REPORT

Horizons Regional Council

Coastal Hazard Report
Akitio and Herbertville

Report prepared for:

Horizons Regional Council

Report prepared by:

Tonkin & Taylor Ltd

Distribution:

Horizons Regional Council	**2 copies**
Tonkin & Taylor Ltd (FILE)	**1 copy**

October 2010

T&T Ref:61457

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Executive summary

Horizons Regional Council (HRC) and Tararua District Council (TDC) have commissioned Tonkin & Taylor (T&T) to undertake a coastal hazard assessment for Akitio and Herbertville in the Tararua District.

The Akitio coastal village is situated along a narrow strip (generally < 80 m) of low lying (generally < 3.4 m RL) coast at the base of predominant hills/cliffs. Significant features are the Akitio River mouth to the north and a rock platform towards the south.

The coast line can be delineated into 5 areas:

- Area 1 – river mouth
- Area 2 – river mouth to school
- Area 3 - school to boat club
- Area 4 - boat club and camp ground (salient)
- Area 5 south of salient

The predominant coastal feature of Herbertville is the Wainui River, with the majority of the Herbertville township situated on low lying river derived sediments. The Wainui River exists into a coastal lagoon that periodically exits out to sea. For periods of time the river is closed off from the sea and fills the inshore lagoon. This type of feature is called a 'hapua'.

Shoreline change analysis from 1944 to 2010 for Akitio has shown that the majority of the shoreline is in dynamic equilibrium, with no significant trend of erosion or accretion. Shoreline accretion is evident in Area 1 along the river bank. Shoreline erosion is also evident within Area 1 along the road adjacent to the river of -0.07 m/yr. Shoreline erosion of -0.2 m/yr is occurring within Area 4.

The Herbertville shoreline is has been accreting since 1944 at rates up to 0.9 m per year.

CEHZ setbacks have been determined for Akitio and Herbertville which take into account current long term shoreline trends, short term shoreline fluctuations and shoreline retreat associated with predicted sea level rise to 2060 (0.36 m) and 2110 (0.9 m and 1.5 m).

Along the Akitio shoreline, CEHZ setbacks to 2060 range from 34 m to 59 m. While CEHZ setbacks to 2110 (0.9 m) range from 59 m to 94 m and 2110 (1.5 m) range from 84 m to 119 m. Based on the CEHZ estimates, much of the low lying Akitio shoreline is likely to be at risk of coastal erosion by 2060. CEHZ setbacks for Herbertville are 40 m, 50 m, and 60 m for 2060, 2110 (0.9 m) and 2110 (1.5 m) respectively.

Coastal inundation including wave set up and run up have been estimated for Akitio using a number of water level and wave scenarios. The wave parameters for 1% and 2% AEP events have been calculated using wave hind cast data from a global wave model. Calculated inundation values compared well with measured values for a storm event in2008.

Inundation modelling showed that the low lying Akitio shoreline is at high risk of inundation in the next 50 years by to a 2% AEP storm event resulting in inundation (including wave set up) to 3.39 m RL.

Considering the high risk of both coastal erosion and inundation affecting the majority of low lying dwellings and infrastructure, Akitio requires significant engineered solutions to mitigate coastal hazards from 2060 and beyond.

Mitigation can be undertaken at relatively low cost to remediate localised, short to medium term coastal hazard issues. However, a long term engineered solution to provide protection of a 2% AEP wave and water level event to 2060 is likely to cost in excess of $5,000,000.

1 Introduction

Horizons Regional Council (HRC) and Tararua District Council (TDC) have commissioned Tonkin & Taylor (T&T) to undertake a coastal hazard assessment for Akitio and Herbertville in the Tararua District.

A recent review of the District Plan by the Tararua District Council has identified the need for an assessment of coastal hazards along parts of the Tararua District Coastline.

The purpose of the assessment is to identify the nature and scale of coastal hazards within the two identified areas. The assessment will enable the councils (HRC and TDC) to make informed decisions as to how to manage existing and new development in these areas.

1.1 Site locations

Akitio and Herbertville are located on the south east coast of the North Island, south of Cape Turnagain. Figure 1-1.

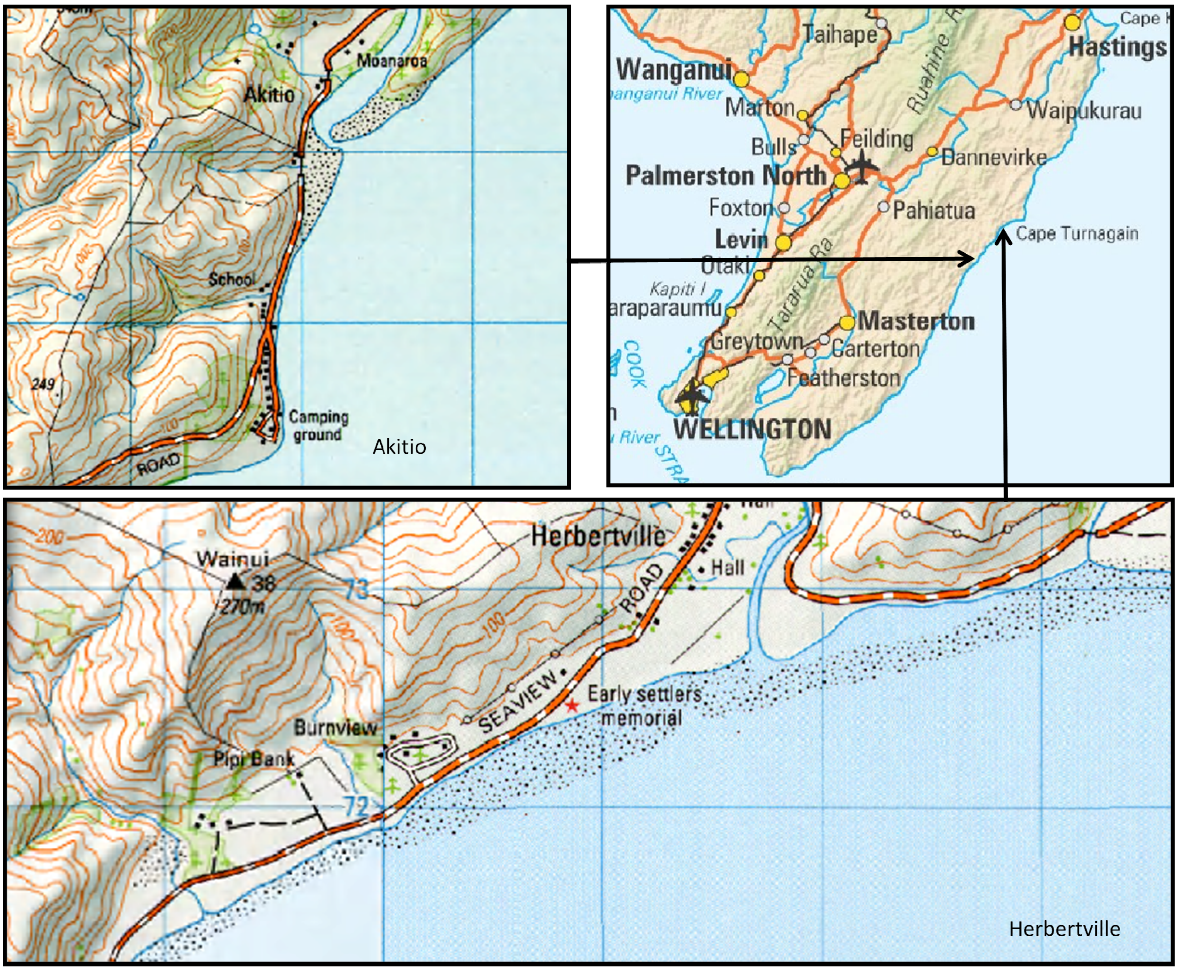


Figure 1-1 Site locations

1.2 Project scope

To undertake the coastal hazard assessment the following tasks are required:

Task 1 - Investigate and identify the nature and scale of coastal hazards that exist in relation to the areas as detailed in the project brief (i.e. Akitio and Herbertville) including:

- Coastal storm surge hazard assessment
- Wave run-up hazard assessment
- Coastal erosion hazard
- Coastal hazard mapping
- Coastal hazard accuracy assessment
- Integration of hazard zones into District Plan
- Identification of potential mitigation options.

Task 2 - Determine ASCH (Area Sensitive to Coastal Hazards) for Herbertville

- Using design water level from Akitio study assess inundation areas (requires onsite survey or LiDAR)
- Determine erosion hazard using available historical aerial images
- Plot ASCH based on most landward extent of both inundation and erosion hazards (not including tsunami hazard).

Assumptions:

- Some of the information of the Akitio assessment will be assumed to be also relevant for Herbertville (i.e. design water levels).
- ASCH (Area Sensitive to Coastal Hazards) is deemed to be the 100 year planning time frame for Coastal Hazards (i.e. one line representing the likely extent of coastal hazards to 2110)
- ASCH to be used as a 'Red Flag' for any proposed developments and requires more detailed assessment, if property seaward of the ASCH.

2 Background information

2.1 Akitio site description

The Akitio coastal village is situated along a narrow strip of low lying coast at the base of predominant hills/cliffs. Significant features are the Akitio river mouth to the north and an attached rock platform towards the south.

In between these two features the coast line can be delineated into 5 areas (Figure 2-1 Appendix A).

2.1.1 Area 1 – river mouth

The river mouth area to the north consists of a low lying sandy spit feature originating from a headland. The spit feature is likely to be inundated and overtopped regularly during large storm events. As such, the spit dimensions and location of the river mouth varies over time (Photo 1 Appendix B). Changes in river mouth location and spit dimensions are largely caused by wave events, rather than river flood events.

Rock protection runs along much of the road in this area. The rock protection is to mitigate the migrating river and from wave events that penetrate through the river mouth and focus on the shoreline (Photo 2 Appendix B).

The backshore vegetation is generally a combination of grass and marram grass.

2.1.2 Area 2 – river mouth to school

The geology of the coastal margin between the river mouth and the school is generally unconsolidated (historic) slip debris. The slip debris has been reformed by marine processes to form the current beach and backshore system. Combined with the slip material is sediment sourced from the Akitio River (Photos 3 and 4 Appendix B).

Based on the geology and topography of the area, the shoreline is likely to have been immediately at the base of the hills at some point in the past. The erosion effects of marine processes at the base of the hills are likely to have caused undercutting of the slope causing slumping of the hill side. The slip debris has then resulted in the shoreline migrating seaward to the current location.

The road reserve width (between the road the current erosion scarp) along the coastal road varies from approximately 17 m to less than 5 m.

A secondary (more recent) slip feature is evident just north of the school. The slip can be clearly seen in aerial photos (Area 2b in Figure 2-1 Appendix A). The slip has resulted in the shoreline protruding approximately 20 m further seaward (than the surrounding shoreline) in this area. The protruding shoreline is currently experiencing erosion. Rock protection has been placed along the seaward edge of the protruding shoreline area (date unknown), presumably to try and mitigate previous erosion events (Photo 5 Appendix B).

The backshore vegetation is generally a combination of grass and marram grass.

2.1.3 Area 3 - school to boat club

South of the slip feature, a low lying road reserve runs seaward of the road to the beach access near the boat club. The strip of reserve varies in width from approximately 17 m at the north (school) end and the south (boat club) to less than 5 m towards the middle of the area.

The reserve area near the school has experienced recent erosion putting the Norfolk pines at risk of toppling over. In an attempt to mitigate the erosion, tipped rock has been placed along the reserve (Photo 6 Appendix B).

Rock protection is also evident in front of the toilet block (Photo 7 Appendix C), although the recent erosion has not affected this area. From the toilet block south to the boat house there is evidence of erosion, which is reducing the width of the reserve.

The backshore vegetation is generally a combination of grass and marram grass.

2.1.4 Area 4 - boat club and camp ground (salient)

The sheltering effect of the rock platform has formed a low lying sandy area (salient). Within the area is the boat club building and camp ground (Photo 9 Appendix B).

Rock protection has also been placed in front of the boat club (Photo 9 Appendix B).

Marram grass is predominant along the low lying salient to the south (Photo 10 Appendix B).

2.1.5 Area 5 south of salient

The area along the southern facing shows some recent signs of erosion. However, considering the absence of any buildings or infrastructure, there is no current erosion risk.

2.2 Herbertville site description

The predominant coastal feature of Herbertville is the Wainui River, with the majority of the Herbertville township situated on low lying river derived sediments. To the north of Herbertville is Cape Turnagain. The Herbertville shoreline extends approximately 5km along the coast and varies in beach width from 300 m near the river mouth to < 50 m along the base of cliffs at the northern and southern ends.

The Wainui River exists into a coastal lagoon that periodically exits out to sea. For periods of time the river is closed off from the sea and fills the inshore lagoon. This type of feature is called a 'hapua'. The lagoon empties of water when the seaward edge is breached. The breach is likely to migrate along the coast (Photo 11 - Appendix B).

The backshore vegetation is generally a combination of grass and marram grass.

2.3 Shoreline changes

2.3.1 Historic shoreline change

Aerial photographs from six time periods (Table 2-1) were geo-referenced and analysed to determine the historic coastal erosion trends over 66 years. The seaward line of vegetation represents a common shoreline feature and was digitised for each photograph. This feature was chosen because it forms a sharp discontinuity in colour contrast on photographs. The shoreline features from both sites are plotted in Figures 2-2 and 2-3 in Appendix A. These trends of shoreline movement are derived from a limited set of photographs and shoreline changes are also possible between photograph dates due to cyclical (seasonal to decadal) climate change.

Table 2-1 Schedule of aerial photographs

Location	Date	Reference
Akitio and Herbertville	25/05/1944	SN286
Akitio and Herbertville	05/05/1967	SN1913
Akitio and Herbertville	30/11/1978	SN5310
Akitio only	21/11/1994	SN9382
Akitio and Herbertville	31/03/2005	SN50510
Akitio and Herbertville	11/05/2010	Project Acquirement

The aerial photograph analysis for Herbertville used contact prints (i.e. no ortho-rectification), which were geo-referenced and digitised using standard GIS techniques (ARCGIS software). The estimated relative accuracy between images for this methodology is 5 m (+/- 0.08 m/yr).

The aerial photograph analysis for Akitio was undertaken by New Zealand Aerial Mapping (NZAM), and included ortho-rectified images and digitising using 3D photogrammetry. The estimated relative accuracy between images for this methodology is 2 m (+/- 0.03 m/yr).

The digitised shoreline features were then analysed using a GIS program called Digital Shoreline Analysis System (DSAS). DSAS was used to calculate shoreline change statistics at 10 m intervals along both sites.

2.3.2 Akitio historic shoreline change

Analysis of shoreline changes at Akitio over the last 66 years suggests that there are four sections with distinct shoreline change trends (refer to Table 2-2 and Figure 2.2, Appendix A).

Section 1 is located along the northern section of the Akitio River bank behind the headland. The shoreline in Section 1 is building out at an average rate of 0.38 m/yr.

Section 2 is also located on the Akitio River bank, and is directly south of Area 1 behind the spit feature. The shoreline in Section 2 is eroding at an average rate of -0.07 m/yr.

Section 3 is the central section of the Akitio shoreline and is in dynamic equilibrium, with no apparent long term trend of erosion or accretion.

The southern shoreline identified as Section 4 is eroding at an average rate of -0.18 m/yr. Section 4 has a maximum erosion rate of 22 m over 66 years (-32 m/yr).

Table 2-2 Akitio shoreline analysis results

Section	Distance (m)	Status	Avg rate (m/yr)	R2	Max rate (m/yr)
1	700	Accreting	0.38	0.75	0.84
2	700	Eroding	-0.07	0.59	-0.18
3	800	Dynamic equilibrium	-0.02	0.08	-0.21
4	1000	Eroding	-0.18	0.34	-0.32

2.3.3 Herbertville historic shoreline change

The results show the majority of the Herbertville shoreline is building out (accreting). In our opinion the Herbertville coastline can be split into three sections with distinct long term trends (refer to Table 2-3 and Figure 2.3, Appendix A).

Section 1 is located on the eastern side of the river mouth and is building out at an average rate of 0.64 m/yr. Based on visual observations from aerial photographs, the river mouth migrates east for periods of time and the flow is directed along the Section 1 shoreline. This causes the shoreline position to fluctuate over time and is reflected in a lower regression coefficient (R2) to the other areas.

Section 2 is located directly south west of the river mouth and has the largest accretion rates of 0.91 m/yr on average. The maximum distance of shoreline movement in this area is approximately 114 m of accretion over 66 years (1.73 m/yr).

The southern section of shoreline is identified as Section 3 and is building out at a rate of 0.52 m/yr on average.

Table 2-3 Herbertville shoreline change

Section	Distance (m)	Status	Avg rate (m/yr)	R2	Max rate (m/yr)
1	400	Accreting	0.64	0.90	0.84
2	900	Accreting	0.91	0.94	1.73
3	700	Accreting	0.52	0.88	0.81

2.3.4 Short term shoreline fluctuations

Short term fluctuations in shoreline positions are caused by; storm events, changes in seasons (summer/winter), climate regimes such as El Nino/La Nina and longer term climate oscillations (Interdecadal Pacific Oscillation - IPO). The aerial images used in the historical analysis are 'snap shots' in time of the shoreline position. Monitoring of the shoreline at more frequent intervals over a period of time can detect shorter term fluctuations.

However, no long term beach profile dataset exists for Akitio or Herbertville. The closest long term beach profile dataset to the study areas is at Waimarama (Hawke's Bay Regional Council), some 90 km to the northeast.

The Waimarama beach profile dataset was used to estimate short term fluctuations. Analysis of the dataset produced a short term fluctuation of 30 m. The short term fluctuation was used in determining the CEHZ for many of the Hawke's Bay Beaches (T&T 2006).

The short term fluctuations are however unlikely to represent the short term fluctuations occurring at Akitio, due to different beach characteristic, but is considered appropriate for Herbertville. Therefore, as a proxy for short term fluctuation at Akitio, we consider the envelope of shoreline position is appropriate, until more information can be obtained (Figure 2.4 Appendix A).

2.4 Water levels

All elevations are referenced to Napier Vertical Datum 1962, which is based on water level information at Port of Napier and is considered to be an appropriate estimate of mean sea level for Akitio.

2.4.1 Predicted tides

The closest long term water level recorder is at the Port of Napier some 130 km north east of the site (see Table 2-4 for tide levels). Based on the NIWA tide forecaster, the tide levels at Akitio and Herbertville are within 0.03 m of those predicted for Napier. High tide at Akitio is predicted to occur approximately 10 minutes before high tide at Napier. Therefore, we consider the tidal information for Napier is appropriate for both Akitio and Herbertville.

Typically, the tidal range is 1.82 m and 1.06 m for spring and neap tides respectively. The mean high water springs level is 0.92 m RL and the maximum range is 2.02 m RL.

Table 2-4 Tidal levels for Port of Napier

Tidal Level	Level (m, CD[1])	RL (m, above MSL)
Highest Astronomical Tide (HAT)	1.96	1.02
Mean High Water Springs (MHWS)	1.86	0.92
Mean High Water Neaps (MHWN)	1.46	0.52
Mean Sea Level (MSL)	0.94	0.00
Mean Low Water Neaps (MLWN)	0.40	-0.54
Mean Low Water Springs (MLWS)	0.04	-0.90
Lowest Astronomical Tide (LAT)	-0.06	-1.00
[1]Chart Datum source: New Zealand Nautical Almanac (LINZ, 2010).		

2.4.2 Storm surge

A storm surge component for estimating coastal inundation was derived from a Monte Carlo simulation for Port of Napier (Worley 2002a). The simulation provided estimates of storm surge for 50 year (~2% AEP) and 100 year (1% AEP) events (Table 2-5). These values are added to the MWHS (0.92) value above to give indicative storm tide elevations.

Table 2-5 showing storm surge return periods based on a Monte Carlo simulations of Part of Napier water level data (sourced from Worley 2002a)

Return Period	Storm surge (m)
5 years	0.87
10 years	0.89
25 years	0.91
50 years	0.93
100 years	0.95

2.4.3 Future sea level rise

The Ministry of Environment (2008) guideline recommends a base value sea level rise of 0.5 m by 2100 (relative to the 1980-1999 average). Furthermore, the Ministry of Environment (2008) suggest assessing the potential consequences from a range of possible higher sea level rises, with, at the very least, consideration of the consequences of mean sea-level rise of at least 0.8 m and an additional sea level rise of 10 mm per year beyond 2100. Figure 2.5 shows the recommendations as set out by MfE.

Baseline sea level rise recommendations for different future timeframes

Timeframe	Base sea-level rise allowance (m relative to 1980–1999 average)	Also consider the consequences of sea-level rise of at least: (m relative to 1980–1999 average)
2030–2039	0.15	0.20
2040–2049	0.20	0.27
2050–2059	0.25	0.36
2060–2069	0.31	0.45
2070–2079	0.37	0.55
2080–2089	0.44	0.66
2090–2099	0.50	0.80
Beyond 2100	10 mm/year	

Figure 2.5 Extract from MfE 2008 showing baseline sea level rise recommendations for different future timeframes

Since the MfE guidelines, sea level rise estimates including better predictions of the effects of the polar ice caps melting have been produced (RSNZ 2010). The RSNZ paper cites sea level rise elevations over the next 100 years in the order of 1.5 m above current levels, due to potential melting of polar ice caps.

The planning timeframes used for this study are 2060 and 2110 (i.e. approximately 50 and 100 years from present). The resulting sea level rise components are 0.36 m and 0.9 m respectively from 1990 levels (taken as the mid range of 1980 to 1999). An additional 2110 level of 1.5 m was analysed to review the upper risk limits. Therefore, the rate of sea level rise from 1990 to 2060 and 2110 is calculated as 5.14 mm/yr to 2060 and either 7.5 mm/yr or 12.5 mm/yr to 2110. Based on the water level gauge at the Port of Auckland, New Zealand sea level has increased at an average rate of approximately 1.4 mm/yr between 1890 and 2000 (Hannah, 2004).

2.5 Waves

2.5.1 NWW3 ocean wave hind cast

NOAA WAVEWATCH III (NWW3) wave hind cast data from 1997 to 2008 near Castle Point (Lat -41.00, Long 176.25) was used to estimate the wave climate offshore of Akitio. The NWW3 node is located some 40km south of Akitio and is considered representative of the offshore wave climate due to the similar offshore bathymetry and alignment of the coastline (refer Figure 2.6).

The wave data was analysed into direction quadrants to enable inshore wave modelling using representative wave parameters Hsig (significant wave height) and peak period (Figure 2.7).

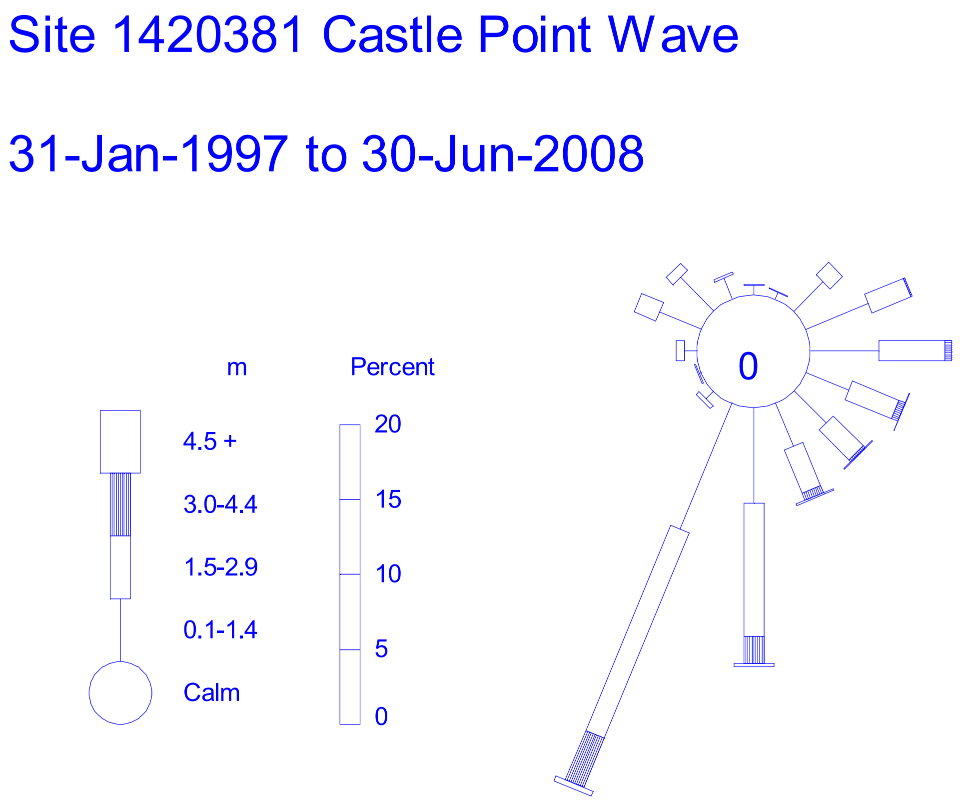


Figure 2.6 showing wave rose of NWW3 data

The NWW3 wave data was used as a basis for wave induced setup and run up elevations (refer section 4.5 and 4.5.4) for the coastal inundation hazard assessment. The combination of high wave heights and long periods are a dominating factor in wave induced setup and run up.

Analysis of the wave data shows that waves from the southern quadrant are both generally larger in height and also longer in period than the other quadrants.

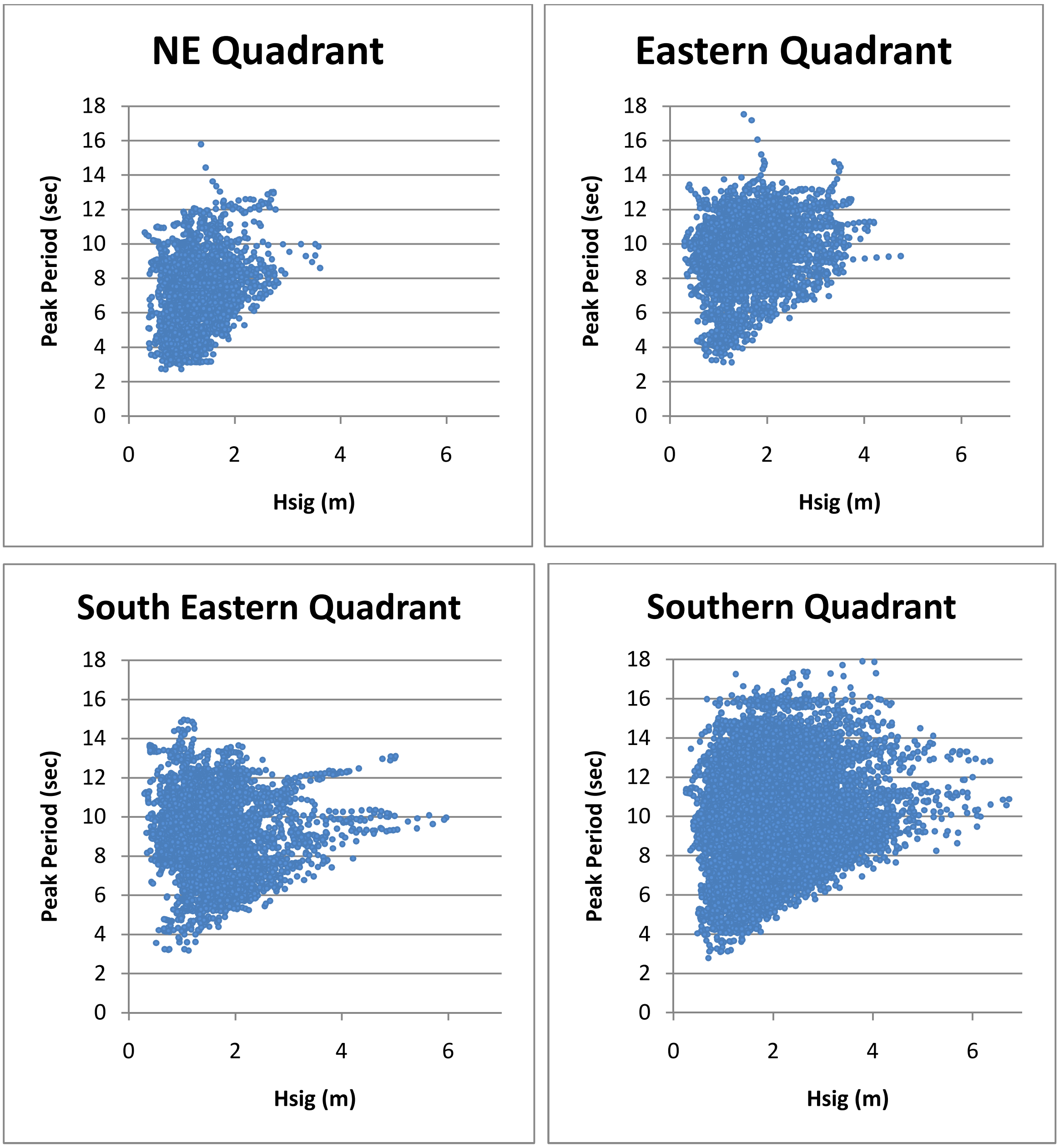


Figure2.7 showing maximum wave height Vs peak period for the 4 directions (NE, E, SE and E).

3 Coastal erosion hazard zone assessment

3.1 Methodology

The 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
- Short term fluctuations in shoreline movement
- Dune stability.

The coastal erosion hazard zones are based on the methodology outlined by Council in the RFT (Equation 1).

$$\boldsymbol{CEHZ = [SL \ + (LT)]T + ST + SE + D} \qquad \text{(Equation 1)}$$

Where:

CEHZ is the width of the coastal erosion hazard zone for sand shores.

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

LT = Historic long term rate of horizontal shoreline movement (m/yr). Future shoreline movement may differ from historic trends due to climatic patterns associated with Interdecadal Pacific Oscillation (IPO) and global climate change. To provide an appropriate precautionary approach, areas of inferred long term accretion as well as areas showing dynamic stability has the long term erosion component set to zero. This means that historic accretion is not extrapolated into the future.

T = Planning time frame (years).

ST = Horizontal distance of shoreline retreat from both storm induced erosion (m).

SE **=** Horizontal distance of shoreline retreat from 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).

Appropriate Factors of Safety or consideration of uncertainty will be incorporated into each individual component of the equation. Further descriptions of the coastal erosion hazard components are set out in the following sections.

The methodology in determining CEHZ's above applies only to areas where the back shore is consistent in geology and/or topography. In areas where the back shore geology and or topography are likely to alter erosion mechanisms and rates, the CEHZ may not apply. This will require site specific assessments, such as geotechnical investigations, to confirm sub-surface conditions.

3.2 Sea level rise effects (SL)

In our opinion, no adjustments for vertical land displacement (uplift from earthquakes) are appropriate to consider for sea level rise during the planning period of 2110.

Future sea level rise will permit waves to attack the backshore and fore dunes more frequently. Sandy open coasts that have been relatively stable over time are likely to show a bias towards erosion with rising sea levels, unless the supply of sand to the beaches can keep pace with erosion.

One approach 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 translation over time (Komar, 1998). The landward translation of the beach profile (**X**) can be defined as a function of SLR (**Δs**) and the active beach slope (**tanα**). This method of describing the equilibrium beach concept is a variation of the Bruun rule and is given in Equation 2 and displayed in Figure 3-1.

$$\boldsymbol{X} = \frac{\Delta s}{\tan \alpha} \qquad \text{(Equation 2)}$$

Where:

X is the landward translation of the beach profile due to sea level rise (m).

Δs = increase in sea level rise (m).

tanα = average slope of the embayment.

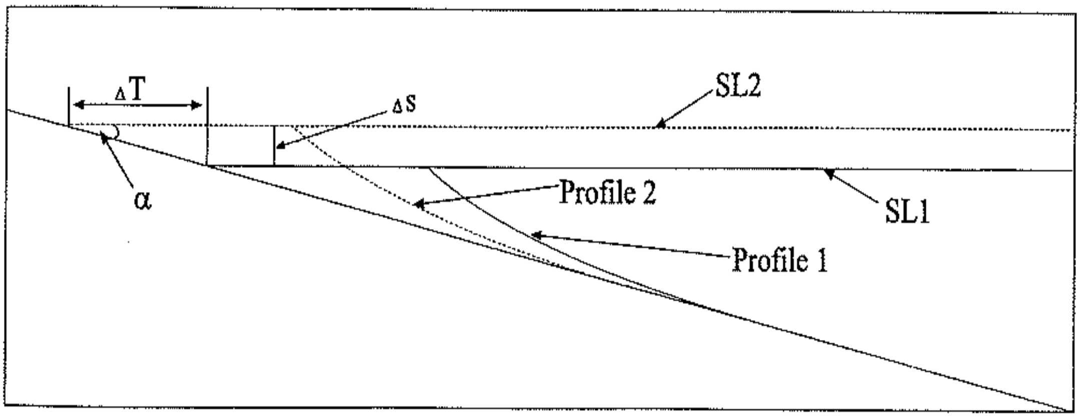


Figure 3-1 Horizontal translation distance of the beach profile under SLR (adopted from Hennecke and Cowell, 2000).

Assuming a predicted sea level rise of 0.36 m by 2060 and 0.9 m or 1.5 m by 2110, the landward movement of the beach profile are given for both Akitio and Herbertville in Table 3-1.

Table 3-1 Sea level rise summary

Planning Period	**Intertidal Slope (tanα)**	**SLR scenario (m)**	**SLR rate (mm/yr)**	**SL Distance (m)**
Akitio				
2060	0.02	0.36 m	5.14	13
2110A	0.02	0.9 m	7.5	38
2110B	0.02	1.5 m	12.5	63
Herbertville				
2060	0.05	0.36 m	5.14	5

Planning Period	Intertidal Slope (tanα)	SLR scenario (m)	SLR rate (mm/yr)	SL Distance (m)
2110A	0.05	0.9 m	7.5	15
2110B	0.05	1.5 m	12.5	25

3.3 Long term rates of shoreline movement (LT)

The long term retreat rate (LT) is an estimate of the average shoreline movement at the toe of the dune. The long term trends were based on linear regression analysis of the dune toe position captured from historical aerial photographs (refer to Section 2.3). The analysis provides a linear regression rate (LRR), which utilises all data points over the survey period and is more sensitive to cyclic trends than an end point rate (Dolan *et. al.,* 1991).

The Herbertville shoreline is experiencing long term accretion and was set a long term shoreline movement rate of zero. The positive shoreline movement trend was not used in this assessment because the historic rate of accretion is unlikely to keep ahead of future shoreline erosion due to predicted sea level rise.

The Akitio open coast shoreline (i.e. excluding the river bank section) can be divided into two sections with distinct shoreline movement trends. The majority of the shoreline to approximately 900 m south of the Akitio River mouth is in dynamic equilibrium, with no apparent long term trend of erosion or accretion (refer to Section 2.3.2). This section of shoreline was set a shoreline movement rate of zero. The southern section of shoreline is eroding at an average rate of -0.17 m/yr and was set a shoreline movement rate of -0.2 m/yr.

3.4 Short term shoreline movement (ST and SE)

The short term erosion rate takes account of both storm induced erosion (SE) and fluctuations around the observed long term trend of shoreline movement (ST).

Short term erosion may occur in response to severe wave storms moving toward the coast from the south west to north east quadrant. However, there are also short term fluctuations in shoreline position over a longer period than an individual storm event. These fluctuations are in response to natural variations in climatic conditions and sediment supply. For example, there may be variations in the direction and magnitude of shoreline movements associated with El Nino Southern Oscillation (ENSO), which typically occur within a three to seven year cycle.

Due to a lack of any beach profile data for both Akitio and Herbertville, short term shoreline movement was assessed from site investigation, historic aerial photographs and using estimates from other similar coastal areas.

DSAS software was used to calculate the maximum horizontal distance between the dune toe positions taken from historic aerial photographs at Akitio. This method calculates the maximum shoreline change envelope every 10 m along the entire shoreline. The Akitio shoreline was divided into four areas with distinct shoreline change envelopes ranging from 15 m in the north to 30 m in the south. Figure 2.4 Appendix A shows the four distinct areas and the associated values set for short term movement in this assessment.

The DSAS method could not be used at the Herbertville site because the shoreline has a consistent accretion trend and therefore does not display a dynamic shoreline change envelope. Beach profile data from Hawke's Bay Regional Council for the Waimamara area was analysed to represent short term shoreline changes at Herbertville. The average value for 3 standard deviations across the 18 Waimarama beach profiles is 30 m. This value was used to represent possible shoreline change from both short term variations and storm cut at Herbertville.

3.5 Dune stability (DS)

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 (32^{o}). The height of existing backshore was taken from LiDAR survey as RL 4 m at Akitio and RL 3.5 m at Herbertville. Based on these parameters the dune stability factor was taken as 6 m and 5 m for Akitio and Herbertville respectively. The dune stability factor was applied as a horizontal distance from the resulting short term erosion dune toe position.

3.6 Planning time frame

Three time frames were applied to provide a sufficient time scale for planning and accommodating development:

- Current Erosion Risk Zone (2010) - CERZ
- 2060 Erosion Risk Zone (50 years) - 2060ERZ
- 2110 Erosion Risk Zone (100 years) - 2110ERZ.

3.7 Factor of safety

As the LiDAR survey data and GIS techniques provide a high degree of accuracy in determining dune stability (D), we did not include a factor of safety in determining (D).

The long term shoreline change (LT) was based on delineation of the shoreline using 3d photogrammetric techniques, which are considered to have high accuracy. Therefore we did not include a factor of safety in determining (LT).

The short term fluctuation (SE) is considered to be a conservative value and therefore did not include a factor of safety in determining (SE).We also consider that sufficient conservatism is incorporated within the (SL) estimates to negate the need for a further Factor of Safety component.

3.8 River mouth zone

Flows from river catchments can have a significant effect on local beach profiles and fluctuation of the outlet position. Shorelines adjacent river mouths are dynamic features and can often experience a larger magnitude of shoreline change than the open coast. Therefore the coastal erosion risk zones do not apply to areas within 200 m of either the Akitio or Wainui River mouth.

3.9 CEHZ setback results

The CEHZ was delineated into 4 zones (Akitio 1 – 4) along the Akitio shoreline starting south of the river mouth. A summary of the CEHZ setback determinations are contained in Table 3.2 below and are shown in Figures 3.4 and 3.5 Appendix A. Note that the 1.5 m SLR to 2110 is not mapped, but the retreat distance (SL2110B) is included for comparison in the Table 3.2.

Table 3.2 showing setback parameters and resulting CEHZ setbacks for Akitio and Herbertville

Site	SL2060 (m)	SL2110A (m)	SL2110B (m)	LT (m/yr)	ST (m)	DS (m)	CERZ (m)	2060ERZ (m)	2110ERZ (m)
Akitio1	-13	-38	-63	0	-15	-6	-21	-34	-59
Akitio2	-13	-38	-63	-0.2	-25	-6	-31	-54	-89
Akitio3	-13	-38	-63	-0.2	-30	-6	-36	-59	-94
Akitio4	-13	-38	-63	-0.2	-25	-6	-31	-54	-89
Herbertville	-5	-15	-25	0	-30	-5	-35	-40	-50

3.10 CEHZ mapping

The foundation for the coastal erosion hazard zone mapping methodology is a series of shore normal (perpendicular) transects, with equal spacing along the coast. Transects were constructed from the baseline which follows the general shape of the coast (Figure 3-2).

The GIS methodology builds up the attribute information for each transect from the input data as described in previous sections. The attribute information is stored in tabular format and is linked spatially to each transect in GIS (Figure 3-2). When all necessary attribute information is captured, the coastal erosion hazard zone distances can be calculated based on Equations 1 and 2.

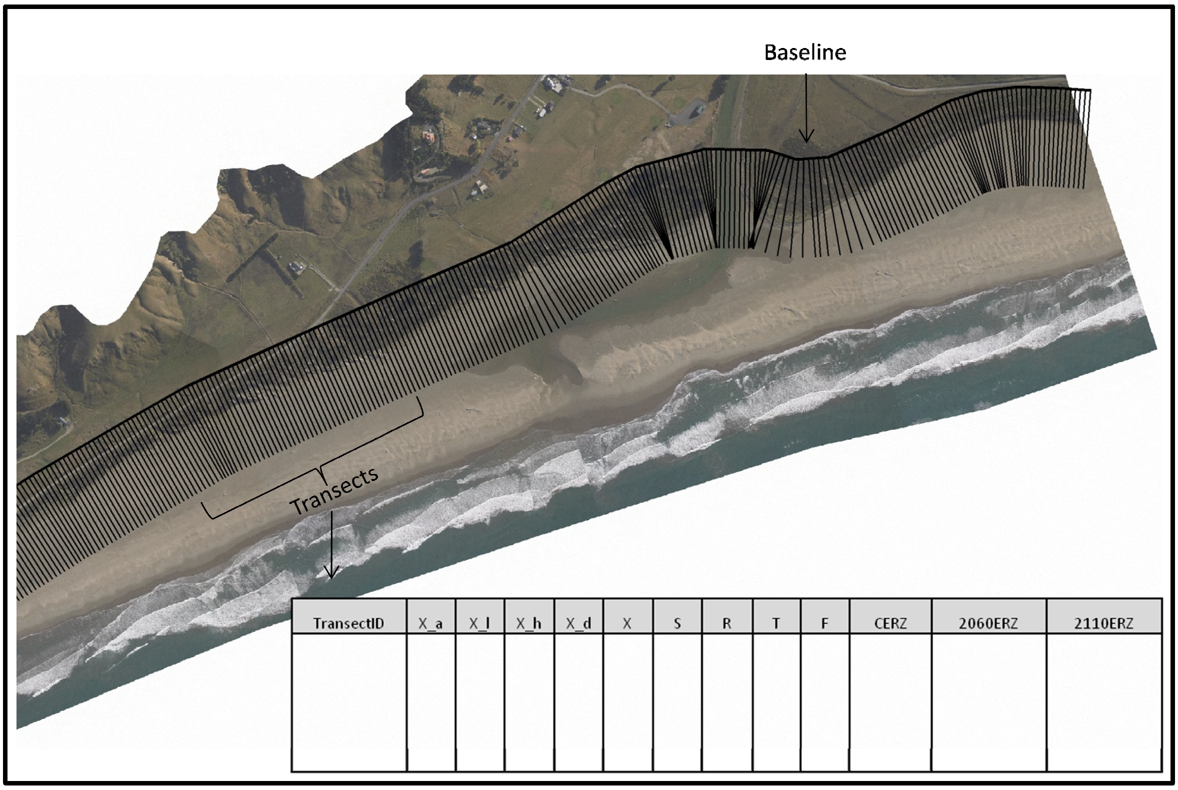


Figure 3-2 Conceptual diagram of the shore normal transects.

The CEHZ horizontal distances are measured inland from the dune toe along each transect. The two distances are transformed into XY coordinate points for each transect. The positions are then joined to form two shore parallel polylines, which delineate the coastal erosion hazard risk zones (Figure 3-3).

In areas where the erosion mechanism or rates are likely to be affected by backshore geology/topography, the CEHZ lines were identified as 'dashed' lines. The areas were typically where the back shore slope increased significantly (i.e. elevated land). The elevated areas are also likely to have different geology. Therefore, these areas may be at low risk of coastal erosion, but may also be prone to other hazards such as slip failure.

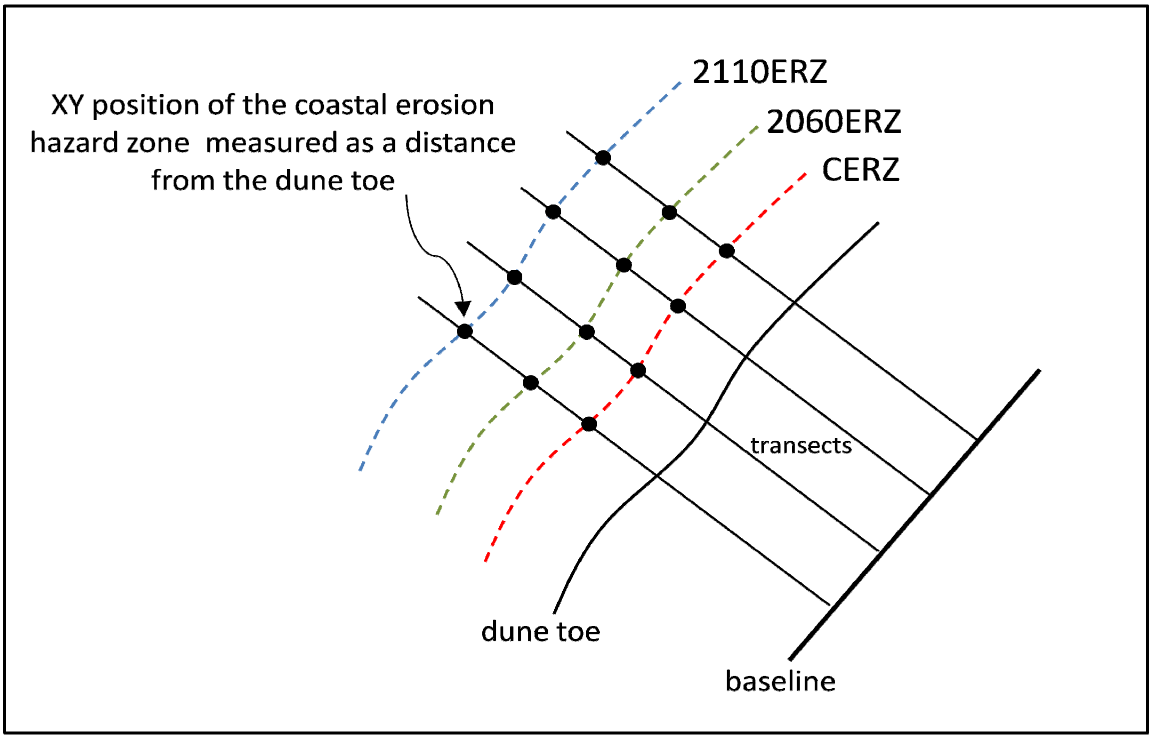


Figure 3-3 Conceptual diagram of the coastal erosion hazard risk zone delineation.

A summary of the GIS mapping methodology is provided in Table 3-3.

Table 3-3 GIS model methodology summary

Pre-processing	- Prepare model input data - Create transects
Processing	- Calculate coastal erosion hazard components based on model input data - Calculate the three coastal erosion hazard zones as a horizontal distance from the dune toe along each transect - Convert the horizontal distances into three sets of XY points along each transect based on trigonometry - Map the coastal erosion hazard zones by joining the three sets of XY points into polylines; CERZ, 2060ERZ and 2110ERZ.

3.10.1 CEHZ origin determination

The CEHZ offset origin is the most landward toe of dune taken from historic aerial photographs. For Akitio the existing dune toe is in the most landward position and was digitised for this study based on the NZAM ortho photo taken on 11 May 2010.

Herbertville is experiencing long term accretion with the most landward (measured) dune toe position in 1944. Generally, the latest shoreline is used as an origin for CEHZ determination, as per Akitio. However, the long term accretion along the Herbertville shoreline is likely to be attributed to the 'hapua' lagoon feature which has provided protection of the shoreline. Therefore, considering the ephemeral and migratory nature of the hapua and river mouth, we recommend using the most landward (1944) shoreline as the CEHZ origin for Herbertville. T account for possible future changes in shoreline protection of the hapua and/or migration of the river mouth.

The coastal erosion hazard zone is measured horizontally inland from the baseline feature at right angles from the general alignment of the shoreline.

3.10.2 CEHZ validation

The GIS model has been tested and validated during development in terms of ensuring the calculations are correct and the results verify what is occurring on the ground.

The model output (i.e. the two coastal erosion hazard risk zones) was validated at six locations:

- Akitio – Area 1 (E1888927 N5498608)
- Akitio – Area 2 (E1888886 N5497747)
- Akitio – Area 3 (E1888831 N5497528)
- Akitio – Area 4 (E1888492 N5497407)
- Herbertville – West (E1900831 N5510697)
- Herbertville – East (E1902250 N5511115)

The following validation checks were made at each of the six locations:

- The coastal erosion hazard risk zone distances are an accumulation of the correct coastal erosion hazard components (i.e. the correct equation has been applied).
- The coastal erosion hazard risk zones are plotted at the correct distance from the dune toe feature (i.e. the correct transformation from horizontal distance to XY position has been applied).

All six locations passed the two validation checks listed above.

As well as the validation checks above, checks were also undertaken to ensure the CEHZ lines appeared reasonable in context with the topography and geology. In areas where the underlying geology or topography does not fit with the methodology in determining the CEHZ's, the CEHZ was deemed to be 'indicative'.

4 Coastal flooding hazard zone assessment

4.1 Methodology

Coastal flooding hazard zones (CFHZ) were assessed for storm surge and wave events only (i.e. no tsunami or river flooding). For consistency with Horizons One Plan, inundation levels for current, 2060 and 2110 time frames are required for 1 % AEP (storm surge and wave height) events.

The 1% AEP (I) sea inundation level in terms of RL at the open coast was estimated via Equation (3) for the Akitio coast.

$$I = MHWS + S_s + R_{2\%} + \gamma + SLR \qquad \textit{Equation (3)}$$

Where:

MHWS = MHWS tide level based on tide data (Port of Napier).

S_s = storm surge (1% AEP).

$R_{2\%}$ = wave run-up elevation (including wave setup) using 1% AEP wave.

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

SLR = future predicted sea-level rise to 2060 (0.36 m) and 2110 (0.9 m) SLR for the 'current' time frame is therefore set at zero.

As well as the three inundation risk zones (Current, 2060 and 2110) above, other scenarios are included in the analysis to provide some relativity and sensitivity. Therefore in addition to the above the following scenarios were analysed:

$I = MHWS + S_s + R_{2\%}$

$I = MHWS + S_s + R_{2\%} + \gamma + SLR$ (1.5 m to 2110)

$I = MHWS + S_s$ (2% AEP) $+ R_{2\%}$ (2% AEP) $+ \gamma + SLR$ (0.36 to 2060)

Still water levels (Storm Tides - ST) were also estimated for the 6 scenarios using Equation 4, which included wave set up rather than wave run up. The storm tide provides a representation of the likely average water level at the shoreline.

$$ST = MHWS + S_s + \eta_{max} + \gamma + SLR \qquad \textit{Equation (4)}$$

Where:

η_{max} = maximum wave setup

The parameters required to determine the required AEP storm surge elevations and wave heights are discussed further in the following sections.

4.2 Sea levels

Sea level data was obtained from the nearest recording tide gauge at Napier, approximately 130 km northeast of the study area Mean High Water Springs (MHWS) was taken as 0.92 m RL (above mean seal level). Refer to Section 2.4 for water level information.

4.3 Storm surge

The storm surge component for estimating coastal inundation was derived from a Monte Carlo simulation for Port of Napier (Worley 2002a). The simulation provided estimates of storm surge for 50 year (~2% AEP) and 100 year (1% AEP) events (Table 2.5 - Section 2.4.2). These values are added to the MWHS value.

4.4 Sea level rise

Sea level rise estimates are consistent with the Ministry of Environment (2008) guidelines (refer Section 3.2) being:

- 0.36 m to 2060
- 0.90 m to 2110.

A sensitivity assessment of sea level rise of 1.5 m to 2110 has also been included, based on RSNZ 2010.

4.5 SWAN near shore wave modelling

Wave modelling has been carried out using SWAN to provide a typical nearshore wave climate at Akitio for a basis of understanding shoreline change and to provide input conditions into coastal inundation empirical calculations.

SWAN is a third-generation wave model that computes random, short-crested wind-generated waves in coastal regions. The SWAN model accommodates the following coastal process:

- Wind generation
- White capping
- Bottom friction
- Quadruplet wave-wave interaction
- Triad wave-wave interactions
- Depth induced breaking.

The SWAN model extended approximately 2 km m offshore and included fine recti-linear grid at a 20 m resolution). The model bathymetry was derived by combining river survey, LiDAR survey and bathymetric survey.

4.5.1 Ascertaining likely wave characteristics inducing greatest run up elevations

Initial SWAN model runs were used to ascertain the likely wave characteristics that would induce the greatest wave set up and run up along the Akitio coast. Based on the NWW3 data for the 4 directions (NE, E, SE and S) the scenarios in Table 4.2 were used. The wave height along the Akitio coast was used to ascertain wave run up elevations. A still water level of 0.92 m RL was used, which is the MHWS elevation.

Table 4.2 showing initial SWAN model run scenarios

SE quadrant		E quadrant		NE quadrant		S quadrant	
Wave (m)	**Period (s)**	**Wave (m)**	**Period (s)**	**Wave (m)**	**Period (s)**	**Wave (m)**	**Period (s)**
1	15	1.5	17.5	1.3	16	4	18
5	13	3.5	14.5	2.7	13	6.4	12.8
6	10	4.75	9	3.5	9.8	6.7	10.8

Analysis of the resulting run up elevations (using methodology in Section 4.5.4) showed that the scenario of waves with 4.0 m and 18 sec period from the southern quadrant generally resulted in the greatest run up along the coast.

Therefore, based on the run up calculations derived from the initial SWAN model runs we consider that wave data from the southern quadrant only are used to derive further wave statistics for coastal hazard inundation.

While the Initial SWAN modelling showed that longer period waves from the southern quadrant produced the greatest run up, a maximum wave height from the quadrant within the period range for each year is required to determine a 1% AEP wave height. The tables below (4.3 and 4.4) show a breakdown of wave heights and occurrences for wave periods from the southern quadrant (NWW3 data).

Although the 16 – 18 sec period does produce some occurrences of wave events, we consider that the frequency of these events is very low. Therefore we consider that wave events producing wave periods in the 14 – 16 sec range are more likely to occur during high tide events. The yearly maximum wave heights within this period range were then used for a Gumbel distribution to determine 100 and 50 year return period wave heights.

Table 4.3 showing maximum wave height from the Southern quadrant (NWW3) binned by peak period

Year	0-10s	10 to 12s	12 to 14s	14 to 16s	16 to 18s
1997	5.71	4.27	5.20	5.21	0.00
1998	4.27	4.73	4.25	4.37	1.40
1999	4.93	5.90	4.89	4.50	3.59
2000	4.28	5.60	4.61	3.51	1.39
2001	6.16	6.09	5.15	3.99	0.00
2002	5.22	6.00	4.55	3.45	0.00
2003	5.00	4.06	4.48	4.37	1.79
2004	5.38	6.73	3.72	3.73	4.06
2005	4.36	4.49	3.48	2.89	0.00

2006	4.71	5.90	4.73	3.42	3.41
2007	4.40	4.34	6.35	3.71	2.69

Table 4.4 showing total number of days per year when waves with within the peak period range occur

Year	0-10s	10 to 12s	12 to 14	14 to 16	16 to 18
1997	70.5	30.9	22.9	8.8	0.0
1998	68.0	42.5	19.3	8.0	0.3
1999	80.5	36.1	17.3	11.4	1.1
2000	82.0	36.9	17.3	6.3	0.3
2001	79.3	41.9	19.9	5.3	0.0
2002	77.4	57.0	19.0	2.9	0.0
2003	60.3	40.9	25.6	6.6	0.1
2004	78.4	48.0	24.9	4.5	0.9
2005	76.8	43.6	18.5	3.3	0.0
2006	80.8	29.0	23.3	8.6	1.9
2007	82.4	45.8	34.5	10.4	0.5

4.5.2 Modelling of 100 year return period wave event

Using the yearly maximum wave heights derived in the previous section a 1% AEP maximum wave height of **5.6 m** was determined using a Gumbel distribution (Figure 4.1).

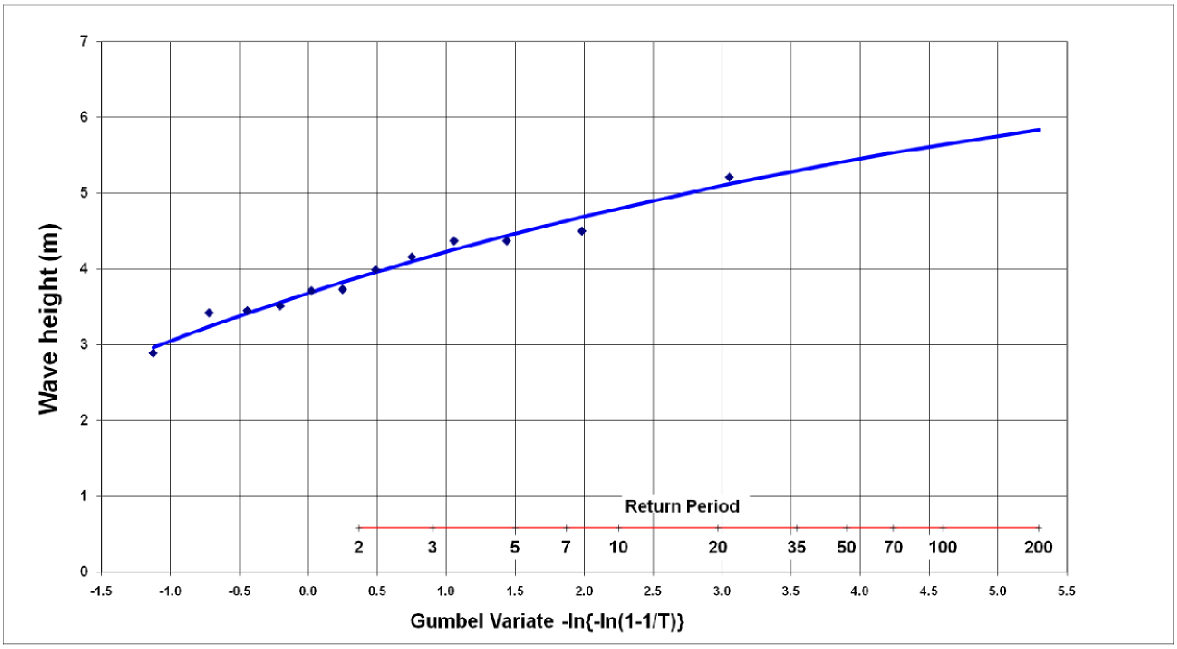


Figure 4.1 showing Gumbel distribution of maximum wave heights from the southern direction within 14 to 16 sec period.

The 1% AEP period wave (5.6 m) was modelled with a 16 sec period wave from the south (180^{o}) with the following still water level scenarios (Section 4.1):

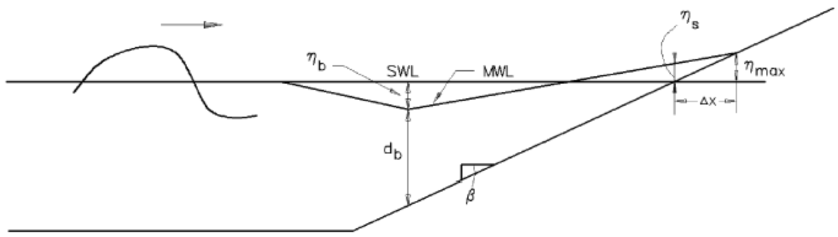
- 0.9 m RL - MHWS
- 1.85 m RL - MHWS + 0.95 m 100 yr return period storm surge
- 2.21m RL - MHWS + 0.95 m 100 yr return period storm surge + 0.36 m SLR to 2060
- 2.75 m RL - MHWS + 0.95 m 100 yr return period storm surge + 0.9 m SLR to 2110
- 2.75 m RL - MHWS + 0.95 m 100 yr return period storm surge + 1.5 m SLR to 2110

Also modelled was a 2% AEP (~50 y Return Period) wave (5.4 m) and water level (MHWS +0.93) with 0.36 m SLR to 2060.

The wave heights from the models were used to calculate set up and run up along the Akitio coastline.

4.5.3 Wave setup

The variation of the still water level (SWL) due to wave setup was evaluated using the offshore wave conditions derived from the global hindcast model (NOAA WAM) (refer to Section 2.5). The maximum wave setup was calculated using the methodology detailed in Chapter II-4-3 of the Coastal Engineering Manual. Relevant parameters describing the derivation of wave setup are shown below.



Where:

η_b = wave set down (m)

η_s = wave setup at the still water level (m)

η_{max} = maximum wave setup (m)

d_b = depth of wave breaking (m)

SWL = still water level (m)

MWL = mean water level (m)

β = beach slope from the break point to the upper beach

Composite beach slopes from the break point (~-3m RL) at MHWS to the upper beach for each shoreline section were evaluated from both the LiDAR and bathymetric survey undertaken for this study. Wave heights used for the set up calculations were obtained from point 150 m offshore of the MHWS elevation along the Akitio coast.

From the resulting setup elevations the largest set up value used for the entire Akitio shoreline. The modelling shows variation in set up elevations along the shoreline, however in reality such localised water level gradients are unlikely. Therefore, the maximum water level for each scenario is recommended to be used when estimating super elevated still water levels.

Typical inundation elevations including set up for each wave/water level scenario are shown in Table 4.5 below and Figure 4.2 Appendix A.

Table 4.5 showing water elevation scenarios (RL m) including wave set up for the various section along Akitio shoreline. Highlighted values are recommended still water elevations along the entire Akitio shoreline.

Area	1% AEP Wave MHWS		2 % AEP wave + 2%AEP water level + SLR2060 (0.36 m)		1% AEP wave + 1%AEP water level		1% AEP wave + 1%AEP water level + SLR2060 (0.36 m)		1% AEP wave + 1%AEP water level + SLR2110 (0.9 m)		1% AEP wave + 1%AEP water level + SLR2110 (1.5)	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
1	1.54	1.68	2.91	3.05	2.93	3.08	3.35	3.50	3.97	4.12	4.65	4.80
2	1.57	1.71	2.95	3.07	2.98	3.10	3.40	3.51	4.02	4.12	4.71	4.80
3	1.53	1.71	2.88	3.04	2.91	3.07	3.32	3.48	3.94	4.08	4.62	4.77
4	1.30	1.54	2.70	2.89	2.73	2.91	3.15	3.32	3.79	3.95	4.48	4.65
5	1.56	2.00	2.93	3.39	2.95	3.42	3.37	3.84	3.99	4.46	4.69	5.15

4.5.4 Wave run-up

Estimates of wave run up was undertaken using, the method of Nielsen and Hanslow (1991), outlined in Equations 5 to 7 below:

$$R_{L\,2\%exceedance} = SWL + 1.89 \times L_{zwm}$$ *Equation (5)*

Where:

$$L_{zwm} = 0.6(H_{orms} \times L_o)^{0.5} \tan\beta.$$ where $\tan\beta \geq 0.10$, and *Equation (6)*

$$L_{zwm} = 0.05(H_{orms} \times L_o)^{0.5}$$ where $\tan\beta < 0.10$ *Equation (7)*

And:

H_{orms} = root mean squared deep water wave height

L_o = deepwater wave length

β = beach slope

Note that the 2% exceedance refers to the run up distribution and not the input wave AEP.

Composite beach slopes from the toe of the intertidal beach (~-0.9 m RL) to the upper beach for each shoreline section were evaluated from both the LiDAR and bathymetric survey undertaken for this study. Wave heights used for the set up calculations were also obtained from point 150 m offshore of the MHWS elevation along the Akitio coast.

The resulting run up elevations were then projected to land and intersected with the LiDAR derived DEM. The methodology enables varying set up elevations (due to along shore changes in wave height and slope) along the shoreline to be accurately delineated landward. Typical inundation elevation including run up for each section are shown in Table 4.6 below and Figure 4.3 Appendix A. Figure 4.4 Appendix A shows the three risk zones for current, 2060 and 2110 time frames.

Table 4.6 showing water elevation scenarios (RL m) including wave run up for the various sections along Akitio shoreline.

Section	1% AEP Wave MHWS		2 % AEP wave + 2%AEP water level + SLR2060 (0.36 m)		Current: 1% AEP wave + 1%AEP water level		2060: 1% AEP wave + 1%AEP water level + SLR2060 (0.36 m)		2110: 1% AEP wave + 1%AEP water level + SLR2110 (0.9 m)		1% AEP wave + 1%AEP water level + SLR2110 (1.5 m)	
	Min (m)	Max (m)	Min (m)	Max (m)	Min (m)	Max (m)	Min (m)	Max (m)	Min (m)	Max (m)	Min (m)	Max (m)
1	2.88	3.08	4.42	4.60	4.45	4.63	4.91	5.08	5.58	5.74	6.32	6.47
2	2.93	3.13	4.48	4.62	4.51	4.65	4.97	5.10	5.64	5.77	6.38	6.50
3	2.83	3.12	4.35	4.57	4.38	4.60	4.83	5.04	5.50	5.69	6.24	6.44
4	2.42	2.87	4.08	4.37	4.12	4.40	4.60	4.86	5.32	5.55	6.09	6.33
5	2.90	3.56	4.44	5.06	4.47	5.09	4.93	5.54	5.61	6.20	6.36	6.92

4.6 Freeboard

A freeboard of 0.2 m is added to the final inundation levels (i.e. in addition to the elevations in Table 4.6) for both 2060 and 2110 planning periods. The freeboard is to account for accuracy of mapped inundation zones.

4.7 CFHZ validation

To validate the above inundation results, post storm inundation elevations were compared with estimated inundation levels. Inundation occurred along areas of the Akitio shoreline during a storm event in February 2008. Landward extents (debris or 'wrack' lines) were surveyed a few days after the storm event. Water level data from Port of Napier and wave data from Nww3 were used as input for the inundation calculations.

The same methodologies as the previous sections were used to estimate wave induced inundation (including wave run up). Table 4.7 below shows the comparison of inundation levels for each of the 5 surveyed locations.

Table 4.7 showing comparison of surveyed and estimated inundations levels for February 2008 storm event.

Location	Surveyed Inundation level (RL m)	Estimated inundation (wave run up and set up) level (RL m)	Difference (estimated–surveyed) (m)
Peg 1(Area 4)	2.50	2.35	-0.15
Peg 2 (Area 4)	2.60	2.40	-0.20
Peg 5 (Area 3)	2.55	2.78	0.23
Peg 6 (Area 3)	2.81	2.82	0.01
Peg 5 (Area 3)	3.55	2.82	-0.73

The estimated inundation levels generally compared favourably with the surveyed elevations, matching or slightly under predicting surveyed levels. At Peg 5 estimated run-up exceeded the surveyed inundation level. While there is likely to be some variability associated with the comparison, we consider the inundation methodology used to estimate inundation levels produces realistic results.

5 Herbertville ASCH

To determine a broad base Area Sensitive to Coastal Hazards (ASCH) for Herbertville the most landward extent of erosion to 2110 (Section 3) and inundation to 2110 was used. A first cut indicative inundation elevation of 4.46 m is used based on a 1% AEP wave setup and 1% AEP water level with 0.9 m SLR to 2110 as determined for Akitio.

Inundation with wave set up was used rather than wave run up due to likely significant difference in run up elevations between Akitio and Herbertville due to near shore and back shore topography .

Figure 4.5 shows the CEHZ and inundation zone as well as the other inundation (Set up only) scenarios used for Akitio.

Figure 4.6 shows the resulting ASCH for Herbertville.

6 Akitio coastal hazard mitigation

The following sections identify the various coastal hazards and possible mitigation options along the Akitio shoreline. The mitigation options are to combat both current and future coastal inundation and erosion hazard issues. We note that these options are land protection, rather than beach protection solutions, and any physical land protection works are likely to create a reduced beach amenity over time. The options are to be used as guidance for the community and councils as to the most appropriate mitigation.

Considering the various options below, further consultation with the community and council is recommended to identify the current and long term planning of Akitio.

The hazards and mitigation options are broken down into issues that what we consider require attention for the following time frames:

- Current – localised hazard issues that require attention sooner rather than later i.e. to combat current hazard risks
- Medium term - mitigation of hazards that will address risks for the next 5 to 10 years for large areas of the Akitio Shoreline
- Long term (2060-2110) - mitigation of hazards along the entire Akitio shoreline, based on guidance within the relevant statutory documents (Building Code, District Plan, Regional Plan and Resource Management Act).

All costs quoted are 'Rough Order Costs' which are likely to be +/- 30%. For costing purposes the rock supply is based on a commercial rate of $\$160/m^3$ (placed). Cost estimates for the chosen option(s) would be refined through the detailed design stage.

The construction cost estimates do not include professional services. Professional services costs for detailed design is estimated at approximately 10% - 20% of the physical works cost and is likely to include the following items:

- Detailed Design
- Detailed construction drawings
- Preparation of tender documents
- Construction supervision.

The professional services cost estimate for resource consent requirements is also estimated at approximately 10% of the physical works and is likely to include the following items:

- Assessment of Environmental Effects
- Ecological assessment
- Resource consent application

Does not include any consultation, hearing or possible Environment Court costs.

The cost estimate for professional services would be refined after the detailed design stage.

6.1 Current hazard mitigation

The hazard issues that currently need attention include the various rock protection works and areas where the road may be at risk from coastal erosion in the near future. The mitigations proposed are generally temporary /short term options to remediate current erosion issues.

At some point in the future, longer term time frame mitigation options are likely to be required (refer Sections 5.2 and 5.3).

Note that we are unsure of the legality (i.e. are the protection works consented) of any of the current rock erosion protection works so any remedial work may require a consent application to both regional and district councils. If the existing erosion protection works are not consented, we recommend a comprehensive consent for the entire Akitio shoreline, once the appropriate options have been decided. The comprehensive approach is preferred as opposed to individual consents for each area of works.

6.1.1 River bank/road rock protection (Area 1)

The current course of the Akitio River and location of the river is causing episodic erosion along the river bank margins. Erosion protection in the form of rock riprap is currently mitigating the erosion along portions of the road/river bank. While the rock protection is functioning, some parts of the rock rip rap require maintenance. There are also unprotected areas near the river mouth that may put the road at risk in the future.

The river mouth area is dynamic and is only causing risk to the road due to the current location of the river mouth. The location of the river mouth is allowing wave energy to penetrate through to the road/river bank during storm events.

Historically the river mouth has migrated north and south between the northern headland and the approximate current position. Should the river mouth migrate back north sometime in the future, the hazard risk to the road will be significantly reduced.

The area north of the current rock protection has exhibited significant shoreline/river bank accretion. The accretion is likely due to the change in river mouth location near the northern headland to the current southern location.

6.1.1.1 Option 1 - Do nothing

The current episodic erosion is due primarily by the location of the river mouth and waves penetrating into the area during wave events over the high tide period. These are infrequent events, but can have significant effects on the rock protection and may put the road at further risk.

- Pros
  - Low/no cost
  - Possible natural migration of river mouth north
- Cons
  - Possible damage to road during significant, but infrequent wave events
  - Periodic maintenance may increase over time

6.1.1.2 Option 2 New rock protection

To mitigate possible future erosion, rock protection of the currently unprotected shoreline could be undertaken. The crest level is recommended to be at or near current top of bank elevation. Therefore, the rock protection will ensure the integrity of the road for the immediate future, but not taking into account predicted SLR to either 2060 or 2110.

The crest is also likely to be over topped during large wave events coinciding with high (spring) tides.

Construction costs to undertake this work are likely to be in the order of $750 – $1,500 per linear meter and likely require some 350 m of works. Therefore costs likely to be in the order of $260,000 to $525,000. Costs fluctuations are due to varying crest elevations and required volume of rock required.

- Pros
  - Added confidence that risk to road significantly reduced for the short to medium term (10-20 years)
  - Possible natural migration of river mouth north
- Cons
  - Likely short to medium term life of structure
  - Overtopping is likely in some areas
  - Structure may not be required if river migrates north
  - Ongoing periodic maintenance
  - Cost

6.1.1.3 Option 3 - Realign the Akitio River

Periodic dredging of the Akitio River and breaching of the spit to locate the river mouth further north is likely to significantly reduce the effects of the river and river mouth on the at risk road/river bank. The alignment change would involve an excavator to dig a channel through the spit at low tide to create a new river mouth. Over the subsequent flood at ebb tides the river flow is likely to assist in keeping the channel open.

Works to be undertaken during spring tides to allow maximum tidal flushing through the new channel. A spring low tide will also allow access to the spit from the south, through the existing river. As the effects of the spit breach are likely to be minimal and the costs also minimal, a trial could be undertaken to assess the viability.

- Pros
  - No requirement for rock protections along road/river bank
  - No permanent engineering structures
  - Mimics the natural river mouth migration process
  - Works may not require consenting (maintenance works)
  - Relatively low cost (excavator hire age for a day)
  - Minimal risk, if new channel does not remain open then will return to status quo.
- Cons
  - Likely retreat of the (currently accreting) river bank adjacent to new river mouth
  - Periodic maintenance dredging to keep river mouth and river alignment away from road
  - New river mouth may close of after a short time (perhaps days) and revert back to the southern location

6.1.2 Slip north of school (Area 2b)

The erosion occurring along the slip shoreline in Area 2b is exacerbated by the existing failed rock protection measures. The ‘ad hoc ‘nature of the rock protection with no filter material behind rock armour units is allowing the fines to be dispersed. The rock protection is also causing end effects north and south of the area. The erosion seems worse along the slip due to the shoreline protruding further seaward than the adjacent shorelines.

Note that no significant long term shoreline erosion evident from the historical shoreline assessment (Section 2.3.2) in this area due to the 'hard line' of rock protection.

6.1.2.1 Option 1 Do nothing

Considering the current erosion scarp is some 2 m from the road, the do nothing option is likely to result in some loss of the road in the very near future. Therefore we do not consider this as a viable option.

6.1.2.2 Option 2 – Extension and remedial works of existing failed rock protection

Approximately 150 m of rock protection will be required to mitigate the erosion which includes tie backs into the stream south of the slip areas and north to reduce end effects. Based on an estimated linear rate of $1,500 per meter for construction the estimated cost is $225k. As the backshore elevation is above the likely still water level to 2060, the works are likely to provide adequate protection for at least 50 years.

- Pros
    - Likely to provide both long term erosion and inundation protection
- Cons
    - Likely to reduce high tide beach width
    - May produce end effects
    - Periodic maintenance
    - Likely to limit dune formation seaward of the protection works.
    - Cost

6.1.2.3 Option 3 - Realignment of shoreline and road

Realignment of the slip shoreline with the adjacent shoreline can allow the beach to behave more consistently and reduce adverse effects on adjacent shoreline. However, realignment of the road will also require realignment of the road into the slip debris. The works associated with realigning the road are likely to include stabilisation of the slip.

- Pros
    - Likely to provide greater high tide beach width
    - Allow the shoreline to behave consistently
    - Reduce end effects
    - May allow formation accretion of beach
    - May reduce amount of shoreline protection required
- Cons
    - Requires significant stabilisation of slip
    - Rock protection may still be required along foreshore
    - Likely high cost.

6.1.3 Reserve area in front of School (Area 3)

Recent erosion has reduced the reserve area, putting at risk a number of Norfolk pine trees. In an attempt to mitigate the erosion and save the trees, tipped rock has been placed along the northern 40 m of the foreshore. The tipped rock has probably reduced the erosion rate, however is not likely to continue to reduce the erosion significantly due to the 'ad-hoc' nature of placement.

6.1.3.1 Option 1 Do nothing

As the historical shoreline position along Area 3 generally fluctuates, there is the possibility that some accretion may occur in the near future. Conversely, the erosion phase may continue with the loss of the Norfolk pines.

- Pros
  - No/low cost
  - Possible natural accretion phase may occur
  - Likely end effects at south end of rock placement
- Cons
  - Uncertainty
  - Possible loss of Norfolk pines and reserve area
  - Existing ad-hoc rock protection may exacerbate erosion.

6.1.3.2 Option 2 – Extension and remedial works of existing rock protection

The existing rock protection to be replaced with more formal rock protection along the foreshore. Rock protection to extend to the stream mouth at the north end of the reserve and the toilet block to the south. Costs are likely to be around $500 per linear meter with a total length of 100 m. Therefore, total cost estimate of $50,000

- Pros
  - Provides protection of reserve and trees
  - Provides barrier to shoreline fluctuations
- Cons
  - Cost
  - Loss of natural character
  - Possible reduction of high tide beach width.

6.1.3.3 Option 3 – Removal of trees, re-contouring and planting

The reserve area could be re designed to encourage a more natural beach environment with native sand binding vegetation. Using sand binding vegetation is likely to encourage formation of a sand dune along the foreshore. Over time the sand dune acts as a sand buffer potentially reducing the effects of shoreline fluctuations.

While the sand dune/buffer does not stop erosion, establishment of sand binding vegetation provides a mechanism to allow faster rejuvenation of the foreshore after an erosion event.

- Pros
  - Relatively low cost with community input (beach care group)
  - Enhanced natural character
  - Reduced maintenance once dune vegetation established
- Cons
  - Likely loss of Norfolk Pines

    - May take a number of years for vegetation to establish and formation of dune buffer
    - Does not stop landward movement of shoreline during erosion events

6.1.4 Toilet Block (Area 3)

The area in front of the toilet block does not currently show evidence of significant erosion. However the presence of the rock protection along the foreshore suggests erosion was a problem in the past and is likely in the future. The only reason for the rock protection is to protect the toilet block.

The rock protection is 'ad-hoc' with no formal placement or appropriate design. However, the rock is likely to provide some protection. While there is no immediate need to address the toilet block area, there may be a need to remediate the area depending on any changes to the adjacent shorelines.

6.1.5 Boat club

The boat club and car park has required erosion protection due to the retreat of the shoreline since 1967. A 90 m rock seawall has provided appropriate protection since built and visually looks to be in good state of repair. However, end effects (exacerbated erosion) are evident at both the north and south end.

The only reason for the rock protection is to protect the boat club.

6.1.5.1 Option 1 Do nothing

Keeping the status quo is likely to result in gradual increase of erosion at the north and south ends of the rock sea wall.

6.1.5.2 Option 2 Extension and remedial works of existing rock protection

Extending the rock seawall north to the beach access is likely to move the end effects to an area that is less prone to end effects. The southern extent of the seawall could be extended and tied back into the back shore to reduce end effects. Costs are likely to be approximately $500 to $1000 per linear meter, with approximately 100 m required. Therefore costs likely to be from $50k to $100k.

- Pros
    - Reduces end effects north and south of the seawall
    - Continued protection of the boat club from erosion.
- Cons
    - Increased length of rock protection decreasing natural character
    - End effects still likely to be evident, especially at the southern end
    - Loss of high tide beach width in front of seawall.

6.1.5.3 Option 3 Relocation Or removal of boat club and existing rock protection

As the only reason for the rock protection is to protect the boat club, removal or relocation of the boat club will remove the need for the rock protection. Removal of the rock protection is likely to allow the shoreline to retreat approximately 12 m to be in alignment with the adjacent shoreline.

- Pros

  - Return to natural shoreline
  - Continued protection of the boat club from erosion.
- Cons
  - Community opposition to removal of boat club
  - Potential for future increased erosion risk to buildings landward of current rock protection.

6.2 Medium term hazard mitigation

Medium term hazard mitigation is to address coastal erosion and inundation issues on a broader scale than in Section 6.1. The medium term hazard mitigation is to provide a 'bigger picture' of options that can be undertaken which may incorporate the localised areas in Section 5.1, but provides a more consistent approach along the shoreline.

Primarily, the medium term options are concerned with the beach, foreshore and backshore areas from the reserve in front of the school boat club south of the boat club. Within the shoreline area are a number of the localised issues contained within Section 5.1 as well as the majority of dwellings within Akitio Village.

The medium term mitigation options are not intended to address inundation and erosion issues to 2060 and beyond, but are an intermediate step which may reduce the current effects of erosion and inundation.

6.2.1 Establishment of frontal dune

The current back shore vegetation includes marram grass along a grass verge with no significant frontal dune system. Although marram grass has traditionally been planted along shorelines, the effectiveness in beach rehabilitation and forming a frontal dune is relatively poor compared to alternative dune forming vegetation.

Establishing appropriate backshore vegetation can increase the amount of sand stored on the beach, resulting in an increase in beach volume and elevation. The increase in beach volume assists in creating a buffer against storm erosion.

The establishment of the frontal dune is also likely to reduce the effects or storm induced inundation by raising the backshore elevation. Raising the backshore elevation is likely to reduce the landward extent of inundation caused by wave run up.

Establishing vegetation on the bank slope and crest assists in coastal erosion management by reducing weathering and increasing slope stability. Plant roots are important for binding soils and reducing soil saturation on coastal banks. Planting on the bank crest and slope will also help to trap wind blown sand from migrating east onto the grass reserve and road. Sand can also be trapped by installing sand fences along the backshore and providing public access ways to the beach.

Examples of native species suitable for backshore planting are:

- Pingao - coastal dune flax that can withstand salt spray.
- Spinifex - coastal grass that forms a network of runners and is an effective sand trap
- Muehlenbeckia complexa - hardy shrub ideal for dense ground cover on coastal banks
- Wharariki - low lying mountain flax that can withstand strong coastal winds.

Planting is a relatively low cost option that will help stabilise the bank and beach with no adverse effects to the coastal environment. The planting and beach rehabilitation can become a community project as much of the planting work does not require significant engineering.

We recommend that beach rehabilitation start at the southern end (north of the boat house) and move north. The southern end is more sheltered from wave events and has an increased sand volume than the northern end, increasing chance of dune vegetation becoming established. Once established the dune vegetation can be extended further north over time.

6.3 Long term hazard mitigation

Long term hazard mitigation is to address potential inundation and erosion hazards to at least 2060. The recommended mitigation is to also address the entire Akitio shoreline from the river mouth to south of the salient. The recommended engineered approach to mitigate future hazards should be used to assist council in ascertaining the long term viability of Akitio as a sustainable coastal village.

If preservation of the existing access and community at Akitio is warranted, then an engineering option will be required. Should an engineered approach to mitigate future hazards be untenable by the community and/or council, then the only other viable option is removal or relocation of dwellings. Considering the social and economic implications of removing dwellings, further assessment is recommended once a decision on engineering options is assessed.

6.3.1 Increase road height

Increasing road height along the Akitio shoreline is to ensure access into Akitio village and provide protection to dwellings along the low lying backshore area, landward of the road.

Total length of road along the Akitio shoreline (to the boat club) requiring increased elevation to mitigate 2% AEP event to 2060 is approximately 1.6km.

6.3.2 Rock revetment construction

The rock revetment is intended to form the seaward face of the recommended raised road to mitigate predicted increased water levels and erosion to 2060. The establishment of a frontal dune, recommended in Section 5.2 is likely to be retained/incorporated. Therefore, the road and incorporated rock revetment can be classed as a 'back stop wall' for the short to medium term.

Note that in some areas the rock revetment may encroach onto the foreshore.

For the short to medium term the majority of the current shoreline position is likely to remain and function as normal, however at some point in the future the shoreline position is likely to retreat to the base of the rock revetment. Therefore, there is likely to be no high tide beach along the revetment in the future (2060).

Revetments provide a hard line defence to protect the road from toe erosion due to coastal processes. Any revetment structure should aim to have low visual impact and maintain the natural character of the coastal environment. A revetment could be constructed as either a new structure or as part of reconstruction/upgrade of an existing structure.

The revetment crest should be a minimum of 4.0 m RL in order to reduce over-topping during 2% AEP storm event including allowance for sea level rise to 2060. This level is based on a design water level of 3.5 m RL and a free board for wave run-up (refer to Section 4). The method is likely to require excavation down to competent ground to reduce scour effects. Based on nearshore wave height and water depth, potential scour depth is 1.0 m below the existing bank toe elevation.

The rock grading for a double layer rock armour structure (slope 1:1.5) should be between 0.6 and 1.0 m diameter with a D_{50} of 0.8 m (Van der Meer method; CIRIA, 1991).

Total length of road along the Akitio shoreline (to the boat club) requiring rock revetment to mitigate 2% AEP event to 2060 is approximately 2.0 km.

6.4 Cost estimate summary for coastal protection works

The construction cost estimates for the medium and long term mitigation options are presented in Table 5.1 below. The construction cost estimates are based on a 100 m section of shoreline and the linear metre rates can be used to estimate costs for longer sections of shoreline. The total cost estimate includes a contingency set at 20 % of the physical works.

Table 6.1: Construction cost summary for potential coastal protection options.

Item	**100 m of Rock Revetment only (Long term mitigation)**	**100 m of Frontal Dune Establishment (Medium term mitigation)**
Physical Works	$220,000	$12,000
Contingency (20%)	$44,000	$3,000
Total Cost Estimate (excluding GST)	$264,000 ($2,640/LM)	$15,000 ($150/LM)

Note that the cost are for construction of the rock revetment only and does not include cost of increasing road base elevation to RL 4.0 m, sealing, road marking etc along 1.6 km of the existing road.

Therefore, indicative costs for establishing a frontal dune along 1 km of Akitio Village foreshore (medium term hazard mitigation) is approximately $150,000.

Similarly, indicative costs for construction of 2.0 km of rock revetment along the raised road (medium term hazard mitigation) are approximately $5,280,000 (not including cost of re establishing road at 4.0 m RL).

7 Conclusion and summary

The Akitio coastal village is situated along a narrow strip (generally < 80 m) of low lying (generally < 3.5 m RL) coast at the base of predominant hills/cliffs. Significant features are the Akitio river mouth to the north and an attached rock platform towards the south.

In between these two features the coast line can be delineated into 5 areas:

- Area 1 – river mouth
- Area 2 – river mouth to school
- Area 3 - school to boat club
- Area 4 - boat club and camp ground (salient)
- Area 5 south of salient

The predominant coastal feature of Herbertville is the Wainui River, with the majority of the Herbertville township situated on low lying river derived sediments. The Wainui River exists into a coastal lagoon that periodically exits out to sea. For periods of time the river is closed off from the sea and fills the inshore lagoon. This type of feature is called a 'hapua'.

Shoreline change analysis from 1944 to 2010 for Akitio has shown that the majority of the shoreline is in dynamic equilibrium, with no significant trend of erosion or accretion. Shoreline accretion is evident in Area 1 along the river bank. Shoreline erosion is also evident within Area 1 along the road adjacent to the river of -0.07 m/yr. Shoreline erosion of -0.2 m/yr is occurring within Area 4.

The Herbertville shoreline is has been accreting since 1944 at rates up to 0.9 m per year.

CEHZ setbacks have been determined for Akitio and Herbertville which take into account current long term shoreline trends, short term shoreline fluctuations and shoreline retreat associated with predicted sea level rise to 2060 (0.36 m) and 2110 (0.9 m and 1.5 m).

Along the Akitio shoreline, CEHZ setbacks to 2060 range from 34 m to 59 m. While CEHZ setbacks to 2110 (0.9 m) range from 59 m to 94 m and 2110 (1.5 m) range from 84 m to 119 m. Based on the CEHZ estimates, much of the low lying Akitio shoreline is likely to be at risk of coastal erosion by 2060. CEHZ setbacks for Herbertville are 40 m, 50 m, and 60 m for 2060, 2110 (0.9 m) and 2110 (1.5 m) respectively.

Coastal inundation including wave set up and run up have been estimated for Akitio using a number of water level and wave scenarios. The wave parameters for 1% and 2% AEP events have been calculated using wave hind cast data from a global wave model. Calculated inundation values compared well with measured values for a storm event in2008.

Inundation modelling showed that the low lying Akitio shoreline is at high risk of inundation in the next 50 years by a 2% AEP storm event resulting in inundation (including wave set up) to 3.39 m RL.

Considering the high risk of both coastal erosion and inundation affecting the majority of low lying dwellings and infrastructure, Akitio requires significant engineered solutions to mitigate coastal hazards from 2060 and beyond.

Mitigation can be undertaken at relatively low cost to remediate localised, short to medium term coastal hazard issues. However, a long term engineered solution to provide protection of a 2% AEP wave and water level event to 2060 is likely to cost in excess of $5,000,000.

8 Applicability

This report has been prepared for the benefit of (the client's name) with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

Tonkin & Taylor Ltd

Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
Rick Liefting	Richard Reinen-Hamill
Senior Coastal Scientist	Senior Coastal Engineer
Mark Ivamy	
Coastal Scientist	

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Appendix A: Figures

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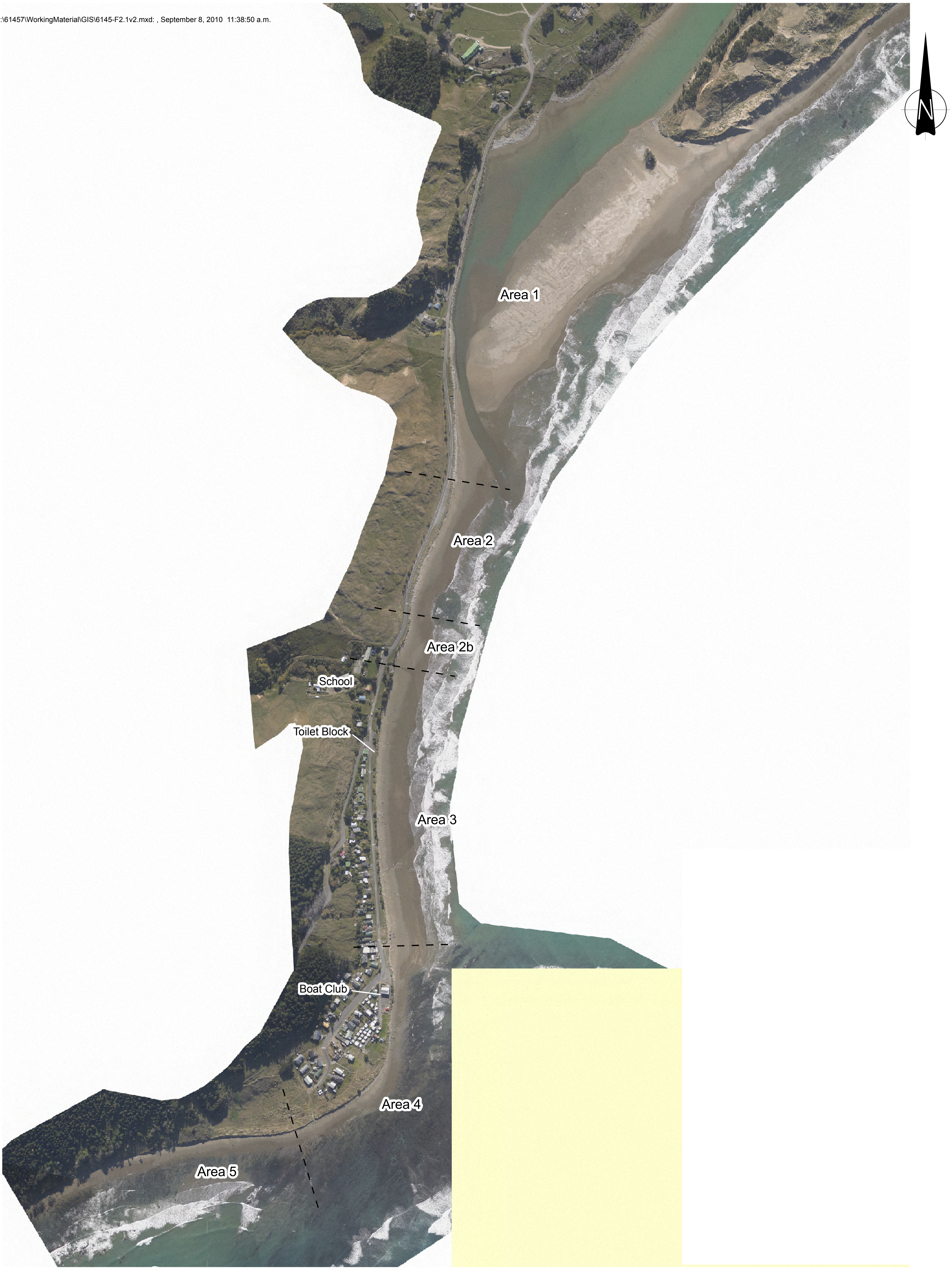

Notes: Background aerial sourced from Horizons

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Akitio
Site Plan

FIGURE No. Figure 2.1.A3

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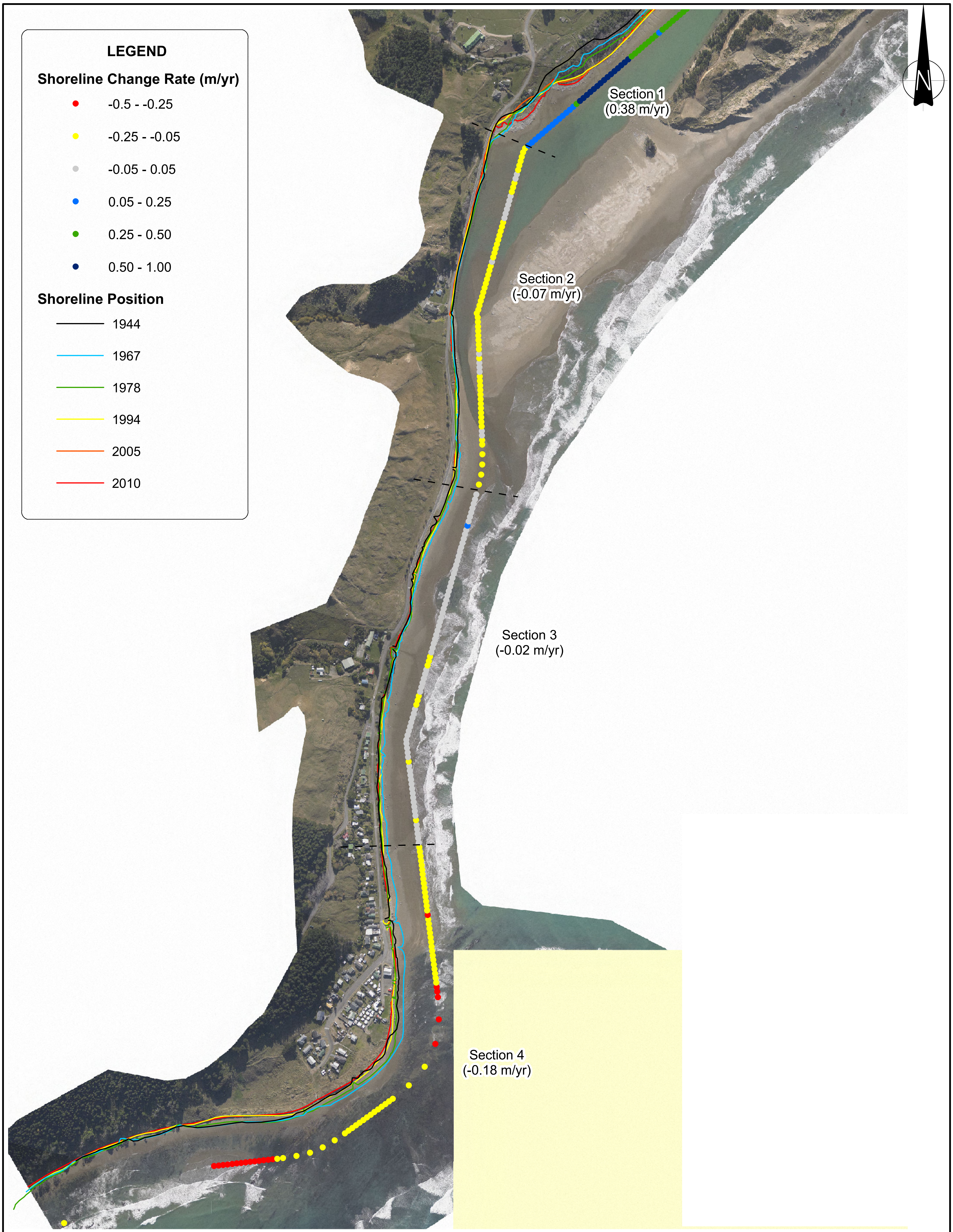

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Horizons Regional Council
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Akitio
Shoreline Change Rate

FIGURE No. Figure 2.2.

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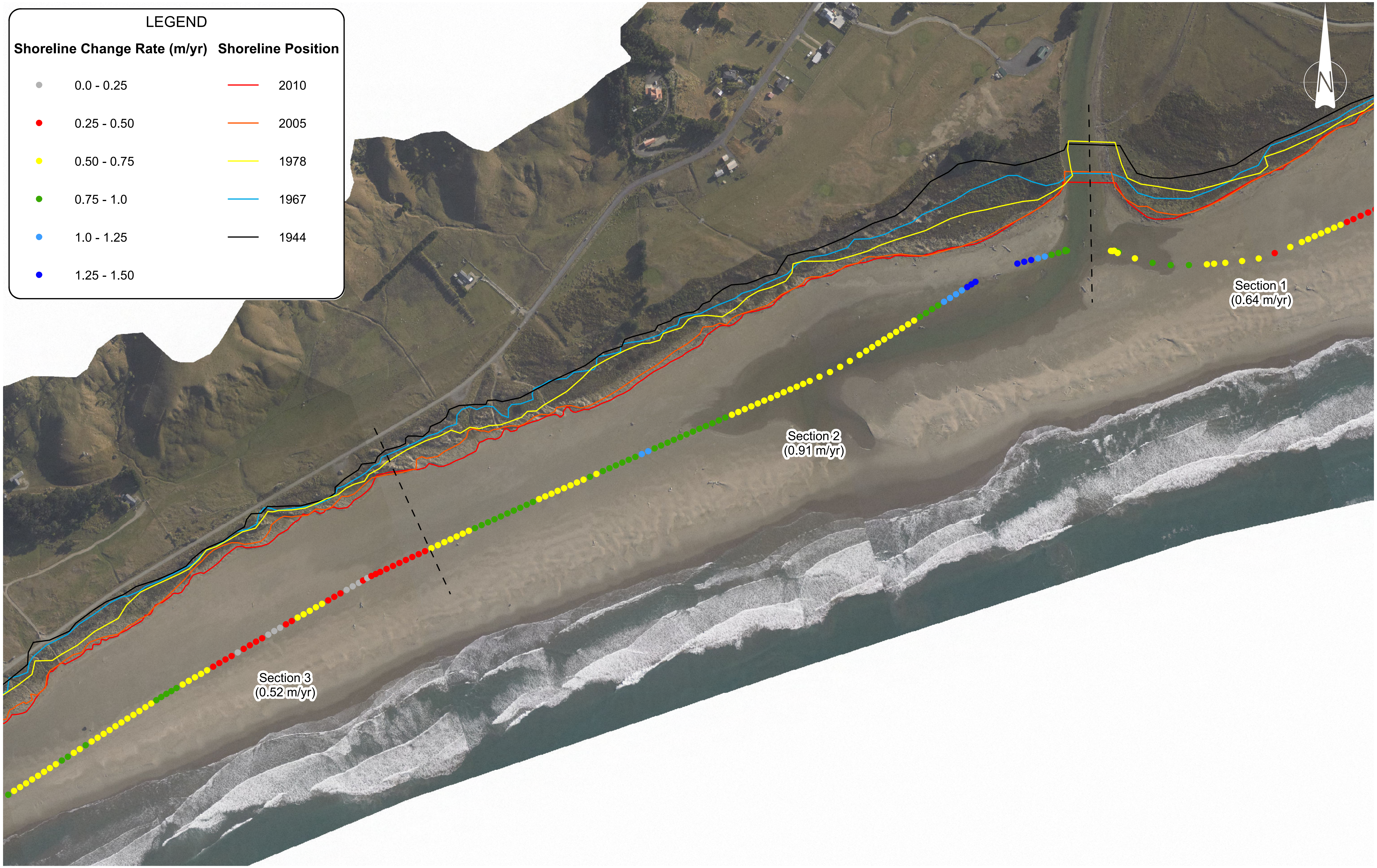

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Horizons Regional Council
Coastal Hazard Assessment
Herbertville
Shoreline Change Rate

FIGURE No. Figure 2.3.

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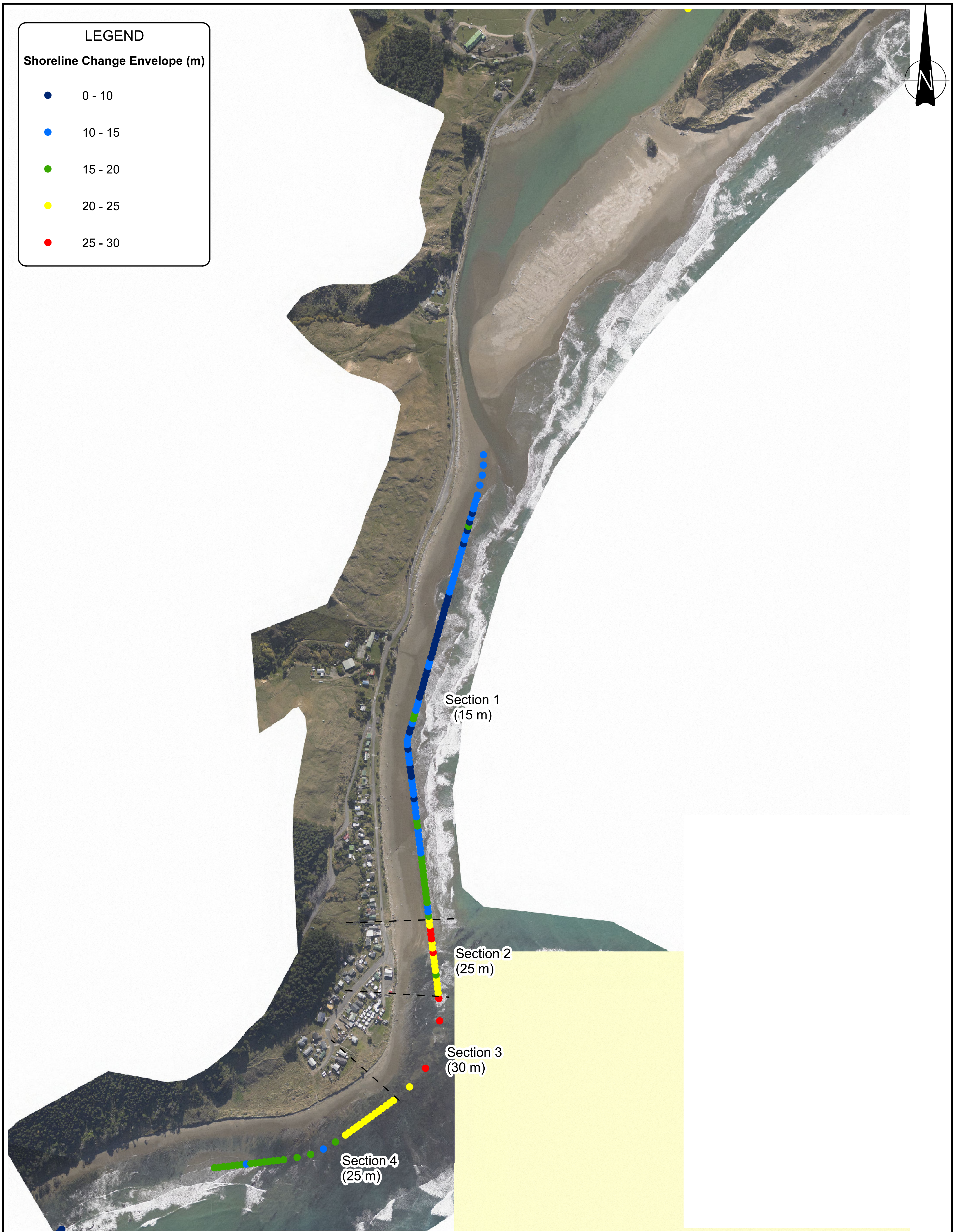

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Shoreline Change Envelope

FIGURE No. Figure 2.4.A3

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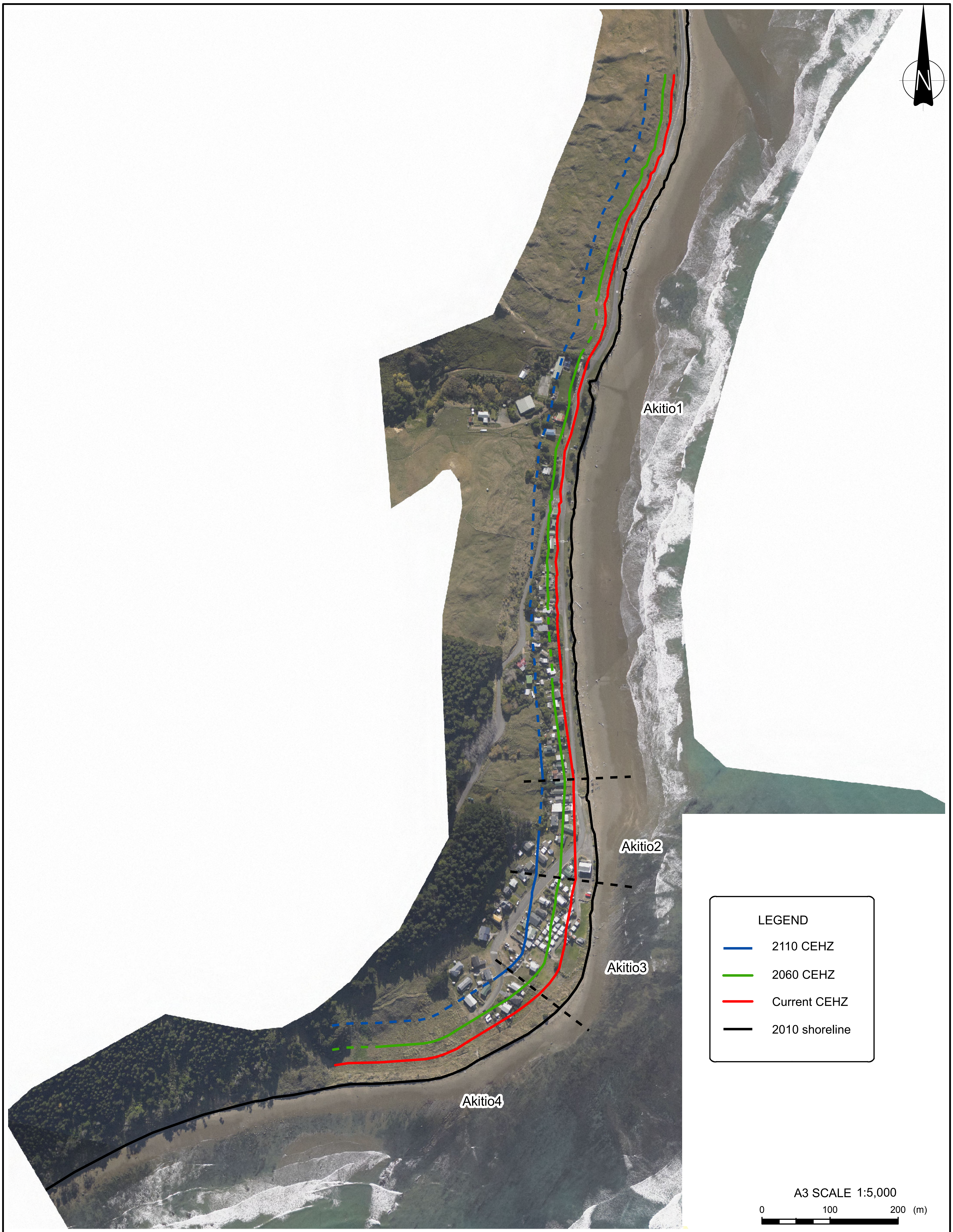

Notes: Background aerial sourced from Horizons.
Dotted line segments represent the transition zone, from beach to cliff material.

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HRC and TDC
COASTAL HAZARD ASSESSMENT
Akitio
Coastal Erosion Hazard Zone

FIGURE No. Figure 3.4.

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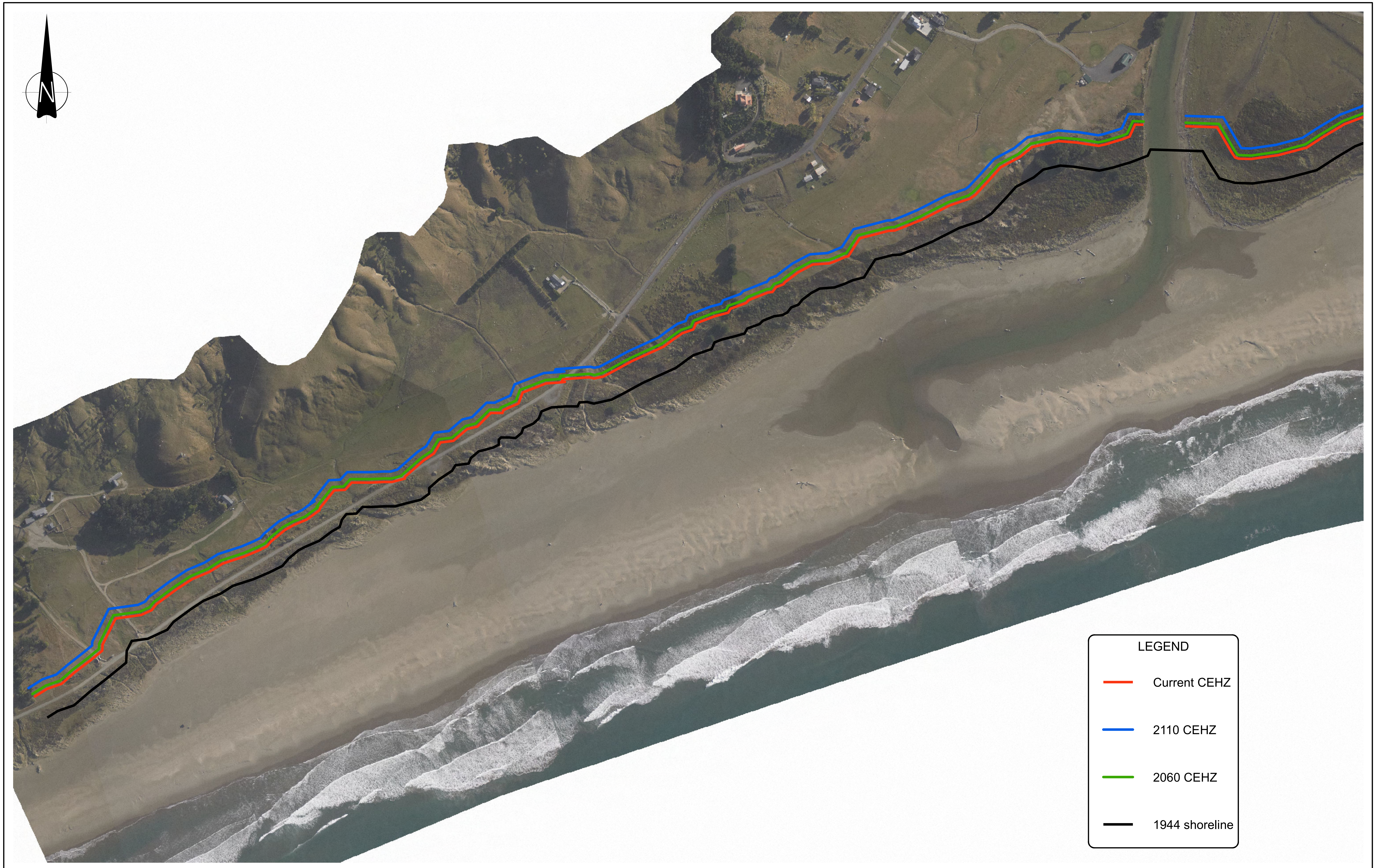

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Horizons Regional Council
Coastal Hazard Assesment
Herbertville
Coastal Erosion Hazard Zones

FIGURE No. 61457-F3.5.

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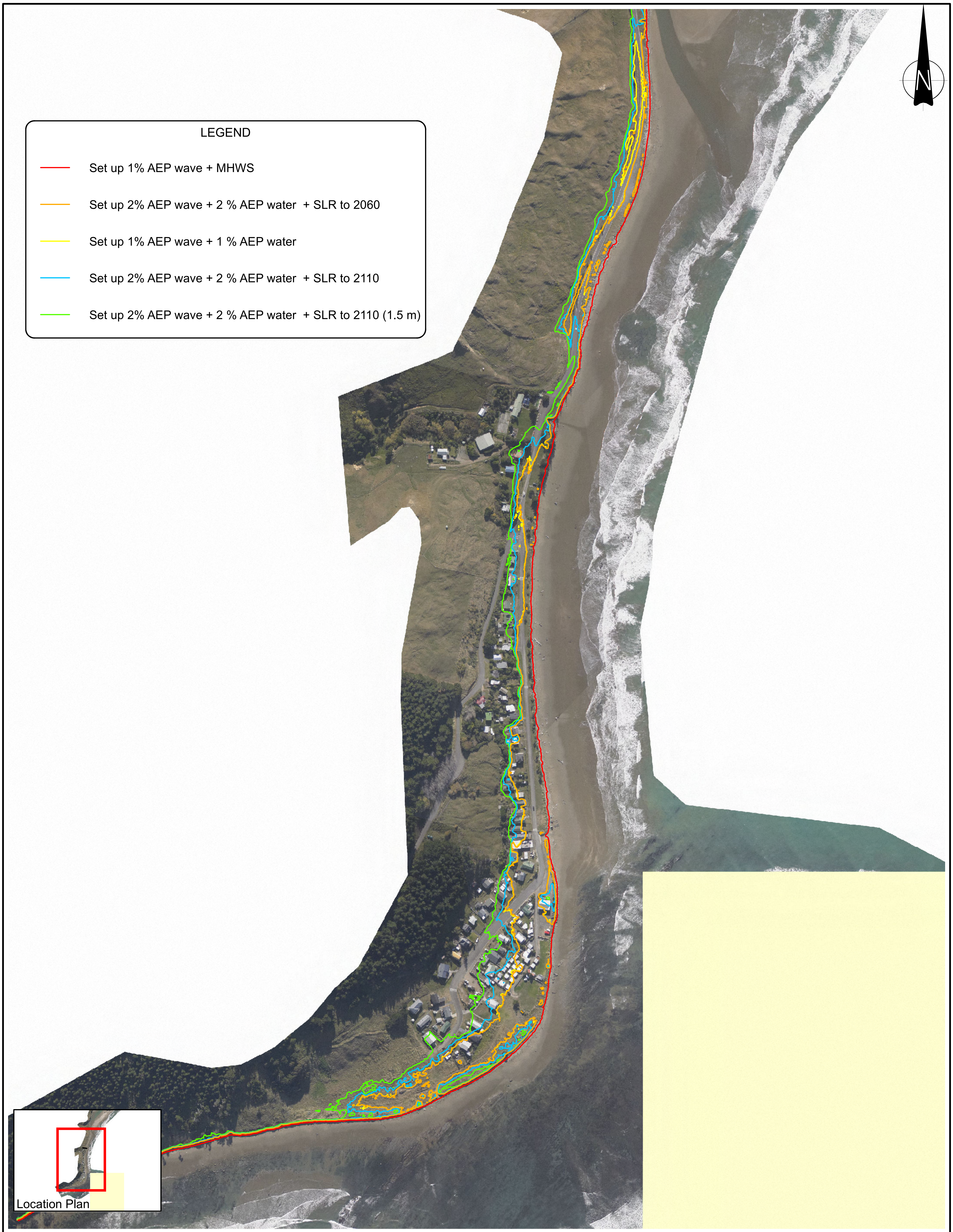

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Akitio Wave Set Up Inundation
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FIGURE No. Figure 4.1A3

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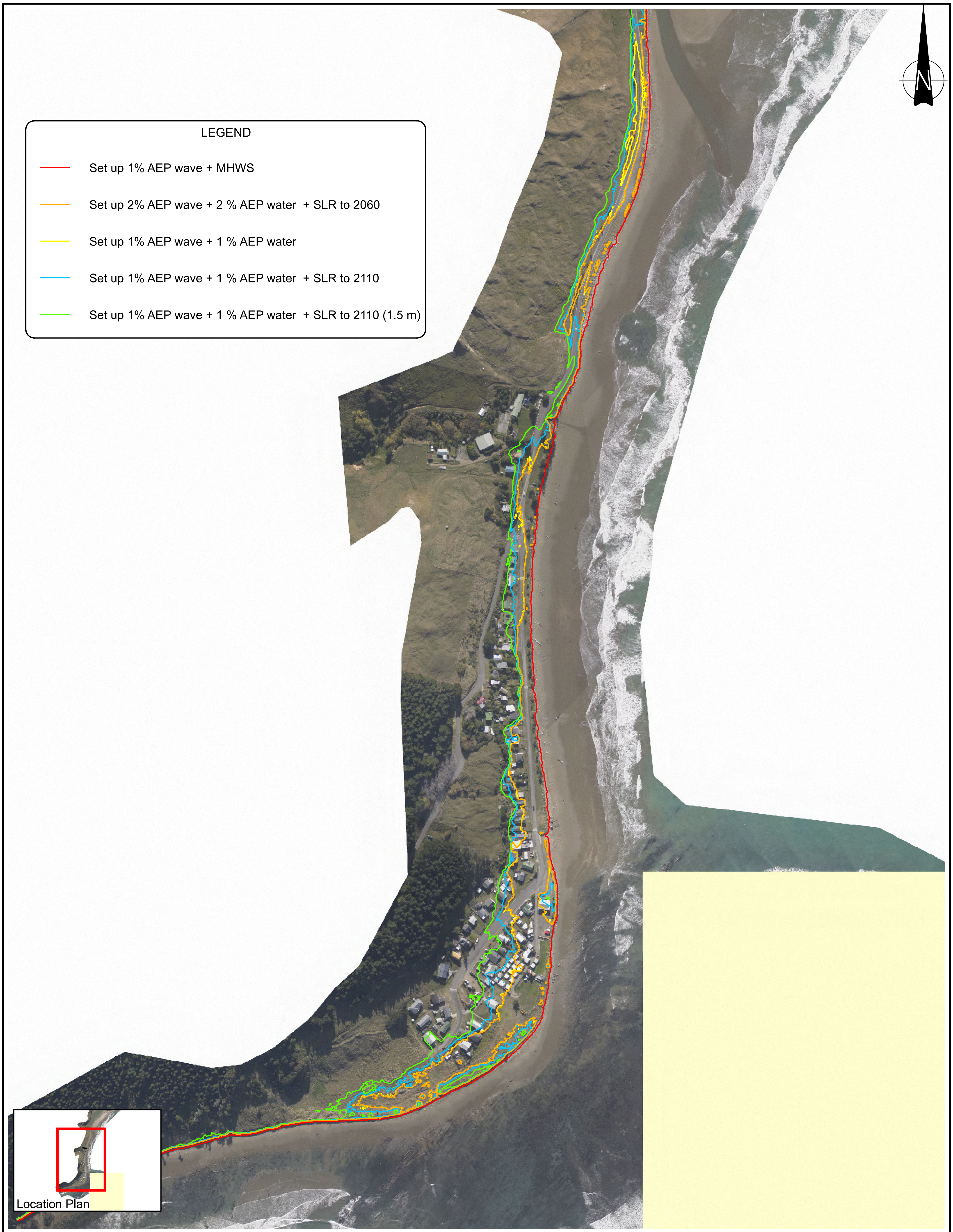

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Akitio Wave Set Up Inundation
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FIGURE No. Figure 4.2A3

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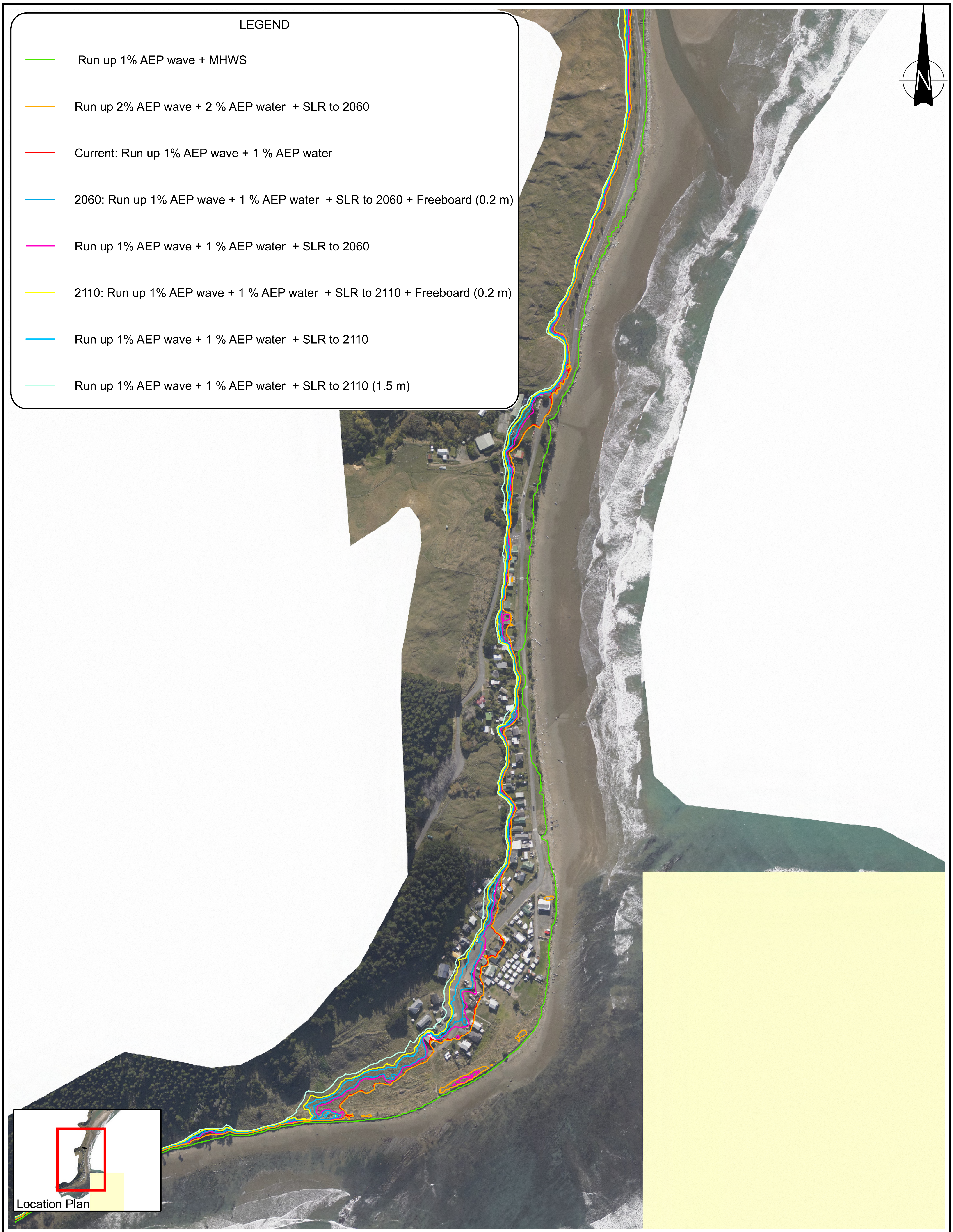

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FIGURE No. Figure 4.3

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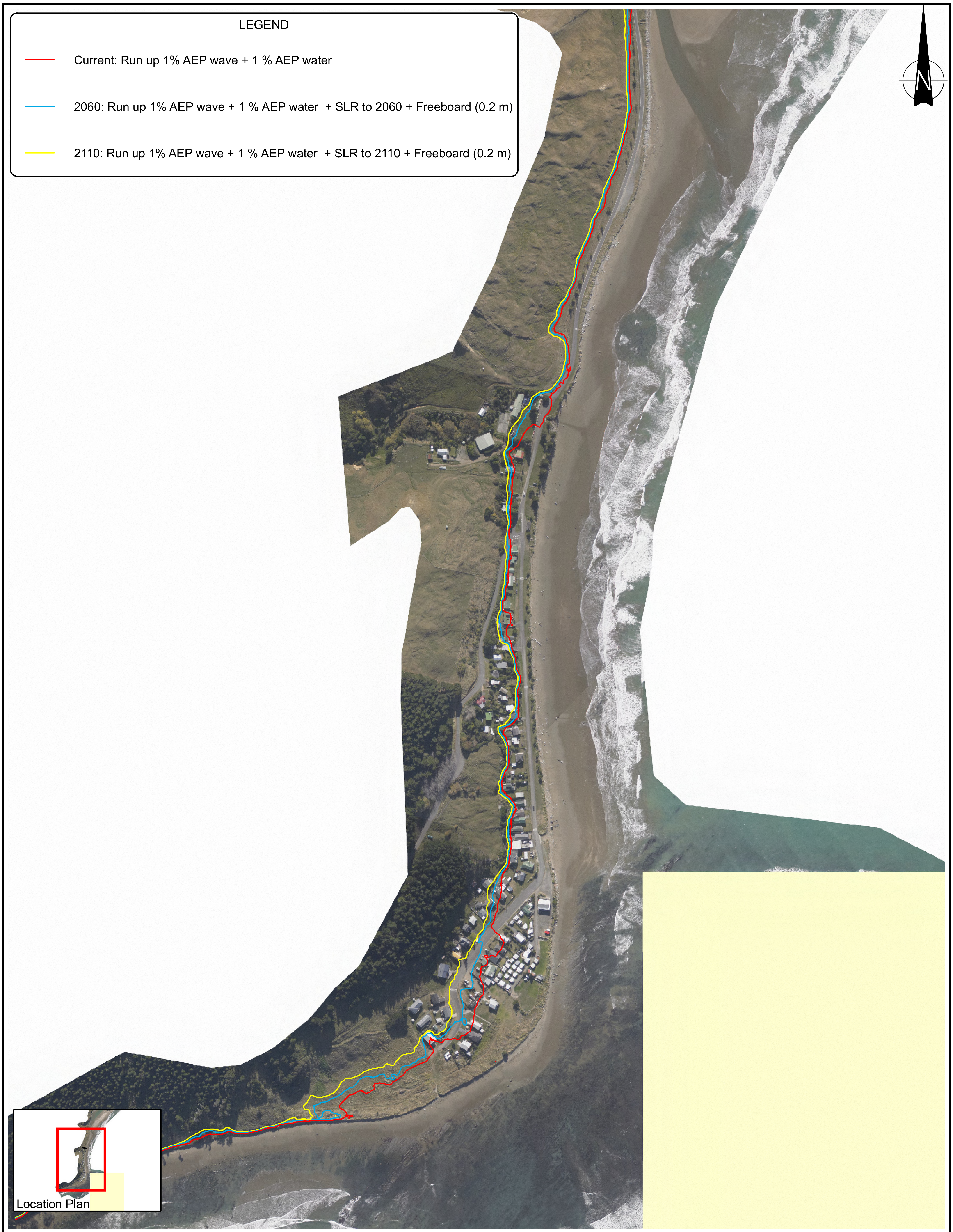

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FIGURE No. Figure 4.4

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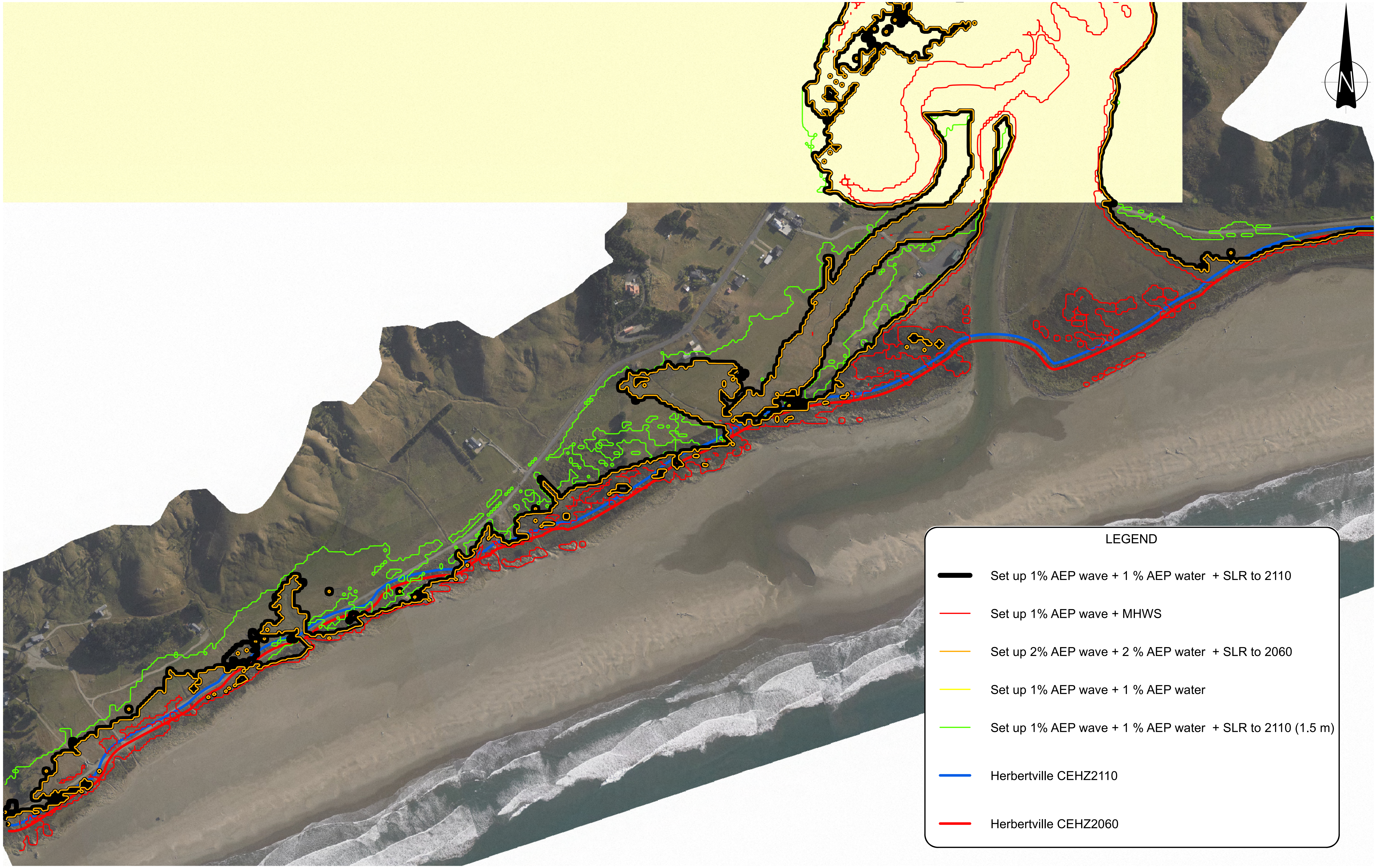

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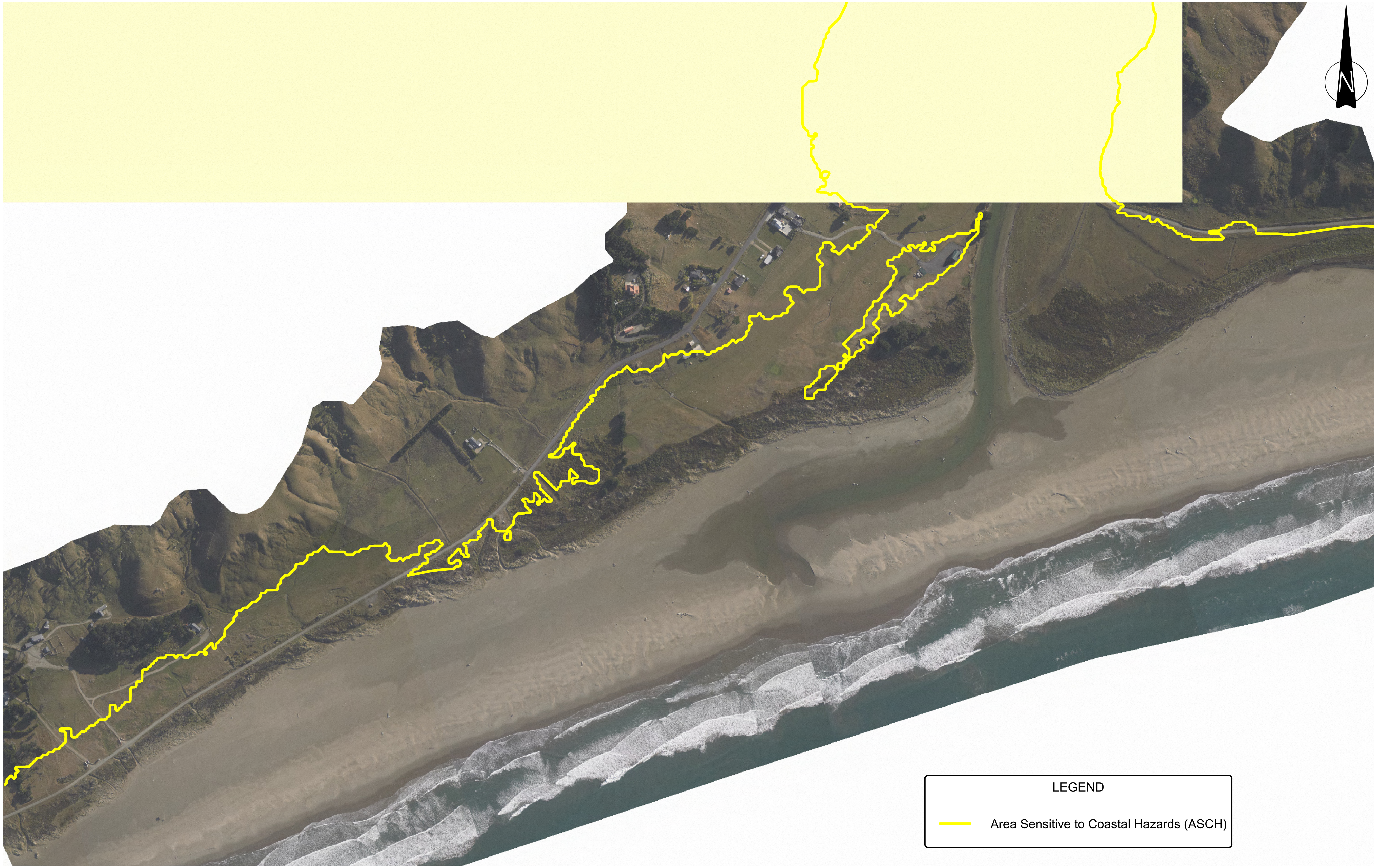
FIGURE No. Figure 4.5A3
Rev. 0



Notes:

A3 SCALE 1:6,000

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Tonkin & Taylor
Environmental and Engineering Consultants
105 Carlton Gore Rd, Newmarket, Auckland
www.tonkin.co.nz

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SCALE (AT A3 SIZE) 1:6,000

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HRC AND TDC
COASTAL HAZARD ASSESSMENT
Herbertville ASCH
Sheet A3

FIGURE No. Figure 4.6A3

Rev. 0

Appendix B: Site Photos

Photo 1 looking south east showing extent of the Akitio River spit and current location of the river mouth.

Photo 2 looking west showing erosion scarp near river mouth

Photo 3 looking west showing erosion scarp exposing back shore slip material

Photo 4 showing exposed slip debris just north of school

Photo 5 looking west showing protruding shoreline with placed rock protection and recent erosion scarp exposing slip debris backshore.

Photo 6 looking west showing low lying reserve with Norfolk pines and tipped rock erosion protection.

Photo 7 looking west showing erosion protection in front of toilet block

Photo 8 looking west showing area in front of the camp ground. Note localised erosion (end effects) at end of the rock protection.

Photo 9 looking west showing rock erosion protection in front of the boat club.

Photo 10 looking north showing low lying salient formation in the lee of the rock platform/reef. Note extensive marram grass

Photo 11 looking north towards Herbertville with Cape Turnagain in the distance. Note the wide beach, which periodically turns into a lagoon.