

**Himatangi Beach
Coastal Hazards Assessment
Manawatu District**



July 2015

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July 2015
Report No: 2015/EXT/1422
ISBN: 978-1-927259-2-0

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Executive Summary

The shoreline at Himatangi Beach has been assessed for coastal hazards. A comprehensive CEHZ (Coastal Erosion Hazard Zone) and CIHZ (Coastal Inundation Hazard Zone) assessment was undertaken.

It has been shown that there is no risk to properties at Himatangi Beach from either coastal inundation or erosion over the 100 year planning timeframe considered by this investigation. The mapping of the hazard zones shows that the landward extent of these zones is located within the dunes behind the beach at Himatangi.

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COASTAL HAZARDS ASSESSMENT

HIMATANGI BEACH

1 Introduction

A progressive review of the information available on coastal hazards by Horizons Regional Council (HRC) has identified the need for an assessment of the nature and scale of coastal hazards along the coastline at Himatangi.

The purpose of this assessment is to identify the coastal hazards to which the area is subjected, in order that the respective councils can then make informed decisions as to how to manage existing and new activities and development in the area.

1.1 Scope of Work

Task 1. To identify an 'area sensitive to coastal hazards' along the full shoreline at Himatangi.

Task 2. To investigate and identify in detail the nature and scale of coastal hazards that exist using the following criteria:

- a. Design waves and storm surge wave run-up (SWRU) in a 1% AEP storm (note care is to be taken in any wave model that the extreme waves are not damped out due to model limitations).
- b. Storm surge in a 1% AEP storm including:
 - i. Barometric set-up;
 - ii. Wind set-up;
 - iii. Wave set-up;
 - iv. Sea level rise of 0.31 m to 2065 and 0.95 m to 2115, being the most recent estimate from the Ministry for the Environment Guidelines 2008; and
 - v. Any other physical processes.
- c. Erosion hazards to be based on:
 - i. Short-term fluctuations in shoreline position (ST);
 - ii. The shoreline response to storm erosion (SE);
 - iii. Long-term response to storm erosion (LT);
 - iv. Dune stability factor (DS);
 - v. The magnitude of shoreline retreat from sea level; and
 - vi. Any other processes (e.g. river mouth migration).
- d. Appropriate freeboard and safety margins.
- e. Scenarios are to be modelled for:
 - i. The existing (2015) situation;
 - ii. Inclusion of climate change impacts to the year 2065 including an agreed provision for wave intensification; and
 - iii. Inclusion of climate change impacts to the year 2115 including an agreed provision for wave intensification.

1.2 Deliverables

A report outlining or including:

1. The investigations undertaken and the calculations, findings and conclusions reached to meet **Task 2** as defined in the scope.
2. The analysis of the options investigated, the finding and conclusions reached in evaluating the coastal hazard risk to meet **Task 1** as defined in the scope.
3. That the requirements described in **Task 2**, of the scope, have been satisfied.

4. Plans and maps, as appropriate, that are in a form compatible with the GIS systems of HRC [ESRI's ArcGIS] and Manawatu District Council (MDC), and that map and plan information includes NZTM and NZMG projections and comes with completed metadata files.

River flooding inundation is not considered in this investigation.

Tsunami risks are not included in this investigation as they have previously been studied by GNS Science.

2 Site Description

Himatangi is located on the west coast of the North Island, at the mouth of the Kaikokopu Stream, approximately 30 km west of Palmerston North. Refer Figure 2.1 below.

Figure 2.1 - Location of Himatangi Beach



The settlement is located along approximately 750 m of the coast and extends approximately 1.2 km inland.

3 Historic Shoreline Change

To analyse the historic shoreline change at Himatangi Beach aerial photographs from 1963, 1995, 2005 and 2011 were geo-referenced and analysed.

The historic aerial photographs were digitised and geo-referenced using ArcGIS software. The estimated relative accuracy between images for this methodology is +/- 2.5 m.

In each photograph, the seaward line of vegetation was digitised as it represents a common shoreline feature. This feature was chosen because it forms a sharp discontinuity in contrast in the photographs. Figure 3.1, overleaf, shows these lines overmarked on the 2011 aerial image of Himatangi Beach.

As clearly shown in Figure 3.1, the shoreline at Himatangi is accreting (building out). This is in keeping with patterns seen along the west coast of the Horizons Region that are supported by anecdotal evidence and observed trends.

Analyses of changes in the shoreline position were undertaken at 11 cross sections, spaced at 100 m intervals along the beach. The locations of the cross sections and the full results of the analysis can be seen in Appendix A.

The analysis showed that the rate of beach accretion has been relatively steady since 1963. Between 1963 and 1995 the seaward vegetation line moved approximately 1.65 m/year. Since 1995, this movement has occurred at a slightly slower rate of approximately 1.19 m/year. This later rate of accretion occurred over both the 1995-2005 and 2005-2011 timeframes.

Due to the inaccuracies present in this type of analysis, it is considered appropriate to assume that the rate of accretion of the shoreline is the full long-term average rate that occurred between 1963 and 2011. Hence, the historic rate of shoreline change is an accretion of 1.5 m/year.

Figure 3.1 - Historic Shoreline Position Change



4 Coastal Erosion Hazard Zone Assessment

4.1 Methodology

To maintain a consistent approach to the assessment of coastal erosion hazards in the Horizons Region, the methodology used in the Waikawa to Waitarere Coastal Hazard Assessment (Tonkin and Taylor, 2013), has been used in this assessment.

This methodology to determine the Coastal Erosion Hazard Zones (CEHZ) includes the cumulative addition of:

- Predicted climate change effects;
- Expected long-term erosion rates;
- Episodic storm induced erosion and short-term fluctuations in shoreline movement;
- Dune stability; and
- Inlet migration.

4.1.1 Open Coast

The equation below models the CEHZ for the open coast:

$$\text{CEHZ (open coast)} = [\text{LT}]\text{T} + \text{SLR} + \text{ST} + \text{DS} + \text{FS} \quad (1)$$

Where:

CEHZ= The width of the Coastal Erosion Hazard Zone for open coast sandy shoreline (m).

LT = Historic long-term rate of horizontal shoreline movement (m/yr).

T = Planning timeframe (years).

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

ST = Horizontal distance of shoreline retreat from both storm induced erosion and short-term fluctuations in the long-term trend of shoreline movement (m).

DS = Horizontal retreat of the vertical erosion scarp based on the angle of repose for loose sand (m).

FS = Factor of safety/uncertainty (m).

4.1.2 Components

4.1.2.1 Planning Timeframe (T)

The three timeframes described in the project brief have been applied. These timeframes are defined below:

- Current Erosion Hazard Zone (2015) - CEHZ;
- 2065 Erosion Hazard Zone (50 years) - 2065EHZ; and

- 2115 Erosion Hazard Zone (100 years) - 2115EHZ.

4.1.2.2 Sea Level Rise Effects (SLR)

The frequency of wave attack, on the fore dunes, is likely to increase with future sea level rise. It is likely that this will lead to increased erosion, even at beaches that have been relatively stable over time.

As with the Waikawa to Waitarere Coastal Hazard Assessment (Tonkin and Taylor, 2013), it is not considered appropriate to make any adjustments for vertical land displacement (uplift from earthquakes) when considering sea level rise during the 100 year planning timeframe.

The approach that has been used is to assume that the sediment supply and active beach width remains constant during a change in sea level (equilibrium beach concept). The beach profile is likely to respond to these conditions with an upward and landward transition over time (Komar, McDougal, Marra, & Ruggiero, 1999). The landward translation of the beach profile (SLR) can be defined as a function of sea level rise and the active beach slope. This method of describing the equilibrium beach concept is a variation of the Bruun Rule and is given in equation 2 below.

$$SLR = \frac{\Delta s}{\tan \alpha} \quad (2)$$

Where:

SLR = Landward translation of the beach profile (m).

Δs = Predicted rise in sea level (m).

α = Average intertidal slope.

The predicted sea level rise has been assumed to be 0.31 m to 2065 and 0.95 m for 2115, which are the most recent estimates from the (Ministry for the Environment (MfE), 2008).

An analysis of surveyed cross sections, which can be seen in Appendix B, shows that the average inter-tidal slope at Himatangi is approximately 1:20.

Using these assumptions, and the model described above, the potential landward movement of the beach has been calculated for both of the sea level rise scenarios. The results are summarised in Table 4.1 below:

Table 4.1 - Landward Movement due to Sea Level Rise

Planning timeframe	Inter-tidal slope	Sea level rise (m)	SLR distance (m)
2015 - 2065	1:20	0.31	6.2 m
2015 - 2115	1:20	0.95	19 m

4.1.2.3 Long-Term Rates of Shoreline Movement (LT)

The historic trend in the movement of the shoreline at Himatangi Beach is discussed in **Section 3**. The analysis of historic aerial photographs has shown that the shoreline has been accreting at a rate of approximately 1.5 m/yr.

It is important to note, that future shoreline movement may differ from the historical trend due to changing climatic patterns associated with the Inter-decadal Pacific Oscillation (IPO) and global climate change.

As a conservative approach to the determination of the Coastal Erosion Hazard Zone at Himatangi Beach, the long-term rate of shoreline movement (LT) has been considered to be zero. This means that continued accretion of the beach is not included in the model.

4.1.2.4 Short-Term Shoreline Movement (ST)

Both storm induced erosion and fluctuations are the long-term trend of shoreline movement are included in the short-term erosion rate.

The most obvious cause of short-term erosion is the effect of severe wave storms attacking the coast. However, one must also consider short-term fluctuations over a longer period than a single storm event. These fluctuations include responses to natural variations in climatic conditions such as the El Nino Southern Oscillation (ENSO), which usually occurs on a 3-7 year cycle.

To determine the likely magnitude of short-term shoreline movement, it is usual to examine aerial photographs or survey information that has been gathered at frequent intervals over a period of time. Unfortunately, there is not a sufficient quantity of this data to make an accurate estimate of short-term shoreline movement at Himatangi Beach. However, (Tonkin and Taylor, 2013), calculated a conservative estimate of 30 m, and applied that as the ST factor along the entire coast between Waikawa and Waitarere. It is considered that this conservative estimate is appropriate to apply at Himatangi.

4.1.2.5 Dune Stability (DS)

As defined by Tonkin and Taylor (2013), the dune stability factor delineates the area of potential risk landward of the erosion scarp. This parameter is based on the height of the existing backshore and the angle of repose for loose dune sand (34°), and is described by equation 3 below.

$$DS = \frac{h}{2 \tan \alpha} \quad (3)$$

Where:

DS = dune stability factor (m).

h = height of the existing backshore (m).

α = angle of repose for loose dune sand.

The maximum height of the dunes at Himatangi was taken as 8 m. This height was determined from the survey information contained in Appendix B. From this height, and using the model described above, the dune stability factor at Himatangi Beach has been calculated to be 5.9 m.

4.1.2.6 Factor of Safety (FS)

Due to the uncertainties in the short-term shoreline movement (ST) it is appropriate to allow a factor of safety in the determination of the Coastal Erosion Hazard Zones. In keeping with the Waikawa to Waitarere Coastal Hazard Assessment (Tonkin and Taylor, 2013), a factor of safety of 5 m was deemed to be appropriate for this assessment.

4.1.3 Inlets

The position of the shoreline at many beaches is heavily influenced by river inlets. However, at Himatangi Beach the outlet of the Kaikokopu Stream is controlled by the engineering works downstream of the Surf Lifesaving Club. Since these works help to maintain the vehicular access to the beach it is considered that the structures will be maintained so as to ensure their structural integrity.

This 'fixing' of the inlet means that, in terms of Coastal Erosion Hazard, the influence of the inlet is negated and the beach can be considered as an open coast.

4.2 Coastal Erosion Hazard Zones

Table 4.2 below presents a summary of the CEHZ distances calculated for the planning timeframes at Himatangi Beach.

Table 4.2 - CEHZ Results

SLR		LT (m/yr)	ST (m)	DS (m)	FS (m)	CEHZ		
2065 (m)	2115 (m)					Current (m)	2065 (m)	2115 (m)
6.2	19	0	30	5.9	5	40.9	47.2	59.9

4.3 Coastal Erosion Hazard Zone Mapping

The seaward line of vegetation, which was digitised from the 2011 aerial imagery, was used as the CEHZ offset origin. The CEHZ is measured horizontally inland from the baseline at right angles to the general alignment of the foreshore. The Coastal Erosion Hazard Zones have been mapped digitally and are shown on the 2011 aerial photograph in Figure 4.1, overleaf.

5 Coastal Inundation Hazard Assessment

The Coastal Inundation Hazard Zones (CIHZ) at Himatangi have been assessed for storm surge and wave run-up events. Inundation from river flooding or tsunami events have not been modelled as they do not form part of the project brief.

5.1 Components of Storm Surge and Wave Run-Up

Storm surges are caused by coastal storms raising sea levels through a combination of the drop in atmospheric pressure and wind driving the sea onto land. The rising of the sea level in response to falling atmospheric pressure is known as barometric set-up. Wind set-up is the change in water levels due to the wind. Note: wind set-ups can be of two types, being either that caused on open coast or differential effects across and estuary.

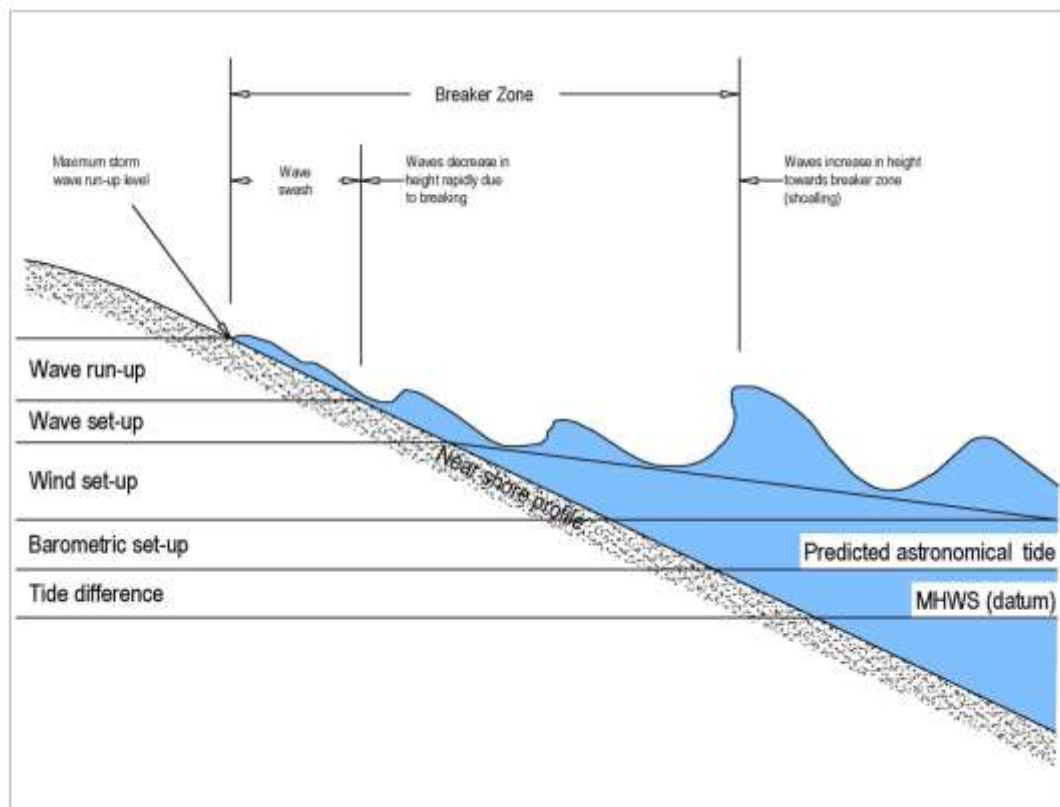
The 'stillwater' level is considered to be the combination of the astronomical tide, barometric set-up and wind set-up.

In addition to the stillwater level, there is a near-shore increase in water levels caused by waves approaching land. This is called wave set-up.

For the open coast a significant component of the peak water level reached on the land is the wave run-up. Wave run-up is defined as the sum of wave set-up and wave swash.

Figure 5.1, below, illustrates these components graphically.

Figure 5.1 - Components of Storm Surge and Wave Run-Up



5.1.1 Astronomical Tide

The astronomical tide is caused by the gravitational attraction of the sun and moon on the Earth's waters. The astronomical tides are estimated based on around 600 components and presented in the New Zealand Nautical Almanac and other publications. The almanac presents detailed data on the Port Taranaki site (LINZ, 2015) and further data on the Secondary Ports site at Manawatu River (LINZ, 2015). This data is summarised in Table 5.1 below.

Table 5.1 - Astronomical Tide Levels

Astronomical Tide Levels	Port Taranaki		Manawatu River	
	(m above Chart Datum)	(m above Wellington Datum)	(m above Chart Datum)	(m above Wellington Datum)
Mean High Water Springs (MHWS)	3.59	1.806	2.4	0.436
Mean High Water Neaps (MHWN)	2.8	1.106	1.8	0.106
Mean Sea Level (MSL)	1.95	0.256	1.3	-0.394
Mean Low Water Neaps (MLWN)	1.07	-0.624	0.9	-0.794
Mean Low Water Springs (MLWS)	0.28	-1.14	0.2	-1.494

5.1.2 Barometric Set-Up

Depressed atmospheric pressures cause a rise in water levels. This is sometimes referred to as a bulge of water following the centre of the depression. The general increase in water levels is one centimetre per hectapascal (or millibar) drop in atmospheric pressure. Conversely, there is also a corresponding drop in sea level associated with rises in atmospheric pressure.

The standard equation for calculating the barometric set-up is

$$\Delta H_{BAR} = z(1014 - pa)$$

Where:

- ΔH_{BAR} = Surface elevation change (m).
- z = Barometric factor with standard value of 0.01.
- pa = Atmospheric pressure at sea level (HPa or mb).

The value of z varies slightly from place to place, but the given factor is a good guide. When a depression moves quickly the change in water levels is superimposed as a surge on prevailing water levels. The surge behaves as a long wave with a wave length approximately equal to the width of the depression. Along the open coast these long waves may increase in amplitude as a result of shoaling on the near shore bathymetry, while within harbours amplitudes may increase or decrease depending on the harbour geometry and conveyance capability through the mouth. The response of sea level to pressure changes occurs relatively slowly,

taking in the range 2 to 12 hours, and affects large areas in approximately uniform manner (Tonkin and Taylor, 2002).

Barometric set-up can also be harmonically enhanced by the speed of travel of the storm. This occurred in central parts of New Zealand in Cyclone Giselle (the “Wahine” storm) in April 1968 (Heath, 1979).

5.1.3 Wind Set-Up

The surface shear stress caused by the wind (called the geostrophic wind) travelling over the sea surface drives water from the prevailing wind direction. An on-shore wind thus drives a ‘wedge’ of water against the land. The magnitude of the height of this wedge is known as the wind set-up. There are complex equations available to calculate the wind set-up dependant on several factors including:

- Intensity, duration and direction of high winds;
- Coastline bathymetry (the wind set-up) is greater in shallow waters); and
- Coastline geometry. The concave shape of the Manawatu-Wanganui Region is minorly conducive to enhancing surge levels. This is a much documented phenomenon during hurricanes, with significantly elevated levels on the Gulf of Mexico coastline in the Caribbean Sea.

5.1.4 Total Stillwater Level

The total stillwater level is the combined total of the astronomical tide, barometric set-up and wind set-up. These may not act with peak magnitude simultaneously. For example, the peak set-ups may occur at low tide (although generally there would not be a significant attenuation in set-ups by the preceding or succeeding high tide), similarly the surge on various parts of the New Zealand coast caused by both Cyclones Giselle (1968) and Fergus (1996) occurred on neap tides.

5.1.5 Wave Set-Up

Wave set-up is a super-elevation of the water surface over normal surge elevation due to onshore mass transport of water by wave action alone. It is caused by energy dissipation due to the shoaling of incoming waves, and is more pronounced in environments with steeper beach slopes and hence the depth of breaking is closer to the shoreline.

It is smaller in restricted fetch environments, low beach slopes and shallow water depths (Tonkin and Taylor, 1999).

5.1.6 Wave Run-Up

The wave run-up component is the elevation above the combined level from the other components reached by the wave swash. It is dependent upon breaking wave height and period, beach slope and the resistance characteristics of the beach sediment. It increases with wave height, period and beach slope and decreases with coarser sediments.

It is also affected by the degree of sheltering from headlands, islands or reefs, and wave angle with the shoreline.

5.2 Methodology

In order to provide a consistent approach to the consideration of coastal inundation hazards in the Horizons Region, the methodology used in the Waikawa to Waitarere Coastal Hazards Assessment (Tonkin and Taylor, 2013), has been applied.

Coastal inundation at Himatangi Beach has been assessed for the three timeframes required as part of the brief. These scenarios being

- 2015 The existing situation;
- 2065 Possible inundation levels for the next 50 years; and
- 2115 Possible inundation levels for the next 100 years.

Design levels for each scenario were assessed for tidal conditions plus wave effects. The wave effects have been assessed assuming that the coastline can be considered as an open coast. This is a reasonable approximation as the maximum height of waves in the mouth of the Kaikokopu Stream will be limited by the depth of the inlet.

The equation below was used to model the 1% AEP still water plus 1% AEP wave effects.

Wave run-up (including wave set-up)

$$CIHZ = MHWS + S_s + \gamma + SLR + R_{2\%} \quad (5)$$

Where:

MHWS = Mean High Water Springs.

S_s = Storm surge.

R_{2%} = 2% wave run-up elevation (including wave set-up).

γ = IPO/ENSO/annual variation in MSL of 0.25 m.

SLR = Sea Level Rise.

5.2.1 Components

5.2.1.1 Mean High Water Springs

As discussed in Section 5.1.1 Mean High Water Springs at the Manawatu River mouth is 0.436 m (above Wellington Datum). However, as noted by (Blackwood, 2007), the tidal range decreases southwards along the coastline. Hence, the MHWS at Himatangi will be slightly higher than at the mouth of the Manawatu River.

To determine an appropriate level for MHWS at Himatangi Beach a linear interpolation was made between the levels at the Wanganui River Entrance and the Manawatu River. This produced an estimate of MHWS at Himatangi Beach of 0.561 m (above Wellington Datum).

5.2.1.2 Storm Surge

A storm surge component of 0.9 m has been chosen for Himatangi Beach. This estimate of the effects of barometric set-up and wind set-up was chosen as it is in

keeping with the assumptions made at Waikawa and Waitarere, which are considered to be appropriate for the beach under consideration in this assessment.

5.2.1.3 Wave Run-Up ($R_{2\%}$)

To estimate wave run-up elevation there are a number of empirical formulae that are commonly used by engineers. In recent assessments of wave run-up on the beaches of the west coast of the Horizons Region four such formulae have been applied. These formulae are of (Holman, 1986), (Ruggiero, Komar, McDougall, Marra, & Beach, 2001) as modified by (Tonkin and Taylor, 2002), (Mase, 1989) and (Hedges & Mase, 2004). These formulae are below:

Holman

$$R_{Max} = C \left(\frac{g}{2\pi} \right)^{0.5} \beta H_B T \quad (6)$$

Where:

- R_{Max} = Maximum wave run-up above the stillwater elevation (m).
- C = A coefficient that varies from 0.83 (rocky slope) to 1.5 (smooth slope), a value of 1.07 was found by (Tonkin and Taylor, 1999) to fit fine sandy beaches.
- β = Beach slope.
- H_B = Breaking wave height (m), based on a transformation of $H_{1\%}$.
- T = Wave period.

Ruggiero et al

$$R_{Max} = 0.27C(\beta H_B L)^{0.5} \quad (7)$$

Where the additional parameters are:

- C = A coefficient that was found by (Tonkin and Taylor, 2002) to be 0.9 for several eastern Bay of Plenty beaches; and
- L = Wave Length (m).

Mase

$$\frac{R_{2\%}}{H_0} = (1.86\xi_0)^{0.71} \quad (8)$$

Where:

- H_0 = Deep-water significant wave height.
- ξ_0 = Iribarren number.

Hedges and Mase

$$R_{2\%} = (0.34 + 1.49\xi_0)H_s \quad (9)$$

Where:

H_s = Deep-water significant wave height.

5.2.1.4 Components of Wave Run-Up at Himatangi Beach

As part of the storm surge and wave run-up design levels for Foxton Beach (Blackwood, 2007) consideration was given to the estimates of wave heights and periods. The following wave characteristics were defined for Foxton Beach:

H_s = 12 m.

$H_{1\%}$ = 18.36 m.

T_p = 14.5 s.

It is considered that these characteristics provide a good estimate of the 1% AEP storm at Himatangi Beach due to the geographical proximity to Foxton.

5.2.1.4.1 Beach Slope (β)

From the beach surveys, contained in Appendix B, an average beach slope of 0.034 m/m has been calculated and assumed to be representative of Himatangi Beach.

5.2.1.4.2 Iribarren Number (ξ_0)

The Iribarren number, or surf similarity parameter, is a dimensionless parameter that is defined as:

$$\xi_0 = \frac{\tan \alpha}{\sqrt{H_0/L_0}} \quad (10)$$

Where:

L_0 = deep-water wave length (m).

α = bed slope.

Using this formula, the Iribarren number at Himatangi Beach has been calculated to be 0.18.

5.2.1.5 Wave Run Up Estimates

Table 5.2, overleaf, summarises the wave run-up elevations calculated by each of the methods described above. The calculations can be seen in Appendix C.

Table 5.2 - Estimates of wave run-up

Method	Wave run-up (m)
Holman	10.38
Ruggiero et al	2.88
Mase	5.45
Hodges & Mase	7.24

The elevation of wave run-up calculated using the method described by Holman appears to be high, while the estimate from the Ruggiero Model is low when compared with the size of waves observed at Himatangi.

It is for this reason that the average run-up elevation calculated by the Mase and Hodges & Mase Models has been used to estimate the elevation of wave run-up at Himatangi Beach.

The 1% AEP storm wave run-up at Himatangi Beach is estimated to be 6.35 m.

5.2.1.6 Sea Level Variation (y)

Tonkin and Taylor (2013), noted that sea level variation occurs due to thermal expansion and contraction due to changes in sea surface temperatures and associated currents. This variation in sea level is related to climate cycles of varying time periods.

The Waikawa to Waitarere Coastal Hazards Assessment concluded that a sea level variation of +/- 0.25 m would apply to both the 50 and 100 year scenarios. This is also considered to be an appropriate sea level variation to be applied at Himatangi Beach for the 2065 and 2115 scenarios.

It is also considered appropriate to include this sea level variation in the assessment of the current inundation hazard to allow for La Niña effects.

5.2.1.7 Sea Level Rise

As defined in the project brief (see section 1.1), sea level rise is taken as 0.31 m for the 2065 scenario and 0.95 m for the 2115 scenario.

5.2.1.8 Inlet Wave Effects

Wave heights in an inlet are considered to be depth limited. As such, the waves that run-up the Kaikokopu Stream, towards the surf lifesaving club, are not likely to increase in size over the planning timeframe considered by this investigation. As such, the risk of coastal inundation to the surf lifesaving club is not considered likely to change.

5.3 Coastal Inundation Hazard Results

Table 5.3, below, summarises the components and final coastal inundation levels for the existing 2065 and 2115 scenarios.

Table 5.3 - Coastal Inundation Hazard Elevations

Parameter	Scenario		
	Existing	2065	2115
MHWS (m above Wellington Datum)	0.561	0.561	0.561
Sea level rise (m)	0	0.31	0.95
Storm surge (m)	0.9	0.9	0.9
Sea level variation (m)	0.25	0.25	0.25
Wave run-up (m)	6.35	6.35	6.35
Total CIHZ elevation (m above Wellington Datum)	8.06	8.37	9.01

5.4 Coastal Inundation Hazard Mapping

The Coastal Inundation Hazard Zones at Himatangi Beach have been mapped by plotting the inundation level on the surveyed ground level. This has given two spatial points for each inundation scenario.

A line has been plotted between these points and overmarked on the 2011 aerial map to indicate the CIHZ. These Coastal Inundation Hazard Zones can be seen in Figure 5.2.

Due to the lack of more extensive topographic information around Himatangi it is not possible to extend these lines further along the shoreline. However, a visual inspection of the dunes confirmed that the dunes are of a relatively constant size along the shoreline at Himatangi. This means, that it is reasonable to assume, that there will be no properties along Hunia Terrace that are likely to be at risk from coastal inundation during the 100 year planning timeframe considered by this investigation.

Figure 5.2 - Himatangi Beach Coastal Inundation Hazard Zones



6 Summary

The shoreline at Himatangi Beach has been assessed for coastal hazards. This has included a comprehensive assessment of the CEHZ (Coastal Erosion Hazard Zone) and the CIHZ (Coastal Inundation Hazard Zone).

CEHZs were assessed for the current (2015), 50 year (2065) and 100 year (2115) planning timeframes. Table 6.1, below, summarises the CEHZ setback distances as measured from the toe of the foredunes (as seen in 2011).

Table 6.1 - Summary of CEHZ setback distances

CEHZ Setback Distances		
2015	2065	2115
40.9 m	47.2 m	59.9 m

CIHZs were assessed for the same planning timeframes as the CEHZs. The 1% AEP elevations (above Wellington Datum) are summarised in Table 6.2 below.

Table 6.2 - Summary of CIHZ elevations

CIHZ Elevations (m above Wellington Datum)		
2015	2065	2115
8.06	8.37	9.01

7 Bibliography

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Appendix A. Historic Shoreline Change

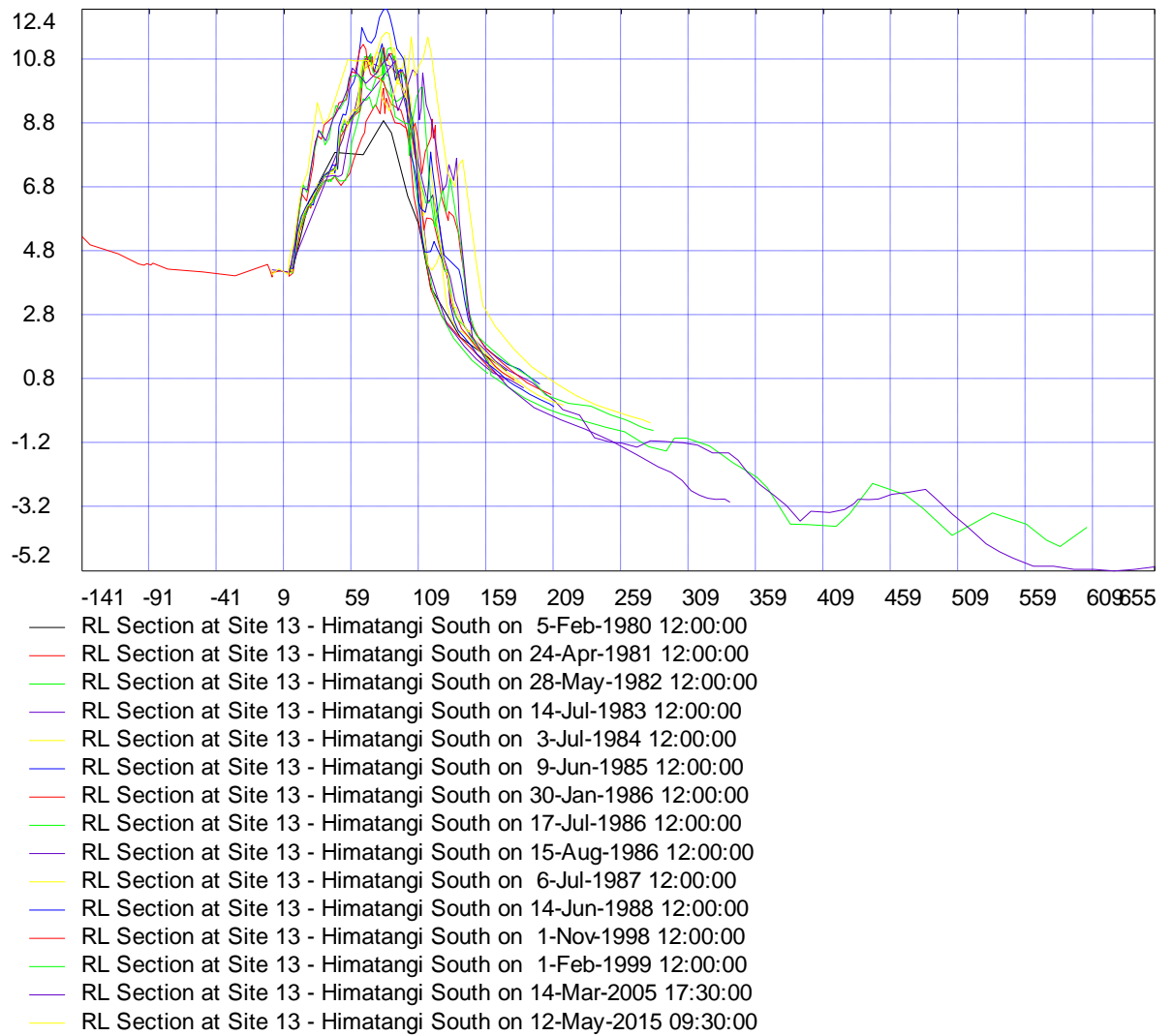


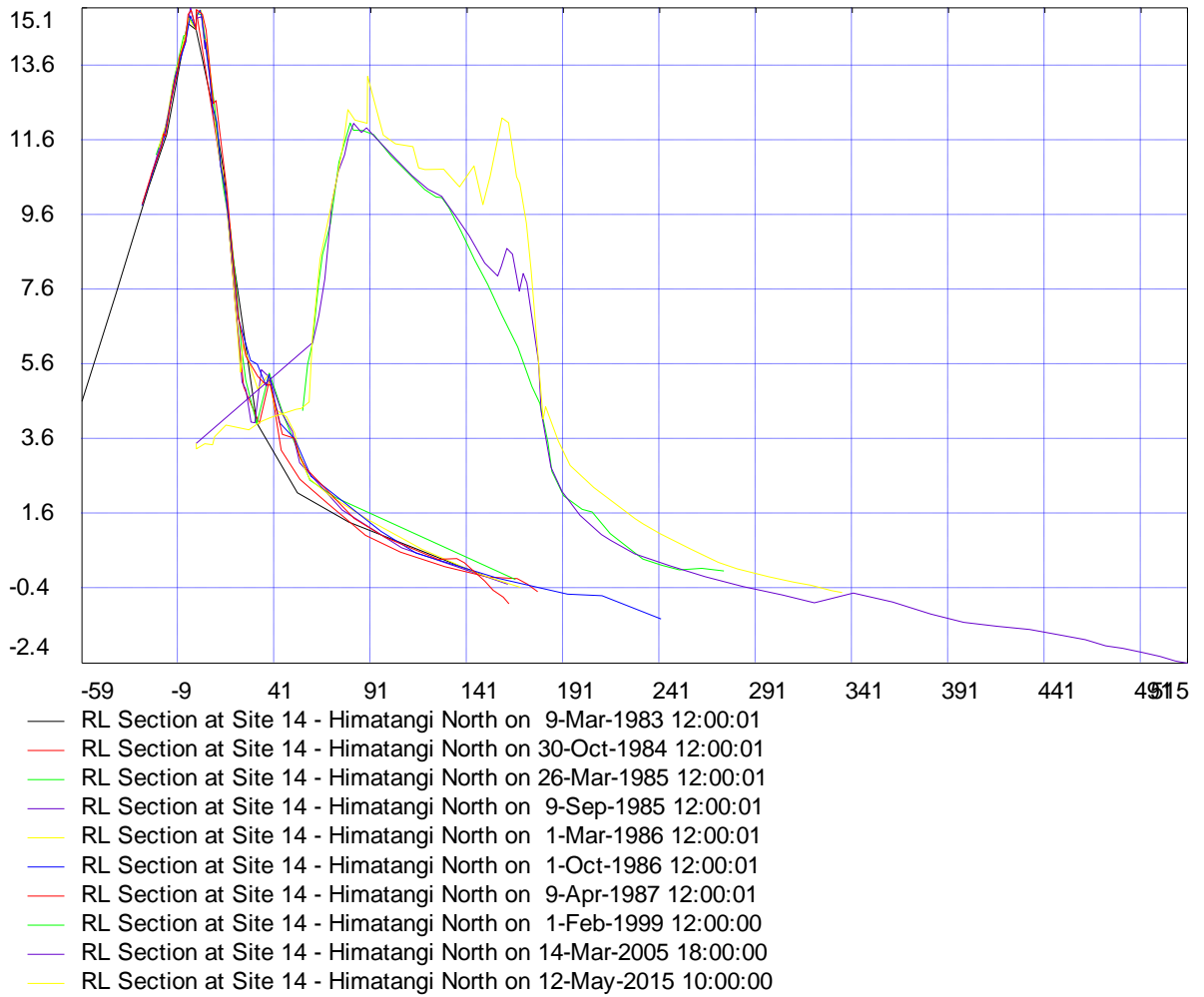
HIMATANGI BEACH
 Shoreline Position Change - Cross Sections

0 75 150 300 Meters

Cross Section	Quantum of seaward migration (m)							
	1963 - 1995		1995 - 2005		2005 - 2011		1963 - 2011	
	Total (m)	Average (m/yr)	Total (m)	Average (m/yr)	Total (m)	Average (m/yr)	Total (m)	Average (m/yr)
1	50.80	1.59	6.54	0.65	10.30	1.72	67.64	1.41
2	54.16	1.69	12.77	1.28	10.04	1.67	76.97	1.60
3	57.47	1.80	19.06	1.91	8.91	1.49	85.44	1.78
4	83.35	2.60	9.93	0.99	8.20	1.37	101.48	2.11
5	68.59	2.14	11.06	1.11	11.82	1.97	91.47	1.91
6	60.55	1.89	16.03	1.60	5.88	0.98	82.46	1.72
7	35.01	1.09	22.84	2.28	1.91	0.32	59.76	1.25
8	45.64	1.43	16.23	1.62	6.78	1.13	68.65	1.43
9	51.43	1.61	7.08	0.71	4.99	0.83	63.50	1.32
10	37.84	1.18	5.85	0.59	3.01	0.50	46.70	0.97
11	37.65	1.18	3.82	0.38	6.51	1.09	47.98	1.00
		1.65		1.19		1.19		1.50

Appendix B. Survey Data





Appendix C. Wave Run-Up Calculations

Himatangi Beach - Wave Run-up Calculations

Holman:

$$R_{max} = C \left(\frac{g}{2\pi} \right)^{0.5} \beta H_b T$$

$C = 1.07$ (for sandy beaches from Tanihira (2001))
 $\beta =$ beach slope = 0.05 m in 0.034
 $H_b =$ breaking wave height
 $T =$ wave period = $10s$

$$\therefore R_{max} = 1.07 \times \left(\frac{9.8}{2\pi} \right)^{0.5} \times 0.05 \times 10.36 \times 10.5$$

~~R_{max}~~

H_b - breaking wave height

from USACE shore protection manual

Assuming that unobstructed deep water height = deep water significant wave height = $12m$

i.e. $H_s = H_b = 12m$

$$\therefore \frac{H_b}{gT^2} = \frac{12}{9.807(10.5)^2} = 0.00582$$

\therefore From Figure 2-72

$$H_b/H_s = 1.05$$

$$\therefore H_b = 1.05 \times 12 = 12.6m$$

$$\therefore R_{max} = 1.07 \times \frac{9.8}{2\pi}^{0.5} \times 0.05 \times 10.36 \times 10.5$$

$$R_{max} = 15.28m \quad 10.38$$

Regions et al:

$$R_{max} = 0.27 C (\beta H_b L)^{0.5}$$

where the additional parameters are:

$C =$ a coefficient found by Tanihira (2001) to be 0.9 for several eastern Bay of Plenty beaches
 $L =$ wave length

$$L = \frac{gT^2}{2\pi} = 328m$$

$$\therefore R_{max} = 3.49m \quad 2.8m$$

Mogk

$$R_{max} = H_b (1.86 \beta)^{0.71}$$

$H_b = 12$

$$\beta = \text{beach number} = \frac{C_{surf}}{\sqrt{H_b L}}$$

$L =$ deep water wave length

$$\therefore \beta = 0.26 \quad 0.1708$$

$$\therefore R_{max} = 12 \times (1.86 \times 0.26)^{0.71}$$

$$R_{max} = 7.44m \quad 5.05$$

