%	100	1.42	1.61
Note: WVD53 2005-2011 base is 0.196 m (e.g. MSL 2005-2011 = 0.196 m WVD53). To be relative to current MSL, need to add an additional 0.054 m to the elevations.			

At the shoreline these extreme storm tide levels are also influenced by wave set up discussed in Section 2.5. The resulting predicted extreme sea levels at shore used in this coastal hazard assessment are presented in Section 4.

2.3.3 Effect of Climate Change

Possible future changes in storm surge and wave climate around New Zealand were investigated by NIWA as part of the Wave and Storm-surge Projections project (WASP). For storm surge, the 99th percentile peaks were calculated for different climate change scenarios and compared to the 30-year hindcasts from 1970 to 2000. The results were that changes in storm surge heights are relatively modest or inclusive around New Zealand, with the south Taranaki Bight being one of the two places to show consistent significant change in storm surge height over all emission scenarios. These changes ranged from +5 to +10%, or to up 0.05 m increase in storm surge height (MfE, 2017).

Given the small magnitude of the above predicted change in storm surge elevation, no change in the storm tide levels other than to account for relative sea level rise (RSLR) are applied in this assessment. The future projections of RSLR are set out in Section 3, and the effect of these on the extreme sea levels are included in Section 4.

2.4 Wave Climate

The wave climate of the Kāpiti coast is dominated by waves from a general North West direction and is influenced by the presence of Kāpiti Island. Locations north of the Island generally receive more energetic waves than to the south of the Island, with a wave shadow effect reducing wave heights directly behind the Island and altering storm wave approach to more westerly in the lee of the southern half of the island (e.g. Paraparaumu and Raumati).

The wave climate for the Kāpiti coast has been modelled by MetOcean Ocean Solutions (2007)[5] who present output at 16 locations from Ōtaki to Paekākāriki from 9 years of hindcast from 1997 to 2006, and by Stephens et al (2011)[6] who present output for Waikanae and Paekākāriki from 45 years of hindcast from 1957 to 2002. The earlier modelling by Met Ocean was validated with wave records from the Kupe Gas Field and Maui Oil field in the Taranaki Bight, which showed good correlation between the measured and modelled wave parameters. However, the latter

[5] Used SWAN ocean wave progradation model to bring to nearshore wave parameters generated from NOAA WAVEWATCH III deep water wave generation site.
[6] Same modelling software as used by MetOcean Solutions (2007).

modelling by NIWA (Stephens et al, 2011) calibrated against seven available wave buoys found an under-prediction of wave heights to the Bearing Head (east of Wellington Harbour) buoy measurements in severe storm events, resulting in a factor of 1.5 being applied to modelled heights for the largest 5% of hindcast waves. Stephens et al (2011) notes that *"this adds a degree of conservatism to the extreme wave analysis, but is also reasonable because of the 10-year Baring Head buoy record is barely sufficient for accurate prediction of 1% AEP* (e.g. 1% chance of occurring in any given year, or Average Recurrence Interval (ARI) of 100 years) *wave heights"*.

A summary of extreme wave results presented in both studies are presented in Appendix A, and the 1-year (e.g. 63% AEP) and 100-year return period (e.g. 1 % AEP) significant wave heights are presented in Figure 2.6. These results show that the modelled wave heights from Stephens et al (2011) from the longer hindcast modelling are considerably higher than those obtained by MetOcean Solutions (2007), highlighting the conservative approach mentioned above. Stephens et al (2011) further noted that although "*the scaling factor of 1.5 produces a good match to the buoy data at high AEP's (63-10%), it increasing over predicts the buoy data at low AEP's , being 0.7 m higher at 1% AEP*. The report also importantly notes that there was a closer agreement between the hindcast and measured wave heights at the Maui-A platform in the Taranaki Bight, further suggesting that the degree of conservatism with a 1.5 scaling factor is higher for the recorded wave heights at the Kāpiti coast sites.

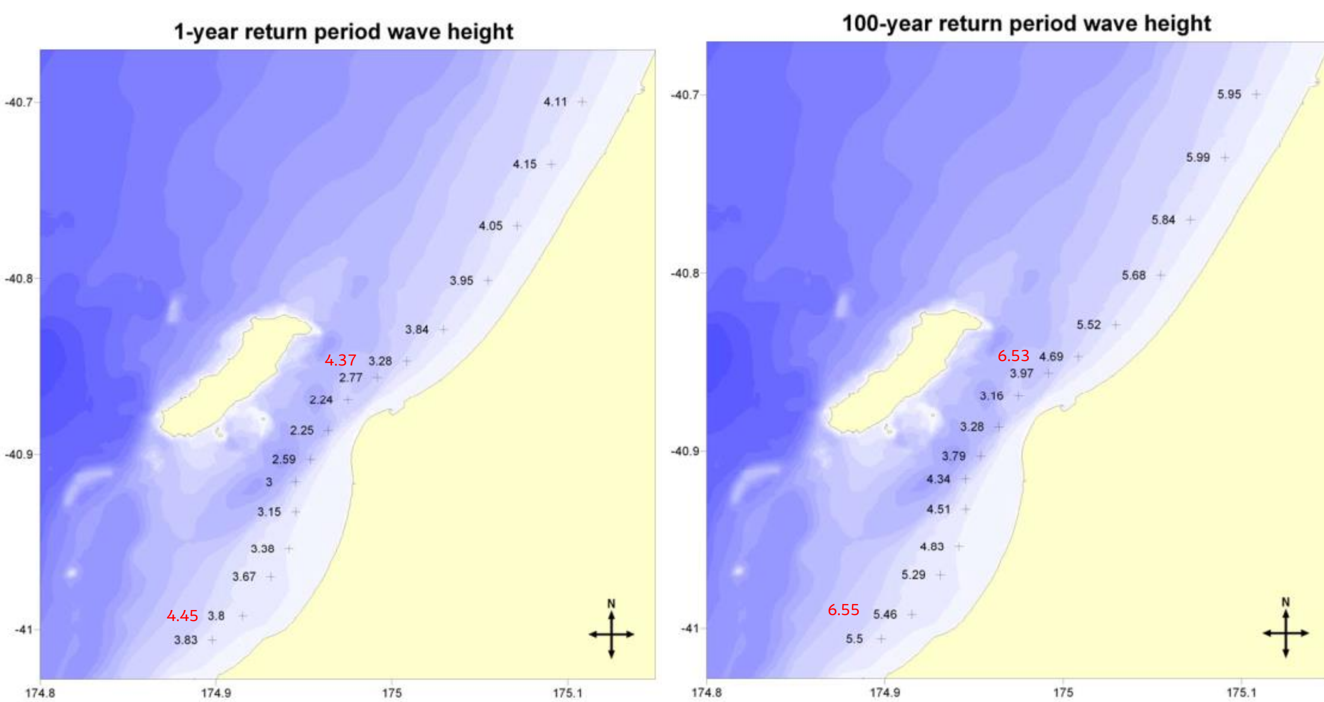

Figure 2.6: 1-year and 100-year return period significant wave heights locations (latitude and longitude) from MetOcean Solutions (2007) (Black values) and Stephens et al (2011) (Red values).

2.4.1 **Impact of Climate Change**

As with storm surge, the impact of climate change on the New Zealand wave climate was investigated by NIWA as part of the WASP project. As reported in MfE (2017), the results indicated that in general increases of 0-5% in the 99th percentile of significant wave height would apply around New Zealand, with the largest increases on the western and southern coasts exposed to Southern Ocean swell waves.

More recent work presented in Bryan and Coco (2020) indicated that that there is evidence that in general the New Zealand is becoming more energetic with the number of extreme storm events, number of storm clusters, and duration of clusters all increasing. However, the maps presented in that paper suggest that is in not the case for the south Taranaki Bight and the west coast of the Wellington region, with the data showing a decrease in the number of storm events for this area.

Based on these results, no change in future wave heights with climate change are assumed in this assessment.

2.5 Joint Probability of Storm Tide and Wave Height

Stephens et al (2011) found that there is a clear positive relationship between storm tide and wave height on the west coast of the Wellington region due to:

i. The great majority of weather systems approach the Wellington region from the west. As the low-pressure systems and fonts approach, they generate wind waves and swell that propagate towards the west coast.

ii. The semi-enclosed nature of the South Taranaki Bight and its exposure to weather systems approaching from the west makes it the primary generation zone for storm surge around the Wellington region; hence the same weather systems generate waves and storm surge.

iii. Small tidal amplitude means that the correlation between storm tide and waves are not masked by large tides.

As a result, there is a high probability that high storm tides will coincide with large storm waves on the Kāpiti coast, hence increasing the frequency of events potentially causing erosion and inundation.

Under the joint probability, a specified AEP or ARI event could result from a number of different combinations of storm tides and wave heights, with the probability of the individual components occurring being considerably less that the joint probability of them occurring together. For example, the results from Stephens et al (2011) indicate that for both Paekākāriki and Waikanae, a 100-year joint probability event could occur from combination of a 10 year ARI storm tide with a wave height likely to occur monthly, or a 10 year wave height combined with a monthly return period storm tide.

Lane et al (2012) applied the joint probabilities from Stephens et al (2011) to examine the relationship of storm tide to wave height for four past storm events (12 September 1976, 16 January 1980, 6 September 1994, 7 November 1994) assessed as having close to a joint 1% AEP along the Kāpiti coast. The joint probability plot for these storm events are shown in Figure 2.7, which as indicated in Section 2.1.1 shows that the largest joint probability event was September 1976, with a joint probability close to 0.5% AEP, as result of the combination of a moderate storm tide and large wave heights (Hs = 6.3 m, Tp = 13.8 sec[7]). In comparison the storm event of September 1994 was a combination of a large storm tide (driven by very high MHWPS tide plus surge) and a small wave height (Hs = 2.3 m, Tp = 9.5 sec), and the November 1994 event was driven by a small storm tide and a large wave height (Hs = 5.6 m, Tp = 10.4 sec).

[7] Storm wave parameters given in Bosserelle and Lane (2019). As per Stephens et al (2011) the Hs heights are scaled up 1.5x from simulated by the WASP models.

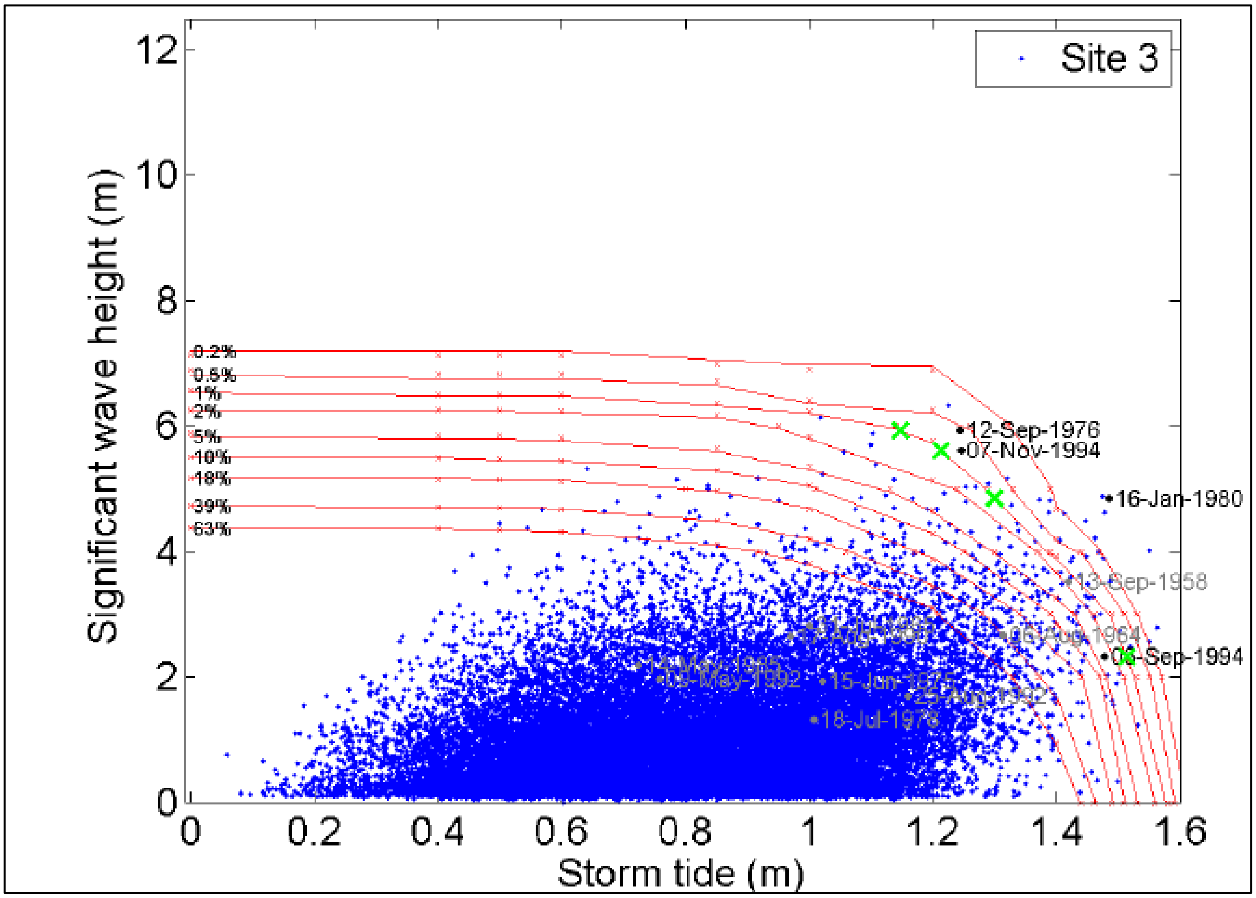

Figure 2.7: Joint probability of storm-tide and significant wave height for four selected storm events (black dots) offshore from Waikanae. (source Lane et al 2012). Note the green crosses green crosses mark the intersection of the selected events with the 1%AEP contour following "sliding" of the storm-tide to fit the contour. Significant wave heights are scaled by x1.5m from those simulated by the WASP model (as per Stephens et al 2011).

2.6 Wave Set-up and Run-up

Wind-generated waves also raise the effective sea level at the shoreline by the following two processes.

i. Wave set-up, being the temporary increase in mean still water level at the coast that results from the release of wave energy in the surf zone as waves break. Wave setup is an integral component of the total water level that potentially could cause direct or near-continuous inundation onto coastal land.

ii. Wave run-up is the maximum vertical extent of wave "up-rush" on a beach or structure above the instantaneous still-water or storm-tide level, and thus constitutes only a short-term fluctuation in water level relative to wave setup, tidal and storm-surge time scales. However, the combined storm-tide plus wave run-up level is relevant for coastal inundation from overtopping of dunes and structures, to erosion of beaches and dunes, and to the wave impact on seawalls and coastal structures.

As well as wave height and period, wave set-up and run-up elevations are strongly dependant on beach foreshore slope. Therefore, beach profile variability is an important parameter on determining extreme water levels along the Kāpiti coast.

Lane et al (2012) applied the wave hindcasts from Stephens et al (2011) and estimated beach slopes to calculate wave set up and run-up[8] at six open coast and two river mouth sites along the Kāpiti coast for the four past storm events referred to in Section 2.5 (Sept 1976, 16 Jan 1980, 6 Sept 1994, 7 Nov 1994). The resulting range of calculated maximum wave set-up and run-up (excluding set-up) heights for these events is presented in Table 2.4. The set-up and run-up heights presented in Table 2.4 are above the storm tide level at the time of the maximum wave conditions. Therefore that level needs to be added to the set-up heights to obtain the extreme sea level at shore, and these extreme water levels added to the run-up height to get maximum run-up elevations required for the erosion and inundation hazard exposure assessments. These total extreme water levels are presented in Section 4.

Table 2.4: Calculated range of maximum wave set-up and run-up heights for storms assessed as have close to 1% joint storm tide/wave height AEP. From Lane et al (2012). Note that run-up heights above set-up (e.g. are above the extreme water levels presented in Section 4).

Location	Beach slope	Range of Set-up heights (m)	Event with Max Set-up	Range of Run-up heights (m)	Event with Max Run-up
Ōtaki Beach	0.023	0.40 – 0.75	12-Sep-76	0.44 – 1.09	12-Sep-76
Ōtaki River	0.038	0.42 – 0.73	12-Sep-76	0.59 -1.14	12-Sep-76
Te Horo	0.384(1)	0.38 – 0.74(1)	12-Sep-76	2.05 – 5.23(2)	12-Sep-76
Waikanae	0.060	0.42 – 0.81	12-Sep-76	0.54 – 1.35	12-Sep-76
Waikanae River	0.043	0.10 – 0.18	7-Nov-1994	0.07 – 0.11	7-Nov-94
Paraparaumu	0.411(1)	0.08 – 0.16(1)	7-Nov-1994	0.30 – 0.49(1)	7-Nov-94
Raumati	0.100	0.14 -0.31	12-Sep-76	0.28 – 0.65	12-Sep-76
Paekākāriki	0.030	0.37 – 0.71	12-Sep-76	0.51 – 1.09	12-Sep-76

Notes:

(1) Although calculated with beach slopes outside the criteria given by Stockdon et al (2006) for the calculations and much steeper than present in multiple profile surveys 2000-2018, and, the results appear to be in the right of magnitude relative to the adjacent sites and subsequent modelling in Bosserelle and Lane (2019).

(2) Calculated with beach slopes outside the criteria given by Stockdon et al (2006) for the calculations and much steeper than present in multiple profile surveys 2000-2018. Results are rejected as being out of the expected range based on the results from the adjacent sites. Using range of surveyed beach slopes, maximum run-up (above set-up) at Te Horo from September 1976 storm calculated to be 1.85 m to 2.77 m.

It is noted in Lane et al (2012) that the beach slopes at Te Horo and Paraparaumu are much steeper (1:2.5) than the slope criteria for the calculation formula, hence there is uncertainty in the set-up and run-up values for these sites, with them considered to be a potential significant over-estimate, particularly for run-up at Te Horo. For this current analysis we have re-examined the beach slopes given by Lane et al (2012) for all sites compared with those obtained by repeated beach surveys since 2000. We found that the relative foreshore slopes (0 m to 2 m contour) surveyed at Te Horo and Paraparaumu were much flatter than those given by Lane et al (2012), and were within the slope criteria required (e.g. mean slopes over all surveys of 1:13 for Te Horo and 1:45 for Paraparaumu). For all other sites, the beach slopes given in Lane et al (2012) were within the surveyed range, and therefore considered to be an appropriate representation for the calculation of set-up and run-up.

As part of updated storm surge inundation modelling for the Ōtaki Beach to Peka Peka area, Bosserelle and Lane (2019) recalculated the wave set-up using the process-based model XBGPU, which explicitly simulates the processes that cause wave set-up as part of the hydrodynamic modelling. The resulting wave set-up elevations were

[8] Set-up and Run-up calculated using formula from Stockdon et al (2006). Calculations relevant for beach slope range of 1:9 to 1:100.

similar to the bathtub type approach used in Lane et al (2012), giving confidence in the set-up values for Te Horo given in Table 2.4. For set-up at Paraparaumu, the values presented in Table 2.4 appear to be low, but accepted based on adjacent sites and consideration of the maximum reduction in wave climate due to the wave shadow effect of Kāpiti Island. The run-up values for Paraparaumu from Table 2.4 are similarly accepted for the same reasons.

Unfortunately, Bosserelle and Lane (2019) do not present updated wave run-up calculations for Te Horo. However, by inputting wave height and period parameters for the September 1976 event into the Stockdon et al (2006) run-up equation with the range of surveyed profile slopes at Te Horo (1:7.5 to 1:16.5), gives a range of maximum run-up heights (above set -up) from 1.85 m to 2.77 m. These are considered to be more acceptable run-up levels than those from Lane et al (2012) presented in Table 2.4. For Peka Peka, (not presented in Table 2.4) the flatter surveyed beach slopes (1: 16.5 to 1:33) result in a range of maximum run-up heights (above set-up) of 1.60 m to 1.85 m.

Table 2.4: Calculated range of maximum wave set-up and run-up heights for storms assessed as have close to 1% joint storm tide/wave height AEP. From Lane et al (2012). Note that run-up heights above set-up (e.g. are above the extreme water levels presented in Section 4).

Location	Beach slope	Range of Set-up heights (m)	Event with Max Set-up	Range of Run-up heights (m)	Event with Max Run-up
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Paraparaumu	0.411(1)	0.08 – 0.16(1)	7-Nov-1994	0.30 – 0.49(1)	7-Nov-94
Raumati	0.100	0.14 -0.31	12-Sep-76	0.28 – 0.65	12-Sep-76
Paekākāriki	0.030	0.37 – 0.71	12-Sep-76	0.51 – 1.09	12-Sep-76

Notes:
(1) Although calculated with beach slopes outside the criteria given by Stockdon et al (2006) for the calculations and much steeper than present in multiple profile surveys 2000-2018, and, the results appear to be in the right of magnitude relative to the adjacent sites and subsequent modelling in Bosserelle and Lane (2019).
(2) Calculated with beach slopes outside the criteria given by Stockdon et al (2006) for the calculations and much steeper than present in multiple profile surveys 2000-2018. Results are rejected as being out of the expected range based on the results from the adjacent sites. Using range of surveyed beach slopes, maximum run-up (above set-up) at Te Horo from September 1976 storm calculated to be 1.85 m to 2.77 m.

In general, the results reflect the wave climate, with the set-up and run-up elevations to the south of Kāpiti Island being less than to the north, and the wave shadow effect at Raumati producing the lowest elevations.

2.6.1 Impact of Climate Change

Climate change could affect wave set-up and run-up in the following three ways:

i. By producing a more energetic wave climate. As outlined above, there is no evidence to date to suggest that any significant changes will occur in the wave climate of the Kāpiti coast.

ii. By larger waves breaking closer to shore due to greater water depth, therefore producing greater set-up and run-up heights. However, under the assumption of conservation of an equilibrium profile shape with SLR, the erosion from the beach will be deposited on the nearshore to raise the seabed by the same vertical magnitude as the magnitude of SLR. Therefore, future wave set-up and run-up heights relative to sea level should remain similar to current day but will reach higher elevations relative to land level due to SLR. Along structured sections of the coastline, the presence of set-up/ run-up occurring at higher wave levels will interact with the structures more frequently, likely resulting in more frequent beach scour in front of the structures and overtopping of low structures with corresponding increase the likelihood of backscour, therefore increasing the likelihood of subsequent failure of structures.

iii. By reduced nearshore and beach slope, resulting in greater set-up and run-up. As above, the SLR response models assume conservation of profile shape, and since slope is dependent on sediment size and wave/current energy which is considered to not change, it is therefore assumed that nearshore and beach slope will remain similar to current conditions.

Based on these factors, it is considered appropriate to apply similar wave set-up and run-up heights as present to future scenarios but recognising that they will reach higher elevations relative to land level due to SLR. The magnitudes of future RSLR are discussed in Section 3, and the resulting total extreme sea levels at shore are presented in Section 4.

3. Projections of Relative Sea Level Rise

Relative sea level rise (RSLR) is the combination of sea level rise (SLR) from global climate change (referred to as absolute rise), and local vertical land movements (VLM). The Wellington region is situated astride a complex network of crustal faults associated with the subduction of the Pacific Plate under the Australian Plate, some 20–40 km beneath the surface of the lower North Island. As such, the region has a more complicated spatial and temporal pattern of long-term RSLR than other stable parts of New Zealand (Bell et al, 2018).

The future rate of RSLR will play an important role in the extent, frequency and magnitude of future coastal erosion, inundation, and high groundwater. Therefore, the following projections of future RSLR are relevant to the assessment of all three hazards, and are presented here together, rather than within the methodology of the individual hazards.

3.1 Historical Rates of Relative Sea Level Rise

From long-term tidal records at Wellington Harbour, the average historic rate of RSLR from 1900 to 2017 was 2.28 ± 0.15 mm/yr (Bell et al, 2018), which is greater than the global average rate of rise of 1.7 ± 0.2 mm/yr between 1901 and 2010 (IPCC, 2014). Bell et al (2018)) also reported that the rate of RSLR at Wellington increased from an average of 0.72 mm/yr for the 62 years up to 1960 to an average of 2.74 ± 0.20 mm/yr for the period since 1961 to 2017. The contribution from climate change over the more recent 55 year period was estimated as being 2.22 ± 0.19 mm/yr (based on assumption that tectonic subsidence was occurring at a rate of 1.8 mm/yr since 1998).

This rate of RSLR implies that the sea level in the Wellington region (including the Kāpiti coast) has risen in the order of 0.16 m ± 0.1 since 1961 to 2020. This magnitude of rise is greater than experienced at the other main New Zealand ports with long-term records (e.g. Auckland, Lyttleton, Dunedin) (Hannah, 2016) and the rates of rise are higher than the global rise of 2.0 ± 0.3 mm/yr in the 40-year period from 1971 to 2010 (IPCC, 2013).

3.2 Projections Due to Climate Change

The Intergovernmental Panel on Climate Change's (IPCC's) *Fifth Assessment Report* (IPCC, 2014) developed four climate change and SLR projections, termed RCPs (Representative Concentration Pathways), based on the following global greenhouse gas emissions scenarios.

- RCP2.6 – low emissions
- RCP4.5 – moderate then declining emissions
- RCP 6.0 – moderate emissions
- RCP8.5 – continuing status quo high emissions

Within each RCP, percentiles are used to quantify the distribution of the SLR projection with the median (50^{th} percentile) plotted as the main curve.

MfE (2017) presents four SLR scenarios which are based on three of the IPCC RCP scenarios (RCP2.6, RCP4.5, RCP8.5) and a higher RCP8.5+ scenario (83^{rd} percentile of RCP8.5) to take into account possible instabilities in the polar ice sheets. The resulting SLR projections from these scenarios extend out to 2150 and include a small additional SLR above the global projections to account for NZ wide regional offset in rates of historical rise.

Since their 2013 assessment, IPCC have released a *Special Report on the Ocean and Cryosphere in a Changing Climate (SROCC)* (IPCC, 2019), which published updated SLR projections from the previous AR5 report. Although a full update of these SLR projections is not expected to be published until the AR6 report is released in 2021, the

2019 IPCC Special Report indicated that the median of the RCP8.5 scenario SLR projection has increased by 0.1 m by 2100, and the upper bound has increased by 0.12 m. This increase in SLR projection by 2100 is a result of a better understanding of the contribution that the melting of the Antarctic Ice Sheet will have on SLR over the next 100 years and beyond.

The resulting adjustments to the four sea level projections from MfE (2017) for the IPCC (2019) *SROCC* updates are presented in Figure 3.1.

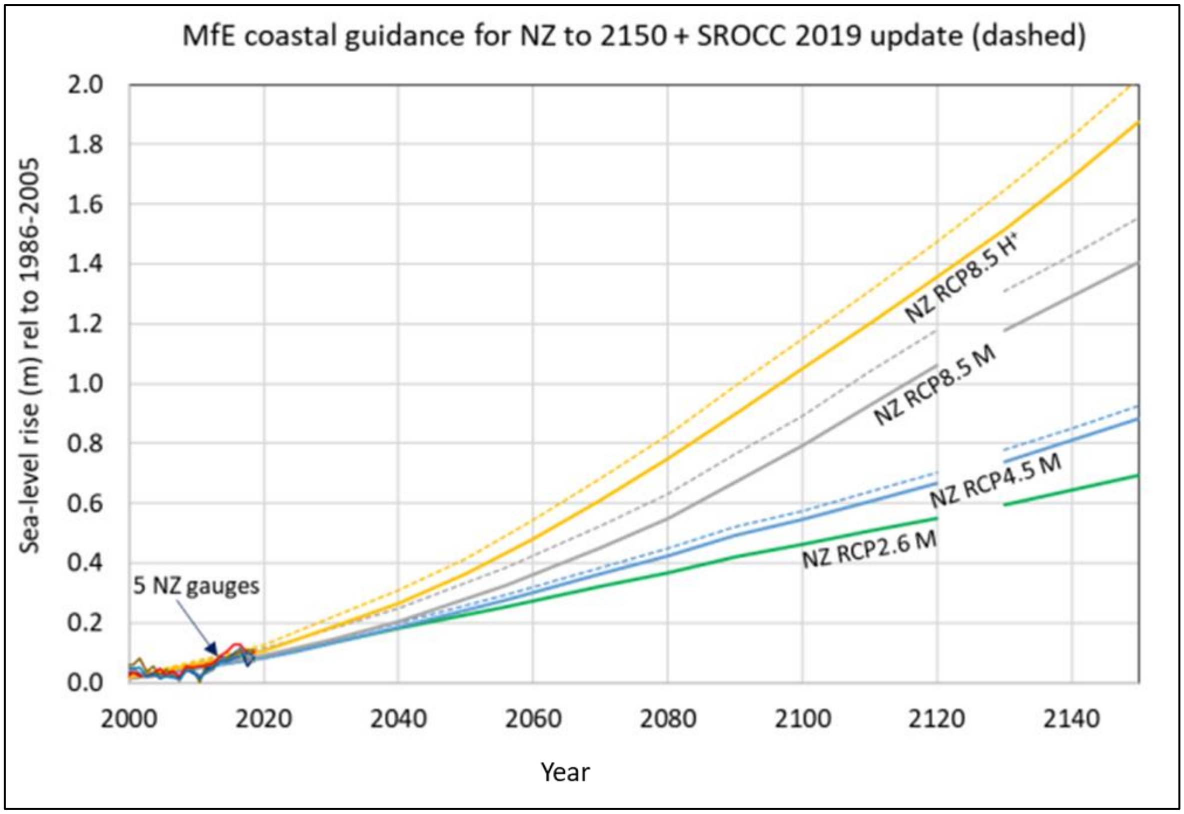

Figure 3.1: Sea level projections for New Zealand until 2150 from MfE (2017) coastal hazard guidance plus updates for IPCC (2019) SROCC (dashed lines). Source MfE (2020).

The projections show the increasing uncertainty of the SLR projections with increasing time frames, with the range of median SLR projections increasing from 0.14 m in 2050 (reasonable certainty) to 0.3 m by 2070 and to 0.8 m by 2120 (deep uncertainty).

It should also be noted that these projections are set from a base sea level in the 1986-2006 period (assumed mid-point base date of 1995). However, for our assessment we are interested in the rise from current sea levels, hence have reset the base sea level to 2020 (e.g. all rise is from zero sea level in 2020). The resulting SLR from 2020 levels due to climate change are set out in Table 3.1.

Table 3.1: New Zealand SLR projections from 2020 Adapted from MfE (2017) and including updates from IPCC (2019).

NZ SLR Scenario Year	RCP 2.6 Median (m)	RCP4.5 Median (m)	RCP 8.5 Median (m)	RCP 8.5H+ (m)
2020	0	0	0	0
2030	0.05	0.05	0.08	0.09
2040	0.10	0.12	0.14	0.18
2050	0.15	0.18	0.24	0.31
2070	0.24	0.31	0.43	0.58
2100	0.38	0.51	0.79	1.04
2120	0.47	0.63	1.07	1.37
2150	0.61	0.84	1.46	1.93

3.3 Projections of Local Vertical Land Movements

As indicated above, local vertical land movements (VLM) have had a significant influence on RSLR for the Wellington region due to subsidence from slow slip activity and vertical uplift following recent large earthquake events. However, the certainty of future projections is limited by the measurements of vertical land movement being restricted to a little over 20 years (Bell et al, 2018) and the inability to predict displacement from future earthquake events.

Using cGPS[9] gauge records, Beavan and Litchfield (2012) and Bell and Hannah (2012) reported that the lower North Island had subsided at average rates of 1-3 mm/yr over the previous 10 years, with the average rate of subsidence for the Wellington Region being 1.8 mm/yr, with the Kāpiti Coast being closer to 1 mm/yr. More recent analysis by Bell et al (2018) notes that in general the Wellington region is subsiding at rates of between 2-5 mm/yr over the last 20 years, with the Kāpiti Coast site averaging 5.15 mm/yr and Paekākāriki site averaging 3.45 mm/yr. However, this subsidence is offset by slow slip events (SSE) that periodically uplift the region by as much a 1 mm/year (averaged over 20 years). These events appear to occur every 6-8 years and can last for up to a year, with the largest events occurring in 2003, 2008, and 2013, and becoming progressively larger towards the west coast, which represents the transition from the Pacific to the Australian plates (Bell et al, 2018). In addition to these ongoing subduction zone processes, the region has been displaced by three recent large earthquake events (e.g. two from the Seddon sequence in 2013, and one from Kaikoura in November 2016) with post-seismic displacement after these events being uplift of up to 50 mm along the Kāpiti coast (Bell et al, 2018). The net effect over the last 10 years at Kāpiti coast has been that the subsidence due to the subduction of the Pacific plate under the Australian plate and coseismic displacement has been cancelled out by the Kaikoura earthquake post-seismic deformation. However, for the sites with 20 years of record, the lack of significant earthquakes in the first part of the period gives a net 20-year subsidence as rates of 1-3mm/year, with the Paekākāriki site showing net subsidence of -18 mm over the 20-year period (Bell et al, 2018).

The recent analysis from Bell et al (2018) concludes that it is difficult to provide a definitive long-term trend of VLM for any site in the Wellington region. This is largely due to the effects and ongoing influences on crustal movement of the recent earthquake events since 2013, and that the complex deformation pattern in the region is likely to remain in the future. There is no reason to expect that the regional long term trend of subsidence being driven by

[9] cGPS: Continuous GPS (Global Positioning System) records provided by USA satellite system.

the Australian-Pacific Plate subduction is going to stop, and this is therefore included in the resulting RSLR projects in Section 3.4. However, while it is possible to estimate the secular subsidence (long-term) and estimate with less certainty the SSE rate; it is not possible to estimate the displacement of future earthquake events, and therefore is difficult to incorporate into long-term projections of RSLR.

3.4 Resulting Relative Sea Level Rise Projections

From the above discussion, and bearing in mind the uncertainty with predicting a long-term rate of future VLM, a local VLM range of -1 to -3 mm/yr for on-going long-term subsidence has been added to the SLR projections from MfE (2017) plus IPCC (2019), as given in Table 3.1. The resulting range of RSLR projections for Kāpiti for each of the RCP scenarios are given in Table 3.2, with the highlighted dates of 2050 (e.g. 30 years' time from base date of 2020), 2070 (50 years' time) and 2120 (100 years' time) being the timeframes selected for this coastal erosion hazard assessment.

Table 3.2: RSLR projections from 2020 for the Kāpiti coast incorporating climate change (including updates from IPCC 2019) and local VLM of -1 to -3 mm/yr.

RSLR Scenario Year	RCP 2.6 Median (m)	RCP4.5 Median (m)	RCP 8.5 Median (m)	RCP 8.5H+
2020	0	0	0	0
2030	0.06 – 0.08	0.06 – 0.08	0.09 – 0.11	0.10 – 0.12
2040	0.12 – 0.16	0.14 – 0.18	0.16 – 0.20	0.20 – 0.24
2050	0.18 – 0.24	0.21 – 0.27	0.27 – 0.33	0.34 – 0.40
2070	0.29 -0.39	0.36 – 0.46	0.48 - 0.58	0.63 – 0.73
2100	0.46 – 0.62	0.59 – 0.75	0.87 – 1.03	1.12 – 1.28
2120	0.57 – 0.77	0.73 -0.93	1.17 - 1.37	1.47 -1.67
2150	0.74 – 1.00	0.97 – 1.23	1.59 – 1.85	2.06 -2.32

As can be seen from Table 3.2, the uncertainty in the magnitude of RSLR increases over time with both the RCP scenarios and the rate of local VLM. For example, the uncertainty range in RSLR by 2050 is 0.22 m (e.g. 0.18 – 0.44 m RSLR), increasing to 0.44 m by 2070 (e.g. 0.29 -0.73 m RSLR) and 1.10 m by 2120 (e.g. 0.57 – 1.67 m RSLR). These increasing uncertainties at each of the timeframes are shown in Figure 3.2.

Based on the range of uncertainty for each timeframe, the projections of RSLR applied in this assessment to determine the exposure of the Kāpiti coast to future coastal erosion and inundation hazards with climate change are as presented in Table 3.3. The lower projections at each time frame cover the RCP2.6 scenario with VLM of -1 mm/yr, while the upper projections cover the RCP8.5H+ scenario with VLM of -3 mm/yr. Due to the large uncertainty of the 2120 RSLR projections, two intermediate projections are also included in the assessment.

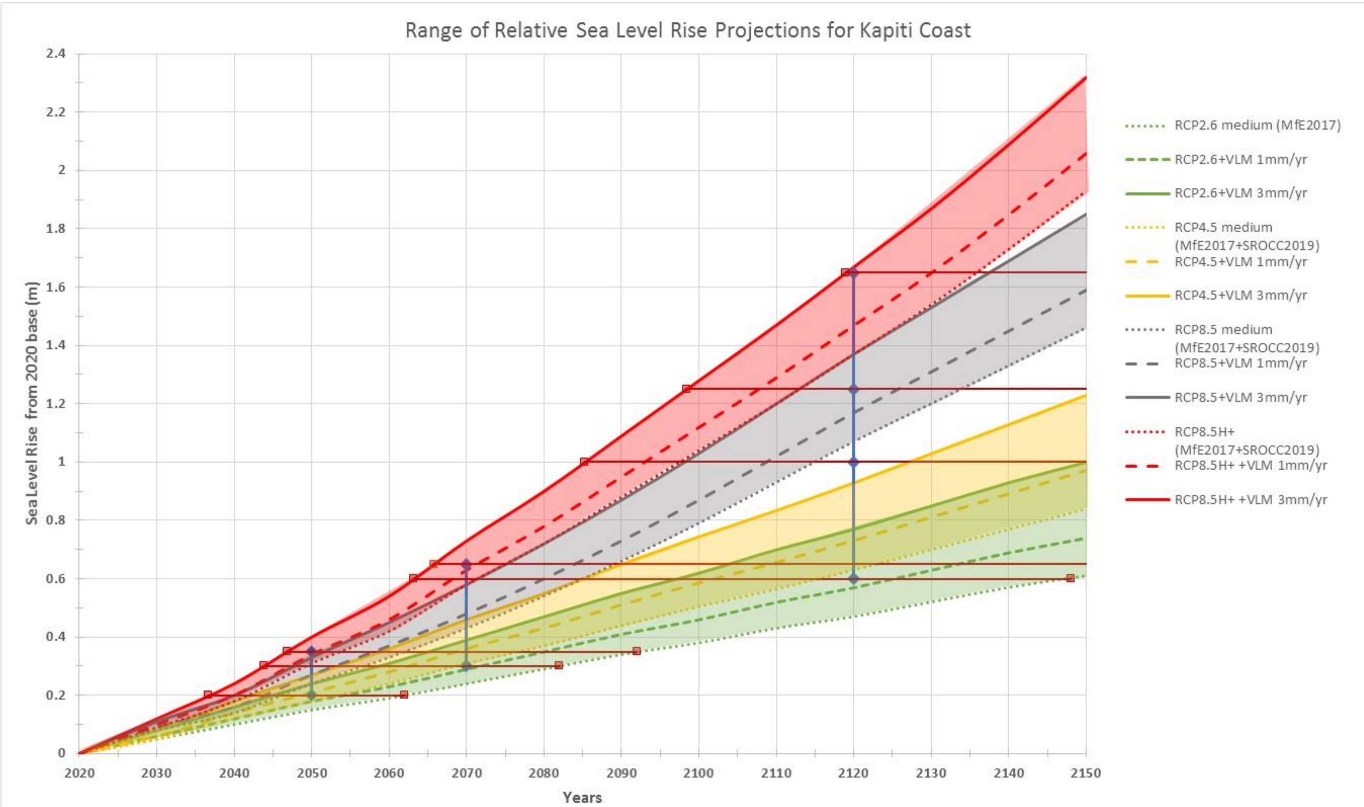

Figure 3.2: RSLR projections for the Kāpiti coast from a 2020 base level incorporating climate change scenarios and local Vertical Land Movement (VLM). The range of projections used in this assessment for the timeframes of 2050, 2070, and 2120 are also shown, along with the range of uncertainty in the timing when these projections may occur.

Table 3.3: RSLR projections from 2020 used in this coastal hazard assessment.

Year	Lower Projection of RSLR since 2020	Intermediate Projection of RSLR since 2020	Upper Projection of RSLR since 2020
2050	0.2 m		0.40 m
2070	0.3 m		0.70 m (Erosion) 0.65 m (Inundation)[1]
2120	0.6 m (Erosion) 0.65 m (Inundation)[(1)]	1.0 m, 1.25 m	1.65 m

[1]For inundation, the extent of the hazard is less sensitive to the timing of SLR. Therefore, a rise of 0.65 m has been applied as the upper projection for 2070 and the lower projection for 2120.

As shown in Figure 3.2, there is also uncertainty on when these magnitudes of RSLR may occur, depending on both the climate change scenario and the rate of local VLM. For example, the lower projection for 2050 may occur as early as 2036 under a RCP 8.5H+ climate change scenario with a VLM rate of -3 mm/yr, or as late as 2062 under a RCP2.6 climate change scenario with a nil net rate of VLM (e.g. more uplifting large earthquake events). Similarly, the lower projections of RSLR for 2120 may occur as early as 2063 or as late as 2150. It is therefore important that the planning for coastal hazard adaptation options consider trigger levels for action based on magnitudes of RSLR rather than timeframes.

Importantly, Figure 3.2 also shows that SLR will continue beyond the longest timeframe used in this assessment (100 years until 2120), with worst-case scenarios of RSLR being greater than 2 m by 2150.

4. Extreme Sea Levels at Shore

Extreme sea levels at shore are the combination of storm tide and wave set-up. As with RSLR, the current and future elevations of these extreme sea levels play an important role in the extent, frequency and magnitude of future coastal erosion and inundation hazards. Therefore, the following projections of extreme water levels at shore are presented here rather than within the methodology of the individual hazards.

4.1 With current sea levels

From their analysis of past storm events, Lane et al (2012) presents representative simulated 1% AEP storm tide plus wave set up levels based on the 6 September 1994 event[10] for five locations along the Kāpiti coast. The resulting extreme water levels at shore were presented relative to the WVD53 (2005-2011), in which MSL is 0.196 m. However, as pointed out in Section 2.3, as a result of RSLR since this time, MSL is now assed to be 0.25 m WVD53 (Bosserelle and Lane, 2019). Therefore, for our assessment, we have made the corresponding adjustment (+0.054 m) to the extreme water levels from Lane et al (2012) to be relevant to current land elevations (denoted as WVD53 (2020). The resulting extreme water levels at shore from both Lane et al (2011) and adjusted to WVD53 (2000) for this assessment are presented in Table 4.1.

Table 4.1: Current 1% AEP extreme sea levels at shore.

Location	1% AEP extreme water level at shore (storm tide + wave set-up) relative to WVD53 (2005-11) Source: Lane et al (2012, Table 3-3)	1% AEP extreme water level at shore (storm tide + wave set-up) relative to WVD53 (2020)
Ōtaki North	2.38 m	2.43 m
Waikanae Beach	2.26 m	2.31 m
Paraparaumu Beach	2.09 m	2.14 m
Raumati Beach	2.09 m	2.14 m
Paekākāriki	2.09 m	2.14 m

4.2 Future Projections with Relative Sea Level Rise

Using the range of projections of future RSLR presented in Table 3.3, the resulting future extreme sea levels at shore used in this assessment are as presented in Table 4.2.

[10] This was the event closet to the joint storm tide / wave height 1% AEP

Table 4.2: Projected future 1% AEP extreme sea levels at shore (Storm tide + wave set-up) with range of RSLR presented in Table 3.3.

		Projected 1% AEP extreme sea levels at shore (storm tide + wave set-up) with future RSLR relative to WVD53 (2020)					
		By 2050		By 2070		By 2120	
	Areas Covered in this assessment	Lower projection (RCP2.6)	Upper projection (RCP8.5H+)	Lower projection (RCP2.6)	Upper projection (RCP8.5H+)	Lower projection (RCP2.6)	Upper projection (RCP8.5H+)
Ōtaki North	Ōtaki to Te Horo	2.63 m	2.78 m	2.73 m	3.08 m	3.03 m	4.08 m
Waikanae Beach	Peka Peka to north side Waikanae River	2.51 m	2.66 m	2.61 m	2.96 m	2.91 m	3.96 m
Paraparaumu Beach	South side of Waikanae River and Paraparaumu	2.34 m	2.49 m	2.44 m	2.79 m	2.74 m	3.79 m
Raumati Beach	Raumati	2.34 m	2.49 m	2.44 m	2.79 m	2.74 m	3.79 m
Paekākāriki	Queen Elizabeth Park and Paekākāriki	2.34 m	2.49 m	2.44 m	2.79 m	2.74 m	3.79 m

5. Past Hazard Assessments

5.1 Past Erosion Hazard Assessments

Early reports of coastal erosion along the Kāpiti coast include Donnelley (1959), Gibb and Wilshere (1976), Gibb (1978) and Gibb and Depledge (1980). These reports summarise that following district wide shoreline advance from around 4000 B.C., erosion trends between Paekākāriki and Paraparaumu commenced somewhere between A.D. 150 and 1874. This erosion was considered to be largely driven by lack of sediment supply south of the cuspate foreland at Paraparaumu (see Section 2.1) and erosion episodes in large storm events, including July 1954, July 1957 and September 1976 (see Section 2.1.1). However, the reporting of shoreline change is limited to a small number of locations and is therefore not considered to constitute a district wide assessment of coastal erosion hazards.

5.1.1 Lumsden (2003)

The first district wide assessment of susceptibility to future coastal erosion hazards was undertaken by Lumsden (2003) as part of developing coastal erosion management strategies along the entire length of the district's shoreline. The assessment involved collecting data on wave conditions, water levels, bathymetry, and beach profiles for input into a geometric model of theoretical foredune erosion in storm events developed by Komar et al (1999) for the low sloping dissipative beaches of the Oregon coast, USA, that are similar to those found along the Kāpiti coast. A schematic of this geometric model is presented in Figure 5.1. Lumsden (2003) adopted this model for future shoreline erosion predictions by incorporating SLR of +0.2 m by 2050 and +0.45 m by 2100 from the IPCC 1995 projections into the extreme water level component of the erosion calculation.

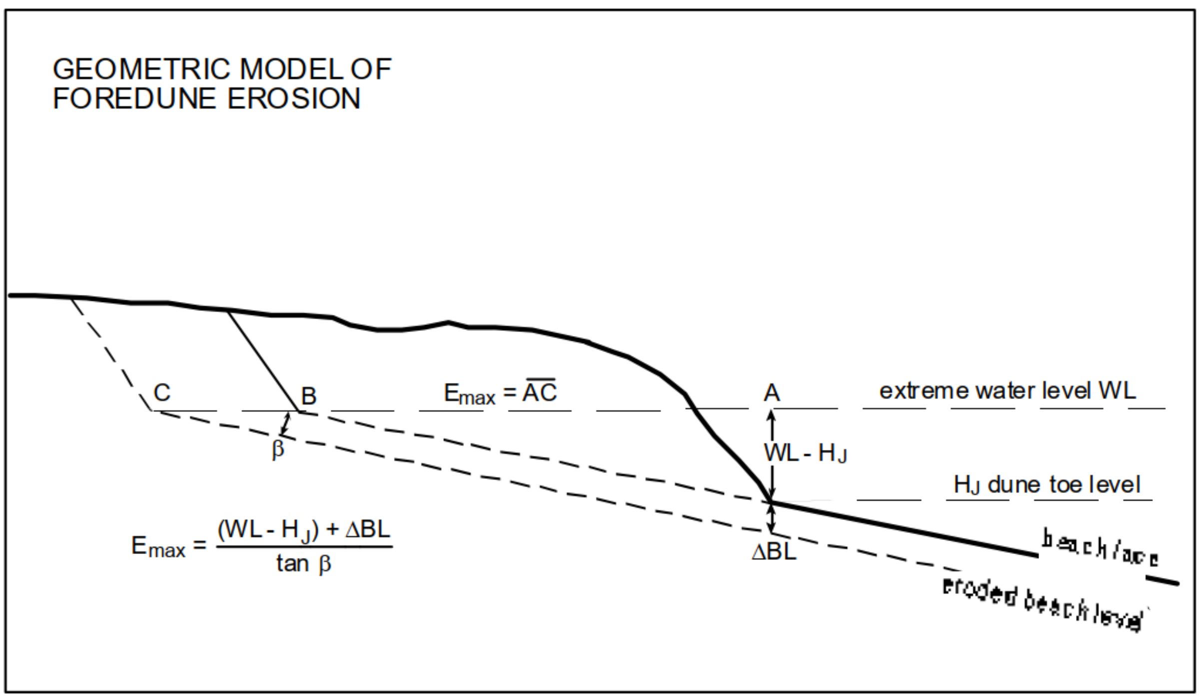

Figure 5.1: Geometric model of foredune erosion from Komar et al (1999) adapted by Lumsden (2003) to predict further shoreline retreat along the Kāpiti Coast with SLR.

From this approach, Lumsden (2003) calculated the future theoretical potential foredune erosion across seven Coastal Hazard Management Areas (CHMA's) for the following two scenarios:

- A 2% AEP storm event with a +0.2 m SLR. This represents a theoretical storm erosion potential within a 50-year timeframe.
- A 1% AEP storm event with a +0.45 m SLR. This represents a theoretical storm erosion potential within a 100-year timeframe.

The resulting theoretical erosion distances are presented in Table 5.1.

From these calculations, and with consideration of historical foredune movements presented by Gibb (1978), Lumsden proposed the following two development setback positions:

1. The Primary Development Setback (PDS) being the land at risk from fluctuations in natural beach erosion under existing conditions.
2. The Secondary Development Setback (SDS) which delineates additional land that may be at risk from erosion over the next 100 years (i.e. is additional to the PDS zone).

The proposed setback distances are also presented in Table 5.1.

Table 5.1: Proposed theoretical coastal erosion and development setback distances from Lumsden (2003).

Coastal Hazard Management Area (CHMA)	Location	Theoretical 50-year Erosion (2% AEP storm, SLR +0.2 m)	Theoretical 100 year Erosion (1% AEP storm, SLR +0.45 m)	Historical foredune movements 1874-1975 from Gibb (1978)	Primary Development Setback (PDS) Distance -current conditions	Total Secondary Development Setback (SDS+PDS) – 100-year Erosion
CHMA-1	Paekākāriki	-49 m	-63 m	-18 to -60 m retreat	25 m	60 m
CHMA-2	QE Park	-63 m	-81 m	0 to 30 m accretion	100 m (1)	100 m
CHMA-3	Raumati south	-72 m	-99 m	-24 to -37 m retreat	30 m	60 m
CHMA-4	Raumati north	-50 m	-75 m	-24 to -37 m retreat	25 m	50 m
CHMA-5	Paraparaumu	-63 m	-91 m	171 to 195 m accretion	25 m	50 m
CHMA-6	Waikanae to Peka Peka	-54 m	-80 m	49 to 171 m accretion	25 m (2)	50 m (2)
CHMA-7	Peka Peka to Ōtaki	-70 m	-98 m	50 to 95 m accretion	25 m (2)	50 m (2)
Notes: (1) PDS recommended to be set the same as SDS to limit development for purpose of maintaining natural character within Queen Elizabeth Park.						

(2) for Urban Areas. Consideration should be given to 100 m PDS setback for rural areas to restrict buildings for aesthetic reasons, with same setback for SDS.

Lumsden (2003) recognised that these setback distances were not precise with limitations in available data and uncertainty of future events and trends. As a result, he recommended that the setbacks should be reviewed on a 5-yearly basis to reflect new knowledge and conditions. In the current context, additional limitations of Lumsden's (2003) assessment include:

- Estimates of SLR have increased significantly since IPCC (1995) assessment, with the then estimate of the 100-year rise now being in the range of estimates for a 50-year timeframe.
- The calculation of run-up elevation is not from a joint probability of water levels and waves.
- The resulting theoretical erosion distances indicate that the greatest erosion will occur over a 50-year period, with reducing erosion over the 50 to 100-year period. This is contrary to the accepted effect of accelerating SLR accelerating shoreline retreat.
- The assessment is deterministic rather than probabilistic.
- Consideration of historical shoreline movements and the effect of structures in the proposed setback distances is not transparent and appears to be subjective.
- The same setback distances are applied across whole CHMA lengths, so does not recognise potential variability within the areas.

5.1.2 **Coastal Systems Limited (2008 & 2012)**

The purpose of the Coastal Systems Limited (CSL) 2008 assessment of coastal erosion hazards in the Kāpiti Coast District was to provide a more detailed localized assessment of potential coastal erosion zones for 50-year timeframes, by employing updated information and data in robust and defendable methodologies that used accepted industry best practice. The assessment covered all 38 km of sandy beach and 12 inlets within the district which used an empirically based deterministic approach to quantify future potential erosion distances by summing several components. This assessment considered three 50-year open coast scenarios in relation to the future of the existing seawalls (i.e. seawall hold, seawall fail and repaired, and seawall removed), and two 50-year inlet scenarios (inlets managed and unmanaged (natural)). The magnitude of SLR used in this assessment was 0.3 m over the 50-year time frame based on the most likely estimate from the IPCC (2007) 4th climate change assessment.

The findings of the CSL (2008) were not implemented in the KCDC District Plan and were subsequently updated with a second assessment in 2012 (CSL, 2012) to take account of directives in the NZCPS 2010, the MfE guidelines on climate change, and more recent information on coastal processes not available in 2008 (e.g. wave and longshore sediment transport modelling). While the same empirical approach of summing several erosion components was employed to meet the requirements of the NZCPS and the MfE guidelines, an additional 100-year open coast scenario of unmanaged seawalls (e.g. seawall removed) was assessed. For inlets, both managed (e.g. maintained and repaired seawalls) and unmanaged seawalls (e.g. seawall removed) were assessed. The magnitude of SLR for this 100-year assessment was taken to be 0.9 m.

Both the 2008 and 2012 CSL assessments applied the same future erosion prediction formula, being for the open coast:

$$\mathbf{CEPD = LT + ST + RSLR + DS + CU}$$

Where:

CEPD = coastal erosion prediction distance;

LT = longer-term shoreline change;

ST = shorter-term shoreline fluctuation;

RSLR = shoreline retreat associated with SLR;

DS= dune stability; and

CU = combined uncertainty.

And for inlets:

IEPD = IMC – (LT + ST + RSLR + DS +CU)

Where:

IEPD = inlet (cross-shore) erosion hazard prediction distance.

IMC = inlet migration curve, being the most landward location of the inlet shoreline migration envelope.

A summary of the methods involved in calculating each of the above components is presented below.

The results from the 2012 updated assessment are presented for 60 transect locations along the Kāpiti Coast District Coastline and can be summarised as follows:

- 50-year open coast: Predicted erosion for managed shoreline scenario ranged from 25.6 m to 120 m from the reference shoreline, and for unmanaged shoreline scenario from 25.6 m to 72.2 m. It was noted that the only place where erosion distances changed between the 2008 and 2012 assessments was along the north Raumati-south Paraparaumu coastline.
- 100-year open coast unmanaged predicted erosion distances range from 39.4 m to 129.7 m from the reference shoreline.
- For inlets north of the cuspate foreland at Paraparaumu, which are generally larger than to the south, predicted 50-year shorelines ranged between 33 m to 120 m landward of the adjacent open coast, compared to 10 m to 88 m for the smaller south coast inlets.

As the result of issues raised by residents and other stakeholders concerning the hazard methodologies and coastal hazard zones, the CSL assessment was subjected to an external review by a panel of coastal experts (Carley et al, 2014). Based on its review, the opinion of the panel was that the hazard lines recommended by CSL were not sufficiently robust to be incorporated into the Proposed District Plan, and recommended that a number of changes in methodology be considered by KCDC in the development of a revised set of hazard lines to be included in their District Plan. The panel did not reject all aspects of the CSL assessments work or approach, however they identified that there were some aspects that were overly conservative and/or mis-applied to an RMA planning approach for the District Plan. A summary of the feedback from the panel for each of the erosion components is presented below.

5.1.2.1 Extrapolation of Long-Term Rate

The long-term historical shoreline change was derived by linear regression analysis of historical shorelines at 68 transects, taken primarily from vertical aerial photos where the vegetation-front is used as the shoreline indicator. Cadastral survey maps were also used around Raumati and Paekākāriki where there was a limited dataset of aerial imagery due to structures being built in the 1950s. Where positive accretion rates were calculated, future LT extrapolation was set to zero as a precautionary measure. For transects where the historical trends of shoreline movement did not appear to be linear, the methodology was adapted, which was reported to generally produce more conservative/precautionary results for input into the future extrapolations.

In assessing the LT value where seawalls were present, the CSL methodology used linear regression modelling for an '*earlier period* (1870s to early 1950s) and also for a *later period* (1940s to 2007) to determine the impact of the seawalls on erosion rates. There are limitations of the calculation of the earlier period rate giving the uncertainty in the metrics used to identify the shoreline in cadastral surveys, which are noted in this report. The earlier period rates were used to identify the LT rate in the 'seawalls removed' scenario, with an additional estimated 'catch-up erosion' added when extrapolating into the future to account for the 50 years of erosion that seawalls had prevented.

Expert Panel Review

The review of this method generally accepted the data that was used by CSL in the assessment, the metrics of measurement (e.g. vegetation line, calculation of pre-post seawall construction rates), and acknowledged that CSL had noted the limitations involved with using cadastral survey maps to measure the pre-seawall construction rates. Carley et al (2014) did note however some concerns around the extrapolation of long-term rates into the future, including:

- Concerns around the double counting of the contribution of SLR in the 20th century, and recommended that this be removed from the extrapolation rate to ensure it was the effects of the sediment budget and construction which were extrapolated, and the effects of SLR were only included in the 'SLR' factor;
- Concerns that there may also have been double counting when the "catch-up erosion" was added to the extrapolation when a seawall was assumed to not be maintained; and
- Concerns that property owners that lived on the cuspate foreland where the shoreline is accreting were denied the benefits of living in that area (e.g. a positive accretion rate was not extrapolated, and LT = 0). The review recognised this was a common practice in the industry, however this was a conservative assumption to make.

The review panel recommended the following from KCDC:

- Within the next decade, KCDC undertake an analysis of beach-sediment budgets.
- Over the next decade, probabilistic estimate of the long-term change should be developed.

5.1.2.2 Effect of Future Accelerated Sea Level Rise

The CSL assessment used the Bruun Rule (Bruun, 1962) approach to calculate the effect of SLR, however it used the average inter-tidal beach slope in the calculations, which has a steeper profile than the slope calculated out to the cross-shore sediment transport closure depth. In the CSL (2008) report, the effect of SLR was limited to 50-60 year projections based on current IPCC SLR projections at the time (0.31 m-0.42 m over 50 years), and in the 2012 update, projections of 0.9 m over 100 years was used based on the MfE (2008) Coastal Hazards Guidance Manual. The report is silent around how the effects of SLR were calculated for beaches backed with seawalls, but it is assumed that the Bruun Rule was still applied at these sites, with relevant beach parameters input from representative beach profiles at these sites in front of the structures.

Expert Panel Review

The review noted that the results of the CSL assessment were reasonable, however there were significant uncertainties based on the selection of the beach-profile slope used in the calculations, and revisions of the results could be required with the updated SLR projections from IPCC (2014). The review is also silent on the use of this method in front of seawalls.

5.1.2.3 Dune stability

The CSL assessments used the Clark and Small (1982) slope replacement theory to calculate the retreat of the scarp top following storm erosion of the foredune along to the entire Kāpiti Coast District shoreline.

Expert Panel Review

The review found that this method was appropriate to use along the north section of the shoreline, north of Raumati where the dunes were generally sandy. However, for the southern section of the shoreline, the review panel made the following recommendations:

- Specialist Geotechnical Engineering advice to be sought regarding slope stability along more elevated portions of the coast south of Raumati, where there were more complex slope stability issues surrounding sand grain size adopted and the assumptions around dry sand.
- Omit the dune stability component from hazard zone calculations for an engineered seawall maintenance/repair/rebuild scenario, as an engineered seawall would be designed to ensure slope stability. It was acceptable to leave it in when investigating the scenario of seawall failure or removal.

5.1.2.4 Short-term Storm Erosion

The short-term (ST) component is described as cross-shore fluctuations induced by major storms, climatic cycles, or sediment variation. The ST component was calculated using the standard error of estimate (SEE) from the residuals in the linear regression of LT rates for each transect with 3 x SEE encompassing 99% of population values being the metric was used. The reports noted that this approach was better than using a 'maximum inter-survey approach' as it was able to determine the short-term shoreline fluctuation independent of any long-term effect by utilizing the fitting errors or residuals.

Expert Panel Review

The review panel describe short term as a hazard which could happen this year, or at any time in the future. They compare the CSL method, being a geologic/geographic approach, to the prior Lumsden (2003) hazard assessment, which is based on extreme combinations of ocean processes, waves, and tides in storm events. The review did not believe that the results of the short-term analysis by CSL were accurate responses to major storms, and definitely not the extreme but rare storm events that have AEP of 1-2%. The panel therefore concluded that the CSL method to not be sufficiently robust to represent the design conditions needed to account for potential short-term erosion and flooding hazards.

It was recommended by the review panel that the methodology employed by Lumsden (2003) is used in the next hazard assessment, as it considers wave conditions, extreme wave heights and swash runup levels. It was also recommended that an engineering analysis be undertaken of the existing shore-protection structures, and that this information be considered in analysing short-term impact on structures and likelihood of them to fail over a 50 – 100-year period.

5.1.2.5 River/Stream Mouth Erosion Assessment

The approach used to calculate the predicted inlet (cross-shore) erosion hazard distances (IEPD) was a modification of the open coast assessment model:

$$\mathbf{IEPD = IMC - (LT + RSLR + DS + CU)}$$

Where:

IMC = inlet migration curve;

LT = longer-term shoreline change;

RSLR = retreat of the shoreline associated with SLR;

DS= dune stability; and

CU = combined uncertainty.

In this approach, CSL identified that the calculation of these zones differed for managed and natural inlets. The IMC component captures the envelope of change in shoreline position around the vicinity of each inlet based on historical aerial imagery, and creates an interpolated curve connecting the landward limits of inlet positions. This curve was then shifted landward of the IMC by the distance equal to the sum of the hazard components (LT, SLR, DS) for the adjacent open coast site.

Expert Panel Review

The review panel noted this was a good first approximation of inlet erosion hazards. However, it outlines the following limitations of the method employed by CSL:

- The IMC is defined by the historical changes in the inlet position, which in some instances has been constrained by inlet management, and therefore there are limitations around projecting this line forward in a 'unmanaged' scenario, as it is not representative of natural processes;
- The IMC is used as a reference location to move landward, rather than depicting longshore variations in inlet position;
- The approach assumes that the inlet shoreline will migrate landward under the influence of coastal processes, when in reality the inlet is primarily driven by fluvial processes; and
- It assumes that the coast will be erosional/recessionary, despite evidence that some parts of the coast and inlets have been in net accretion in the past.

The review noted that this area of coastal hazards methodology is very under-developed, and there is no common approach to identifying future inlet erosion zone due to the complexities around both coastal and fluvial processes influencing these sites. The panel review recommend that a better analysis would require a site-by-site analysis which better accounts for unique characteristics and historical behaviour of each inlet to be examined in isolation.

5.2 Past Coastal Inundation Assessments

5.2.1 Storm tide inundation

Assessments of potential coastal inundation hazards in the Kāpiti Coast District have been undertaken by NIWA as part of investigations of these hazards within the Wellington region for GWRC (Stephens et al, 2011; Lane et al, 2012; Bosserelle and Lane, 2019).

Stephens et al (2011) presents estimates of potential coastal inundation caused by combined (joint Probability) storm tide and waves at selected sites, including two sites within the Kāpiti Coast District (Paekākāriki and

Paraparaumu), for both the current climate and projected climate change. The results of the analysis showed that on the Kāpiti coast "*hazardous events are most likely to involve a combination of large waves coinciding with a high storm tide, because storm tide and waves are highly correlated. The exposure increases to the north along this coast, due to increasing tidal range and exposure to larger waves from the west*."

Lane et al (2012) uses the results from Stephens et al (2011) and sea level trends from Bell and Hannah (2012) in a 3D hydrodynamic model to simulate storm tide inundation for selected past storm events with a joint AEP of close to 1%, including four events covering the Kāpiti coast (September 1976, January 1980, September 1994, November 1994) See Figure 2.7). The September 1994 event was considered to be the most representative of a 1% joint AEP event and was used in bathtub approach modelling of inundation extents and depths from storm tide plus wave set-up for present day sea level and future SLR of 0.5 m, 1.0 m, and 1.5 m. Due to LiDAR limitations, the results for the Kāpiti coast modelling were concentrated around Waikanae and Ōtaki Beach.

Using 2013 LiDAR, Bosserelle and Lane (2019) extend the inundation results from Lane et al (2012) to the Peka Peka and Te Horo area, and overlapped in the area around Ōtaki. From comparison of the results from both studies in this area, the results of the 2019 modelling are considered to be better due to a higher resolution of modelling (XBGPU), better consideration of processes driving inundation (e.g. infragravity waves), and a better consideration of stopbanks on the Ōtaki River. The assessment therefore concluded that the results presented should supersede those from the 2012 study.

5.2.2 Elevated Groundwater

The effect of climate change, including SLR, on groundwater levels has previously been assessed by SKM for KCDC (SKM, 2012). The study is preliminary in nature and was primarily intended to provide an initial regional understanding of whether the aquifer system is prone to being heavily impacted by climate change. It is not intended to accurately forecast local-scale impacts (i.e. at the scale of individual properties or even suburbs). More detailed assessments, including numerical groundwater modelling, would be required to determine local-scale impacts.

Groundwater surfaces were estimated for four future climate scenarios which include three increments of future SLR; 0.5 m, 1.0 m, and 1.5 m with no specific timeframes for when these rises would be reached. The study output includes maps showing the broad areas that may be more prone to prolonged periods of wet or damp ground conditions and digital datasets for the estimated depth of the groundwater surface above or below ground level. Although the modelling approach has limitations (in particular the effects of drainage of groundwater by streams, ditches and the stormwater network are not included) and cannot be considered accurate at the local scale, the outputs provide an indication of the expected change in groundwater levels as a result of climate change.

6. Coastal Erosion Methodology

6.1 General Approach

Our methodology involves a probabilistic approach to produce mapped Projected Future Shoreline Positions (PFSP) for pre-determined increments of SLR from the combination of:

- Continuation of long-term trends and patterns of shoreline movement (e.g. retreat, accretion; dynamic equilibrium[11]);
- The effects of projected future relative sea level rise (RSLR) on these long-term trends and patterns;
- A dune stability factor based on the angle of repose of the sediment found along the Kāpiti coast; and
- A short-term storm erosion component to define maximum storm erosion that may occur at or near the end of any planning timeframe, and/or the magnitude of erosion that may occur from the failure of seawalls.

These components are combined in the following formula to calculate coastal erosion distances to the PFSP.

CED = (LT x T) + SL + DS + ST

Where:

CED = coastal Erosion Distance to the PFSP;

LT = past Long-term rate of shoreline movement;

T = the time frames over which the past long-term rates are extrapolated in the future. For this assessment these are set at 30, 50, and 100 years;

SL = erosion due to future accelerated RSLR for selected range of rise over the above timeframes;

DS = dune stability factor; and

ST = short term storm erosion.

The calculations of all CED's are taken from the current 'Reference Shoreline Position', being the beach vegetation line on unprotected sand beaches, the back of the gravel beach from the Ōtaki River to Te Horo, and position of seawalls on beaches protected by these structures. This position of the 'Reference Shoreline Position' is chosen due be being easily identified on aerial imagery and in the field, and for unprotected shorelines the ability to show long-term trends in both accretion and erosion.

A 'Present Day' erosion susceptibility is calculated using the short term and dune stability factor components to show the magnitude of erosion that could occur if a 1% AEP storm event were to occur with the shoreline in its current position. A conceptual diagram of this calculation is presented in Figure 6.1.

All four components are used to determine the future coastal erosion hazard susceptibility, as shown conceptually in in Figure 6.2. This approach is consistent with the requirements of Policy 24 of the NZCPS: *Identification of coastal*

[11] Long term retreat is the long term erosion of the beach causing the reference vshoreline to move landward. Long term accretion is long term beach growth which results in the reference shoreline moving in a seaward direction. Dynamic equilibrium is the long term fluctuation of reference shoreline position about a static position.

hazards (see Section 1.3), and with the best practice recommendations in MfE (2017) *Coastal Hazard and climate Change Guidance to Local Government* (see Section 1.4) and Ramsay et al (2012) *Defining coastal hazard zones for setback lines: A guide to good practice.* The methodology to determine each of the components of the erosion hazard equation also addressed as far as possible the limitations in the previous hazard assessments identified by the expert peer review to the CSL (2012) assessment (Carley et al, 2014) (see Section 5.1.2).

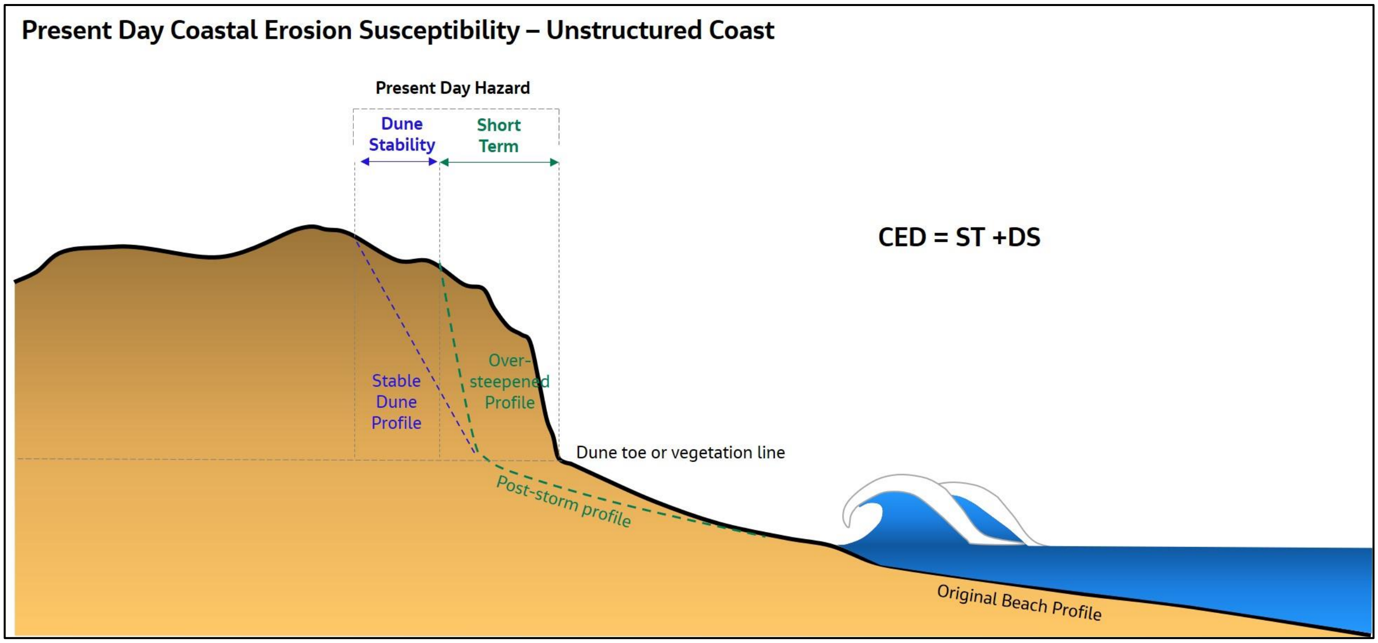

Figure 6.1: Conceptual diagram of components calculated for present day coastal erosion distance.

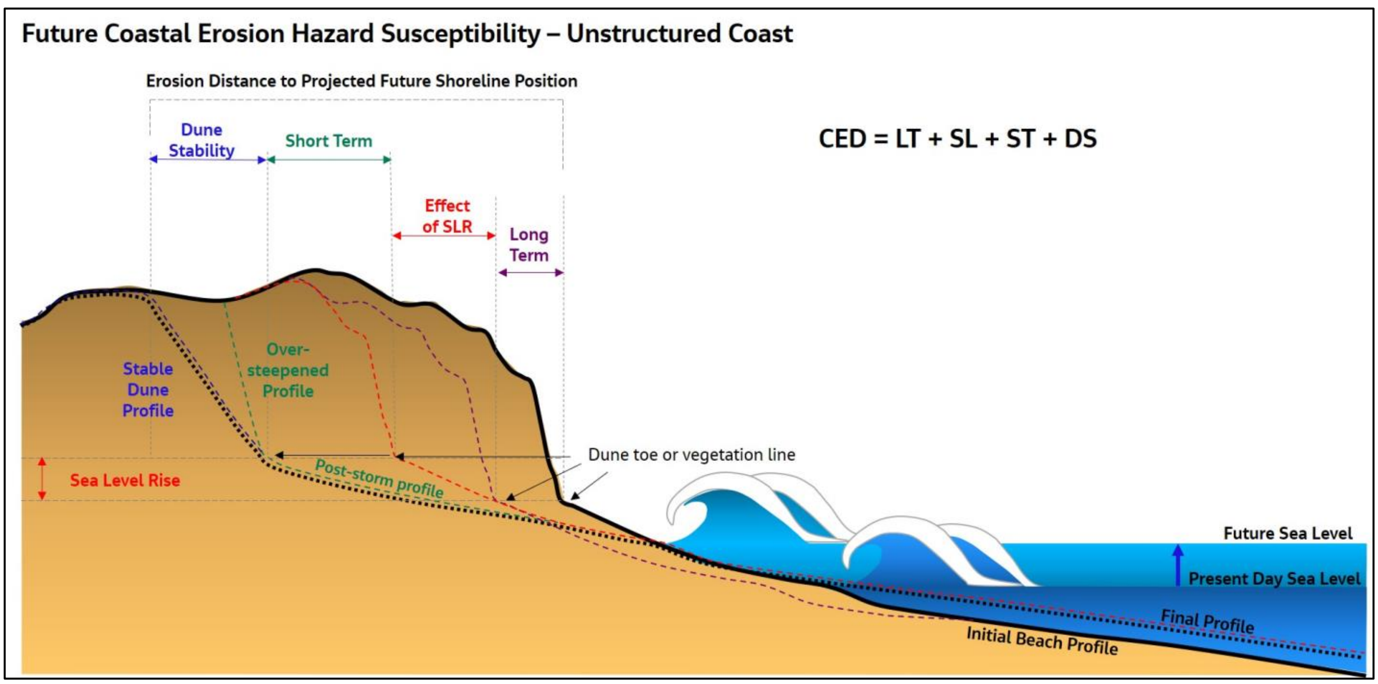

Figure 6.2: Conceptual diagram of components calculated for future coastal erosion hazard susceptibility on an unstructured coast.

This general approach to coastal erosion assessments works well for unmodified open coast shorelines, which are present along approximately 80% of the Kāpiti Coast district's shoreline. However, as outlined in Section 2.1, the remaining 20% at Paekākāriki, Raumati South, Raumati, and parts of Paraparaumu is dominated by an ad hoc series of seawalls that have had a large influence on past and present rates of shoreline movement and will continue to do so as sea level continues to rise. Thus, the presence of seawalls needs to be appropriately taken into account in the erosion assessment under this general approach. For meaningful assessment results in these areas, assumptions around the future presence of these structures, and adjustments to the general methodology need to be made, which are overviewed in the following Section (6.1.1) and the adjustments to the methodology of each of erosion components to account for the presence of seawalls are detailed in Sections 6.3- 6.6 below.

As set out in Section 2.1, the coastline of the Kāpiti Coast District also includes a number of coastal hydrosystems (i.e. river/stream mouths) where the general open coast erosion approach is not applicable due to the influences of the fluctuations in the river mouth position and fluvial processes on changes in shoreline position. The methodology for assessing the future changes in shoreline/river mouth position in these hydrosystems is outlined in Section 6.7.

6.1.1 Decision making for inclusion of structures in assessment

The decision making around how to assess the continued presence of seawall structures along the Paekākāriki, Raumati and Paraparaumu coastal cells of the Kāpiti Coast District shoreline involved consideration of the following three potential scenarios.

- Scenario A: The 'Most likely' scenario in which structures are in place and functioning until their maximum residual life as identified in the KCDC coastal structure database[12]. At the end of their maximum residual life it is assumed that they have failed, are removed, and not replaced. Following this time the coast transitions back to a natural shoreline and natural erosion processes recommence. The components used to calculate these distances for both the present day and future timeframes is shown schematically in Figures 6.3 and 6.4 below.
- Scenario B: The 'Maximum Erosion' scenario in which all structures fail and are removed in 2020. Structures are not replaced and the shoreline transitions back into a natural shoreline with natural erosion processes recommencing from current day.
- Scenario C: The 'Least Erosion' scenario under which all structures are maintained and continue to function beyond their maximum residual life to 2070 or 2120.

For the other coastal cells (e.g. Ōtaki to Waikanae, Queen Elizabeth Park) where there are currently no seawall structures, there is a fourth Scenario D: A 'No Structures' scenario under which the assumption is that no structures will be built in these locations in the future, with natural shorelines movements occurring throughout all time frames.

The decision-making process around how the LT, SL, DS, and ST components are included in the CED calculation under each structure's scenario are shown in Figure 6.5. For the Paekākāriki, Raumati and Paraparaumu coastal cells, the resulting PFSP's mapped in Volume 2 of this assessment are for Scenario A ('Most likely'). The calculation of the present day hazard (short term and dune stability) for structured coasts assumed that the storm causes the structure to fail, and the erosion hazard follows this failure (For the other coastal cells without structures, it is only relevant to calculate the CED for Scenario D (No Structures), with the resulting PFSP being mapped in Volume 2 for this scenario. This decision making regarding structures is about how they are included in the coastal hazard assessment. Any consideration of decisions around the cost benefit of maintaining existing structures or replacement with new structures is beyond the scope of this assessment.

[12] KCDC coastal structures database prepared by Tonkin and Taylor (2016) and updated in 2021 for Raumati structures.

Based on the coastal structure database[14] structures were grouped into sections of shoreline based on an alongshore average of the maximum residual life of the structure from the structure database. These shoreline sections are a minimum of 200m long (four transects) with the residual life rounded up to be in 10 year bands. For example, if a groupings maximum residual life was 5 years, for calculation purposes this was rounded up to 10. We have not made any other assumptions around the structures, except that after this timeframe the structure has failed and therefore would serve no function, and would not be replaced. River mouth training wall structures have all been assumed to have a residual life of 30 years.

The groupings of shoreline structures based on indicative maximum residual life is presented in Appendix B.

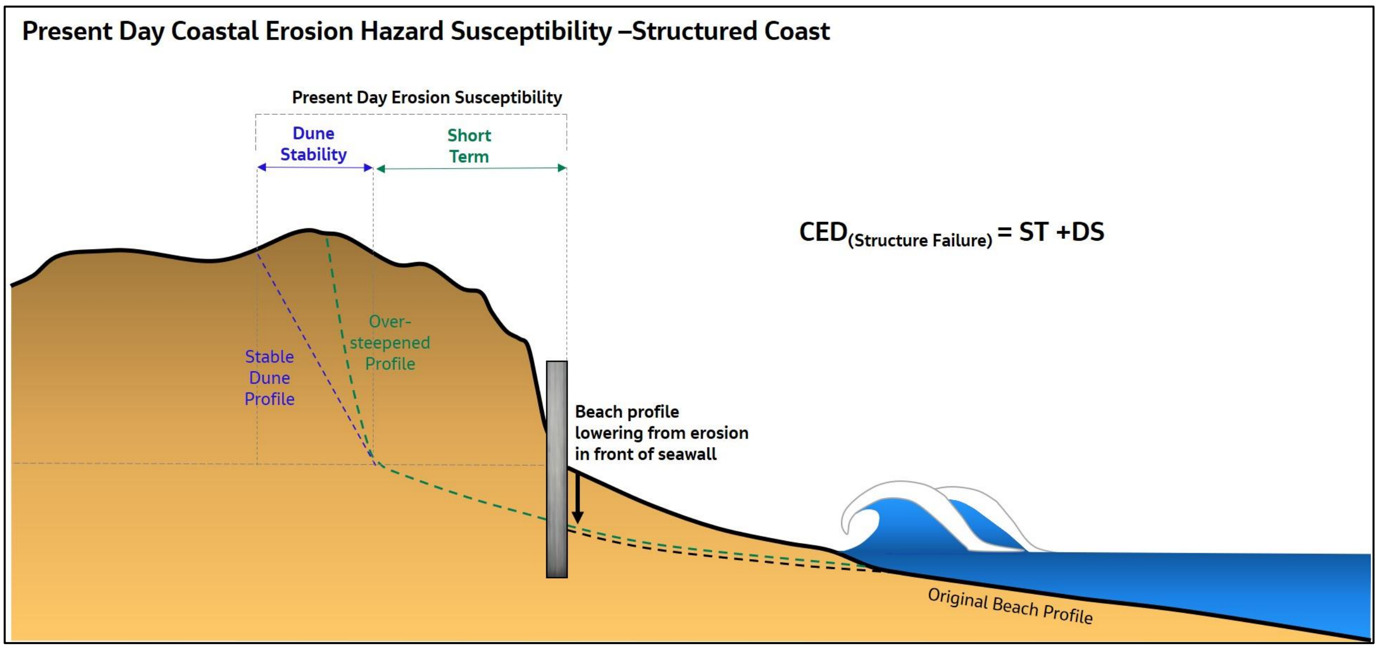

Figure 6.3: Conceptual diagram of processes occurring around a structure coast, and use of components in present day hazard calculation.

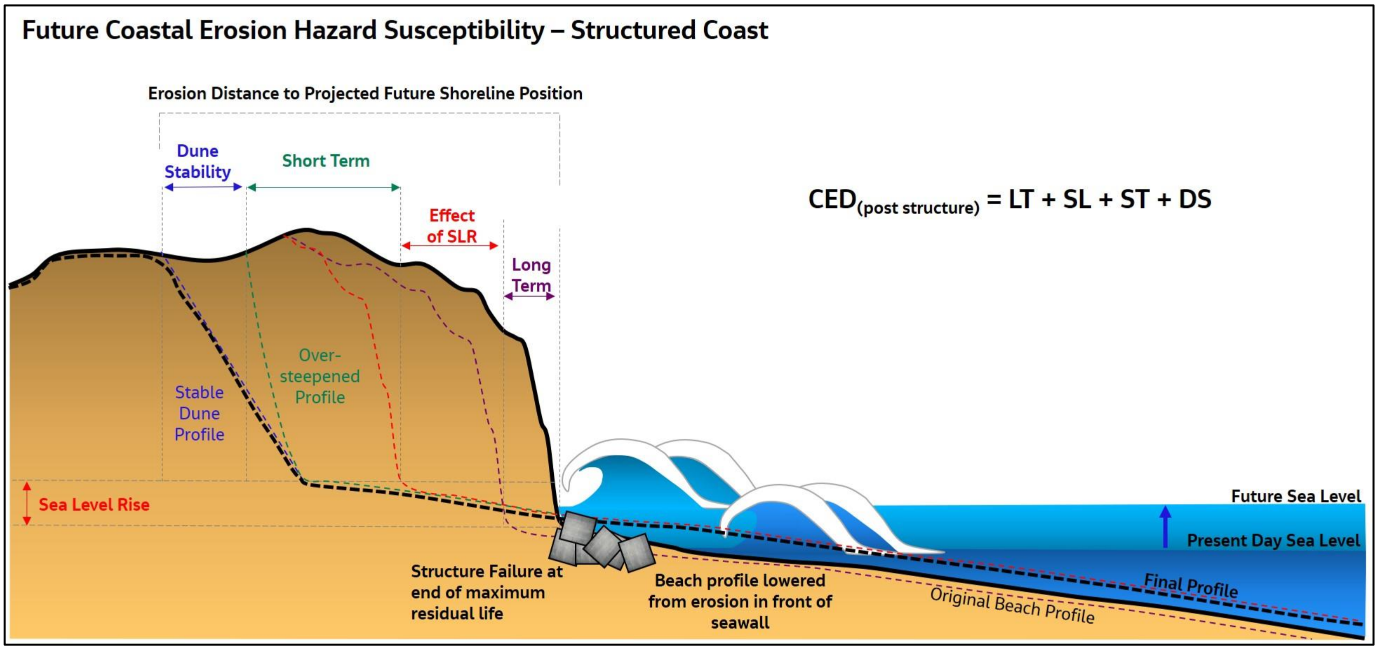

Figure 6.4: Conceptual diagram of components used to calculated future coastal erosion hazard susceptibility following structure failure (or assumed end of residual life).

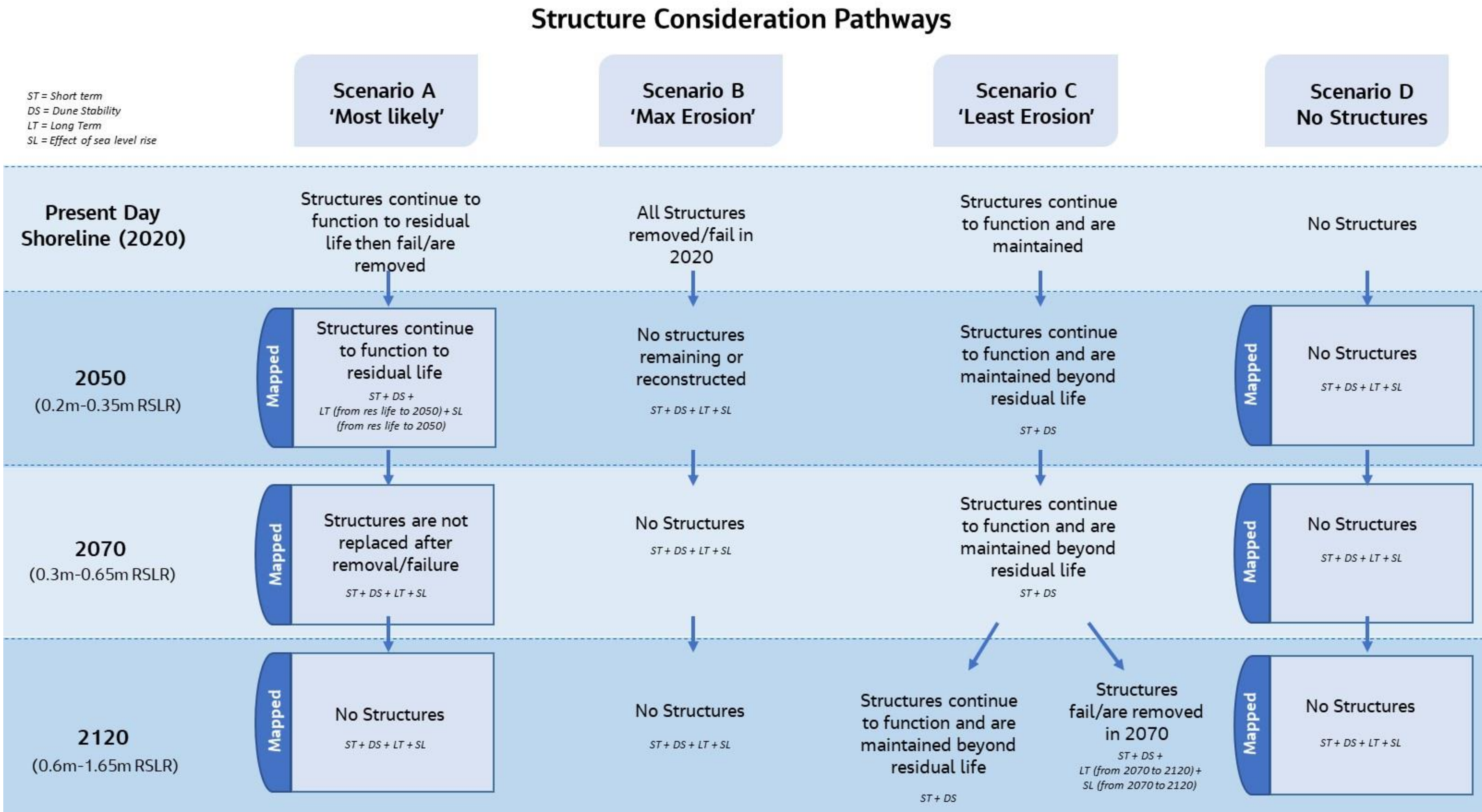

Figure 6.5: Structure Consideration Pathways which outline the assumptions around the presence/failure of structures, and what components are used to calculate the Coastal Erosion Distances (CED) at that timeframe.

6.1.2 Assessment Cells

In line with the GWRC Coastal Vulnerability Study (Mitchell Daysh, 2019), the Kāpiti Coast District shoreline has been divided into eight coastal cells for the presentation in Volume 2 of the coastal erosion assessment results. These cells have been chosen to represent areas of similar coastal morphology, process, and presence of seawall structures, as presented in Table 6.1, and shown in Figure 6.6.

There are also ten coastal hydrosystems (i.e. river/stream mouth areas) listed in Table 6.1 and shown in Figure 6.6 where the river/stream influences the shoreline position and are therefore assessed with the alternative methodology presented in Section 6.7. As shown, some of these hydrosystems form appropriate limits to the coastal cells and are located between the coastal cells (e.g. Waimeha, Waikanae, Wainui) while other hydrosystems are located within the coastal cells. Note that although the presentation of results in Volume 2 are divided into the coastal cells, the assessments methods and mapping outputs are continuous across adjacent cells to ensure that future shoreline positions conserve current open coast shoreline plan shapes and that future positions of the mouths of hydrosystems are tied into the adjacent future open coast shorelines. Note also that inundation cells are different from the hydrosystem cells, and are presented in Section 7.4.

Table 6.1: Coastal and hydrosystem assessment cells.

Coastal Cell	Beach Type	Beach modification	Historical Shoreline movement	Beach Orientation
Ōtaki North	Sand	Unmodified	Accretion	WNW
Ōtaki River to Te Horo	Composite sand and gravel	Unmodified	Accretion	WNW
Peka Peka (Te Horo to Waikanae)	Sand	Unmodified	Accretion	WNW
Waikanae Beach	Sand	Unmodified	Accretion	NW
Paraparaumu	Sand	Seawall modified	Erosion	NNW and WNW
Raumati- Raumati South	Sand	Seawall modified	Erosion	West
Queen Elizabeth Park	Sand	unmodified	Erosion	WNW
Paekākāriki	Sand	Seawall modified	Erosion	WNW
Hydrosystem Cell	**Coastal Cell Location**		**Dominant Process**	
Waitohu Stream	In Ōtaki North cell		Coastal – accretionary shoreline	
Ōtaki River	In Ōtaki River to Te Horo cell		Fluvial	
Peka Peka Inlet	In Peka Peka cell		Coastal – accretionary shoreline	
Waimeha Stream	Between Peka Peka and Waikanae Beach cells		Coastal – accretionary shoreline	
Waikanae River	Between Waikanae Beach and Paraparaumu cells		Fluvial	
Tikotu Creek	In Paraparaumu cell		Coastal - accretionary shoreline	
Wharemauku Stream	In Raumati cell		Coastal – erosional shoreline	
Whareroa Stream	In Queen Elizabeth Park cell		Coastal – erosional shoreline	
Wainui Stream	Between Queen Elizabeth Park and Paekākāriki cells		Coastal – erosional shoreline	

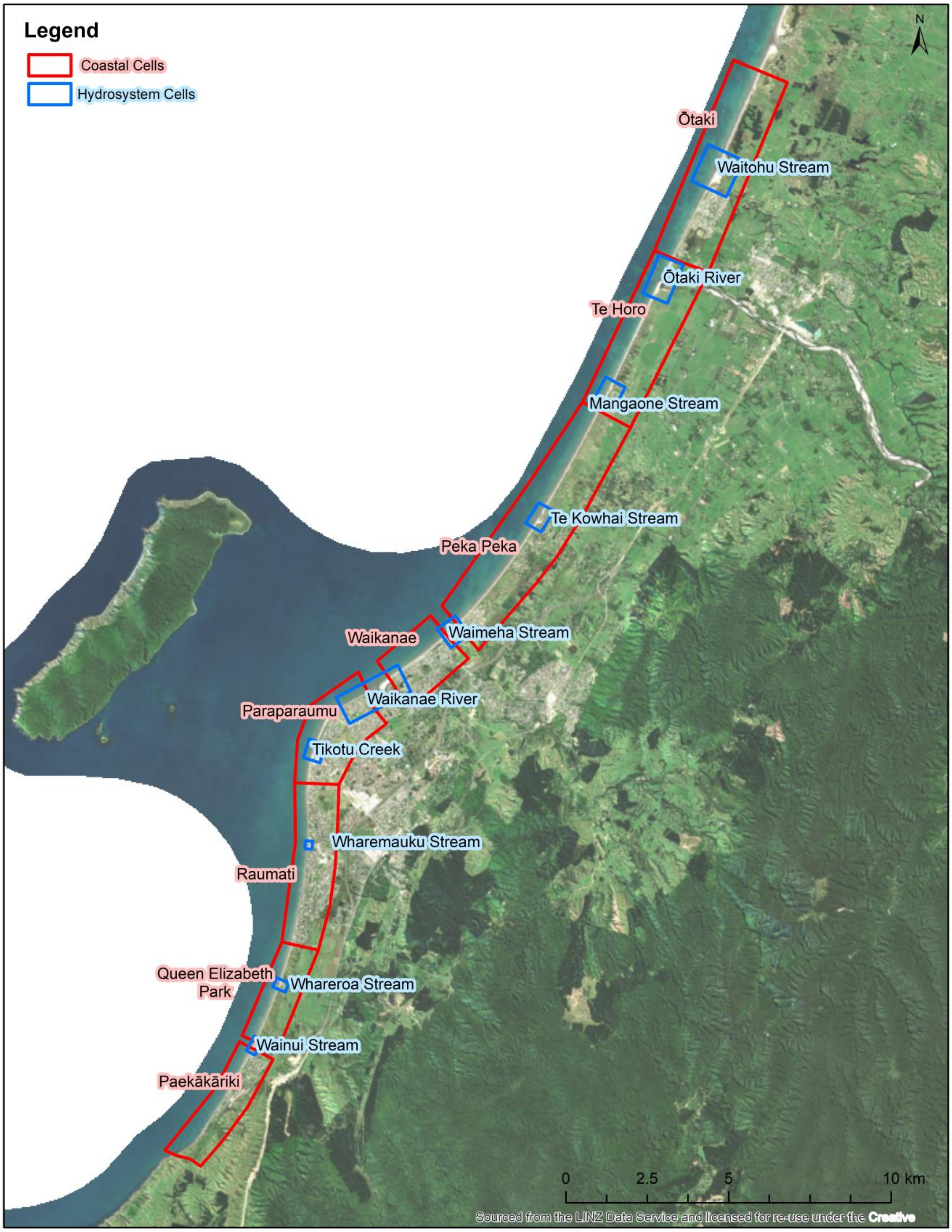

Figure 6.6: Coastal and hydrosystem cells for erosion assessment.

6.2 Probabilistic Approach

Using a probabilistic approach is considered to be a best practise industry standard, which deals with the uncertainty surrounding the data used and assumptions applied in the methods to obtain the results obtained for each of the components of the CED calculation for determination of the PFSP.

For a probabilistic approach, a range of values for each input parameter are applied to each component calculation rather than just applying a single value under a deterministic approach. Values for each parameter are selected randomly from a given distribution based on the expected natural variability above and below average values, and the random selection and component combination calculation is repeated a large number of times. This approach provides both a "best-estimate" and an understanding of the potential range of outcomes.

The probability calculations involved using the mathematical software MATLAB R2019b to run a 'Monte Carlo' simulation where for each determination transect and SLR scenario, 10,000 realizations of the PFSP lines were made by combining random values from each of the long-term (LT), short-term (ST), dune stability (DS) and SLR erosion distributions. The resulting distribution of the PFSP realizations show the range of where the projected shoreline will be in relation to the present-day shoreline, and what the probability is that the erosion could extend beyond a given distance. Figure 6.7 below shows the PFSP distribution output for a single transect, where the bars represent the number of realizations from the 10,000 trials.

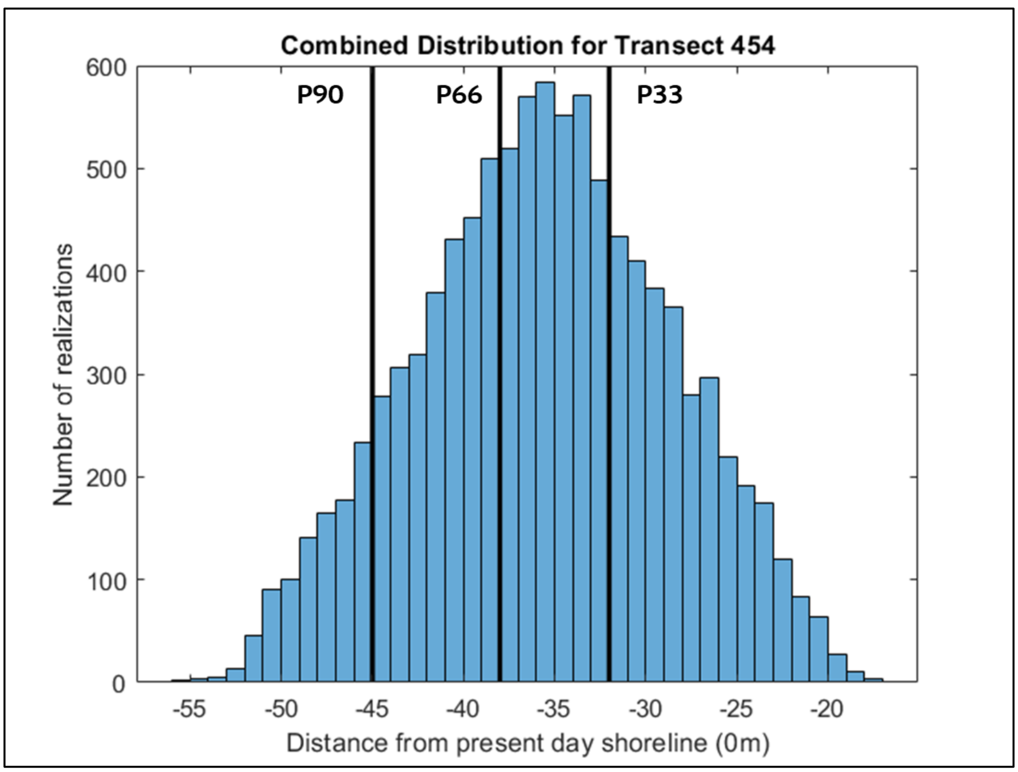

Figure 6.7: Example of the probability of PFSP distances from the present-day shoreline. The bars represent the 10,000 realizations of the projected future shoreline position made from drawing random ST, LT, SLR and DS values from their respective distributions.

To run the 'Monte Carlo' simulations, a triangular distribution was assumed for each component of the CED (LT, ST, SL, DS) at each transect using minimum, mean, and maximum values obtained or assumed from the data. This was considered the best distribution to apply given the limited nature of data available to define the uncertainty in the value of these components. The minimum, mean, and maximum values to define the triangular distribution are included in the Volume 2 results for each coastal cell. Where there are data limitations (i.e. no beach profiles for ST and LT components), the formation of a probability distribution assessment is still possible, however it requires some generic assumptions to be made around the minimum, mean and maximum parameter values, and we have termed these 'quasi-distributions'.

Under this probabilistic approach, a 'most likely' scenario and a 'very unlikely' scenario, have been developed to help guide future discussions on dynamic adaptive planning pathways and coastal asset planning. In this assessment the 'most likely' scenario covers the 33-66% probability of occurrence range of 10,000 random observations of the CED for each timeframe and SLR scenario, and 'very unlikely' scenario being the 90th percentile of the probability distribution. These scenarios are consistent with the terminology of likelihoods recommended by MfE (2017, Appendix F), with the 'very unlikely' position being the landward limit of the 'likely' range of positions.

For interpretation of the coastal erosion hazard maps, there is a 33% chance that the PFSP will be within the zone mapped as being the 'most likely' position, and only a 10% chance that the PFSP will be landward of the P90 position.

6.3 Extrapolation of Long-Term Shoreline Movements (LT)

Historical aerial imagery was collated from KCDC public database and Retrolens for the entirety of the Kāpiti Coast District coastline between 1948 and 2017 to analyse long-term shoreline trends and rates of change. The dates of the aerial imagery used and the coastal cells that they cover are presented below in Table 6.2.

Table 6.2: Dates and spatial coverage of aerial imagery used in the Digital Shoreline Analysis System (DSAS) shoreline analysis.

Date	Coastal Cells Covered	Date	Coastal Cells Covered
09/12/1948	All cells	**05/02/1977**	Raumati, Paraparaumu, Waikanae, Peka Peka, Te Horo
17/04/1952	Paekākāriki and Queen Elizabeth Park	**30/11/1978**	Te Horo and Ōtaki
18/01/1954	Paekākāriki	**17/03/1983**	Peka Peka, Te Horo, Ōtaki
29/11/1956	Raumati and Paraparaumu	**20/11/1987**	Raumati, Paraparaumu, Waikanae, Peka Peka
15/01/1957	Waikanae, Peka Peka, Te Horo, Ōtaki	**20/01/1988**	Paekākāriki and Queen Elizabeth Park
08/22/1961	Paekākāriki	**01/03/1998**	All cells
15/04/1966	Queen Elizabeth Park, Raumati, Paraparaumu, Waikanae	**01/01/2007**	All cells
01/11/1966	Paekākāriki	**24/02/2017**	All cells
23/05/1968	Waikanae, Peka Peka, Te Horo, Ōtaki		

The imagery sourced from Retrolens was georeferenced in ArcGIS using consistent stable features such as buildings and roads to ensure the correct positioning and scale of the shoreline. For more recent images where there are more stable features which can be accurately identified, we can have confidence that the position of the feature did not change in the time period between images, therefore can have a high level of confidence in the accuracy of the georeferencing. However, in earlier photographs where there are fewer stable features that can be identified due to lack of development and the change of housing stock, and therefore we are less confident with the accuracy of the georeferencing. We expect that the georeferencing of earlier images (e.g. prior to 1978) could be ± 5 m.

The reference shorelines in each image were manually digitized in GIS using coastal features which could be observed in all photographs and are considered to be representative of long-term shoreline change. The accuracy of this digitization was dependent on the quality of the aerial imagery, and the accuracy of the georeferencing. The resolution, shadowing, and light exposure are issues in earlier images which makes it difficult to identify features. In areas where the coastal feature was unidentifiable, a shoreline was not produced for that time period.

The digitized reference shorelines were used in the GIS based (Digital Shoreline Analysis System (DSAS)) tool to calculate the net shoreline change and rates of the shoreline movements since 1948 at 50 m spaced transects perpendicular to the shoreline orientation. At each transect, the DSAS tool calculates rates of shoreline movements in metres per year (m/yr) both as “end point” movement (i.e. first to last), and as a linear regression rate (LRR) and R^2 value[13] over the total number of digitized reference shorelines at that location. For the 38 km of shoreline within the Kāpiti Coast district, this approach resulted in over 760 measurements of shoreline change covering close to a 70-year period. The location of these transects within each erosion assessment cell is shown in the maps of data reference points presented in Appendix C.

For sections of the coast without the presence of coastal protection structures, the historical trend component of the PFSP equation was determined from the long-term LRR of shoreline change when the R^2 value for the transect was high (e.g. > 0.5), indicating at least some degree of linear trend of shoreline change. For transects where the $R^2 < 0.5$, further analysis of temporal pattern of the shoreline change over the individual time periods was undertaken to determine if using the entire period of analysis was appropriate. In some instances, earlier shorelines were removed from the analysis as the rate of movement in these early periods were not representative of current day processes. For example, river mouth migration and residential development adjacent to the Waikanae estuary meant that there were significant periods of shoreline advancement[14] between 1948-1956, which meant that the overall R^2 value was low due to the longer term lower accretion rates in the years between 1956-2017. To give a more accurate indication of the long-term rate which should be extrapolated into the future, the 1948 shoreline was removed from the analysis at transects on the southern side of the Waikanae Estuary, and the LRR rate used was based on long-term shoreline movement from 1956 onwards.

For the Te Horo cell, a large section of transects (558-607) had R^2 values lower than 0.5. The shoreline movement over the available aerial imagery was assessed between these transects and adjacent sites, which showed that similar trends of shoreline change at different magnitudes occurred. Due to the poor linear trend, It was determined that the linear regression rate for these transects could not be used, and the End Point Rate (EPR) was used instead. A ± 50% of the EPR was applied to form the probability distributions for these sites, which was determined to be appropriate, as there was more uncertainty in the future using the EPR given the inconsistent trends across these transects. For adjacent sites with high R^2 values and the LRR has been used, of which there is more certainty around the future trends at the site, and the 90% confidence interval applied for the probability distributions at these transects equate to between 20-40% of the raw LRR rate. Therefore, applying a ± 50% factor to the EPR to form the probability distribution is appropriate to demonstrate this uncertainty.

For shoreline sections where a seawall structure was present, therefore had a zero shoreline movement ‘post-structure’, there is a need for a ‘non-structure’ rate for the extrapolation into the future to account for shoreline erosion if or when a structure fails and the coastline returns to an unmodified coast. A common approach in

[13] R^2: the square of the correlation coefficient is a measure of the strength and direction of the linear relationship between two variables. Positive correlation has values ranging from 0 to 1, where 1 indicates the strongest possible linear relationship. For this assessment the two variables are erosion distance and time.

[14] Shoreline advancement is when the shoreline reference point is moved seaward as a result of multiple factors such as development along the shoreline, reclamation, changes to adjacent river mouth positions, and not necessarily only a result of natural shoreline accretion.

these situations is to use the shoreline change from an adjacent un-modified section. However, this is not practical for the Raumati and Paekākāriki shoreline cells due to the continuous length of seawall structures. For these cells, a 'pre-structure' rate was used for the future extrapolation, obtained from the CSL (2008 & 2012) assessment, where an assessment of shoreline change pre-structures was undertaken at 1 km intervals for the entire Kāpiti Coast District shoreline using pre-structure aerial imagery (1954), and cadastral surveys from the 1870's. This is referred to in CSL (2008) as the "earlier rate". CSL acknowledges the limitations of the calculation of this earlier rate, as cadastral surveys and aerial imagery use different shoreline indicators (e.g. the high water line and the vegetation line respectively), which introduces an unresolvable systematic error when combining the two data sources, and can result in an over-estimation of shoreline erosion. For this study, CSL (2008) "earlier rates" were used as the long term rate for relevant shoreline transects as presented in Table 6.3. The location of this CSL profiles relative to the DSAS transects are presented in the Data Reference Maps in Appendix C.

Table 6.3: Details of CSL (2008) 'earlier rates' used as pre-structure rates in this study, and what transect these rates were applied to in this study.

CSL (2008) site reference	Relevant DSAS Transects applied in this study	Rate of shoreline movement (m/yr)	CSL (2008) site reference	Relevant DSAS Transects applied in this study	Rate of shoreline movement (m/yr)
C0-17	1-23 (Paekākāriki cell)	-0.08	C8-02	177-188 (Raumati cell)	-0.112
X0-33	24-25 (Paekākāriki cell)	-0.13	C8-72	189-199 (Raumati cell)	-0.231
X0-65	31-32 (Paekākāriki cell)	-0.203	C9-11	200-206 (Raumati cell)	-0.201
C0-73	33-34 (Paekākāriki cell)	-0.201	C9-43	207-215 (Raumati cell)	-0.021
X0-82	35-43 (Paekākāriki cell)	-0.151	C10-29	216-228 (Raumati cell)	0.205
C1-51	44-61 (Paekākāriki cell)	-1.31	C10-40	229-232 (Raumati cell)	0.168
C2-62	62-81 (Paekākāriki cell)	-0.062	C10-61	233-239 (Raumati cell)	0.078
C6-76	153-159 (Raumati cell)	-0.193	C11-17	240-247 (Raumati cell)	0.353
C7-10	160-167 (Raumati cell)	-0.237	C11-41	248-253 (Raumati cell)	0.304
C7-56	168-176 (Raumati cell)	-0.231			

For these transects with structures, the calculation of the long-term shoreline change component (LT) involved extrapolating the 'pre-structure rate' for the timeframe over which structure is assumed to be no longer present. For example, if a structure has a residual life of 20 years, then the long-term rate is only extrapolated from 2040 onwards, and therefore only 10 years of the extrapolated long-term rate is included in the shoreline calculation for the 2050 timeframe.

An exception to this approach was applied along the section of shoreline between 169 – 183 Manly Street, Paraparaumu that is backed with a concrete block seawall. At this location, covering three DSAS transects, the earlier 'pre-structure' rate from CSL (2008) was for accretion at a higher rate than what was calculated in the DSAS, and hence created a large isolated discontinuity in shoreline plan shape when applied to the three transects. To avoid this situation, the DSAS 'pre-structure' rates were extrapolated rather the CSL (2008) earlier rates. This is considered an appropriate approach for this situation.

For the probability distribution of the LT component at sites with protection structures, since there were no error bands around the CSL (2008) 'earlier rates', a 'quasi-distribution' was developed with assumed distribution of ± 50% of the rate measured by CSL.

6.4 Effect of Future Accelerated Sea Level Rise

A key point in the consideration of the erosion effects of SLR, is to only deal with the potential effects of future accelerated rise, and not 'double account' of the contemporary rates of rise that are already included in the extrapolation of historical long-term shoreline movements. In this assessment, this is dealt with by discounting the future rate of SLR over the specified time frames by the known contemporary SLR, so that the potential effects are limited to only those associated with the acceleration in rate of SLR.

For un-protected shorelines, the discount off future SLR is the contemporary rate of 2.74 mm/yr (RSLR for Wellington 1961-2015 from Bell et al (2018)). However, for shorelines protected with structures where the historical shorelines movements prior to 1948 from CSL (2008) have been used to assess potential movements once structures are removed, the appropriate discount off future rates of SLR is 0.72 mm/yr, being the Wellington average rate for 1891-1960, from Bell et al(2018).

6.4.1 Sand Beach

A standardized 'Bruun Rule' (Bruun,1962) approach was used to determine sand beach retreat with the incremental increases in sea level over the 30, 50 and 100 year timeframes. Although this method is widely used in the international literature and is recommended in MfE (2017), it is also widely criticized for its limitations. The model involves the assumptions of conservation of an equilibrium profile shape with the volume eroded seaward from the beach being that required to raise the nearshore profile out to the closure depth for cross-shore sediment transport by the same vertical increase as the increase of SLR, as shown below in Figure 6.8. Therefore, the resulting horizontal shoreline retreat is dependent on the beach-nearshore slope from dune crest to the closure depth and is expressed by the following equation.

$$\boldsymbol{Retreat}\ (\Delta \boldsymbol{x}) = \frac{\boldsymbol{L} \times \boldsymbol{s}}{(\boldsymbol{h} + \boldsymbol{d})}$$

Where:

L = the horizontal distance to the closure depth from the beach crest;

s = projected SLR over the planning timeframe;

h = the height of the dune above Mean Sea Level (MSL); and

d = the average closure depth below MSL.

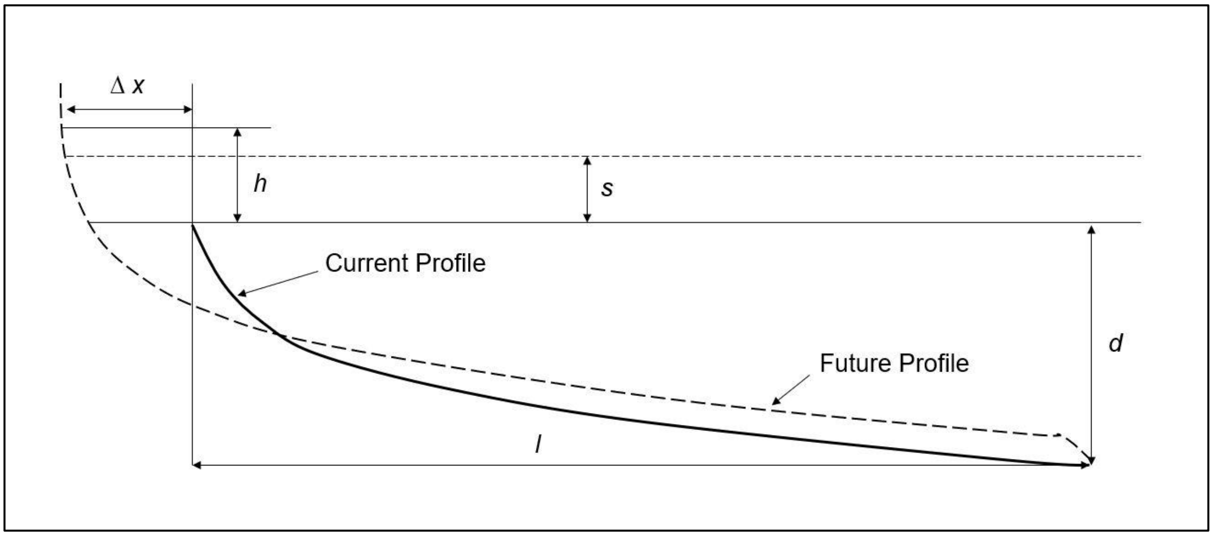

Figure 6.8: Schematic of Bruun Rule components.

Height of the dune above MSL was obtained from the average beach profile envelope calculated in BMAP from repeated beach profiles surveyed for KCDC at 26 sites across the district between 2000 and 2018. The locations of these profiles are shown in the Data Reference Point Maps presented in Appendix B. Maximum and minimum dune elevations for the probability distribution were obtained from the maximum and minimum profile envelopes respectively.

Closure depth was calculated using Hallermeiers (1981) equation. In the absence of time series wave data, significant wave height and wave period for input in the calculations were taken as the 1-year return period value for the closest site presented in the MetOceans (2007) assessment, which is considered to be a good proxy for the 12 hour per year exceedance values used by Hallermeiers (1981). The closure depth distribution for the probability analysis was formed from using the inner and outer Hallermeiers as the minimum and maximum values of the distribution and taking the mid-point of these two values as the mean. Distances and depths offshore to the respective closure depths were calculated using the Bathymetric profiles surveyed on 2000 as presented in Lumsden (2003). The locations of these bathymetric profiles are shown in the Data Reference Point Maps presented in Appendix C.

For sections of shoreline that are currently protected by structures, the effect of SLR on future shoreline retreat is restricted to the time frame following assumed structure failure and removal at the end of their residual life (see Section 6.1.1). Up until this time, the indicative effect of SLR on beach lowering and increased wave attack on the structures was determined by profile translation under Bruun Rule assumptions of conservation of volume to indicate whether accelerated failure from toe scour or overtopping was likely to alter the residual life of the structure. Following the assumed failure/removal, the indicative shoreline retreat due to SLR was calculated by the Bruun Rule using the average natural height of the dunes behind the seawalls obtained from 5-10 LiDAR profiles extracted from the 2017 LiDAR up to 200 m either side of the beach transect site. For example, if a structure had a residual life of 20 years, then SLR component was calculated as the erosion from the rate of rise between 2040-2050 for the 2050 shoreline.

Consideration was also made of the reduced erosion effect of SLR as the shoreline erodes into land of higher elevation than the present day, which involved sensitivity testing of the SLR effect for greater dune height within the 100-year scenario. At each survey profile, and LiDAR profile was extrapolated out to the preliminary 100-year shoreline (with 1 m of SLR) to determine what the land elevation (or dune height) could be at that time. If this elevation was greater than the height used for the original SLR calculations, it was rerun for the 2070-2120 period using the higher elevation. The results of the sensitivity testing showed that four profiles (200, 250, 260, and 280) could have 14 to 16 m less erosion over 100 years if consideration is made of the higher landward elevation. These higher elevations are included in the probability distribution for the SLR calculations. Profiles 220, 230, 270 and 290 had a lesser effect of 2-6 m over the 100-year period if higher landward elevations were considered, while at all other profiles the land elevations at the 100-year shoreline were either the same as the present day, or were projected to be seaward of the present day shoreline due to on-going accretion being at higher rates that the effect of SLR.

6.4.2 Composite Beach

A modification to the original Bruun Rule was required to assess the potential effect of SLR on the composite beach in the Ōtaki River to Te Horo coastal cell. During the site visit, a wide surging surf zone was identified along this section of beach, and therefore while in literature it is referred to as a mixed sand and gravel beach, this wider surf zone indicates that sometimes there is a flatter nearshore environment as opposed to the steep nearshore step characteristic of true mixed sand and gravel beaches. It is considered most likely that the beach fluctuates between mixed sand and gravel and composite beach properties, but due to a flat surf zone slope it is assessed as being a composite beach for the purpose of determining potential SLR erosion effects. This approach has been used in coastal hazard assessments for similar beaches on the Canterbury Coast (Jacobs, 2020), and is based on the assumption that the presence of gravel will slow the erosional effect of SLR. The approach involves multiplying the Bruun Rule result by the average percentage of sand found across the beach. Although there was no sediment sampling data available from the beach profile sites used, observations of the sediment size distribution averaged across the whole profile were noted on the site visit which formed the basis of quasi-distributions of the sand/gravel ratio, along with consideration of the longshore distance from the gravel source (Ōtaki River). These observations indicated that the presence of gravel in the beach profile stopped approximately 5 km south of the Ōtaki River mouth. The following assumptions about sand/gravel contributions were made about each of the 3 profiles used within the Te Horo cell.

- Beach profile 440 is located on the south side of the Ōtaki river mouth, and it was observed in the site visit that there was likely to be 50% gravel contribution to the profile.
- Beach profile 430 is located 1.4 km south of profile 440, and it was assumed than gravel contribution decreased to 35%, with sand contribution increasing to 65%.
- Beach profile 420 is located 3.4 km south of profile 440, and on the site visit it was observed that there was still a small presence of gravel in the profile, which depleted to 0% over 1.5 km to the south. Based on these observations, it was assumed that gravel contribution here was 15% and sand contribution was 85%.

This sediment size modification slows the rate of retreat from the original Bruun Rule formula to account for how much gravel is present in the beach, with the modified retreat formula applied to Composite Beaches being:

$$\boldsymbol{Bruun_{Composite}} = \frac{\boldsymbol{L \times a}}{\boldsymbol{(h+d)}} \times \%\ \boldsymbol{of\ Sand}$$

Where:

L = horizontal distance to closure depth from dune crest;

s = SLR over the planning timeframe;

h = height of beach crest above MSL; and

d = average closure depth below MSL.

6.4.3 Limitations

There are a number of well documented limitations to the Bruun Rule approach for estimating the effect of SLR on coastal erosion. The most relevant of these for the Kāpiti Coast District coast include:

- It assumes only two-dimensional cross-shore sediment movements hence does not include consideration of longshore sediment transport inputs/losses or plan shape controls (e.g. headlands).
- It does not take into consideration progressive elevation changes in the backshore over a temporal scale as the shoreline retreats, which as elevation increases in the backshore, there is more volume of material to erode and hence the erosion rate may slow, and therefore could be over-conservative in some locations, as noted above.
- It also does not take into account changes in the material in the backshore, which could also slow or speed up erosion rates. This is not particularly relevant for Kāpiti Coast as most backshore environments are either peat (Gibb, 1978) or old coastal sand dune plains.
- Is only applicable to equilibrium beach profiles, hence the resulting profile will only be reached at some time after SLR has reached the stated level, therefore results are considered to be very conservative for the time frames and magnitudes of SLR used in the calculations.
- The effect of SLR is dependent on the total beach/nearshore slope out to the closure depth (termed closure slope), where steeper slopes require less sediment volume to raise the nearshore bed, and therefore less erosion distance is predicted. The nearshore profile south of the cuspate foreland differs to the typical equilibrium sand beach profile due to the widely flatted nearshore caused by the downdrift sand bank from the cuspate foreland, which then drops steeply into the Rauoterangi channel. This non-equilibrium nearshore profile results flatter closure slopes to the inner Hallermeiers limits, and hence greater erosion with SLR, compared to the steeper slopes out to the outer Hallermeiers limit due to the steep drop off into the Rauoterangi channel.
- Todd and MacDonald (2020) noted that the composite beach modification to the Bruun Rule better accounts for the cross-shore sediment transport loses of sand from these beach profile with SLR, as well as retention of gravels on the upper beach/berms. However, they also recognised that this method raises a contradiction between the Bruun Rule assumption of conservation of equilibrium nearshore depth by beach erosion volume, and less beach erosion volume with offshore transport of the gravels from the upper berm/crest region being limited. They concluded that In the long-term, this differential rate of loss of sand compared to the gravel components could result in the beach converting to a more mixed sand and gravel form unless abrasion of the gravel component keeps pace with the offshore sand losses due to SLR.

6.5 Short Term-Storm Erosion (ST)

The Carley et al (2014) panel review recommended that the geometric model used in Lumsden (2003) (Section 5.1.1) was applied in future assessments. In an initial pilot study in the Raumati area as part of this project, this approach was adopted to calculate short term erosion distances for input into the CED equation. As a result of structures being built along the shoreline, the beaches in front of structures have lowered. When these lowered beach elevations are used in the geometric approach, short term erosion distances of up to 100 m were produced. Sensitivity testing of this method on open coastal and structured profiles are presented in Appendix B. Based on observations of erosion in Raumati following the significant September 1976 storm (e.g. largest on record) by Gibb and Wiltshere (1976), which identified that 15 m of erosion occurred where structures were present, it was determined that the results of using this approach were not realistic.

In this assessment, a quasi-distribution was developed for each coastal cell based on the observations noted by Gibb and Wiltshere (1976) for the September 1976 storm, which was classified by Lane et al (2012) as being close to a 0.5% AEP joint wave and storm tide event. The observations from Gibb and Wiltshere (1976) are summarised below in Table 6.4. For the probability distribution, the mean value was taken as the upper limit of the short-term observation (e.g. Mean for Paekākāriki was 5 m) and a minimum and maximum were generated as ±50% of the mean value. The taking of the existing upper magnitude of storm retreat observations as the mean value could be considered to be a conservative approach, however this is counteracted by the increased frequency of this magnitude event in the future due to SLR. Sensitivity testing of these observed short term erosion distances was undertaken using the SBEACH model for unmodified sites (Appendix B), which produced similar results to the short term storm observations in Waikanae. However, it is noted that there are limitations with model input data for the September 1976 storm event (e.g. lack of water level and wave time series).

Table 6.4: Observations of short-term storm erosion from the September 1976 storm, reported in Gibb and Wiltshere (1976).

Coastal Cell	Short Term Erosion Observation
Ōtaki	Not given. Assumed as 5-10 m(1)
Te Horo	Not given. Assumed as 5-10 m(1)
Peka Peka	5-10 m
Waikanae	1-5 m
Paraparaumu	1-5 m
Raumati	10-15 m
Queen Elizabeth Park	1-10 m
Paekākāriki	1-5 m
[1]Assumed on basis that NW storm wave heights at Te Horo and Ōtaki would be similar, if not greater, than at Peka Peka, therefore similar storm erosion effects.	

6.6 Dune Stability (DS)

The dune stability (DS) factor delineates the area potentially susceptible to erosion landward of the erosion scarp. The parameter assumes that storm erosion results in an over-steepened scarp which must adjust to a stable angle of response for loose sand. The dune stability width is dependent on the height of the existing dune and the angle of repose for loose sand. The use of 'dry' sand angle of repose is appropriate as it is the process of sand becoming less cohesive as it dries that promotes the slope failure.

For this assessment, dune stability was calculated using the following standard industry practice formula, which the Carley et al (2014) review panel noted was appropriate for the low dunes in northern part of the District. While it is accepted to be non-conservative, it is based on the "slope replacement theory of cohesiveless sand" (Clark and Small, 1982) as set out in Appendix E from CSL (2008). The h/2 factor is due to the eroded cross sectional area approximately equal to the cross-section of deposition as talus, and since scarp face after stability adjustment is perpendicular to pre-storm face: $h/(DTR+STR) = \tan \alpha$, therefore:

$$\mathbf{STR = h/2(\tan \alpha)}$$

Where:

STR is the landward distance, the scarp top must retreat to achieve dune stability (DS);

h = the height of the escarpment; and

α = the stable slope angle of repose of dry sand (30-34 degrees).

The formula is based on the "slope replacement theory of cohesiveless sand" (Clark and Small, 1982) as set out in Appendix E from CSL (2008). The h/2 factor is due to the eroded cross sectional area being approximately equal to the cross-section of deposition as talus, and the scarp face after stability adjustment being approximately perpendicular to pre-storm face as shown in Figure 6.9.

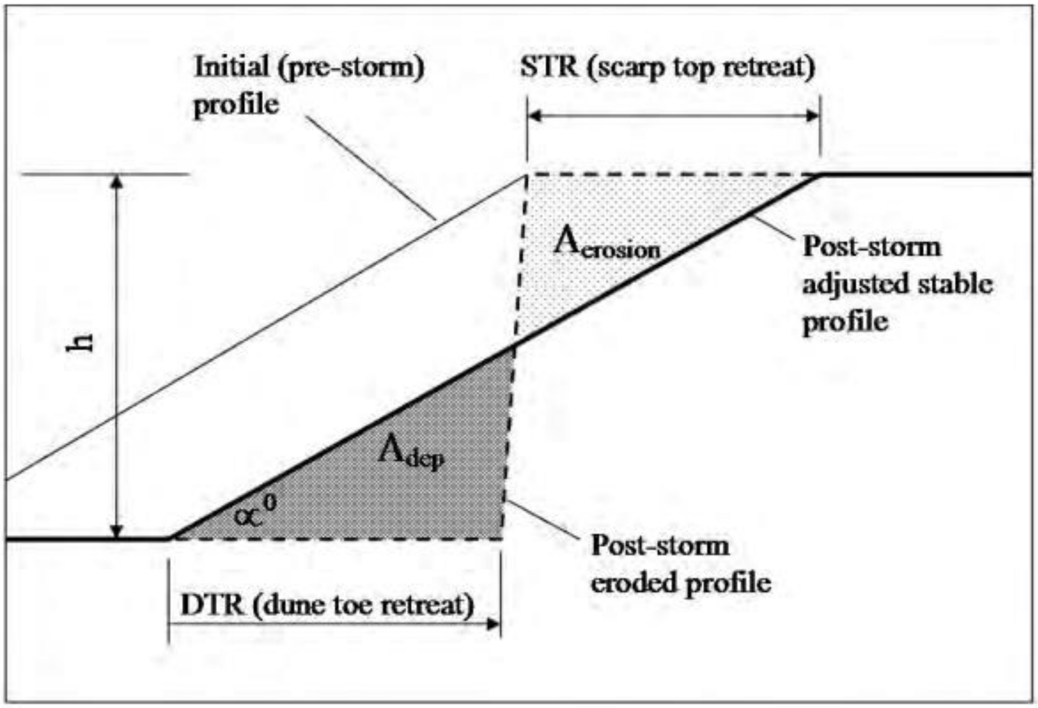

Figure 6.9: Schematic of theory of dune stability calculations. (from CSL, 2008 Appendix E).

The height of the escarpment was calculated using beach profile data, where the dune toe elevation was subtracted from the dune height to give an estimate of the potential maximum height of the escarpment. This was calculated at all beach profiles, and the minimum, mean and maximum escarpment heights were used for the probability distribution. Dune heights and toe elevations were taken from beach profile survey data where possible, and when structures were present, these were taken from LiDAR in line with measurements used for calculating the effects of SLR on structured coasts (Section 6.4). A range of stable angle of reposes for dry sand from 30 degrees to 34 degrees were also incorporated into the probability distribution to account for changes in the grain size and wetness of the sand. The resulting probability distribution at each profile involved: a 30 degree angle of repose paired with the largest escarpment height to give a maximum STR value; a 32 degree angle of repose was paired with the mean escarpment height to give a mean STR value; and a 34 degree angle of repose was paired with the smallest escarpment height to give the minimum STR value.

The dune stability factor was not applied in the Te Horo cell due to the presence of a composite beach, within which the low gravel crest barrier retreats by rollover rather than dune scarp formation.

6.7 Hydrosystem (River/Stream Mouth) Erosion Assessments

For this assessment we have defined ten river/stream mouth locations shown in Figure 6.6, where an alternative approach was required to estimate the position of the future shoreline within the mouth environment.

In the early stages of developing this methodology, discussions were held with coastal hydrosystems expert Dr Deirdre Hart and GWRC senior policy advisor (hazards) Dr Iain Dawe who agreed that there are currently limited tools available to determine the effects of SLR on coastal hydrosystems. However it is likely that the following changes to river/stream mouth environments may be observed with SLR:

- The mouth environments will likely migrate landward with the adjacent shoreline;
- Where there are spit barriers (e.g. Ōtaki River, Waikanae Estuary) we could expect to see some 'squeeze' of the waterbody as the barrier migrates landward with adjacent shoreline into the coastal lagoon/hapua;
- If there are higher banks upstream of the inlet, they will be able to hold more water and therefore the throat of the river would be less inclined to migrate; and
- The landward migration of the shoreline will have an effect on river mouth training structures.

We determined that, in line with the expert panel review comments (e.g. Carley et al, 2014), each inlet needed to be assessed individually, and consideration needed to be had for:

- The position of the mouth environment in relation to the adjacent future shoreline position;
- The topography and elevation of the land surrounding the mouth environment;
- The conservation of area and volume of the available water ponding within the mouth environment;
- The relationship of the future width and depth of the mouth throat to its current position; and
- The occurrence of structures and assumptions around their future existence.

Based on the individual responses to these considerations for hydrosystem, we developed a decision tree approach for determining how the future position of each mouth inlet would be assessed. As a result, four assessment methods were developed to account for the different hydrosystem characteristics. An overview of the decision tree approach is presented in Figure 6.10, and breakdown of the characteristics to inform the method used at each inlet is presented in Table 6.5. A brief summary of each method is presented in Sections 6.7.1 to 6.7.4.

Following the decision tree approach, a range of 'Hydrosystem Extents' were developed for mouth location for each timeframe and SLR scenario. These zones are subjective and are based on the considerations above. These zones indicate the extent of potential longshore migration of the river/stream mouth in the future, with the landward extent of the zone being the anticipated maximum landward position of the hydrosystem environment.

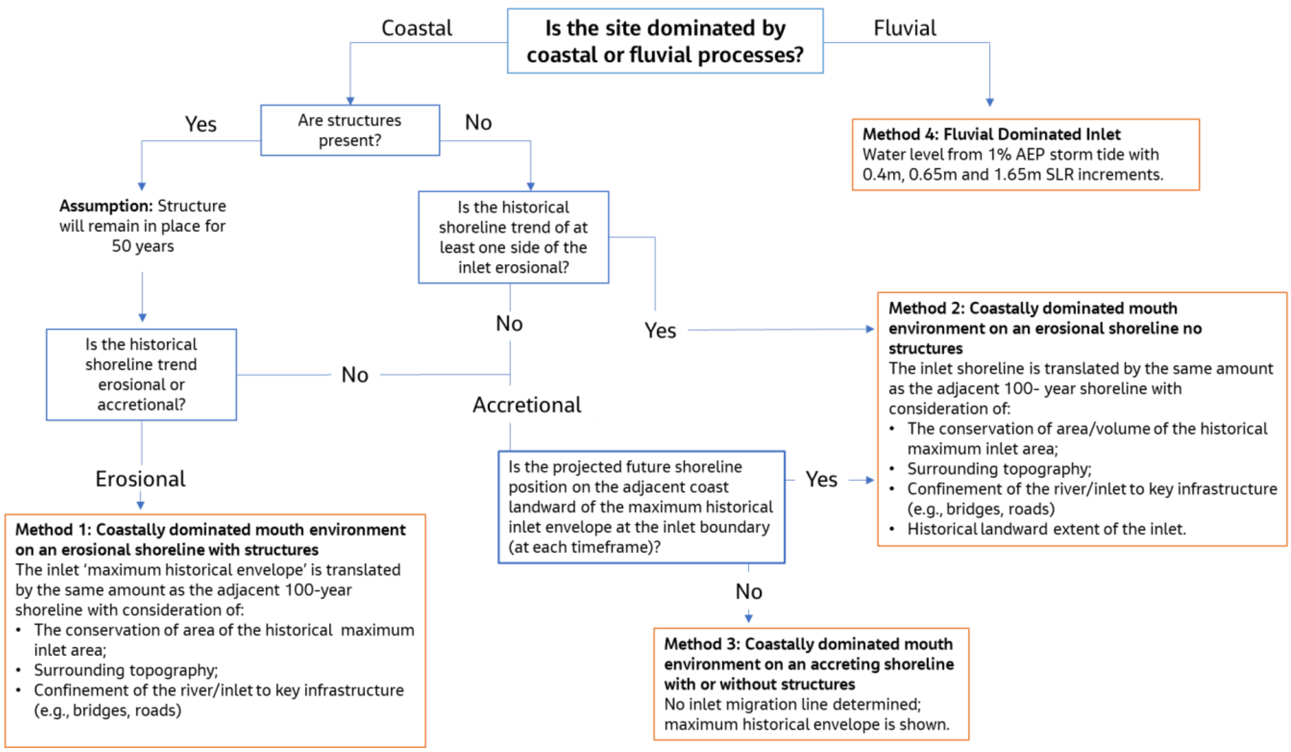

Figure 6.10: Decision tree for applying appropriate method to each river/stream mouth environment.

Table 6.5: Key characteristics of each hydrosystem and the primary method adopted from the decision tree approach used in the assessment. Note that by 2120, some streams which initially used method 3 had to use method 2 on one side of the inlet as the PFSP line was landward of the historical inlet extent.

	Dominating processes		Structures Present		Adjacent Shoreline Historical trend		Primary Method
Hydrosystem	Fluvial	Coastal	South	North	South	North	
Wainui Stream		✓			Accretion	Erosion	Method 2
Whareroa Stream		✓	✓		Erosion	Erosion	Method 1
Wharemauku Stream		✓	✓	✓	Erosion	Erosion	Method 1
Tikotu Stream		✓	✓	✓	Accretion	Accretion	Method 3
Waikanae Estuary	✓		✓	✓	Accretion	Accretion	Method 4
Waimeha Stream		✓			Accretion	Accretion	Method 3
Te Kowhai Stream		✓			Accretion	Accretion	Method 3
Mangaone Stream		✓			Accretion	Accretion	Method 3
Ōtaki River	✓			✓	Accretion	Accretion	Method 4
Waitohu Stream		✓			Accretion	Accretion	Method 3

6.7.1 Method One – Coastally dominated mouth environment on an erosional shoreline with structures

Method one is used when mouth environments have more coastal influence than fluvial influence to their morphology. In these inlets, structures are currently present generally in the form of training walls which are restricting migration of the inlet. An assumption is made that all river mouth training structures will be maintained for at least 30 years based on current consents and asset management by KCDC, and over this timeframe there is not expected to be any movement of the inlet in relation to the structure. The historical shoreline trends at these inlets are generally erosional, and therefore over the first 50-year period, the shoreline will continue to erode, weakening the river mouth training structures.

At the 100-year timeframe, the position of the maximum historical envelope is translated landward by the same amount as the adjacent shoreline, and general shape of the mouth inlet is maintained. Consideration of the topography of the land is included in the translation of inlet position, with the inlet being confined to areas generally <10 m of elevation over a 100-year period where appropriate. The landward extent of the zone is blended into the adjacent future shoreline position.

For mouths with training structures on one bank, Method two (Section 6.7.2) is applied to the un-controlled, eroding bank.

6.7.2 Method Two – Coastally dominated mouth environment on an erosional shoreline with no structures

Method two is used when no structures are present within the mouth environment, and the shoreline both within and adjacent to the inlet is historically erosional and expected to continue to erode with SLR. At each timeframe and scenario, the position of the maximum historical envelope is translated landward by the same amount as the adjacent shoreline, and general shape of the inlet is maintained. Consideration of the topography of the land is included in the translation of inlet position, with the inlet being confined to areas generally <6 m of elevation over a 30 to 50-year period, and <10 m of elevation over a 100-year period where appropriate. The landward extent of the zone is blended into the adjacent future shoreline position.

6.7.3 Method Three – Coastally dominated mouth environment on an accreting shoreline with or without structures

Method three is used when there is a historically accreting shoreline, with or without structures, which is unlikely to be overturned by SLR in the future. From a coastal processes perspective, when coastal erosion along the adjacent coast is unlikely to occur in the future with SLR, there is no reason to suggest that the mouth environment would migrate landward. An exception to this could be the occurrence of large flood events that could cause scouring and erosion around the outlet that exceeds the historic recorded envelope of change. It is recognised that this is possible with climate change induced rainfall combined with SLR. However, due to the predominant coastal influence on these small inlets, the historical maximum landward extent has not been approached in contemporary times, and with continued shoreline accretion is considered to be an appropriate approximation of future conditions under the most extreme conditions.

This is considered to be a conservative approach, as there is no coastal process reason that with continued accretion, the inlet would permanently migrate landward of the current inlet extent. For any structures that exist within these inlets, it is assumed that they will be maintained for at least 30 years.

However, for the 2120 timeframe at Mangaone Stream, Te Kowhai Stream, Waimeha Stream, and Waitohu Stream, the effect of RSLR results in the PFSP being located landward historical maximum landward extent of the mouth environment, resulting in method two being adopted for this timeframe.

6.7.4 Method Four – Fluvial Dominated Hydrosystem

Method four was used to assess the effect of SLR on fluvial dominated coastal hydrosystems (e.g. Ōtaki and Waikanae). For this method, we use the results of the inundation assessment by applying the 10% AEP river flow combined with 1% AEP storm tide for the following two RSLR scenarios as used in the inundation assessment:

- A 'medium' RSLR of 0.65 m; being approximately the upper projection of rise by 2070 (RCP 8.5+) and the lower projection of rise by 2120 (RCP2.6).
- A 'high' RSLR of 1.65 m; being approximately the upper projection of rise by 2120 (RCP8.5+).

This approach is based on the principle that for these large river mouth environments, the morphology is shaped by both fluvial and extreme coastal events. It is assumed inundation from this combination of events would produce vegetation die back, and shape the morphology of the mouth environment.

6.7.5 Assumptions

The following assumptions have been made in the development of the river/stream mouth migration zones:

- There is no consideration for large anomaly events taking place over the next 100-year period (e.g. earthquakes, land clearance) which could increase the sediment input to the inlet.
- Historical shoreline trends will continue into the future.
- Fluvial inputs into the coastally dominated inlets will not change.
- Coastally dominated mouth environments will generally try to maintain their shape and area over a long time period.
- Erosion rates will decrease as the land elevation increases due to there being more material to be removed by the erosion processes. Topography will have an influence on where the inlet migrates to and how the river position could change.

6.8 Mapping Outputs

A scale of 1:7500 will be used for the mapping outputs in the Volume 2 report. For each map, where the erosion susceptibility is present at the specified timeframe, a 'most likely' band (P33-P66) is shown, as well as the potential shoreline position zone beyond which there is only a 10% probability that the shoreline will be located within the specified timeframe (e.g. current shoreline to P90 position).

As presented in Table 3.3, the following bands and zones are mapped for the following scenarios at each timeframe:

- 0.2 m SLR 2050 (Most likely band and potential shoreline zone up to P90)
- 0.4 m SLR 2050 (Most likely band and potential shoreline zone up to P90)
- 0.3 m SLR 2070 (Most likely band and potential shoreline zone up to P90)
- 0.7 m SLR 2070 (Most likely band and potential shoreline zone up to P90)
- 0.65 m SLR 2120 (Most likely band and potential shoreline zone up to P90)
- 1 m SLR 2120 (Most likely band only)
- 1.25 m SLR 2120 (Most likely band only)

- 1.65 m (Most likely band and potential shoreline zone up to P90)

An example of how the most likely band and potential shoreline zone is presented is shown below in Figure 6.11. For this mapping product, only the 'hazard' has been mapped, so that where accretion was projected to occur over a specified time frame (i.e. if the long term accretion rate is higher than the effect of SLR) then only the 'present day' hazard (short term and dune stability) has been mapped.

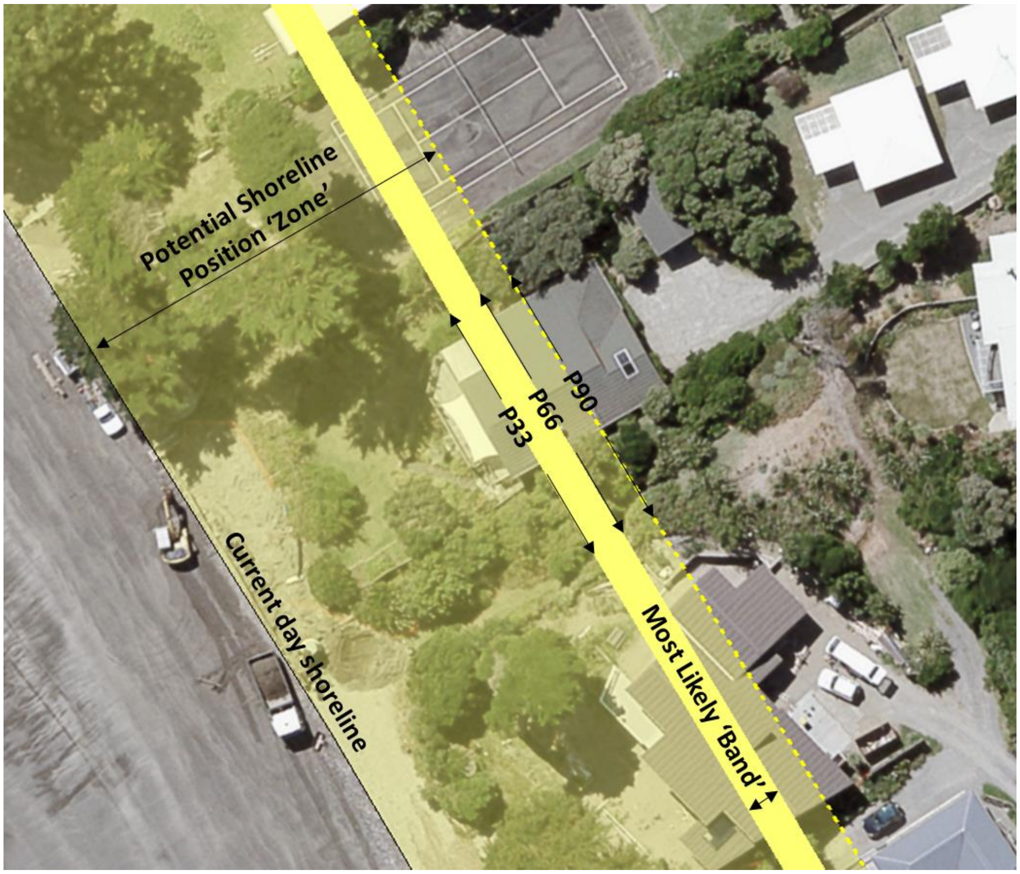

Figure 6.11: Example mapping output detailing how each probabilistic output is included in the map.

The projected shoreline position is calculated from a 'smoothed' present day reference shoreline, which for mapping purposes has had small changes in the orientation of the shoreline smoothed out (e.g. corners of walls, sand dune blow outs). Once the probabilistic outputs were plotted, these were reviewed to determine where there were any discontinuities along the future shoreline, and further investigation was undertaken to determine why these discontinuities had occurred. These discontinuities generally occurred at the cell boundaries, in association with a change in discrete profile inputs being used for calculations.

An example of this is where the bathymetry downdrift of the Paraparaumu cuspate foreland (discussed in Section 6.4.3) has an effect on the amount of erosion due to SLR between the Raumati and Paraparaumu cell, resulting in a large discontinuity of the projected erosion at the boundary between these cells. In this case we have assumed that there is a linear change between the two input bathymetric profile which influences the closure slope (as the closure depth and beach height parameter inputs are all similar). Therefore, the effect of the SLR is also linearly reduced in a northward direction across two transects on either side of the boundary between these cells. In doing so, this altered the erosion distances produced by the probabilistic outputs, however, it has allowed for important and overriding plan shape considerations of the shoreline which are consistent with the coastal processes operating.

6.9 Coastal Erosion Assessment Limitations

It is recognised that there are a number of limitations in the methods used to assess future coastal erosion, some of which are highlighted in the above sections on the erosion components. The following discussion consolidates all of the limitations encountered and resulting assumptions make in this assessment to overcome these limitations.

6.9.1 Decision making for inclusion of structures in assessment

There are limitations around our knowledge and ability to predict the future management of coastal protection structures for inclusion in both the effect of SLR and extrapolation of the long term trend into the future. As a result the following assumptions were made:

- Coastal protection structures assumed residual life based on maximum residual life from Tonkin and Taylor (2016 & 2021) databases (see Section 6.1.1).
- Rivermouth training wall structures assumed to have a residual life of 30 years based on discussion with KCDC.
- At the end of the residual life the structure would fail and not form any protection.

6.9.2 Probabilistic Approach

Generally, a probabilistic approach is used to overcome uncertainties and limitations in the data inputs required for the erosion calculations. However, the following limitations are still noted under this approach.

- There are several limitations around data availability (e.g. beach profiles) which mean that there is a limited range of temporal variability in the data. This results in outputs of the probabilistic approach appearing to be more certain, however this is only due to the limited range of input data.
- Quasi-Distributions have had to be applied where there is only one source of data. The use of the quasi-distribution and method to make up the distribution is justified in their respective sections. This is applicable for:
    - Short term erosion factor where observations from the September 1976 storm is used as the basis of the distribution.
    - Long term historical shoreline change where structures are present, and the CSL (2008) "earlier rates" are used.
- Due to the limited datasets, triangular distributions of minimum, mean and maximum value are used as opposed to normal distributions due a reliable standard deviation not being able to be determined from the small datasets.

6.9.3 Extrapolation of Long-Term Shoreline Movements

Two different methods were used for the extrapolation of the long term rate, depending on the occurrence of structures along the coastline. The following limitations of the methods used to calculate the long term rate include:

- The most recent aerial imagery available is from 2017, and therefore this has been used as the reference shoreline for this assessment. This does not include recent changes to shoreline structures along the coast, or more recent erosion since 2017.
- The long term trend is derived from approximately eight time-stamps across the 70-year period.

- Manual georeferencing of historical imagery was required for this assessment. This is difficult over older time periods (pre-1978) in un-developed areas due to no stable reference points being available. This results in inaccuracies in the georeferencing and can have a flow on effect into the calculation of the long term rate. We expect that this error in earlier images could be in the order of +/- 5 m.
- The accuracy of digitization of the shoreline is dependent on the quality of the aerial imagery, and the accuracy of the georeferencing. The resolution, shadowing, and light exposure are issues in earlier images which make it difficult to identify shoreline features. In areas where the coastal feature was unidentifiable, a shoreline was not produced for that time period.
- The use of linear regression rate is not always applicable at shorelines which have long term fluctuations of shoreline trends and have a low R^2 value. This limitation was minimized by using the End Point Rate (EPR) where appropriate (e.g. Te Horo) or removing early historical shorelines to establish more stable long term trends that were representative of the current day, and removed any false shoreline advancement. This is detailed in Section 6.3.
- Where the EPR was used, an assumed triangular distribution (e.g. quasi probability distribution) with upper and lower bounds of +/- 50% of the EPR was applied.
- Where a structure is present, a 'pre-structure' rate needed to be used which is representative of what the shoreline trend was prior to structures being built. These rates were obtained from the CSL (2008 & 2012) assessment, where an assessment of shoreline change pre-structures was undertaken at 1 km intervals for the entire Kāpiti Coast District shoreline using pre-structure aerial imagery (1954), and cadastral surveys from the 1870's. This is referred to in CSL (2008) as the "earlier rate". CSL acknowledges the limitations of the calculation of this earlier rate, as cadastral surveys and aerial imagery use different shoreline indicators (e.g. the high water line and the vegetation line respectively), which introduces an unresolvable systematic error when combining the two data sources, and can result in an over-estimation of shoreline erosion. When applying this rate, a +/- 50% upper and lower bound was applied to the "earlier rate" to develop the triangular quasi-probability distribution at these transects.
- Assumptions needed to be made around any increase in sediment budget into the future. We have assumed for this assessment that going forward there will be no change to the sediment budget, and current trends will continue into the future.

6.9.4 Effect of future accelerated sea level rise

The following limitations and assumptions were made for calculating the effect of future accelerated sea level rise:

- The standard "Bruun Rule" approach is widely used in the international literature and is recommended in MfE (2017), it is also widely criticized for its limitations. It assumes only two-dimensional cross-shore sediment movements hence does not include consideration of longshore sediment transport inputs/losses or plan shape controls (e.g. headlands).
- While some sensitivity testing was undertaken, this approach does not take into consideration elevation changes in the backshore over a temporal scale as the shoreline retreats. As elevation increases in the backshore, there is more volume of material to erode and hence the erosion rate may slow, and therefore could be over-conservative in some locations, as noted above.
- This approach also does not take into account changes in the material in the backshore, which could also slow or speed up erosion rates. This is not particularly relevant for Kāpiti Coast as most backshore environments are either peat (Gibb, 1978) or old coastal sand dune plains, however will have some effect where there is infrastructure in the backshore.

- This model is only applicable to equilibrium beach profiles, hence the resulting profile will only be reached at some time after SLR has reached the stated level, therefore results are considered to be very conservative for the time frames and magnitudes of SLR used in the calculations.
- The effect of SLR is largely dependent on the total beach slope out to the closure depth, where steeper slopes require less sediment volume to raise the nearshore bed, and therefore less erosion distance is predicted. The nearshore profile south of the cuspate foreland differs to the typical equilibrium sand beach profile. This is due to the widely flatted nearshore caused by the downdrift sand bank from the cuspate foreland, which then drops steeply into the Rauoterangi channel. As a result, inner closure depths tended to give larger SLR erosion response than outer closure depths due to the flatter slopes in the nearshore for the inner Hallermeiers limits, compared to the steep slopes out to the resulting outer Hallermeiers limit.
- There were several limitations around the availability of data, including:
  - There was no time series wave data available at the time of this assessment, and in calculating closure depth wave height and wave period for input in the calculations were taken as the 1-year return period value for the closest site presented in the MetOceans (2007) assessment. However, this is considered to be a good proxy for the 12 hour per year exceedance values used by Hallermeiers (1981).
  - The bathymetry data is very granular, resulting in large changes in the predicted future erosion due to RSLR between the bathymetric profiles at Paraparaumu and Raumati, which result in large discontinuity in the projected erosion distances between these cells, which are not justified from the coastal processes and shoreline plan shore perspective.
  - The most recent bathymetry data available for this assessment was from Lumsden (2003). There could have been small changes to the bathymetry over the past 17 years.
  - There is a short and inconsistent record of beach profile surveying, which results in small variability in beach heights and widths which are then inputted into the probability distribution.
- There is a limitation around the uncertainty of SLR and when different amounts of SLR will occur. This is addressed through the use of a range of SLR magnitudes that encompasses all SLR projections for each time period.
- For the composite beach approach, there was no sediment sampling data available from the beach profile sites used. Observations of the sediment size distribution averaged across the whole profile were noted on the site visit which formed the basis of quasi-distributions of the sand/gravel ratio, along with consideration of the longshore distance from the gravel source (Ōtaki River). The observations from the site visit indicated that the presence of gravel in the beach profile stopped approximately 5 km south of the Ōtaki River mouth.
- Todd and MacDonald (2020) noted that the composite beach modification better accounts for the cross-shore sediment transport loses of sand from the beach profile with SLR, as well as retention of gravels on the upper beach/berms. However, they also recognised that this method raises a contradiction between the Bruun Rule assumption of conservation of equilibrium nearshore depth by beach erosion volume, and less beach erosion volume with offshore transport of the gravels from the upper berm/crest region being limited. In the long-term, this differential rate of loss of sand and gravel components could result in the beach converting to a more mixed sand and gravel form unless abrasion of the gravel component keeps pace with the offshore sand losses due to SLR.

6.9.5 Short term Erosion

Due to other methods used to assess the short term erosion hazard not producing realistic results for the Kāpiti Coast shoreline, the only relevant data which could be included in the assessment was observations from the September 1976 storm event. A quasi-distribution has to be used to incorporate this data into the assessment. This took the mean upper limit of the short-term observation as the ‘mean’ for the quasi distribution, and determined a minimum and maximum as ±50% of the mean value. The taking of the existing upper magnitude of storm retreat observations as the mean value could be considered to be a conservative approach, however this is counteracted by the increased frequency of this magnitude event in the future due to SLR.

Sensitivity testing of these observed short-term erosion distances was undertaken using the SBEACH model for unmodified sites (Appendix B). However, it is noted that there are limitations with model input data for the September 1976 storm event (e.g. lack of water level and wave time series).

6.9.6 **Dune Stability**

There was no available data on the consolidation or 'wetness' of sand along different sections of the Kāpiti Coast shoreline, and therefore assumptions were made around the angle of repose. A range of 30-34 degrees was incorporated into the probability distribution which was representative of a range of angle of reposes for dry sand of varying sizes. It was assumed that sand is dry when it fails.

6.9.7 **Hydrosystems**

There are a significant number of limitations on the determination of future coastal hydrosystem positions due to the fact that this is a relatively under-developed field of research. There is currently limited understanding of how these coastal hydrosystems will respond in the future due to the complex influences which determine their morphology. Due to the limited understanding of these systems and data available, the following assumptions had to be made in determining where the future inlet position might be:

- There is no consideration for large anomaly events taking place over the next 100-year period (e.g. earthquakes, land clearance) which could increase the sediment input to the lower catchment and mouth/estuary environments.
- Historical shoreline trends will continue into the future.
- Fluvial inputs into the coastally dominated hydrosystems will not change.
- Coastally dominated hydrosystems will generally try to maintain their shape and area over a long time period.
- Erosion rates will decrease as the land elevation increases due to there being more material to be removed by the erosion processes. Topography will have an influence on where the mouth environment migrates to and how the river position could change.

7. Coastal Inundation Methodology

7.1 Overview

The dune ridges along the Kāpiti Coast District shoreline are generally higher than current estimates of extreme sea levels including allowances for SLR over the next 100 years. Therefore, depending on shoreline erosion characteristics, the dunes will continue to provide a barrier to direct inundation by the sea along the majority of the coast. For example, the indicative "bathtub area" potentially under the influence of storm tide and SLR over a 100-year timeframe (i.e. land below 3.96 m WVD-53 based on 1 % AEP storm tide and RSLR in 2120 at Waikanae as presented in Table 4.2) is shown in Figure 7.1, with the potentially susceptible area including only a narrow strip along the beaches in front of the dune system. However, the map also shows two other areas potentially susceptible to coastally driven flooding, being (1) the lower reaches of the rivers and creeks which flow into the sea through openings in the dune ridges where the extent and magnitude of flooding in fluvial events could be exacerbated by high sea levels, and (2) low-lying areas that are not connected to the sea by overland pathways and therefore would not be inundated directly by surface flow but may be connected by the stormwater drainage network which would provide a pathway for flooding.

The past coastal inundation assessments summarised in Section 5.2 are limited to the modelling of impacts of extreme storm tides (1% AEP) and RSLR, without consideration of the combined impacts of these coastal conditions on flooding from heavy rain (pluvial flooding) and high flow in the stormwater network, streams and rivers (fluvial flooding), which is a widespread natural hazard in the Kāpiti Coast district.

Areas susceptible to these fluvial or pluvial flood hazards have been mapped in detail by KCDC and GWRC for the 1% AEP using flood models as shown in Figure 7.2. While the existing models do include an allowance for coincident storm tide (20% AEP) and the effects of climate change (RSLR of 0.8 m and 17% increase in extreme rainfall depth), the mapping outputs do not allow the analysis of change in the flood susceptibility (e.g. extent and magnitude) from different storm tide, sea level, and river flow scenarios, or the interpretation of the impact of SLR on the flood prone areas. New flood models are currently being prepared by both KCDC and GWRC. The models will be used to produce updated flood hazard maps for the District Plan. The new models can also be used to provide a detailed multi-hazard assessment of the combined effects of coincident coastal storm events and RSLR on fluvial and pluvial flood hazards. However, these models will not be available within the programmed time for completing this coastal hazard assessment.

Therefore, the main objective of the inundation assessment in this project is to provide an initial understanding of the areas most likely to be affected by coastal flooding from storm tides, and how this will change under RSLR to the consistent levels as used ion the erosion assessment. This will be done using current available data and a simpler "bathtub" approach to provide an interim assessment ahead of more detailed models and outputs becoming available. However, the outputs of the current assessment will allow coastal inundation hazards to be considered alongside the erosion hazards in developing coastal hazard adaptation pathways under the *Takutai Kāpiti* project. Where needed, the development and assessment of these pathways can be further informed by more detailed modelling using the new flood models as they become available during the course of the project.

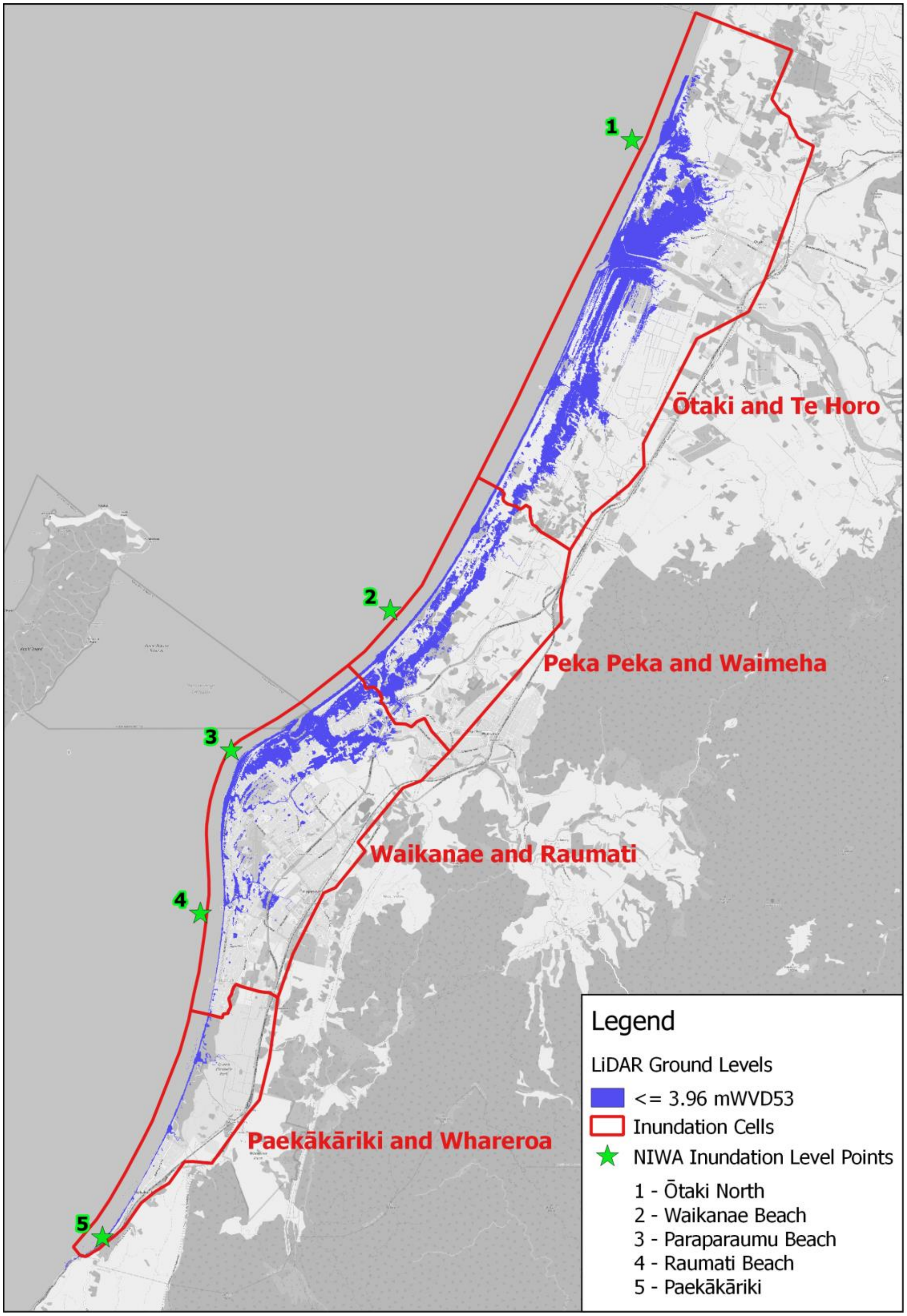

Figure 7.1: Areas of land potentially susceptible to coastal inundation hazard over a 100-year period (i.e. ground level lower than 3.96 m WVD-53), the coastal inundation cells considered in this project and the location of points at which sea levels for inundation assessment were previously derived by NIWA (Lane et al, 2012).

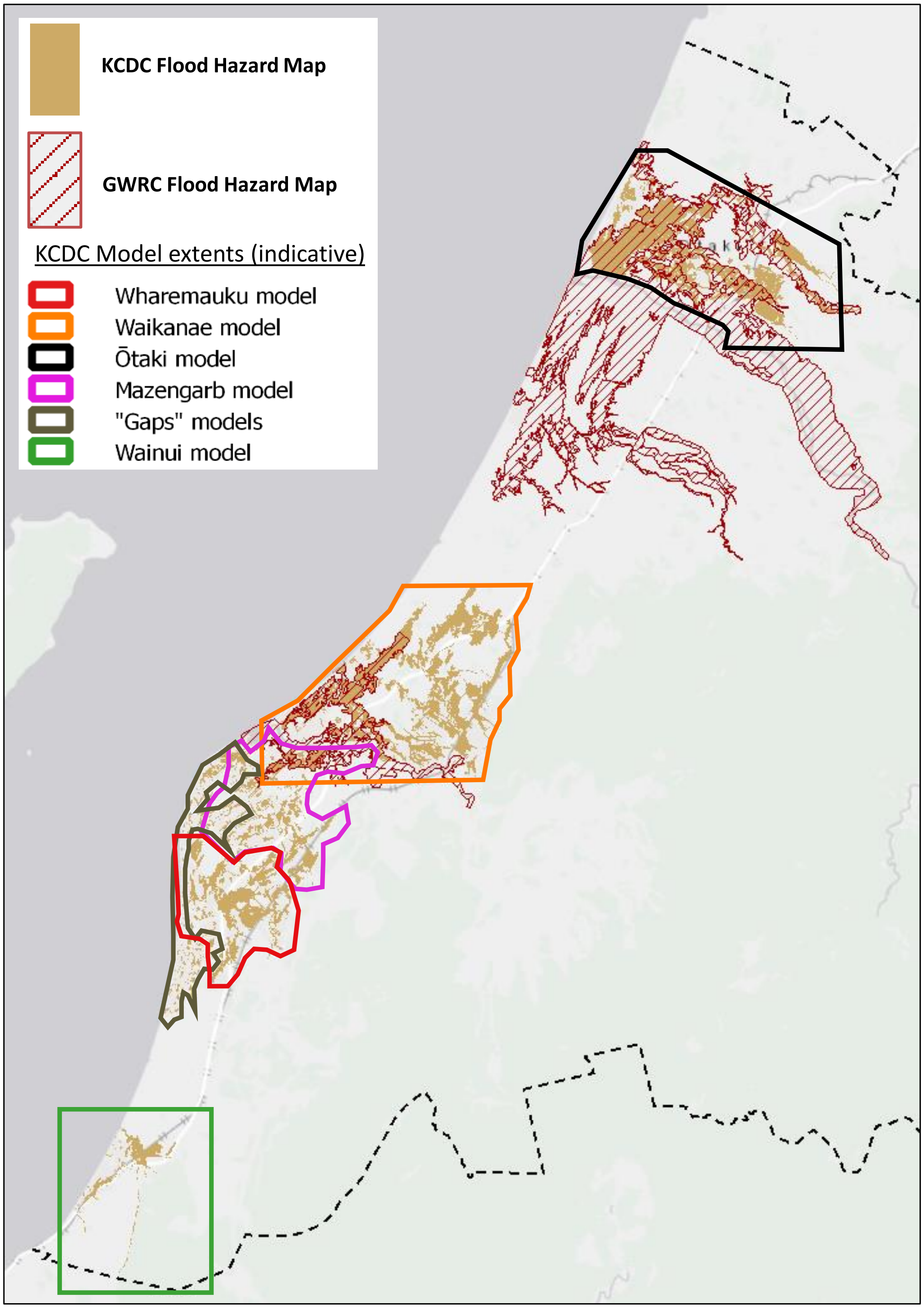

Figure 7.2: Current KCDC and GWRC flood hazard maps showing areas at risk of fluvial and pluvial flooding for 1% Annual Exceedance Probability and the approximate extents of the current KCDC stormwater models.

7.2 Approach

Our approach in this assessment is to make use of existing data to provide an initial assessment of the area most likely to be affected by flooding from coastal storms for a range of potential RSLR scenarios that are consistent with those used in the coastal erosion assessment. This is required as the existing flood hazard model assessments all have limitations in how far they address the needs of this *Takutai Kāpiti* project. In order to provide an initial assessment of coastal flood hazard that overcomes some of these limitations, we have used a simpler "bathtub" method to produce new coastal flood maps. However, as noted above the new flood models being developed by KCDC are expected to become available during the course of the project, which will allow a more detailed simulation of the combined flooding from coastal storms, pluvial and fluvial events and can be used to further support the development of adaptation pathways as required.

A summary of the factors included in the existing flood and storm surge assessments compared to the "bathtub" method to be used in this assessment is presented in Table 7.1.

Table 7.1: Factors included in existing flooding assessments compared to this current assessment.

Factors to be considered in the project inundation assessment	How the current maps consider these factors			How the Takutai Kāpiti bathtub method considers these factors	
	KCDC flood hazard maps	GWRC flood maps	GWRC storm surge maps		
Coverage of the full coastline	X	X	X	✓	Covers the whole coastline, using available ground level data and existing estimates of inundation water levels
A range of sea level rise values	X (only 0.8 m)	X (only 0.8 m)	✓ (four scenarios: 0 m, 0.5 m, 1.0 m, 1.5 m	✓	Consider 0 m, 0.40 m, 0.65 m and 1.65 m based on max RCP8.5+ scenarios for2050, 2070 and 2120
Extreme storm tide level	✓ (5% AEP)	✓ (5% AEP)	✓ (1% AEP)	✓	1% AEP static storm tide level including wave setup
Different types of inundation pathway	Do not include all rivers	Do not include SW network	Do not include SW network or smaller streams	✓	Conservative - considers all land potentially at risk (below sea level) including those without obvious direct pathways
Fluvial/pluvial contribution to flooding	✓	✓	X	X	Not considered. Where this effect is important it can be assessed using new flood models which are under development

7.2.1 The "Bathtub" method

In this method, a digital elevation model (DEM) of the ground surface is used to map all areas of ground which lie below a given sea level. For our assessment we have used ground level data from the 2017 KCDC LiDAR aerial survey. The sea levels considered in the method are defined by the storm tide level, including wave setup, and allowances for future rises in the mean level of the sea.

This method maps all land which is potentially vulnerable to inundation by the sea, however the method does not explicitly take account of inundation pathways. The resulting maps show all land below the sea level considered, including areas which are not currently directly connected to the sea by overland flow paths. In some cases, these areas are connected to the sea by the stormwater drainage network, including drainage pipes below the ground as well as open channels, or by streams and rivers which flow into the sea. In other cases, pathways may not currently exist, but these areas could become vulnerable to coastal flooding if developed due to the effect of the sea level on the outfalls of drainage systems for new development.

Figure 7.3 shows a conceptual cross-section through the coastline to illustrate how the bathtub method is applied to identify all areas below sea level and how this relates to an example inundation pathway through the stormwater network.

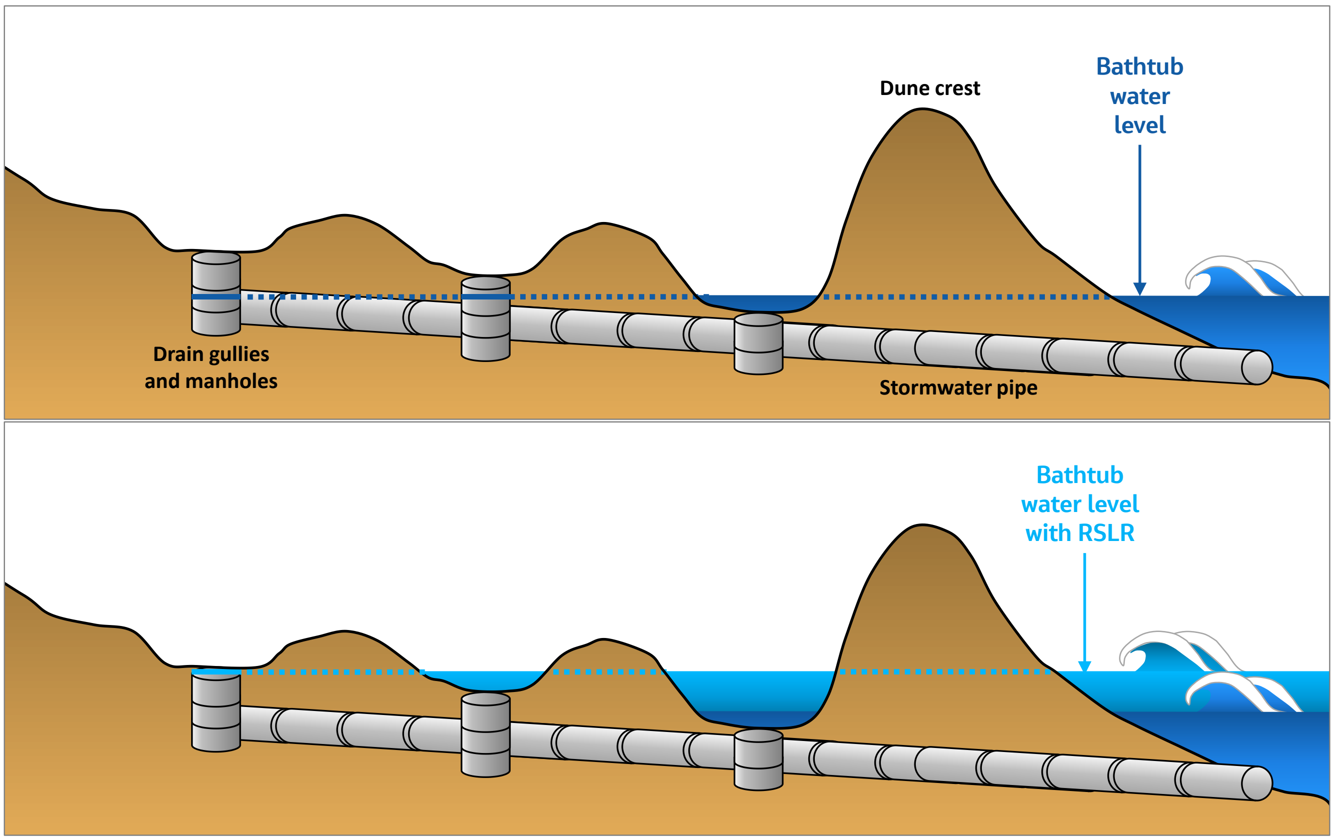

Figure 7.3: Conceptual cross-section through the coastline to illustrate the application of the bathtub method – without RSLR (upper image) and with RSLR (lower image). In this example, low lying land protected from the sea by the high dunes along the coastline is vulnerable to flooding from the sea through the stormwater drains. In other cases the inundation pathway could be a stream or river.

The effect of extreme sea level on coincident rainfall and high river flows cannot be considered explicitly in the bathtub method. Elevated water level in the sea, streams or rivers due to coastal storms can increase pluvial and fluvial flooding if they coincide. Figure 7.4 shows conceptually how the sea levels considered in the bathtub method could affect flooding from the stormwater network if they coincide with heavy rain.

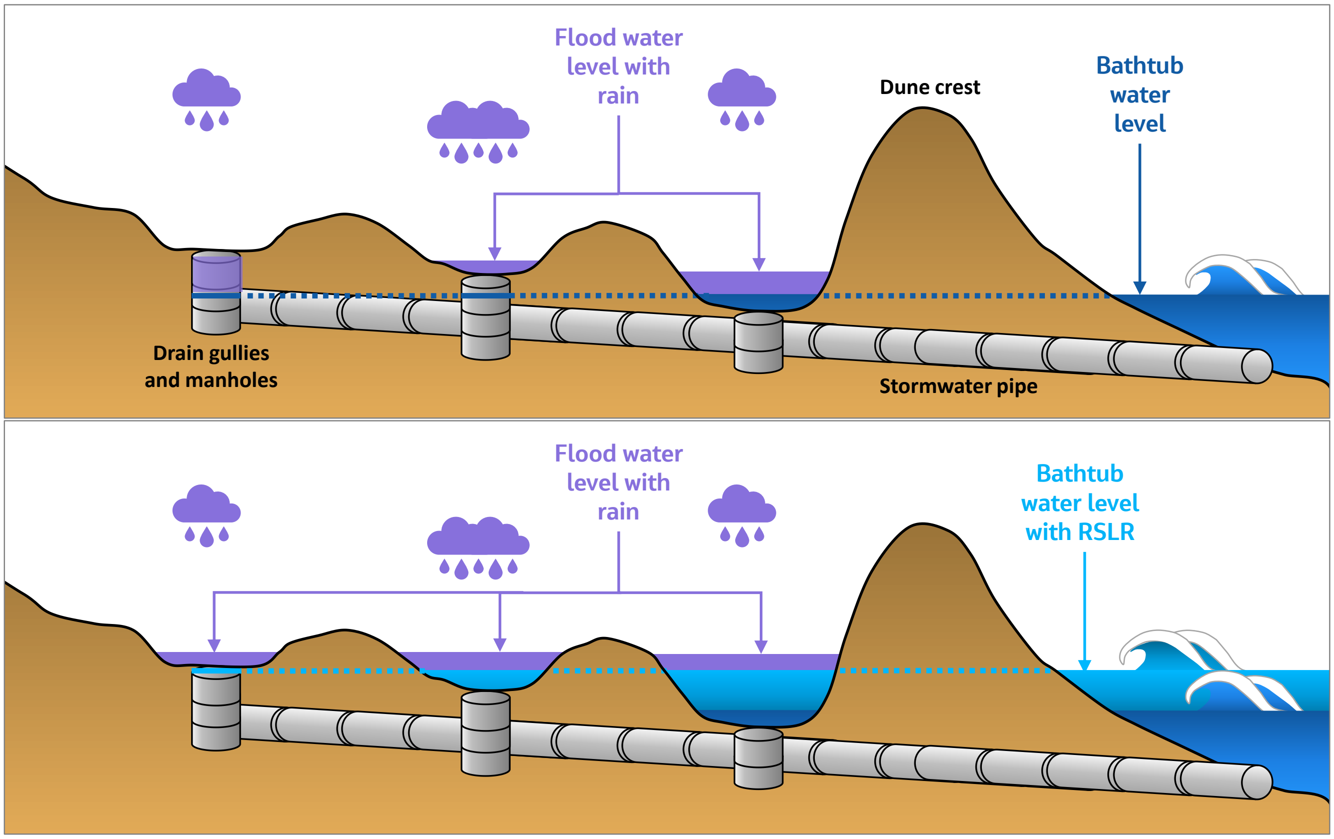

Figure 7.4: Conceptual cross-section through the coastline illustrating the potential combined effects of coincident high sea level and a rainfall event compared to the bathtub method – without RSLR (upper image) and with RSLR (lower image).

7.2.2 Comparison of bathtub method and hydrodynamic modelling

The main relative benefits and limitations of the bathtub method compared to more complex hydrodynamic modelling of flooding are summarised in Table 7.2.

Table 7.2: Factors included in existing flooding assessments compared to this current assessment.

	Benefits	Limitations
Bathtub	- Identifies all land potentially at risk from coastal inundation. - Includes an indication of areas that could become at risk if developed.	- Conservative, does not take account of volume available for inundation or capacity of the pathways which can both influence the inundation extent. - Does not identify areas where pluvial/fluvial flooding is increased due to coastal hazard.
Hydrodynamic model	- Can be used to assess combined hazard from multiple sources (coastal, fluvial, pluvial). - Takes account of volume of inundation and flow capacity of inundation pathways.	- Models will usually have a threshold level of detail and will not include all flow pathways (e.g. smaller drains and pipes) so may exclude some areas potentially at risk of inundation. - Models will usually represent the "current" state of development and may not indicate areas at risk if developed in the future.

Although the bathtub method cannot provide an assessment of the combined effect of multiple coincident sources of flood hazard, the mapping provides a complete initial assessment of the land potentially at risk of coastal inundation over a range of future sea level rise scenarios. The new flood models developed by KCDC will then allow further investigation of combined flooding, where needed, in the *Takutai Kāpiti* project.

7.2.3 Groundwater

For each inundation cell we have considered the potential for elevated groundwater levels to pond or contribute to inundation in areas beyond those mapped as at risk from coastal inundation by the bathtub method. We have used the available previous estimates of the groundwater surface for future climate change scenarios (SKM, 2012), which include the effect of SLR.

In areas where the 2012 study outputs show the groundwater level is estimated to be above ground level and the ground level is below mean sea level (including any RSLR allowance), gravity drainage of groundwater will be impeded and the hazard from groundwater ponding is more likely. These areas are included in the bathtub mapping of land below storm tide level (including RSLR) and have not been mapped separately.

In areas of higher ground (above MSL including RSLR) where groundwater levels are estimated to exceed ground level, drainage will tend to limit the depth of surface water resulting from groundwater breakout. Deeper surface water ponding may occur locally in undrained areas of higher ground, such as groundwater dependent wetlands, but the 2012 study water levels are not sufficiently accurate to map the corresponding inundation extents.

Although the depth of inundation from elevated groundwater outside the coastal inundation zone may generally be limited by drainage networks, both natural and constructed, high groundwater levels pose other hazards to infrastructure and people. However, detailed consideration of these hazards is outside the scope of this coastal hazard assessment.

7.3 Scenarios

For our inundation assessment we have applied a combination of sea levels and storm tide into the four representative scenarios set out in Table 7.3, of which the components are detailed below in 7.3.1 to 7.3.3.

Table 7.3: Representative Inundation scenarios.

Mean sea level	Extreme Sea Level[(a)]	Groundwater scenario from SKM (2012) considered
2020	1% AEP	Not considered
2020 + “Low” RSLR of 0.40 m	1% AEP	Not considered
2020 + “Medium” RSLR of 0.65 m	1% AEP	Scenario 1
2020 + “High” RSLR of 1.65 m	1% AEP	Scenario 4
(a) storm tide levels include wave setup allowance		

7.3.1 Extreme sea levels

We have adopted the 1% AEP storm tide levels at shore, including wave setup, derived from Lane et al (2012) at the locations along the Kāpiti coast indicated in Figure 7.1 and as presented in Table 4.1. The levels in Table 4.1 also include adjustment of the MSL to 2020 levels. The resulting levels range from 2.43 m-WVD53 for Ōtaki to 2.14 m-WVD53 for Paraparaumu to Paekākāriki. The wave setup included in these estimates was derived for the open coast, whereas the primary inundation pathways along the Kāpiti coast are through river mouths where wave setup tends to be less, therefore these estimates are likely to be somewhat conservative.

To test the relative sensitivity of extreme sea levels to storm tides for AEP's less than 1%, we have used the joint probability analysis of storm tides and wave heights from Lane et al (2012) to estimate the 10% AEP storm tide levels for two relevant sites on the Kāpiti Coast District coast (i.e. between Waikanae Beach and Paraparaumu Beach (Site 3) and south of Paekākāriki (Site 2)). For wave set-up calculation, we have applied the findings of Tanaka and Tinh (2008) for wave setup in similar river mouths to estimate the upper bound wave setup (i.e. 14% of wave height) for input into the total storm tide level for each of the 10% AEP pairings of storm tide and wave height for Site 3, which are presented in Table 7.4.

Table 7.4: Estimated10% AEP extreme sea levels at shore for Site 3 (from joint probability analysis, Lane et al, 2012). (maximum level indicated in bold). WVD53 is referenced to 2005-2011 mean sea level.

Storm tide (m WVD53 2005-11)	Significant wave height (m)	Estimated wave setup (m)	Estimated total storm tide (m WVD53 2005-11)
0.00	5.51	0.77	0.77
0.40	5.51	0.77	1.17
0.49	5.51	0.77	1.26
0.59	5.49	0.77	1.36
0.84	5.39	0.75	1.59
0.99	5.12	0.72	1.71
1.19	4.60	0.64	1.83
1.22	4.48	0.63	**1.85**
1.38	3.28	0.46	1.84
1.40	2.98	0.42	1.82
1.54	0.00	0.00	1.54

The maximum level estimated for the 10% AEP storm tide is 1.85 m-WVD53 referenced to 2005-11 MSL or 1.90 m-WVD53 referenced to 2020 MSL. The corresponding 1% AEP storm tide level for Paraparaumu Beach is 2.09 m-WVD53 referenced to 2005-11 MSL or 2.14 m-WVD53 referenced to 2020 MSL. The difference between the estimated levels for the 10% and 1% AEP storm tides at this location is therefore approximately 0.24 m or around 15% of the overall range in RSLR we have considered for our assessment (1.65 m). The two main components of storm tide (surge and wave setup) are both limited by physical factors and the increase in storm tide for AEPs smaller than 1% is likely to be less than the difference between the 10% and 1% AEP levels.

Therefore in our assessment we have only used the 1% AEP storm tide levels because:

(1) flood hazards are defined in the District Plan Maps by the 1% AEP event;

(2) estimates of the 1% AEP storm tide level are available from previous detailed tide and wave modelling; and

(3) the difference between the 1% AEP storm tide level and the level for other AEPs is small in comparison to the differences in projected RSLR for the scenarios we are considering.

7.3.2 Relative sea level rise

Section 3.4 (Table 3.2) sets out the magnitudes of RSLR, including allowance for vertical land movements, under different RCP scenarios for various time frames up to 2150. As set out in Table 3.3, since the inundation assessment is less sensitive to the timing of the SLR than the erosion assessment, the representative scenarios used in this assessment is limited to the following three magnitudes of RSLR, which cover time frames of 2050, 2070 and 2120:

- A "low" RSLR of 0.40 m; being approximately the upper projection of rise by 2050 (RCP 8.5+) and the lower projection of rise by 2070 (RCP2.6).
- A "medium" RSLR of 0.65 m; being approximately the upper projection of rise by 2070 (RCP 8.5+) and the lower projection of rise by 2120 (RCP2.6).
- A 'high' RSLR of 1.65 m; being approximately the upper projection of rise by 2120 (RCP8.5+).

7.3.3 Groundwater levels

The available data for the effect of climate change on groundwater levels (SKM, 2012) considers four scenarios:

- Scenario 1: 0.5 m SLR and 17% increase in winter rainfall.
- Scenario 2: 1.0 m SLR and 17% increase in winter rainfall.
- Scenario 3: 1.0 m SLR and 40% increase in winter rainfall.
- Scenario 4: 1.5 m SLR and 40% increase in winter rainfall.

The rainfall scenarios were based on guidance at the time (MfE, 2008). The 17% and 40% increases in rainfall are conservative 'extreme' rainfall estimates and relate to mid- and high-range scenarios of likely increases in temperature.

In our assessment we have considered Scenario 1 for the "medium" RSLR scenario and Scenario 4 for the "high" RSLR scenario as shown in Table 7.3.

7.4 Approach to individual inundation cells

7.4.1 Definition of inundation cells

Figure 7.1 shows the four inundation cells considered in our assessment. The cells have been defined by two main criteria:

- The locations of estimated inundation water levels by Lane et al (2012) in relation to the main inundation pathways (streams and rivers).
- The approximate drainage catchment boundaries for the pathways.

This allows each discrete storm tide level to be applied to a separate section of the coast without discontinuity in the mapped flood extent using the bathtub method.

The "Ōtaki and Te Horo" cell extends from the northern boundary of the District southwards as far as the estimated limit of the Mangaone Stream drainage catchment (Te Hapua Road), covering the inundation pathways to which the Ōtaki North storm tide level applies.

The "Peka Peka and Waimeha" cell extends south from Te Hapua Road to the estimated boundary between the Waimeha Stream and the Waikanae River drainage catchments, covering inundation pathways to which the Waikanae Beach storm tide level applies.

The storm tide levels at the remaining points (Paraparaumu Beach, Raumati Beach and Paekākāriki) are the same and the remaining section of the District is split between the largely urban area in the "Waikanae and Raumati" cell and the largely rural area in "Paekākāriki and Whareroa" cell along the southern limits of Raumati South.

7.4.2 Ōtaki and Te Horo

The main inundation pathways in the Ōtaki and Te Horo cell are:

- The Waitohu Stream, the Ōtaki River, the Mangaone Stream and the drains which outfall to these streams;
- The stormwater outfalls from small drainage catchments along the coast which outfall directly to the sea; and
- Direct inundation of the shoreline.

Table 7.5 provides details of the scenarios considered in this inundation cell.

Table 7.5: Sea levels for inundation scenarios for the Ōtaki and Te Horo inundation cell.

Rise in mean sea level	Peak Tide (mWVD53)	Notes
None	1% AEP (2.43 m)	▪ MSL (2020) is 0.25 m-WVD53 ▪ 1% AEP Storm tide (including wave setup) is 2.43 m-WVD53 in 2020 (ref. Table 4.1, Ōtaki North)
"Low" 0.40 m	1% AEP (2.83 m)	
"Medium" 0.65 m	1% AEP (3.08 m)	
"High" 1.65 m	1% AEP (4.08 m)	

The maps in Figure 7.5 show, in red, the areas identified in the SKM (2012) study where groundwater levels could rise above ground level if the mean sea level rises by 0.5 m or 1.5 m and winter rainfall increases (SKM Scenario 1 and 4). The map also shows, in blue, the areas where ground levels are below the MSL for the "medium" and "high" values of SLR considered in our assessment (0.65 m and 1.65 m respectively) together with the stormwater network layout and main open channels in the Ōtaki and Te Horo cell.

The maps show that:

i) For the "medium" SLR scenario, areas of potential groundwater "breakout" are limited mainly to the Ōtaki River mouth and alongside the Mangaone Stream where there is potential for gravity drainage of elevated groundwater or the areas are beyond the expected inundation zone. We have therefore not included groundwater ponding in the inundation maps for this scenario.

ii) For the "high" SLR scenario the SKM (2012) study indicates extensive areas of potential groundwater "breakout". North of the Ōtaki River these areas are generally drained by streams. Where the ground level is below MSL (including SLR) and ponding is more likely, these areas are already included in the bathtub inundation mapping. To the south of the Ōtaki River the depths of ponding indicated by the SKM (2012) study are relatively deep (0.5 m to 1.5 m). The groundwater levels do not account for drainage paths and will tend to overestimate depths. Inclusion of these depths of water as groundwater ponding depths in the bathtub inundation maps is not considered realistic.

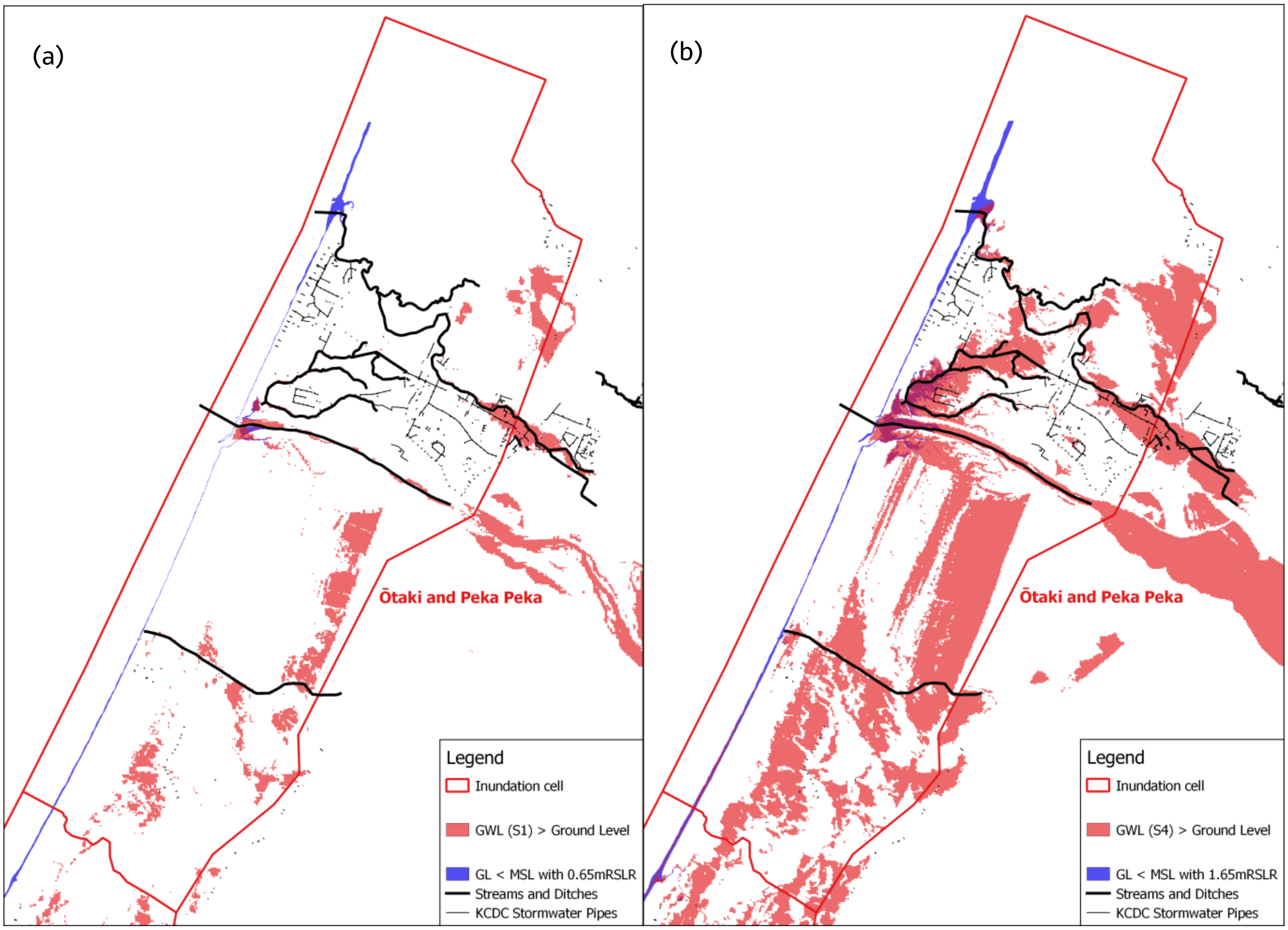

Figure 7.5: Ōtaki and Te Horo inundation cell : areas of elevated groundwater levels (GWL) under climate change (Scenarios 1 and 4, SKM 2012) and ground level (GL) below mean sea level (MSL) for (a) 0.65 m and (b) 1.65 m relative sea level rise (RSLR).

7.4.3 Peka Peka and Waimeha

The inundation pathways in the Peka Peka and Waimeha cell are:

- Te Kowhai Stream, Waimeha Stream and the drains which outfall to these streams;
- The stormwater outfalls from small drainage catchments along the coast which outfall directly to the sea; and
- Direct inundation of the shoreline.

Table 7.6 provides details of the scenarios considered in this inundation cell.

Table 7.6: Sea levels for inundation scenarios for the Peka Peka and Waimeha inundation cell.

Rise in mean sea level	Peak Tide (mWVD53)	Notes
None	1% AEP (2.31 m)	▪ MSL (2020) is 0.25 m-WVD53 ▪ 1% AEP Storm tide (including wave setup) is 2.31 m-WVD53 in 2020 (ref. Table 4.1, Waikanae Beach)
“Low” 0.40 m	1% AEP (2.71 m)	
“Medium” 0.65 m	1% AEP (2.96 m)	
“High” 1.65 m	1% AEP (3.96 m)	

The maps in Figure 7.6 show, in red, the areas identified in the SKM (2012) study where groundwater levels could rise above ground level if the MSL rises by 0.5 m or 1.5 m and winter rainfall increases (SKM Scenario 1 and 4). The map also shows, in blue, the areas where ground levels are below the MSL for the “medium” and “high” values of SLR considered in our assessment (0.65 m and 1.65 m respectively) together with the stormwater network layout and main open channels in the Peka Peka and Waimeha cell.

The maps show that for both scenarios high groundwater levels are estimated in the dunes at the north end of the Waikanae cell (0.5 m to 1 m above ground level in Scenario 1; and 1 m to 3 m above ground level in Scenario 4). The SKM (2012) study notes that model validation was poor in this area and the effects of local drainage paths are not included in the estimates. Inclusion of these depths of water as groundwater ponding depths in the bathtub inundation maps is not considered realistic. For these reasons we have not included additional ponding in the bathtub inundation maps for this cell.

(a)

(b)

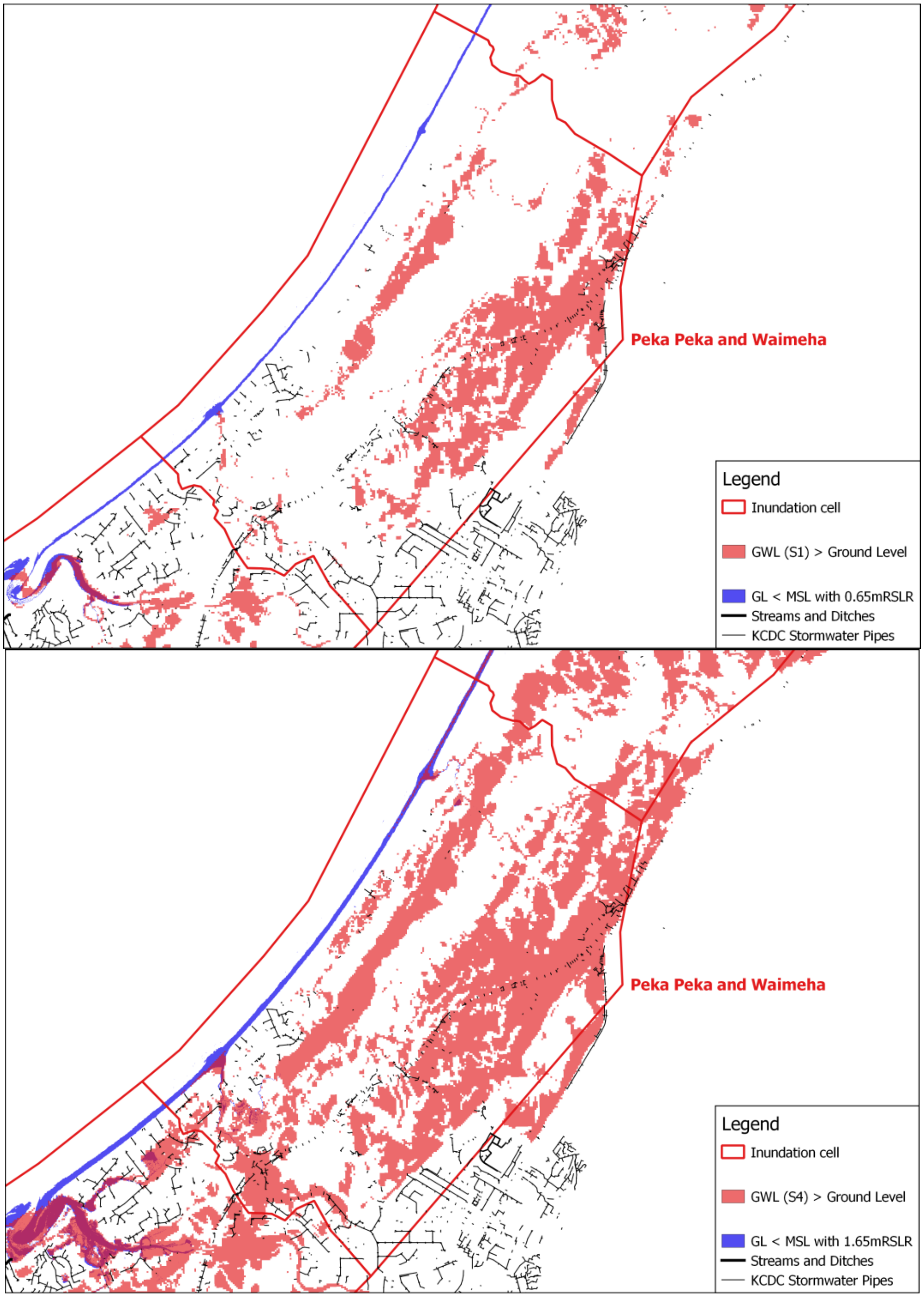

Figure 7.6: Peka Peka and Waimeha inundation cell: Areas of elevated groundwater levels (GWL) under climate change (Scenarios 1 and 4, SKM 2012) and ground level (GL) below mean sea level (MSL) for (a) 0.65 m and (b) 1.65 m relative sea level rise (RSLR).

7.4.4 Waikanae and Raumati

The inundation pathways in the Waikanae and Raumati cell are:

- Waikanae River, Tikotu Creek, Wharemauku Stream and the drains which outfall to the streams;
- The stormwater outfalls from small drainage catchments along the coast which outfall directly to the sea; and
- Direct inundation of the shoreline.

Table 7.7 provides details of the scenarios considered in this inundation cell.

Table 7.7: Sea levels for inundation scenarios for the Waikanae and Raumati inundation cell.

Rise in mean sea level	Peak Tide (mWVD53)	Notes
None	1% AEP (2.14 m)	▪ MSL (2020) is 0.25 m-WVD53 ▪ 1% AEP Storm tide (including wave setup) is 2.14 m-WVD53 in 2020 (ref. Table 4.1, Paraparaumu Beach and Raumati Beach)
"Low" 0.40 m	1% AEP (2.54 m)	
"Medium" 0.65 m	1% AEP (2.79 m)	
"High" 1.65 m	1% AEP (3.79 m)	

The maps in Figure 7.7 show, in red, the areas identified in the SKM (2012) study where groundwater levels could rise above ground level if the MSL rises by 0.5 m or 1.5 m and winter rainfall increases (Scenario 1 and 4). The map also shows, in blue, the areas where ground levels are below the MSL for the "medium" and "high" values of SLR considered in our assessment (0.65 m and 1.65 m respectively) together with the stormwater network layout and main open channels in the Waikanae and Raumati cell.

The maps show that in both cases:

- Ground levels in all the areas of potential groundwater "breakout" are above MSL (including an allowance for a "high" rise in sea level) and so there is potential for gravity drainage of elevated groundwater.
- The current network of stormwater pipes and drainage channels extends through all the areas of potential groundwater "breakout".

For these reasons it is unlikely that the rise in groundwater will cause large areas of deeper permanent surface ponding and we have therefore not included additional ponding in the bathtub inundation maps for this cell.

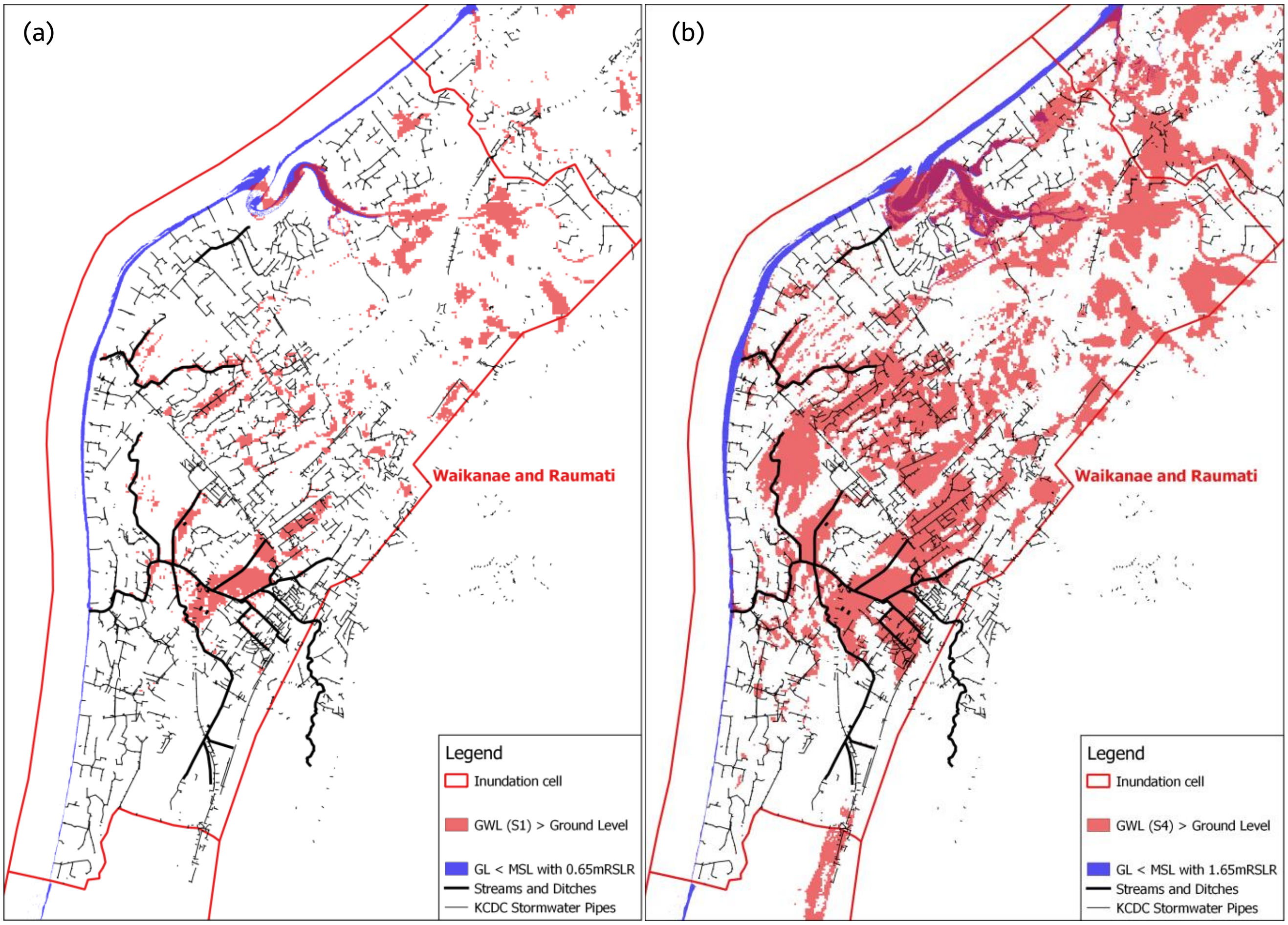

Figure 7.7: Waikanae and Raumati inundation cell: Areas of elevated groundwater levels (GWL) under climate change (Scenarios 1 and 4, SKM 2012) and ground level (GL) below mean sea level (MSL) for (a) 0.65 m and (b) 1.65 m relative sea level rise (RSLR).

7.4.5 **Paekākāriki and Whareroa**

The inundation pathways in the Paekākāriki and Whareroa cell are:

- The Whareroa Stream, Wainui Stream, and the drains which outfall to the streams;
- The stormwater outfalls from small drainage catchments along the coast which outfall directly to the sea; and
- Direct inundation of the shoreline.

Table 7.8 provides details of the scenarios considered in this inundation cell.

Table 7.8: Sea levels for inundation scenarios for the Paekākāriki and Whareroa inundation cell.

Rise in mean sea level	Peak Tide (mWVD53)	Notes
None	1% AEP (2.14 m)	▪ MSL (2020) is 0.25 m-WVD53 ▪ 1% AEP Storm tide (including wave setup) is 2.14 m-WVD53 in 2020 (ref. Table 4.1, Raumati Beach and Paekākāriki)
“Low” 0.40 m	1% AEP (2.54 m)	
“Medium” 0.65 m	1% AEP (2.79 m)	
“High” 1.65 m	1% AEP (3.79 m)	

The map in Figure 7.8 shows, in red, the areas identified in the SKM (2012) study where groundwater levels could rise above ground level if the MSL rises by 1.5 m and winter rainfall increases (Scenario 4). The map also shows, in blue, the areas where ground levels are below the MSL for the “high” value of SLR considered in our assessment (0.65 m and 1.65 m respectively) together with the stormwater network layout and main open channels in the Paekākāriki and Whareroa cell.

The map shows that even for the “high” SLR scenario the estimated areas of potential groundwater breakout in these cells are limited to the stream channels and we have therefore not considered groundwater ponding in the inundation assessment for these cells.

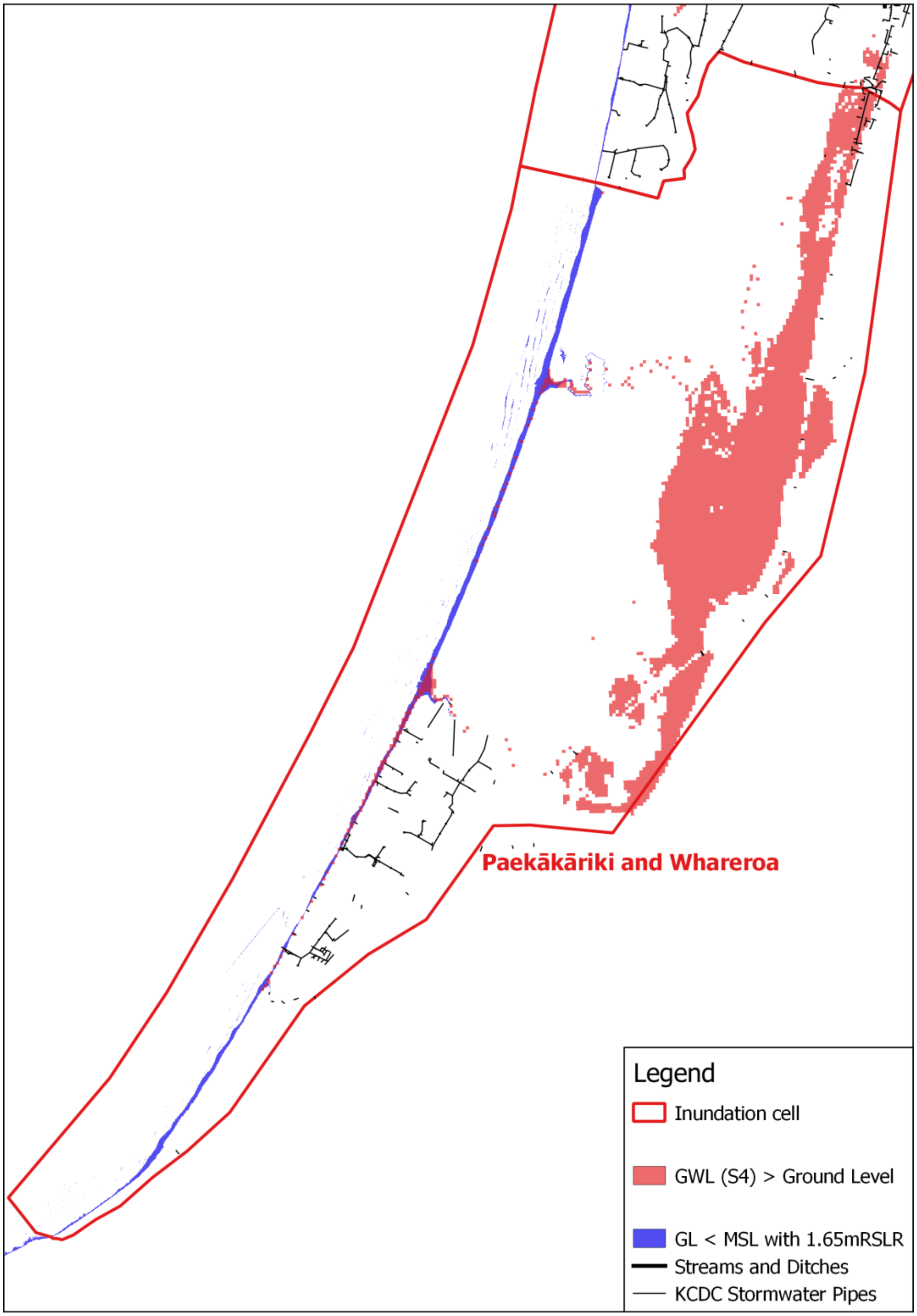

Figure 7.8: Paekākāriki and Whareroa inundation cell: Areas of elevated groundwater levels (GWL) under climate change (Scenario 4, SKM 2012) and ground level (GL) below mean sea level (MSL) for 1.65 m relative sea level rise (RSLR).

7.5 Beach Run-up modelling

Wave run-up is a result of the uprush of water after waves collapse on the beach. It varies on both a temporal and spatial scale, and is very difficult to incorporate into bathtub modelling, which uses a static water level. Wave run-up can increase inundation depths and spatial extents in areas where the wave can overtop the dune or a structure, resulting in the creation of an inundation pathway, as shown in Figure 7.9.

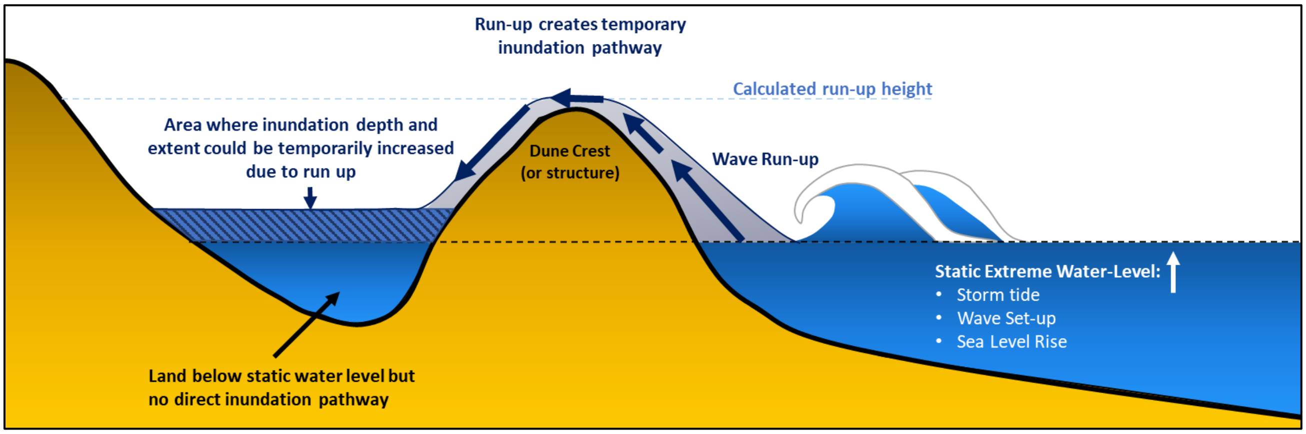

Figure 7.9: Schematic showing how run-up has been incorporated bathtub modelling. Areas from the bathtub modelling have been identified as being potentially affected by an increase in inundation depth and spatial extent where an inundation pathway has been created via run-up.

The purpose of including an element of wave run-up in the inundation assessment was to identify potential areas where run-up overtopping has the potential to overtop the dune of beach ridge and therefore result in an increase in inundation depth and extent. The assessment is limited to identification of locations where this could occur based on the calculated wave runup levels for present day beach elevations and slopes. However, the effect of this run-up overtopping has not been quantified in terms of increased water depths or spatial extent of flooding. This would be a subsequent step which could be done following the updated flood models by KCDC at a later stage.

The 2017 LiDAR was used to determine the shoreline areas along the entire coastline where the dune/structure elevation and immediate hinterland is below the run-up levels presented in Table 2.4 (from Lane et al, 2012) (except for Te Horo, where the results from Lane are rejected and the re-calculated estimated maximum run-up level of 2.77 m from the September 1976 storm is used) at

(1) current sea level;

(2) with 0.65 m RSLR; and

(3) with 1.65 m of RSLR.

These areas are defined as “potential run-up overtopping” areas. Areas which had been identified in the initial bathtub assessment as being inundated, and then identified as having an additional inundation pathway via runup are cross hatched to show that runup and overtopping could increase the inundation depth and extent at this area.

This approach is only adopted in areas where the immediate hinterland of the dune/structure was lower that the dune/structure, and therefore water would pool behind. Where the dune/structure was overtopped but the hinterland elevation continued to increase immediately behind the dune/structure, it is assumed that there will only be minimal increase to inundation depths on a highly variable temporal scale, and water will run back down the slope not adding a large volume to the additional pooling of water.

7.6 Coastal Inundation Assessment Limitations

A summary of the limitations of the methodology used to conduct the coastal inundation assessment are as follows:

- The inundation mapping is based on previous estimates of storm tide level at five locations along the coast (Lane et al, 2012). These tide levels include allowance for wave setup along the open coast, however the wave setup for inundation pathways through the river mouths may be smaller, and therefore the bathtub could be conservative inside river mouth areas.
- A single storm tide level has been adopted for each of the defined inundation cells. Inundation cells have been delineated based on the estimated catchments of the main stream pathways associated with the location of each storm tide level. However, these storm tide levels are likely to vary a small amount throughout the inundation cell, which has not been encapsulated in the bathtub model.
- The bathtub assessment is based on ground levels derived from 2017 LiDAR survey by KCDC. Whilst this is the most recent dataset available, there may be changes to the ground levels, development, and shoreline elevation since 2017 that are not accounted for in the bathtub model.
- All areas of ground below storm tide level have been mapped as vulnerable to inundation in each scenario without an assessment of potential pathways.
- The potential for contribution from elevated groundwater under RSLR that has been considered is based on the limited high level data available. However, this data is not sufficiently detailed to provide quantitative estimates of the contributions.
- The bathtub approach does not take into account the capacity of inundation pathways, or the volume of water available for inundation over the period of a storm tide event (with consideration for the tidal cycle), which can limit the extent and depths of inundation. Therefore, the bathtub model could be considered to be conservative.
- The bathtub model does not allow for contribution of coincident rainfall or fluvial flow.
- Additional areas that could be effected by wave run-up overtopping dunes/structures has been identified, however the spatial extent or increase in water depth has not been determined. This would require a volume overtopping assessment, and could be carried out in a later stage of the *Takutai Kāpiti* project to better define the inundation hazard in more defined areas where runup could be an issue.

8. Vulnerability Assessment

In this assessment we have quantitatively assessed the *sensitivity* of key council infrastructure and community services to the current and future exposure to coastal hazards. The purpose of this assessment is to determine when various assets or services may become affected by SLR. In assessing this, we are able to develop a high level vulnerability profile for each coastal cell.

This assessment has not considered any social, ecological, or culturally significance assets, sites, or services, as this will require input from the community to understand what assets or sites the community values. It is understood this is likely to form part of the assessment in Phase Two of the project.

The vulnerability assessment considered the following community services and critical infrastructure:

- Properties
- Key roads which run parallel to the shoreline
- Schools
- Medical Centres
- Public coastal stormwater outlets
- Wastewater treatment plants
- Water supply bores
- Pump stations

All community service and infrastructure data was provided by KCDC. Property data was extracted from the LINZ data service.

An asset or service was assessed as being 'affected' if the location of the asset/service intersected with the hazard susceptibility at the 50-year or 100-year timeframe. For the inundation hazard, water depths are not reported as part of this vulnerability assessment.

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Appendix A. Extreme Event Wave Parameters from MetOcean Solutions (2007) and Stephens et al (2011)

Name (MOS 2007)	Location	Max (Hs) (m)	P95 Hs (m)	P99 Direction	P99 Tp (s)	Significant wave height (m) for specified return periods									
						1-yr		2-yr		10-yr		50-yr		100-yr	
						MOS (2007)	Stephens (2011)	MOS (2007)	Stephens (2011)	MOS (2007)	Stephens (2011)	MOS (2007)	Stephens (2011)	MOS (2007)	Stephens (2011)
kpto01	North Ōtaki	4.75	2.6	291	9.11	4.11		4.42		5.09		5.7		5.95	
kpto02	South Ōtaki	4.83	2.63	293	9.14	4.15		4.46		5.14		5.75		5.99	
kpto03	Central Te Horo	4.74	2.57	297	9.05	4.05		4.36		5.01		5.6		5.84	
kpto04	North Peka Peka	4.7	2.52	299	9.07	3.95		4.24		4.87		5.45		5.68	
kpto05	South Peka Peka	4.63	2.42	307	8.93	3.84		4.12		4.74		5.3		5.52	
kpto06	North Waikanae	3.92	2.07	319	8.37	3.28		3.52		4.03		4.5		4.69	
kpto07	South Waikanae	3.31	1.74	330	7.57	2.77	4.37	2.98	4.72	3.41	5.51	3.81	6.24	3.97	6.53
kpto08	North Paraparaumu	3.13	1.43	324	5.88	2.24		2.4		2.73		3.04		3.16	
kpto09	South Paraparaumu	3.29	1.41	267	5.61	2.25		2.42		2.79		3.14		3.28	
kpto10	North Raumati	3.27	1.61	267	6.99	2.59		2.79		3.22		3.62		3.79	
kpto11	Central Raumati	3.42	1.86	278	7.91	3		3.23		3.71		4.16		4.34	
kpto12	South Raumati	3.62	2.00	288	8.32	3.15		3.38		3.87		4.33		4.51	
kpto13	Central QE II Park	3.92	2.17	293	8.71	3.38		3.62		4.15		4.63		4.83	
kpto14	North Paekākāriki	4.38	2.3	297	8.69	3.67		3.94		4.53		5.07		5.29	
kpto15	South Paekākāriki	4.53	2.36	303	8.57	3.8	4.45	4.08	4.83	4.69	5.61	5.24	6.29	5.46	6.55
kpto16	South of Paekākāriki	4.59	2.38	308	8.54	3.83		4.11		4.72		5.28		5.5	

Appendix B. Residual life groupings for coastal structures

J:\IE\Projects\02_New Zealand\IS355300\22 Technical\GIS\MXD\Jacobs Structure Residual Life Maps.mxd

CLIENT Kapiti Coast District Council	
PROJECT Takutai Kapiti Coastal Hazards Assessment	
SCALE 1:12,241 @ A3	PROJECT CODE IS355300
PROJECT MANAGER AK	DRAWN KM
PROJECT DIRECTOR BC	DATE 04/23/2021

Base Aerial Imagery from 2017

Residual life of structures

Paekākāriki

Max residual life as per T&T 2016 and 2021 assessments.
Grouped across multiple structures for use in this assessment

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CLIENT Kapiti Coast District Council	
PROJECT Takutai Kapiti Coastal Hazards Assessment	
SCALE 1:13,621 @ A3	PROJECT CODE IS355300
PROJECT MANAGER AK	DRAWN KM
PROJECT DIRECTOR BC	DATE 04/23/2021

Base Aerial Imagery from 2017

Residual life of structures

Raumati

Max residual life as per T&T 2016 and 2021 assessments.
Grouped across multiple structures for use in this assessment

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Appendix C. Data Reference Points used in CED Calculations

J:\IE\Projects\02_New Zealand\IS355300\22 Technical\GIS\MXD\Data Reference Points.mxd

CLIENT Kapiti Coast District Council	
PROJECT Takutai Kapiti Coastal Hazards Assessment	
SCALE 1:19,000 @ A3	PROJECT CODE IS355300
PROJECT MANAGER AK	DRAWN KM
PROJECT DIRECTOR BC	DATE 05/21/2021

Base Aerial Imagery from 2017

KCDC Coastal Hazards Assessment
Data Reference Points
Map 1 (Ōtaki)

Jacobs
Level 2, Wynn Williams building,
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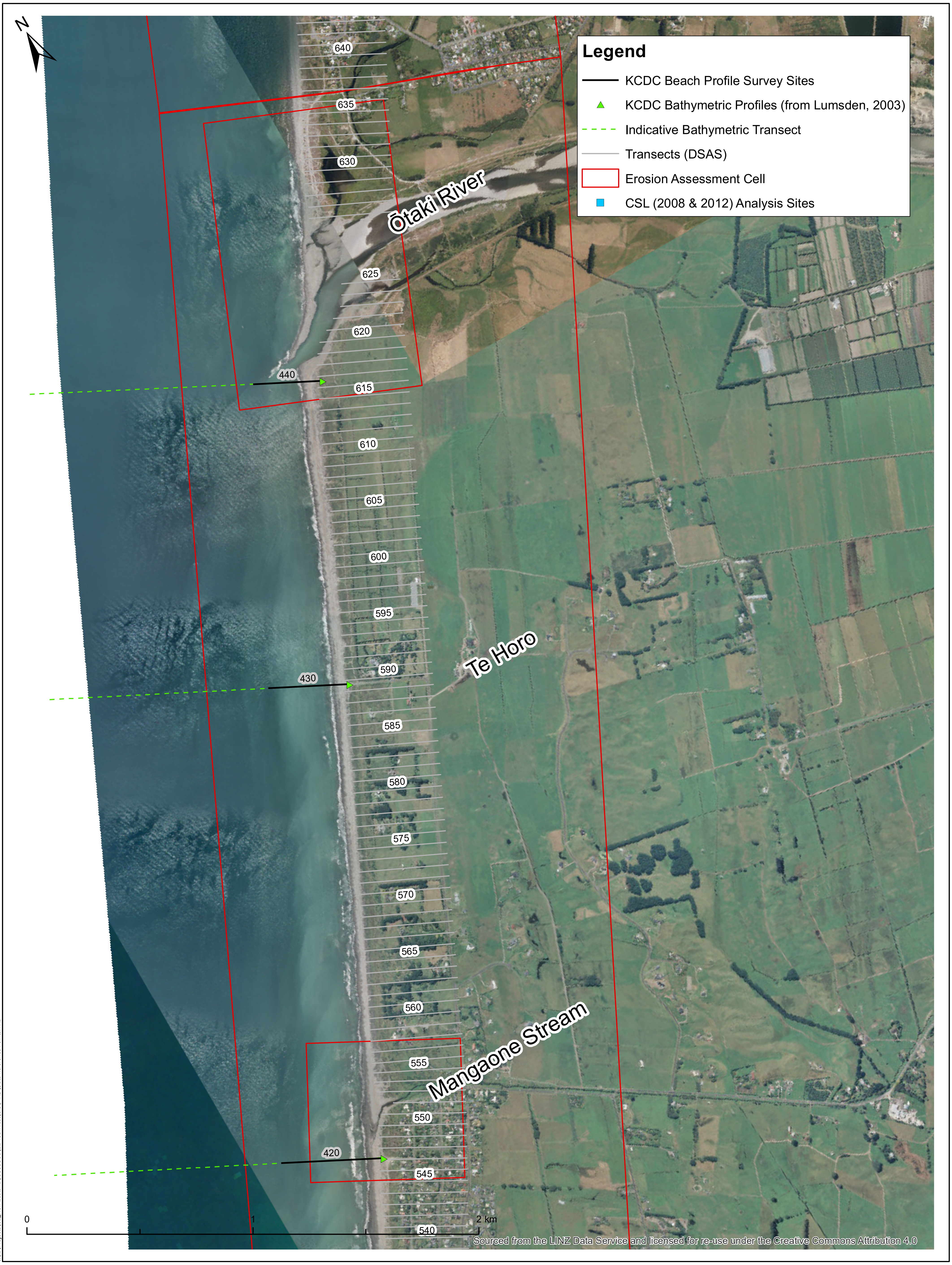

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CLIENT Kapiti Coast District Council	
PROJECT Takutai Kapiti Coastal Hazards Assessment	
SCALE 1:15,000 @ A3	PROJECT CODE IS355300
PROJECT MANAGER AK	DRAWN KM
PROJECT DIRECTOR BC	DATE 05/21/2021

Base Aerial Imagery from 2017

KCDC Coastal Hazards Assessment
Data Reference Points
Map 2 (Te Horo)

Jacobs
Level 2, Wynn Williams building,
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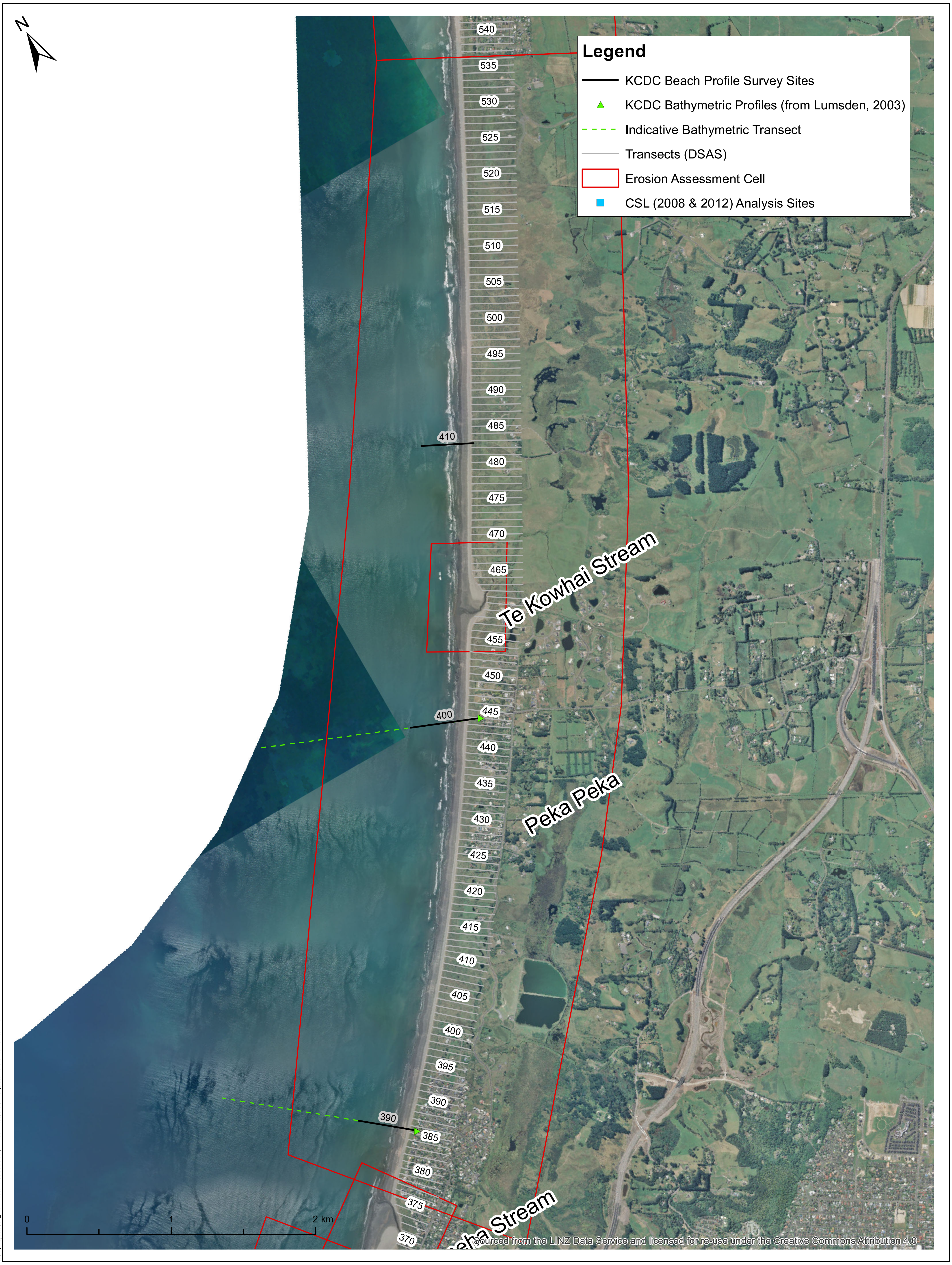

CLIENT	Kapiti Coast District Council
PROJECT	Takutai Kapiti Coastal Hazards Assessment
SCALE 1:23,500 @ A3	PROJECT CODE IS355300
PROJECT MANAGER AK	DRAWN KM
PROJECT DIRECTOR BC	DATE 05/21/2021

Base Aerial Imagery from 2017

KCDC Coastal Hazards Assessment
Data Reference Points
Map 3 (Peka Peka)

Jacobs
Level 2, Wynn Williams building,
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CLIENT Kapiti Coast District Council		
PROJECT Takutai Kapiti Coastal Hazards Assessment		
SCALE 1:11,500 @ A3	PROJECT CODE IS355300	
PROJECT MANAGER AK	DRAWN KM	
PROJECT DIRECTOR BC	DATE 05/21/2021	

Base Aerial Imagery from 2017

KCDC Coastal Hazards Assessment
Data Reference Points
Map 4 (Waikanae)

Jacobs
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CLIENT Kapiti Coast District Council		
PROJECT Takutai Kapiti Coastal Hazards Assessment		
SCALE 1:14,000 @ A3	PROJECT CODE IS355300	
PROJECT MANAGER AK	DRAWN KM	
PROJECT DIRECTOR BC	DATE 05/21/2021	Base Aerial Imagery from 2017

KCDC Coastal Hazards Assessment
Data Reference Points
Map 5 (Paraparaumu)

Jacobs
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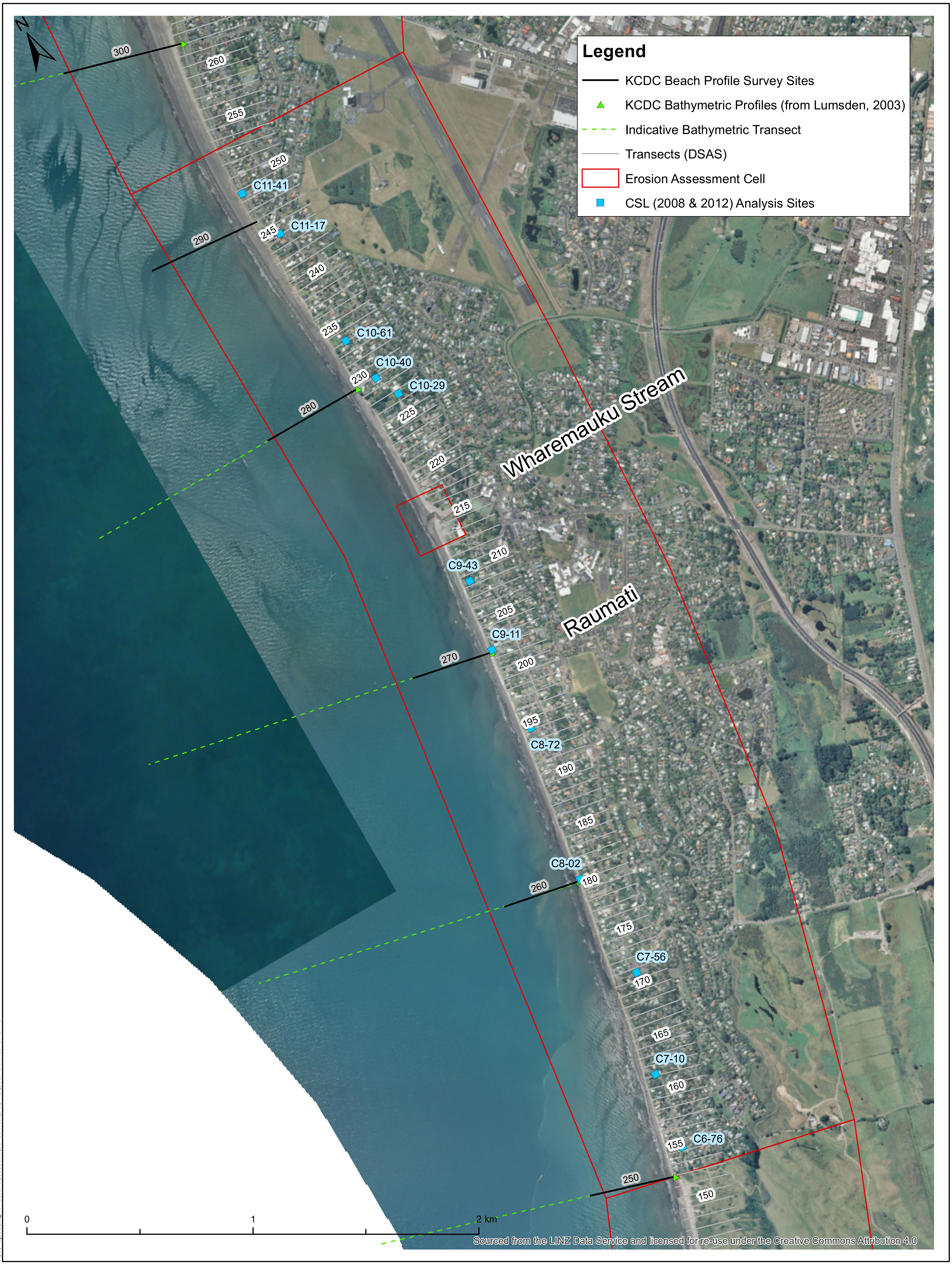

CLIENT Kapiti Coast District Council	
PROJECT Takutai Kapiti Coastal Hazards Assessment	
SCALE 1:15,000 @ A3	PROJECT CODE IS355300
PROJECT MANAGER AK	DRAWN KM
PROJECT DIRECTOR BC	DATE 05/21/2021

Base Aerial Imagery from 2017

KCDC Coastal Hazards Assessment
Data Reference Points
Map 6 (Raumati)

Jacobs
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CLIENT	Kapiti Coast District Council
PROJECT	Takutai Kapiti Coastal Hazards Assessment
SCALE 1:10,500 @ A3	PROJECT CODE IS355300
PROJECT MANAGER AK	DRAWN KM
PROJECT DIRECTOR BC	DATE 05/21/2021

Base Aerial Imagery from 2017

KCDC Coastal Hazards Assessment
Data Reference Points
Map 7 (Queen Elizabeth Park)

Jacobs
Level 2, Wynn Williams building,
47 Hereford Street,
Christchurch Central 8013,
New Zealand
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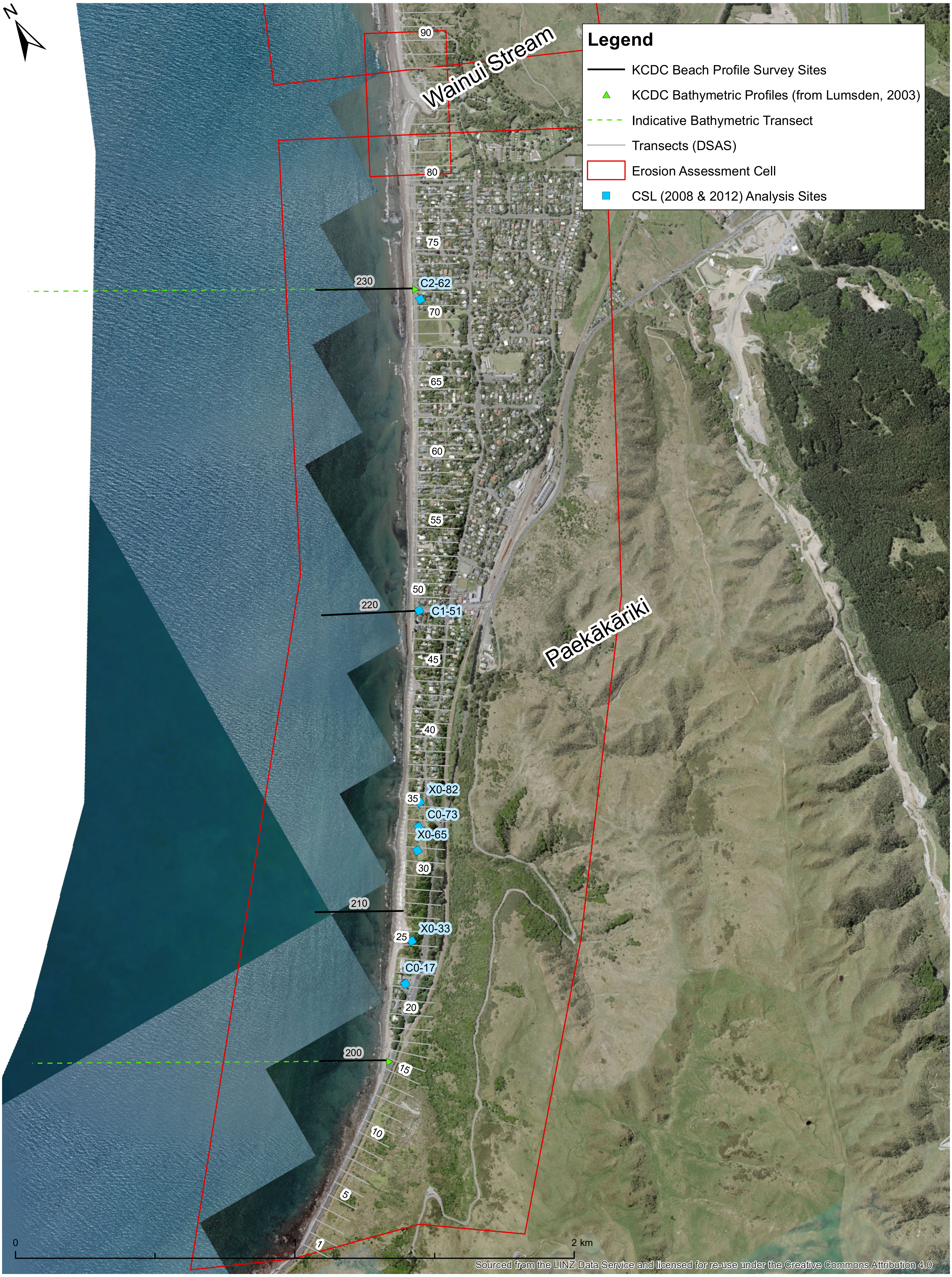

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CLIENT Kapiti Coast District Council		
PROJECT Takutai Kapiti Coastal Hazards Assessment		
SCALE 1:12,500 @ A3	PROJECT CODE IS355300	
PROJECT MANAGER AK	DRAWN KM	
PROJECT DIRECTOR BC	DATE 05/21/2021	

Base Aerial Imagery from 2017

KCDC Coastal Hazards Assessment
Data Reference Points
Map 8 (Paekākāriki)

Jacobs
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New Zealand
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Appendix D. Short-term Component Sensitivity Testing

The short-term erosion component in the KCDC erosion assessment was calculated based on observations recorded in the September 1976 storm by Gibb and Wiltshere (1976). The quasi-distribution formed based on these observations used the highest observation of the area as the mean, and used +/- 50% of the mean as the maximum and minimum values of the distribution. The taking of the existing upper magnitude of storm retreat observations as the mean value could be considered to be a conservative approach, however this is counteracted by the increased frequency of this magnitude event in the future due to SLR.

Sensitivity testing was carried out to determine whether the quasi-distributions could be considered too conservative for use in this assessment, and whether alternative approaches recommended in the Carley et al (2014) panel review, and by Beca in reviewing this assessment, were more appropriate. This sensitivity testing looked at two alternative methods:

- Lumsden (2003) Geometric model approach recommended in the Carley et al (2014) expert panel review. and
- SBEACH numerical modelling approach recommended in the Beca (2020) independent review of this assessment.

The following will discuss the sensitivity testing of these two methods against the observations by Gibbs and Wiltshere (1976) to test whether the two methods could be used in this assessment, and whether similar results to the 1976 observations could be produced using these methods.

D.1 Geometric Model Approach

The Carley et al (2014) panel review recommended that the geometric model used in Lumsden (2003) was applied in future assessments. Lumsden (2003) uses an approach developed by Komar et al (1999) for the low sloping dissipative beach of Oregon, USA, that are similar to those found along the Kāpiti Coast. A schematic of this geometric model is presented in Figure 5.1. This approach is based on using extreme combinations of ocean processes, waves, and tides in storm events, and looking at how an extreme water level would cut into the foredune. The equation for this approach is as follows:

$$\boldsymbol{E}_{\max} = \frac{(\boldsymbol{WL} - \boldsymbol{H_j}) + \Delta\boldsymbol{BL}}{\boldsymbol{tan\beta}}$$

Where:

WL = total water level (tide + runup);

H_j = elevation of toe of foredune;

B = angle of beach face dominated by wave swash; and

ΔBL = vertical shift in beach level change due to rip current or general beach erosion.

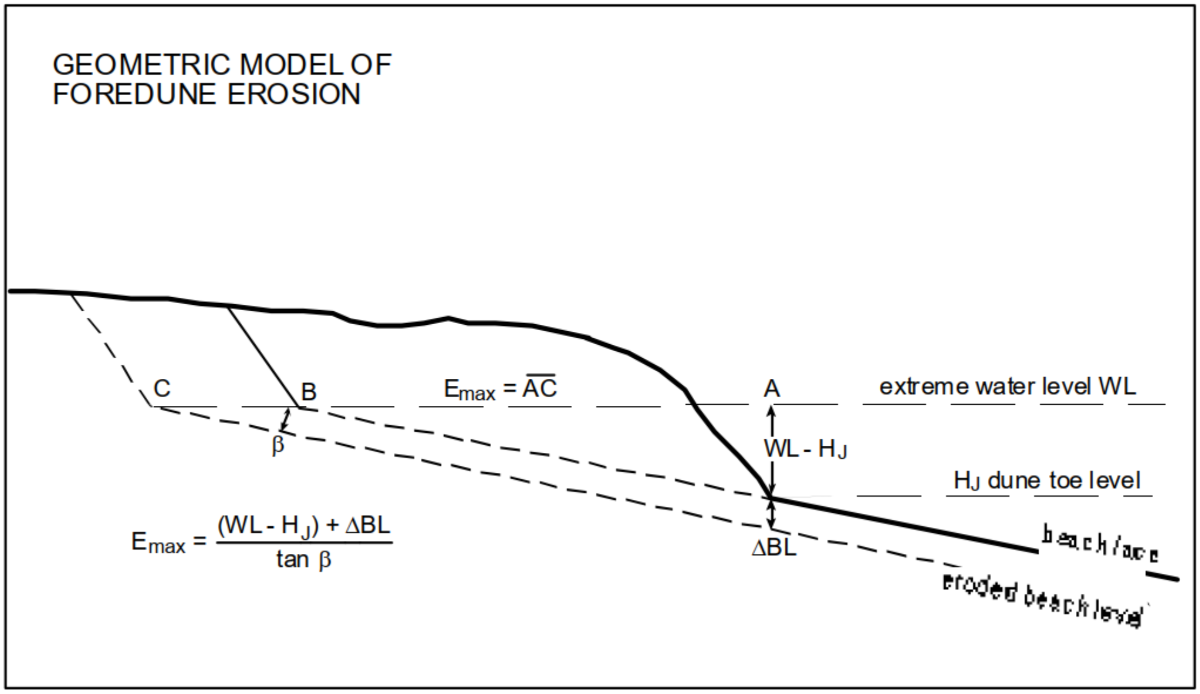

Figure B.1: Geometric model of foredune erosion from Komar et al (1999) adapted by Lumsden (2003) to predict further shoreline retreat along the Kāpiti Coast with SLR.

This model was tested at two sites along the KCDC coastline, (1) Structured coastline at Raumati; and (2) open coast non-structured site at Ōtaki. KCDC beach profiles were used at each site to determine profile characteristics to form a distribution. Extreme water levels were taken from NIWA (2012), and distributions were developed based on the 1% AEP water level, plus the run-up calculated from three modelled storm events (12/09/1976; 06/09/1994; and 07/11/1994). The extreme water levels applied to the two sites presented in Table B.1 below.

Table B.1: Extreme water levels used in geometric model sensitivity testing.

	Runup in Event			**Extreme water levels**	**Extreme water levels (Distributions)**		
Location	06/09/1994	07/11/1994	12/09/1976	Storm tide + runup to 2020 incl. recent SLR	Min	Mean	Max
Ōtaki Beach	0.44	0.68	1.09	2.43	2.87	3.11	3.52
Raumati	0.28	0.47	0.65	2.14	2.42	2.61	2.79

D.1.1 Raumati

The inputs and results for five beach profiles in Raumati into the Geometric model are presented below in Table B.2.

Table B.2: Geometric model inputs and results for profiles at Raumati.

Profile	Min/Mean/Max	WL	H_j	Tanβ	ΔBL	ST Erosion Distance
250	Maximum Erosion	2.79	0.15	0.013	NA	211.2
	Mean Erosion	2.61	0.53	0.027	NA	78.5
	Minimum Erosion	2.42	0.63	0.057	NA	31.3
260	Maximum Erosion	2.79	0.64	0.021	NA	100.8
	Mean Erosion	2.61	0.78	0.034	NA	54.0
	Minimum Erosion	2.42	0.72	0.051	NA	33.1
270	Maximum Erosion	2.79	0.64	0.021	NA	100.8
	Mean Erosion	2.61	0.78	0.031	NA	58.7
	Minimum Erosion	2.42	0.72	0.048	NA	35.4
280	Maximum Erosion	2.79	1.26	0.024	NA	64.7
	Mean Erosion	2.61	1.36	0.027	NA	46.0
	Minimum Erosion	2.42	1.39	0.032	NA	32.8
290	Maximum Erosion	2.79	1.20	0.020	NA	80.4
	Mean Erosion	2.61	1.83	0.026	NA	29.6
	Minimum Erosion	2.42	2.12	0.033	NA	9.1

As a result of structures being built along the shoreline, the beaches in front of the structures have lowered, and dune toe elevation (beach elevation at the base of the current structure) at the southern end of the cell is 0.15-0.63 m in elevation. This increases to 1.2-2.1 m of elevation at the northern end. Observations of erosion in Raumati following the significant September 1976 storm (e.g. largest on record) by Gibb and Wiltshere (1976), which identified that 15 m of erosion occurred where structures were present, and then were destroyed in the storm.

The results of using the geometric model at Raumati (without structures) showed that it could result in 31-211 m of erosion at the southern end of Raumati, and between 9-80 m of erosion at the northern end. Given that the observation of erosion following the September 1976 storm was 10-15 m, it was determined that applying this geometric model to structured shoreline in Raumati produced unrealistic results which would add a significant level of conservativeness to the resulting erosion distances for the cell. We therefore determined that this method was not appropriate for use in the Raumati cell.

D.1.2 Ōtaki Beach

No storm erosion observation was made in the Gibb and Wiltshere (1976) report, however we have assumed that it would be similar or more than the 5-10 m observed at Peka Peka, given that is has a similar wave climate and exposure along the open coast as Te Horo and Peka Peka. Both profiles within the Ōtaki cell were input into the geometric model, with the inputs and results of the geometric model are presented below in Table B.3.

Table B.3: Geometric model inputs and results for profiles at Ōtaki Beach.

Profile	Min/Mean/Max	WL	H_j	Tanβ	ΔBL	ST Erosion Distance
450	Maximum Erosion	3.52	0.7	0.008	NA	338.4
	Mean Erosion	3.11	1.43	0.017	NA	101.0
	Minimum Erosion	2.87	2.02	0.023	NA	36.2
460	Maximum Erosion	3.52	1.88	0.024	NA	69.8
	Mean Erosion	3.11	2.52	0.031	NA	19.0
	Minimum Erosion	2.87	2.91	0.043	NA	-0.9
460 Average profile across all water levels	Maximum Erosion	3.52	2.52	0.031	NA	32.1
	Mean Erosion	3.11	2.52	0.031	NA	19.0
	Minimum Erosion	2.87	2.52	0.031	NA	11.3

The results of the geometric model showed that at profile 450, the short term erosion ranged from 36.2 to 338.4 m. At profile 460, the short term erosion component ranged from 69.8 m of erosion to +0.9 m of accretion. When the average profile parameters were used against the range of extreme water levels, the results produced were more realistic that when the range of profile heights were tested against the range of water levels to get a probability distribution. Overall, the results at both profiles produced unrealistic results, which in line with the results produced at Raumati, would add a significant level of conservativeness to the resulting erosion distances for the cell. It was therefore concluded that this method was not appropriate for use at open coast sites.

D.2 SBEACH Numerical Modelling Approach

As part of the independent review of this methodology, it was recommended that SBEACH numerical modelling was used to test the sensitivity of the observations noted in the Gibbs and Wiltshere (1976). The observations at Waikanae beach following the September 1976 storm were noted as being 1-5 m of erosion. The September 1976 storm was then simulated in SBEACH using the following key data inputs:

- Waikanae onshore beach profile (Profile 370) combined with bathymetric profile 43;
- Simulated 48-72 hour tidal cycle with a storm tide of 1.25 m at the peak (excluding setup), estimated from Figure 2.6 in Lane et al (2012);
- Wind of 45 km/hr (estimated from observations of 20-30 knots per hour in Gibbs and Wiltshere (1976)); and
- Wave height of 6.53 m (from Stephens et al (2011)); wave period 8.4 – 14 seconds.

The SBEACH model was run 13 times using variations of the above key data to determine the sensitivity of the inputs, and to determine if using this data it would produce an erosion distance in the order of magnitude similar to the observations noted by Gibbs and Wiltshere (1976) of 1-5 m of erosion. The results of the various runs indicated that erosion was between 1-5 m, with the maximum erosion produced being between 3-4 m of erosion at the 2 m contour, and 1-2 m of erosion around the 5.5 m contour, as presented below in Figure B.2. While these model results fit within the observations noted in the 1976 storm, all runs produced erosion distances at the lower end of this erosion range. We believe that the results produced through the SBEACH modelling in Waikanae were likely to be on the lower end of erosion that would be expected in a rare storm event, and therefore have used the slightly higher observations by Gibbs and Wiltshere (1976) to form the quasi-distributions for the short term erosion component.

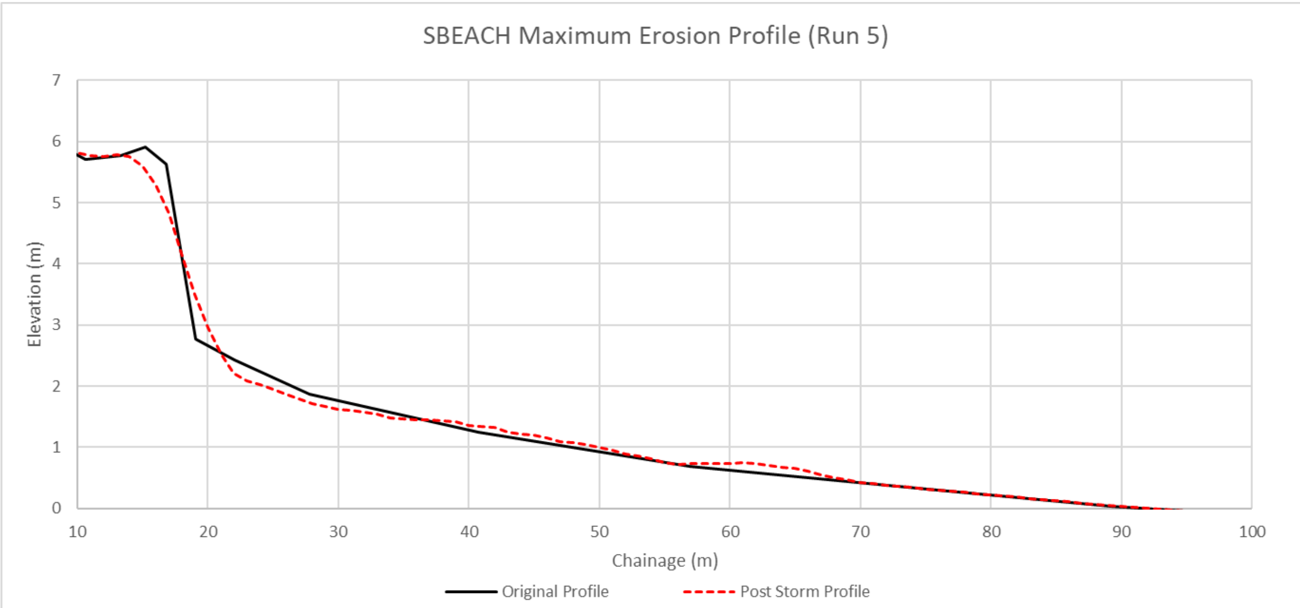

Figure B.2: Run 5 of SBEACH simulation which created the maximum onshore erosion (3-4 m of erosion at 2 m contour, 1-2 m erosion at top of dune).

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Kapiti Coast District Council
175 Rimu Rd
Paraparaumu

28 May 2021

Attention: Lyndsey Craig

Dear Lyndsey

Takutai Kapiti Coastal Hazard Exposure and Vulnerability Methodology Review

Kapiti Coast District Council (Council) commissioned Beca on the 13 November 2020 to provide ongoing peer review services for the Takutai Kapiti Coastal Hazard Exposure and Vulnerability Assessment that is being completed by Jacobs New Zealand Limited (Jacobs) for Council.

I confirm that I am a Technical Director in Coastal Science for Beca Ltd with over 20 years' experience in the field of coastal science, coastal hazards, climate change and metocean engineering. I frequently provide coastal hazard peer review services for New Zealand Councils. Review services for the Takutai Kapiti Coastal Hazard Exposure and Vulnerability Assessment have been completed for the following:

- Review of Jacobs assessment, methodology and data gap memorandum.
- Review of the draft Coastal Hazard and Risk Assessment technical report (dated 12 May 2021) that covers coastal erosion and coastal inundation hazards.

I can confirm that during the project I have provided intermittent review and feedback which has been integrated into the project approach and methodology.

Based on my review, I can confirm that the coastal erosion hazard methodology outlined in the Jacobs report dated 12 May 2021:

- Is consistent with the assessment guideline intent outlined in MfE, 2017:*Coastal Hazards and Climate Change – Guidance for Local Government*.
- Adopts current assessment techniques that have been used to define coastal hazards for similar environs in New Zealand;
- Considers uncertainty of the individual parameters contributing to coastal erosion form future sea level rise; and
- Is considered appropriate considering the level of information and data available.

It is noted that the inundation hazard methodology has adopted a simplified inundation technique to inform the assessment with the intent of being superseded by more detailed assessments that are being completed by others later in the year. Nevertheless, the inundation assessment is considered suitable for informing adaptation options.

Yours sincerely

Connon Andrews
Technical Director – Coastal Science

on behalf of
Beca Limited

Phone Number: +64 9 300 2418
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28 June 2021

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Paraparaumu 5254

Tēnā Koe Lyndsey

Takutai Kāpiti Coastal Hazard Exposure and Vulnerability Methodology Review

Kāpiti Coast District Council requested technical support from Greater Wellington Regional Council in October 2020 to provide peer review and feedback into the Takutai Kāpiti coastal hazard exposure and vulnerability assessment being undertaken by Jacobs New Zealand Limited.

I am the Senior Regional Hazards Analyst and policy advisor for the Wellington Regional Council. I have been employed at the Council since 2006. I hold a PhD specialising in coastal processes and geomorphology and have been involved in coastal research for over 25 years, at university level, within consultancy and currently in local government.

As the natural hazards for analyst Wellington Regional Council I provide scientific analysis, commentary and research into natural and coastal hazards that affect the Greater Wellington region and to write and/or provide expert advice and evidence for hearings, the Environment Court and policy that deals with managing the risks from natural hazards. I provide advice to policy analysts, resource managers, consents officers, engineers and elected councillors in the region, and to business's and the wider public.

Through the review of this work I provided feedback from the inception of the project and into the development of the methodology through to a peer review of the final written document.
I am satisfied that the methodology to undertake the coastal vulnerability assessment is appropriate for the purposes of informing and guiding community based decision making for coastal adaptation in the short, medium and long term planning horizons and to provide direction for District Plan coastal hazard approaches.

It is based on a reanalysis of existing data and information, of which a significant body of knowledge has accumulated for the Kapiti Coast over the past 30-40 years. It is consistent with New Zealand guidance for coastal hazard assessments and acknowledges the uncertainty in the data and future projections focussing objectively on the numerical statistics without introducing additional subjective levels of uncertainty.

Ngā mihi

Dr Iain Dawe
Senior Hazards Analyst
Environmental Policy

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Iain.dawe@gw.govt.nz