Report

Tonkin+Taylor

# Coastal Hazard Assessment for Christchurch and Banks Peninsula (2017)

**Prepared for**
Christchurch City Council

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## Executive summary

Christchurch City Council (Council) commissioned Tonkin & Taylor Ltd (T+T) in 2015 to prepare a report and maps that identified areas susceptible to coastal hazards (inundation, erosion and sea level rise) for the main coastal settlements selected by Council. The report was subject to a detailed expert panel peer review in 2016. The present report updates the original 2015 report taking into account the recommendations of the peer review panel.

The areas potentially susceptible to coastal hazards were termed coastal erosion hazard zones (CEHZ) and coastal inundation zones (CIHZ). The zones have been mapped over both a 50 year (approximately – taken to be 2065) and 100 year (taken to be 2120) planning time frame for both the open and harbour coast for four Intergovernmental Panel on Climate Change (IPCC) emission scenarios (median projections for RCP2.6, RCP4.5 and RCP8.5 and the 83rd percentile of RCP8.5).

The New Zealand Coastal Policy Statement 2010 (NZCPS, 2010) is a national policy statement under the Resource Management Act 1991. The NZCPS (2010) states policies in order to achieve the purpose of the Act in relation to the coastal environments of New Zealand. Both the Environment Canterbury Regional Policy Statement (RPS) and the Christchurch District Plan will give effect to the NZCPS (2010). The CEHZ methodology used for this project has been developed in accordance with the Objectives and Policies of the NZCPS (2010) directly relevant to the assessment of coastal erosion hazard.

The CEHZ methodology used in this study combines standard and well-tested approaches for defining coastal erosion hazard zones by adding together component parameters. This method has been refined for the open coast to include either a normal distribution, extreme value distribution or triangular distribution parameter bounds which are combined by stochastic simulation. The resulting distribution is a probabilistic forecast of potential hazard zone width, rather than including single values for each component and one overall factor for uncertainty.

This approach produces a range of hazard zones (probability distribution) corresponding to differing likelihoods which may be applied to risk-based assessments as advocated by the NZCPS (2010) and supported by best practice guidelines (e.g. Ramsey et al., 2012). Following consultation with Council in 2014, the P66% CEHZ value at 2065 and the P5% CEHZ value at 2120 were adopted as likely and potential CEHZ values (termed CEHZ2065 and CEHZ2120 respectively).

We implemented separate methodologies to assess coastal hazards for the open coast and the harbour coast sites due to the different physical coastal processes driving each of the two environments. The harbour coast CEHZ methodology is deterministic rather than probabilistic and accounts for the sheltered environment and differing morphologies of these sites. The method is consistent with current best practice guidelines (e.g. Ramsey et al., 2012).

The CIHZ was mapped using two methods:

- Connected "bath-tub" method – maps the area of land below the inundation level based on LiDAR derived topography, where there is a connection pathway to the sea. This method was used for sites located within both the Lyttelton and Akaroa Harbours and the open coast.
- Dynamic model method (TUFLOW) – simulates the physics of the tide and inundation levels to dynamically map the inundation levels based on LiDAR derived topography and detailed bathymetry of the estuary. This method is beneficial for wide flat areas and was implemented for Avon-Heathcote Estuary and the Brooklands Lagoon.

We recommend continuing to regularly monitor the shoreline position and inundation levels across the region to provide measured data, including continuing beach profile monitoring and digitising shorelines from aerial imagery or by GPS survey. We also recommend the adopted baselines and both the CEHZ and CIHZ values are reassessed at least every 10 years or following significant changes

in either legislation or best practice and technical guidance, which could potentially result in significant changes to the inundation or erosion hazard zones.

# 1 Introduction

Christchurch City Council (Council) commissioned Tonkin & Taylor Ltd (T+T) to prepare a report and maps that identified areas susceptible to coastal hazards (inundation, erosion and sea level rise) for the main coastal settlements selected by Council (T+T, 2015b). The report was subject to a detailed expert panel peer review in 2016 (Kenderdine et al., 2016). The present report updates the original 2015 report taking into account the recommendations of the peer review panel.

The areas potentially susceptible to coastal hazard were termed coastal erosion hazard zones (CEHZ) and coastal inundation zones (CIHZ). The zones have been mapped over both a 50 year (2065) and 100 year (2120) planning time frame for four IPCC climate change scenarios (median projections for RCP2.6, RCP4.5 and RCP8.5 and the 83$^{rd}$ percentile of RCP8.5 expressed in the report as RCP8.5+).

## 1.1 Previous work

Environment Canterbury Regional Council (ECan) developed two CHZs for the Canterbury region as set out in the Regional Coastal Environment Plan (RCEP, 2005):

- Coastal Hazard Zone 1 (CHZ1) – landward limit of the active beach system including any long-term rates of erosion to 50 years.
- Coastal Hazard Zone 2 (CHZ2) – landward limit of the active beach system including any long-term rates of erosion to 100 years.

The southern Pegasus Bay shoreline was assessed to be an accreting shoreline and therefore only Coastal Hazard Zone 1 was mapped as the landward limit of the active beach system in this area.

The delineation of the CHZ for Southern Pegasus Bay was completed prior to 2005 and there is now over 10 years of additional data that could be included in any new assessments. Understanding the physical processes and drivers of change is a key process that must be carried out to enable a robust coastal hazard assessment and is a fundamental requirement of Policy 24 of the NZCPS (2010). The Waimakariri River is a major source of sediment for the Southern Pegasus Bay shoreline, resulting in a historic trend of shoreline accretion.

T+T completed an assessment for Christchurch City Council on the effects of sea level rise over a 100 year time frame (T+T, 2013a). The assessment included high level mapping of the areas susceptible to storm inundation and erosion due to sea level rise over a 100 year time frame for both the Avon-Heathcote Estuary and Akaroa Harbour.

T+T also reviewed the existing coastal hazard zones (CHZ) as presented in the Regional Coastal Environment Plan for the Canterbury Region (T+T, 2015a). The review recommended re-assessing the existing CHZ as they did not adequately incorporate the potential effects of future climate change, including sea level rise and other effects as required under the New Zealand Coastal Policy Statement (NZCPS, 2010). The CHZ also needed to be re-assessed to consider the coastal erosion hazard over a shorter (50 year planning) time frame (i.e. to 2065). Also the coastal inundation hazard needed to be assessed for the open coast to identify low-lying areas of land with potential coastal inundation pathways.

## 1.2 Areas considered for present study

The extent of this study includes the coastal settlements located on non-consolidated (loose) sand or gravel shorelines within the Council jurisdictional boundaries (refer to Appendix A for site plan). The sites are listed below and are classified by their coastal environment as either *open coast* or *harbour coast* (harbours and estuaries):

Open coast

- Southern Pegasus Bay from Waimairi Beach to Southshore including the South Brighton spit
- Sumner.

Harbour coast

- Avon-Heathcote Estuary
- Brooklands Lagoon
- Lyttelton Harbour
  - Allandale
  - Teddington
  - Charteris Bay
  - Purau.
- Akaroa Harbour
  - Akaroa Township
  - Takamatua
  - Duvauchelle
  - Wainui.

# 2 Background information

## 2.1 Statutory legislation

### 2.1.1 New Zealand Coastal Policy Statement

The New Zealand Coastal Policy Statement 2010 (NZCPS) is a national policy statement under the Resource Management Act 1991. The NZCPS (2010) states policies in order to achieve the purpose of the Act in relation to the coastal environments of New Zealand. Regional policy statements and regional and district plans must give effect to the NZCPS (2010).

Objective 5 and Policies 3, 24, 25, 26 and 27 of the NZCPS (2010), refer to www.doc.govt.nz for policy statements, are directly relevant to the assessment of coastal hazard.

Objective 5 is to ensure that coastal hazard risks, taking into account 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
- Protecting or restoring natural defences to coastal hazards.

### 2.1.2 Canterbury Regional Policy Statement

The Canterbury Regional Policy Statement (CRPS) became operative on 15 January 2013 with revisions made in February 2017. The CRPS provides an overview of the resource management issues in the Canterbury region and sets out a suite of objectives, policies and methods in order to achieve integrated management of the region's resources.

Chapters 8 and 11 are of particular relevance to the assessment of coastal hazards. These chapters deal with the Coastal Environment and Natural Hazards respectively. The following objectives and policies are relevant to this report in regard to identifying coastal hazards in the Canterbury region:

- Issue 8.1.7 – Natural hazards in the coastal environment

  There is a need to assess the effects of climate change, and coastal hazards such as coastal erosion, on the coastal environment, and develop responses where human assets and natural values are threatened by such coastal hazards.
- Objective 8.2.1 – Increasing knowledge of the coastal environment and its resources

  A programme of information gathering is undertaken on the natural processes, ecosystems and resources in the coastal environment, with the purpose of providing the basis for:

  (1) Development of a coastal strategy(ies) within five years to address the management of the coastal environment in Canterbury

  (2) Consequential changes to the Canterbury Regional Policy Statement, any relevant regional coastal plan(s) and district plans.
- Objective 11.2.3 – Climate change and natural hazards

  The effects of climate change, and its influence on sea levels and the frequency and severity of natural hazards, are recognised and provided for.
- Policy 11.3.5 – General risk management approach

  Subdivision, use or development of land shall be avoided if the risk from natural hazards is unacceptable. When determining whether risk is unacceptable, the following matters will be considered:

  (1) the likelihood of the natural hazard event

(2) the potential consequence of the natural hazard event for: people and communities, property, infrastructure and the environment, and the emergency response organisations. Where there is uncertainty in the likelihood or consequences of a natural hazard event, the local authority shall adopt a precautionary approach.

- Policy 11.3.8 – Climate change

  When considering natural hazards, and in determining if new subdivision, use or development is appropriate and sustainable in relation to the potential risks from natural hazard events, local authorities shall have particular regard to the effects of climate change.

### 2.1.3 Regional Coastal Environment Plan for the Canterbury Region

The Regional Coastal Environment Plan for the Canterbury Region (RCEP) was made operative in 2005. The RCEP manages the natural and physical resources of the Canterbury coastal environment.

Chapter 9 of the RCEP covers coastal hazards and section 9.2 details the following policies regarding management of coastal hazards for the Canterbury coast:

- Policy 9.1

  (a) New habitable buildings should be located away from areas of the coastal environment that are or have the potential to be subject to sea water inundation or coastal erosion.

  (e) Natural features that buffer the effects of coastal hazards should be protected.

## 2.2 Water levels

Water levels play an important role in determining coastal erosion hazard both by controlling the amount of wave energy reaching the backshore causing erosion during storm events and by controlling the mean shoreline position on longer time scales (refer Figure 2-1).

Key components that determine water level are:

- Astronomical tides
- Barometric set-up and wind effects, generally referred to as storm surge
- Medium-term fluctuations, including seasonal effects, El Nino-Southern Oscillation (ENSO) and Inter-decadal Pacific Oscillation (IPO) effects commonly called mean sea level anomaly (MSLA)
- Predicted long-term changes in sea level due to climate change
- Onshore wave transformation processes through wave set-up and run-up.

All water levels shown in this report are presented in terms of Lyttelton Vertical Datum 1937 (expressed as “RL m”), unless stated otherwise.

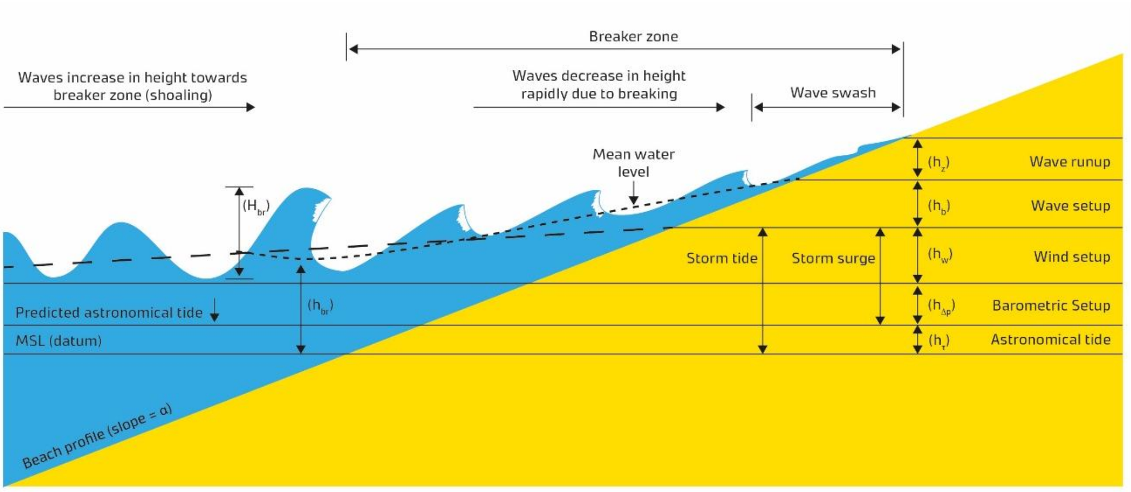


*Figure 2-1 Schematic illustrating sources of coastal-storm inundation*

## 2.2.1 Astronomical tide

The astronomical tides are caused by the gravitational attraction of solar-system bodies, primarily the Sun and Earth's moon. These forces result in ocean long waves interacting with the continental shelf in a complex way to produce a rise and fall in sea levels (tides). In New Zealand the astronomical tides have the largest influence on sea level.

Tidal levels for New Zealand ports are provided by Land Information New Zealand (LINZ) based on the average predicted values over the 18.6 year tidal cycle. Values for Lyttelton are presented within Table 2-1 in terms of Chart Datum, Lyttelton Vertical Datum 1937 (RL m) and CCC Datum. The values in terms of Chart Datum and CCC Datum have been included as a reference, but are not used in the remaining sections of the report. The spring tidal range is approximately 2.4 m and the mean sea level is around RL 0.2 m.

Table 2-1: Tidal levels at Lyttelton Harbour

| Tide state | Lyttelton Chart Datum (m) | Lyttelton Vertical Datum 1937 (RL m) | CCC Datum (m) |
|---|---|---|---|
| Highest Astronomical Tide (HAT) | 2.71 | 1.47 | 10.51 |
| Mean High Water Springs (MHWS) | 2.6 | 1.36 | 10.40 |
| Mean Sea Level (MSL) | 1.41 | 0.17 | 9.21 |
| Mean Low Water Springs (MLWS) | 0.20 | -1.04 | 8.0 |
| Lowest Astronomical Tide (LAT) | 0.17 | -1.07 | 7.97 |
| Source: LINZ (2017) | | | |

Lyttelton Chart Datum to Lyttelton Vertical Datum conversion is -1.24 m (LINZ, 2017). Lyttelton Vertical Datum to CCC Datum conversion is +9.04 m (NIWA, 2011)

## 2.2.2 Storm surge

Storm surge results from the combination of barometric set-up from low atmospheric pressure and wind stress from winds blowing along or onshore which elevates the water level above the predicted tide (refer Figure 2-1). Storm surge applies to the general elevation of the sea above the predicted tide across a region but excludes nearshore effects of storm waves such as wave set-up and wave run-up at the shoreline.

### 2.2.3 Medium-term fluctuations and cycles

Atmospheric factors such as season, ENSO and IPO can all affect the mean level of the sea at a specific time (refer to Figure 2-2). The combined effect of these fluctuations may be up to 0.25 m according to Bell (2012).

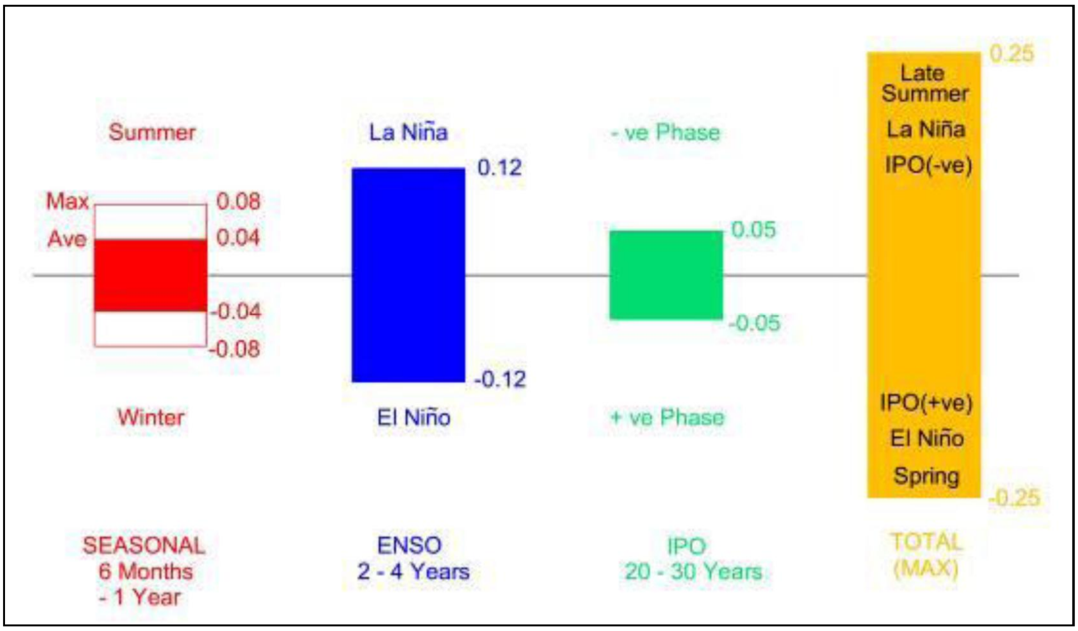


*Figure 2-2: Components contributing to sea level variation over medium to long-term periods (source: Bell, 2012)*

### 2.2.4 Storm tide levels

The combined elevation of the predicted tide, storm surge and medium-term fluctuations is known as the storm tide. The 1% and 2% Annual Exceedance Probability (AEP) storm tide for the open coast including Sumner are predicted by Stephens et al. (2015) and are RL 1.85 m and RL 1.83 m respectively based on a MSL baseline between 1986 and 2005. Storm tide levels for the Lyttelton Harbour and Akaroa Harbour have not been assessed by Stephens et al. (2015).

The storm tide at the port of Lyttelton has been calculated by Goring (2009) and is based on the Lyttelton tide gauge data (1998-2009) using the Empirical Simulation Technique (EST). The 1% and 2% AEP storm tides are RL 1.92 m and RL 1.87 m respectively (Goring, 2009).

No long-term tide gauge data exists for Akaroa although the New Zealand Nautical Almanac (LINZ, 2016) shows MHWS at French Bay and Tikao Bay to be similar with standard tide levels ranging from 0.1 m to 0.2 m higher than the Lyttelton Port MHWS. Therefore this study assumes that the storm tide levels presented in Table 2-2 for Lyttelton can be applied to the Akaroa Harbour sites due to similar astronomic tides and estuary geometry.

Table 2-2: Extreme storm tide for open coast and Lyttelton

| Site | Storm tide level (RL m) | |
|---|---|---|
| | 1% AEP | 2% AEP |
| Lyttelton[1] | 1.92 | 1.87 |
| Open coast and Sumner[2] | 1.85 | 1.83 |

[1]Source: Goring (2009)
[2]Source: Stephens et al. (2015)

### 2.2.5 Long-term sea levels

Historic sea level rise in New Zealand has averaged 1.7 ± 0.1 mm/yr with Christchurch exhibiting a slightly higher rate of 2.0 ± 0.15 mm/yr (Hannah and Bell, 2012). Climate change is predicted to accelerate this rate of sea level rise into the future. NZCPS (2010) requires that the identification of coastal hazards includes consideration of climate change effects, including accelerated sea level rise over at least a 100 year planning period (i.e. 2120). Potential sea level rise over this time frame is likely to significantly alter the coastal erosion hazard (e.g. due to affected coastal processes, coastal water levels).

Modelling presented within the most recent IPCC report (IPCC, 2013) show predicted global sea level rise values by 2100 to range from 0.27 m, which is slightly above the current rate of rise, to 1 m depending on the emission scenario adopted. The RCP2.6 scenario assumes very low greenhouse gas concentration levels by 2100 after first reaching high levels by 2050. The RCP4.5 scenario is a 'stabilization' scenario in which emissions are stabilized shortly after 2100 without exceeding. The RCP8.5 scenario assumes a high rate of emissions continue to rise in the 21st century.

The Ministry of Environment (MfE, 2008) guideline recommends a base value sea level rise of 0.5 m by 2090 (relative to the 1980-1999 average), with consideration of the consequences of sea level rise of at least 0.8 m by 2090 with an additional sea level rise of 10 mm/yr. beyond 2100. Bell (2013) and T+T (2013) recommend that for planning to 2115, these values are increased to 0.7 and 1.0 m respectively. Bell (2013) also recommends that when planning for new activities or developments, that higher potential rises of 1.5 to 2 m above the present mean sea level should be considered to cover the predicted climate change effects beyond a 100 year period.

We have used four sea level rise scenarios that are based around three RCP scenarios derived from IPCC (2013). These are the median projections of the RCP2.6, RCP4.5 and RCP8.5, and (RCP8.5+) the upper end of the 'likely range' (i.e. 83$^{rd}$ percentile) of the RCP8.5 projection. The global-average projections of the potential future scenarios (RCP2.6, RCP4.5, RCP8.5 and RCP8.5+) have been plotted by Stephens (2015) derived from Church et al. (2012), and are shown in Figure 2-3 and Table 2-3 shows the specific values used for the two time periods.

Table 2-3: Sea level rise projections from the 1986-2005 baseline for the four emission scenarios

| Year | RCP2.6 M | RCP4.5 M | RCP8.5M | RCP8.5+ |
|---|---|---|---|---|
| 2065 | 0.30 | 0.33 | 0.41 | 0.55 |
| 2120 | 0.55 | 0.67 | 1.06 | 1.36 |

### 2.2.6 Wave effects

Wave effects include wave set-up and wave run-up. Wave set-up is a local elevation in the mean water level on the foreshore, caused by the reduction in wave height through the surf-zone. Wave run-up is the sum of the wave set-up and the wave swash and is the maximum level that the waves reach on the beach relative to the still water level.

An indicator of wave run-up is recorded within the ECan beach profile dataset (i.e. storm debris line). Three significant storm events have occurred during the beach profile dataset period of 25 years in 1992, 2001 and 2014. Driftwood and storm debris line elevations were surveyed after these storm events, which ranged from RL 2.58 m to 2.8 m. The upper elevation relates to a wave run-up level range of approximately 1.1 m to 1.4 m, based on a tide level of RL 1.7 m (2% AEP Sumner Head) and RL 1.4 m (HAT Sumner Head) respectively.

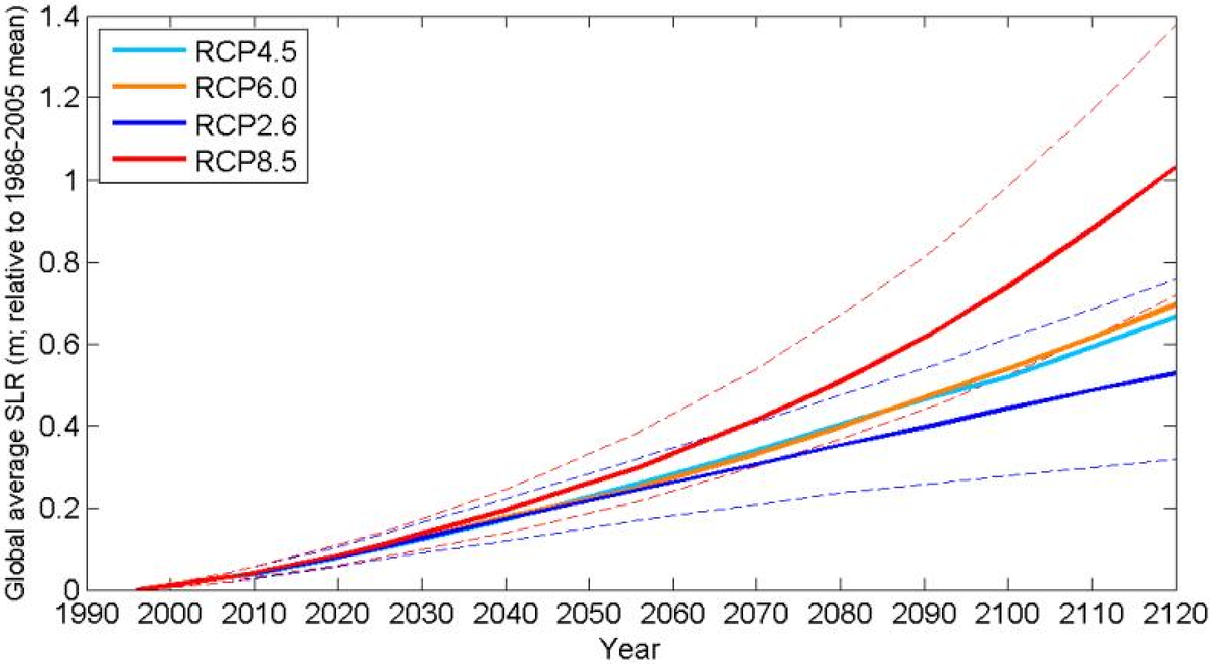


*Figure 2-3: Global-average sea level rise projection trajectories including median projections of RCP2.6, RCP4.5, RCP6.0 and RCP8.5 (solid lines), including 'likely ranges' (dashed lines). The (RCP8.5+) projection is the upper end of the 'likely range' (i.e. 83rd percentile) of the RCP8.5 projection (upper red dashed line). Source: Stephens (2015).*

## 2.3 Waimakariri River sediment supply

This section sets out our review of sediment budget papers, distribution of sediment transport and the effect of long-term transport trends to be used in the probabilistic assessment of coastal erosion.

The Christchurch coast is currently supplied with sand-sized sediment predominantly by local rivers. The most substantial river sediment input comes from the Waimakariri River, which discharges to the coast north of the city, with sediments carried south to nourish the New Brighton coastline via longshore drift.

This longshore drift is driven by remotely- and locally-generated wind waves as well as an eddy of the current that typically flows northward along the east coast of the South Island of New Zealand, but which is reversed in the lee of Banks Peninsula due to sheltering and wave refraction processes (Reynolds-Fleming & Fleming, 2005).

The open coast beaches have historically been naturally replenished with sediment supplied from rivers, with the Waimakariri River responsible for the largest supply (around 77% based on Griffiths and Glasby's figures in Hicks, 1998) and it is expected to be the main supplier of beach sediment to the Christchurch coast based on historic coastal processes. The amount of beach sand-sized sediment supplied by the river has been estimated to be around 20% of the total load (Hicks, 1998).

Hicks (1998) estimated that the supply of sediment to Pegasus Bay coast by the Waimakariri River to be between 360,000 to 1,270,000 $m^3$/yr., of which around half was thought to be carried by longshore currents southwards to nourish the beaches between the mouth of the Waimakariri River and Banks Peninsula (i.e. between 180,000 $m^3$/yr. to 635,000$m^3$/yr.). There is a wide range in possible sand supply. This is due to the wide range of published estimates of suspended load estimates and the assumptions of how much of this suspended load is of a size and density that it will end up in the beach system rather than in the offshore environment. We note Hicks (1998) preferred values towards the lower end of the range of estimates as there may be additional trapping of suspended load in the lower reaches of the river. Our comparison of measured beach change data supports the assumption of less, rather than more, sediment entering the beach system from the Waimakariri River.

Hicks (1998) research represents the most recent quantitative assessment available, although comparisons with earlier work on this topic by Duns (1995) and Kirk (1979) indicate a level of uncertainty in the Pegasus Bay sediment budget. This uncertainty grows when considering that wind and wave climates could undergo directional shifts with future climate changes as well as possible anthropogenic effects.

Based on our knowledge of sediment supply on gravel and mixed-sand gravel river systems in the Hawke's Bay (Tukituki River) and along the Eastbourne Coast in Wellington Harbour (Olsen, 2009) that have been affected by earthquake events, we are aware there can be a considerable time lag between the increase in catchment related supply in the upper reaches of the catchment and it possibly reaching the coast. We postulate that this time lag provides a possible proxy for potential catchment related effects as a result of future climate change.

For the Tukituki River catchment aggradation is occurring at the foothills of the Herataunga Plains, but is not reaching the coast. This may be due to the lowering of the plains as a result of the 1931 Hawke's Bay earthquake changing the hydraulic grade and the recognition of a longer timescale required before the material from the catchment can reach the coast. As the Orongorongo River connects more directly from the hills to the coast, there has been a gradual change from a sandy coast to a gravel coast along the eastern shoreline of Wellington Harbour. The Wairarapa earthquake occurred in 1855 causing landslides and infilling of the steep valley catchments. The resulting increase in sediment supply to the coast caused sediment to move slowly along a 10 m wide strip of the open coast, reaching the entrance to the harbour in 1941 some 85 years later. The time lag for catchment related changes to be seen on the coast was also recognised in the assessment of the effects of dams on the Waitaki River (Hicks and Todd, 2003).

This suggests that even if increased rainfall is predicted as a result of climate change there may be a significant lag before the increased rainfall results in a possible increase in sediment supply to the river system from the catchment.

# 3 New data collection

## 3.1 Shorelines

Digitised historic shorelines have been provided by ECan covering a time period between 1941 and 2011 (refer to Table 3-1). This set of shoreline information provides a total of five time-periods spaced approximately every 15 to 20 years for analysing long-term trends over a 69 year period (1942–2011).

The historic shorelines are based on digitising the shoreline proxy, taken to be the seaward edge of dune vegetation, from geo-referenced historic aerial photographs. The seaward edge of the dune vegetation was digitised to represent the dune toe, which was taken as the shoreline proxy. This shoreline proxy was chosen because the change in contrast from dune vegetation to beach sand can more accurately be identified on the historic black and white aerial photographs rather than the water line.

Table 3-1: Summary of aerial photographs input dataset

| Date captured | Run number | Source | Scale |
|---|---|---|---|
| 14/10/1941 | SN 152 | NZAM Aerial Photograph | 1:16,000 |
| 10/05/1955 | SN 872 | NZAM Aerial Photograph | 1:16,000 |
| 22/08/1979 | SN 5468 | NZAM Aerial Photograph | 1:24,000 |
| 08/03/2001 | SN 50038c | NZAM Aerial Photograph | 1:24,000 |
| 24/02/2011 | SN 521 | NZAM Aerial Photograph | 1:24,000 |

The shoreline data digitised from aerial photographs was verified against the source information by T+T. Verification and quality control focused on the accuracy of the shoreline proxy representation including the position and frequency of the polyline nodes. The geo-referencing of the historic aerial photographs was independently checked over a minimum of three ground control points (GCP) to verify the horizontal accuracy.

Three potential measurement errors have been estimated for the historic shoreline position:

- The geo-referencing error (Er) represents the potential offset of an image from a known point based on ground control points collected during the geo-referencing process
- The digitising error (Ed) represents the potential operator inconsistency in digitising a shoreline using ArcGIS software
- Shoreline proxy error (Es) is the estimated uncertainty in identifying the shoreline, which is more for black and white images. Example of features that cause shoreline proxy error include scale, shadow, overhanging trees and the uncertainty in identifying the correct dune vegetation edge based on black and white contrast.

Refer to Table 3-2 for a summary of the estimated shoreline data error values. The resultant potential error in shoreline position can be calculated as between 2 and 4 m (0.025 and 0.05 m/yr) using a sum of independent errors approach whereby:

$$E_{sum} = \sqrt{E_{\mathrm{r}}^2 + E_{\mathrm{d}}^2 + E_s^2} \qquad \text{(Equation 1)}$$

Table 3-2: Shoreline data error summary

| Potential measurement error (metres) | Data type | |
|---|---|---|
| | A | B |
| Geo-referencing error (Er)<br>Digitising error (Ed)<br>Shoreline proxy error (Es) | 1<br>1<br>2 | 2<br>1<br>3 |
| Total potential error (Et) (metres) | 2.45 | 3.7 |
| *Rounded* | *2 m* | *4 m* |
| Data Type A = post 2000 aerial source, B = pre 2000 aerial source | | |

## 3.2 Beach profiles

The natural cross-shore beach profile is expected to fluctuate in response to changes in the beach processes and sediment supply. ECan has undertaken regular beach profile surveys along southern Pegasus Bay between 1990 and 2016. Profiles C0362 and C0431-C0703 have also been surveyed once or twice in 1978, with no surveys undertaken between 1979 and 1990. Profiles C0853 – C0856 (2004), C0863 (2000) and C1111 (2008) have been surveyed for approximately 10 years. The surveys are captured with a Leica TCA1100L total station in conjunction with a Sokkia prism survey pole. The vertical and horizontal accuracy is ±30 mm. The surveys are completed twice a year, in summer and winter, and the cross-shore extent includes the backdune out to at least mean sea level. A summary of the beach profile data is presented in Table 3-3. Refer to Appendix B for beach profiles cross-sections of the 9/5/1990 and 9/8/2016 surveys, the minimum, average and maximum envelope profiles and Appendix E (Figure E1-E6) for a site plan for a location plan of beach profile positions.

Table 3-3: Summary of beach profile data for southern Pegasus Bay

| Beach profile description | | First survey date | Last survey date | Survey period (years) | Number of surveys |
|---|---|---|---|---|---|
| Code | Name | | | | |
| C1130 | Waimairi Beach (Larnach Street) | 9/05/1990 | 9/08/2016 | 26 | 54 |
| C1111 | Waimairi Beach (Beach Road) | 7/08/2008 | 11/08/2016 | 8 | 16 |
| C1100 | North New Brighton (Pandora Street) | 9/05/1990 | 11/08/2016 | 26 | 53 |
| C1086 | North New Brighton (Pacific Road) | 9/05/1990 | 11/08/2016 | 26 | 56 |
| C1065 | North New Brighton (Effingham Street) | 9/05/1990 | 11/08/2016 | 26 | 53 |
| C1041 | North New Brighton (Cygnet Street) | 9/05/1990 | 11/08/2016 | 26 | 53 |
| C1011 | North New Brighton (Bowhill Road) | 9/05/1990 | 11/08/2016 | 26 | 53 |
| C0952 | New Brighton (Rawhiti Street) | 9/05/1990 | 11/08/2016 | 26 | 55 |
| C0924 | New Brighton (Lonsdale Street) | 9/05/1990 | 11/08/2016 | 26 | 52 |
| C0889 | New Brighton (Hawke Street) | 9/05/1990 | 20/07/2016 | 26 | 54 |
| C0863 | New Brighton (226 Marine Parade) | 1/12/2000 | 8/08/2016 | 16 | 31 |
| C0856 | New Brighton (231 Marine Parade) | 21/07/2004 | 8/08/2016 | 12 | 24 |
| C0853 | New Brighton (233 Marine Parade) | 21/07/2004 | 8/08/2016 | 12 | 24 |
| C0848 | New Brighton (Hood Street) | 9/05/1990 | 8/08/2016 | 26 | 54 |
| C0815 | New Brighton (Rodney Street) | 9/05/1990 | 8/08/2016 | 26 | 54 |
| C0781 | New Brighton (Mountbatten Street) | 9/05/1990 | 20/07/2016 | 26 | 53 |

| Beach profile description | | First survey date | Last survey date | Survey period (years) | Number of surveys |
|---|---|---|---|---|---|
| Code | Name | | | | |
| C0748 | South New Brighton (Jervois Street) | 9/05/1990 | 20/07/2016 | 26 | 54 |
| C0703 | South New Brighton (Bridge Street) | 1/08/1978 | 20/07/2016 | 38 | 54 |
| C0650 | South New Brighton (Beatty Street) | 1/08/1978 | 20/07/2016 | 38 | 54 |
| C0600 | South New Brighton (Jellicoe Street) | 1/08/1978 | 18/07/2016 | 38 | 50 |
| C0531 | South New Brighton (Halsey Street) | 19/12/1978 | 18/07/2016 | 38 | 54 |
| C0513 | South New Brighton (Caspian Street) | 18/12/1978 | 18/07/2016 | 38 | 56 |
| C0471 | South Shore (Heron Street) | 31/07/1978 | 20/07/2016 | 38 | 55 |
| C0431 | Southshore (Penguin Street) | 1/08/1978 | 20/07/2016 | 38 | 56 |
| C0396 | Southshore (Plover Street) | 9/05/1990 | 19/07/2016 | 26 | 54 |
| C0362 | Southshore (Tern Street) | 1/08/1978 | 20/07/2016 | 38 | 55 |
| C0350 | Southshore (Torea Street) | 9/05/1990 | 20/07/2016 | 26 | 53 |
| C0300 | Southshore (South of Pukeko Place) | 9/05/1990 | 19/07/2016 | 26 | 57 |
| C0271 | Southshore (End Rockinghorse Road) | 9/05/1990 | 19/07/2016 | 26 | 58 |

## 3.3 LiDAR

Council sourced LiDAR data was processed in GIS using ArcGIS software (Spatial Analyst Licence) to form a digital elevation model (DTM). The LiDAR survey was undertaken in 2011 after the 2010-2011 Canterbury Earthquakes (Table 3-4). Metadata supplied with the source LiDAR indicates the survey equipment had a vertical accuracy ±0.07 m. The generated DTM has a grid cell size of 2 m by 2 m. Dune crest elevations were extracted from the DTM as a 3D polyline along the dune crest alignment using standard transect methods with a node spacing of 2 m. LiDAR was also used to establish the elevation of the dune toe for both sites. This information is required for the shoreline change analysis of the beach profile datasets.

Table 3-4: LiDAR source and commissioning agencies

| DEM | Source LiDAR | Commissioning Agencies |
|---|---|---|
| Post-Sept 2010 | NZAM, 5 Sep 2010 | Ministry of Civil Defence and Emergency Management |
| Post-Feb 2011 | NZAM, 8-10 Mar 2011<br>AAM, 20-30 May 2011 | Ministry of Civil Defence and Emergency Management<br>Christchurch City Council |
| Post-June 2011 | NZAM, 18 & 20 Jul, 11 Aug, 25-27 Aug, and 2-3 Sep 2011 | Earthquake Commission |
| Post-Dec 2011 | NZAM 17-18 Feb, 2012 | Earthquake Commission |

# 4 Re-assessment methodology

Due to the different processes driving the open coast and harbour environments different methodologies were implemented to assess coastal hazards. Data used for determination of the open coast inundation and erosion hazard zones are summarised in Section 4.1 and 4.2 respectively. The harbour coast inundation and erosion hazard zone determination methodologies are outlined in Sections 4.3 and 4.4 respectively. Sumner also required a site specific approach due to both its location partly within the harbour, estuary entrance and open coast, and the presence of the Sumner rock revetment along a large length of the coastline. The assessment methodology for both inundation and erosion for Sumner is outlined in Section4.5.

## 4.1 Open coast coastal inundation hazard zone (CIHZ)

Prior to the T+T (2015b) study there was no existing coastal inundation hazard identified for the open coast. The coastal inundation level in this study was based on the combination of the following components (refer to Section 2.2):

- Storm tide based on extreme event analysis of the Sumner Head tide gauge
- Wave set-up using empirical relationship included in the Coastal Engineering Manual (USCAE, 2006)
- Sea level rise based on extrapolation of the four selected RCP emission scenarios.

Wave set-up was calculated based on the Coastal Engineering Manual method (CEM, II-4-3). This method takes into account the wave climate and beach slope. Table 4-1 outlines the input wave climate and beach slope parameters used for this assessment.

Table 4-1: Input parameters used for the wave set-up calculations

| Site | Deep water wave height ($H_o$)[1] | Deep water wave period ($T_o$)[1] | Beach slope (tan**β**)[2] |
|---|---|---|---|
| | 1% AEP | 1% AEP | |
| New Brighton | 5.98 m | 14.45 sec | 0.01 |
| Sumner | 5.32 m | 12.75 sec | 0.006 |
| Taylor's Mistake | 5.32 m | 12.75 sec | 0.01 |

Notes:
[1]Wave climate data sourced from T+T (1998)
[2]Beach slope taken from the break point.

Table 4-2 summarises the storm inundation component values used to calculate the CIHZ levels for the open coast. The total CIHZ level is calculated by summing the three inundation component values using a "building-block" approach. This approach represents a conservative upper bound of the inundation hazard. The maximum CIHZ level at the three sites for the 2065 and 2120 time frames are RL 3.9 m and RL 4.7 m respectively. These levels are considered to be generally in line with previous reporting (T+T, 1998) although generally higher (i.e. 500 mm higher) than the observed upper levels of storm debris of approximately RL 2.8 m recorded since 1990 (refer to Section 2.2.6).

The sea level rise (SLR) values shown in Table 4-2 are based on a sea level averaged between 1986 and 2005. The storm tides derived by Stephens et al. (2015) are based on mean sea level baseline between 1986 and 2005. A rise in sea levels between these periods of 2.5 to 3.5 cm should therefore be discounted. However, because the total CIHZ levels are rounded to 1 decimal place (i.e. the nearest 0.1 m), discounting the sea level rise values (<0.035 m) will not change the total CIHZ levels.

Table 4-2: Coastal Inundation Hazard components values

| Site | Time period | Storm Tide (m RL) | Wave set-up (m) | RCP2.6 M | | RCP4.5 M | | RCP8.5 M | | RCP8.5+ | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | SLR (m) | Total CIHZ level (RL m) | SLR (m) | Total CIHZ level (RL m) | SLR (m) | Total CIHZ level (RL m) | SLR (m) | Total CIHZ level (RL m) |
| New Brighton | 2065 | 1.85 | 1.49 | 0.3 | 3.6 | 0.33 | 3.7 | 0.41 | 3.8 | 0.55 | 3.9 |
| | 2120 | 1.85 | 1.53 | 0.55 | 3.9 | 0.67 | 4.1 | 1 | 4.4 | 1.36 | 4.7 |
| Sumner | 2065 | 1.85 | 1.27 | 0.3 | 3.4 | 0.33 | 3.5 | 0.41 | 3.5 | 0.55 | 3.7 |
| | 2120 | 1.85 | 1.31 | 0.55 | 3.7 | 0.67 | 3.8 | 1 | 4.2 | 1.36 | 4.5 |
| Taylor’s Mistake | 2065 | 1.85 | 1.29 | 0.3 | 3.4 | 0.33 | 3.5 | 0.41 | 3.6 | 0.55 | 3.7 |
| | 2120 | 1.85 | 1.33 | 0.55 | 3.7 | 0.67 | 3.9 | 1 | 4.2 | 1.36 | 4.5 |
| All levels reduced to Lyttelton Datum 1937 (LVD-1937) and rounded to 1 decimal place | | | | | | | | | | | |

The inundation zones (CIHZ) were mapped by extrapolating the total inundation level inland based on a digital elevation model (DEM) derived from LiDAR surveyed post the 2010-2011 Canterbury Earthquakes. The CIHZ maps are presented in Appendix I. For both Taylor’s Mistake and Sumner inundation pathways exist landward. However, the elevation of the foredunes located along the open coast shoreline from Waimairi to the Avon-Heathcote Estuary mouth are generally sufficient to mitigate the coastal inundation hazard. There are two sites at New Brighton where the foredunes have been modified and inundation pathways exist through the foredunes:

- New Brighton Library
- North New Brighton Community Centre and North Beach Surf Lifesaving Club.

The inundation pathways at both sites are relatively narrow and the quantity of inundation will be affected by tide levels and friction. Therefore the volume of water able to propagate inland will be restricted. It is expected that the inundation level will decrease inland further away from the shoreline due to the limited volume of seawater able to pass through the pathway within the time period of a typical storm event. This was quantified by making use of the hydrodynamic model that was developed for inundation prediction within the Avon-Heathcote Estuary and the Brooklands Lagoon areas.

## 4.2 Open coast coastal erosion hazard zone (CEHZ)

The method for the open coast that extends from Waimairi Beach to the Avon-Heathcote Estuary (excluding the South Brighton Spit) involved dividing the open coast into a series of cells where the shorelines observed long-term behaviour can be characterised as being reasonably similar and then calculating the potential extent of erosion considering the erosion drivers. The methodology for Sumner and the South Brighton Spit are described in Section 4.5.2 and Section 4.2.6.

### 4.2.1 Defining coastal behaviour cells

The open coast was divided into eight coastal cells (A-G) based on shoreline composition and behaviour which can influence the resultant hazard. Factors which may influence the behaviour of a cell include:

- cell morphology
- profile geometry
- backshore elevation
- historic shoreline trends.

All the open coast cells have a similar morphology with a dune backshore and a relatively flat fine sand beach. Cell G represents the distal end of the New Brighton spit where the shoreline has historically fluctuated and the morphology is relatively low-lying. The main influence on the cell division along the open coast is the historic shoreline trends (refer to Table 4-3 for a summary of the cell divisions).

All cells have experienced accretion over the long-term with the highest rates occurring at the north and south extents of the open coast site (i.e. cells A, F and G). The lowest rates of accretion have occurred at cell B where the backshore has been modified and carparks, structures and other public access routes have altered the dune morphology. Some areas along cell B have minimal established dune vegetation, which reduces the dune capacity to trap wind-blown sand and accrete seaward. Refer to Section 4.2.3.5 for a full description of the components values adopted for each cell.

Table 4-3: Summary of the behaviour cell characteristics for the open coast

| Site | Christchurch open coast | | | | | | |
|---|---|---|---|---|---|---|---|
| Cell | A | B | C | D | E | F | G |
| Chainage[1], (m) | 0-1950 | 1950-3700 | 3700-5200 | 5200-6300 | 6300-7250 | 7250-8650 | 8650-9600 |
| ECan beach profiles within each cell | C1130<br>C1111<br>C1100<br>C1086<br>C1065<br>C1041 | C1011<br>C0952<br>C0924<br>C0889<br>C0863<br>C0856<br>C0853 | C0848<br>C0815<br>C0781<br>C0748 | C0703<br>C0650<br>C0600 | C0531<br>C0513 | C0471<br>C0431<br>C0396<br>C0362 | C0350<br>C0300<br>C0271 |
| Morphology | Unmodified dune backshore | Modified dune backshore | Unmodified dune backshore | Unmodified dune backshore | Unmodified dune backshore | Unmodified dune backshore | Low-lying distal spit backshore |
| Historic shoreline movement | Accretion (high) | Accretion (low) | Accretion (average) | Accretion (average) | Accretion (average) | Accretion (high) | Fluctuates |

[1] Chainage is a distance measure from the origin taken as the start of cell A at E1577557m N5186179m (NZTM)

### 4.2.2 Coastal Erosion Hazard parameters

The CEHZs were established from the cumulative effect of four main parameters as indicated in Figure 4-1:

$$CEHZ = SC + DS + (LT \times T) + SL \qquad \text{(Equation 2)}$$

Where:

SC = Storm cut/Short-term erosion defined by the horizontal storm cut distance (m).

DS = Dune slope is characterized by the horizontal distance from the base of the eroded dune to the crest of a stable angle of repose (m).

LT = Long-term rate of horizontal coastline movement (m/yr.).

T = Time frame (years). In this instance a period of 48 and 103 years will be used for CEHZ2065 and CEHZ2120 respectively (i.e. approximately 50 and 100 years).

SL = Horizontal coastline retreat due to possible accelerated sea level rise (m).

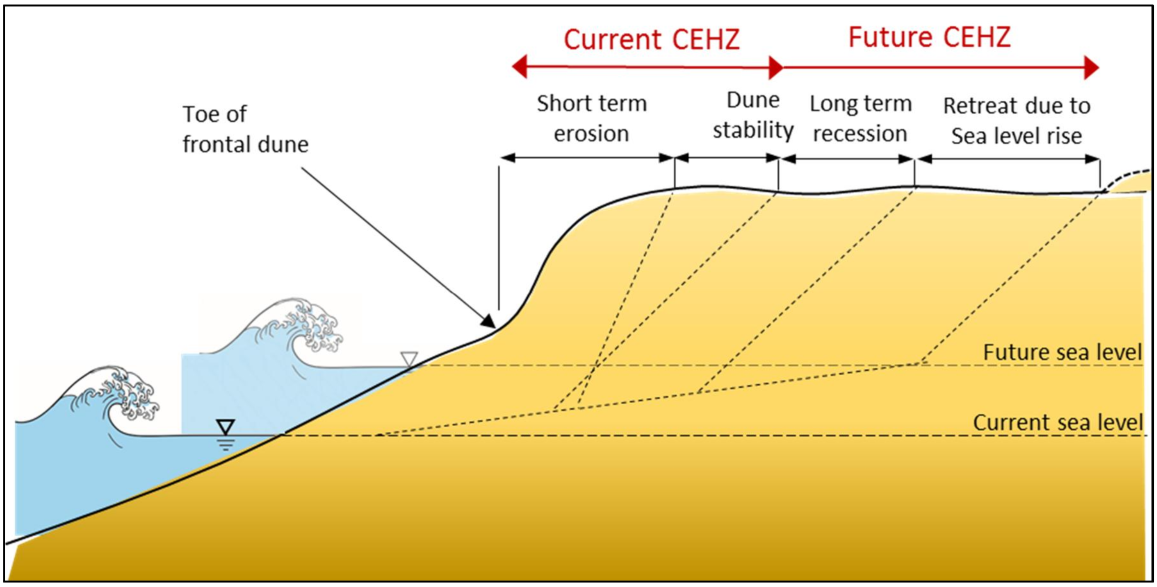


*Figure 4-1: Definition sketch for open coast CEHZ*

The figure illustrates that erosion hazard over a particular time period and at any particular beach shoreline comprises the combination of short-term erosion (storm cut) and the associated instability of the land area behind the dune (DS) with the observed historic long-term behaviour of the beach (LT) and an allowance of future change resulting from expected sea level rise (SL).

The CEHZ baseline to which values are referenced is the most recent dune toe derived from aerial photographs captured in 2011, except where the dynamic spit shoreline begins to fluctuate south of Tern Street. In the dynamic spit area the baseline was taken as the most inland extent of fluctuation (envelope) based on the historic photograph analysis.

The Envirolink guide to good practice (Ramsey et al, 2012) recommends moving from deterministic predictions to probabilistic projections, and that the recognition and treatment of uncertainty is a key source of variance between CHZ predictions by practitioners. We have adopted a probabilistic approach which is consistent with the Envirolink guide, and includes the following steps:

- Use probability distribution functions in the form of a normal distribution, a triangular distribution or extreme value distribution (see Figure 4-2). A normal distribution should be used where sufficient data is available and this data is (near) normally distributed. A triangular distribution should be used where limited data is available. The triangular distribution contains the best estimate (mode), lower and upper bounds of the four components (excludes T which is fixed) based on either available data or heuristic reasoning based on experience. An extreme value distribution should be used where extreme values are not included in the available data and should be included.

- Probability distributions constructed for each components are randomly sampled and the extracted values used to define a potential CEHZ distance. This process is repeated 10,000 times using a Monte Carlo technique. An example of a probability distribution of the resultant CHZ width is shown in Figure 4-3.
- Utilise the probabilistic distributions to map the range of CEHZ distances for each time frame and assign a pragmatic probability or likelihood for each CEHZ.

The probabilistic approach recognises there will always be inherent uncertainties associated with projections and provides a much more transparent way of capturing and presenting such uncertainty. We note that this method results in a range of potential hazard zone distances. The probabilistic method also aligns with risk assessment approach where the results can be aligned with a range of likelihood scenarios if required (refer to Section 4.2.5 and Table 4-13).

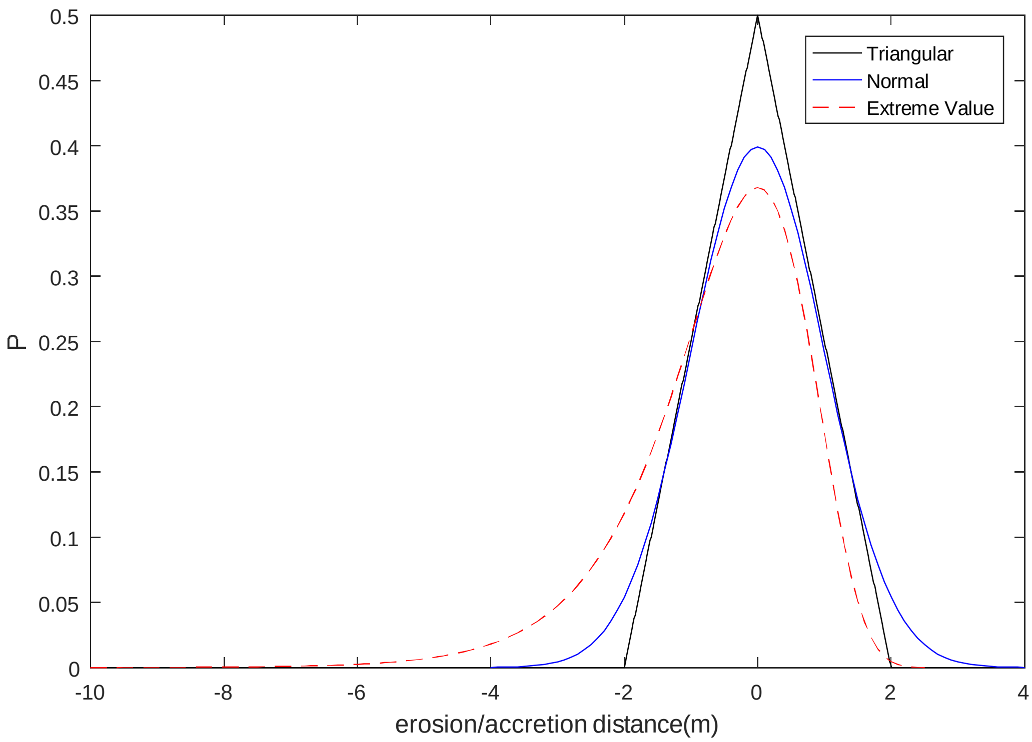


*Figure 4-2 Example of triangular, normal and extreme value distribution*

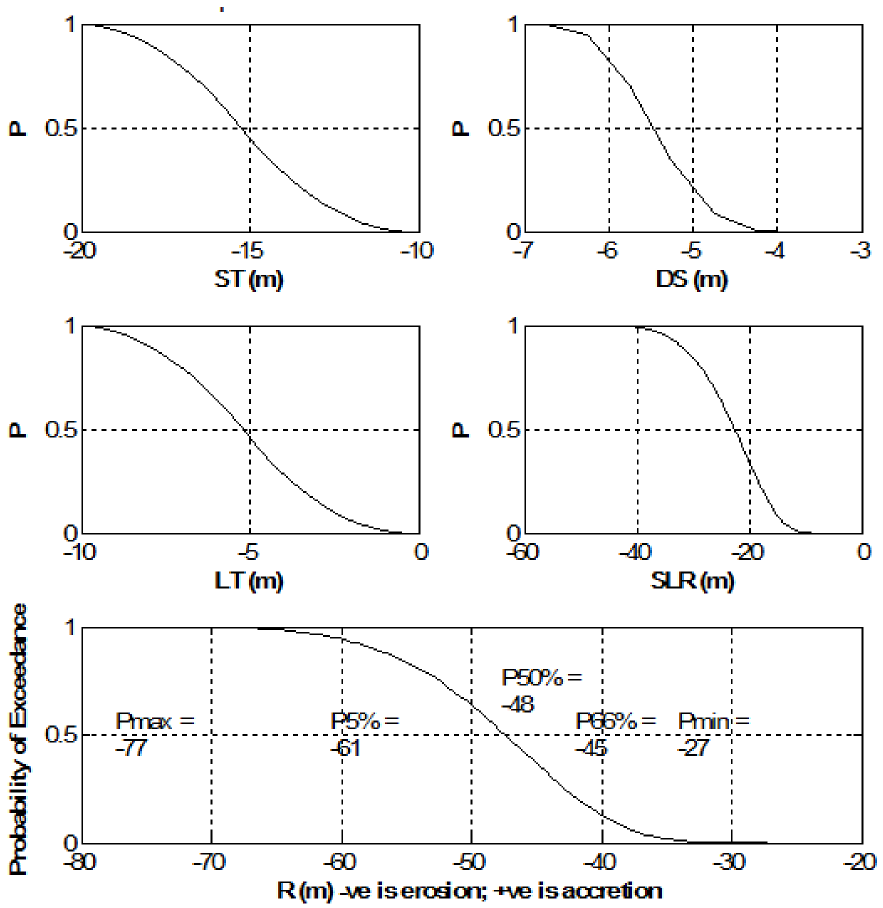


*Figure 4-3: Example of cumulative distribution functions of parameter samples and the resultant CHZ distances*

### 4.2.3 Component derivation

The CEHZ components identified in Section 4.2 and Equation 2 (refer Section 4.2.2) have been assessed for each behaviour cell and are described in the following sections below.

#### 4.2.3.1 Planning time frame (T)

Two time frames were applied to provide information on current hazards and information at appropriate time scales for considering future planning and development options:

- 2065 Coastal Erosion Hazard Zone (approx. 50 years): CEHZ2065
- 2120 Coastal Erosion Hazard Zone (approx. 100 years): CEHZ2120.

#### 4.2.3.2 Storm cut (SC)

Shorelines undergo short-term cycles of storm-induced erosion (i.e. storm cut) followed by periods of re-building. Where a coast experiences shoreline erosion (i.e. landward movements) due to single or clusters of storms, the short-term erosional component of the cycle needs to be accounted for in any coastal hazard assessment. The post-storm recovery, or accretional part of such cycles, does not need to be accounted for in this short-term (storm cut) component. This is because short-term accretion is not a local coastal hazard (such as on the Christchurch open coast) Long-term trends in accretion should already be accounted for in the long-term shoreline trend component (refer to Section 4.2.3.4).

Internationally, the short-term erosion hazard posed by storm cycle dynamics is typically achieved by including a ‘storm demand’ (volume) or ‘storm cut’ (horizontal transgression) component. Erosion volumes or horizontal transgression distances are estimated where possible from available data by assessing the effects of extreme storm events on the beach system. In this assessment, short-term shoreline movements were assessed by analysing the dune toe position from beach profile analysis.

#### 4.2.3.2.1 Methodology

Based on visual inspection of the beach profile data-base the dune toe level was estimated to be around 2.5 m RL. The horizontal movement of the dune toe based on the ECan beach profile analysis was used to assess the storm cut distance using inter-survey storm cut. A numerical model assessment of storm erosion potential was also undertaken, but found to underestimate the storm cut compared to measured data and therefore not used for this study (refer to Appendix C for numerical model assessment).

The inter-survey storm cut is the landward horizontal retreat distance measured between two consecutive surveys (i.e. distance between excursion distances). Figure 4-4 show measured excursion distances over time for profile C0848 (Hoods Street, New Brighton). We note that due to the relatively long period between surveys these distances may not represent the largest excursion that may have occurred between these time periods. However, the data set provides the best source of information to analyse.

It can be seen from Figure 4-4 that while the beach experienced net accretion, the shoreline fluctuates over time. The ongoing accretion is likely to be periodic, responding to pulses of sediment supplied from the Waimakiriri River (refer to Section 2.3). This periodic short-term trend is apparent between 1993 and 1995, where the shoreline built out approximately 14 m over a 2 year period (refer to Figure 4-4). This response may have also been a result of beach recovery from a series of prior coastal storms during the winter of 1992, which cumulatively caused significant erosion.

Periods of erosion caused by southerly storm events and tropical cyclone events are also apparent within the dataset, with erosion (i.e. storm cut distances) of up to 12 m occurring over a 2 year time period at New Brighton (e.g. C0952). At this location it can be seen that the dune toe experienced a much lower rate of accretion during the period between 2000 and 2016, with some beach profile sites recording net erosion over this 16 year period (e.g. C0748, refer to Appendix B, Figure B17).

The beach profile analysis results for all profiles are shown in Table 4-4 and show the mean and largest inter-survey storm cut. A full set excursion distances and profile plots for all profiles is presented in Appendix B.

The mean inter-survey storm cut ranges from -1.9 m to -5.6 m. The largest inter-survey storm cut ranges from -4.3 m to -22.2 m. The largest inter-survey storm cut (-22.2 m) is measured at profile C0889 and is situated adjacent to the Brighton Pier seawall. Due to the influence of the seawall and dune management works the shoreline levels in front and along the seawall are likely affected and therefore do not represent natural shoreline movements.

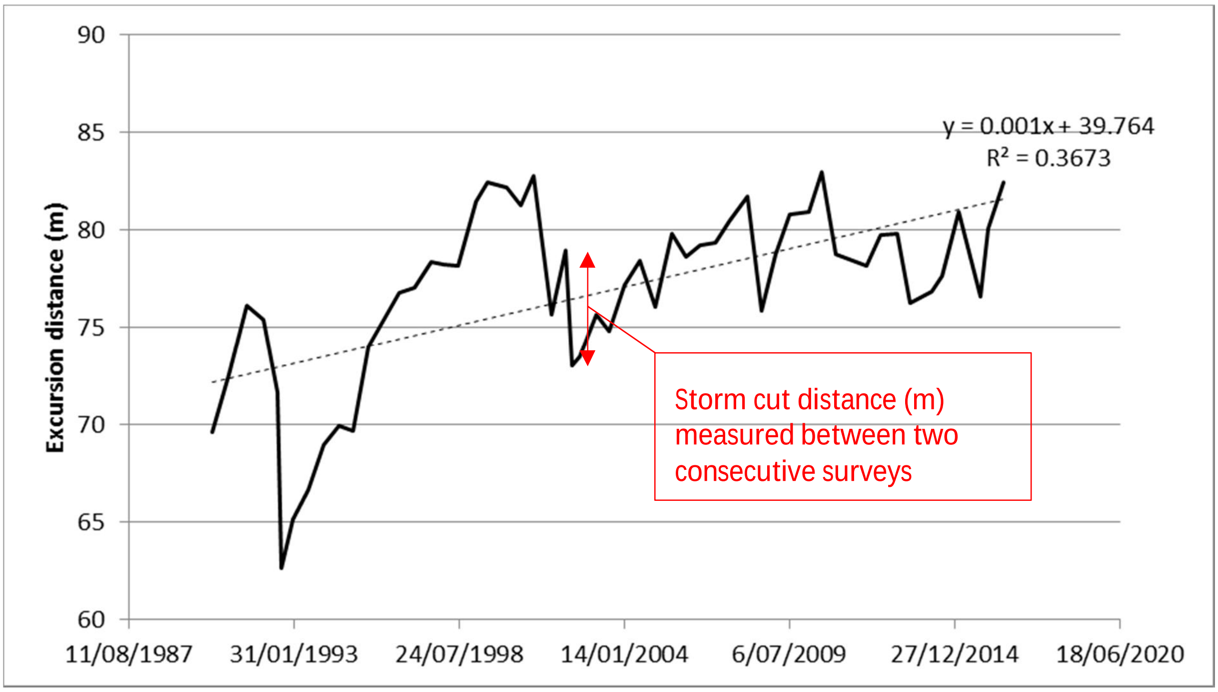


*Figure 4-4: Example of dune toe linear regression plot for C0848 Hood Street (New Brighton)*

Table 4-4: Mean and maximum inter-survey storm cut

| Beach Profile | Rate of change (m/year) | Mean inter-survey storm cut (m) | Largest inter-survey storm cut (m) |
|---|---|---|---|
| C1130 | 0.01 | -3.0 | -11.2 |
| C1111 | -0.15 | -2.4 | -4.3 |
| C1100 | 0.26 | -2.4 | -9.6 |
| C1086 | 0.37 | -2.9 | -9.8 |
| C1065 | 0.22 | -2.1 | -10.7 |
| C1041 | 0.15 | -2.8 | -10.4 |
| C1011 | -0.22 | -3.5 | -11.3 |
| C0952 | 0.11 | -2.5 | -10.6 |
| C0924 | 0.33 | -1.9 | -6.5 |
| C0889* | 0.29 | -5.6 | -22.2 |
| C0863* | 0.22 | -3.2 | -8.0 |
| C0856 | 0.07 | -1.9 | -6.7 |
| C0853 | -0.11 | -2.1 | -6.1 |
| C0848 | 0.37 | -2.8 | -9.0 |
| C0815 | 0.29 | -2.8 | -13.5 |
| C0781 | 0.44 | -2.5 | -9.2 |
| C0748 | 0.66 | -2.6 | -10.2 |
| C0703 | 0.58 | -3.6 | -10.7 |
| C0650 | 0.80 | -2.2 | -12.9 |
| C0600 | 0.80 | -2.8 | -9.9 |

| Beach Profile | Rate of change (m/year) | Mean inter-survey storm cut (m) | Largest inter-survey storm cut (m) |
|---|---|---|---|
| C0531 | 0.55 | -2.2 | -8.3 |
| C0513 | 0.69 | -2.2 | -9.4 |
| C0471 | 0.62 | -2.6 | -8.9 |
| C0431 | 0.58 | -3.0 | -8.0 |
| C0396 | 0.84 | -2.6 | -15.4 |
| C0362 | 1.24 | -2.6 | -12.4 |

*Profile situated adjacent to Brighton Pier seawall

#### 4.2.3.2.2 Adopted values and distribution

Appendix D shows a matrix (Table D1) of the inter-survey storm cut distances for each alongshore beach profile and for each survey date including a heat map (Figure D1). The table includes both the alongshore mean and maximum storm cut distance for each survey date. The storm cut distances measured at profiles CCC0863 and CCC0889 are likely influenced by the backshore seawall and have therefore been omitted from the analysis.

Although Kenderdine et al. (2016) stated that separate storm cut distributions should be applied to each cell as has been applied for the long-term components, examining Table D1 shows that the storm cut distances are similar along the open coast. Based on the values shown in Table D1 and considering that the exposure to wave action along the open coast (cell A-F) is approximately similar we have adopted a single distribution for the entire open coast shoreline (excluding Sumner).

The analysed storm cut distances are based on a 26 year dataset. In order to extrapolate extreme values derived from a limited number of observations (i.e. 26 years of 6-monthly surveys), an extreme value analyses have been undertaken. These have been carried out adopting the following distances:

1 alongshore-mean

2 alongshore-maximum.

For this approach the alongshore mean or maximum storm cut distance (refer to Table D1, Appendix D) for the entire dataset have been considered. This follows the rationale that for a given storm event a single profile may not detect the maximum erosion that has occurred along the beach (i.e. at a rip head) whereas the maximum erosion is more likely to be observed from a dataset including multiple alongshore profiles.

A range of data selection methods has been reviewed including Peaks Over Threshold (POT) and Annual Maximum (AM) approaches. The POT method includes a threshold level (i.e. minimum storm cut distance) that can be used to increase the population size of shorter datasets and/or omit smaller events which may not belong to the same statistical population. The AM method includes the maximum observed data for each year within a time series. For the Christchurch open coast beach profile data set that includes bi-annual survey data, the AM method adopts the larger of the two observed storm cut distances for each year.

A range of candidate distributions were tested with the observed data on the basis that the extreme tail of a distribution often has a rather simple and standardized form, regardless of the shape of the more central parts of the distribution (WAFO, 2012). The distributions tested include Generalized Extreme Value (GEV) distribution, Weibull distribution and Generalised Pareto Distribution (GPD) along with a range of estimation methods to fit to distributions to the observed data. Analysis is undertaken using the methods described in (Shand et al., 2011) using toolboxes provided in WAFO

(2012). A GEV Type 1 (Gumbel) distribution was found to have the best fit to the observed data and has been adopted.

The probability density of an extreme value distribution includes both a location or mean (μ) and shape parameter (σ). The location parameter indicates the position of the distribution mean with the shape parameter determines the tail behaviour of the distribution. The following location and shape parameters have been derived:

- $\mu = -3.13$ m
- $\sigma = 2.39$ m

An example of a PDF (Probability Density Function) of the Gumbel distribution based on the above parameters and histograms showing results using the Monte Carlo technique is shown in Figure 4-5.

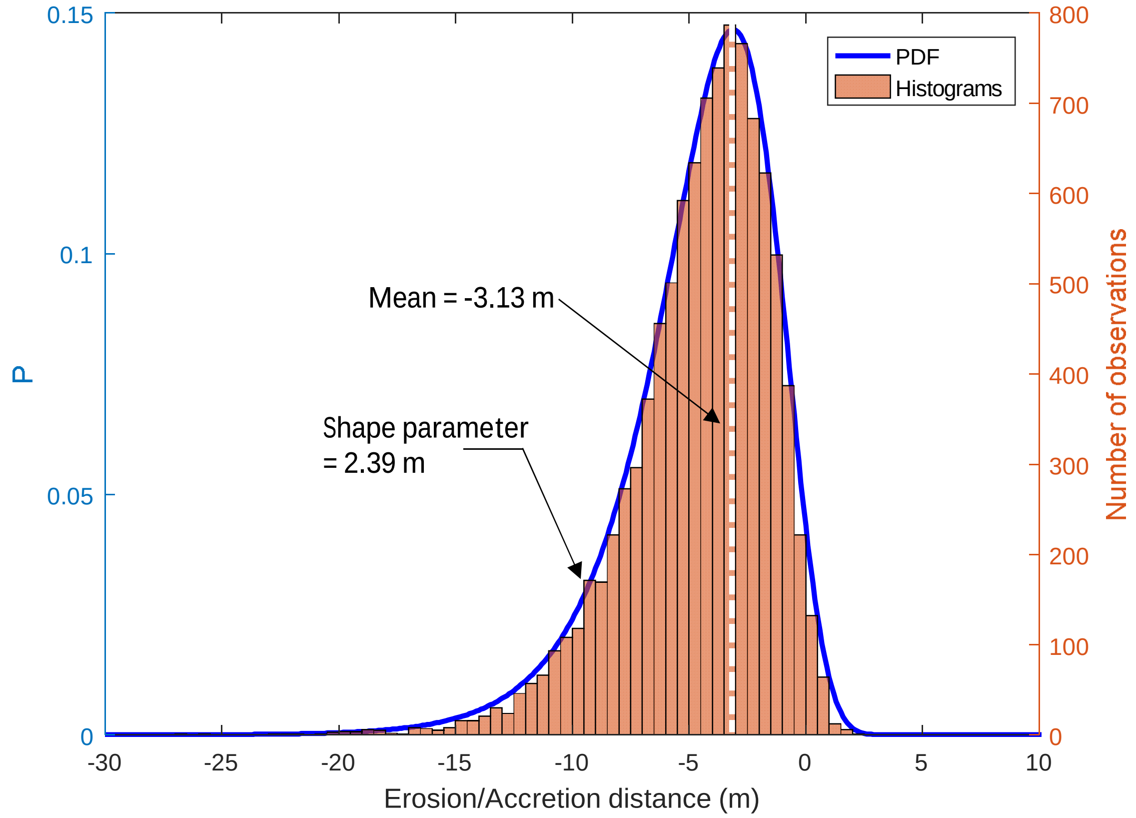


*Figure 4-5: PDF of Gumbel distribution for the storm cut component with a mean (location parameter) of -3.13 m and shape parameter of 2.39 m. The histograms show the number of observations for the erosion/accretion distances resulting from random sampling 10,000 times using the Monte Carlo technique.*

#### 4.2.3.3 Dune stability (DS)

The dune stability factor delineates the area of potential risk landward of the erosion scarp resulting from reduced bearing capacity within an area behind the dune scarp. The parameter assumes that storm erosion results in an over-steepened scarp which must adjust to a stable angle of repose for loose dune sand. The dune stability width is dependent on the height of the existing backshore and the angle of repose for loose dune sand. This has been obtained from an examination of historic reports and a review of the beach profile data. The dune stability factor is outlined below:

$$DS = \frac{H_{dune}}{2(\tan a_{sand})}$$ (Equation 3)

Where $H_{dune}$ is the dune height from the eroded base to the crest and $\alpha_{sand}$ is the stable angle of repose for beach sand. The stable angle ranges from 30 to 34 degrees and these values have been adopted (refer to Table 4-5). Average beach heights and the range in each cell were obtained by analysis of the LiDAR data sets (refer to Table 4-6). Parameter bounds are defined based on the variation in dune height along the coastal behaviour cell and potential range in stable angle of repose.

Table 4-5: Dune stability component values

| Dune stability component value bounds | | | |
|---|---|---|---|
| Cell | Lower (degrees) | Mode (degrees) | Upper (degrees) |
| A-G | 30 | 32 | 34 |

Table 4-6: Dune height component values

| Dune height component value bounds | | | |
|---|---|---|---|
| Cell | Lower (degrees) | Mode (degrees) | Upper (degrees) |
| A | 4 | 5.5 | 7 |
| B | 5 | 5.5 | 6.5 |
| C | 5 | 6 | 7.5 |
| D | 5 | 6 | 7 |
| E | 4 | 5.5 | 7 |
| F | 4 | 5 | 6 |

#### 4.2.3.4 Long-term (LT)

The long-term rate of coastline movement includes both ongoing trends and long-term cyclical fluctuations. These may be due to changes in sea level, fluctuations in coastal sediment supply or associated with long-term climatic cycles such as IPO.

Long-term trends have been evaluated by the analysis of the historic shoreline positions. These have been derived from geo-referenced historic aerial photographs. The historic shoreline data was analysed using the GIS-based Digital Shoreline Analysis System (DSAS) model to evaluate long-term trends. DSAS processes the shoreline data and calculates shoreline change statistics at 10 m intervals along the entire site. Rates of long-term shoreline movement are derived using linear regression analysis. By calculating trends along the entire shoreline, rather than at a low number of discrete points (i.e. beach profile surveys), alongshore variation in long-term trends can be determined more accurately and either be used to inform parameter bounds or to separate the site into coastal behaviour cells.

Maps displaying the DSAS rate of shoreline change output results at 10 m intervals along the shoreline are presented in Appendix E (Figure E1-E6). All areas of Southern Pegasus Bay have experienced net accretion over the last 70 years. Figure 4-6 displays a graph of the DSAS results with the historic shoreline movement rate plotted along the open coast from cell A to F (chainage 0 to 8600 m). The graph plots both the linear regression rate (LRR) and the upper and lower bound 90% confidence intervals (i.e. there is 90% confidence that the LRR values are between these bounds).

The 90% confidence intervals (CI) range from 0.02 m/yr. to 0.68 m/yr., and is on average 0.21 m/yr. The upper and lower bound 90% CIs are shown in Figure 4-6 to illustrate the uncertainty of the alongshore long-term shoreline trend.

Figure 4-6 shows several positive and negative spikes in LRR values. In particular positive spikes are evident around chainage 1000 m and 5000 m and these are likely induced by the surf club access way (anthropogenic) development. The LRR values for these spikes have therefore been excluded from the assessment. The remaining spikes including the negative spike around chainage 5000 are likely caused by local disturbances where the shoreline locally eroded/accreted further compared to the adjacent shoreline. These spikes in LRR values have been retained to account for local disturbances potentially occurring within the respective cells.

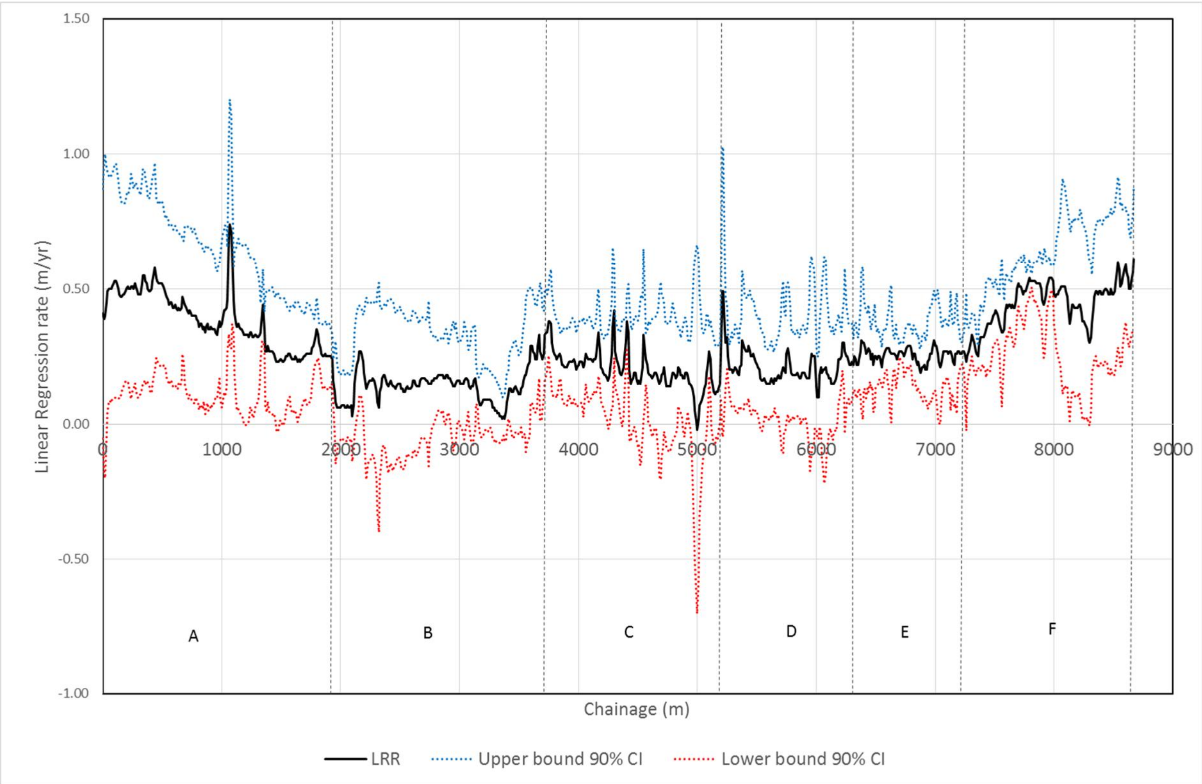


*Figure 4-6: Summary of DSAS results for the open coast including Linear Regression Rate (LRR) including upper bound and lower bound 90% Confidence Intervals (CI)*

The greatest rate of accretion along Southern Pegasus Bay was recorded in the Southshore area (Cell F) followed by Cell A. These results are consistent with the beach profile analysis results at the dune toe. The lowest rate of accretion was recorded at New Brighton (Cell B). This result is expected as the New Brighton area has the greatest amount of anthropogenic changes (i.e. coastal protection structures) within the dunes along the Southern Pegasus Bay shoreline. These coastal protection structures are typically designed to protect landward assets and usually affecting the seaward and adjacent natural coastal zones. The dune area located adjacent to the New Brighton Library and pier fronting Marine Parade has therefore little or no sand-binding dune vegetation. This is mainly due to the area having high public use and structures have been constructed in the active dune area (e.g. New Brighton Library and Marine Parade car park), with associated sand fences installed and beach scraping activities undertaken periodically to limit dune accumulation seaward of this area. Figure 4-7 shows the New Brighton pier area during the July 2001 storms exemplifying that the presence of the seawalls (constructed around 1920) does not allow dune development in this area.

*Figure 4-7: New Brighton pier area during the July 2001 storms (source: Justin Cope, ECan)*

Appendix E (Figure E7-E12) shows the horizontal shoreline position over time with respect to the 2011 shoreline including the five historic shorelines. Three points have been selected within each cell. These figures include linear regression trends between 1941 and 2011, and between 1955 and 2011. The figures show a larger accretion rate at 1941-1955 compared to 1955-2011 for cell A to C, and show a reduction in accretion rate between 1955 and 2011. For cell E to F the accretion rate is approximately constant between 1941 and 2011. Several shoreline position graphs show a relatively steep accretion trend between 1994 and 2011 (e.g. C2, D1-D3) which may be related to dune restoration activities.

##### 4.2.3.4.1 Comparison of dune growth versus beach growth

The long-term shoreline changes in the previous section are based on the horizontal position of the vegetation line/dune toe line. However, long-term changes in horizontal position of the (high tide) beach line may vary. In case the vegetation line did not adequately represent the actual growth in the beach profile, we used the beach profile data set to examine the rate of change of both the dune toe level (taken to be the 2.5 m contour) and the mid-upper/high-tide beach level (taken to be the 1 m contour). It is noted that this analysis is over a shorter time period than the DSAS assessment and therefore unlikely to fully match the longer term assessment. The average rate of change was calculated using linear regression techniques for 13 selected profiles using all the data at each profile from May 1990 to July/August 2016. This resulted in two profiles within each cell and three profiles within cell B. The results of this analysis are set out in Table 4-7. Appendix B includes beach profile plots showing the first and last surveyed profile as well as the calculated minimum, maximum and average beach profile at each location.

These results show regression rates are variable along the coast, with the northern profiles, closest to the Waimakariri River, having significantly less growth, while the southern profiles, more sheltered by Banks Peninsula showing higher rates of regression. Generally the beach accretion rates tend to be lower than the dune accretion rates.

Table 4-7: Assessment of long-term annual change of the dune and beach

| Beach Profile Description | | Regression rate (m/yr.) | |
|---|---|---|---|
| Profile | Name | 2.5 m | 1 m |
| C1130 | Waimairi Beach (Larnach Street) | 0.01 | 0.07 |
| C1065 | North New Brighton (Effingham Street) | 0.22 | 0.15 |
| C1011 | North New Brighton (Bowhill Road) | -0.22 | -0.11 |
| C0952 | New Brighton (Rawhiti Street) | 0.11 | 0.11 |
| C0889 | New Brighton (Hawke Street) | 0.29 | 0.40 |
| C0815 | New Brighton (Rodney Street) | 0.29 | 0.07 |
| C0748 | South New Brighton (Jervois Street) | 0.66 | 0.47 |
| C0703 | South New Brighton (Bridge Street) | 0.58 | 0.62 |
| C0600 | South New Brighton (Jellicoe Street) | 0.80 | 0.88 |
| C0531 | South New Brighton (Halsey Street) | 0.55 | 0.58 |
| C0513 | South New Brighton (Caspian Street) | 0.69 | 0.33 |
| C0431 | Southshore (Penguin Street) | 0.58 | 0.29 |
| C0362 | Southshore (Tern Street) | 1.24 | 1.17 |
| *Average rate (05/1990 to 07/2016)* | | *0.45* | *0.39* |

Because these results are based on a relative short-term time period, the horizontal position of both the dune toe and high tide beach have been assessed for a longer time period at two selected locations. Aerial photographs from 1940-1944 and 2011 have been used for this.

Figure 4-8 shows the 1940-1944 and 2011 historic aerial photographs in the vicinity of C0531 (Cell E) and C1130 (Cell A) including the digitised dune toe and high tide beach lines. It can be seen from this figure that the progradation (seaward movement) of the dune toe is larger than the progradation of the high tide beach between 1940-1944 and 2011 at both locations.

Shoreline trends between 1994 and 2011 for both the dune toe and high tide beach contour are shown in Figure 4-8 (centre panels) including the long-term trend derived from DSAS results. This shows that at profile C1130 the accretion rate of the high tide beach is larger than the accretion rate of the dune toe between 1994 and 2011. This may suggest that the in general larger accretion rate of the dune toe is spatial and temporal variable.

The lower panels in Figure 4-8 show the 1994, 2011 and average beach profiles. It can be seen that the dunes have experienced greater accretion than the beach between 1994 and 2011, in particular for profile C1330 and to a lesser degree at C0531.

Based on the above assessment dune progradation is in general larger than beach progradation with the dune/beach profile potentially oversteepening. Figure 4-9 shows a sketch of this including the (larger) seaward growth of the dunes and the (lesser) growth of the high tide beach.

Cause and effect of a larger dune accretion rate

It is likely that the larger seaward growth of the dunes is a result of dune management. Dune enhancement measures have been applied since the 1870s when Marram grass was introduced with more enhanced measures such as dune reshaping and foredune planting of native sand-binding species applied since the 1990s (pers. Comm. Justin Cope 31/05/2017). Figure 4-10 (left panel) shows a photograph of foredune plants planted at North New Brighton (C1041) from February 1992.

While a larger progradation of the dunes may potentially lead to an oversteepened beach/dune profile, it may also increase the vulnerability of the dunes to storm-induced erosion. Because beach progradation is less than dune progradation, the beach profile is relatively flat compared to when progradation rates of beach and dune are equal. Therefore the dune is more exposed to wave action during storm events which could lead to a larger storm cut of the dunes. Figure 4-10 shows photographs of the dunes at North New Brighton (C1041) from February 1992 (left panel) and August 1992 (right panel), showing the pre- and post-storm dunes. An erosion scarp of several metres (vertically) can be seen.

C0531 (Cell E)

C1130 (Cell A)

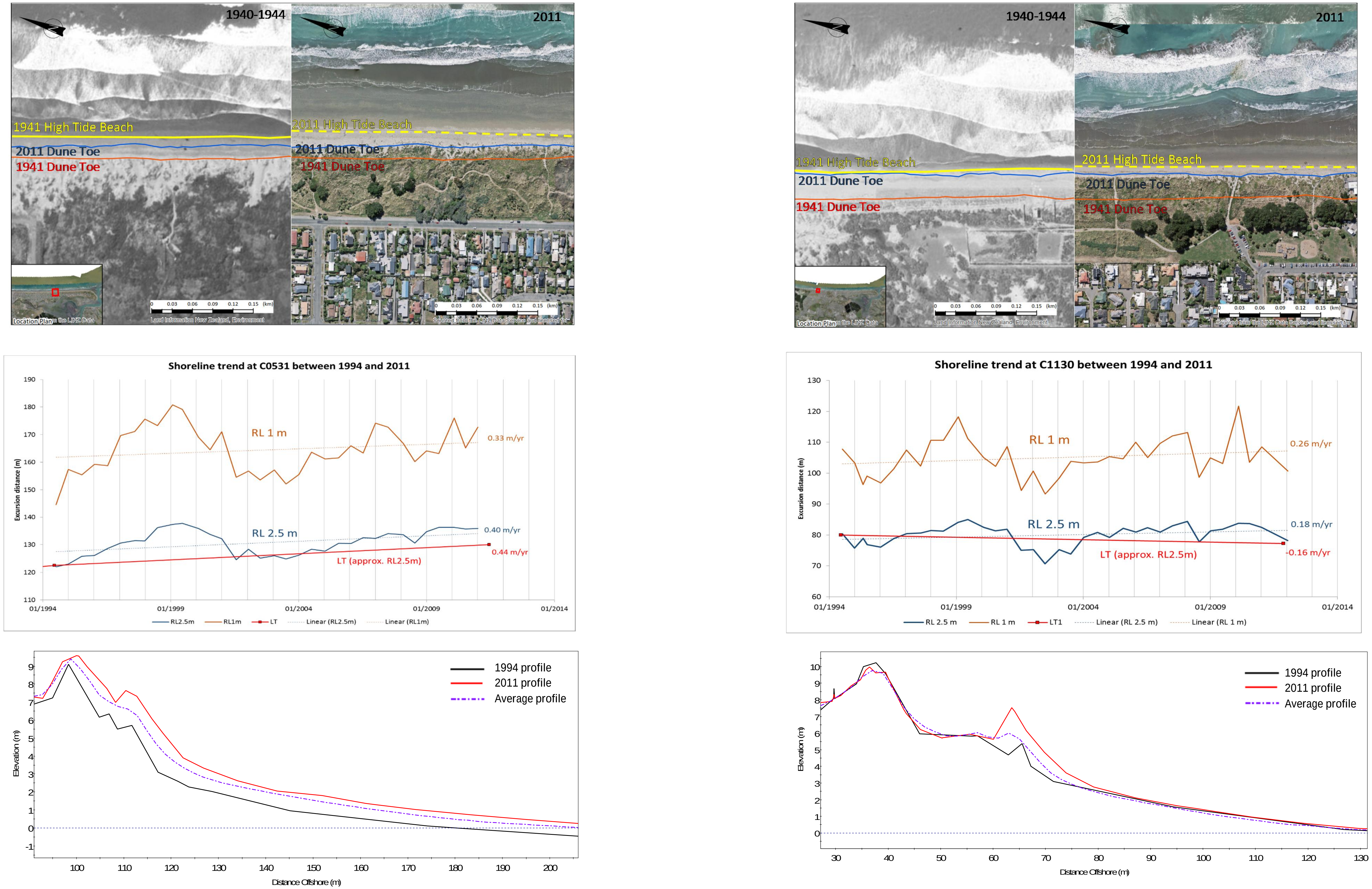


*Figure 4-8: Top panels: Dune toe versus high tide beach changes between 1940 and 2011 shown on historic aerial photographs (source: Canterburymaps.govt.nz). Centre panels: Shoreline trends between 1994 and 2011 for RL1m contour (taken to be high tide beach) and RL2.5m (taken to be dune toe) compared with the long-term trend from DSAS analysis. Lower panels: Beach profile surveys from 1994, 2011 and the average profile between 1994 and 2011. Left panels show figures for C0531 (Cell E) and right panels show figures for C1130 (Cell A).*

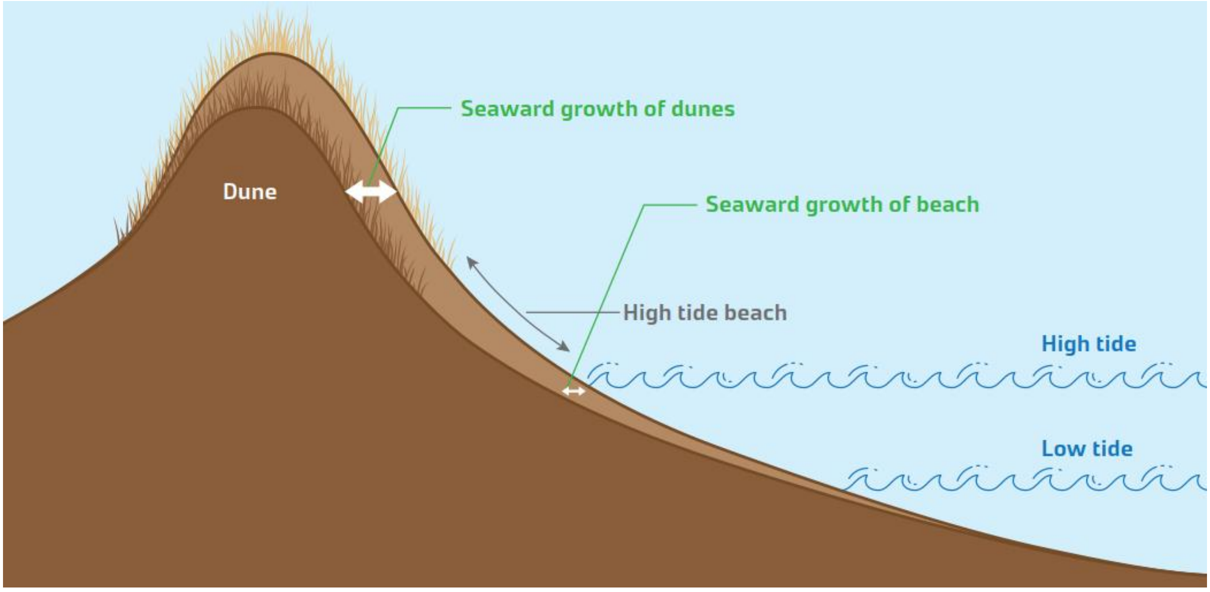


*Figure 4-9: Sketch showing how dune growth (accretion of dune) as a result of improved vegetation management may be greater than accretion of the beach face. This may potentially mask beach retreat.*

*Figure 4-10: Photograph taken in vicinity of C1041 (North New Brighton) in February 1992 showing the dunes including dune management measures (left), and photograph taken in vicinity of C1041 in August 1992 showing the post-storm dunes (right). Source: Justin Cope (ECan).*

#### 4.2.3.4.2 Effect of sediment supply on long-term shoreline changes

This section sets out an evaluation of the estimated sediment supply from the Waimakariri River (refer to Section 2.3) with the implications of this supply on shoreline change.

Based on a sediment budget compartment length of 16 km (the length of the coastline from the Waimakariri River outlet to the Avon-Heathcote Estuary inlet), an active beach profile height (measured from the dune crest to a depth offshore where wave induced sediment transport is negligible, of 20 m as assumed by Hicks (1988)) and with no losses either offshore or to the estuary, the range of possible shoreline accretion is 0.56 to 2 m/yr. This was calculated by dividing the supply of sediment (180,000 $m^3$/yr. or 635,000 $m^3$/yr.) by the length and depth (16 km/20 m) of the beach to give the rate of horizontal change. This estimated accretion rate of 0.56 m/yr. of the entire profile height of 20 m is sensitive to the input assumptions of suspended sediment load volume, the fine sand fraction percentage and the sediment budget compartment dimensions.

The DSAS analysis of shoreline change measured by an examination of the dune vegetation line over a period of 70 years (1941 to 2011) resulted in an average change of 0.278 m/yr. in the southern (8.6

km) portion of cell. This rate of observed change is significantly lower than the estimated rates of 0.56 to 2.0 m/yr. This could mean that less sediment is supplied to this area than the sediment supply calculations suggest, or that the supply is not evenly distributed over the profile (i.e. growth of the profile does not occur uniformly) or that there are additional losses out of the system or some combination thereof.

Based on the comparison of regression rates for the dune toe and high tide beach it seems that some of the rates of change from the DSAS may be more linked to dune management improvements rather than ongoing sediment supply. The average trend of accretion from the beach profile analysis is 0.45 m/yr. at the dune toe and 0.39 m/yr. on the beach face (refer to Table 4-7). Both these values are also lower than the derived rate of 0.56 m/yr. based on the lower bound supply rate of 180,000 $m^3$/yr. from the river (Hicks, 1998), suggesting that even this lower bound rate could be optimistic (i.e. too large).

Based on the observed long-term dune toe accretion rate of 0.278 m/yr. the total additional sand supplied to this area is around 48,000 $m^3$/yr. (0.278m/yr. x 8,600 m x 20 m) with the shorter term beach profile data at the 2.5 m beach contour suggesting some 77,000 $m^3$/yr. Using these same rates for the entire coastal cell results in a supply of around 90,000 to 144,000 $m^3$/yr. between the Waimakariri River and the estuary inlet as more justifiable values than the 180,000 $m^3$/yr. lower value derived by Hicks (1998). For the purposes of this study we have based the existing situation on the DSAS long-term trend data (i.e. 48,000 $m^3$/yr. supplied to the 8.6 km beach system).

We note that additional research on the supply of sediment to the Christchurch's beach systems from the Waimakariri and other rivers, including other sources such as the continental shelf would be useful. Also the potential deposition patterns on the nearshore beach system would be useful. This is useful to better understand the effect of the river and to estimate likely changes in potential deposition patters with future changes in river flow levels and Pegasus Bay wave energies.

The 2010–2011 Canterbury earthquakes caused minor subsidence along the northern New Brighton shoreline and minor uplift along the southern shoreline in the order of 0.1 to 0.2 m (Beavan et al., 2012). This adjustment may modify littoral transport processes. However, ECan have not noted any indication of a response in the beach profile record to date.

#### 4.2.3.4.3 Potential large scale changes to sediment supply from Waimakariri River

Sediment supply from the Waimakariri River could be affected by tectonic events, such as large scale earthquakes within the catchment or climate change effects. Based on the observable changes to river systems from the Kaikōura Earthquake Sequence, significant sediment loads can be deposited into the river and valley systems. Based on observation of gravel movement on other rivers, the change to processes can then take decades to centuries to work through, with larger volumes of sediment supplied to the upper catchment no always reaching the coastal environment, to the earthquake related changes to the hydraulic grade of the river systems.

Previous research indicates precipitation over the Canterbury Plains is affected by large atmospheric circulation patterns, such as ENSO and IPO, which are expected to respond to projected climate change resulting in a change in the frequency, magnitude and seasonal distribution of rainfall (Ummenhofer and England, 2007; Ummenhofer et al., 2009). NIWA (MfE, 2016) also forecasts that the Canterbury Plains will likely experience less rainfall over the short to medium term.

The Canterbury Plains represent a dryland and drought prone environment under current conditions and the plains agriculture is already heavily reliant on irrigation water, with the region's catchments having been under extreme water use pressures for over a decade (e.g. OECD, 2007; Glubb et al., 2012).

Further to the demand for water from the Waimakariri River and the groundwater resource, there are currently significant pressures on the Waimakariri River sediment resources, with implications for the coastal sediment budget. Gravel and its associated sand 'bycatch' is being extracted from the lower reaches of the river, near the State Highway 1 bridge, in an area flanked by stopbanks to protect surrounding settlements from the hazard of Waimakariri River flooding. This sediment extraction pressure is likely to increase with future demand from the construction industry as well as with the desire to maintain the river channel depth and thus the design standard of the stopbanks (the flood carrying capacity of the Waimakariri River).

Given the range of future anthropogenic water resource management options under NIWA's regional climate change predictions (refer to MfE, 2016), and alongside sediment resource and flood hazard considerations, significant future alterations to the Waimakariri River sediment supply to Pegasus Bay are possible.

There is no local research on the effect of climate change on the Waimakariri River sediment supply and both local international research suggests that this is a complex process, with many feedback loops. A comprehensive study on the impact of climate change on the River Rhine (Asselman, et al., 2000) identified that most of the sediment transport in that river occurred during mean flow conditions, rather than in peak (flood) flows. While the Rhine has a larger catchment and length, both the Rhine and Waimakariri Rivers are snow-fed transitioning from an Alpine environment to a flood plain and have similar mean flood flows (2,000 -2,500 $m^3/s$ for Waimakariri River and 2,300 $m^3/s$ for the Rhine River). The study identified that climate change tends to result in small increases to the mean annual flow (MAF), but also results in both the high and low flow conditions. However, most of the sediment transport occurred around the MAF and therefore a reduction in MAF could result in a reduction of sediment transport. The rise in sea level could also increase the migration of the salt wedge in the lower reaches of the river and the flocculation effect of the saline water could also affect the sedimentation processes in the lower reaches of the river reducing sediment discharge to the coast. Asselman et al. (2000) evaluated that sediment supply to the lower reaches of the Rhine could reduce by between 5 to 40% as a result of climate change and land use scenarios.

It is likely that all the physical effects identified in the Rhine study as a result of climate change could occur at the Waimakariri River. However, it is not possible to accurately assess the climate change and anthropogenic effects on sediment supply to the coast. For the purposes of understanding the potential effect of climate change on sediment supply we have assumed a 10% reduction in sediment supply at 2065 and a 30% reduction in sediment supply at 2120. No positive contribution was included as it was considered that the potential effect of higher rainfalls increasing sediment supply to the coast would not occur immediately as discussed in Section 2.3. Therefore the potential of increased sediment supply, from a major tectonic event is not able to be accurately predicted within the time period under consideration.

The 10% and 30% reductions were applied to the 48,000 $m^3/yr$. volume calculated to be the present accumulation volume and a net rate of change in m/yr. was calculated using the same method as used to evaluate average sediment supply rates. The net effect will be a reduction in the observed long-term rate of accretion and a variation in the long-term rate of change of -0.027 m/yr. at 2065 and -0.083 m/yr. at 2120. Taking this rate of change into account changes the long-term rates of change at each cell as shown in
Table 4-8.

Table 4-8: Range of long-term trends taking into account climate change effects

| Cell | Annual rate (m/yr.) | | |
|---|---|---|---|
| | Present day | 2065 | 2120 |
| A | 0.38 | 0.35 | 0.30 |
| B | 0.14 | 0.11 | 0.06 |
| C | 0.20 | 0.17 | 0.12 |
| D | 0.21 | 0.18 | 0.13 |
| E | 0.26 | 0.23 | 0.18 |
| F | 0.44 | 0.41 | 0.36 |

#### 4.2.3.5 Adopted values

A normal distribution has been adopted to use for the long-term component as recommended by Kenderdine et al. (2016). Input parameters for a normal distribution are a mean and SD, and these have been derived based on the LRR values within each cell for this study.

Two scenarios of long-term trends have been assessed for this study:

1 Average sediment budget scenario
2 Reduced sediment budget scenario (i.e. taking into account climate change effects).

For scenario 1 (average sediment budget) the present day rates have been adopted and assumed to continue to 2120 with a constant sediment supply from the Waimakariri River. The adopted values for scenario 1 are shown in Table 4-9. The SD values have been derived based on the 90% confidence intervals of beach profile residuals. This was found to be the most reliable method (refer to Appendix F).

For scenario 2 (reduced sediment budget) the mean long-term rates as shown in Table 4-8 have been adopted in combination with the SD for each cell as shown in Table 4-9.

Table 4-9: Long-term component values for 'Average sediment budget' scenario

| Cell | Mean (m) | SD (m) based on 90% CI of residuals |
|---|---|---|
| A | +0.38 | 0.10 |
| B | +0.14 | 0.10 |
| C | +0.20 | 0.07 |
| D | +021 | 0.07 |
| E | +0.26 | 0.06 |
| F | +0.44 | 0.06 |

#### 4.2.3.6 Effects of sea level rise (SL)

Adopted sea level values

We have applied sea level rise values for four IPCC scenarios over a 100 year time frame (i.e. 2120) as required by the NZCPS (2010).

An average historic rate of sea level rise of 2.0 mm/yr. has been deducted from the projected sea level rise values for use in this assessment on the basis that the existing long-term trends and processes already incorporate the response to the historic situation. Because the RCP projections are based on a mean sea level between 1986 and 2005, a further deduction has been applied to discount the sea level rise that has occurred between 1995 (average of 1986-2005) and 2011. The base year for the projections to 2120 is 2011. Table 4-10 presents the projected sea level rise values and adjusted sea level rise values that are used in this coastal hazard assessment.

Table 4-10: Adopted sea level rise values

| Time frame | RCP2.6 M (m) | RCP4.5 M (m) | RCP8.5 M (m) | RCP8.5+ (m) |
|---|---|---|---|---|
| Projected 2065 | 0.30 | 0.33 | 0.41 | 0.55 |
| Adjusted 2065 | 0.14 | 0.18 | 0.24 | 0.37 |
| Projected 2120 | 0.55 | 0.67 | 1.06 | 1.36 |
| Adjusted 2120 | 0.29 | 0.41 | 0.80 | 1.08 |
| Note: the adjusted values include a discount of 2.0 mm/yr based on average historical trends, and discount of sea level rise occurred between 1995 (average of 1986-2005) and 2011. | | | | |

### Beach response

Geometric response models propose that as sea level is raised, the equilibrium profile is moved upward and landward conserving mass and original shape (refer Figure 4-11). The most well-known of these geometric response models is that of Bruun (1962, 1988) which proposes that with increased sea level, material is eroded from the upper beach and deposited offshore to a maximum depth, termed closure depth. The increase in sea bed level is equivalent to the rise in sea level and results in landward recession of the shoreline. The model can be defined by the following equation:

$$SL = \frac{L_*}{B + h_*} S \qquad \text{(Equation 4)}$$

Where SL is the landward retreat, $h_*$ defines the maximum depth of sediment exchange taken as the closure depth, $L_*$ is the horizontal distance from the shoreline to the offshore position of $h_*$, B is the height of the berm/dune crest within the eroded backshore and S is the sea level rise.

The EnviroLink best practice guidelines for defining coastal hazard zones in New Zealand states the Bruun Rule is applicable to open coast sandy beaches (Ramsey et al., 2012). The Bruun Rule has also been tested in the Environment Court and was accepted as a suitable approach to predict the beach response to sea level rise for the purposes of coastal hazard planning (Skinner v Tauranga District Council, A 163/02).

The Bruun Rule is considered to provide an acceptable “order of magnitude” estimate of shoreline retreat distance due to a rise in sea level (Ramsey et al., 2012). However, it is governed by simple, two-dimensional conservation of mass principles and is limited in its application in the following aspects:

- The rule assumes that there is an offshore limit of sediment exchange or a ‘closure depth’ beyond which the seabed does not raise with sea level
- The rule assumes no offshore or onshore losses or gains
- The rule assumes an equilibrium beach profile where the beach may fluctuate under seasonal and storm influences but returns to a statistically average profile (i.e. the profile is not undergoing long-term steepening or flattening)
- The rule does not accommodate variations in sediment properties across the profile or profile control by hard structures such as substrate geology or adjacent headlands.

While some have questioned the actual existence of a closure depth (Cooper and Pilkey, 2004), the Bruun Rule is not necessarily reliant on its physical existence. While long-term sediment exchange may occur to very deep water depths (i.e. the ‘pinch-out’ point), this “ultimate limit” profile adjustment extent is only valid if either the profile response is instantaneous or if sea level changes and then stabilises with the profile ‘catching up’. As sea level rise is expected to be ongoing and a lag in profile response is apparent, the outer limit of profile adjustment is likely to be left behind. The closure depth can therefore be more realistically defined as the point at which the profile adjustment can keep up with sea level change and becomes a calibration parameter in lieu of an adequate depth dependent lag parameter.

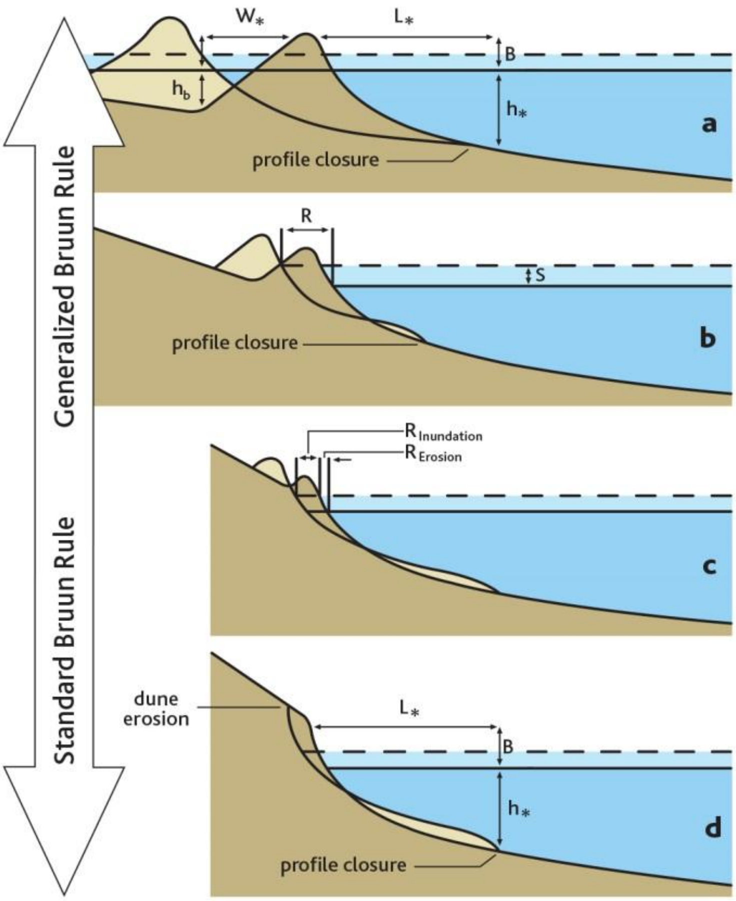


*Figure 4-11: Schematic diagrams of the Bruun model modes of shoreline response (after Cowell and Kench, 2001)*

To define the sea level rise retreat (SL) component distributions, the Bruun rule estimates using the outer Hallermeier closure depth definition (di) have been adopted as upper bound values, estimates using the inner Hallermeier closure definition (dl) provides the modal (most likely) values and results using the beach face slope (Komar, 1999) provide the lower (almost certain) bounds. The beach face is defined by average mean low water spring position and average beach crest height. The Hallermeier closure definitions are defined as follows (Nicholls et al., 1998):

$$d_l = 2.28H_{s,t} - 68.5(H_{s,t}^2 / gT_s^2) \approx 2 \times H_{s,t}$$ (Equation 5)

$$d_i = 1.5 \times d_l$$ (Equation 6)

Where dl is the closure depth below mean low water spring, $H_{s,t}$ is non-breaking significant wave height exceeded for 12 hours in a defined time period, nominally 1 year, and $T_s$ is the associated period.

For this study the deep water (non-breaking) wave climate parameters of $H_s$ and $T_s$ were based on the ECan wave buoy data recorded over a 14 year period between 1999 and 2013. The wave buoy is located in deep water east of Banks Peninsula. The resulting $H_s$ and $T_s$ parameters are 4.2 m and 10.8 s respectively. Based on these wave climate parameters the inner closure depth is calculated as 8.5 m below mean low water spring using the Hallermeier method defined in Equation 5 (refer section 4.2.3.6, equivalent to 9.5 m below mean sea level). The outer closure depth is calculated as 13 m (equivalent to 14 m below mean sea level. The average dune crest is approximately 8.5 m above mean sea level. This results in a total active profile height of between 18 m to 23 m (8.5 m dune height and 9.5 m to 14 m closure depth).

### 4.2.4 Combination of parameters

For each coastal cell, the relevant component bounds influencing the CEHZ have been defined according to the methods described as above and is summarised in Table 4-11. Table 4-12 shows the input parameters for each CEHZ within each cell.

Table 4-11: Theoretical erosion hazard parameter bounds

| Parameter | Distribution | Input values |
|---|---|---|
| Storm cut, SC (m) | Extreme Value | Mean & shape parameter |
| Dune stability, DS (m) | Triangular | Lower bound: $H_{max}$ & $\alpha_{min}$<br>Mode: $H_{mean}$ & $\alpha_{mean}$<br>Upper bound: $H_{min}$ & $\alpha_{max}$ |
| Long-term, LT (m/yr.) | Normal | sea level rise scenarios:<br>Average sediment budget scenario: Mean & SD<br>Reduced sediment budget scenario: Mean & SD |
| Sea level rise, SLR (m) | N/A | sea level rise scenarios:<br>RCP2.6 M<br>RCP4.5 M<br>RCP8.5 M<br>RCP8.5+ |
| Closure slope | Triangular | Lower bound: Slope from dune crest to outer Hallermeier closure depth<br>Mode: Slope from dune crest to inner Hallermeier closure depth<br>Upper bound: Slope across active beach face to typical swash excursion |

Table 4-12: Input parameters for each CEHZ component within each cell

| Site | | Christchurch open coast | | | | | |
|---|---|---|---|---|---|---|---|
| Cell | | A | B | C | D | E | F |
| Chainage, m (from N/W) | | 0-1950 | 1950-3700 | 3700-5200 | 5200-6300 | 6300-7250 | 7250-8650 |
| Morphology | | Dune | Dune | Dune | Dune | Dune | Dune |
| Storm cut (m) | Mean | -3.13 | -3.13 | -3.13 | -3.13 | -3.13 | -3.13 |
| | Shape parameter | 2.39 | 2.39 | 2.39 | 2.39 | 2.39 | 2.39 |
| Dune elevation (m RL) | Min | 4 | 4.5 | 5 | 4.5 | 4 | 3.5 |
| | Mode | 5.5 | 5.5 | 6 | 6 | 5.5 | 5 |
| | Max | 7 | 6.5 | 7.5 | 7 | 7 | 6 |
| Stable angle (degree) | Min | 30 | 30 | 30 | 30 | 30 | 30 |
| | Mode | 32 | 32 | 32 | 32 | 32 | 32 |
| | Max | 34 | 34 | 34 | 34 | 34 | 34 |
| Long-term (m) -ve erosion +ve accretion | Mean[1] (present) | 0.38 | 0.14 | 0.2 | 0.21 | 0.26 | 0.44 |
| | Mean[1] (2065) | 0.38 | 0.14 | 0.2 | 0.21 | 0.26 | 0.44 |
| | Mean[1] (2120) | 0.38 | 0.14 | 0.2 | 0.21 | 0.26 | 0.44 |
| | Mean[2] (present) | 0.38 | 0.14 | 0.2 | 0.21 | 0.26 | 0.44 |
| | Mean[2] (2065) | 0.35 | 0.11 | 0.17 | 0.18 | 0.23 | 0.41 |
| | Mean[2] (2120) | 0.3 | 0.06 | 0.12 | 0.13 | 0.18 | 0.36 |
| | SD | 0.1 | 0.1 | 0.07 | 0.07 | 0.06 | 0.06 |
| Closure slope | Min | 0.006 | 0.006 | 0.006 | 0.005 | 0.005 | 0.005 |
| | Mode | 0.018 | 0.016 | 0.014 | 0.013 | 0.012 | 0.012 |
| | Max | 0.035 | 0.026 | 0.038 | 0.041 | 0.027 | 0.029 |
| SLR 2065 (m) | RCP2.6 M | 0.14 | | | | | |
| | RCP4.5 M | 0.18 | | | | | |
| | RCP8.5 M | 0.24 | | | | | |
| | RCP8.5+ | 0.37 | | | | | |
| SLR 2120 (m) | RCP2.6 M | 0.29 | | | | | |
| | RCP4.5 M | 0.41 | | | | | |
| | RCP8.5 M | 0.8 | | | | | |
| | RCP8.5 83+ | 1.08 | | | | | |

[1]Average Sediment Budget scenario
[2]Reduced Sediment Budget scenario

Probability distributions constructed for each parameter are randomly sampled and the extracted values used to define a potential CEHZ distance. This process is repeated 10,000 times using a Monte Carlo technique and the probability distribution of the resultant CEHZ width is forecast. Figure 4-12 presents an example of the results for each CEHZ component and the resultant CEHZ distance for Cell A at 2120 for both the average sediment budget scenario and reduced sediment budget scenario including the four sea level rise scenarios. The example shows both the histogram and the cumulative distribution frequency graphs.

The example results show that for the average sediment budget and RCP8.5+ scenario the possible CEHZ range from 32 to -148 m, with a P50% (50% probability of exceedance) value of -25 m (Figure 4-12; left panel). This result can be interpreted as a 50 % chance of coastal erosion exceeding 25 m by 2120. The P5% is -77 m, which is substantially below the maximum extent of -148 m. Furthermore, these results show the difference in erosion values for the four sea level rise scenarios. The possible CEHZ values for the RCP2.6 M scenario ranges from 57 to -32 m compared to 32 to -148 m for the RCP8.5+ scenario. This shows that for the lower RCP scenario that the long-term accretion rate is dominant (i.e. larger) over the sea level rise component, which nullifies the sea level rise component. The sea level rise component becomes more dominant for the higher RCP scenarios.

For the reduced sediment budget and RCP8.5+ scenario (Figure 4-12, right panel) the possible CEHZ ranges from 22 to -165m with a P50% of -34 m. The slightly larger erosion distances compared to the average sediment budget scenario are caused by the reduced long-term accretion rates and show that these may potentially become significant in the future.

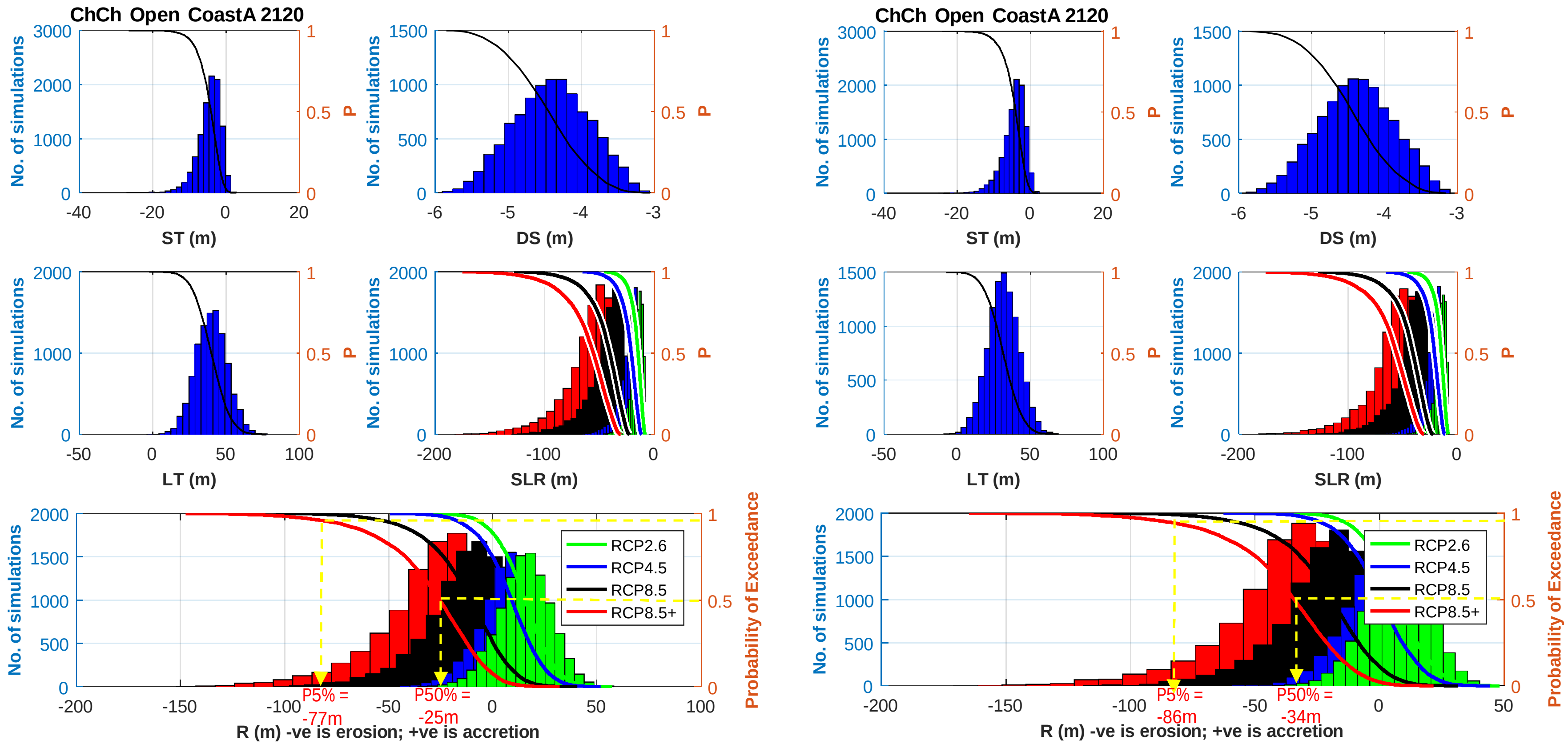


*Figure 4-12: Example histograms and cumulative distribution functions of parameter samples and the resultant CEHZ distances for cell A to 2120 for the average sediment budget scenario (left) and reduced sediment budget scenario (right).*

### 4.2.5 Risk-based approach

A risk-based approach to managing coastal hazard is advocated by both the NZCPS (2010) and the CRPS (2013) with both the likelihood and consequence of hazard occurrence requiring consideration. For example, the NZCPS (2010) suggests consideration of areas both 'likely' to be affected by hazard and areas 'potentially' affected by hazard (refer to Section 2.1.1). While the term 'likely' may be related to a likelihood over a defined time frame based on guidance provided by MfE (2008), i.e. a probability greater than 66% as shown in Table 4-13, the term 'potential' is less well defined. This assessment therefore aims to derive a range of hazard zones corresponding to differing likelihoods which may be applied to risk assessment.

Table 4-13: Likelihood of scenario occurring within the selected planning horizon

| Designation | Frequency | Description | IPCC definition |
|---|---|---|---|
| | | | Virtually certain (> 99% chance that a result is true) |
| A | Almost certain | Is expected to happen, perhaps more than once | Very likely (90–99%) |
| B | Likely | Will probably happen | Likely (66–90%) |
| C | Possible | Might occur; 50/50 chance | Medium (33–66%) |
| D | Unlikely | Unlikely to occur, but possible | Unlikely (10–33%) |
| E | Rare | Highly unlikely, but conceivable | Very unlikely (1–10%) |
| | | | Exceptionally unlikely (< 1%) |

### 4.2.6 Mapping the CEHZ

Coastal erosion hazard zone distances are mapped as offsets to the existing baseline of the 2011 dune toe. The CEHZ2065 and CEHZ2120 for the South Brighton Spit zone (Cell G) is offset from the Inlet Migration Curve (IMC) baseline, due to the shoreline fluctuation in this area. The IMC is defined as the most inland shoreline position over the fluctuating spit area (Shand, 2012). The assessment includes the changes that have occurred since the Canterbury earthquakes due to changes in land level and assess potential effect of future sea level rise. Refer to Figure 4-13 for an illustration of the IMC delineation for cell G. Figure 4-13 shows the historic shorelines fluctuate within cell G and the IMC represents the landward edge of the shoreline fluctuation and is used as the baseline for offsetting the CEHZ distance.

Where the erosion hazard distances differ between adjacent coastal cells, the mapped CEHZ is merged over a distance of at least ten times the difference between erosion hazard distances. This provides a smooth transition between adjacent cells. Where appropriate transitions are mapped along contours or material discontinuities.


Notes: Aerial photograph taken in 2011

A3 SCALE 1:4,000

0 50 100 150 200 Meters

LEGEND

Inlet Migration Curve

Shoreline (dune toe position)

1941

1994

1955

2011


Tonkin+Taylor

105 Carlton Gore Road, Auckland

www.tonkintaylor.co.nz

| | | |
|---|---|---|
| DRAWN | PPK | Sep.17 |
| CHECKED | | |
| APPROVED | | |
| ARCFILE | 851857.004_F4-13.mxd | |
| SCALE (AT A3 SIZE) | 1:4,000 | |
| PROJECT No. | 851857.004 | |

# CHRISTCHURCH CITY COUNCIL

Coastal Hazard Assessment

Inlet Migration Curve

FIGURE No. Figure 4-13

Rev. 0

Path: P:\851857\851857.0040\WorkingMaterial\GIS\Map Documents\851857.004_F4-13.mxd Date: 7/09/2017 Time: 11:07:40 AM

### 4.2.7 Uncertainties and limitations

Uncertainty may be introduced to the assessment by:

1. an incomplete understanding of the components influencing the coastal erosion hazard zone
2. an imprecise description of the natural processes affecting, and the subsequent quantification of each individual parameter
3. errors introduced in the collection and processing of data
4. variance in the processes occurring within individual coastal cells.

Of these uncertainties, the alongshore variance (item 4) of individual coastal cells may be reduced by splitting the coast into continually smaller cells. However, data such as beach profiles are often available only at discrete intervals, meaning increasing cell resolution may not necessarily increase data resolution and subsequent accuracy. We believe we have refined the cells as far as practical based on factors which could significantly affect results. Residual uncertainty may be allowed for by selecting a lower probability CEHZ value.

The first two uncertainty items listed above (item 1-2) are being continually developed within coastal research fields. However, there is generally a lag time between scientific developments, and their use in practical assessment as they are refined, tested and made generically applicable. This assessment has used relatively new techniques by incorporating probabilistic assessment of components.

Similarly, numerical models are beginning to better resolve the physical processes responsible for coastal erosion although as noted above the inability to consider infra-gravity waves does affect SBEACH's ability to represent erosion for flat dissipative beaches. However, complex coupled models are computationally expensive and heavily reliant on quality, long-term data. Without such data, complex model results are largely meaningless. We have attempted to balance the use of numerical modelling where useful (wave and beach response) with analytical and empirical assessment to ensure results are robust and sensible.

The re-assessment methodology developed by T+T incorporates the uncertainty in the individual components within the individual parameter bounds. Greater uncertainty utilises wider parameter bounds while less uncertainty utilises narrower bounds. This allows independent uncertainty terms to be combined within the probabilistic framework rather than utilising a single factor or adding uncertainty to each term as has been done previously.

Uncertainties in individual components will reduce as better and longer local data is acquired (item 3), particularly around rates of short- and long-term shoreline movement and shoreline response to sea level rise. Data collection programmes such as beach profiling are essential to reducing this uncertainty and should be continued. Our approach can also allow for uncertainties and data limitations by the user defined selection of the P value output.

### 4.2.8 Anthropogenic effects

Human influences can affect the coastal erosion hazard. Erosion protection works have been installed along portions of the New Brighton shoreline to protect public assets (e.g. car parks and the New Brighton Library). The dune height along these types of areas are also reduced which can increase the inundation hazard.

While properly designed coastal protection works along beach can reduce erosion rates while in place, the shoreline position is generally returned to its long-term equilibrium position rapidly once the structure fails or is removed. We have therefore evaluated the hazard extent excluding the effects of any structures. This identifies the potential land area that could be affected, or the area that is benefitting with the structure. The CEHZ lines along shoreline protected by coastal structures are situated roughly along the landward side of the structures. Appendix H shows the CEHZ lines along the erosion protection works and shows the area that could be affected by erosion if the protection works fail. Informed decision around the future maintenance or re-consenting of structures can then be made.

Dune planting and fencing has been undertaken along sections of New Brighton. If this strategy is not maintained over the time frame of the CEHZ period (50 or 100 years), then we could expect a greater area of land to be susceptible to coastal erosion hazard in this area.

## 4.3 Harbour coast coastal inundation hazard zone (CIHZ)

The coastal inundation level was mapped by combining the following components (refer to Section 2.2 for more background information):

- Storm tide
- Wave set-up
- Wind set-up
- Sea level rise.

### 4.3.1 Storm tide

The combined elevation of the predicted tide, storm surge and medium-term fluctuations is known as the storm tide (refer to Section 2.2.4). Goring (2009) has calculated the 1% AEP storm tide level for Port Lyttelton.

The Lyttelton Port of Christchurch storm tide results have been adopted for sites within Lyttelton Harbour. No long-term tide gauge data exists for Akaroa and this study assumes the storm tide levels presented in Table 4-14 for Lyttelton can be applied to the Akaroa Harbour sites. Additional wind set-up values have been calculated for the Akaroa Harbour, which increases the storm tide level for some sites at the head of the harbour. The 1% AEP was adopted for both the 2065 and 2120 planning time frames.

Table 4-14: Extreme storm tide

| Site | Storm tide level (RL m) |
|---|---|
| | 1% AEP |
| Port Lyttelton | 1.92 |

### 4.3.2 Wave set-up

Waves can super-elevate the mean water level during the breaking process (termed wave set-up). Wave set-up represents a constant flow of water over a coastal barrier (e.g. dune system including backshore, beach and shoreface) above the storm tide level and is generally included in static flood assessments for the purposes of hazard mapping. The additional wave run-up is not considered in the inundation calculation because it attenuates inland and is unlikely to cause widespread inundation over areas several tens of metres from the coast (Ramsey et al., 2012). However, wave run-up may be an important consideration for assets located close to the existing shoreline (e.g. port and road infrastructure).

#### 4.3.2.1 Avon-Heathcote Estuary entrance

Wave set-up is a process that can elevate water levels at entrances to inlets. However, it is noted that the research on wave set up at estuary and river entrances is still developing and there are still knowledge gaps on all the relevant physical processes and interactions (Dunn, 2001). However, research by Irish and Canizares, (2009) indicate that wave induced gradient flow contributes an additional 15 to 35% to the total storm tide level.

The wave-breaking process is capable of raising the mean sea level and this includes wave breaking and set-up on the ebb-tide delta as well as a contribution to water levels from the gradient between the wave set-up on the adjacent coast and at the inlet (see Figure 4-14). This process is also likely at the entrance to the Avon-Heathcote Estuary.

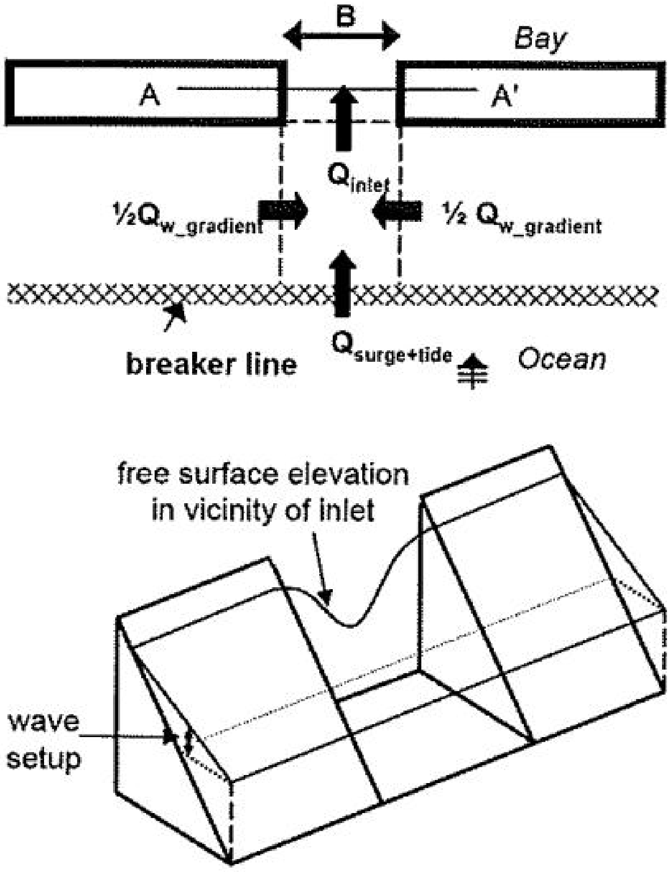


*Figure 4-14: Schematic view of wave induced flow through tidal inlets during storms (Source: Irish and Canizares, 2009)*

Stephens et al. (2015) recently completed coastal calculator identifies an offshore extreme wave height off Banks Peninsula of around 8.5 m and an associated 1% AEP nearshore wave height of 3.34 m off New Brighton Beach at around the 5 m depth contour. This resulted in a wave set-up of 0.69 m using the upper beach slope of 1(V):12(H) based on beach profile data.

The wave transformation model Unibest-LT (WL|Delft Hydraulics, 1994) was used to simulate the wave-breaking process from deep water to onshore with the bathymetric information available from the hydrographic chart NZ6321 that includes depth contours and points at various resolution. A deep water boundary condition of 5 m and a mean wave period of 9 seconds was used and the model derived a wave height at the 5 m contour of 3.39 m, similar to the nearshore wave height derived by NIWA.

The amount of wave set-up on the ebb tide delta was then assessed using the Unibest-LT model. A maximum set-up solely from the wave-breaking process along the transect was calculated to be 0.3 m. As there is a 0.39 m difference in elevation between the adjacent beach set-up of 0.69 m and the set-up on the ebb tide delta of 0.3 m there will be a contribution resulting from alongshore gradient set-up. Assuming between 15% and 35% (based on Irish and Canizares, 2009) of this difference is added to the wave-breaking induced set-up adds between 0.06 m and 0.137 m. Taking 0.1 m, the resulting increase in elevation at the estuary inlet as a result of wave induced breaking and set-up gradients is estimated as 0.4 m. Note that for future time frames (e.g. 2065-2120) the estimated wave set-up may become non-conservative since warming seas and increased storm intensities will likely lead to higher wave set-up and storm surges in the future (Myeong et al., 2016).

#### 4.3.2.2 Akaroa Harbour

The wave climate for Akaroa Harbour has been previously assessed by Todd et al. (2008) considering both wind waves and refracted swell waves. The largest source of increased water level was used for this assessment which was generally the wind wave source.

#### 4.3.2.3 Lyttelton Harbour

The wave climate for Lyttelton Harbour has been calculated for each site individually based on fetch, wind stress and water depth to assess wave set-up (T+T, 2015b). Fetch and depth limited wave height prediction methods by Young and Verhagen (1996), Goda (2003) and CRESS (Coastal and River Engineering Support System, 2012) have been evaluated and compared using the hourly wind speeds converted from 3 second gusts (AS/NZS 1170.2:2011). The methods by Young and Verhagen (1996) and Goda (2003) incorporate an average depth and beach slope, where the method by CRESS allows multiple sections with variable water depth and beach slope. Due to tidal channels and flats present at Lyttelton Harbour we have adopted the method by CRESS to incorporate depth variations.

Offshore swell waves entering Lyttelton Harbour are not expected to be greater than the wind wave climate and have not been modelled separately. Swell waves are considered to be depth limited and also reduced by refraction as they curve into the sites and shoal over the shallow intertidal flats.

Wave set-up is predicted by using the method as described in the Coastal Engineering Manual (CEM, section II-4-3) (USACE, 2002). This method takes into account the wave height and length and beach slope. Beach slopes between MHWS and LAT, using depth contours from LINZ charts, have been adopted to calculate wave set-up. Wave set-up ranged from 0.15 m to 0.28 m within the Akaroa and Lyttelton Harbour sites (refer to Table 4-15). The wave set-up calculation for each site was based on the direction of the longest fetch distance.

### 4.3.3 Wind set-up

In basins or semi-enclosed basins onshore wind stresses causing wind set-up at the shoreline can become important and should be taken into account for static flood assessments. Wind set-up is included in the storm tide calculated by Goring (2009) for Lyttelton. However, further wind set-up is expected at the head of the harbours due to the additional fetch distance to the site.

The wind set-up has been assessed by comparing formulations by CIRIA (2007) (Construction Industry Research and Information Association) and CRESS. Both methods include wind speed, water depth and fetch length. However, CRESS allows multiple sections with variable depth where CIRIA assumes an average water depth. Wind set-up predictions by CIRIA and CRESS show similar results and range between 0.08 and 0.25 m (refer to Table 4-15). Note two sites within Lyttelton Harbour (Purau and Charteris Bay) do not incur additional wind set up due to being located relatively close to the Port tide gauge location.

### 4.3.4 Sea level rise

Long-term changes in mean sea level should be considered in assessing future inundation levels. Historic sea level rise in Christchurch over the last 100 years is estimated at around 2.0 mm/year

(Hannah and Bell, 2012). Climate change is predicted to accelerate this rate of sea level rise into the future. Section 2.2.5 outlines the current state of scientific knowledge and best practice guidance on sea level rise projections. We have included modelling using sea level rise based on the IPCC projections for the median values of the RCP2.6, RCP4.5 and RCP8.5 scenarios and the RCP8.5+ percentile scenario.

Since tidal characteristics are expected to remain unchanged by future sea level rise, storm tide characteristics are expected to remain similar (MfE, 2008). Therefore, to predict future extreme inundation levels, sea level rise can simply be added to the present day storm tide levels.

### 4.3.5 Coastal inundation values

#### 4.3.5.1 Akaroa Harbour and Lyttelton Harbour

The 2120 coastal inundation levels for both the Lyttelton Harbour and Akaroa Harbour sites are presented in Table 4-15 to
Table 4-18, and displayed as maps in Appendix I (levels reduced to Lyttelton Vertical Datum 1937). The corresponding coastal inundation levels for the 2065 planning time frame are presented in the same tables, and are also mapped in Appendix I. The total inundation levels are based on combining the four components described in Section 4.3. The main difference in the total level between the two time frames is the sea level rise component. We note that the wave set-up and wind set-up component values are the same for both time frames. The SLR values shown in Table 4-15 to Table 4-19 are based on a sea level averaged between 1986 and 2005 (refer to explanation in Section 4.1).

Table 4-15: Coastal Inundation levels for the RCP2.6 M Scenario

| Site | 1% AEP storm tide (m)[1] | Wave set-up (m) | Additional Wind set-up (m) | Sea level rise to 2065 (m) | Sea level rise to 2120 (m) | Total 2065 Inundation Level (m)[2] | Total 2120 Inundation Level (m)[2] |
|---|---|---|---|---|---|---|---|
| Allandale | 1.92 | 0.23 | 0.16 | 0.3 | 0.55 | 2.6 | 2.9 |
| Teddington | 1.92 | 0.21 | 0.25 | 0.3 | 0.55 | 2.7 | 2.9 |
| Charteris Bay | 1.92 | 0.24 | n/a | 0.3 | 0.55 | 2.5 | 2.7 |
| Purau | 1.92 | 0.26 | n/a | 0.3 | 0.55 | 2.5 | 2.7 |
| Wainui | 1.92 | 0.24 | 0.02 | 0.3 | 0.55 | 2.5 | 2.7 |
| Duvauchelle | 1.92 | 0.28 | 0.08 | 0.3 | 0.55 | 2.6 | 2.8 |
| Takamatua | 1.92 | 0.15 | n/a | 0.3 | 0.55 | 2.4 | 2.6 |
| Akaroa North | 1.92 | 0.18 | n/a | 0.3 | 0.55 | 2.4 | 2.7 |
| [1] Lyttelton Vertical Datum 1937 (LVD-37) | | | | | | | |
| [2] Rounded to 1 decimal place | | | | | | | |

Table 4-16: Coastal Inundation levels for the RCP4.5 M Scenario

| Site | 1% AEP storm tide (m)[1] | Wave set-up (m) | Additional Wind set-up (m) | Sea level rise to 2065 (m) | Sea level rise to 2120 (m) | Total 2065 Inundation Level (m)[2] | Total 2120 Inundation Level (m)[2] |
|---|---|---|---|---|---|---|---|
| Allandale | 1.92 | 0.23 | 0.16 | 0.33 | 0.67 | 2.6 | 3.0 |
| Teddington | 1.92 | 0.21 | 0.25 | 0.33 | 0.67 | 2.7 | 3.1 |
| Charteris Bay | 1.92 | 0.24 | n/a | 0.33 | 0.67 | 2.5 | 2.8 |
| Purau | 1.92 | 0.26 | n/a | 0.33 | 0.67 | 2.5 | 2.9 |
| Wainui | 1.92 | 0.24 | 0.02 | 0.33 | 0.67 | 2.5 | 2.9 |
| Duvauchelle | 1.92 | 0.28 | 0.08 | 0.33 | 0.67 | 2.6 | 3.0 |
| Takamatua | 1.92 | 0.15 | n/a | 0.33 | 0.67 | 2.4 | 2.7 |
| Akaroa North | 1.92 | 0.18 | n/a | 0.33 | 0.67 | 2.4 | 2.8 |
| [1] Lyttelton Vertical Datum 1937 (LVD-37) | | | | | | | |
| [2] Rounded to 1 decimal place. | | | | | | | |

Table 4-17: Coastal Inundation levels for the RCP8.5 M Scenario

| Site | 1% AEP storm tide (m)[1] | Wave set-up (m) | Additional Wind set-up (m) | Sea level rise to 2065 (m) | Sea level rise to 2120 (m) | Total 2065 Inundation Level (m)[2] | Total 2120 Inundation Level (m)[2] |
|---|---|---|---|---|---|---|---|
| Alandale | 1.92 | 0.23 | 0.16 | 0.41 | 1.06 | 2.7 | 3.4 |
| Teddington | 1.92 | 0.21 | 0.25 | 0.41 | 1.06 | 2.8 | 3.4 |
| Charteris Bay | 1.92 | 0.24 | n/a | 0.41 | 1.06 | 2.6 | 3.2 |
| Purau | 1.92 | 0.26 | n/a | 0.41 | 1.06 | 2.6 | 3.2 |
| Wainui | 1.92 | 0.24 | 0.02 | 0.41 | 1.06 | 2.6 | 3.2 |
| Duvauchelle | 1.92 | 0.28 | 0.08 | 0.41 | 1.06 | 2.7 | 3.3 |
| Takamatua | 1.92 | 0.15 | n/a | 0.41 | 1.06 | 2.5 | 3.1 |
| Akaroa North | 1.92 | 0.18 | n/a | 0.41 | 1.06 | 2.5 | 3.2 |
| [1] Lyttelton Vertical Datum 1937 (LVD-37) | | | | | | | |
| [2] Rounded to 1 decimal place | | | | | | | |

Table 4-18: Coastal Inundation levels for the RCP8.5+ Scenario

| Site | 1% AEP storm tide (m)[1] | Wave set-up (m) | Additional Wind set-up (m) | Sea level rise to 2065 (m) | Sea level rise to 2120 (m) | Total 2065 Inundation Level (m)[2] | Total 2120 Inundation Level (m)[2] |
|---|---|---|---|---|---|---|---|
| Alandale | 1.92 | 0.23 | 0.16 | 0.55 | 1.36 | 2.9 | 3.7 |
| Teddington | 1.92 | 0.21 | 0.25 | 0.55 | 1.36 | 2.9 | 3.7 |
| Charteris Bay | 1.92 | 0.24 | n/a | 0.55 | 1.36 | 2.7 | 3.5 |
| Purau | 1.92 | 0.26 | n/a | 0.55 | 1.36 | 2.7 | 3.5 |
| Wainui | 1.92 | 0.24 | 0.02 | 0.55 | 1.36 | 2.7 | 3.5 |
| Duvauchelle | 1.92 | 0.28 | 0.08 | 0.55 | 1.36 | 2.8 | 3.6 |
| Takamatua | 1.92 | 0.15 | n/a | 0.55 | 1.36 | 2.6 | 3.4 |
| Akaroa North | 1.92 | 0.18 | n/a | 0.55 | 1.36 | 2.6 | 3.5 |
| [1] Lyttelton Vertical Datum 1937 (LVD-37) | | | | | | | |
| [2] Rounded to 1 decimal place | | | | | | | |

The coastal inundation was mapped for the Akaroa and Lyttelton Harbour sites using the “bath tub” method. The “bath tub” method extrapolates the storm inundation level inland where pathways exist based on a digital elevation model (DEM) derived from LiDAR surveyed post the 2010-2011 Canterbury Earthquakes.

The “bath tub” mapping approach described above assumes that if an inland area is connected to the open coast via a drain/river then this area will be inundated to the equivalent level as the adjacent open coast. This assumption is based on there being no time lags or diminished volumes in flooding the inland areas. Since the sites within both Lyttelton and Akaroa Harbour have relatively steep backshore topography, the “bath tub” approach can be considered a suitable method. A GIS script has been used to discount pools or depressions that are not connected to the sea. The “bath tub” method results in a mapped inland extent of flooding inundation from the sea.

#### 4.3.5.2 Avon-Heathcote Estuary and Brooklands Lagoon

The situation is different for the wide low-lying areas inland of the Avon-Heathcote Estuary and Brooklands Lagoon, where friction will reduce the volume of water that can inundate an area over the peak of the tidal cycle. Therefore, the “bath tub” method which assumes instantaneous inundation of the entire area is not suitable for these locations. We have adopted a different method for the Avon-Heathcote Estuary and Brooklands Lagoon and applied a hydrodynamic model (TUFLOW) to assess the plausible inland extent of coastal inundation.

The model uses LiDAR derived topography and detailed bathymetry of the estuaries and simulates the physics of the tide and inundation levels to dynamically map the land susceptible to coastal inundation hazard. The model bathymetry is gridded to a 5 m x 5 m grid, meaning that the actual topography is represented by a lattice of 5 m x 5 m horizontal cells, with elevation in each cell being taken as an average of the actual ground elevations within each cell. This approach is required for a model-based assessment, such as this. The results from this assessment are output to the same grid as the bathymetry. As a result of this, the “edges” of defined floodplains appear as a series of orthogonal lines at the edge of the 5 m x 5 m grid.

As with the open coast assessment, a 0.4 m allowance was added to predicted storm tide levels to take into account wave set-up at the estuary entrance as an additional factor to add to the boundary levels.

The TUFLOW model used to assess the effects of sea level rise in Christchurch was based on the model derived by T+T for the Earthquake Commission (EQC). The details of the model are outlined in the Increased Flooding Vulnerability: Overland Flow Model Build Report; Volume 3 (T+T, 2014). A summary of the relevant model parameters and calibration testing is provided in Appendix J.

Table 4-19: Summary of coastal inundation level input components for TUFLOW

| | | | RCP2.6 M | | RCP4.5 M | | RCP8.5 M | | RCP8.5+ | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Storm Tide (m) | Set-up allowance (m) | SLR (m) | Total CIHZ level (RL m) | SLR (m) | Total CIHZ level (RL m) | SLR (m) | Total CIHZ level (RL m) | SLR (m) | Total CIHZ level (RL m) |
| 2065 | 1.85 | 0.4 | 0.3 | 2.6 | 0.33 | 2.6 | 0.41 | 2.7 | 0.55 | 2.8 |
| 2120 | 1.85 | 0.4 | 0.55 | 2.8 | 0.67 | 2.9 | 1.06 | 3.3 | 1.36 | 3.6 |
| All levels reduced to Lyttelton Datum 1937 (LVD-1937) | | | | | | | | | | |

By making use of this hydrodynamic modelling approach, it can be seen how extreme levels are damped through the estuary systems. The high/low tide situations are also clearly visible. In Figure 4-15 the instantaneous modelled water depths in the Avon-Heathcote Estuary for the time of low tide at the estuary mouth are shown, using an example result, this being for the 2065 RCP8.5 scenario. In Figure 4-16 the same result file is used, but plotted at the time of high tide at the estuary mouth.

Evident from these two figures is the time lag and amplitude damping of the tidal response through the estuary system.

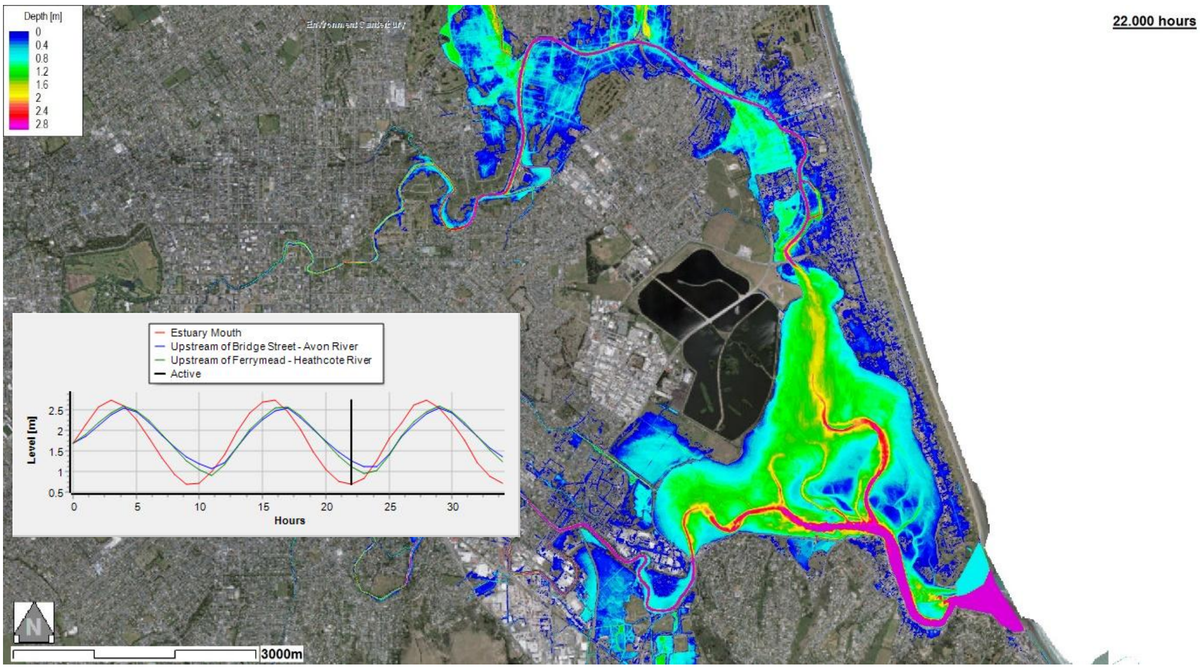


*Figure 4-15 Instantaneous water depth, Avon-Heathcote Estuary, 2065 RCP8.5 for time = 22 hours*

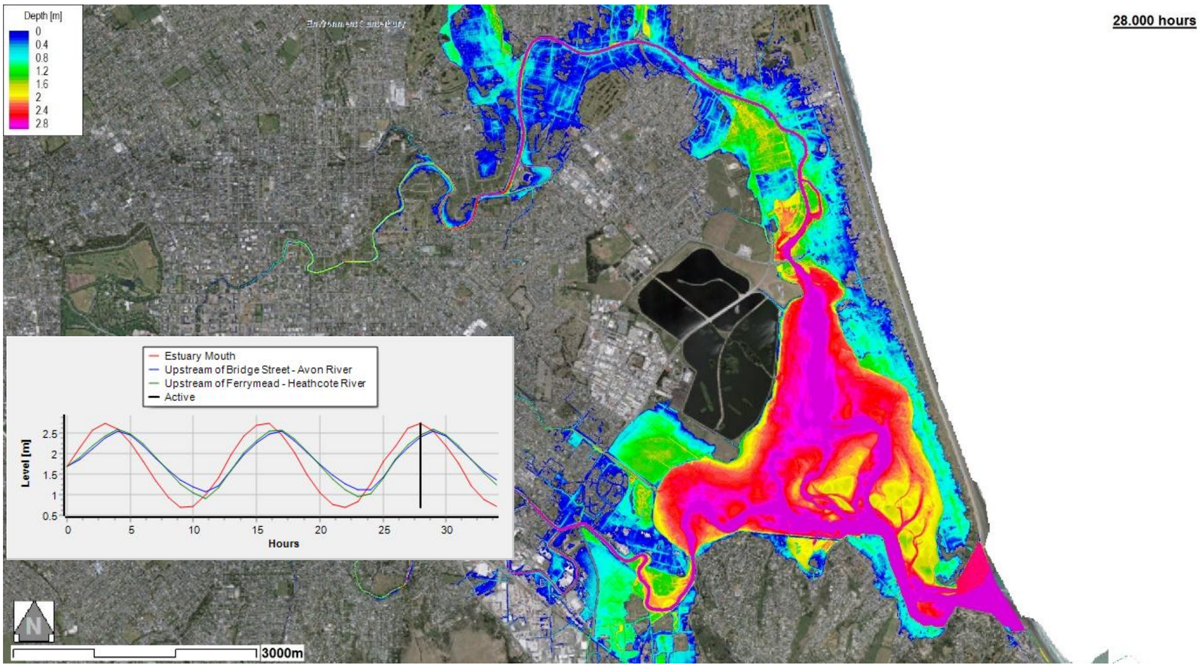


*Figure 4-16: Instantaneous water depth, Avon-Heathcote Estuary, 2065 RCP8.5 for time = 28 hours*

In Figure 4-17 and Figure 4-18 peak flood depth is plotted within the Avon-Heathcote Estuary for the 2065 RCP4.5 and 2120 RCP8.5 scenarios. Overlaid in text are peak water levels attained through simulation of the tidal cycle, using a spring tide amplitude. Note that these peak levels do not occur simultaneously, but are peaks attained during the whole of the simulation (unlike the instantaneous plots shown in Figure 4-15 and Figure 4-16).

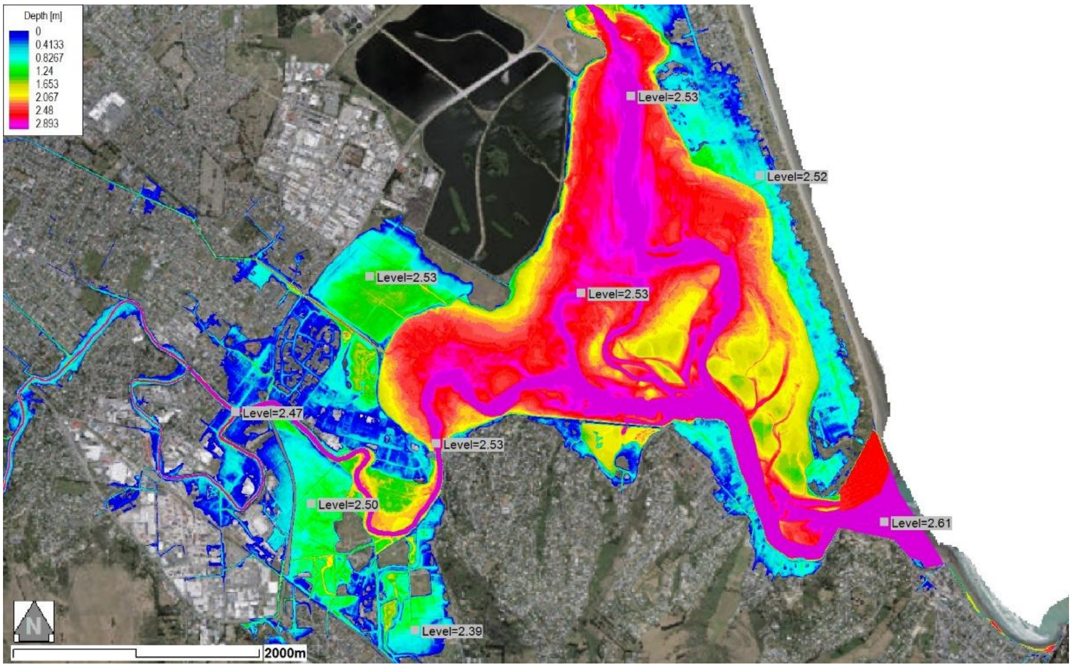


*Figure 4-17: Sample plot of peak flood depth for Avon-Heathcote Estuary, 2065 RCP4.5*

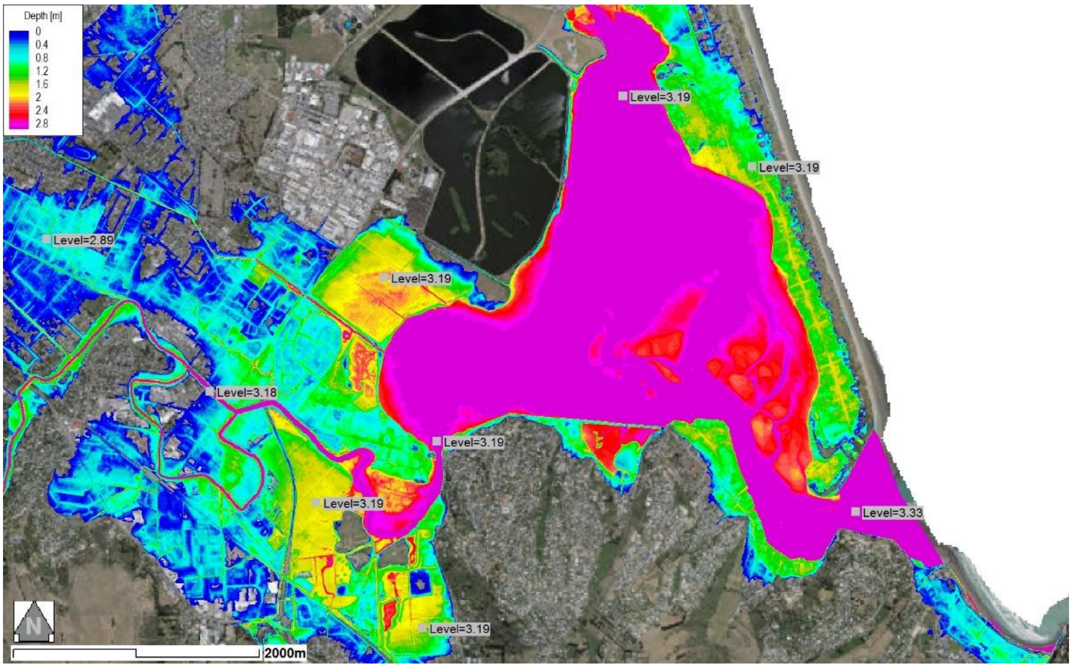


*Figure 4-18: Sample plot of peak flood depth for Avon-Heathcote Estuary, 2120 RCP8.5*

The peak levels attained by hydrodynamic simulation of the tidal cycle have been used in setting the maximum extents of the CIHZ.

## 4.4 Harbour coast coastal erosion hazard zone (CEHZ)

The area of the Avon-Heathcote Estuary subject to potential erosion hazard has been defined by Bridge Street to the north and the bridge crossing the Heathcote River (refer Figure 4-19). Generally the estuary shorelines have been subject to significant modification (refer to Jupp et al., 2007) for pre-earthquake shoreline type assessment). The majority of the coastline along the southern margin forms a major transport corridor (i.e. road) and is generally a hardened/managed shoreline. The Christchurch Wastewater Plant and transport corridors form a significant portion of the western flank of the estuary. The inner shore of the spit has some coastal protection and some evidence of ongoing erosion along portions of the unprotected shoreline. Along the inner shore spit protection works have failed and are likely designated for removal due to earthquake effects including subsidence and liquefaction (D. Hart pers. comm.).

*Figure 4-19: Avon-Heathcote Estuary showing extents of area considered for the erosion hazard assessment (dashed red lines). Managed and modified coasts are shown in green and Council managed seawalls are shown as a solid blue line*

The majority of the shoreline around Akaroa Harbour and Lyttelton Harbour are hard cliff shorelines which are not expected to significantly retreat from coastal erosion over the next 100 years as evidenced by the relatively narrow wave cut platform within the intertidal areas which would have formed over the last 6,500 years when water levels were around present day levels. However the settlements located at the head of the bays are located on soft shorelines with narrow beaches and relatively low-lying backshores and are affected by storm induced erosion. The sites within Lyttelton Harbour and also Duvauchelle and Takamatua located in Akaroa Harbour consist of silty sand or fine sand beaches with wide, shallow intertidal nearshore zones. Akaroa and Wainui both have relatively steep nearshore zones with mixed sand and gravel beaches.

The CEHZ methodology for harbour coasts is based on the same equation set out for open coasts first described in Section 4.2.2.

$$CEHZ = SC + DS + (LT \times T) + SL \qquad \text{(Equation 2)}$$

Where:

| | | |
|---|---|---|
| SC | = | Storm cut/Short-term erosion defined by the horizontal storm cut distance (m). |
| DS | = | Dune stability is characterized by the horizontal distance from the base of the eroded scarp to the backshore crest of a stable angle of repose (m) |
| LT | = | Long-term rate of horizontal coastline movement (m/yr.) |
| T | = | Time frame (years). In this instance a period of 50 and 100 years is used for CEHZ1 and CEHZ2 respectively (i.e. 2065 and 2120) |
| SL | = | Horizontal coastline retreat due to possible accelerated sea level rise (m). |

The components have been derived in a similar way for harbour coasts with a modified term for shoreline retreat due to the effects of sea level rise. The dune/bank stability (DS) and planning time frame (T) components were assessed using the same method as open coasts outlined in Section 4.2.3.3and Section 4.2.3.1 respectively and these are summarised in Table 4-20. The derivation of the other three components is explained below, which is based on site specific assessments of the Lyttelton and Akaroa Harbour environments (refer to Appendix G).

## 4.4.1 Component derivation

### 4.4.1.1 Storm cut (SC)

The short-term erosion due to potential storm cut varies around the various harbour sites, largely due to the steepness of the beach slope and the level of the intertidal area. Table 4-20 shows the shoreline types for the areas inspected and where the coastal edge is a bank, then no short-term fluctuation/storm cut can be expected (i.e. erosion is a long-term process). For the coarse sandy environments a storm cut of 2 m was considered realistic and a smaller storm cut of 1 m was considered appropriate for the finer sandy beaches at the head of the harbours. For the erosion assessment a storm cut distance of either 1 m or 2 m were applied for beach areas and 0 m (zero meter) for banks.

### 4.4.1.2 Long-term (LT)

The results of the aerial photographs assessment shows that most of the shorelines along the harbour sites are relatively stable with little horizontal movement evident apart from the coastlines at Allandale and Charteris Bay. A long-term erosion rate of 0.05 m/yr. was used at Allandale and 0.1 m/yr. was used for Charteris based on the aerial photographs analysis. The long-term component has been set at zero for all other harbour coast sites (refer Table 4-20).

### 4.4.1.3 Sea level rise

The harbour coast beaches consist of either silty sand, fine sand, shell or mixed sand and gravel and have a wide intertidal zone with no extensive dune system. The majority of the terrestrial sediments supplied to the beach areas are from the catchment via the streams that discharge to the coast and, to a lesser degree, from erosion from the cliff coasts adjacent. Therefore harbour coast beaches are expected to behave differently to sandy open coast beaches in response to a rise in mean sea level.

The effect of sea level rise on estuarine type shorelines can be highly variable and complex and will depend on the interrelationship between:

- backshore topography and geology
- sediment supply and storage
- the wave energy acting on the shoreline.

Although small amounts of accretion of some of the shoreline is apparent at some sites (e.g. Purau), it is expected that the acceleration in sea level rise is likely to exceed accretion rates at some point in time in the future (MfE, 2008). Similarly it can be expected that while sedimentation of these estuarine environments is occurring, sea level rise will be greater than the rate of sedimentation and therefore there will be an increase in water depth within the estuary. The greater water depth will allow greater wave heights to act on the shoreline, suggesting that there will be an increased erosion potential. However, as it is a lower energy environment, erosion is likely to occur more episodically and more slowly than a more energetic open coast environment. It is also possible that increased rainfall intensity could increase the supply of sediment to the coast in these relatively short steep catchments. However, this sediment likely consists of fine silt or clay and may not reduce expected future erosion.

The traditional Bruun Rule (Bruun, 1962, 1988) developed for open coast uniform sandy beaches that extend down beyond where waves can influence the seabed does not directly apply for estuarine and gravel type shores where the upper beach is a markedly different composition from the intertidal areas. The upper beach typically consists of consolidated banks or gravels and coarser sands whereas the intertidal areas comprise finer and cohesive sediment.

However, a more simplified equilibrium beach concept that assumes that the upper beach profile is likely to respond to increasing sea level rise with an upward and landward translation over time (Komar et al., 1999) is appropriate. The landward translation of the beach profile (SL) can be defined as a function of sea level rise (**Δ**s) and the upper beach slope (tan**α**). The upper beach slope above the intersection of the beach and the fronting intertidal flats was adopted. The equilibrium profile method relationship is given in Equation 7.

$$SL = \frac{\Delta s}{\tan \alpha} \qquad \text{(Equation 7)}$$

Where:

SL = the landward translation of the beach profile due to sea level rise (m)

Δs = increase in sea level rise (m) taken to be a maximum of 0.5 m to 2065 and 1.0 m to 2120 where the present height of the beach above MHWS is higher than the projected sea level rise increase, or is the height of the beach above MHWS where the beach is lower than these values.

tanα = average slope of the upper beach.

As there are areas where no significant change in shoreline position was observed from the historic aerial photograph analysis there was no discount for historic sea level rise in the values of sea level rise used.

The decision to use the height of the beach crest above MHWS as a limit for sea level rise effects was based on the understanding that in low energy environments there may be insufficient energy to reform beach crests to match the increase in sea level and that once sea levels exceed the crest height, inundation becomes the more significant controlling effect. Therefore the maximum potential extent of erosion as a result of sea level rise for low-lying beach areas was assumed to be controlled by the height of the beach crest or bank above the MHWS. Where the beach crest was higher than the projected sea level rise increases, the sea level value was used. This means that when sea level exceeds the crest height and inundation occurs, there is no additional increase in erosion of the present day shoreline. This method approximately follows the method by Komar et al. (1999), with the MHWS adopted as the dune-toe level.

As shown in Appendix G (Table 1) Teddington and Takamatua are very low-lying, with less than 0.2 m elevation above MHWS while the backshore at Duvauchelle is less than 0.5 m above MHWS.

Charteris Bay, Purau and Akaroa North all have beach crests less than 1 m above MHWS and only Wainui and Allandale have backshore levels greater than 1 m above MHWS.

Based on the largely modified shoreline of Avon-Heathcote Estuary, nominal erosion hazard distances have been considered with resulting hazard distances set in Section 5.2. These nominal distances have been considered with a sole purpose of demonstrating potential erosion effects if these structures are not retained in good order along this shoreline (i.e. to prevent future erosion along the structure). The location of the seawalls within the Avon-Heathcote Estuary, Lyttelton and Akaroa Harbours are included in the CEHZ maps (refer to Appendix H).

Table 4-20: Coastal erosion hazard assessment component values for the harbour coasts

| Site | Slope (1:n) | Height of beach crest above MHWS (m) | Dune slope/ bank stability, DS (m) | Long-term retreat rate, LT (m/yr.) | Storm cut, SC (m) | Sea level rise retreat, SL to 2065 (m) | Sea level rise retreat, SL to 2120 (m) |
|---|---|---|---|---|---|---|---|
| Allandale[1] | 2 | 1.87 | 1.8 | 0.05 | 0 | 1 | 2 |
| Teddington[1] | 2 | 0.17 | 1.4 | 0 | 0 | 0.3 | 0.3 |
| Charteris Bay | 8 | 0.87 | 1.4 | 0.1 | 2 | 4 | 7 |
| Purau | 9 | 0.87 | 1.9 | 0 | 2 | 5 | 8 |
| Wainui | 5 | 2.47 | 2.3 | 0 | 2 | 3 | 5 |
| Duvauchelle | 30 | 0.47 | 2.2 | 0 | 1 | 14 | 14 |
| Takamatua | 27 | 0.17 | 1.9 | 0 | 1 | 5 | 5 |
| Akaroa North | 12 | 0.67 | 1.9 | 0 | 2 | 6 | 8 |

[1]Coastal edge is bank

## 4.5 Sumner

### 4.5.1 Sumner inundation hazard zone

The inundation hazard zone assessment for the Sumner shoreline has been included in the open coast coastal inundation hazard zone assessment (refer to Section 4.1). In so doing, the "bath-tub" approach has been applied. This represents a situation where the existing sea wall and non-return gates on connected waterways do not keep coastal flooding away from the low-lying ground in Sumner.

Using the Digital Elevation Model (DEM) derived from LiDAR it is not possible for features such as seawalls to be fully represented. However, it is known that in some parts, the existing seawall is not continuous, and there are areas through which inundation could pass. As a result of this, the "bath tub" approach adopted is shown to be connected with the inland areas, which are then mapped as being potentially prone to inundation.

### 4.5.2 Sumner coastal erosion hazard zone

The majority of the Sumner shoreline is protected by rock revetments. Council considers that the Sumner rock revetment protects a strategic asset (The Esplanade and Main Rd) and will continue to be maintained to protect the land from coastal erosion. However, CEHZs have been assessed for the Sumner shoreline to characterise the erosion hazard to 2065 and 2120 recognising that existing structures and hard edges will need to be maintained and managed to avoid future erosion hazard effects.

The Sumner shoreline has been split into three behaviour cells based on shoreline composition and behaviour as listed below:

- a sheltered shoreline protected by a rock revetment (adopted as cell H)
- an unprotected unconsolidated shoreline (adopted as cell I)
- an exposed shoreline protected by a rock revetment (adopted as cell J).

The extents of these cells/sections are shown in Figure 4-20.


*Figure 4-20: Site map of Sumner (source: GoogleEarth) including delineation of shoreline and the 1920-1925, 1941 and 1955 historic shorelines*

The approximately 400 m long sheltered shoreline (cell H) is situated within the Avon-Heathcote Estuary entrance (see Figure 4-20). This section is likely sheltered from the high energy open ocean waves because of the presence of the ebb tidal delta, which acts as an energy dissipater. A rock revetment protects Main Rd along this section of the shoreline with cliffs backing the road. We consider that based on the sheltered nature of the modified shoreline, nominal erosion hazard distances are applied to cell H equal to the nominal erosion hazard distances applied to the Avon-Heathcote Estuary shoreline (refer to Section 4.4.1).

The approximately 450 m long unprotected unconsolidated shoreline (cell I) extends from the sheltered rock revetment shoreline to the north-west to Castle Rock to the south-east. Figure 4-20 shows historic shorelines from 1920-1925 (digitised from canterburymaps.govt.nz), 1941 and 1955 (both provided by CCC). Figure 4-20 shows a dynamic character of the shoreline which is typical for an unconsolidated unprotected shoreline situated within an estuary entrance. This behaviour is roughly similar to that of cell G. We consider that adopting the IMC methodology, similar to what was applied to cell G (refer to Section 4.2.6), would be the most appropriate for cell I.

The exposed open coast shoreline (cell J) extending from Castle Rock to Sumner Head (Heberden Ave, see Figure 4-20) is protected by a rock revetment with a length of approximately 1200 m. The open coast CEHZ methodology is considered to be applied to cell J (refer to Section 4.2). However, because of the presence of the seawall the storm cut component (SC) and the long-term component (LT) have been set to zero. Because the crest of the rock revetment has been used as a baseline for cell J the dune stability (DS) component has been set to zero also. The potential coastal erosion hazard zone distance are therefore solely based on horizontal coastline retreat due to possible accelerated sea level rise (SL component).

The sea level rise retreat component values (refer to section 4.4.1.3) for cell J have been derived using the same methodology and sea level rise values as set out in Section 4.2.3.6. The closure slope parameter bounds have been derived as described in Table 4-11 and are shown in Table 4-21. Note that no beach is present along cell J due to the presence of the revetment and the upper bound parameter (beach face) value of cell F (closest to cell J) has therefore been adopted as the upper bound parameter value for cell J.

Table 4-21: Closure slope parameter bounds for cell J (Sumner)

| | Lower bound | mode | Upper bound |
|---|---|---|---|
| Closure slope | 0.005 | 0.008 | 0.029[1] |

[1]Based on upper bound parameter value for cell F for the open coast

# 5 Coastal erosion hazard assessment results

Components have been assessed for each coastal cell based on the data and methodologies described in Section 4. The open and harbour coast CEHZ results are presented in Section 5.1 and Section 5.2 respectively.

## 5.1 Open coast CEHZ values

For each coastal cell a range of CEHZ probabilistic values are calculated and following consultation with the Christchurch City Council in 2014, the P66% value for 2065 (value with a 66% likelihood of being exceeded by 2065) and the P5% value for 2120 (5% likelihood of being exceeded by 2120) were adopted as prudent likely and potential coastal erosion hazard zones values termed the CEHZ2065 and CEHZ2120 respectively. The 66% and 5% likelihoods taken as respectively 'likely' and 'potential' are derived from MfE (2008), shown in Table 4-13, and are suggested to be considered by the NZCPS (2010).

The P66% and P5% CEHZ distances based on the probabilistic assessment are presented in Table 5-1 for both the current (2015) and future (2065 and 2120) time frames. The full set of both the histogram and cumulative distribution function graphs from the probabilistic assessment output are presented in Appendix K for each site. The present-day coastal erosion hazard is based on the storm cut (SC) and dune stability (DS) components only.

Table 5-1 shows that the present day CEHZ results show a greater inland extent of erosion than the future 2065 time frame results for some RCP scenarios. This is due to the long-term accretion trend in these locations being greater than the potential retreat due to sea level rise (in particular for lower RCP scenarios) over the next 50 years.

The difference in P66% CEHZ distance for the various RCP scenarios in 2065 is in the order of -10 m to -15 m for both sediment budget scenarios, with greater landward distances of erosion for higher RCP scenarios. The P5% results for 2120 show that the difference in CEHZ value between the highest and lowest RCP scenario is in the order of up to -100 m (e.g. cell E) for both sediment budget scenarios. This shows the significance of sea level rise component in particular for longer time frames (e.g. beyond 2120).

The significance of considering the average sediment budget or reduced sediment budget scenario is less compared to the sea level rise component. For the 2065 time frame the CEHZ distances the difference is in the order of several metres, and for the 2120 time frame the difference is in the order of -10 m. Typically a greater landward CEHZ distance is found for the reduced sediment budget scenario, which includes a reduced long-term accretion rate.

For both 2065 and 2120 the smallest landward CEHZ distances are found for cell A (up to -86 m). The greatest distances are found for cells B, D and E (up to -119 to -127 m). The CEHZ2065 distances range from 6 m to -46 m across all cells depending on adopted RCP and sediment budget scenario. The CEHZ2120 distances range from 22 m to -127 m depending on adopted RCP and sediment budget scenario. The CEHZ values have been mapped with respect to the adopted baseline and are presented in Appendix H.

The South New Brighton spit is expected to be susceptible to erosion from both the open coast and the harbour coast edges. Due to the relatively low-lying land on the harbour side of the spit, erosion is expected to potentially effect the full alongshore length of the spit along the southern 2.5 km (i.e. south of Caspian Street) over the 2120 time frame. Figure H-29 (a and b) located in Appendix H shows the land susceptible to erosion over the 2065 and 2120 time frame.

Table 5-1: Probability (66% and 5%) of CEHZ exceedance results (in metres) for Southern Pegasus Bay for both the 2065 and 2120 time frame

| Cell | Scenario | Present Day | | 2065 | | | | 2120 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Average Sediment Budget | | Reduced Sediment Budget | | Average Sediment Budget | | Reduced Sediment Budget | |
| | | 66% | 5% | 66% | 5% | 66% | 5% | 66% | 5% | 66% | 5% |
| Cell A | RCP2.6 | -7 | -15 | 5 | -9 | 4 | -11 | 20 | -6 | 12 | -14 |
| | RCP4.5 | | | 4 | -11 | 2 | -13 | 14 | -16 | 6 | -24 |
| | RCP8.5 | | | 0 | -17 | -1 | -18 | -4 | -50 | -12 | -58 |
| | RCP8.5+ | | | -6 | -28 | -8 | -30 | -17 | -77 | -26 | -86 |
| Cell B | RCP2.6 | -7 | -15 | -8 | -22 | -10 | -24 | -8 | -34 | -16 | -42 |
| | RCP4.5 | | | -10 | -25 | -12 | -27 | -15 | -45 | -24 | -53 |
| | RCP8.5 | | | -15 | -32 | -16 | -33 | -38 | -82 | -46 | -92 |
| | RCP8.5+ | | | -22 | -45 | -24 | -46 | -54 | -113 | -63 | -123 |
| Cell C | RCP2.6 | -8 | -15 | -5 | -18 | -7 | -19 | 0 | -23 | -9 | -31 |
| | RCP4.5 | | | -7 | -20 | -8 | -22 | -6 | -34 | -15 | -42 |
| | RCP8.5 | | | -10 | -27 | -12 | -28 | -25 | -72 | -33 | -81 |
| | RCP8.5+ | | | -16 | -40 | -18 | -41 | -38 | -102 | -47 | -111 |
| Cell D | RCP2.6 | -8 | -15 | -4 | -18 | -6 | -19 | 1 | -23 | -7 | -32 |
| | RCP4.5 | | | -6 | -21 | -7 | -22 | -5 | -36 | -13 | -45 |
| | RCP8.5 | | | -9 | -28 | -11 | -29 | -22 | -76 | -31 | -86 |
| | RCP8.5+ | | | -15 | -42 | -17 | -43 | -35 | -108 | -43 | -119 |
| Cell E | RCP2.6 | -7 | -15 | -4 | -16 | -5 | -18 | 2 | -21 | -7 | -29 |
| | RCP4.5 | | | -6 | -20 | -7 | -21 | -6 | -36 | -14 | -43 |

| Cell | Scenario | Present Day | | 2065 | | | | 2120 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Average Sediment Budget | | Reduced Sediment Budget | | Average Sediment Budget | | Reduced Sediment Budget | |
| | | 66% | 5% | 66% | 5% | 66% | 5% | 66% | 5% | 66% | 5% |
| | RCP8.5 | | | -10 | -28 | -12 | -30 | -29 | -83 | -38 | -91 |
| | RCP8.5+ | | | -18 | -44 | -20 | -46 | -47 | -120 | -55 | -127 |
| Cell F | RCP2.6 | -7 | -14 | 6 | -7 | 4 | -8 | 22 | -1 | 14 | -9 |
| | RCP4.5 | | | 4 | -10 | 2 | -11 | 15 | -15 | 6 | -23 |
| | RCP8.5 | | | 0 | -18 | -2 | -20 | -8 | -63 | -16 | -70 |
| | RCP8.5+ | | | -8 | -34 | -9 | -35 | -25 | -99 | -33 | -107 |

## 5.2 Harbour coast CEHZ values

The results of the coastal erosion hazard assessment for the bays within Akaroa and Lyttelton Harbours are set out in Table 5-2. The results show relatively small areas of potential susceptibility to erosion by 2065 (2 to 17 m) and this increases to 2 to 20 m by 2120. Due to uncertainties in information and knowledge of processes, we recommend a minimum CEHZ of 5 m be adopted for planning purposes and that all these offsets be applied from the vegetation line as established from the latest aerial photograph assessment. The decision to make the set-back (i.e. CEHZ) a minimum of 5 m only applies to Teddington, as all the other locations have set-back extents of at least 5 m.

Due to the largely modified shoreline around the Avon-Heathcote Estuary (refer to Appendix G; Table 1) a nominal 5 m is recommended around the coast to characterise the erosion hazard to 2065, increasing to 10 m by 2120 recognising that existing structures and hard edges will need to be maintained and managed to avoid future erosion hazard effects.

Table 5-2: Summary of CEHZ component and resultant values based on the equilibrium profile method

| Site | CEHZ 2065 (m) | CEHZ 2120 (m) |
|---|---|---|
| Allandale | 5 | 9 |
| Teddington | 2 (5[1]) | 2 (5[1]) |
| Charteris Bay | 12 | 20 |
| Purau | 8 | 12 |
| Wainui | 7 | 9 |
| Duvauchelle | 17 | 17 |
| Takamatua | 7 | 7 |
| Akaroa North | 10 | 12 |
| Avon-Heathcote Estuary | 5 | 10 |
| Note 1: recommend that the calculated value of 2 m be increased to 5 m due to uncertainties and limited information | | |

# 6 Coastal inundation hazard assessment results

The land susceptible to coastal inundation hazard was identified for both the 2065 and 2120 time frame as coastal inundation hazard zones (CIHZ). The inundation levels are presented in Section 4.3.5. The CIHZ for the harbour coast was mapped using two methods:

- Connected "bath-tub" method – maps the area of land below the inundation level based on LiDAR derived topography, where there is a connection pathway to the sea. This method was used for sites located within both the Lyttelton and Akaroa harbours.
- Dynamic model method – simulates the physics of the tide and inundation levels to dynamically map the inundation levels based on LiDAR derived topography and detailed bathymetry of the estuary. This method is most appropriate for wide flat areas and was used for Avon-Heathcote Estuary and the Brooklands Lagoon.

The CIHZ for the open coast was mapped using the "connected bath-tub" approach, except for the area between Waimairi and the New Brighton spit, where the hydrodynamic model was used. The reasons are explained in Section 4.3.5.

The results of the CIHZ assessments for the harbour and open coasts were mapped for both the 2065 and 2120 time frame and are presented in Appendix I.

# 7 Discussion

Coastal processes (e.g. water levels, waves, and sediment transport) and future shoreline positions are difficult to accurately forecast over a 100 year time frame due to the potential for morphological feedbacks (i.e. response of shorelines due dynamic interaction between coastal processes and shoreline position) to slow or increase the rates of historic trends. These forecasts become more uncertain when considering the effect of potential sea level rise and interrelationships with other systems (i.e. spit and estuary inlets).

Some areas of the open coast have areas of relatively narrow dune vegetation where backshore areas comprise revetment, grass reserve or residential development. We expect dune recovery to be negatively affected where native dune vegetation has been removed. Removal of dune vegetation could result in a greater erosion response in both the long-term and short-term than historically experienced because sediment is unable to accumulate with no erosion buffers created along these areas.

The probabilistic method used for the open coast includes the uncertainty within each component to produce a range of CEHZ distances. The results of this assessment will assist the Christchurch City Council and others to consider this uncertainty when selecting a probability of exceedance output in accordance with risk-based guidance provided in the NZCPS (2010).

The harbour coast CEHZ assessment adopts a building-block method to identify land susceptible to coastal erosion hazard. Although we have not yet developed the harbour coast methodology to incorporate the probabilistic approach, the methods are commonly applied and are still in accordance with best practice guidelines.

We recommend continuing to monitor the shoreline position at both open coast and harbour coast sites by mapping shoreline positions from aerial photographs or GPS surveys along with continuing the traditional beach profile dataset carried out by Environment Canterbury. The shoreline monitoring will provide measured/recorded data to help resolve these uncertainties for future re-assessments.

# 8 Summary and conclusions

Christchurch City Council (Council) commissioned Tonkin & Taylor Ltd (T+T) in 2015 to prepare a report and maps that identified areas susceptible to coastal hazards (inundation, erosion and sea level rise) for the main coastal settlements selected by Council. The report was subject to a detailed expert panel peer review in 2016. This report updates the original 2015 report and addresses the recommendations of the peer review panel.

The areas potentially susceptible to coastal hazards were termed coastal erosion hazard zones (CEHZ) and coastal inundation zones (CIHZ). The zones have been mapped for both the open and harbour coast over both an approximate 50 year (2065) and 100 year (2120) planning time frame for both the open and harbour coast for four climate change scenarios (median projections for RCP2.6, RCP4.5 and RCP8.5 and the 83$^{rd}$ percentile of RCP8.5).

The CEHZ methodology used in this study combines standard and well-tested approaches for defining coastal erosion hazard zones by the addition of component parameters. This method has been refined for the open coast to include parameter bounds which are combined by stochastic simulation. The resulting distribution is a probabilistic forecast of potential hazard zone width, rather than including single values for each component and one overall factor for uncertainty.

This approach produces a range of hazard zones (probability distribution) corresponding to differing likelihoods which may be applied to risk-based assessments as advocated by the NZCPS (2010) and supported by best practice guidelines. Following consultation with Council, the P66% CEHZ value at 2065 and the P5% CEHZ value at 2120 are adopted as likely and potential CEHZ values (termed CEHZ1 and CEHZ2 respectively).

We implemented separate methodologies to assess coastal hazards for the open coast and the harbour coast sites due to the different processes driving each of the two coastal environments. The harbour coast CEHZ methodology accounts for the low-lying morphology typical of these sites. Although we have not yet developed the harbour coast methodology to incorporate the probabilistic approach, the method is in accordance with best practice guidelines.

The CIHZ was mapped using two methods:

- Connected "bath-tub" method – maps the area of land below the inundation level based on LiDAR derived topography, where there is a connection pathway to the sea. This method was used for sites located within both the Lyttelton and Akaroa Harbours and the open coast.
- Dynamic model method (TUFLOW) – simulates the physics of the tide and inundation levels to dynamically map the inundation levels based on LiDAR derived topography and detailed bathymetry of the estuary. This method is beneficial for wide flat areas and was implemented for Avon-Heathcote Estuary, Brooklands Lagoon, Sumner and for the New Brighton coast.

We recommend continuing to regularly monitor the shoreline position and inundation levels across the region to provide background data, including continuing beach profile monitoring and digitising shorelines from aerial imagery or by GPS survey. We also recommend the adopted baselines and both the CEHZ and CIHZ values are reassessed at least every 10 years or following significant changes in either legislation or best practice and technical guidance.

# 9 Applicability

This report has been prepared for the exclusive use of our client Christchurch City Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

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# Appendix A: Site location plan

# Appendix B: Beach profile output plots

# Appendix C: SBEACH assessment

# Appendix D: Storm cut matrix

# Appendix E: DSAS maps (including beach profile locations)

# Appendix F: Assessment of input parameters for long-term distribution

# Appendix G: Site specific assessments of the Lyttelton and Akaroa Harbour environments

# Appendix H: CEHZ result maps

# Appendix I: CIHZ result maps

# Appendix J: TUFLOW model description

# Appendix K: Open Coast CHEZ probabilistic model outputs

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