Hurunui District Coastline Hazard and Risk Assessment

Hurunui District Coastal Hazard and Risk Assessment

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Appendix Booklet

Executive Summary

Hurunui District Council (HDC) commissioned Jacobs to undertake an assessment of how coastal hazards will change with projected climate change scenarios over the next 100 years, and the risks to coastal settlements and critical council infrastructure. The hazards covered in the assessment are coastal erosion, coastal inundation and rising groundwater leading to shallow groundwater levels within five coastal settlements of Leithfield Beach, Amberley Beach, Motunau, Gore Bay, and Claverley, plus the section of Conway Flat Road that runs close to the coastal cliffs.

The purpose of this assessment is to help aid engagement with coastal communities on the potential consequences of the changing hazard and the development of a council strategy for adaptive pathways to manage the future hazards within their coastal communities.

The assessment of potential consequences of coastal hazards and subsequent risk to properties and dwellings covers six coastal settlements defined in the District Plan: Leithfield Beach, Amberley Beach, Motunau, Gore Bay, Conway Flat, and Claverley, as well as council identified critical infrastructure located within the coastal environment surrounding the settlements.

Coastal Erosion Hazard and Risk

For this assessment Projected Future Shoreline Positions (PFSP) were estimated for 30, 50 and 100-years' time from the combination of extrapolation of historical rates of shoreline movement, the effects of future accelerated sea level rise (SLR) under RCP8.5 and RCP8.5+ scenarios, and short-term storm retreat. The results indicated than all settlements are most likely to be subjected to coastal erosion over all the timeframes considered, even the currently accreting shorelines at Leithfield Beach and Claverley. Largest PFSP distances from the current shoreline position were estimated to be at Amberley Beach, Motunau and Gore Bay, being 20-45 m by 2050, 25-75 m by 2070, and 65-170 m by 2120. Estimated erosion distances were slightly less at Leithfield Beach, particularly at the southern end closest to sediment supply from the Waimakariri and Ashley Rivers. Considerably less erosion was projected at Conway Flat and Claverley with retreat distances less than 10 m by 2030 with the majority being due to short-term storm effects, up to 13 m by 2070, and maximum of 24 m by 2120.

Properties in Amberley Beach, Motunau and Gore Bay settlements are the most risk from the coastal erosion hazards due to the close proximity of the settlements to the shorelines. In Motunau and Gore Bay, a number of properties will be affected in 30 years, and this incrementally increases as sea level rises to be 100% (106 properties) and 46% (69 properties) of the current properties at Gore Bay and Motunau respectively within 100 years under the RCP8.5+ SLR scenario. At Motunau the number of properties assessed as being at risk is likely to be an under estimate due to the lower river mouth terrace not being included in the erosion assessment.

At Amberley Beach, no properties are projected to be at risk from coastal erosion within the 30-year timeframe, but there are 6% (15 properties) mapped as being affected in 50 years, and 33% (45 properties) within 100 years. At Leithfield Beach the 200m vegetated back shore buffer between the settlement and the shoreline provides protection for properties from coastal erosion for up to 100 years. At Claverley, although the PFSP distances are much lower, 15% (2 properties) of the current properties in the settlement could be at risk from coastal erosion within 30 years, and up to 62% (8 properties) are projected to be at risk within 100 years.

The most at-risk critical infrastructure from coastal erosion was assessed to be the coastal segments of road at Amberley Beach (Golf Links Rd), Conway Flat (Conway Flat Rd) and Claverley (Claverley Rd). Sections of all these roads, plus sections of Cathedral Rd and Gore Bay Rd in Gore Bay are projected to be affected by erosion within 30 years. The only additional piece of critical infrastructure assessed likely to be affected by coastal erosion is

the wet well on the lower river mouth terrace at Motunau. The coastal inundation bund at Amberley Beach was also assessed as likely to be totally lost to coastal erosion within 30 years.

Coastal Inundation Hazard and Risk

The worst affected settlements by potential coastal inundation as assessed in the bathtub modelling to be Leithfield Beach and Amberley Beach, with both settlements having coastal inlets with barrier beaches below the modelled static water levels, and low-lying topography over which this water could easily spread to inundate. At Leithfield Beach, the spatial extent of the coastal inundation hazard from a 1% AEP static water level was modelled to potentially cover 99% of the settlement under all scenarios, with modelled average inundation depths across the settlement increasing from 0.5 m in current sea levels to 2 m for 100-year SLR under the RCP8.5+ scenario. At Amberley Beach, with the current inundation bund elevations, the extent of inundation was modelled to be 20 to 30% of the settlement for current and 30-year sea levels respectively, increasing to 90% by 2070 and up to 99% by 2120. Modelled inundation depths for static water levels were less than at Leithfield Beach, being up 0.3 m for 30 years of SLR and up to 0.5 m for 50 years. However, wave run-up overtopping would be greater, and could increase inundation depths by up to 0.5 m for 1% AEP coastal storms with 50 years of SLR. It is noted that the bathtub modelling method produces very conservative results as it does not account for temporal variances of the event, or any hydrodynamic factors.

At Gore Bay the northern part of the settlement footprint is susceptible to coastal inundation under all scenarios including current sea levels from over topping of the low ridge in front of the combined mouths of the Buxton Creek and the Jed River, with the extent of potential inundation mapped for a 1% AEP static water level increasing from 10% under current sea level conditions to 15% in 50 year SLR and 20% in 100 years with SLR. Modelled average inundation depths increased from 0.3 m for current sea levels to 1.3 m under the 100-year RCP8.5+ scenario. The northern end of Gore Bay is also susceptible wave run-up overtopping over the low beach barrier along Gore Bay Road, which could add considerable inundation volume, increasing the inundation extent to cover around 35% of the total settlement under current and 30-year scenarios, and increasing inundation depths.

At Motunau, possible inundation under all scenarios is limited to around 10% of the settlement footprint located on the low river terrace, with depths of inundation under the bathtub modelling approach increasing from an average of 0.7 m for current sea levels to 1.8 m with SLR over the next 50 years. Along Conway Flat Rd, any potential inundation hazard is limited to the mouths of the numerous small streams and watercourses than discharge to the beach fronting the coastal cliffs. At Claverley, no coastal inundation hazard was detected from the 1% AEP static water level modelling, with wave run-up overtopping only potentially effecting the settlement in the 100-year RCP8.5+ scenario.

The most affected settlements in terms of risk to properties and dwellings from coastal inundation are Leithfield Beach and Amberley Beach, where even in the current day scenario 60% (Amberley Beach) to all or nearly all (Leithfield Beach) properties and dwellings intersect with the coastal inundation hazard footprint. For Amberley Beach this percentage increases to 80% (88 dwellings) under the 30-year SLR scenarios, and near 100% (108 dwellings) under the 50-year scenarios. For Gore Bay, properties and dwellings at risk from static water level inundation are limited to around 3% (2 dwellings) of the total settlement located at the northern end along Gore Bay Rd to the Buxton Creek under current conditions, and only increases to around 8% (8 dwellings) under both 100-year SLR scenarios. However, the inclusion of wave run-up overtopping increases this inundation risk to around 10% for both total settlement properties (13) and dwellings (8) under current conditions, and up to 40% 34 dwellings and 51 properties) under 100-year SLR scenarios. At Motunau, the at-risk properties and dwellings are limited to the lower river mouth terrace under all scenarios, being around 10% (12 dwellings) of the total settlement in all timeframes.

The wastewater treatment ponds at Amberley Beach and plant at Motunau are not expected to be subjected to any coastal inundation over the next 100 years. However, the wet wells in Amberley Beach, Leithfield Beach and Motunau are all expected to be inundated to some degree in future 1% AEP inundation scenarios.

Of the critical roads assessed, Golf Links Rd at Amberley Beach is at risk of inundation during 1% AEP storm events under all scenarios including current day levels. Although depths are shown to be only in the order of 0.2 m with 30 years of SLR, the addition of run-up overtopping water and velocities is likely to create issues for vehicle access in storm events well before this time. For the roads assessed at Conway Flat and Claverley, only segments are mapped as being at risk of coastal inundation from 50 years on, with inundation depths not likely to be an issue to closer to 100 years. At Gore Bay, the northern entrance via Gore Bay Rd is potentially at risk from inundation under current day 1% AEP storm conditions with inundation depths up to 0.2 m and increasing to 1 m with 100 years of SLR. At the southern entrance to the settlement parts of Cathedral Rd are also at risk from inundation by 1% AEP storm wave run-up overtopping under the 50-year RCP8.5+ scenario.

As well as water levels, future SLR will also increase the annual probability that the present day 1% AEP event will occur. Within 30 years this magnitude water level is two to five times as likely to occur in any one year, and within 50 years five to 15 times as likely to occur in any year. Within 100 years SLR under the RCP8.5+ scenario, this magnitude event would become an annual occurrence.

Groundwater Hazard and Risk

The settlements most susceptible to groundwater level rise in future scenarios are Leithfield Beach and Amberley Beach, due to the low-lying nature of the settlements and the shallow water tables. In Leithfield Beach at present, significant areas of existing development and infrastructure are located in areas of shallow groundwater (<1m BGL). Under the RCP 8.5+ 50yr SLR scenario the majority of the settlement is predicted to have groundwater levels shallower than 1m BGL, with areas shallower than 0.5m BGL encroaching on the settlement for the RCP 8.5+ 100yr SLR scenario. In Amberley Beach, the western margin of the settlement area is predicted to have groundwater levels shallower than 1m BGL, with some areas shallower than 0.5m BGL in the northwest with 1.3m of SLR in 100 years under the RCP8.5+ scenario. At both settlements saline incursion is predicted in the unconfined aquifer.

At Motunau and Gore Bay, the majority of the settlements are elevated and are not considered to be at risk from future groundwater rise scenarios. The main areas of risks at both settlements are near river mouths where ground water has the potential to rise to within 1 to 0.5m of the ground surface under the RCP8.5+ SLR scenario. Due to the high elevations of both Claverley and Conway Flat, under the RCP 8.5+ 100-year SLR scenarios it was determined that there was no significant impact and the areas were not at risk from future groundwater level rise.

Leithfield Beach is predicted to be the most at-risk settlement in terms of dwellings impacts by groundwater rise, with an increase from 2% (5 dwellings) currently exposed to average groundwater less than 0.5m BGL to 42% (112 dwellings) exposed with a 1.3 m SLR within 100 years. At Amberley Beach, the number of dwellings predicted to be impacted by groundwater shallower than 0.5m BGL increases from 0% to 11% (15 dwellings) by 100 years with 48% (66 dwellings) of dwellings in areas of groundwater shallower than 1m BGL, compared to 7% (9 dwellings) under the current scenario. At the rest of the settlements, no dwellings are expected to have groundwater within 1 m of ground level even under the 100-year RCP8.5+ SLR scenario.

Recommendations

For coastal erosion it is recommended that continued on-going monitoring of shoreline changes in both position and profile is required to verify and validate the extrapolation of past long-term rates and the role of accelerated SLR in future rates of shoreline retreat. Required enhancement of the current Environment Canterbury profile network enhancements include surveying of nearshore profiles at composite and MSG beaches for input in geometric models of SLR effects. It is also recommended that future research and assessment on the effects of

SLR on erosion at the Motunau River Mouth is required, and more detailed three-dimensional numerical modelling of geomorphic shoreline response to SLR be considered at some stage over the next 10 years for Amberley Beach, Motunau and Gore Bay.

From the results of the bathtub modelling, it is recommended that further hydrodynamic modelling of the inundation hazards is warranted at Leithfield Beach, Amberley Beach and Gore Bay to better quantify the threshold for overtopping and inundation, the spatial extent and magnitude (e.g. inundation depths) of the hazard, and risks posed to the dwellings and critical infrastructure that can be utilised as part of the decision making toward adaptive planning pathways. Any future modelling should also incorporate the effect of future erosion and changes to beach topography on future inundation hazards. It is also recommended that the risk assessment of dwellings could be improved by including floor level data for Leithfield Beach, Amberley Beach and Gore Bay.

Should further refinement of the rise in shallow groundwater hazard assessment be required to increase confidence in the outcomes, then additional data will be required to be collected. This would include accurate survey and levelling of groundwater monitoring locations, collection of contemporary, high frequency water level data at Leithfield Beach, Amberley Beach and inland areas so that data can be used to validate or refine current modelling. In order to further refine potential risk to the Leithfield Beach community water supply bore, it is recommended that a review of the test pumping data be undertaken to assess if the data can be used to estimate distance to an offshore discharge point.

1. Introduction

1.1 Project Background

Hurunui District Council (HDC) commissioned Jacobs to undertake an assessment of how coastal hazards will change with projected climate change scenarios over the next 100 years, and the risks to coastal settlements and critical council infrastructure. The hazards covered in the assessment are coastal erosion, coastal inundation and rising groundwater leading to shallow groundwater levels.

The assessment of potential consequences of coastal hazards and subsequent risk to properties and critical infrastructure covers the following:

- Coastal settlements of Leithfield Beach, Amberley Beach, Motunau, Gore Bay and Claverley (Figure 1.1) as defined in the Hurunui District Plan.
- Critical three waters infrastructure located within the coastal environment being the Amberley and Motunau wastewater treatment plants, the Leithfield Beach water supply bore, and wet wells in several settlements.
- Selected road corridors outside the settlements at Amberley Beach, Conway Flat and Claverley.

The quantification of the extent of future hazards and risks is required to inform Council about the hazards and how they will impact on coastal settlements and critical infrastructure over time. The purpose of this work is to help to aid engagement with coastal communities on the potential consequences of the changing hazard and the development of a council strategy for adaptive pathways to manage the future hazards within their coastal communities.

1.2 Project Scope

The scope of the project is to:

- Create hazard maps of the likely extent of future coastal erosion and sea water inundation hazards and the rise of groundwater level and salinity under a series of accepted sea level rise (SLR) scenarios.
- Undertake a high-level risk assessment by intersecting the above hazard maps with data on settlements and critical infrastructure to estimate the impact of coastal erosion, inundation and groundwater rise associated with SLR on communities and council services.
- Report the outcomes of the above assessments (this report) and present these in a workshop with Council.

1.3 Structure of the Report

This report is structured as follows:

Section 2 outlines the methodologies employed for the assessment of coastal erosion, coastal inundation and rising groundwater hazards with SLR and the assessment of risks from these hazards.

The resulting hazard maps and risk assessments are discussed for each township in turn from south to north with:

- Leithfield Beach in Section 3
- Amberley Beach in Section 4
- Motunau in Section 5
- Gore Bay in Section 6
- Conway Flat in Section 7
- Claverley in Section 8

Each section provides an overview of the settlement footprint, followed by summaries of extent and magnitude of each of hazards and the risk to dwellings and critical infrastructure. Section 9 contains summary conclusions from the study.

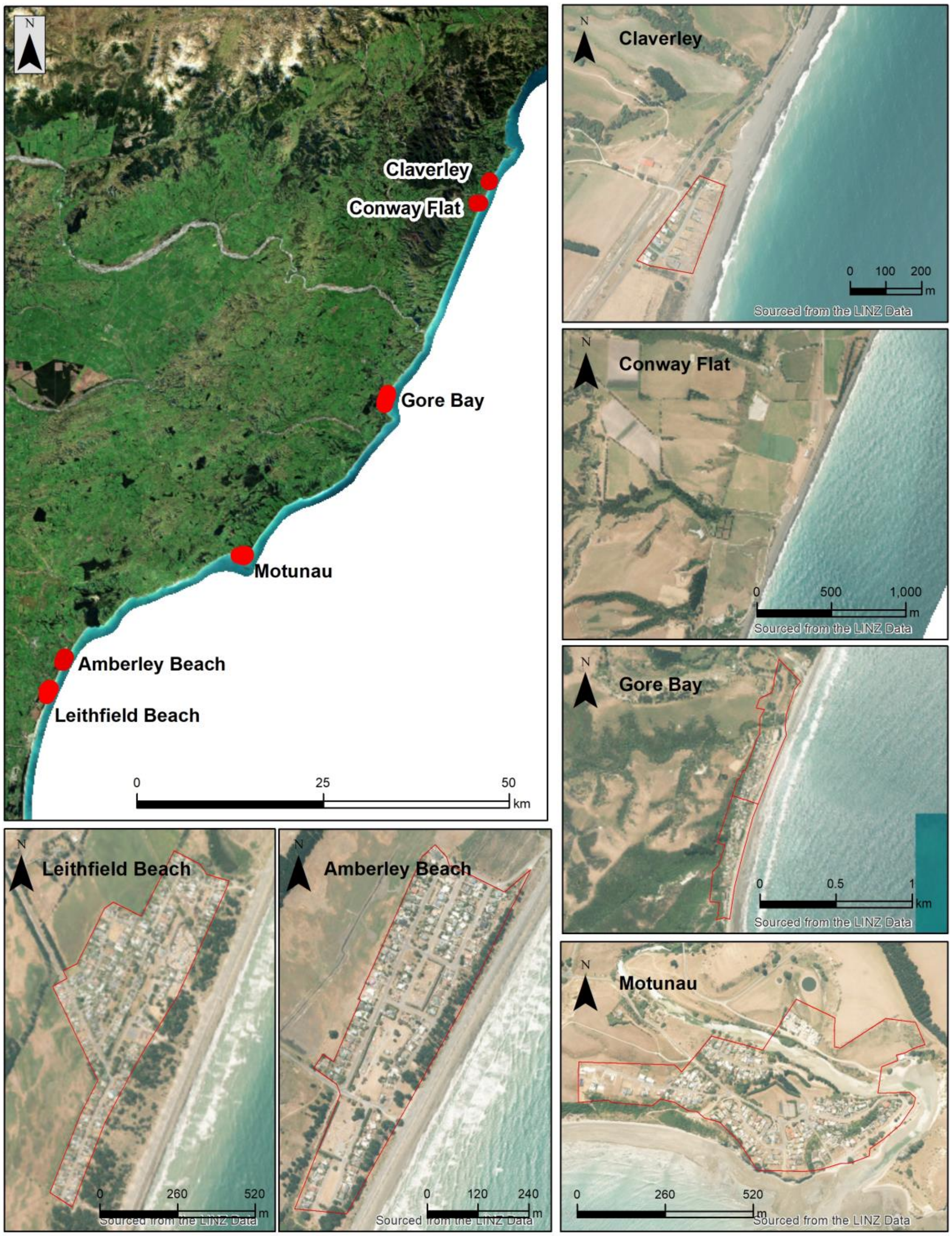

Figure 1.1: Location overview of assessed settlements along the Hurunui District Coastline.

1.4 Shoreline Morphologies of the Coastal Settlements

As can be seen in Figure 1.1, the coastline of the Hurunui District comprises of the following range of shoreline morphologies: Mixed Sand and Gravel (MSG) beaches, composite beaches, alluvial cliffs, and mudstone cliffs. Each of these morphologies responds differently to coastal processes, hence have experienced a range of coastal erosion and inundation hazard magnitudes, and will respond differently to SLR effects on erosion, inundation and rising groundwater levels.

A brief description of the morphology of shoreline at each settlement is summarised below:

- *Leithfield Beach:* Settlement is situated on a low coastal plain behind a composite beach with double ridge system comprised of fine gravel sized sediment and a relatively flat sandy lower foreshore. The back ridge is above wave run-up elevations, so there is currently no overtopping of this ridge during extreme coastal storm events. From the shoaling wave break patterns on Google Earth images, it is assumed that the nearshore profile has a similar flat gradient as the sand beach.

- *Amberley Beach*: Settlement is situated on a low coastal plain behind a composite beach with single coarse gravel ridge system, topped up by an artificial gravel bund on the crest with a sandy lower foreshore. Although the artificial gravel bund has raised the beach crest height, wave overtopping can still occur in extreme coastal storm events. From the shoaling wave break patterns on Google Earth images, it is assumed that the nearshore profile has a similar flat gradient as the sand beach.

- *Motunau*: Settlement is situated on uplifted mudstone cliffs dissected by the Motunau River with some of the settlement also located along a low alluvial river terrace. The high (30-40 m) mudstone cliffs are fronted by a low gravel beach at the toe and an intertidal nearshore mudstone shore platform.

- *Gore Bay*: Settlement is situated on a narrow raised coastal plain (approx. 7-10 m above MSL) located behind a beach system that varies between composite and a Mixed Sand and Gravel (MSG) beach state. Back of active beach terminates in low gravel cliff or raised former gravel barrier, so the beach has a limited washover slope. There is a sandy lower foreshore (particularly at south end) with flat gradient across the surf zone from shoaling wave break patterns on Google Earth images suggesting that nearshore profile is similar to a sand beach, hence the morphology is considered to be more composite beach than MSG.

- *Conway Flat:* Covers the Conway Flat Road rather than a settlement, which is situated near the 10 -12 m high alluvial coastal cliff forming the coastal edge of the southern upper terrace of the Conway River. At the toe of the cliff is an MSG beach with a limited washover slope due to the presence of the cliff. A single breaker line on Google Earth images indicates beach profile has steep nearshore face common of MSG beaches.

- *Claverley*: Settlement is situated on a lower coastal plain behind a high MSG beach ridge that has a limited washover slope to hinterland. A single breaker line on Google Earth images indicates beach profile has steep nearshore face common of MSG beaches.

A brief description of the dwelling and population numbers for each settlement, plus the critical infrastructure is provided in the results section for each settlement.

1.5 Groundwater and Coastal Processes

In coastal regions, groundwater generally flows towards the coast (from west to east). Near the coast, there is also an upwards flow of groundwater from deeper layers to the surface. This upwards discharge of deeper groundwater can continue offshore through the sea bed where confined aquifers are present (such as Leithfield Beach and Amberley Beach). There will also be discharge of groundwater from shallower unconfined aquifers. In unconfined coastal aquifers, density differential causes the fresh groundwater to float on the denser seawater. Near the coast the thickness of fresh water overlying saline water becomes negligible so the average level can reasonably be approximated by mean sea level.

The position of the saline interface is approximated by the Ghyben-Herzberg relationship. Assuming a specific gravity of 1 for fresh water and 1.025 for saline water (equivalent to 25,000 mg/L total dissolved solids), the saline interface theoretically occurs at a depth below mean sea level that is 40 times the height of fresh water above sea level.

The conceptual model of groundwater near the coast in Figure 1.2 illustrates this process.

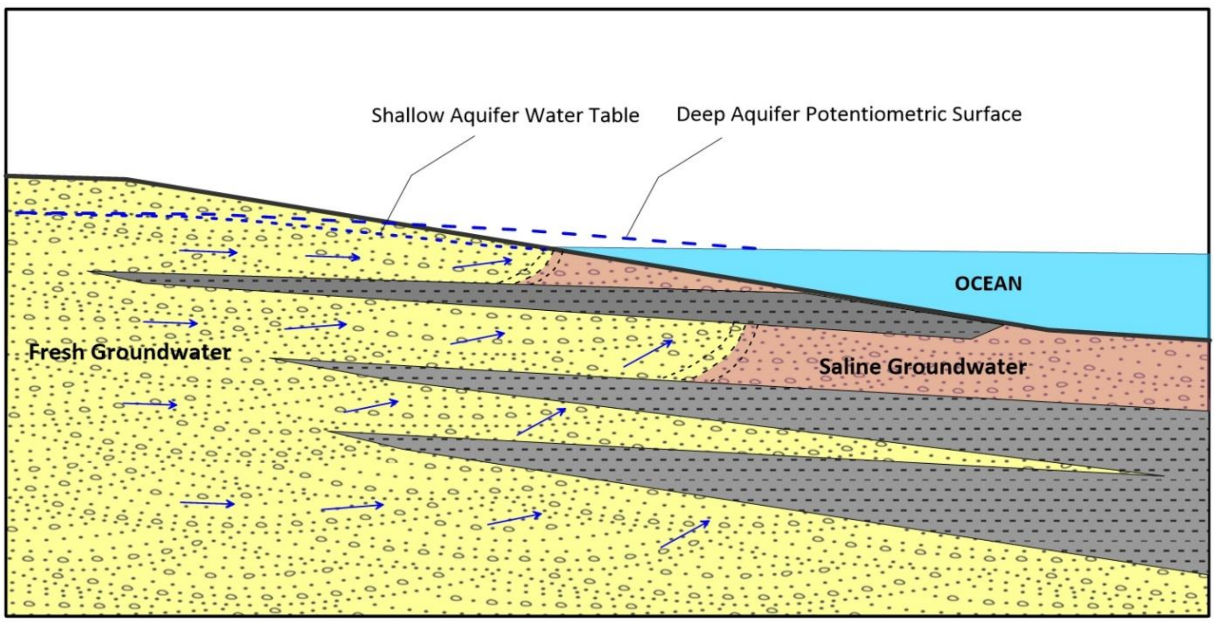

Figure 1.2: Conceptual model of saline interface (after PDP, 2011)

At a ratio of 1:40, the depth of the saline interface is highly sensitive to small changes in groundwater level. Groundwater in unconfined aquifers also responds to fluctuations in sea level, such as from the tides and storm surges. The influence of these shorter-term variations diminishes with distance inland depending on the aquifer properties; in an unconfined aquifer the influence may extend up to several hundred metres, whereas in a confined aquifer it may extend up to several kilometres.

Long-term changes in sea level will also result in long-term changes in groundwater levels. Similarly, to tidal, and seasonal variations, the extent of the influence of SLR into coastal aquifers depends on many factors including topography, recharge and abstraction. In addition to the long-term raising of sea level, climate change will also exacerbate extreme climate-associated events, including storm surges and wave runup and overwash (Hoover et al, 2017). These event impacts will be superimposed upon SLR increases in groundwater levels and salinity. Unconfined coastal aquifers with low hydraulic gradients are theoretically most sensitive to SLR, but the impacts of storm surges could also be significant near to the coast.

As shown in Figure 1.2, because of the densities of sea and fresh water, a saline wedge occurs below fresh water discharging from the land. The interface between fresh and saline water changes with tides and seasons, as well as with longer-term variations in climate (SLR and rainfall discharge) and freshwater abstraction. However, in reality the interface will be a diffuse transition zone from the fresh groundwater to saline groundwater. As sea level rises, the interface between saline and fresh water will migrate landwards causing increased saline intrusion and changes to natural habitats and impacts on exposed assets.

1.5.1 Groundwater Levels and Climate Change

How groundwater levels may vary with climate change is not well documented although focused research is now being undertaken (Taylor et al., 2013). There are three key interacting physical processes relevant to how groundwater will vary in the Hurunui coastal area with climate change:

1. The effective recharge of varying climate and rainfall patterns

Groundwater recharge varies spatially according to changes in land cover, irrigation, soils and climate. Recharge is strongly influenced by climate variability, including El Nino/Southern Oscillation and Pacific Decadal Oscillation. However, wetter conditions do not always produce greater recharge (Green et al., 2011). Typically, long duration extreme rainfall is understood as necessary for groundwater recharge, but some studies have revealed that high recharge can be experienced in some geologies from shorter duration more intense rainfall. Green *et al.* (2011) reviewed several studies which demonstrated both increases and decreases in recharge are possible with climate change. None of these studies had immediate applicability to the hydrogeology of the Hurunui area. It has, however, been predicted that decreases in winter rainfall on the east coast may provide less groundwater recharge.

2. Sea level rise (SLR)

Rising sea level will raise the groundwater surface as it slopes down from the plains to the sea. As above, the aquifer properties will determine the magnitude and inland extent of this influence. PDP (2011) present two possible scenarios where recharge and abstraction remain constant, these being;

1. A constant flux scenario where the same hydraulic gradient is re-established at a higher level, and
2. A constant head scenario where there is a point inland where the freshwater head remains fixed and freshwater discharge decreases as sea level rises.

3. Abstraction and land use

The literature emphasises that alongside climate-induced variations in groundwater level (and quality), anthropogenic changes will have a significant impact (e.g. abstraction for potable water or artificial drainage of groundwater). Whilst not directly a result of climate change, both abstraction and land use are likely to change as the Hurunui district adapts to the impact of climate change. Land use will greatly influence recharge and Taylor *et al.* (2013) highlights studies which have predicted that impacts of abstraction from coastal aquifers are likely to dominate over SLR on changes in groundwater level and salinity.

2. Methodology

2.1 Literature Review

Relevant literature from HDC council records and previous consulting reports undertaken by Environment Canterbury and DTec Consulting were reviewed for information regarding historical erosion and flooding events, both inland and coastal, to inform further understanding of coastal hazards within the local area and provide information to 'ground-truth' predictive mapping. A list of the relevant literature reviewed is included in Appendix A.

The following national guidance and case study documents pertaining to managing coastal hazards were also reviewed for relevance and consideration for this assessment. These documents included:

- New Zealand Coastal Policy Statement (2010).
- Ramsay, D. L. et al. (2012) Defining coastal hazard zones and setback lines. A guide to good practice.
- Wright, J. (2015) Preparing New Zealand for rising seas: Certainty and Uncertainty.
- Ministry for the Environment. (2017) Coastal Hazards and Climate Change: Guidance for Local Government.

2.2 Data Collation

Data was collated from various sources to inform the GIS modelling as well as ground truthing the model results. Relevant Environment Canterbury databases collated for use to inform modelling included:

- Historical erosion database
- Beach profile monitoring database
- Coastal storm database
- Aerial imagery database
- LiDAR imagery database
- Groundwater level data from 21 wells in the vicinity of Leithfield Beach and Amberley Beach.

2.2.1 LiDAR Imagery

LiDAR imagery was obtained from Environment Canterbury and used for inundation bathtub modelling and groundwater modelling. Due to the locations of the settlements being spaced apart along the Hurunui coastline, several LiDAR datasets were required in order to provide full coverage at each settlement. The following datasets were acquired and used to cover the six settlements:

- 165 FPFA1204 Motunau (2018)
- 166 FPFA1204 Pegasus Bay (2018)
- 128 FPFA1033 Kaikoura (2012)
- 151 Kaikoura LiDAR (2017)

2.2.2 Aerial Imagery

The latest aerial imagery (2018-2019) was obtained from Environment Canterbury. Historical imagery was obtained from LINZ online data service (2004, 2015) and Retrolens (1950-2000). Details of the specific aerial imagery used for each settlement is presented in Table 2.1.

Table 2.1: Aerial imagery used in determination of long-term historical shoreline movements at each settlement.

	Leithfield Beach	Amberley Beach	Motunau	Gore Bay*	Conway Flat	Claverley
Aerial Imagery Dates	11/11/2018	27/01/2019	14/11/2018	09/01/2015	09/01/2015	27/01/2019
	05/12/2000	04/03/2004	04/03/2004	04/03/2004	04/03/2004	11/02/2004
	06/03/1974	14/11/1986	14/11/1986	14/11/1985	29/09/1975	28/01/1985
	24/03/1956	20/08/1967	19/09/1968	25/02/1979	25/08/1950	20/04/1966
		10/10/1959	10/10/1950	03/11/1965		25/08/1950
				07/10/1955		
DSAS transects	223-257	173-213	141-168	82-130	25-77	1-21
	*Environment Canterbury produced report in 2011 *Aerial photo analysis of the Gore Bay coastline, 1955-2004*. The lines from this analysis were obtained and used for this study with the addition of 2015 imagery (latest).					

2.2.3 Water Level Data

The groundwater level data available from 21 wells on the coastal plain in the vicinity of Leithfield Beach and Amberley Beach are summarised in Table 2.2. The length of these records range from 1 year to 52 years, with measurements at individual wells ranging from 14 readings to 1592 readings.

2.2.4 Critical Assets

The nature and identification of the critical assets were discussed with Council at the project kick-off meeting, where several assets were identified to be included in the risk assessment. These critical assets included:

- Leithfield Beach community water supply bore
- Five wet wells (Motunau (2), Amberley Beach (2), Leithfield Beach (1))
- Wastewater treatment ponds (Motunau, Amberley Beach)
- Roads
  - Golf Links Road (Amberley Beach)
  - Conway Flat Road (Conway Flat)
  - Claverley Road (Claverley)
- Dwellings in each settlement.

Spatial information for the listed critical assets was supplied by Council (water supply bore, wet wells and waste water treatment ponds). Dwelling footprints were obtained from a building footprint layer from LINZ data service, where it was assumed there was one dwelling per property and sheds/garages were removed from the layer in ArcGIS.

Table 2.2: Leithfield Beach and Amberley Beach Water Level Data Summary

Bore ID	Easting (m)	Northing (m)	Min mon date	Max mon date	Years of data	Number of measurements	Groundwater Elevation (m above mean sea level (MSL))				
							Mean	**Min**	**50th percentile**	**85th percentile**	**Max**
BW24/0260	1578005	5220629	17 Jun 2015	05 Sep 2019	4.22	245	28.30	27.90	28.30	28.40	29.00
M34/0052	1577867	5222152	20 Nov 1962	15 Oct 1986	23.90	114	44.10	42.50	44.00	44.80	45.90
M34/0094	1574543	5225017	29 Aug 1966	05 Sep 2019	53.02	205	67.70	65.40	68.00	69.50	70.50
M34/0095	1577103	5226085	28 Feb 1966	05 Sep 2019	53.52	317	61.30	60.40	61.10	62.00	63.60
M34/0096	1579176	5221772	24 May 1967	15 Oct 1986	19.39	72	29.37	25.21	30.19	31.80	32.54
M34/0140	1578197	5217774	02 Jun 1964	15 Oct 1986	22.37	80	15.90	14.20	16.30	16.70	17.50
M34/0141	1576397	5216874	02 Jun 1964	12 Feb 1981	16.70	56	28.44	25.12	28.66	30.82	31.36
M34/0144	1573598	5216874	02 Jun 1964	15 Oct 1986	22.37	106	59.17	55.14	58.81	60.18	64.15
M34/0153	1578463	5216763	23 May 1966	29 Aug 1985	19.27	86	9.35	7.24	9.10	11.02	12.15
M34/0155	1575345	5214532	24 May 1967	05 Sep 2019	52.28	297	20.50	18.00	20.60	21.00	22.90
M34/0165	1576226	5213157	16 May 1978	05 Sep 2019	41.31	152	6.00	3.10	5.30	9.20	10.50
M34/0178	1572295	5215965	20 Sep 1977	05 Sep 2019	41.96	187	65.30	63.80	65.30	65.60	66.20
M34/0497	1578006	5220626	11 Jul 1990	05 Apr 2018	27.73	1592	28.20	27.60	28.30	28.40	29.50
M34/0798	1579788	5215394	06 Feb 2008	18 Dec 2008	0.87	13	2.88	2.49	2.85	3.19	3.24
M34/5611	1578139	5217933	16 Dec 2007	05 Sep 2019	11.72	140	15.40	13.40	15.70	16.10	16.40
M34/5813	1576253	5222501	29 Nov 2007	05 Sep 2019	11.77	140	51.70	51.10	51.60	52.10	52.80
N34/046	1581573	5220373	10 Jun 1968	15 Oct 1986	18.35	67	3.83	3.20	3.85	4.20	4.47
N34/0146	1582335	5221373	24 Dec 1999	01 Mar 2004	4.19	241	5.90	5.10	5.90	6.20	7.50
N34/0147	1582196	5221173	01 Mar 2004	24 Aug 2006	2.48	139	1.00	0.00	0.80	1.60	2.00
N34/0364	1582152	5220134	22 Nov 2007	18 Dec 2008	1.07	14	0.78	0.39	0.79	0.93	1.37
N34/0365	1581367	5220124	22 Nov 2007	18 Dec 2008	1.07	19	1.46	0.98	1.36	1.75	2.36

2.3 Coastal Erosion Hazard Assessment

A deterministic coastal erosion assessment considering historical long-term erosion, current short-term storm erosion, and estimated future erosion from projected sea level rise was applied to produce a 'Projected Further Shoreline Position' (PFSP) for 30, 50 and 100-year scenarios.

These components were combined in the following formula to calculate the position of the PFSP. This approach meets the requirements of NZCPS Policy 24 for the identification of coastal hazards:

PFSP = (LT X T) + SL + ST

Where:

PFSP = the Projected Further Shoreline Position;

T = Timeframe considered. For this assessment, 30, 50 and 100-year timeframes were selected to correspond to infrastructure management, building code, and land-use planning timeframes respectively;

LT =Extrapolation of the rate of historical long-term shoreline movement (m/yr);

SL =Estimated erosion due to accelerated sea level rise (SLR) over time frame (T); and

ST = Storm term storm erosion.

The following sections outline the methodology applied for calculating each of the erosion components of the PFSP equation.

2.3.1 Extrapolation of Long-term Historical Shoreline Movements

Long-term historical shoreline movements were determined using the GIS based tool DSAS (Digital Shoreline Analysis System). Shorelines were digitised from orthorectified and georeferenced aerial imagery sourced from Environment Canterbury and Retrolens, captured on the dates presented in Table 2.1 for each settlement. As shown in this table between four to six imagery dates were used for each settlement spanning 50-60 years.

The position of all the digitised historical shorelines are plotted on the most recent aerial imagery in Appendix B.

For the majority of the gravel beach environments, the vegetation line was considered to be the most appropriate shoreline reference position for determining long-term change as it responds to both erosion and accretion phases of beach movement and is recognisable on the majority of the imagery. However, for Amberley Beach this was not appropriate due to the anthropogenic change in the beach environment with the construction and continual renourishment of a 1km gravel bund to protect the settlement from inundation and erosion, which impacted presence and movements of the beach vegetation line. Therefore, the wetted line from wave run-up was used as the proxy for shoreline change at Amberley Beach. This feature was identified on the aerial imagery as being where there was a change in the shade of grey on the foreshore due to saturation from wave runup processes. Caution must be exercised when using this feature as it can be significantly affected by the tidal cycle and variability of wave run up due to wave conditions and foreshore slope on the day that the image was captured.

For Motunau and Conway Flat, the cliff top edge was used to identified movement where possible. This feature was used as it is the distance from assets (e.g. dwellings and infrastructure) to the cliff top that is of relevance for planning purposes. However, on some images the ability to determine the cliff edge position was limited due to shadow from the cliff obscuring the position of the edge. When the cliff edge could not be identified, the cliff toe (intersection of the cliff and the back of the beach deposit) was used as a proxy, which for comparative purposes was required to be used at that location across all historical imagery for that section of shoreline. Caution is required in determined the cliff toe position due to potential difficulty in some circumstances differentiating between cliff fall material (e.g. talus slopes) and the beach material.

The GIS based tool DSAS (Digital Shoreline Analysis System) was used to calculate net change between the digitised shoreline and the linear regression rates of shoreline movements in all the aerial images over the 50-60 years of coverage at each settlement. Calculations were made at 50m spaced transects, with the transect numbers at each settlement being presented in Table 2.3, and their locations being presented on the recent aerial imagery in Appendix B.

Table 2.3: DSAS transects at each settlement.

	Leithfield Beach	Amberley Beach	Motunau	Gore Bay*	Conway Flat	Claverley
No. of Transects	35	41	28	49	53	22
Transects IDs	223-257	173-213	141-168	82-130	25-77	1-21

Transects that were located across or in near proximity to river mouths were removed from the analysis as they were considered likely to not be an accurate depiction of the long-term shoreline change due to coastal processes acting in that geomorphic environment. Transects which covered areas where there was too much uncertainty in the location of the reference feature (e.g. due to over exposure of the imagery or low image resolution) were also removed from the analysis.

The R^2[1] value of the Linear Regression Rate (LRR) of shoreline movement from the DSAS output was assessed to determine if it was appropriate to present the long-term rate of movement as the LRR. Transects with R^2 values less than 0.3 indicated that the LRR was not a good approximation of the temporal trend of shoreline movement at the transect. These non-conforming transects were analysed based on their location and context within the settlements coastline, resulting in either of the following actions:

- Where these transects were spaced sporadically amongst other transects, they were taken out of the data analysis.
- Where there were areas with several of these transects in sequence, a further analysis about the change in rate relationship was undertaken to determine what the most appropriate long-term rate was, including whether the End Point Rate (EPR) from the DSAS was more appropriate.

The rates of shoreline movements from the DSAS analysis are shown in the transect colour scales in Appendix B.

The DSAS results were validated by comparing the shoreline change measured by DSAS over the most recent time period (e.g. 2000 or 2004 to most recent imagery) with the surveyed shoreline change from Environment Canterbury beach profiles over the same period. This validation was restricted to individual DSAS transects in close proximity to the beach profile locations, and surveyed change was determined using the same feature as used as the reference shoreline for the DSAS (e.g. vegetation line, contour, crest, bund front). The comparison of methods was calculating the difference between the change in survey profile and the shoreline change calculated from the nearest DSAS transect. A tolerance of ±5m difference in position change of the shoreline feature between the two methods was considered to be acceptable. Results and comments of the DSAS validation can be found in Appendix C.

To remove localised discontinuities in the position from PFSP in the extrapolation of the historical long-term rate of shoreline movement from point source anomalies at individual transects, the raw DSAS results at each

[1] Pearson Correlation Co-efficient

transect are smoothed by applying a moving average calculated from the average of 5 transects either side of the specified transect.

2.3.2 Erosion Impacts of Accelerated Sea Level Rise

MfE (2017) presents four SLR scenarios, developed and adapted for New Zealand conditions based on the scenarios presented in IPCC (2014). Details of the IPCC (2014) and MfE (2017) SLR scenarios are presented in Appendix D.

For this assessment, the RCP8.5 and RCP8.5+ SLR projections from MfE (2017) were used with the following adjustments to make them more appropriate for the assessment of effects of accelerated SLR from current sea levels.

- Offset by -0.05 m to account for SLR that has occurred since 1995, the mid date of the IPCC (2014) baseline for assessing SLR
- Offset the predicted rise by the contemporary rate of rise (e.g. 2 mm/yr) as this is already accounted for in the extrapolation of historical shoreline change

The resulting SLR projections used in this assessment from a 2020 baseline are presented in Table 2.4.

Table 2.4: SLR scenarios used in this assessment

	RCP8.5 SLR Scenario		RCP8.5+ SLR Scenario	
Year	SLR from 2020 Baseline	Rate of accelerated rise	SLR from 2020 Baseline	Rate of accelerated rise
2050 (30 Year)	+0.23 m	5.7 mm/yr	+0.32 m	8.7 mm/yr
2070 (50 Year)	+0.40 m	6.0 mm/yr	+0.56 m	9.2 mm/yr
2120 (100 Year)	+1.01 m	8.1 mm/yr	+1.31 m	11.1 mm/yr

Geometric shoreline retreat models were used to provide order of magnitude estimates of shoreline retreat with the above accelerated SLR. All of the geometric prediction models have limitations around the assumptions applied and the uncertainty of the data required to be inputted into the models. However, their benefits are that they provide a practical method for obtaining a rapid semi-quantitative assessment of the likely order of magnitude of shoreline response to sea level rise.

Full details of the models considered for each of the shoreline morphologies found in the Hurunui District and the sensitivity testing for the different models are included in Appendix D, with the selected models summarised below:

For Composite Beaches (Leithfield Beach, Amberley Beach, Gore Bay)

Modification to the original Bruun Rule involving multiplying the Bruun rule result by the average percentage of sand found across the beach profile (obtained from past sampling by Environment Canterbury at multiple sites across the profile e.g. upper berm, mid foreshore and swash zone) at each settlement. This modification slowed the rate of retreat from the original Bruun Rule formula to account for how much gravel was present on the beach, which responds slower to SLR.

The modified retreat formula applied to Composite Beaches was:

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

Where:

L = Horizontal distance to closure depth from dune crest

s = sea level rise over the planning timeframe

h = height of beach crest above MSL

d = Average closure depth below MSL

For Mixed Sand and Gravel (MSG) Beaches (Claverley)

Geometric models involving rollover processes and volumes for gravel beaches were found to be not applicable for the MGS beaches in the Hurunui District, therefore a modification to the closure depth from the original Bruun rule was applied to these beaches as the sediment transport processes indicate that this will be in the vicinity of the toe of the steep nearshore face found on these beach types. The modification involves applying an assumed toe of the nearshore face located at a depth of 5 m below MSL, and a nearshore slope of 1:10 to the Bruun rule calculations based on the results of the 1987 nearshore surveys at Washdyke, Timaru.

The assumption from the modification is that sediment will still be lost offshore due to profile adjustment with SLR, but as a result of applying a shallower closure depth, there is a corresponding steepening of the closure slope, and hence a reduction in the estimated erosion distances with SLR from these predicted by the original Bruun Rule using the storm wave determination of closure depth.

The resulting modified retreat formula applied to MSG Beaches was:

$$\boldsymbol{Bruun_{MSG}} = \frac{\boldsymbol{L \times a}}{(\boldsymbol{h + dt})}$$

Where:

L = Horizontal distance to closure depth from dune crest

s = sea level rise over the planning timeframe

h = height of beach crest above MSL

dt = Closure depth below MSL defined as the toe of the steep nearshore face

For Mudstone Cliffs (Motunau)

Walkden and Dickson (2008) used sensitivity testing of the SCAPE (Soft Cliff And Platform Erosion) model (Hall & Walkden, 2005) to examine the influence of different beach volumes, erosive forces, sea level rises on the development of equilibrium cliff retreat rates over long time periods (e.g. decadal to centuries). The results of this analysis were that for beach volumes below 30 m^3/m (e.g. the cliff retreat does not contribute significant sediment to the beach) there was a relationship between increase in cliff retreat rates and the ratio of rate of future SLR to the current rate of rise.

Since the mudstone cliffs at Motunau fit the criteria for this model (i.e. are soft sediments and beach volumes < 30 m^3/m), it is considered that it is appropriate to apply this relationship at Motunau. The relationship is expressed by the following equation:

$$\boldsymbol{LT_F} = \boldsymbol{LT_H} \times \left(\frac{\boldsymbol{S_F}}{\boldsymbol{S_H}}\right)^{\boldsymbol{m}}$$

Where:

LT_F = Future cliff retreat rate,

LT_H = Long term historical cliff retreat rate (e.g. DSAS results)

S_F = Future rate of SLR

S_H = Historical rate of SLR (taken as 0.002 m/yr)

m = negative/damped feedback system for influence of beach/platform in front of the cliff face. Based on the results of Walkden and Dickson (2008) a value of *m* = 0.5 is applied.

By reorganising this equation, the increase in erosion rate due to SLR can be expressed as

$$\boldsymbol{LT_{F(SLR)} = LT_H \times \left(\frac{S_F}{S_H}\right)^{0.5} - LT_H}$$

For Alluvial Cliffs (Conway Flat)

Since erosion of alluvial cliffs contributes significant sediment to the beach at the base of the cliffs, the volumes are well above the 30 m^3/m threshold for applying the above relationship. Although Ashton et al (2011) expanded the analysis of Walkden and Dickson (2008) looking at generic changes in the feedback power relationship (i.e. *m value*) for other types of cliff geology and strength (e.g. rock, alluvial glacial outwash terrace), it was still with the assumption of low beach volumes which does not affect the evolution of the cliff-beach/platform profile and the cliff does not contribute significant beach building sediment. To overcome this limitation, and to provide a consistent approach across the whole Canterbury region for the assessment of the effects of SLR on cliff retreat rates, sensitivity testing was carried out for all cliffed sections of the Canterbury Coast. The details of this analysis are outlined in Appendix D (section D.4).

Based on the results of the sensitivity analysis the following modifications to the above Walkden and Dickson (2008) relationship are made for beach volume effect on the future retreat of alluvial cliffs at Conway Flat due to SLR:

$$\boldsymbol{LT_{F(SLR)} = LT_H \times Vol_{effect} \times \left(\frac{S_F}{S_H}\right)^{0.5} - LT_H}$$

Where:

$LT_{F(SLR)}$ = Future cliff retreat rate due to SLR

LT_H = Long term historical cliff retreat rate (e.g. DSAS results)

Vol_{effect} = 0.65 for Canterbury alluvial cliffs

S_F = Future rate of SLR

S_H = Historical rate of SLR (taken as 0.002 m/yr)

2.3.3 Short-term Storm Erosion

The inclusion of short-term erosion as a component in the PFSP calculation is to account for an extreme storm event or series of events resulting in significant erosion close to or soon after the end of the planning timeframe. The impact of these events occurring within the planning timeframe can be accounted for within the extrapolation of long-term rates, but if they occur at the end of the timeframe the impact within the long-term rates may not be accounted for.

For the determination of this parameter the beach profiles from the Environment Canterbury survey database were analysed for the maximum inter-survey change (i.e. generally a year) at selected contours (e.g. 3.5m contour) over the total length of the profile record. The profiles used at each settlement are presented in

Appendix E. To confirm that the maximum inter-survey change was a result of storm events and not a survey anomaly, their occurrence was checked against the Environment Canterbury coastal storm register, which records wave events since May 1999 with significant wave height greater that 4m at the deep-water wave buoy off Banks Peninsula.

The resulting short-term erosion component was applied to all nearby DSAS transects with similar coastal morphologies for inclusion in the PFSP.

It was also proposed to use XBeach-G modelling software for the calculation of short-term storm effects using a 1% AEP storm water level and wave input from Stephens et al (2015). However, a satisfactory output could not be obtained, therefore the results of the maximum erosion change from the profile data were used to inform the PFSP.

2.3.4 Mapping of PFSP

Mapping of the PFPS was undertaken in ArcGIS, with the most recent shoreline from the DSAS assessment (e.g. 2015, 2018, 2019 depending on settlement) being used as the baseline for the mapping. A moving average was applied to the long-term historical rate along an average of five transects (e.g. 250m) either side of the specified transect. For each DSAS transect, along with the calculated historical rate for the transect, the relevant short-term erosion and SLR erosion components were attributed to the transect for each timeframe and corresponding RCP scenario. The sum of these three parameters was then used to transpose the base shoreline in varying distances either landward or seaward from the base shoreline. Maps of the PFPS are presented in Appendix F and components for each shoreline transect is presented in Appendix G.

2.4 Coastal Inundation Hazard Assessment

A "Bathtub" model approach was used across the six settlements to identify the potential extent of inundation from a 1% AEP coastal storm, and consequently the inundation risk to infrastructure and properties in each settlement due to an event of this magnitude. The SLR scenarios applied in the modelling were present day levels, 30, 50 and 100-year SLR under the RCP 8.5 and RCP8.5+ scenarios.

This modelling was undertaken in GIS with the output being identified areas of land which are below the design static water level for each scenario, and therefore infer that the land would be inundated to a certain depth. This method produces conservative results as it does not account for temporal variances of the event, or any hydrodynamic factors such as water moving across a surface. Therefore, for settlements which show significant inundation in the bathtub model, it is recommended that further hydrodynamic modelling is undertaken.

LiDAR data of the Hurunui District for use in the modelling was acquired from Environment Canterbury for each settlement at 1m resolution. The DEM tiles were mosaicked together at each settlement to form a continuous surface in New Zealand Transverse Mercator projection (NZTM2000), and Lyttleton Vertical Datum (LVD1937).

2.4.1 1% AEP Static Water Level

The static water level used in this analysis was made up of the joint probability 1% AEP level from combined storm tide (e.g. astronomical tide, storm surge and wind set-up), wave set-up (super elevation of water level close to the shore with wave breaking processes), and the SLR component. This static water level was calculated using the Canterbury Coastal Calculator (Stephens et al, 2015).

It is important to note that the 1% AEP calculated for this assessment is for present day sea levels and wave climate, and the magnitude of this event has not been increased over the various time periods to account for climate change. For future levels this present day 1% AEP level is added to the SLR for each timeframe and RCP scenario. The static water levels applied for each settlement over various time frames are detailed in Table 2.5. The 30-year RCP 8.5+ scenario was not modelled due to their only being a 30mm difference between the 30 year 8.5 and 8.5+ scenario.

Table 2.5: 1% AEP event static water levels used in this study.

Year	RCP	Leithfield Beach	Amberley Beach	Motunau	Gore Bay (south)	Gore Bay (north)	Conway Flat	Claverley
Present Day		3.51 m	2.84 m	3.82 m	3.26 m	3.41 m	2.95 m	2.95 m
30 year (2050)	8.5	3.79 m	3.12 m	4.1 m	3.54 m	3.69 m	3.23 m	3.23 m
50 year (2070)	8.5	3.9 6m	3.29 m	4.27 m	3.71 m	3.86 m	3.40 m	3.40 m
	8.5+	4.12 m	3.45 m	4.43 m	3.87 m	4.02 m	3.56 m	3.56 m
100 year (2120)	8.5	4.57 m	3.90 m	4.88 m	4.32 m	4.47 m	4.01 m	4.01 m
	8.5+	4.87 m	4.20 m	5.18 m	4.62 m	4.77 m	4.31 m	4.31 m

2.4.2 Inundation Mapping in ArcGIS

The extent and depth of the static water inundation under the bathtub approach was calculated and mapped in ArcGIS. For all settlements hydraulic connection with the ocean was assumed to occur at the waterways present within the settlement boundaries, and at beach frontages where the beach ridge was below the modelled static water level for that SLR scenario. Once having established that there was a hydraulic connection to the ocean within a settlement, all areas below the scenario static water levels were mapped, hence some small isolated areas without hydraulic connection are also included in the maps. The static water levels used in the modelling are those presented in Table 2.5 above, with the inundation extents being presented as water depth above the ground surface. All modelling and mapping presentation are in LVD1937 elevation datum. The inundation maps are presented in Appendix H.

2.4.3 Additional Wave Run-up Considerations

The modelling also included consideration of the effect beach overtopping from wave run-up (maximum vertical extent of wave "up-rush" on a beach above storm-tide level) on inundation extents and depths. It is known that for beaches where wave overtopping occurs, this process can result in inundation in areas where the beach ridge is above the static water level and add significantly to the inundation volumes for areas with lower beach ridges.

While the Canterbury Coastal Calculator (Stephens et al, 2015) includes the calculation of the wave run-up elevation, it was considered that in a bathtub modelling approach the simple additional of this elevation to the static storm tide water level would result in an extreme over prediction of inundation extent and depths, as does not accounting for the for the pulsing of wave run-up overtopping with wave period or the time limitations of overtopping due to tidal cycles. To overcome these limitations, the run-up contribution to the inundation was calculated using the overtopping volume calculator in the Canterbury Coastal Calculator (Stephens et al., 2015) with the static 1% AEP water level used in the calculations varied on an hourly basis over a twelve-hour high tide cycle to calculate the total overtop volume over the beach ridge. No consideration of crest lowering in response to the overtopping or for events lasting longer that one tidal cycle was made in the calculations.

The following iterative steps were used for the mapping of additional inundation extent from wave run-up overtopping volume:

1. The additional water volume was spread over the area of static water inundation to test whether the increased inundation level would exceed the next 0.5 m increment of ground elevation;

2. If so, the addition inundation area to the next 0.5 m increment of ground elevation was calculated and step one was repeated for the new enlarged area.

3. However, if the increased inundation level did not exceed the next 0.5 m increment of ground elevation, the iterative process was stopped and then proceeded to step 4.

4. The additional inundation area was shown in the Appendix H maps as a solid light-yellow area to indicate potential extent of inundation if run-up was included. For areas where ground levels decrease away from the coast (e.g. Amberley Beach) is was considered that the influence of wave run-up on inundation extent would be arbitrarily limited to 200 m from the shoreline. Inundation depths are not included in these additional areas as the 0.5 m elevation increments used in the calculations are coarser than the 0.2 m depth increments applied to the static water inundation areas.

Note that no run-up effects were calculated for Motunau, as the lower river terrace inside the river mouth was not considered to be exposed to run-up processes. For Conway Flat the run-up calculations apply to the mouths of the small creeks present in this study area rather than the alluvial cliffs.

2.5 Rising Groundwater Hazard Assessment

2.5.1 Overview

There are a number of possible approaches to understanding how groundwater characteristics could vary with climate change. Rainfall patterns will affect groundwater recharge and thus the landward elevation of groundwater, whereas SLR (and rise of downstream river levels) will raise coastal groundwater. Both of these already occur seasonally and during significantly wet and/or stormy periods, but long-term changes will occur in a changing climate.

In terms of our ability to predict future impacts of climate change:

1) Changes in groundwater level are more readily quantified than those of salinity;
2) Change due to SLR is more readily predicted than those from varying recharge;
3) Groundwater abstractions significantly influence groundwater levels and salinity but cannot readily be predicted into the future with any certainty and are therefore assumed to continue as they currently are.

Therefore, for this regional-scale coastal area assessment, our method has focused on predicting permanent changes in the groundwater level with sea level rise, assuming no change in rainfall recharge and groundwater abstraction.

2.5.2 Water Level Data Availability

From the available water level data, it was hoped that the 85th percentile (higher) water level would be able to be determined at each location and water level elevation contours plotted to inform head conditions for the inland model boundary. This would provide a consistent approach as used for similar assessments in Christchurch City and Waimakariri District. However, there are limitations to the availability of groundwater data in the coastal margins of the Hurunui District, with the only relevant well data being that summarised in Table 2.2. Due to these limitations there was no real statistical justification to approximating 85th percentile levels, hence the assessments carried out in this study using an indicative average level for informing areas of potential risk due to groundwater level rise. Should a more detailed assessment be required then detailed site specific investigation and data would be required.

The following summaries the availability of the data for each of the study areas.

Leithfield Beach and Amberley Beach

Leithfield Beach and Amberley Beach had most data available, although accurate water level elevation data near the coast and consistent time-series water level data was lacking. Plots of available data in the vicinity of the coastal settlements are provided on Figure 2.1 and Figure 2.2, with the water levels being plotted as elevation above mean sea level (MSL) and calculated from the recorded ground elevations at the well head and ground elevations derived from LiDAR survey. It is noted that in addition to the data presented, some individual water levels for bores and number of discrete measurements were also available.

Key commentary with respect to data availability and reliability is provided as follows:

- There are some considerable differences in water levels based on ground elevations derived from the bore data vs ground elevations derived from LiDAR – N34/0046 (Amberley Beach) is an extreme example with the current ground surface likely to have been altered by quarrying, however the original data also appears too low – possibly influenced by quarrying and there is a definite pump on/pump off fluctuation. Ground elevation discrepancies are typically in the order of 1 to 2m.
- There is a considerable lack of continuous groundwater monitoring and recent groundwater monitoring. Bore N34/0046 (Amberley Beach) and M34/0153 (Leithfield Beach) have almost 20 years of data, however the most recent measurements are over 30 years old. The most recent water level data is over 10 years old and of relatively short duration.
- There are also discrepancies in data continuity, N34/0146 and N34/0147 (Amberley Beach) are only 240m apart but show significant differences in water level elevations. N34/0146 is outside of the model domain but the inconsistent levels still bring the reliability of the data from either bore into question. This is also an area of prior quarrying so the reduced levels at N34/0147 could be due to dewatering.

In light of the above, there is insufficient data available to map 85th percentile water levels at Leithfield Beach and Amberley Beach with any degree of confidence.

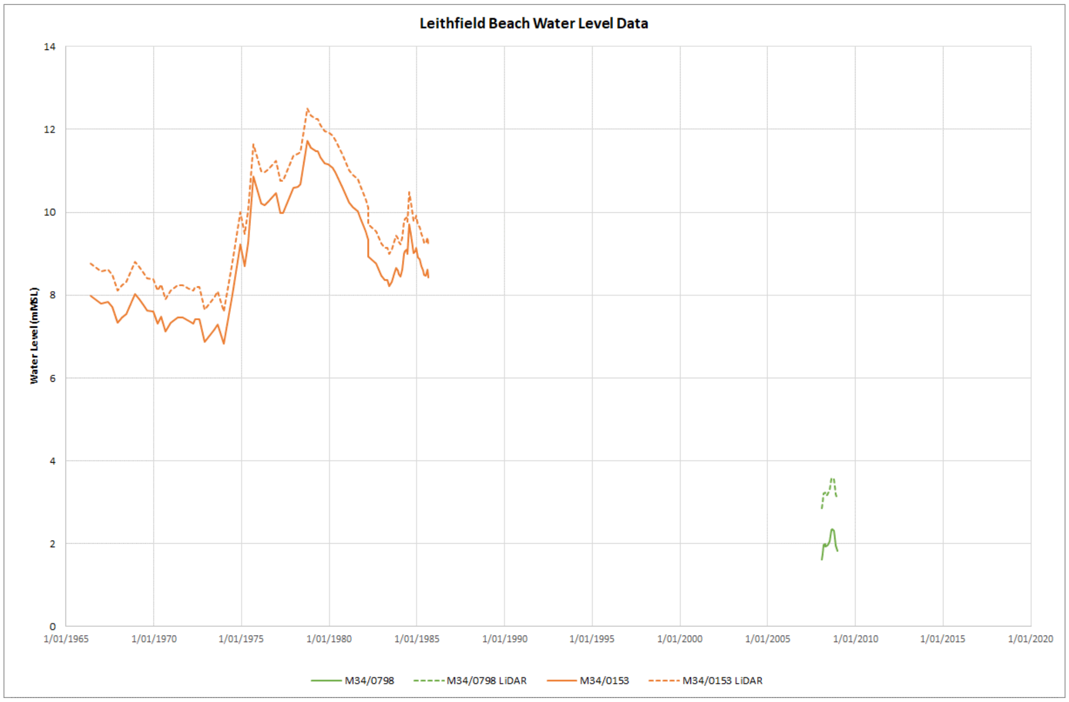

Figure 2.1: Leithfield Beach Water Level data

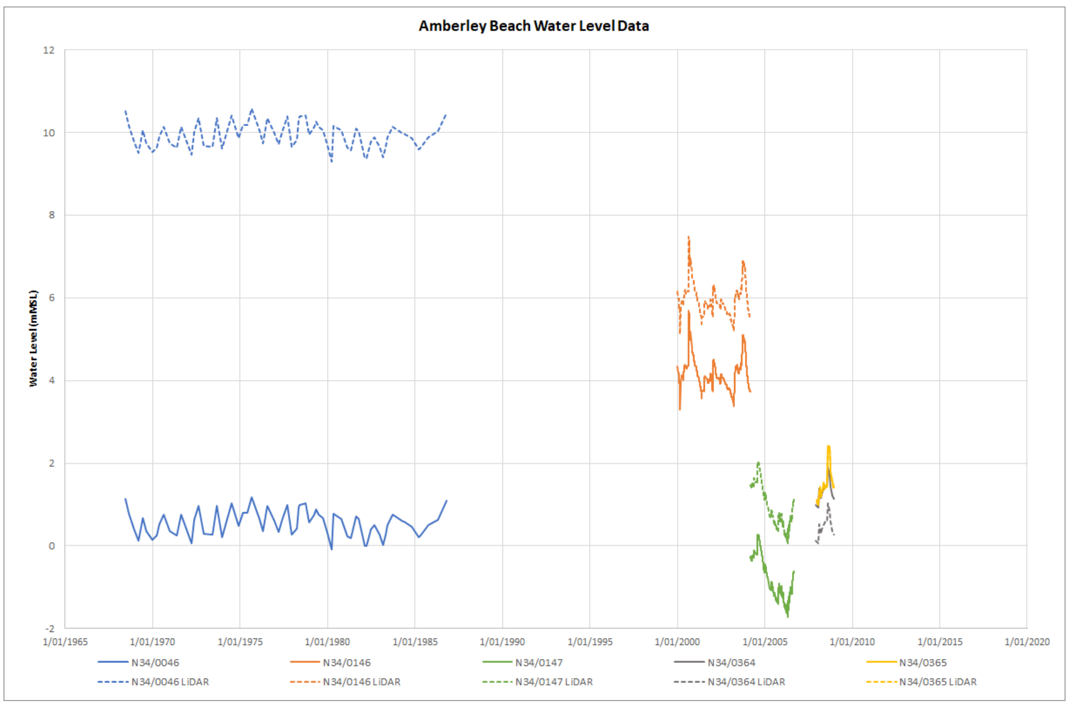

Figure 2.2: Amberley Beach Water Level Data

Conway Flat and Claverley

Both Conway Flat and Claverley have limited available data. Five bores had limited or individual water levels that were used to inform model water levels.

Motunau and Gore Bay

Motunau and Gore Bay had little to no available water level data. Water levels from wells inland were used to inform inland boundary conditions.

2.5.3 Groundwater Level Rise

In general groundwater levels for the current scenario at each location were modelled using the analytic element groundwater modelling software AnAqSim (Fitts, 2017). AnAqSim employs the analytic element method (AEM), which superposes analytic solutions to yield a composite solution consisting of equations for head and discharge as functions of location and time. AnAqSim uses subdomains as described in Fitts (2010), which gives it strong capabilities with respect to heterogeneity and anisotropy. It also employs high-order line elements, spatially-variable area sinks, and finite-difference time steps to allow multi-level aquifer systems and wide-ranging transient flow simulations.

The AEM is fundamentally different from numerical methods like finite element and finite difference (such as FeFlow and ModFlow), where the domain is broken into small blocks or elements and simple head distributions (e.g. linear) are assumed within these blocks or elements. In the AEM, boundaries of the domain are discretized, but the domain itself is not, and as such is infinitely scalable.

Available water level monitoring data (Leithfield Beach and Amberley Beach) was used to inform inland model boundary conditions. Surface water bodies, such as rivers, drains and lagoons, were used to constrain unconfined aquifer water levels, with elevations derived from LINZ (2019) LiDAR data, and the 0m elevation contour from LiDAR data was used to inform the current sea level constant head boundary at the coast.

Where possible, additional water level data was also employed to refine model parameterisation and ensure simulated groundwater levels were consistent with observed data.

Once an acceptable current scenario model was established for each of the study areas, future SLR under the RCP8.5+ scenarios (MfE, 2017) were then simulated. For the modelling scenarios, the forecast SLR were rounded to 0.6 m over a 50-year period (e.g. 2070) and 1.3 m over a 100-year period (e.g. 2120). Given that the differences in forecast SLR under the more conservative RCP8.5 scenario were of the order of 0.16 and 0.3m respectively for the 50-year and 100-year time periods, these were not modelled as the differences were considered to be less than the accuracy allowed by the applied methodology and data availability.

The 1.3m elevation contour derived from LiDAR data was adopted for the coastal boundary of the future 1.3m SLR scenario with constant head boundaries set at 1.3m MSL to replicate the elevated sea level. For the 0.6m SLR scenario, coastal boundaries were retained as for the current scenario, however the constant head boundary increased to 0.6m MSL. Surface water bodies at the coast, such as lagoons and tidal river reaches, were also assumed to rise with sea level.

Given that sea level rise is expected to be a relatively slow process, groundwater equilibration is expected to keep pace with that rate of change. As such, current and future groundwater conditions have been simulated as steady state conditions, representative of long-term average or equilibrium conditions.

Water table elevations for the current and future scenarios at each of the study areas were extracted and are presented and discussed in the following sections.

2.5.4 Saline Interface

The location of the saltwater interface has also been simulated in AnAqSim based on the Ghyben-Herzberg equation. This approach assumes density equilibrium between the fresh and salt water and a sharp interface within a homogeneous aquifer. In reality the interface would be represented by a transitional mixing zone from fresh to saline water and would also be influenced by groundwater flow and aquifer heterogeneity.

The location of the saline interface and the predicted water tables for the current and future scenarios, relative to ground surface are presented in cross section for each of the study areas.

2.6 Risk Assessment and Mapping

A basic risk assessment was undertaken to calculate the number of dwellings and properties which intersected with the respective hazard footprints (inundation footprint excluded wave run-up). Basic statistics including average depth across the settlement, number of properties intersected and number of dwellings intersected were recorded for each settlement to help identify which of the settlements would be most at risk from SLR.

The risk to critical infrastructure identified by the Hurunui District Council, which included roads, waste water treatment ponds and wet wells, was also assessed. A table of statistics including number of dwellings/properties affected and the intersection of the hazard with critical infrastructure is presented in each settlement section.

2.6.1 Dwellings and properties

Coastal erosion risk was assessed using only property boundaries obtained from the LINZ data service. Any property intersecting with the PFSP was included in the risk statistics, regardless of the size of the intersection.

Coastal inundation risk was identified using both property boundaries and dwelling footprints. Each property was filtered to have only one dwelling per property, e.g. sheds and garages were removed from the property. This was based on the assumption there is only one dwelling per property. Dwellings were only assessed if they were located inside the settlement footprint as outlined in the Hurunui District Plan. Conway Flat did not have a settlement footprint as per the District Plan, and therefore no dwellings were assessed for this location.

No floor level data was available for the dwellings located at the settlements, and therefore risk of inundation was identified based on whether the dwelling footprint intersected the flood extent.

For rising groundwater, property boundaries were used to identify risk, with properties overlying groundwater that could rise to within 0.5 m of the ground surface being assessed as being at high risk.

2.6.2 Wet wells and Water Supply Bores

Five wet wells (two in Amberley, one in Leithfield, two in Motunau) and one water supply bore (Leithfield) were identified by HDC to be assessed for the potential risk from coastal inundation and erosion. It is understood that any coastal inundation at these structures would cause issues for these structures as the heads are at ground level, therefore inundation would cause infiltration of saltwater into the structure which would impact the performance of the asset.

For the wet wells, if the modelling indicated that future coastal inundation is possible, then the risk assessment also considered the impacts of saline water on the operation of the sewage ponds. Based on the estimate of increasing groundwater salinity, indications of both the salinity impacts on the water quality from the Leithfield water supply bore and whether underground assets could be susceptible to increased corrosion were also considered in the assessment.

2.6.3 Roads

Coastal erosion and inundation risk on key identified roads at Conway Flat, Calverley and Amberley were identified using the road polygons in ArcGIS with the percentage of the road that would be affected by the hazard footprint being reported. An estimated average inundation depth along the structure was also reported to provide an indication of whether the road could be used (i.e. water depths below 0.3m) during an event as an evacuation route.

3. Leithfield Beach

3.1 Settlement Description

Leithfield Beach is at the northern end of an extensive prograded coastal plain along the fringes of Pegasus Bay. The plain is at its widest (6km) where it joins Bank Peninsula, and tapers to less than 1 km wide in the vicinity of Amberley Beach (Blake 1968). Leithfield Beach is towards the northern end of an extensive prograded coastal plain along the fringes of Pegasus Bay running from Banks Peninsula north to Amberley Beach. The Leithfield Beach settlement, located 10km south east of Amberley township, has a population of 402[2] residents and contains 265 dwellings within the footprint, inclusive of some buildings in the Holiday Park which could be occupied temporarily or permanently. The critical infrastructure of interest to Council at this settlement are two wet wells and the water supply bore as shown in Figure 3.1. Another piece of key infrastructure which may be affected by coastal erosion is the piped outfall to the land drainage channel that runs through the settlement as shown in Figure 3.2. This outfall drains a catchment area of some 550 ha draining springs and rainfall runoff from the low-lying flat land between the Old North Road at Leithfield village and the Leithfield Beach.

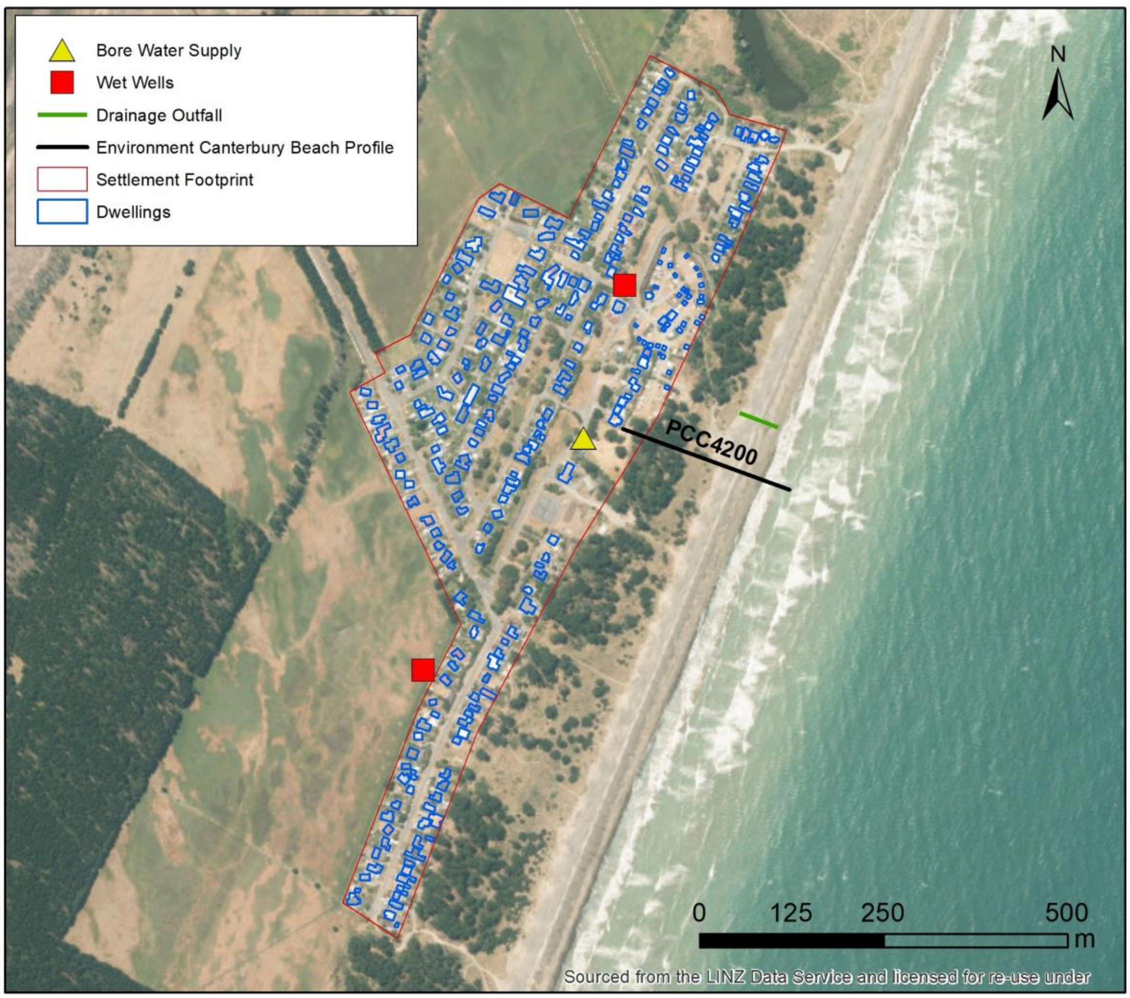

Figure 3.1: Leithfield Beach settlement footprint and critical infrastructure.

[2] Taken from the New Zealand Census Data (2013) as provided by HDC.

The coastal frontage to the settlement is 1.5 km long, with the settlement footprint being separated from the shoreline by a 200m wide series of vegetated backshore ridges, which are up to 6m above MSL in elevation. The beach state varies between MSG and Composite beach depending on sea conditions, however for this assessment is assessed as being a composite beach comprised of gravel upper beach ridges and relatively flat sandy lower foreshore and nearshore profile. To the north of the settlement is a coastal lagoon located behind the beach which is not naturally open to the ocean, however, the beach topography indicates that wave overtopping occurs that does enter the lagoon.

Figure 3.2: Leithfield Beach drainage outfall pipe in centre of the settlement; (a) the Ocean outfall; (b) pipe inlet at the back of the beach ridge.

3.2 Coastal Erosion Hazard Assessment

3.2.1 Historical Long-term Shoreline Movements

The historical shoreline analysis for Leithfield Beach was covered by DSAS transects 223 (north) to 257 (south), with a map of the results being presented in Appendix B.

The overall long-term historical trend along this section of coastline is for shoreline advance over the 60 years covered by the analysis. Accretion rates decrease in a northward direction along the settlement frontage, with the southern 500 m (Transects 246-257) having average rates of +1.4 m/yr, decreasing in the central and northern parts of the settlement (Transects 245-229) to average rates of +0.6 m/yr. A similar accretionary historical shoreline trend was also found in the Waimakariri District, with accretion rates decreasing in a northward direction to +1 m/yr at Waikuku immediately south of the Hurunui District (Jacobs, 2018). This accretionary trend is considered to be due to the northward transport from sediment supplied by the Waimakariri River, supplemented by the supply from the Ashley River for the Leithfield area.

As shown by Figure 3.3, accretion rates have also been decreasing over time, with rates since 1974 being considerably less than those in the 1956-1974 period. For northern transects, there has actually been net retreat since 1974, but 2018 shoreline positions are still seaward of 1956 positions. The reason for this reduction in accretion rate is not clear, as there is no indication in reduction of sediment supply from the rivers or in northerly longshore transport within these time periods. Hicks et al (2018) has also assessed that the sediment supply from the Waimakariri River will most likely remain similar to current conditions or slightly enhanced with climate change, and longshore transports within southern Pegasus Bay should remain similar to present.

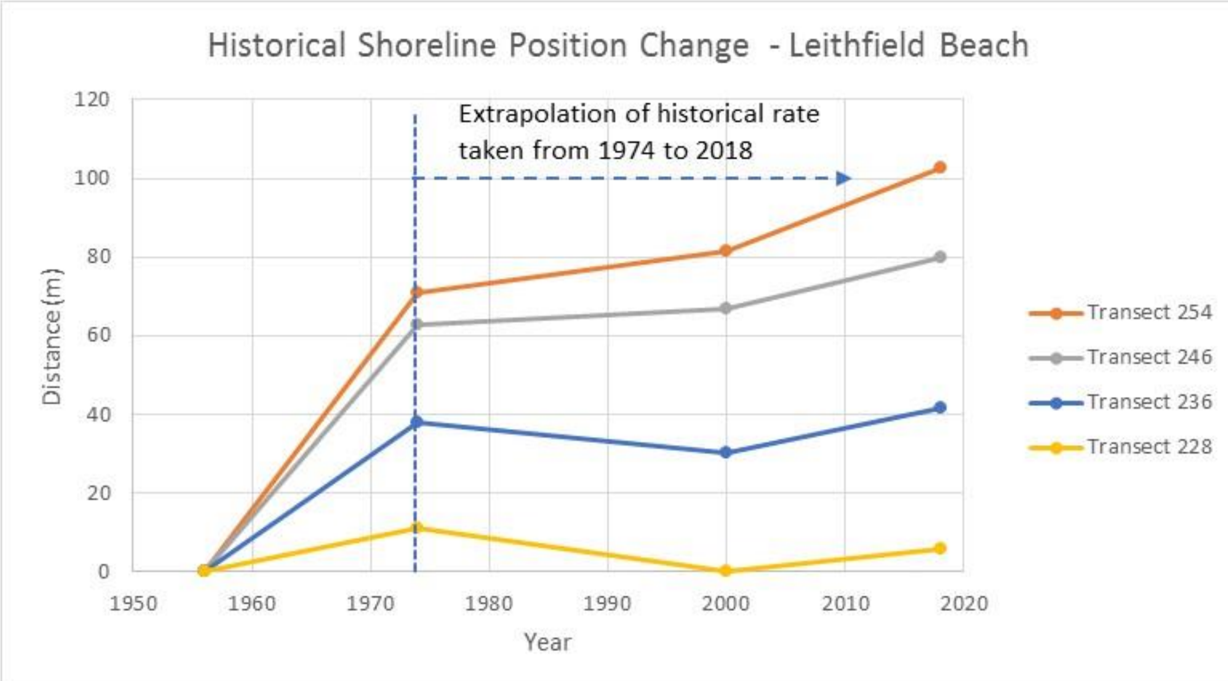

Figure 3.3: Historical shoreline position change for selected DSAS Transects at Leithfield Beach 1956-2018.

As a conservative approach to the extrapolation of historical rates for input into the determination of the PFSP position, only shoreline advance rates since 1974 have been used. The resulting projected shoreline advance distances from extrapolating these rates 30, 50 and 100 years into the future are presented in Table 3.1.

Table 3.1: Projected shoreline advances from extrapolation of rates from 1974 to 2018 for selected DSAS Transects at Leithfield Beach

Scenario	30 years (2050)	50 years (2070)	100 years (2120)
Transect 254 (South end of settlement)	+18.8 m	+ 31.3 m	+62.6 m
Transect 246	+9.6 m	+16.0 m	+31.9 m
Transect 236	+6.7 m	+11.1 m	+22.3 m
Transect 228 (north end of settlement)	+2.5 m	+4.1 m	+8.3 m

3.2.2 Accelerated Sea Level Rise Effects

The effects of projected accelerated SLR on coastal erosion was calculated at Leithfield Beach using the modified Bruun Rule for composite beaches as set out in section 2.3.2 and Appendix D. Beach crest height and sediment size data was taken from Environment Canterbury profile PCC4200, which showed that the crest elevation was 5.7 m (above MSL) and that 84.5% of the beach sediment was sand. Closure depth was calculated to be -8.8 m (below MSL) at a distance of 1900 m from the shore. The resulting shoreline retreat due to accelerated SLR under the over 30, 50 and 100-year timeframes under the RCP 8.5 and RCP8.5+ SLR scenarios are presented in Table 3.2.

It is noticeable from the differences in erosion distances for each RCP scenario that the erosion distance is very sensitive to the magnitude of SLR. For example, a 0.1 m additional rise under scenario 8.5+ by 2050 adds around 10 m to the erosion distance, an addition 0.15 m rise by 2070 adds around 20 m, and an addition 0.3 m rise by 2120 adds around 35 m.

Table 3.2: Calculated erosion distance due to accelerated SLR at Leithfield Beach (ECan profile PCC4200).

Scenario	30 years (2050)		50 years (2070)		100 years (2120)	
	SLR	Erosion Dist	SLR	Erosion Dist	SLR	Erosion Dist
RCP8.5	+0.23 m	-18 m	+0.40 m	-32 m	+1.01 m	-89 m
RCP8.5+	+0.32 m	-28 m	+0.56 m	-51 m	+1.31 m	-124 m

3.2.3 Short-term Storm Effects

As shown in Table 3.3, Environment Canterbury beach profiles between 1991-2019 revealed that the maximum inter-survey erosion ranged from -6.8 m for the beach crest (Nov 1991- May 1992) to -5.7 m for the beach toe (May 1992-Sept 1992). Based on these survey changes and applying a conservative approach, an arbitrary value of 7m has been adopted as the short-term erosion component of the PFSP.

Table 3.3: Maximum short-term erosion measured by ECan beach profiles at Leithfield Beach.

Profile	Feature Measured	Maximum Inter-survey Change	Period of Max Change	Storm Notes
PCC4200	Crest	-6.8 m	Nov 1991- May 1992	Pre storm register but from antidotal records 1992 is known to be stormy period with 7 events prior to Sept, of which 3 were recorded to be significant.
	Toe	-5.7 m	May 1992 - Sept 1992	

3.2.4 Projected Future Shoreline Position (PFSP)

From the combination of the above information on the individual components, the resulting distances from the current shoreline to the PFSP under the RCP 8.5 and RCP 8.5+ scenarios at selected transects are presented below in Table 3.4. Full details of the components at all transects are presented in Appendix G and the ground position in relation to the settlement is shown in Appendix F.

Table 3.4: Distances from current shoreline to PFSP at Leithfield Beach. (Distances rounded to nearest metre)

Timeframe	30 years (2050)		50 years (2070)		100 years (2120)	
Scenario	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Transect 254 (South end of settlement)	-6 m	-17 m	-8 m	-27 m	-34 m	-69 m
Transect 246	-16 m	-26 m	-23 m	-42 m	-64 m	-100 m
Transect 236	-19 m	-29 m	-28 m	-47 m	-74 m	-110 m
Transect 228 (north end of settlement)	-23 m	-33 m	-35 m	-54 m	-88 m	-124 m

It is noticeable from these results that for all parts of the settlement coastal frontage, except the southern end under the RCP 8.5 scenario, that within 30 years the erosion due to SLR is predicted to outstrip the advance due to sediment supply, resulting in net erosion occurring. Within 50 years, this also occurs at the southern end under both SLR scenarios, and by 100 years retreat distances are projected to be 35-70 m at the southern end and in the order of 90-125 m at the northern end of the settlement.

3.2.5 Coastal Erosion Risk

As can be seen from the mapping of the PFSP in Appendix F, coastal erosion with SLR is not projected to intersect any of the 197 properties until after 2070. By 2120 under the RCP 8.5 scenario (e.g. SLR≈1 m) the vegetation line is projected to be very close to property boundaries at the northern end of the settlement footprint, which will likely increase their exposure to coastal inundation due to reduced beach width and bulk (assuming the back of beach is fixed to the current position). However, under the RCP 8.5+ scenario (e.g. SLR≈1 3 m) the shoreline is projected to intersect with 14 properties at this northern end of the settlement. A summary of properties exposed to the erosion hazard in over the different timeframes under both SLR scenarios is presented in Table 3.5.

Critical infrastructure of the water supply bore and both wet wells are not predicted to be affected directly by coastal erosion under any of the scenarios. However, it is considered likely that the drain outfall pipe structure shown in Figure 3.2 would be affected to some degree by the projected erosion within the 30 to 50-year timeframes, by undermining of the ocean end with retreating beach profiles, and/or sediment blockage of landward pipe inlet from beach rollover.

Table 3.5: Number of properties affected by coastal erosion in future SLR scenarios at Leithfield Beach.

Timeframe	Total	30 years (2050)		50 years (2070)		100 years (2120)	
Scenario		RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Number of properties	197	0	0	0	0	0	14

3.3 Coastal Inundation Hazard Assessment

3.3.1 Bathtub Model Results

Coastal inundation bathtub model maps for each settlement under current and future SLR scenarios are presented in Appendix H, with the results for Leithfield Beach summarised below in Table 3.6.

Table 3.6: Summary of the spatial extent of potential inundation hazard in Leithfield Beach

Timeframe	Present day (2020)	30-year (2050)	50-years (2070)		100-years (2120)	
Scenario and 1% AEP static water levels(1)	(3.59 m)	RCP 8.5 (3.82 m)	RCP 8.5 (3.99 m)	RCP 8.5+ (4.15 m)	RCP 8.5 (4.60 m)	RCP 8.5+ (4.90 m)
Approx % of settlement below 1% AEP water level	99%	99%	99%	99%	99%	99%
Average Depth	0.5 m	0.8 m	1.2 m	1.4 m	1.8 m	2 m
[1] 1% AEP static water level = Storm Tide (ST) + wave set-up (WS). All water levels are given in terms of Lyttleton Vertical Datum 1937 (LVD)						

As can be seem from the maps and above results, virtually the entire settlement footprint is below the 1% AEP static water level event with current sea levels, therefore using a bathtub mapping approach is shown in the Appendix H maps to be potentially susceptible to coastal inundation. However, as documented in Section 2.4, the bathtub method produces very conservative results as it does not account for temporal variances of the event, or any hydrodynamic factors. Under this modelling approach there would be considerable variation in the inundation depths within the settlement, with an average depth in the order of 0.5 m.

The mapping indicates that only isolated wave run-up overtopping of the double beach ridge system would occur within the settlement frontage, with the source of majority of the inundation being from overtopping of the lowered beach ridge at the coastal lagoon immediately to the north of the settlement footprint.

Due to the entire settlement footprint being below the threshold for inundation by static water level, no additional inundation areas have been mapped for inundation by wave runup overtopping.

Under the future SLR scenarios, the locations and volumes of beach overtopping will increase, resulting in a corresponding increase in the potential inundation depths as sea levels rise. The bathtub modelling approach indicates that average inundation depths in a 1% AEP event could increase to 0.8 m in 30 years' time, in the range of 1.2-1.4 m in 50 years' time, and range of 1.8-2 m in 100-years' time.

3.3.2 Coastal Inundation Risk

Dwellings and Properties

As shown in Table 3.7, for all scenarios, from the present-day scenario though to the 100-year RCP 8.5+ scenario, all properties (265) with a dwelling intersect the coastal inundation hazard footprint. Depth of water around the dwellings has not been assessed. Only six properties in the settlement footprint without dwellings are modelled to not intersect with inundation hazard footprint with present day sea levels, with that number reducing to one to two properties under the 100-year SLR scenarios.

Table 3.7: Total number of dwellings and properties which intersect with the inundation hazard footprint

Timeframe	Total	Present day (2020)	30-year (2050)	50-years (2070)		100-years (2120)	
Scenario			RCP 8.5	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Dwellings	265	265	265	265	265	265	265
Properties	197	191	191	192	193	195	196

Critical Infrastructure

As indicated in Table 3.8, inundation will occur around the three pieces of critical infrastructure under all inundation scenarios including present day. The inundation depths are given from ground level at the structure, which assuming the head of the structure is at ground level, indicates that these critical infrastructural units are at risk from coastal inundation under all 1% AEP scenarios. Under SLR this level of risk increases from both increased inundation depths and reduced frequency of extreme events as indicated by Figure 3.4.

Table 3.8: Potential Inundation depth at critical infrastructure in Leithfield Beach from Bathtub modelling of 1% AEP event

Timeframe	Present day (2020)	30-year (2050)	50-years (2070)		100-years (2120)	
Infrastructure		RCP 8.5	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Wet Well North	0.87 m	1.2 m	1.34 m	1.5 m	1.95 m	2.28 m
Wet Well South	0.93 m	1.2 m	1.38 m	1.59 m	2.0 m	2.29 m
Water Supply Bore	0.59 m	0.87 m	1.09 m	1.2 m	1.8 m	2.27 m

3.3.3 Recommended Further Inundation Modelling

Given the results of the bathtub modelling, it is recommended that further hydrodynamic modelling of the inundation hazards at Leithfield Beach **is warranted** to better quantify the threshold for overtopping and inundation, the spatial extent and magnitude (e.g. inundation depths) of the hazard, and risks posed to the dwellings and critical infrastructure.

3.3.4 Change in Annual Recurrence Interval

As well as water levels, future SLR will also increase the annual probability that the present day 1% AEP event will occur. As shown in Figure 3.4, the Annual Recurrence Interval (ARI) for the present day 1% AEP event magnitude (e.g. 3.59 m static water level) reduces from the current 100 years to 40-50 years by 2050, to 15-30 years by 2070, and to 1-3 years by 2120. Expressed another way, this magnitude event is twice as likely to occur in any one year by 2050 under both SLR scenarios and could become an annual occurrence by 2120 under the more extreme RCP8.5+ scenario.

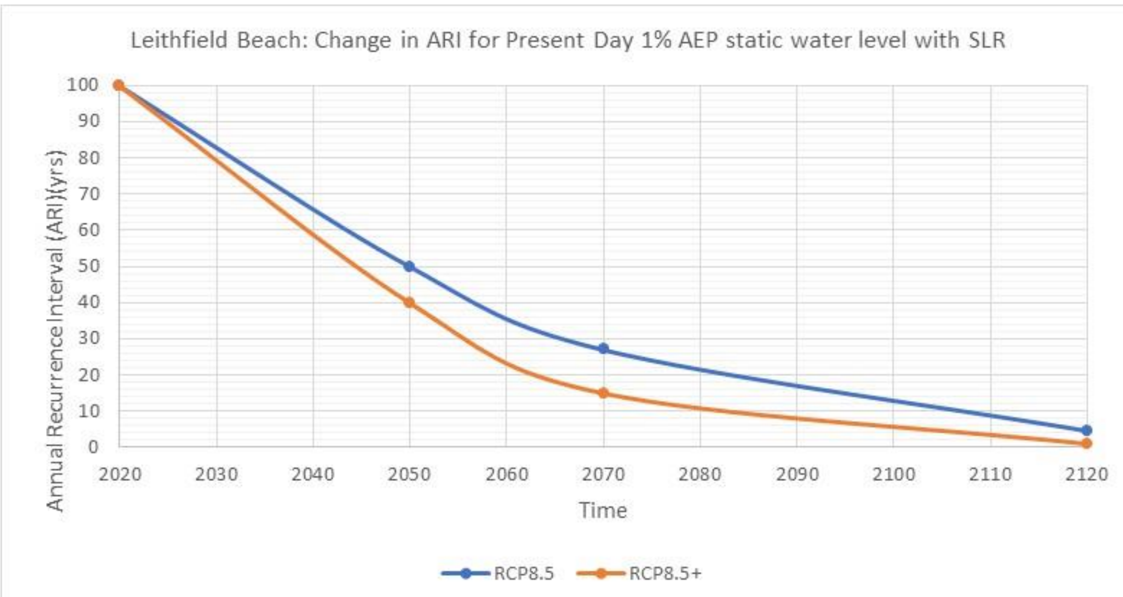

Figure 3.4: Effect of SLR on Annual Recurrence Interval for Present Day 1% AEP static water event for Leithfield Beach.

3.4 Rising Groundwater Hazard Assessment

3.4.1 Existing Groundwater Conditions

At Leithfield Beach and Amberley Beach the geology is dominated by Quaternary alluvial and beach deposits and the northwards continuation of the Christchurch Aquifer System. Although not investigated in any detail, Brown (2001) notes that the groundwater system north of the Ashley River displays a typical Canterbury Plains coastal deposition pattern, with interfingering interglacial marine fine sediment deposits (aquicludes) and glacial period outwash gravels (aquifers). Review of the Canterbury Maps Viewer (https://canterburymaps.govt.nz/) indicates extensive semi-confined or unconfined aquifers associated with the northern Canterbury Plains and in particular, the Kowai River (north and south branches) and Waipara River.

The Environment Canterbury Well Search database (https://ecan.govt.nz/data/well-search) and Canterbury Maps Viewer note numerous wells with water level data in the area. However, around the coastal settlements of Leithfield Beach and Amberley Beach, data is more sparse and recorded bore elevations display significant discrepancy from elevations inferred from LiDAR data.

Groundwater flow is generally to the east towards the coast. Inland from Leithfield Beach and Amberley Beach there is a significant drop in ground elevation from the inland plains area at a fluvial terrace scarp in the order of 10 to 20m in height. This scarp represents a significant control on groundwater elevations with seepage faces typically occurring at the base of the scarp, numerous springs are also mapped along or below the base of the scarp. In the lower lying coastal area the shallow nature of the water table is indicated by numerous drains and wetland areas.

The Environment Canterbury (2016a) report on coastal groundwater discharge in the Waimakariri zone, describes the reduced deposition of fine-grained marine sediments in the northern Waimakariri zone due to longshore currents. This has resulted in a reduced thickness of low permeability confining layers between the glacial alluvial strata, allowing for more groundwater discharge from deeper aquifers via vertical (upwards) seepage. However, as indicated by water supply bore BW24/0051 at Leithfield Beach, confined and artesian groundwater conditions are still present.

Surface water bodies in the area include Leithfield Beach Lagoon (north of Leithfield Beach) and Mimoto Lagoon (south of Amberley Beach).

3.4.2 Leithfield Supply Well

At Leithfield Beach, water supply bore BW24/0051 draws water from an artesian gravel aquifer at a depth of approximately 120 m BGL (screened from 116.7 to 118.7 m BGL). Ground elevation at the well head is recorded as approximately 2.7 m MSL. Water levels at BW24/0051 are recorded in the range 0.3 to 1.1 m above ground level, equivalent to approximately 3.5 to 4.3m MSL.

The Well Search database shows the well was pump tested at a rate of 32 L/s for 3 days. Data is not provided for review or analysis, but comments indicate that the test data required compensation to account for tidal fluctuations. The Council data indicates that from the period September 2019 through early February 2020, BW24/0051 was pumped at an average rate of approximately 2100 kL/day (24.3 L/s).

3.4.3 Adopted Parameters

The following parameters were adopted for incorporation in the AnAqSim groundwater model:

- Quaternary shallow alluvial hydraulic conductivity assumed at 110 m/day; based on Environment Canterbury (2016a).
  - Two local shallow bores (N34/0408 - 9.7m and M34/0321 -38.4 m) have had pumping tests undertaken and provide indicative hydraulic conductivities of 81 and 125 m/day respectively.
  - BW24/0051 at Leithfield Beach has also been tested but is in a confined aquifer screened from 116.7 to 118.7 m. BW24/0051 returned a transmissivity from two tests of the order of 210 to 230 m^2/day, equivalent to hydraulic conductivity of approximately 70 m/day. An aquifer storage coefficient of $1.0x10^{-4}$ was derived.
- Recharge rate –100 mm/year based on work completed by Environment Canterbury (2016b).
- Lagoons and drains simulated as specified head boundaries based on LiDAR elevations.

3.4.4 Rising Groundwater Mapping

A map of depths to the indicative average shallow groundwater conditions at Leithfield Beach under present day sea levels are presented in Appendix I, which indicates that significant areas of existing development and infrastructure at Leithfield Beach are located in areas of shallow groundwater (<1 m BGL).

Depths to indicative average shallow groundwater under future RCP 8.5+ SLR scenarios are also presented in Appendix I, which indicate that due to the low-lying nature of the area and shallow water table, Leithfield Beach is susceptible to future rises in groundwater level with SLR. Under the 50-year scenario, the majority of the settlement is predicted to have average groundwater levels shallower than 1 m BGL, with areas shallower than 0.5 m BGL encroaching on the settlement under the RCP 8.5+ 100yr SLR scenario.

The predicted saline interface with SLR is shown in Figure 3.8, indicating potential significant saline incursion in the unconfined aquifer under the RCP 8.5+ 100yr SLR scenario.

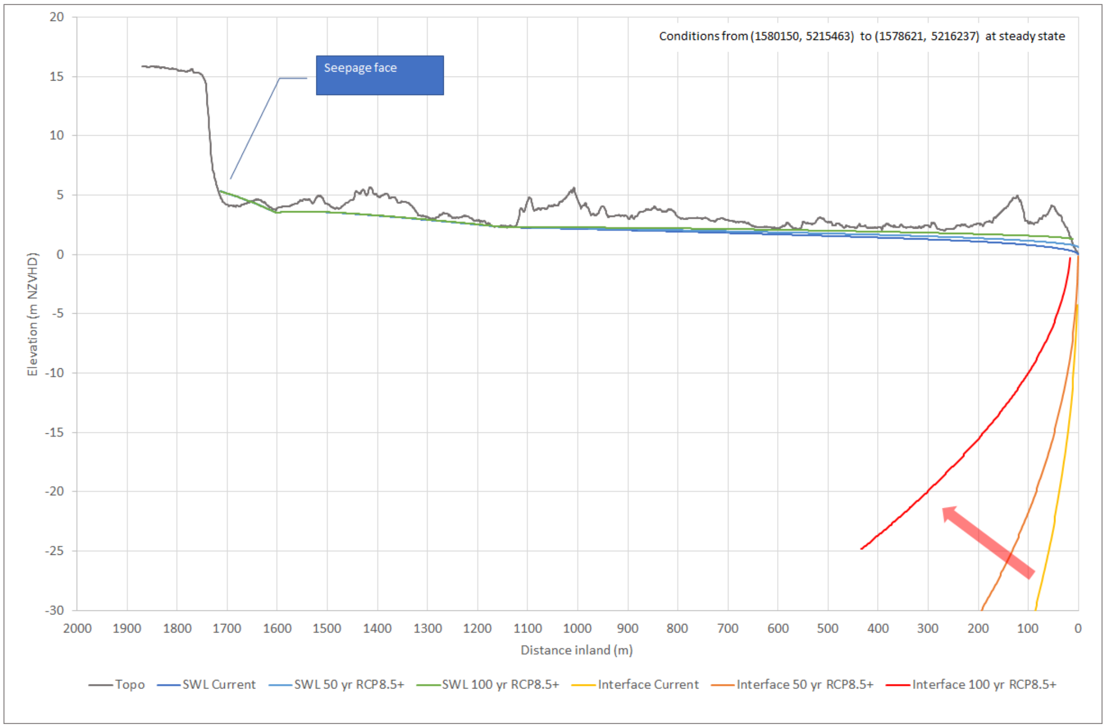

Figure 3.8: Leithfield Beach simulated water levels and saline interface with SLR.

3.4.5 Rising Groundwater Risk

The number of dwellings exposed to different groundwater depths with present and future sea levels is presented in Table 3.9. Note that where a dwelling covers two or more depth categories, the shallowest depth has been applied. The number of dwelling predicted to be at risk from groundwater shallower than 0.5 m increases significantly with SLR particularly for SLR beyond 50 years. At present, only 2% of the total number of dwellings in the settlement are in this high-risk category, increasing only slightly to 6% in 50 years, but increasing to over 40% within 100 years.

Table 3.9: Number of dwellings exposed to indicative average groundwater depths at Leithfield Beach.

RCP 8.5+ SLR Scenario	Depth to Groundwater (m BGL)			
	⩽ 0.5 m	0.5-1 m	1-2 m	> 2 m
Present Sea level (2020)	5	132	128	0
50 year (0.6 m SLR)	16	193	56	0
100 year (1.3 m SLR)	112	130	23	0

Predicted average groundwater depths at the nominated critical infrastructure is presented in Table 3.10. As shown depths to groundwater levels are predicted to decrease from 1 to 2 m BGL with present sea levels, to 0.5 to 1 m BGL and shallower than 0.5 m BGL at the southern wet well under predicted 100-year sea levels.

Table 3.10: Indicative Average Groundwater Depths at Leithfield Beach Critical Infrastructure (m BGL).

Infrastructure	Present day (2020)	100-year RCP 8.5+(SLR=1.3 m)
Wet Well North	1-2 m	0.5-1m
Wet Well South	1-2 m	<0.5m
Water Supply Bore	1-2 m	0.5-1m

3.4.6 Leithfield Water Supply Well

An assessment of potential saline ingress to the Leithfield water supply bore has been undertaken. The utilised aquifer was simulated as a confined aquifer from approximately -100 to -120m MSL.

Environment Canterbury (2016a) notes that in the northern Waimakariri zone the saltwater freshwater interface is assumed to be less than 5 km offshore based on the sub-artesian groundwater levels. Under steady-state conditions the location of the saline interface would notionally start where the potentiometric surface of the confined aquifer reaches 0 m MSL.

To estimate the location of the interface, upwards discharge from the confined aquifer to overlying aquifer and to the ocean was simulated as a head dependent flux off the coast with the distance from the coast varied, by trial and error, until observed heads at BW24/0051 were adequately replicated in the model. The steady state, non-pumping head at BW24/0051 was simulated at approximately 3.9m MSL compared to the 3.5 to 4.3m MSL observed heads.

Scenarios were then run to simulate the RCP 8.5+ SLR scenarios of 1.3 and 1.8m sea level rise with the results presented in Figure 3.9.

For the present-day scenario, pumping was simulated at a number of varying constant discharge rates to find a threshold rate at which continuous pumping was sustainable without saline intrusion or significant up-coning. For this assessment a threshold pumping rate of 20 L/s (1,728 kL/day) has been adopted. Beyond this rate sign of saline up-coning (saline groundwater level rise beneath the well) were indicated in the model.

From Figure 3.9, the position of the saline interface in the confined aquifer is predicted to be approximately 600 m to 1000 m from BW24/0051, hence a position of 340 m to 820 m offshore. Under the future SLR scenarios, the saline interface is predicted to migrate inland by up to 100 m, however remains over 400 m from BW24/0051 under pumping conditions. Under these scenarios, no deterioration of water quality is anticipated resulting from SLR and inland migration of the saline interface.

It is considered that a more detailed assessment of BW24/0051, including review of test pumping data should be undertaken to refine future risk from SLR.

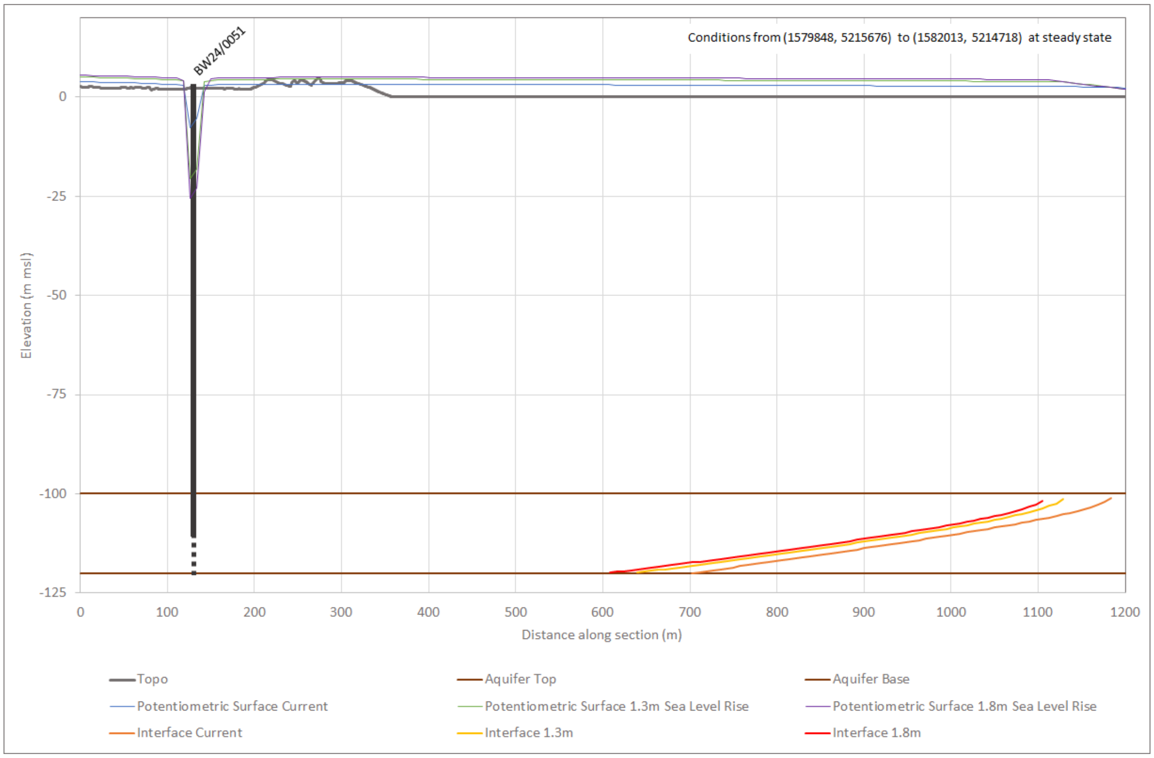

Figure 3.9: Leithfield Beach Well Saline Ingress Assessment.

4. Amberley Beach

4.1 Settlement Description

As shown in Figure 4.1, Amberley Beach is a small settlement with 1 km coastal frontage that contains 108 dwellings and 165 residents[3]. The critical infrastructure of interest to Council are one wet well, the wastewater treatment plant and a section of Golf Links Road behind the beach to the north of the settlement, which is the only access to the Amberley Golf Course.

The settlement is separated from the beach frontage by a narrow 50-70m wide forest area. The beach has been classified as a composite beach with the normal condition being an upper beach gravel storm ridge with a dominantly sand foreshore which the presence of several lines of breakers suggests extents across a shallow gradient nearshore with no distinct nearshore step such as found in a mixed sand and gravel beach environment.

A distinctive feature of the coastal landscape at Amberley Beach is the presence of a man-made bund along the storm ridge of the beach along the whole frontage of the settlement to prevent coastal inundation from beach overtopping (location shown in Figure 4.1). This bund started as a private community protection scheme over a 250 m length to the south of Amberley Beach Rd following overtopping of the low natural storm ridge during a significant coastal storm event in August 1992, resulting in several houses being inundated and the settlement being evacuated by Civil Defence. This was the first time that beach overtopping and inundation had been recorded at Amberley Beach. Following further overtopping and erosion along the reminder of the settlement frontage in a number of storms during 2002, the renourishment was extended along 700 m of the Chamberlain Ave frontage to Golf Links Rd (DTec, 2009). The bund has successfully prevented inundation of the settlement in coastal storm events since this time, however has suffered erosion in significant storm events resulting in nourishment top-ups of the bund in 2009, 2015, and 2018. The condition of the bund in late 2019 is shown in Figure 4.2(a).

To the south and north of the settlement there are small coastal lagoons (Mimimoto Lagoon and unnamed respectively) into which drainage from the small coastal plain discharges, including drains across low laying land immediate west of the settlement. Neither of the lagoons have a permanent opening to the ocean with both having outlet channels normally blocked by beach sediment that allow regress of high lagoon water levels and also the ingress of sea water during coastal storm events. The bridge over the northern lagoon outlet shown in Figure 4.2(b) is part of the Golf Links Rd critical infrastructure.

[3] Taken from the New Zealand Census Data (2013) as provided by HDC.

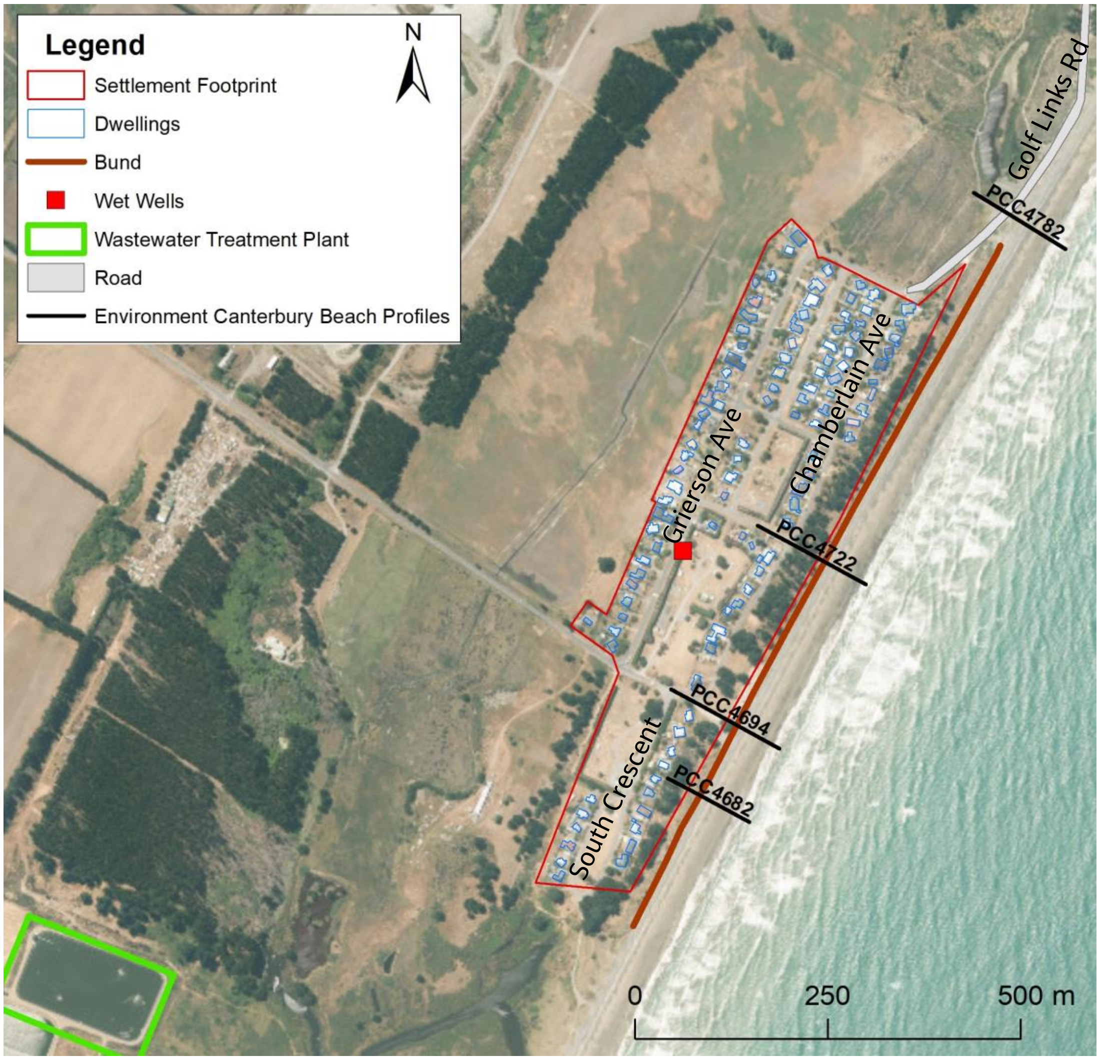

Figure 4.1: Amberley beach settlement footprint and critical infrastructure.

Figure 4.2: (a) 1km Bund along the storm ridge at Amberley Beach (photo July 2019), (b) northern lagoon inlet under the bridge at Golf Links Road, an identified piece of critical infrastructure.

4.2 Coastal Erosion Hazard Assessment

4.2.1 Historical Long-term Shoreline Movements

Amberley Beach is at the northern end of an extensive prograded coastal plain along the fringes of Pegasus Bay tapering to less than 1 km wide in the vicinity of Amberley Beach. This wedge of sand and gravel laid down in the last 6,500 years, indicating net shoreline advance over this geological time period. Over more historical time periods, DTec (2009) combined cadastral maps from 1862, aerial photographs and beach profile surveys to produce the pattern of shown in Figure 4.3 of slowing accretion post 1950 converted to erosion post 1968.

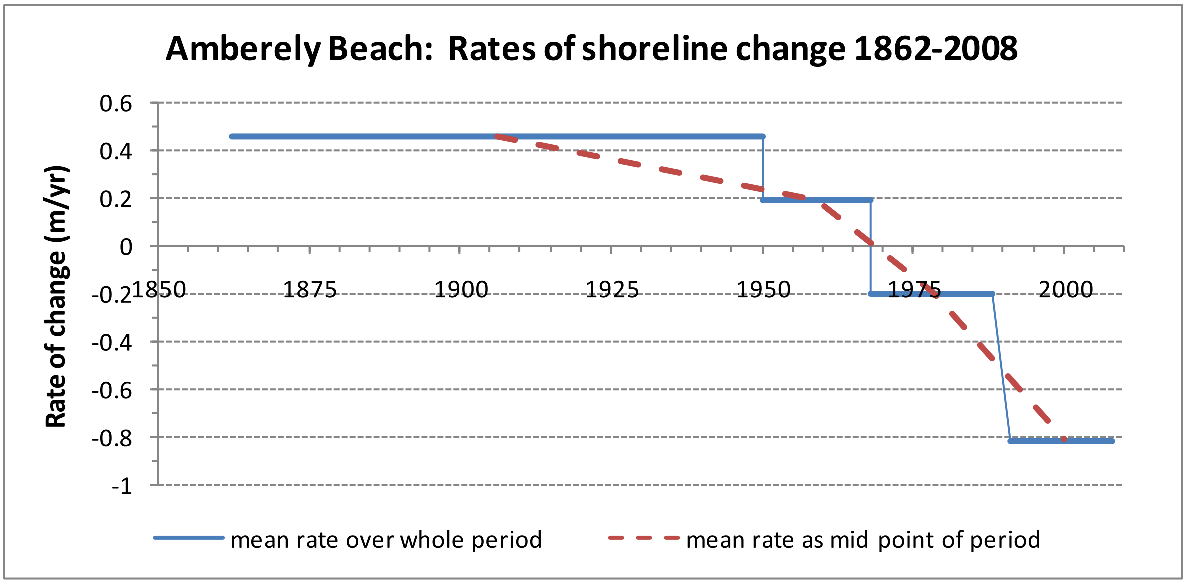

Figure 4.3: Amberley Beach rates of shoreline change 1862- 2008 from DTec (2009)

As set out in Table 2.1, the aerial photography analysis for Amberley Beach under this project covered five sets of imagery over 60 years from October 1959 to January 2019, with the settlement being covered by DSAS transects 173 (north) to 213 (south). A map of the historical shorelines, DSAS transect locations, and resulting erosion rates is presented as part of Appendix B.

As shown in Figure 4.4 for representative transects, the long-term historical trend of shoreline movement from the DSAS were similar as found by DTec (2009) with small scale accretion or stability up to 1986 following by increasing erosion rates over time. Spatially erosion also increased to the north away from sand sediment supply from the Waimakariri and Ashley Rivers (it is assumed that the gravel sediment supply is from the Waipara River via a southerly counter eddy from Double Corner).

In general, historically over the total 60 years of the aerial imagery coverage, the erosion trends at the settlement can be summarised as follows:

- South of the settlement (Transects 206-213): Average rate of -0.32 m/yr since 1959, increasing to -0.48 m/yr since 1986.
- Settlement frontage (Transects 205 to 187): Average rate of -0.35 m/yr since 1959, increasing to -0.69 m/yr since 1986.
- Golf Links Rd (Transects 173-186): Average rate of -0.57 m/yr since 1959, increasing to -0.97 m/yr since 1986.

As a conservative approach to the extrapolation of historical rates for input into the determination of the PFSP position, only shoreline advance rates since 1986 have been used. The resulting projected shoreline advance distances from extrapolating these rates 30, 50 and 100 years into the future are presented in Table 4.1.

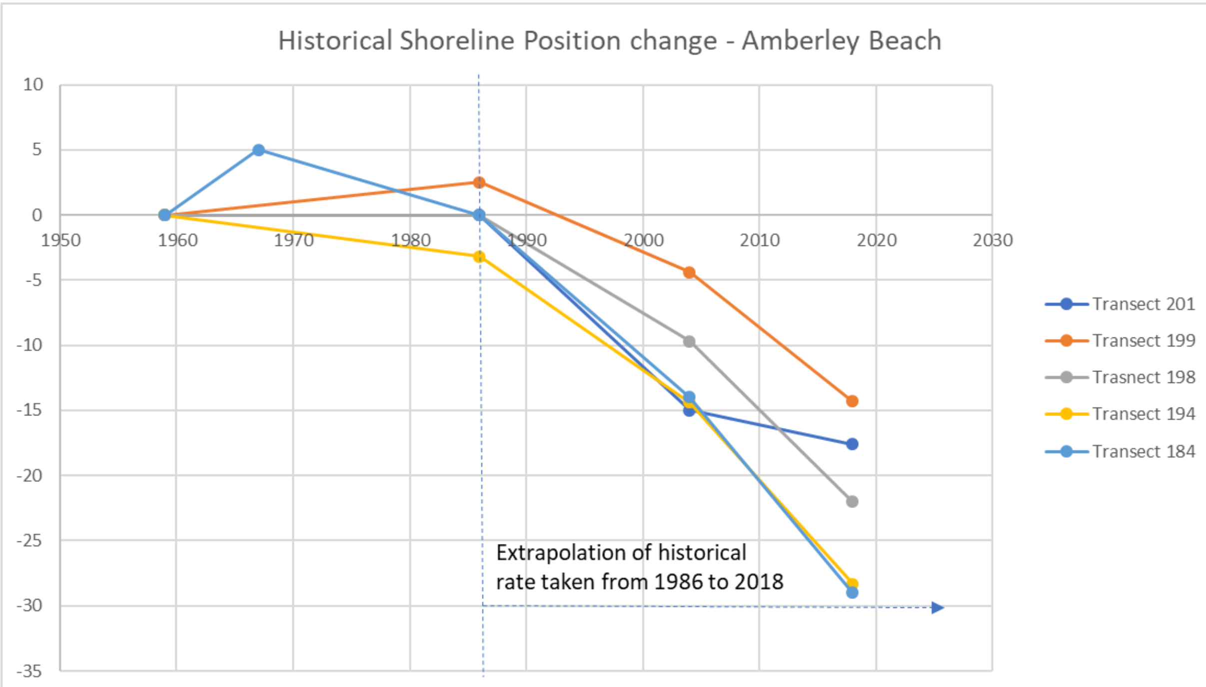

Figure 4.4: Historical shoreline position change for selected DSAS Transects at Amberley Beach 1960-2018.

Table 4.1: Projected shoreline retreat from extrapolation of rates from 1986-2019 for selected DSAS Transects at. Amberley Beach

Scenario	30 years (2050)	50 years (2070)	100 years (2120)
Transect 201 (South Crescent)	-18m	-30m	-60m
Transect 199 (Amberley Beach Rd)	-18m	-30m	-60m
Transect 194 (Chamberlain Ave)	-21m	-35m	-70m
Transect 184 (Golf Links Rd)	-30m	-50m	-100m

4.2.2 Accelerated Sea Level Rise Effects

The effects of projected accelerated SLR on coastal erosion was calculated at Amberley Beach using the modified Bruun Rule for composite beaches as set out in section 2.3.2 and Appendix D. Beach crest height and sediment size data was taken from three Environment Canterbury profiles along the settlement frontage (PCC4694, PCC4722, PCC 4782), which showed that the crest elevation ranged from 4.5 to 5.3 m above MSL and that 29% of the beach sediment was sand. Closure depth was calculated to be -9.3 m (below MSL) at a distance of 1450 m from the shore. The resulting shoreline retreat due to accelerated SLR under the over 30, 50 and 100-year timeframes under the RCP 8.5 and RCP8.5+ SLR scenarios are presented in Table 4.2.

It is noticeable from the differences in erosion distances for each RCP scenario that the erosion distance is not as sensitive to the magnitude of SLR as Leithfield Beach. For example, a 0.1 m additional rise under scenario 8.5+ by 2050 adds around 3 m to the erosion distance, an addition 0.15 m rise by 2070 adds around 5 m, and an addition 0.3 m rise by 2120 adds around 10 m.

Table 4.2: Calculated erosion distance due to accelerated SLR at Amberley Beach.

Profile	Scenario	30 Years (2050)		50 Years (2070)		100 Years (2120)	
		SLR	Erosion Dist	SLR	Erosion Dist	SLR	Erosion Dist
PCC4694	RCP8.5	+0.23m	-5 m	+0.40m	-8 m	+1.01m	-23 m
	RCP8.5+	+0.32m	-7 m	+0.56m	-13 m	+1.31m	-31 m
PCC4722	RCP8.5	+0.23m	-5 m	+0.40m	-8 m	+1.01m	-23 m
	RCP8.5+	+0.32m	-7 m	+0.56m	-13 m	+1.31m	-32 m
PCC4782	RCP8.5	+0.23m	-5 m	+0.40m	-9 m	+1.01m	-24 m
	RCP8.5+	+0.32m	-8 m	+0.56m	-14 m	+1.31m	-33 m

4.2.3 Short-term Storm Effects

As shown in Table 4.3, Environment Canterbury beach profiles between 1991-2019 revealed that the maximum inter-survey erosion ranged from -6.6 m for the front top edge of the bund on the storm ridge (PCC4722, Nov 2007- Nov 2008) to -6.3 m for the front bottom position of the bund (PCC 4694, Nov 2005-Nov 2006, and PCC 4782, Nov 2009-Nov 2010). Based on these survey changes and applying a conservative approach, an arbitrary value of 7 m has been adopted as the short-term erosion component of the PFSP.

Table 4.3: Maximum short-term erosion measured by ECan beach profiles at Amberley Beach.

Profile	Feature Measured	Maximum Inter-survey change	Period of max change	Storm Notes
PCC4782	Bund front bottom	-6.3 m	November 2009 - November 2010	One storm on 8-9 August 2010 recorded on ECan storm register.
	Bund front top	-3.5 m	November 2009 - November 2010	
PCC4722	Bund front bottom	-5.0 m	November 2006 - October 2007	One storm on 24-27 June 2007 recorded on ECan storm register.
	Bund front top	-6.6 m	October 2007 - November 2008	One storm on 30 July – 1 August 2008 recorded on ECan storm register
PCC4694	Bund front bottom	-6.3 m	November 2005 - November 2006	One storm on 21-22 July 2006 recorded on ECan storm register.
	Bund front top	-5.6 m	November 2006 - October 2007	One storm on 24-27 June 2007 recorded on ECan storm register
PCC4682	Bund front bottom	-4.9 m	October 2007 - November 2008	One storm on 30 July – 1 August 2008 recorded on ECan storm register
	Front of storm ridge	-5.3 m	May 1992 – September 1992	Pre storm register but from antidotal records 1992 is known to be stormy period with 7 events prior to Sept, of which 3 were recorded to be significant.

4.2.4 Projected Future Shoreline Position (PFSP)

From the combination of the above information on the individual components, the resulting distances from the current shoreline to the PFSP under the RCP 8.5 and RCP 8.5+ scenarios at selected transects are presented below in Table 4.4. Full details of the components at all transects are presented in Appendix G and the ground position in relation to the settlement is shown in Appendix F.

The lack of sensitivity of the projections to the magnitude of SLR is shown in the close position of the PFSP under the RCP8.5 and RCP8.5+ scenarios at each timeframe. At all timeframes, the extrapolation of long-term erosion will contribute the greatest percentage of the projected erosion, being responsible for 48 -72% over the next 30 years, increasing to 53-76% over a 100-year timeframe. In contrast, the contribution from accelerated SLR to the projected erosion will be 12-27% over the next 30 years, increasing to 18-38% over 100 years.

Table 4.4: Distances from current shoreline to PFSP at Amberley Beach. (Distances rounded to nearest metre).

Timeframe	30-years (2050)		50-years (2070)		100-years (2120)	
Scenario	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Transect 201 (South Crescent)	-30	-32	-45	-50	-89	-98
Transect 199 (Amberley Beach Rd)	-30	-33	-46	-51	-92	-101
Transect 194 (Chamberlain Ave)	-33	-36	-51	-56	-101	-110
Transect 184 (Golf Links Rd)	-40	-43	-63	-68	-126	-135

4.2.5 Coastal Erosion Risk

As can be seen in Appendix F, by 2050 the existing beach bund will be totally eroded and become ineffective as an inundation protection with the storm ridge projected to be located within the current forested backshore. The loss of this bund will occur even without accelerated SLR, and without maintenance could occur within 10-15 years with existing rates of shoreline retreat. Within this timeframe, the shoreline position is not shown to intersect any beach front properties, however, this mapping is of the beach ridge, so it is likely that without intervention rollover processes will result in the landward toe of the beach being within private properties at the south end of South Crescent and along the eastern side of Chamberlain Ave north of Laverys Drive.

Assuming the bund is not maintained, by 2070 there will be up to 15 properties along South Crescent and the eastern side of Chamberlain Ave north of Laverys Drive affected by erosion under both SLR scenarios. Within 100 years the modelling indicates that the PFSP will be located to the west of South Crescent and Chamberlain Ave with 45 (33% of settlement) properties being affected by erosion. A summary of these results is presented below in Table 4.5.

Table 4.5: Number of properties affected by coastal erosion in future SLR scenarios at Amberley Beach.

Timeframe	Total	30-year (2050)		50-year (2070)		100-year (2120)	
Scenario		RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Number of properties	138	0	0	15	15	45	45

Of the critical infrastructure assessed at Amberley Beach, as shown in Appendix F, the wastewater treatment pond and the wet well are modelled to be not affected by erosion within any timeframe. However, the coastal section of Golf Links Rd including the bridge over the coastal lagoon outlet is projected to be eroded by 2050 under both SLR scenarios. As with the bund, this will occur even without accelerated SLR, and without maintenance of the existing erosion protection could be totally lost within 10-15 years with existing rates of shoreline retreat.

4.3 Coastal Inundation Hazard Assessment

4.3.1 Bathtub Model Results

Coastal inundation bathtub model maps for each settlement under current and future SLR scenarios are presented in Appendix H, with the results for Amberley Beach summarised below in Table 4.6.

Table 4.6: Summary of estimated spatial extent of potential inundation hazard in Amberley Beach.

Scenario	**Present Day (2020)**	**30-year (2050)**	**50-year (2070)**		**100-year (2120)**	
Scenario and 1% AEP static water level[1]	**(2.84 m)**	**RCP 8.5 (3.12 m)**	**RCP 8.5 (3.29 m)**	**RCP 8.5+ (3.45 m)**	**RCP 8.5 (3.90 m)**	**RCP 8.5+ (4.2 m)**
Approx % of settlement inundated	20%	33%	90%	95%	99%	99%
Average Depth	0.1m	0.25m	0.3m	0.5m	0.8m	1.2m

[1] 1% AEP static water level = Storm Tide (ST) + wave set-up (WS).
All water levels are given in terms of Lyttleton Vertical Datum 1937 (LVD)

At current sea levels, 1% AEP static water levels are insufficient to overtop the protection bund along the settlement frontage (generally >5 m LVD) but can overtop the lowered beach ridge at the outlets of the coastal lagoons south and north of the settlement. These water bodies connect to the low-lying land to the west of the settlement, with 20% of the settlement footprint located along Grierson Ave being below the static water level and therefore suspectable to inundation under the bathtub modelling approach. Potential inundation depths would be shallow, generally in the range 0.1-0.2m. Although the bathtub method produces very conservative results as it does not account for temporal variances of the event or any hydrodynamic factors, this pattern of inundation occurred in the 1992 storm event with overtopping of the natural beach ridge along the settlement footage, which was at much lower levels prior to the construction of the inundation bund. At current sea levels, wave run-up during a 1% AEP event, predicted to be able to reach elevations of 5.2 m LVD, will overtop parts of the bund, which is consistent with current observations, but will only result in isolated additional shallow inundation along Chamberlain Ave.

Under the 30-year RCP 8.5 scenario the 1% AEP static water level increases to 3.12 m LVD, sufficient to inundate approximately one third of the settlement footprint under the conservative bathtub approach. Average inundation depths would be in the order of 0.25 m and maximum depths up to 0.6 m in the north west corner of the settlement. Wave run-up, predicted to be able to reach elevations of 5.4 m LVD, will overtop the majority of the bund, however, the volume of water should not be sufficient to significantly increase the extent of inundation within the settlement, but will increase inundation depths.

Under the 50-year scenarios, the 1% AEP static water level would increase to 3.29 m LVD under RCP8.5 and 3.45 m LVD under the RCP8.5+ scenario. Under the conservative bathtub approach these water levels would inundate 90% to 95% of the settlement footprint, with average depths in the order of 0.3 m to 0.5 m and maximum depths in the north-west corner being in the order of 1.0 m. The only part of the settlement footprint that is not below the 1% AEP static water level at these sea levels is a narrow strip of current backshore, however, will be subject to beach overtopping with wave run-up predicted to be able to reach elevations in the order of 5.7 m LVD. The volume of water overtopping the beach during a high tide cycle could increase inundation depths in the settlement by in the order of 0.5 m. However, since most of the settlement is already shown to be susceptible to inundation, no additional run-up inundation extent is shown in the Appendix H mapping for this timeframe.

Under both the 100-year SLR scenarios, the whole of the settlement is below the projected 1% AEP static water level, therefore is shown as being totally susceptible to inundation with a bathtub modelling approach. While the extent of inundation within the settlement is similar to the 50-year scenarios, potential inundation depths are approximately doubled with water depths >0.7m across most of the settlement, and over 1m along the western edge (Grierson Ave). As these sea levels, the volume of water overtopping the beach during a high tide cycle could increase the inundation depths in the settlement by greater that 0.5 m

4.3.2 Coastal Inundation Risk

Dwellings and Properties

As indicated in Table 4.7, in the current day scenario, just over half of the dwellings in the Amberley Beach settlement intersect with the coastal inundation hazard footprint. Depth of water around the dwellings has not been assessed. In the 30-year scenario, this increases to 80% of property and dwellings potentially affected. In the 50-year and 100-year scenarios, all property and dwellings in the settlement potentially will be affected.

Table 4.7: Total number of dwellings and properties which intersect with the inundation hazard footprint

Timeframe	Total	Present Day (2020)	30-year (2050)	50-year (2070)		100-year (2120)	
Scenario			RCP 8.5	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Dwellings	108	65	88	106	108	108	108
Properties	138	85	110	136	138	138	138

Critical Infrastructure

As shown in Table 4.8, the wastewater treatment pond is not modelled to be inundated under any of the SLR scenarios presented. The wet well (assuming its head works is located at ground level) and Golf Links Rd are modelled as having shallow inundation in in the order of 0.2 m in a 1% AEP storm event with SLR over the next 30-year, increasing to water depths up to 0.5 m by 2070 in similar frequency events, although it is noted that there will be large variation in inundation depths on Golf Links Rd with variations in ground levels.

Table 4.8: Potential Inundation depth at critical infrastructure in Amberley Beach from Bathtub modelling of 1% AEP event.

Infrastructure	Present Day (2020)	30-year (2050)	50-year (2070)		100-year (2120)	
Infrastructure		**RCP 8.5**	**RCP 8.5**	**RCP 8.5+**	**RCP 8.5**	**RCP 8.5+**
Wastewater Treatment Pond	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated
Wet Well	Not inundated	0.16m	0.36m	0.49m	0.92m	1.2m
Golf Links Road	0.02m	0.21m	0.3m	0.5m	>0.6m	>1.2m

4.3.3 Recommended Further Inundation Modelling

Given the results of the bathtub modelling, it is recommended that further hydrodynamic modelling of the inundation hazards at Amberley Beach **is warranted** to better quantify the threshold for overtopping and inundation, the spatial extent and magnitude (e.g. inundation depths) of the hazard, the risks posed to the dwellings, the wet well and Golf Links Rd, and assess the likelihood of the continued success of the bund to provide inundation protection over future timeframes.

4.3.4 Change in Annual Recurrence Interval

As well as water levels, future SLR will also increase the annual probability that the present day 1% AEP event will occur. As shown in Figure 4., the Annual Recurrence Interval (ARI) for the present day 1% AEP event magnitude (e.g. 2.84 m static water level) reduces from the current 100 years to 15-30 years by 2050, to 6-12 years by 2070, and to 1 year by 2120. Expressed another way, this magnitude event is more than three times as likely to occur in any one year by 2050 and could become an annual occurrence by 2120 under both SLR scenarios.

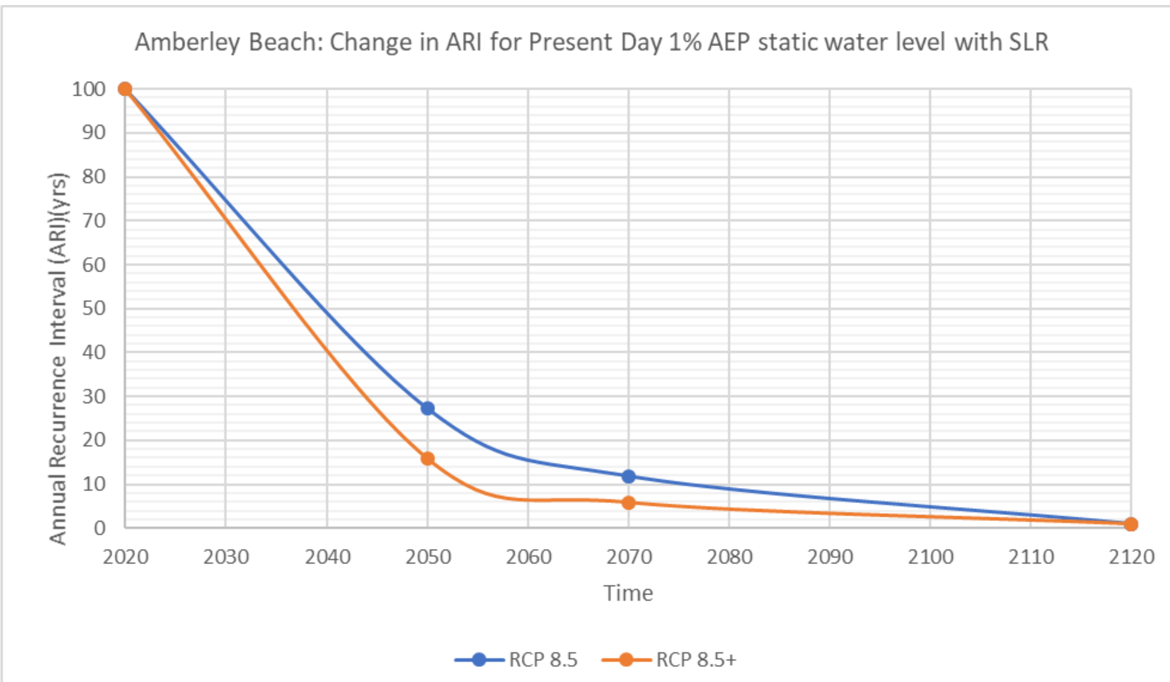

Figure 4.5: Effect of SLR on the Annual Recurrence Interval for Present Day 1% AEP static water level event for Amberley Beach.

4.4 Rising Groundwater Hazard Assessment

4.4.1 Existing Groundwater Conditions

At Amberley Beach the geology is the same as reported in Section 3.4.1 at Leithfield Beach, being dominated by Quaternary alluvial and beach deposits and the northwards continuation of the Christchurch Aquifer System. Although not investigated in any detail, Brown (2001) notes that the groundwater system north of the Ashley River displays a typical Canterbury Plains coastal deposition pattern, with interfingering interglacial marine fine sediment deposits (aquicludes) and glacial period outwash gravels (aquifers). Review of the Canterbury Maps Viewer (https://canterburymaps.govt.nz/) indicates extensive semi-confined or unconfined aquifers associated with the northern Canterbury Plains and in particular, the Kowai River (north and south branches) and Waipara River.

The Environment Canterbury Well Search database (https://ecan.govt.nz/data/well-search) and Canterbury Maps Viewer note numerous wells with water level data in the general Leithfield and Amberley areas. However, around the coastal settlements, data is more sparse and recorded bore elevations display significant discrepancy from elevations inferred from LiDAR data.

Groundwater flow is generally to the east towards the coast. Inland from Amberley Beach there is a significant drop in ground elevation from the inland plains area at a fluvial terrace scarp of the order of 10 to 20m in height. This scarp represents a significant control on groundwater elevations with seepage faces typically occurring at the base of the scarp, numerous springs are also mapped along or below the base of the scarp. In the lower lying coastal area the shallow nature of the water table is indicated by numerous drains and wetland areas. Flooded quarry pits are also present at the Winstone Aggregates and Readymix Quarry properties inland from Amberley Beach.

The Environment Canterbury (2016a) report on coastal groundwater discharge in the Waimakariri zone, describes the reduced deposition of fine-grained marine sediments in the northern Waimakariri zone due to longshore currents. This has resulted in a reduced thickness of low permeability confining layers between the glacial alluvial strata, allowing for more groundwater discharge from deeper aquifers via vertical (upwards) seepage. However, as indicated by water supply bore BW24/0051 at Leithfield Beach confined and artesian groundwater conditions are still present.

As noted in Section 4.1, surface water bodies around the Amberley Beach Settlement include Mimimoto Lagoon to the south and Amberley Beach Lagoon to the north as well as the inundated quarry voids at the Winstone Aggregates and Readymix Quarry properties inland from the settlement.

4.4.2 Adopted Parameters

The same parameters as for Leithfield Beach were adopted for incorporation in the AnAqSim groundwater model, being:

- Quaternary shallow alluvial hydraulic conductivity assumed at 110 m/day; based on Environment Canterbury (2016a).
  - Two local shallow bores (N34/0408 - 9.7m and M34/0321 -38.4 m) have had pumping tests undertaken and provide indicative hydraulic conductivities of 81 and 125 m/day respectively.
  - BW24/0051 at Leithfield Beach has also been tested but is in a confined aquifer screened from 116.7 to 118.7 m. BW24/0051 returned a transmissivity from two tests of the order of 210 to 230 m^2/day, equivalent to hydraulic conductivity of approximately 70 m/day. An aquifer storage coefficient of $1.0x10^{-4}$ was derived.
- Recharge rate –100 mm/year based on work completed by Environment Canterbury (2016b)
- Lagoons and drains simulated as specified head boundaries based on LiDAR elevations.

4.4.3 Rising Groundwater Mapping

The map of depths to the indicative average shallow groundwater conditions at Amberley Beach under present day and future sea levels presented in Appendix I indicate that there are only very limited areas in north west corner of the settlement with shallow groundwater (<1 m BGL), and none with depth to average groundwater < 0.5 m.

The maps of groundwater levels under future sea level rise scenarios, indicate that there only small increase in area along the western settlement boundary exposed to shallow average groundwater (<1m BGL) with a SLR rise of 0.56 m in 50 years, and no areas with average groundwater < 0.5 m BGL. Under the 1.3m SLR scenario over 100 years, the whole of the western margin of the settlement (e.g. west of Grierson Ave) is predicted to have average groundwater levels shallower than 1m BGL, with some areas shallower than 0.5m BGL in the northwest corner.

The predicted saline interface with SLR is shown of Figure 4.6, indicating potential significant saline incursion in the unconfined aquifer propagating up to 700m inland under the RCP 8.5+ 100yr SLR scenario.

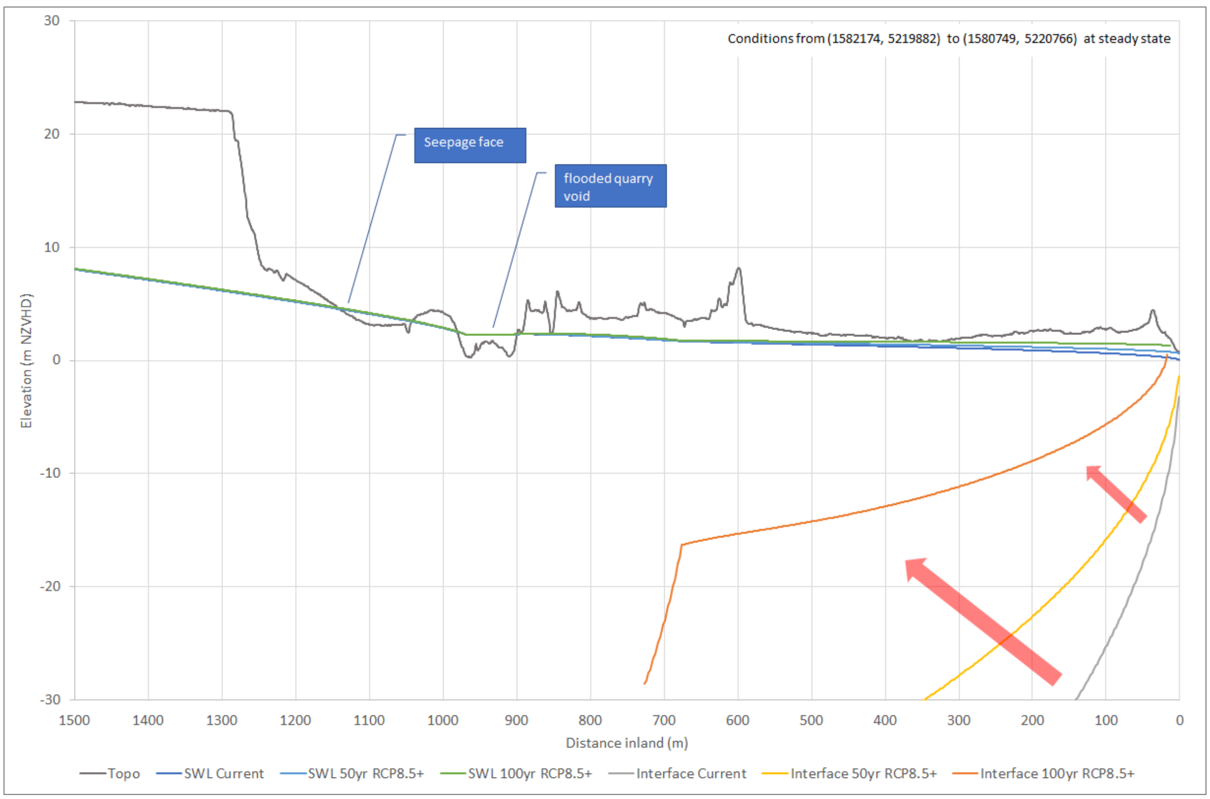

Figure 4.6: Amberley Beach simulated water levels and saline interface with SLR.

4.4.4 Rising Groundwater Risk

The number of dwellings exposed to different groundwater depths with present and future sea levels is presented in Table 4.9. Note that where a dwelling covers two or more depth categories, the shallowest depth has been applied. The number of dwelling predicted to be at risk from groundwater shallower than 0.5 m increases from zero to one over the next 50 years, however increases to 15 houses by 100 years. Within this timeframe over 60% of dwellings will be subjected to average groundwater shallower than 1m BGL, compared to 8% under the current scenario.

Table 4.9: Number of dwellings exposed to indicative average groundwater depths at Amberley Beach.

RCP 8.5+ SLR Scenario	**SLR Scenario**	**Depth to Groundwater (m BGL)**			
		≤ 0.5	**0.5-1**	**1-2**	**> 2**
Present Sea level (2020)	Current day	0	9	83	16

50 year (0.6 m SLR)	50 year/0.6m	1	30	77	0
100 year (1.3 m SLR)	100 year/1.3m	15	51	42	0

Predicted average groundwater depths at the nominated critical infrastructure is presented in Table 4.10. As shown, the Amberley Wastewater Treatment Plant and wet well are not predicted to be impacted by rise a rise in shallow groundwater with SLR over the next 100 years.

Table 4.10: Indicative average groundwater depths at Amberley Beach Critical Infrastructure (m BGL)

Infrastructure	Present day (2020)	100-year RCP 8.5+(SLR=1.3 m)
Wet Well	1-2m	1-2m
Wastewater Treatment Plant	2-5m	2-5m

5. Motunau

5.1 Settlement Description

Motunau is a small coastal settlement located along the top of an uplifted mudstone cliff 30-40 m high, and on the low-lying river terrace at the mouth of the Motunau River, as shown in Figure 5.1 and Figure 5.2. The settlement footprint contains 131 dwellings, but the 2013 census gives only 12 permanent residents living in Motunau[4]. The critical infrastructure of interest to the council at this settlement is the wastewater treatment plant located across the river to the north of the settlement, and two wet wells located on the lower river terrace as shown in figure 5.1.

Figure 5.1: Motunau settlement overview of settlement footprint and critical infrastructure.

[4] Taken from the New Zealand Census Data (2013) as provided by HDC.

Figure 5.2: Oblique view of Motunau settlement showing dwelling on the high mudstone cliffs and along the low terrace at the mouth of the Motunau River (Source Google Earth).

The cliff face at Motunau has under gone episodes of rapid erosion, resulting in house removals and the installation of gabion baskets at the base of the cliff as protection works in the 1980's to early 1990's period. The evidence of this erosion can be seen from the length of exposed drainage pipe in Figure 5.3(a). At the river mouth, rock training walls were installed in the 1970's and more recently armour rock has been placed along the frontage of The Parade to protect from bank erosion as shown in Figure 5.3(b).

Figure 5.3: (a) Erosion of Motunau high coastal cliffs; (b) Motunau River lower terrace - bank armour rock protection immediately upstream of the river mouth.

5.2 Coastal Erosion Hazard Assessment

5.2.1 Historical Long-term Shoreline Movements

An increase in cliff erosion rate in the 1980's prompted a number of studies of the mechanism of erosion (e.g. R.W Morris & Associates 1987, 1988) and the success of using gabion baskets as a wave trip and sediment trap wall at the base of the cliff to slow erosion rates (e.g. RETECH (1990,1991, 1991b)). These reports indicated that the increase in rate of erosion increased in the 1970's following the removal of a large volume of boulders from the shore platform in 1971 to form the river mouth training walls, resulting in increased wave energy being able to directly attack the toe of the cliff and accelerating cliff retreat. However, the gabion wall was not a sustainable protection structure, with the baskets breaking within a few years and not being repaired. Foster (2009) found that the cliff top position only suffered minor erosion in the 1950-1980 period, but retreat of between 10-23 m from 1980 to 1993, and a further 5-16 m retreat from 1993 to 2004. Conversely, Foster's analysis indicated that the width of the beach at Sandy Bay, along the cliffs immediately to the west of the settlement, had decreased approximately 25 m from 1950 to 1968, potentially as a precursor to the cliff erosion, but had stabled from 1968 to 2004.

For this assessment historical erosion was measured from five aerial photographs between 1950 and 2018 (see Table 2.1), with DSAS transects from 141 (east) to 168 (west). The map of historical shoreline positions and erosion rates is presented in Appendix B.

The shoreline reference position for Transects 141-146 inside the river mouth training walls is the vegetation line, however, care is required when interpreting results from these sites as they are influenced by complex morphodynamic processes at the river-coast interface. For Transects 147-152 the shoreline reference position is the cliff top position, which is clearly visible on the imagery. For transects Sandy Bay Transects 154-168 to the west of the settlement, the position of the top of cliff is not clear, so the reference position for measuring shoreline change was taken as the vegetation line at the back of the beach system.

As can be seen from the transect results presented in Figure 5.4, overall, there an erosional trend along the Motunau settlement shoreline, but as found by the previous studies, this erosion is episodic and occurring at different times intervals for the different shoreline morphologies.

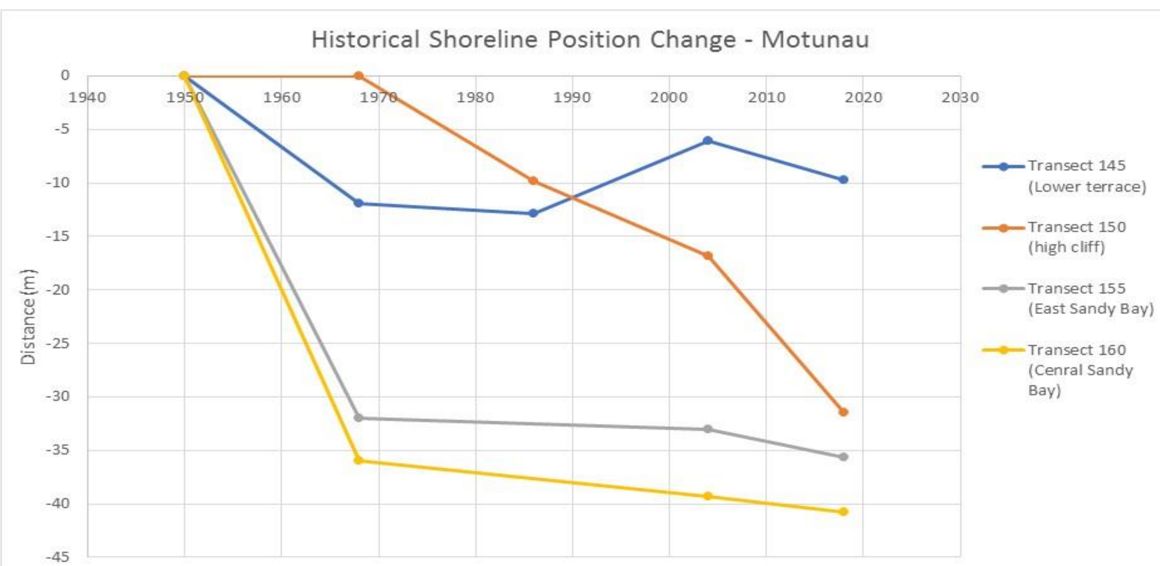

Figure 5.4: Historical shoreline position change for selected DSAS Transects at Motunau 1950-2018.

For transects 141-146 within the river mouth, the pattern of shoreline movement has been influenced by the placement of armour erosion as protection works. For Transect 145 presented in Figure 5.4, close to the mouth training walls, there is a net retreat of around -10 m since 1950, while Transects further upriver (e.g. 141-143) show net advance in the range of 8-17 m since 1950 due to the rock placement.

For the high cliffs (Transects 147-152), stability up to 1968 followed episodic erosion patterns with total retreat distances ranging from -4 m to -30 m (Transect 150 as shown in figure 5.4) over the next 50 years to 2018. For extrapolation of historical rates, a smoothed average retreat rate of -0.26 m/yr to -.035 m/yr has been applied across the cliff transects.

Transects 154-168 along Sandy Bay to the west of the settlement displayed beach retreat trends similar to that described by Foster (2009); rapid retreat at up to 2 m/yr from 1950 to 1968, followed by slow retreat at rates < 0.1 m/yr to give total retreat distances of -35 to -40 m. For the extrapolation of historical rates, smoothed average retreat rate of -0.46 to -0.49 m/yr has been applied across transects in this section of the Motunau coast. The projected future shoreline retreat from the extrapolation of these historical rates is presented in Table 5.1.

Table 5.1: Projected shoreline retreat from extrapolation of rates from 1950 to 2018 for selected DSAS Transects at Motunau.

Scenario	30 years (2050)	50 years (2070)	100 years (2120)
Transect 145 (Lower terrace)	-2.1 m	-3.7 m	-7 m
Transect 150 (High Mudstone cliff)	-9.3 m	-15.5 m	-31.0 m
Transect 155 (East Sandy Bay)	-14.1 m	-23.5 m	-47 0m
Transect 160 (Central Sandy Bay)	-14.4 m	-24.0 m	-48 0m

5.2.2 Accelerated Sea Level Rise Effects

The effects of projected accelerated SLR on coastal erosion of the Motunau mudstone cliffs was calculated using the equation soft sediment cliffs from Walkden & Dickson (2008) as set out in section 2.3.2 and Appendix D. The results are summarised below in Table 5.2. However, it should be noted that due to difficulties in identifying the top cliff edge along Sandy Bay to the west of the settlement, the cliff toe was taken as the point of reference for the retreat calculations.

The effect of SLR was not calculated for the river bank frontages inside the river mouth training walls due to the complicated fluvial -coastal processes interactions acting in this environment, meaning that there is no commonly accepted coastal model that is appropriate to apply in this area.

Table 5.2 Calculated erosion distance due to accelerated SLR at Motunau.

Profile	Scenario	30 Years (2050)		50 Years (2070)		100 Years (2120)	
		SLR	Erosion Dist	SLR	Erosion Dist	SLR	Erosion Dist
Transect 150 (High Mudstone cliff)	RCP 8.5	+0.23 m	-7.7 m	+0.40 m	-14.3 m	+1.01 m	-37.5 m
	RCP 8.5+	+0.32 m	-10.7 m	+0.56 m	-19.8 m	+1.31 m	-47.1 m
Transect 155 (East Sandy Bay)	RCP 8.5	+0.23 m	-9.9 m	+0.40 m	-18.5 m	+1.01 m	-48.6 m
	RCP 8.5+	+0.32 m	-13.9 m	+0.56 m	-25.6 m	+1.31 m	-61.0 m
Transect 160 (Central Sandy Bay)	RCP 8.5	+0.23 m	-11.6 m	+0.40 m	-21.6 m	+1.01 m	-56.7 m
	RCP 8.5+	+0.32 m	-16.2 m	+0.56 m	-29.9 m	+1.31 m	-72.1 m

5.2.3 Short-term Storm Effects

The short term storm effect measured from Environment Canterbury beach profiles showed that between 1997-2019, the maximum annual survey erosion for the beach toe at the base of the cliff across the four profiles analysed ranged from -0.6 to -6m as shown in Table 5.3. Due to the method used to record the cliff surveys, in some instances the cliff edge was not included in the annual survey, and therefore it was in no position to use this feature as an indicator for annual change.

The maximum beach toe erosion recorded was at Profile HCH2477 between June 2017 – May 2018, a year which experienced four coastal storm events, included a relatively large event over three days on 10-12 April 2018, immediately before the latter survey. Adopting this upper limit, an arbitrary value of 6m has been adopted as the short-term erosion component for the PFSP for Motunau settlement. Although the cliff surveys started after the episodic mass failure erosion events in the 1980's, it is considered that 6 m is an appropriate value for these types of events.

Table 5.3: Maximum short-term erosion measured by ECan beach profiles at Motunau.

Profile	Feature Measured	Maximum Inter-Survey Change	Period of max change	Storm Notes (from ECan Storm register)
HCH2458 (Sandy Bay)	Toe	-0.6 m	December 2002 - December 2003	3 events: July, Sept, Oct 2003.
HCH2477 (high cliff)	Toe	-6.0 m	June 2017 - May 2018	4 events: Jun & Jul 2017, Feb & April 2018.
HCH2487 (high cliff)	Toe	-5.3 m	January 2010 - December 2010	2 events: May, Aug 2010
HCH2549 (low river terrace)	Toe	-3.7 m	May 2016 - June 2017	5 events: Sept 2016, Jan(2), April, May 2017
	Scarp	-3.1 m	May 2016 - June 2017	

5.2.4 Projected Future Shoreline Position (PFSP)

From the combination of the above information on the individual components, the resulting distances from the current shoreline to the PFSP under the RCP 8.5 and RCP 8.5+ scenarios at selected transects are presented below in Table 5.4. Full details of the components at all transects are presented in Appendix G and the ground position in relation to the settlement is shown in Appendix F.

The PSFP lines have only been calculated for the high cliffed section of coastline at Motunau. As noted above, PSFP lines have not been calculated for the lower river terrace river frontage inside the river mouth training walls due to the complicated fluvial -coastal processes interactions acting in this environment. It is unknown what the effect of SLR will have on this section of shoreline. While the erosion rate along this section has historically been much lower than the cliffed open coast section of coastline, the low elevation and exposure to both coastal and fluvial processes is likely to cause an acceleration of the erosion trend if the armour rock protection is not of sufficient size and elevation to continue to provide protection. The lower terrace should be continually monitored to determine any increase in erosion rates in the future.

From comparing the individual component results with the over-all projected erosion distances in Table 5.4, it can be seen that the extrapolation of long-term erosion due to current erosion drivers of waves, water levels and mass failure will be the main contribution to retreat over the next 50 years, and that accelerated rates of SLR will only contribute more than 50% of the projected erosion over a 100-year timeframe.

Table 5.4: Distances from current shoreline to PFSP at Motunau. (Distances rounded to nearest metre)

Timeframe	30-year (2050)		50-year (2070)		100-year (2120)	
Scenario	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Transect 150 (High mudstone cliff)	-22 m	-23 m	-34 m	-36 m	-66 m	-69 m
Transect 155 (East Sandy Bay)	-38 m	-39 m	-60 m	-62 m	-118 m	-122 m
Transect 160 (Central Sandy Bay)	-38 m	-40 m	-61 m	-63 m	-120 m	-124 m

5.2.5 Coastal Erosion Risk

As can be seem from the mapping of the PFSP in Appendix F, many properties will be at risk from cliff erosion under all SLR scenarios assessed. A summary of the number of properties at risk (e.g. intersect with the PFSP line) is presented below in Table 5.5. The resulting property numbers are as the lower limit of the properties within the settlement that may be affected at some stage in the future, as it does not include properties located on the low terrace near the river mouth, some of which are located less than 30 m from the current river bank.

Of the 150 properties within the Motunau settlement footprint, 28 are projected to be at risk from cliff erosion over the next 30 years, 36 over the next 50 years, and 60-70 over the next 100 years.

Of the critical infrastructure assessed at Motunau, the wastewater treatment plant and wet well located up the river are not going to be at risk from coastal erosion over the next 100 years. However, the wet well on the lower terrace by the river mouth is located around 32 m from the current river bank, and although not covered by the assessment of the PFSP, is considered to be at possible risk from coastal erosion within 50 years and likely to be at risk within 100 years. However, further assessment of likely river mouth morphology changes with SLR would need to be made in order to more accurately determine the level of risk over these time periods.

Table 5.5: Number of properties affected by coastal erosion in future SLR scenarios.

Timeframe	Total	30-year (2050)		50-year (2070)		100-year (2120)	
Scenario		**RCP 8.5**	**RCP 8.5+**	**RCP 8.5**	**RCP 8.5+**	**RCP 8.5**	**RCP 8.5+**
Number of properties	150	>28	>28	>36	>36	>63	>69

5.3 Coastal Inundation Hazard Assessment

5.3.1 Bathtub Model Results

Coastal inundation bathtub model maps for each settlement under current and future SLR scenarios are presented in Appendix H, with the results for Motunau summarised below in Table 5.6. No run-up effects were calculated for Motunau, as the lower river terrace inside the river mouth was not considered to be exposed to run-up processes.

The results show that under all scenarios possible inundation is limited to around 10% of the settlement footprint located on the low river terrace, with depths of inundation under the bathtub modelling approach increasing from an average of 0.7 m under current sea levels to 1.8 m with SLR over the next 50 years and over 2 m with SLR over the next 100 years. However, the bathtub method produces very conservative results as it does not account for temporal variances of the event or any hydrodynamic factors, therefore these depths are most likely over estimates.

Table 5.6: Summary of the spatial extent of potential inundation hazard in Motunau.

Scenario	Present Day (2020)	30-year (2050)	50-year (2070)		100-year (2120)	
Scenario and 1% AEP static water levels[1]	**(3.82m)**	**RCP 8.5 (4.05m)**	**RCP 8.5 (4.22m)**	**RCP 8.5+ (4.38m)**	**RCP 8.5 (4.83m)**	**RCP 8.5+ (5.13m)**
Approx % of settlement inundated	10%	10%	10%	10%	10%	10%
Average Depth	0.69m	1.3m	1.8m	1.9m	2.2m	2.4m
[1]1% AEP static water level = Storm Tide (ST) + wave set-up (WS). All water levels are given in terms of Lyttleton Vertical Datum 1937 (LVD)						

5.3.2 Coastal Inundation Risk

Dwellings and Properties

Due to the spatial extent of the coastal inundation hazard being limited to the low terrace, the number of dwellings and properties at risk from this hazard in a 1% AEP event is similar over all timeframes and SLR scenarios. As shown in Table 5.7, the numbers of dwellings and properties at risk increase from 12 and 11 respectively under current sea levels to 14 dwellings and properties over a 100-year timeframe. Depth of water around the dwellings has not been assessed, however, this will increase under each timeframe as per the results presented in Table 5.6.

Table 5.7: Total number of dwellings and properties which intersect with the inundation hazard footprint.

Timeframe	Total	Present Day (2020)	30-year (2050)	50-year (2070)		100-year (2120)	
Scenario			**RCP 8.5**	**RCP 8.5**	**RCP 8.5+**	**RCP 8.5**	**RCP 8.5+**
Dwellings	131	12	12	12	12	12	13
Properties	132	11	12	12	13	14	14

Critical Infrastructure

In Motunau the wastewater treatment plant is not predicted to be at any risk of coastal inundation in any sea level rise scenario even under the conservative bathtub modelling. However, the two wet wells in the settlement are at risk of inundation in each timeframe by different increments. As shown in Table 5.8 the south wet well located on the low river terrace will be exposed to greater water depths. The modelled depths at the north wet well is likely to be very conservative, being limited to tidal influences without wave set-up, however may also be exposed to fluvial flood hazards.

Table 5.8: Potential Inundation depths at critical infrastructure in Motunau from Bathtub modelling of 1% AEP event.

Timeframe	Present Day (2020)	30-year (2050)	50-year (2070)		100-year (2120)	
Infrastructure		RCP 8.5	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Wastewater treatment plant	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated
Wet Well North (inland)	Not inundated	0.3m	0.45m	0.61m	1.06m	1.36m
Wet Well South (coastal)	0.2m	0.89m	1.16m	1.31m	1.76m	2.06m

5.3.3 Recommended Further Inundation Modelling

Given the results of the bathtub modelling, it is recommended that further hydrodynamic modelling of the inundation hazards at Motunau **is not warranted** as this time due to the footprint of potential inundation being limited to the lower river terrace, the small number of properties and dwellings potentially at risk, and the limited opportunity for the further development in this area. However, consideration for future proofing options for the wet well located on this terrace against inundation risk over at least the next 30 years is warranted.

5.3.4 Change in Annual Recurrence Interval

As well as water levels, future SLR will also increase the annual probability that the present day 1% AEP event will occur. As shown in Figure 5.5, the Annual Recurrence Interval (ARI) for the present day 1% AEP event magnitude (e.g. 3.82 m static water level) reduces from the current 100 years to 50-60 years by 2050, to 20-30 years by 2070, and to 1-5 years by 2120. Expressed another way, this magnitude event is twice as likely to occur in any one year by 2050 under both SLR scenarios and could become a bi-annual occurrence by 2120 under the more extreme RCP8.5+ scenario.

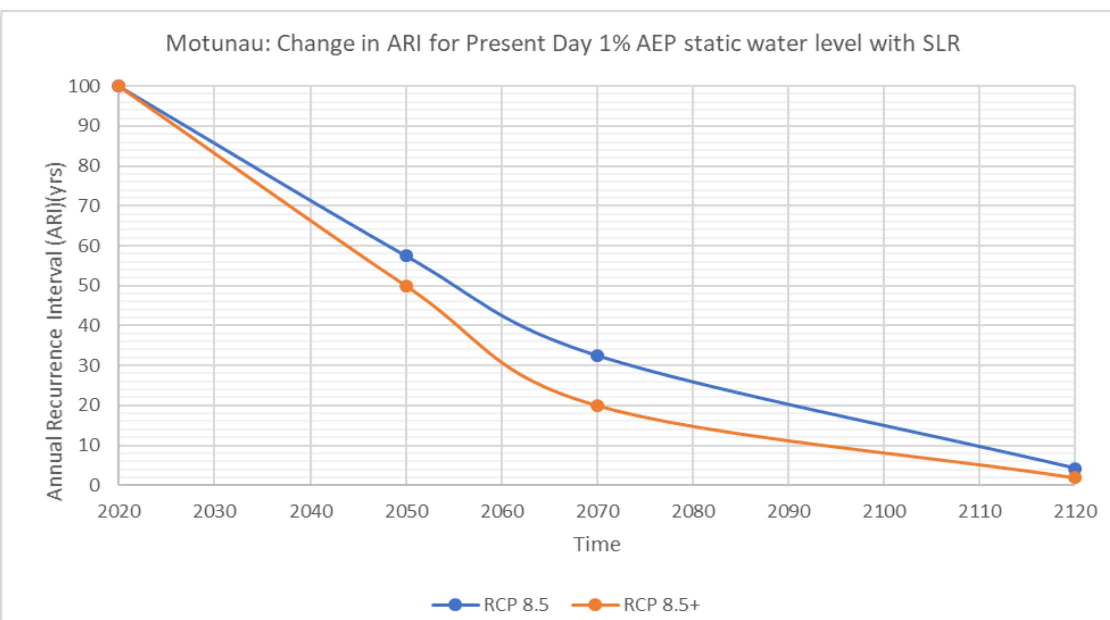

Figure 5.5: Effect of SLR on Annual Recurrence Interval for Present Day 1% AEP static water event for Motunau.

5.4 Rising Groundwater Hazard Assessment

5.4.1 Existing Groundwater Conditions

At Motunau Beach the geology is dominated by Tertiary age (Pliocene and Miocene) sedimentary lithologies of the Motunau Group and Greta Formation. Typical lithologies include siltstones, mudstones and conglomerate. The Pliocene and Miocene sediments are typically of low permeability and regarded as aquitards or hydrogeological basement where overlain by more productive aquifers. A veneer of alluvium exists on the raised marine terraces and in association with the Motunau River. These are not considered to represent a regionally significant aquifer.

The Canterbury Maps Viewer shows no mapped semi-confined or unconfined alluvial aquifers. There are also no bores or water level data in the immediate area.

Review of the New Zealand Geotechnical Database identified two hand augered geotechnical investigation holes at Motunau that intercepted groundwater. The holes intercepted groundwater at approximately 1.90m BGL (38m MSL). This is inferred to represent a minor perched water table at the interface of the unconsolidated alluvium overly of the Motunau Group sediments.

While the intrinsic permeability of the Tertiary sediments is generally low, localised faulting and fracturing, or karst development in limestone units, can result in localised productive aquifers. On a regional scale, bulk hydraulic conductivity is likely to be relatively low. However, one bore (N34/0102) installed in Motunau Group sedimentary rocks approximately 3.4 km inland from Motunau records a maximum yield of 26 L/s. The bore is installed in limestone and is not considered to be representative of the Motunau Beach area.

5.4.2 Adopted Parameters

The following parameters were adopted for incorporation in the AnAqSim groundwater model:

- Tertiary formations simulated as unconfined aquifer with hydraulic conductivity assumed at 1 m/day;
- Recharge rate – 36.5 mm/year to reflect low permeability formation.
- Motunau River and inlet simulated as specified head boundaries based on LiDAR elevations.

5.4.3 Rising Groundwater Mapping

The map of depths to the indicative average shallow groundwater conditions at Motunau under present day and future sea levels are presented in Appendix I. The predicted change to the saline interface with SLR is shown in Figure 5.6.

Since the majority of Motunau settlement is elevated on the high terrace, depth to groundwater over most of the settlement is >10 m under all SLR scenarios and timeframes over the next 100 years. On the lower river mouth terrace, current depth to groundwater is in the range 5-10 m, with 100 years of SLR reducing depths along The Parade to 1 m BGL and to <0.5 m along foreshore areas.

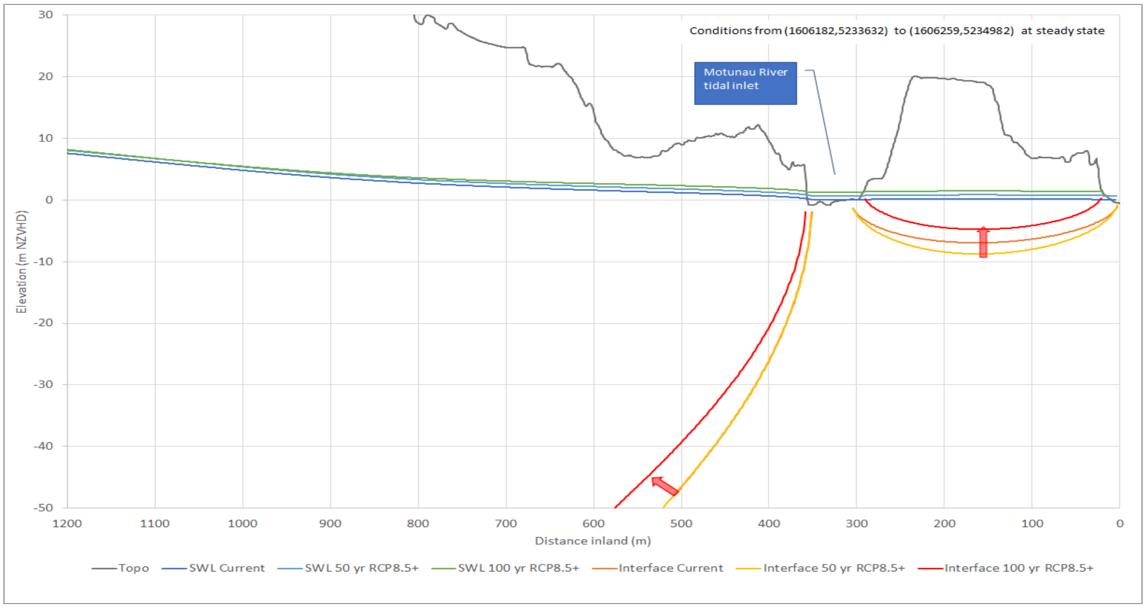

Figure 5.6: Motunau Beach simulated water levels and saline interface with SLR.

5.4.4 Rising Groundwater Risk

The number of dwellings subjected to potentially shallow groundwater, and predicted groundwater depths at nominated critical infrastructure are provided on Table 5.9 and Table 5.10 respectively. No dwellings are predicted to be at risk from groundwater shallower than 1m BGL over any timeframe in the next 100 years. Critical infrastructure is also not predicted to be at risk from rising groundwater levels with SLR.

Table 5.9: Number of dwellings exposed to indicative average groundwater depths at Motunau.

SLR Scenario	**Depth to Groundwater (m BGL)**			
	≤ 0.5	**0.5-1**	**1-2**	**> 2**
Present Sea level (2020)	0	0	0	131
50 year (0.6 m SLR)	0	0	7	124
100 year (1.3 m SLR)	0	0	12	119

Table 5.10: Indicative Average Groundwater Depths at Motunau Critical Infrastructure (m BGL).

Infrastructure	**Present day (2020)**	**100-year RCP 8.5+ (SLR=1.3 m)**
Wet Well North (inland)	2-5m	2-5m
Wet Well South (coastal)	2-5m	1-2m
Wastewater treatment plant	2-5m	2-5m

6. Gore Bay

6.1 Settlement Description

As shown in Figure 6.1, Gore Bay is a small coastal settlement to the north of the Port Robinson headland and around 7 km north of the mouth of Hurunui River. The settlement is located on a narrow raised coastal plain of Holocene sand and gravel bounded by tall alluvial and loess cliffs of Tertiary sediment to the west that mark the likely shoreline position 6500 years ago, and the gravel beach system to the east. The settlement contains 92 dwellings and 27 permanent residents[5].

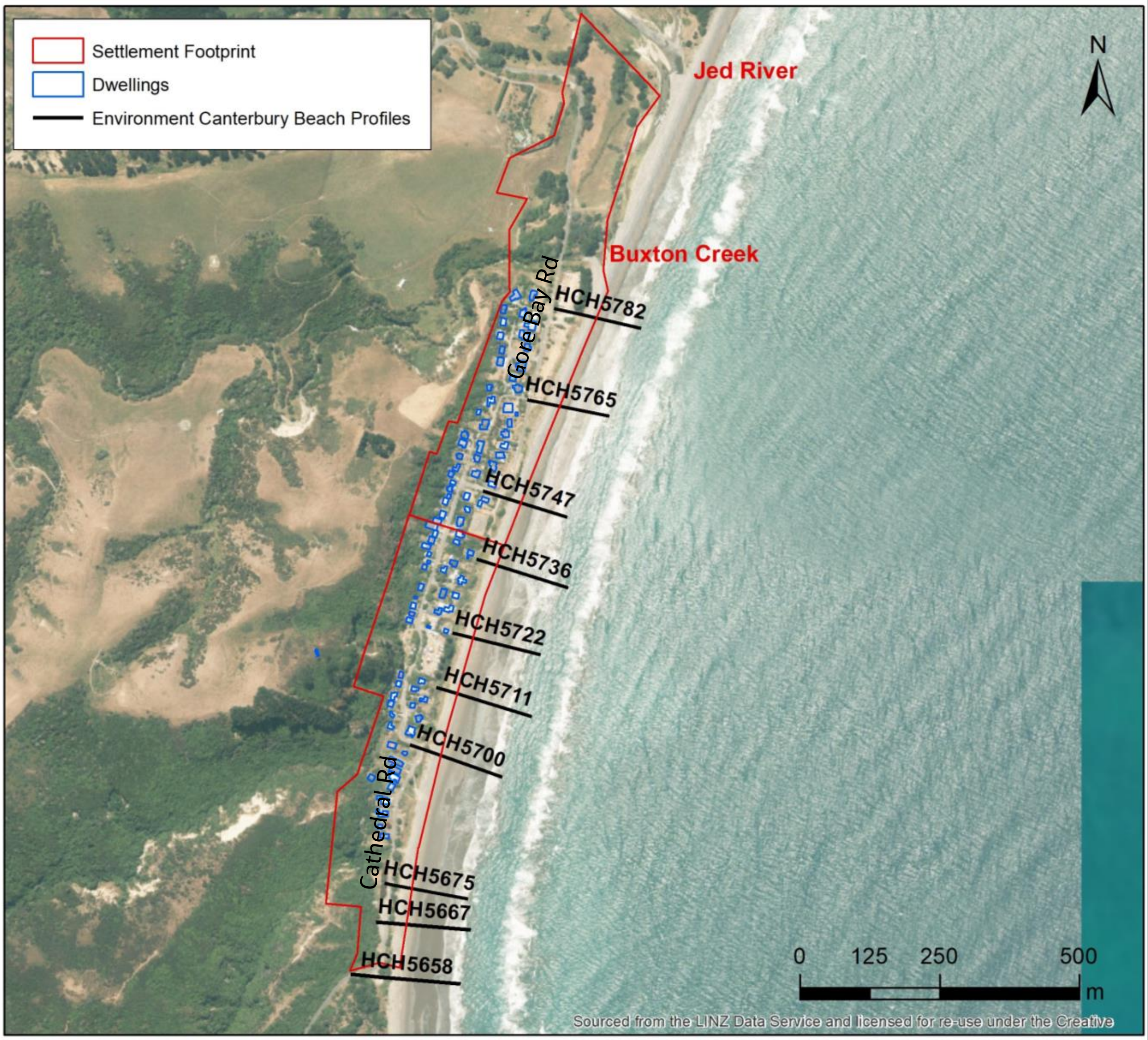

Figure 6.1: Gore Bay settlement footprint and critical infrastructure. Gore Bay is split into north and south for inundation purposes.

[5] Taken from the New Zealand Census Data (2013) as provided by HDC.

The beach system varies between composite and a Mixed Sand and Gravel (MSG) beach state (Figure 6.2(a), with a sandy lower foreshore (particularly at south end) and flat gradient across the surf zone from shoaling wave break patterns on Google Earth images suggesting that nearshore profile is similar to a sand beach. As the southern end of the settlement, the beach is backed by a low scarp or former beach ridge up to 7-8 m AMSL providing so protection from coastal inundation (Figure 6.2(a)). At the north end of the settlement, along Gore Bay Rd (Figure 6.2(b) the coastal plain is lower as it dips towards Buxton Creek and the Jed River that discharge to the beach at the northern limit of the settlement in a combined channel, although neither have a permanent mouth to the ocean.

The council did not identify any critical infrastructure for risk assessment in Gore Bay.

Figure 6.2: Gore Bay (a) composite beach; and (b) Gore Bay Rd at the north end of the settlement

6.2 Coastal Erosion Hazard Assessment

6.2.1 Historical Long-term Shoreline Movements

Based on a study by Vessey (2003), Environment Canterbury (2010) indicated that there have been several periods where erosion was of such concern to the residents of Gore Bay that action was taken including following 1934-1940, 1951-1952, and 1975-1978. As a result, groynes, seawalls and breastworks have been undertaken along much of the Gore Bay coastline, as well as ad hoc measures seaward of individual properties at various times. Environment Canterbury (2010) further notes that since the late 1970's, the Gore Bay coastline has been generally stable with no significant erosion occurring, however, due to the historic erosion that has occurred, it is possible that this may again occur in the future.

For this assessment historical erosion was measured from six aerial photographs between 1955 and 2015 (see Table 2.1), with DSAS transects from 82 (north) to 130 (south). It is noted that the 2015 end date for the analysis is prior to the 2016 Kaikoura Earthquake, so that beach position is not influenced by any increases in local sediment supply as a result of the earthquake. The map of historical shoreline positions and erosion rates is presented in Appendix B.

As can be seen from the transect results presented in Figure 6.3, there has similar trends of shoreline movements along the main part of the settlement to the south of Buxton Creek (Transects 101-130), with fluctuating periods of accretion and erosion to the mid 1980's followed by steady accretion through to 2015. It is noted that the rapid erosion from 1979 to 1985, in which retreat rates were > 1 m/yr at the southern end Gore Bay was not identified by Environment Canterbury (2010). The results of the DSAS also indicate a general south-north trend in reducing magnitude of the shoreline changes within each period, with greater in shoreline position occurring at the south end of the settlement (e.g. Transects 127,118 in figure 6.3) than at the north end (e.g. Transect 101).

Over the total 60 years covered by the analysis, the whole of the coastal frontage to the south of Buxton Creek accreted, with net rates ranging from +0.10 m/yr south of the settlement, increasing to +0.15 m/yr in the centre of the settlement, and decreasing back to +0.05 m/yr at Buxton Creek.

However, as also shown in Appendix B and Figure 6.3 (Transect 87), there is a different trend to the north of Buxton Creek (Transects 100-82), with predominantly erosion being displayed to 1985, which is unfortunately where the aerial imagery ceases. Average net rates of retreat over the 30 years of measurements for this section of coast progressively increasing to the north from -0.05 m/yr north of Buxton Creek to -0.5 m/yr north of the Jed River. This continues the northward trend of reducing sediment supply to withstand erosion events. Although the sites to the south of Buxton Creek show accretion since 1985, the survey results from Environment Canterbury profile HCH5867 located near Transect 87 showed beach ridge retreat of -7.6 m from 1993 to 2015, suggesting that erosion has continued to the north of river, but at reduced rates. It is also notable that since 2015 there has been a small accretion at this profile site, possibility due to increased local sediment sample post the 2016 North Canterbury Earthquake.

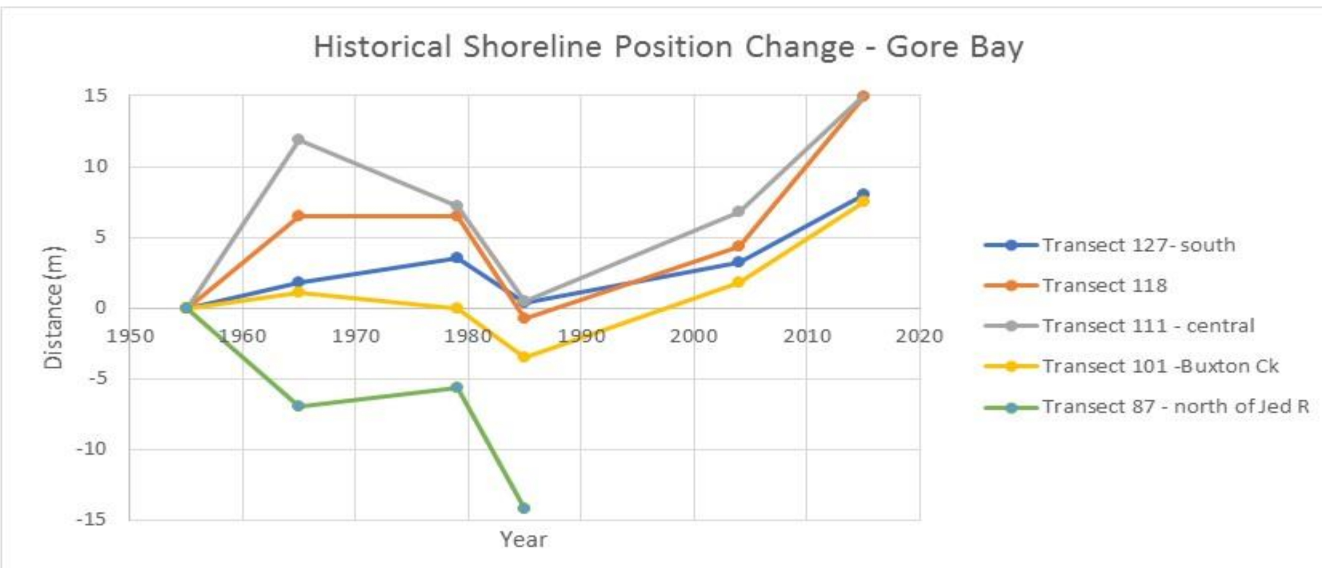

Figure 6.3: Historical shoreline position change for selected DSAS Transects at Gore Bay from 1955 to 2018.

The resulting projected shoreline change distances from extrapolating the historical rates at the transects presented in Figure 6.3 over 30, 50 and 100 years into the future are presented in Table 6.1.

Table 6.1: Projected change in shoreline position from extrapolation of rates from 1955 to 2015 for selected DSAS Transects at Gore Bay.

Scenario	30 years (2050)	50 years (2070)	100 years (2120)
Transect 127 (south of settlement)	+3.3 m	+5.5 m	+11 m
Transect 118 (Southern end of settlement)	+5.1 m	+8.5 m	+17 m
Transect 111 (central settlement)	+3.9 m	+6.5 m	+13 m
Transect 101 (Northern end of settlement)	+0.6m	+1.0 m	+2.0 m
Transect 87 (North of Jed River)[1]	-14.1 m	-23.5 m	-47.0 m
Note (1): Erosion rates 1955-1985 as not available to 2015. Likely that actual erosion rates to 2015 would be less.			

6.2.2 Accelerated Sea Level Rise Effects

Due to the assumed flat nearshore profile of a sand beach, the effects of projected accelerated SLR on coastal erosion was calculated at Gore Bay using the modified Bruun Rule for composite beaches as set out in section 2.3.2 and Appendix D. Beach crest height and sediment size was taken from the 10 Environment Canterbury profiles along the settlement frontage as shown in Figure 6.1, with crest elevation ranged from 3.2 m to 4 m above MSL and that 59% of the beach sediment was sand. Closure depth was calculated to be -11.8 m (below MSL) at a distance of 2800 m from the shore. The resulting shoreline retreat due to accelerated SLR under the over 30, 50 and 100-year timeframes under the RCP 8.5 and RCP8.5+ SLR scenarios are presented in Table 6.2.

The results of the effects of sea level rise in Gore Bay are summarised below in Table 6.2. By 2050, erosion as a direct result of sea level rise could be up to 27m, by 2070 up to 49m, and by 2120 up to 119m.

Table 6.2: Calculated erosion distance due to accelerated SLR at Gore Bay.

Transect	Scenario	30 years (2050)		50 years (2070)		100 years (2120)	
		SLR	Erosion Dist	SLR	Erosion Dist	SLR	Erosion Dist
Transect 127 (southern end of settlement)	RCP8.5	+0.23 m	-17.4 m	+0.40 m	-30.8 m	+1.01 m	-85.4 m
	RCP8.5+	+0.32 m	-27.5 m	+0.56 m	-48.9 m	+1.31 m	-119.3 m
Transect 118	RCP8.5	+0.23 m	-17.1 m	+0.40 m	-30.2 m	+1.01 m	-83.7 m
	RCP8.5+	+0.32 m	-27.0 m	+0.56 m	-47.9 m	+1.31 m	-116.9 m
Transect 111	RCP8.5	+0.23 m	-16.5 m	+0.40 m	-29.3 m	+1.01 m	-81.1 m
	RCP8.5+	+0.32 m	-26.1 m	+0.56 m	-46.4 m	+1.31 m	-113.2 m
Transect 101 (Northern end of settlement)	RCP8.5	+0.23 m	-16.4 m	+0.40 m	-29.1 m	+1.01 m	-80.5 m
	RCP8.5+	+0.32 m	-26.0 m	+0.56 m	-46.1 m	+1.31 m	-112.4 m
Transect 87 (north of Jed River)	RCP8.5	+0.23 m	-16.3 m	+0.40 m	-28.9 m	+1.01 m	-79.9 m
	RCP8.5+	+0.32 m	-25.8 m	+0.56 m	-45.8 m	+1.31 m	-111.7 m

It is noticeable from the differences in erosion distances for each RCP scenario that the erosion distance is very sensitive to the magnitude of SLR. For example, a 0.1 m additional rise under scenario 8.5+ by 2050 adds around 10 m to the erosion distance, an addition 0.15 m rise by 2070 adds around 18 m, and an addition 0.3 m rise by 2120 adds over 30 m to the erosion distance.

6.2.3 Short-term Storm Effects

As shown in Table 6.3, the short-term storm effect measured from the Environment Canterbury beach profiles showed that between 1993-2019, the maximum inter-survey erosion across the eleven profiles analysed ranged from -2.4 m for the top of the back of beach scarp to -9.5 m for the 3 m contour at Profile HCH5736. This maximum annual inter-survey retreat occurred between November 2000 and December 2001, when one storm event was recorded on the Environment Canterbury storm database. However, a second site (HCH5711) also experienced a similar magnitude of annual retreat (-9.3 m) in the December 2002-December 2003 period when 3 storm events were recorded on the database. Several sites experienced retreat in the order of -5 m during periods with between 4- 7 storm events.

Adopting a conservative approach of using the upper limit of the inter-survey erosion, an arbitrary value of -10 m has been adopted as the short-term erosion component for the PFSP calculation for Gore Bay. It is noted that this is the largest short-term component of any of the settlements covered in this assessment, however it is justified on the grounds that it has lower crest elevations and greater wave energy than the composite beaches at Amberley Beach and Leithfield Beach, therefore is likely to suffer both greater offshore losses and rollover in storm events than these locations.

Table 6.3: Maximum short-term erosion measured by ECan beach profiles at Gore Bay.

Profile	Feature Measured	Maximum Inter-survey change	Period of max change	Storm Notes (from ECan Storm register)
HCH5658	Scarp top	-2.4 m	November 1994-December 1995	Pre-Storm Register, no antidotal records of storms within this period.
HCH5667	3m Contour	-2.5 m	December 1993-November 1994	Pre-Storm Register but from antidotal records there were four minor storms within this period.
HCH5675	3m Contour	-4.5 m	December 1993-November 1994	
HCH5700	Back of Beach Toe	-2.6 m	November 2000 – December 2001	One storm in July 2001
HCH5711	3m Contour	-9.3 m	December 2002 – December 2003	3 events: July, Sept, Oct 2003
HCH5722	3m Contour	-4.5 m	May 2016 – June 2017	5 events: Sept 2016, Jan (2), Apr, May 2017
HCH5736	3m Contour	-9.5 m	November 2000 – December 2001	One storm in July 2001
HCH5747	3m Contour	-5.5 m	December 2010-December 2011	4 events: Apr, Jul, Aug (2) 2011
HCH5765	3m Contour	-4.5 m	May 2016-June 2017	5 events: Sept 2016, Jan (2), Apr, May 2017
HCH5782	3m Contour	-4 m	December 1997 – November 1998	Pre-Storm Register, no antidotal records of storms within this period.
HCH5867	3m Contour	-5.5 m	December 2001-December 2002	7 events: Feb, Apr, May, Jun, Jul, Aug, Nov 2002

6.2.4 Projected Future Shoreline Position (PFSP)

From the combination of the above information on the individual components, the resulting distances from the current shoreline to the PFSP under the RCP 8.5 and RCP 8.5+ scenarios at selected transects are presented below in Table 6.4. Full details of the components at all transects are presented in Appendix G and the ground position in relation to the settlement is shown in Appendix F.

Due to the confined nature of the settlement between the coastline and steep cliffs, the PFPS distance for the RCP8.5+ scenario have been truncated to the base cliff where required as different erosional process would occur when the shoreline approached this location, that have not been accounted for in this assessment.

Table 6.4: Distances from current shoreline to PFSP at Gore Bay. (Distances rounded to nearest metre).

Timeframe	**30 years (2050)**		**50 years (2070)**		**100 years (2120)**	
Scenario	**RCP 8.5**	**RCP 8.5+**	**RCP 8.5**	**RCP 8.5+**	**RCP 8.5**	**RCP 8.5+**
Transect 127 (Southern end of the settlement)	-24 m	-34 m	-35 m	-53 m	-84 m	-118 m
Transect 118	-22 m	-32 m	-32 m	-50 m	-77 m	-110 m
Transect 111	-23 m	-32 m	-33 m	-50 m	-78 m	-110 m
Transect 101 (Northern end of the settlement)	-26 m	-35 m	-38 m	-55 m	-88 m	-120 m
Transect 87 (North of Jed River)	-40 m	-50 m	-62 m	-79 m	-137 m	-169 m

It is noticeable from these results that for all parts of the settlement coastal frontage south of the Buxton River (e.g. Transect 101-130), that within 30 years the erosion due to accelerated SLR is predicted to outstrip the advance due to sediment supply, resulting in net erosion occurring. For these sites SLR contributes 65-85% of the distance to the PFSP over the next 30 years, and 75 to 95% over the next 50 years.

For the transects north of the Buxton River, predicted erosion distances are greater due to the added contribution of the extrapolation of the current long-term erosion rate. However, even within 30 years the effects of accelerated SLR will contribute more to the position of the PFSP than the extrapolation of current rates, and by 50 years will be contributing up to 855 of the predicted erosion.

6.2.5 Coastal Erosion Risk

As can be seen from the mapping of the PFSP lines in Appendix F, under the 2050 and 2070 SLR scenarios properties along the southern coastal section of Cathedral Rd and northern parts of Gore Bay Rd are predicted to be at risk from coastal erosion. Under the 100-year scenarios, properties right throughout the settlement will be at risk. As summarised in Table 6.5 below, 13% to 24 % of the total properties of the settlement will be at risk by 2050, 25-35% by 2070, and up to 100% by 2120 when at the worst case under RCP85+ scenario the shoreline is predicted to be along the existing cliff line.

No critical infrastructure at Gore Bay was required to be assessed. However, as discussed above, around 300 m of Cathedral Rd at the southern entrance to the settlement and around 170 m of the southern section of Gore Bay Rd are mapped as be at risk from coastal erosion by 2050, with almost all the roading network through the settlement affected by 2120.

Table 6.5: Number of properties affected by coastal erosion in future SLR scenarios.

Timeframe	Total	30-year (2050)		50-year (2070)		100-year (2120)	
Scenario		RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Number of properties	106	14	25	25	37	63	106

6.3 Coastal Inundation Hazard Assessment

6.3.1 Bathtub Model Results

Coastal inundation bathtub model maps for each settlement under current and future SLR scenarios are presented in Appendix H, with the results for Gore Bay summarised below in Table 6.6. As can be seen from these maps and results, wave run-up overtopping the beach ridge, particularly in the southern part of the settlement could add considerable area to the potential inundation extent.

Table 6.6: Summary of spatial extent of potential inundation hazard in Gore Bay.

Scenario	Present Day (2020)	30-year (2050)	50-year (2070)		100-year (2120)	
Scenario and 1% AEP static water levels[1]	**3.26m (south) 3.41m (north)**	**RCP 8.5 3.49m (south) 3.64m (north)**	**RCP 8.5 3.66m (south) 3.81m (north)**	**RCP 8.5+ 3.82m (south) 3.97m (north)**	**RCP 8.5 4.27m (south) 4.42m (north)**	**RCP 8.5+ 4.57m (south) 4.72m (north)**
Approx % of settlement inundated with static water level	10%	10%	15%	15%	20%	20%
Average inundation Depth	0.3m	0.6m	0.7m	1m	1.2m	1.3m
Approx % of settlement inundated including potential run-up	35%	35%	35%	45%	45%	45%
[1] 1% AEP static water level = Storm Tide (ST) + wave set-up (WS). All water levels are given in terms of Lyttleton Vertical Datum 1937 (LVD)						

At current sea level, the 1% AEP static water level could enter the northern part of the settlement footprint by over topping the low ridge in front of the combined mouths of the Buxton Creek and the Jed River. Under the bathtub modelling approach the northern part of the settlement along Gore Bay Rd and the Buxton Campground would be susceptible to inundation, with average water depths in the order of 0.3 m. However, as documented in Section 2.4, the bathtub method produces very conservative results as it does not account for temporal variances of the event, or any hydrodynamic factors. Never-the-less, as shown in the mapping in Appendix H, wave run-up overtopping could add considerable inundation volume, increasing the inundation extent at the northern end of the settlement to cover around 35% of the settlement, and increasing inundation depths. Through the central and southern parts of the settlement the presence of the high former beach ridge or small cliff behind the current beach is of sufficient elevation to prevent overtopping water entering this part of the settlement.

A similar extent of potential inundation is modelled for SLR over the next 30 years, but average inundation depths from static water level could increase to around 0.6m. Wave run-up overtopping of the low beach ridge at the northern end of the settlement would have a similar impact on inundation extents and depths as in the present-day scenario.

Under the 50-year scenarios, the 1% AEP static water level would increase to 3.81 m LVD under RCP8.5 and 3.97 m LVD under RCP8.5+ scenario. Under the conservative bathtub approach these water levels would inundate 15% of the settlement footprint, all at the northern end, with average depths in the order of 0.7 m to 1 m. Wave run-up over-topping would add similar extents and depths to the northern inundation area as under the previous scenarios, but for the RCP8.5+ scenario could also overtop the high former ridge and small back beach scarps in the southern and central parts of the settlement, increasing the potential inundation extent to around 45% of the total settlement footprint.

Under both the 100-year SLR scenarios, 20% of the settlement is below the projected 1%AEP water levels of 4.3 m to 4.7 m LVD, with average inundation depths in both parts of the settlement being in excess of 1 m. Wave run-up could reach elevations up to 3.5 m higher than static water levels, resulting in overtopping right along the settlement footage resulting in the potential inundation extent increase to 45% of the settlement footprint, and increasing the depth of inundation.

6.3.2 Coastal Inundation Risk

Dwellings and Properties

As shown in Table 6.7, under current sea levels only two dwellings and four properties at Gore Bay intersect with the coastal inundation hazard footprint for static water levels, which increases to 8 dwellings and 13 properties respectively when including the potential additional inundation from wave run-up overtopping at along the southern end of the settlement. The impact of SLR on inundation risk is clearly demonstrated in Table 6.7, with the number of both dwellings and properties at potential risk nearly doubling with rise over the next 30 years. With SLR over 100 years, nearly 40 % of the current dwellings and 50% of properties could be at potential risk.

Table 6.7: Total number of dwellings and properties which intersect with the inundation hazard footprint

Timeframe	Total	Present Day (2020)	30-year (2050)	50-year (2070)		100-year (2120)	
Scenario			RCP 8.5	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
1% AEP Static Water Level Inundation							
Dwellings	92	2	3	7	8	8	8
With run-up	92	8	14	14	22	28	34
Properties	106	4	5	5	8	8	8
With run-up	106	13	25	25	39	50	51

Critical Infrastructure

There was no critical infrastructure required to be assessed at Gore Bay. However, the mapping in Appendix H indicates that the northern entrance to the settlement via Gore Bay Rd is potentially at risk from inundation in current day 1% AEP storm conditions with inundation depths up to 0.2 m and increasing to 1 m with 100 years of SLR. At the southern entrance to the settlement parts of Cathedral Rd are shown to be at risk from inundation by 1% AEP storm wave run-up overtopping under the 50-year RCP8.5+ scenario, and under both 100-year SLR scenarios.

6.3.3 Recommended Further Inundation Modelling

Given the results of the bathtub modelling, it is recommended that further hydrodynamic modelling of the inundation hazards at Gore Bay **is warranted** to better quantify the threshold for overtopping and inundation, the spatial extent and magnitude (e.g. inundation depths) of the hazard, the risks posed to the dwellings, the road access to the settlement.

6.3.4 Change in Annual Recurrence Interval

As well as water levels, future SLR will also increase the annual probability that the present day 1% AEP event will occur. As shown in Figure 6.4, the Annual Recurrence Interval (ARI) for the present day 1% AEP event magnitude reduces from the current 100 years to 30-45 years by 2050, to 10-20 years by 2070, and to 1-3 years by 2120. Expressed another way, this magnitude event is more than twice as likely to occur in any one year by 2050 under both SLR scenarios and could become an annual occurrence by 2120 under the more extreme RCP8.5+ scenario.

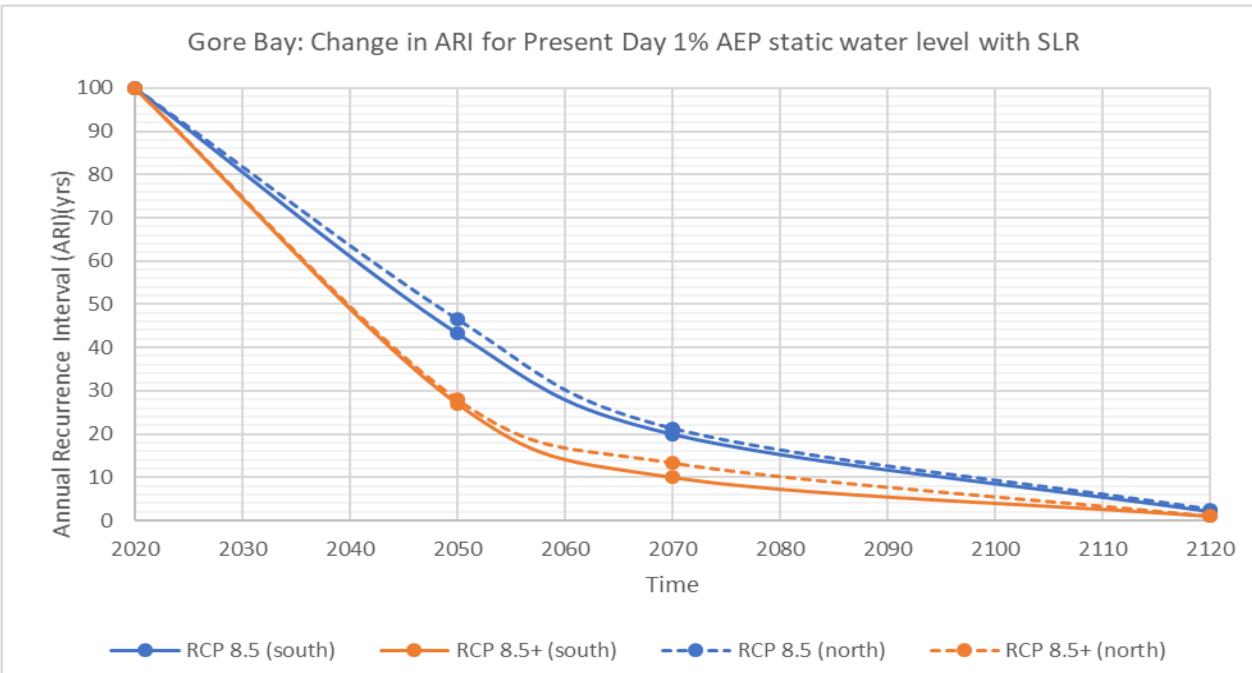

Figure 6.4 Effect of SLR on Annual Recurrence Interval for Present Day 1% AEP static water event for Gore Bay north and south.

6.4 Rising Groundwater Hazard Assessment

6.4.1 Existing Groundwater Conditions

The geology of the raised land behind the narrow Gore Bay coastal plain is Tertiary (Pliocene and Miocene) and Cretaceous Period sedimentary lithologies of Motunau Group, Eyre Group, and Pahu Terrain.

The narrow Gore Bay coastal plain comprises Holocene sands, gravels and alluvium transported north from the Hurunui River, and sourced from local rivers, the Jed River and to a lesser extent from the Buxton River. There is little data indicating the thickness of the sediments. Well O33/0007 indicates an alluvial thickness of only 3.6 m overlying mudstone and limestone, however, this well is located close to Tertiary outcrops west of the settlement. Given the narrow nature of the plain a maximum thickness of the order of 10 to 15 m has been inferred.

Groundwater flow is inferred to be toward the coast, however there is little data available to indicate groundwater elevation and flow direction.

The lower Jed River and coastal plain are mapped as semi-confined or unconfined aquifers on the Canterbury Maps Viewer and are likely to host moderate permeability, several orders of magnitude greater than the underlying lithologies.

6.4.2 Adopted Parameters

The following parameters were adopted for incorporation in the AnAqSim groundwater model:

- Quaternary alluvial and beach deposits hydraulic conductivity assumed at 20 m/day;
- Tertiary formations simulated as unconfined/confined aquifer with hydraulic conductivity assumed at 1 m/day;
- Recharge rate – 100 mm/year to alluvium and 36.5 mm/year to Tertiary to reflect low permeability formation.
- Jed River and inlet, and Buxton River simulated as specified head boundaries based on LiDAR elevations.

6.4.3 Rising Groundwater Mapping

Maps of depths to the indicative average shallow groundwater conditions at Gore Bay under present day sea levels and future SLR are presented in Appendix I. The predicted saline interface is shown in Figure 6.5.

The majority of the Gore Bay settlement is relatively elevated apart from around the combined mouth of the Buxton Creek and the Jed River, depth to groundwater being > 1 m under all SLR scenarios over the whole settlement except for a small area at Buxton Campground under the 100-year RCP8.5+ scenario.

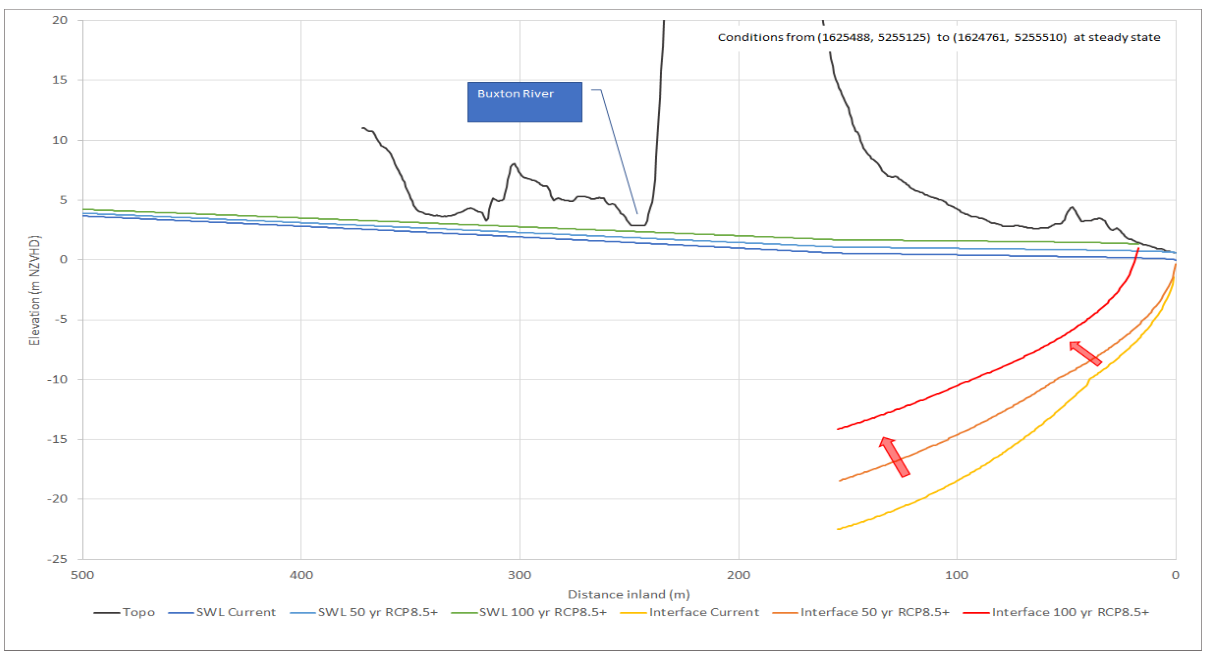

Figure 6.5: Gore Bay simulated water levels and saline interface

6.4.4 Rising Groundwater Risk

The number of dwellings exposed to different groundwater depths with present and future sea levels is presented in Table 6.8. Note that where a dwelling covers two or more depth categories, the shallowest depth has been applied.

No dwellings are predicted to be impacted by groundwater shallower than 1m BGL. The number of dwelling predicted to have groundwater in the range 1 to 2m BGL increases from zero to 7 with SLR over the next 100 years.

There was no critical infrastructure required to be assessed at Gore Bay.

Table 6.8: Number of dwellings exposed to indicative average groundwater depths at Gore Bay.

SLR Scenario	Depth to Groundwater (m BGL)			
	≤ 0.5m	0.5-1m	1-2m	> 2m
Present Sea level (2020)	0	0	0	92
50 year (0.6 m SLR)	0	0	1	91
100 year (1.3 m SLR)	0	0	7	85

7. Conway Flat

7.1 Settlement Description

The area of Conway Flat covered in this assessment was the alluvial coastal cliffs forming the high terrace to the south of the Conway River along which Conway Flat Rd runs very close to the cliff edge as shown in Figure 7.1 and 7.2(a). The Council has identified this road as being critical infrastructure as it is the only access route to the farming community. The road is protected from erosion in places by the placement of armour rock on the cliff face and base (Figure 7.2(b).

There are no dwellings or properties assessed along this 1.5km stretch of shoreline, however it is noted that there is farm buildings and sparsely distributed dwellings along the coastal hinterland, especially at the southern end of the assessment area.

Figure 7.1: Conway Flat overview of assessment footprint and critical infrastructure.

Figure 7.2: Conway Flat Road: (a)Road close to alluvial cliff edge; (b) Armour rock protection on cliff face and toe.

7.2 Coastal Erosion Hazard Assessment

7.2.1 Historical Long-term Shoreline Movements

Jacobs (2017) undertake an assessment of historical coastal erosion rates along the Conway Flat Rd and the impact of the North Canterbury Earthquakes in November 2016 on coastal erosion as part of Council decision making on the best form of protection and likely lifetimes for the road. The historical erosion assessment involved DSAS analysis of historical shorelines from aerial imagery between 1969 and 2015, with the earthquake impact being assessed from aerial images between January 2015 and immediately post-earthquake in November 2016.

For this current assessment the historical shoreline analysis was extended back to 1950 aerial imagery to give a longer period and therefore more representative assessment of historical erosion rates. The most recent photographs used in the assessment were 2015, with the 2016 earthquake effects considered under short-term episodic effects. The analysis is covered by DSAS transects 25 (north) to 77 (south), with a map of the results being presented in Appendix B.

The resulting erosion rates from this latest analysis were similar but generally slightly lower than those obtained by Jacobs (2017) over the shorter period. Average rates along the alluvial cliffs where Conway Flat Rd is closest to the edge (e.g. Transects 38-50) were -0.11 m/yr since 1950 compared to -0.13 m/yr from Jacobs (2017), while along the loess cliff section (Transects 54-67) were similar across both assessments at -0.07 m/yr. The maximum erosion rate at an individual transect is -0.38 m/yr at Transect 50, located at the southern edge of the high alluvial terrace, and most likely influenced by stream erosion from the watercourse that discharges at this location.

The projected future shoreline retreat from the extrapolation of these historical rates is presented in Table 7.1.

Table 7.1: Projected shoreline advances from extrapolation of rates from 1950 to 2015 for selected DSAS Transects at Conway Flat Road

Scenario	30 years (2050)	50 years (2070)	100 years (2120)
Transect 35 (alluvial cliff north of road)	-2.1 m	-3.5 m	-7.0 m
Transect 45 (alluvial cliff)	-3.3 m	-5.5 m	-11 m
Transect 60 (loess cliff)	-1.8 m	-3.0 m	-6 m

Transect 70 (loess cliff-southern end of road)	-3.3 m	-5.5 m	-11 m

7.2.2 Accelerated Sea Level Rise Effects

The effects of projected accelerated SLR on erosion of the coastal cliffs at Conway Flat was calculated using the modified Walkden & Dickson (2008) equation for alluvial cliffs as set out in section 2.3.2 and Appendix D. The results are summarised below in Table 7.2.

As can be seem from Table 7.2, the increase in erosion as a result of the accelerated SLR is small, being generally less than 1 m within 30 years, 1-3 m in 50 years, 2-7 m in 100 years. For all sites the erosion due to accelerated SLR are less than from the extrapolation of the long-term erosion. However, all indicate that the magnitude of the erosion is very sensitive to which SLR scenario is applied with erosion distances approximately doubling between the RCP8.5 scenario and the RCP8.5+ scenario within the same timeframe.

Table 7.2 : Calculated erosion distance due to accelerated SLR at Conway Flat.

Transect	Scenario	30 years (2050)		50 years (2070)		100 years (2120)	
		SLR	Erosion Dist	SLR	Erosion Dist	SLR	Erosion Dist
Transect 35 (Northern end of the road along the cliff)	RCP8.5	+0.23 m	-0.3 m	+0.40 m	-0.8 m	+1.01 m	-2.8 m
	RCP8.5+	+0.32 m	-0.8 m	+0.56 m	-1.5 m	+1.31 m	-4.1 m
Transect 45	RCP8.5	+0.23 m	-0.6 m	+0.40 m	-1.4 m	+1.01 m	-4.9 m
	RCP8.5+	+0.32 m	-1.3 m	+0.56 m	-2.7 m	+1.31 m	-7.1 m
Transect 60	RCP8.5	+0.23 m	-0.3	+0.40 m	-0.7	+1.01 m	-2.6 m
	RCP8.5+	+0.32 m	-0.7 m	+0.56 m	-1.4 m	+1.31 m	-3.9 m
Transect 70 (Southern end of road along the cliff)	RCP8.5	+0.23 m	-0.6 m	+0.40 m	-1.3 m	+1.01 m	-4.6 m
	RCP8.5+	+0.32 m	-1.3 m	+0.56 m	-2.5 m	+1.31 m	-6.7 m

7.2.3 Short-term Storm and Earthquake Effects

One Environment Canterbury beach profile (HCK8510) is located along the Conway Flat cliffs that can be used to estimate the magnitude of short-term storm erosion of the cliff position. As shown in Table 7.3, the maximum inter-survey erosion ranged from -1.22m (Dec 2014- Dec 2015) using the cliff top and -2.51m using the cliff toe.

Table 7.3: Maximum short-term erosion measured by ECan beach profile at Conway Flat.

Profile	Feature Measured	Maximum inter-survey Change	Period of max change	Storm Notes (from ECan Storm register)
HCK8510	Toe	-2.51 m	January 2014 - December 2014	5 events: Jan, Mar, April, Jun, Aug 2014
	Cliff top	-1.22 m	December 2014 - December 2015	7 events: Feb, Apr (2), May, Jun, Jul, Sept 2015, of which May is the most extreme and April the 4th on the register (1999-2019)

The Jacobs (2017) assessment found that the erosion of the alluvial cliff edge from January 2015 to November 2016 averaged -3 m with a maximum of -5 m, the majority of which is attributed to the effect of the November 2016 North Canterbury Earthquake on the unconsolidated cliff material. However, since this period also included the 7 storm events listed in Table 7.3 plus an additional 2 in 2016 prior to November 2016, it is not possible to determine accurately the earthquake effect.

Based on the above findings, an arbitrary value of 3m has been adopted as the short-term erosion component from both storms and earthquakes for the PFSP for Conway Flat Rd.

7.2.4 Projected Future Shoreline Positions (PFSP)

From the combination of the above information on the individual components, the resulting distances from the current shoreline to the PFSP under the RCP 8.5 and RCP 8.5+ scenarios at selected transects are presented below in Table 7.4. Full details of the components at all transects are presented in Appendix G and the ground position in relation to the settlement is shown in Appendix F.

Table 7.4: Distances from current shoreline to PFSP at Conway Flat. (Distances rounded to nearest metre).

Timeframe	30 years (2050)		50 years (2070)		100 years (2120)	
Scenario	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Transect 35 (Northern end of the road along the cliff)	-5 m	-6 m	-7 m	-8 m	-12 m	-14 m
Transect 45	-7 m	-8 m	-10 m	-11 m	-19 m	-22 m
Transect 60	-5 m	- 6 m	-7 m	-8 m	-12 m	-13 m
Transect 70 (Southern end of road along the cliff)	-7 m	-8 m	-10 m	-11 m	-18 m	-22 m

As can be seen from Table 7.4, the projected erosion distances are not large, being less than 10 m within the next 30 years, up to approximately 10 m by 2070 and in the range 12-22 m by 2120 depending on the SLR scenario.

From comparing the individual component results with the over-all projected erosion distances, the accelerated SLR will only contribute < 20% of the projected erosion by 2050, in the order of 15-24% by 2070, and 25-33% by 2120. In the immediate future the largest contribution to the position of the PFSP is short-term episodic erosion from storms or earthquakes, with the extrapolation of current long-term rates becoming the dominant factor within 50 years.

7.2.5 Coastal Erosion Risk

The purpose of the risk assessment is the 1.5 km stretch of Conway Flat Rd that fronts the shoreline. The resulting lengths of road at risk under the different SLR scenarios are presented in Table 7.5, which indicates 35% of the road length is at risk within 30 years, increasing to 65% in 50 years, and 85% within 100 years.

Table 7.5: Lengths of Conway Flat. Road at potential risk from coastal erosion.

Timeframe	Total	30-year (2050)	50-year (2070)		100-year (2120)	
Scenario		RCP 8.5	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Indicative length of road affected	1500m	550m	950m	1000m	1300m	1300m

7.3 Coastal Inundation Hazard Assessment

7.3.1 Bathtub Model Results

Coastal inundation bathtub modelling maps for each settlement under current and future SLR scenarios are presented in Appendix H.

Due to being a cliff environment, any potential inundation hazard is limited to the mouths of the numerous small streams and watercourses than discharge to the beach fronting the cliffs. Under current day and 30-year SLR scenarios any inundation would be as a result of wave run-up overtopping the beach barrier fronting these streams and watercourses.

Under the 50-year RCP 8.5 scenario the 1% AEP static water level could extend up small coastal inlets and may affect approximately 20 m Conway Flat Rd at the southern end of the study area with 0.2m water depth. In the 50-year RCP 8.5+ scenario, the spatial extent of the hazard increases in these isolated inlets and the depth also increases. Approximately a 50m section of the road at the northern extent of the study area could also become inundated but only with 0.1m of water, and the inundation at the previously mentioned southern road section increases to 0.4m water depth.

In the 100-year RCP 8.5 scenario, the spatial extent of the inundation footprint continues to increase, and the potential depth across Conway Flat Rd could increase to 0.5-0.8 m. In the 100-year RCP 8.5+ scenario, the hazard extent continues to increase, and the depth of water across the road at the coastal inlets could increase to 0.8-1m.

7.3.2 Coastal Inundation Risk

Dwellings and Properties

There are no dwellings or properties within the inundation areas along Conway Flat Rd.

Critical Infrastructure

Conway Flat Rd was the only assessed piece of critical infrastructure in this area. As indicated in Table 7.6, only two locations where small streams or water courses cross the road are at risk from inundation in a 1% AEP storm event with SLR.

Table 7.6: Potential Inundation depth at small inlets on Conway Flat Rd from Bathtub modelling of 1% AEP event

Infrastructure	Present day (2020)	30-year (2050)	50year (2070)		100year (2120)	
Infrastructure		**RCP 8.5**	**RCP 8.5**	**RCP8.5+**	**RCP 8.5**	**RCP8.5+**
Road (% of total road affected)	0%	0%	2%	3%	3%	3%
Road (average inundation depth)	Not inundated	Not inundated	0.2m	0.4m	0.8m	1.1m

7.3.3 Recommended Further Inundation Modelling

Given the results of the bathtub modelling, it is recommended that further hydrodynamic modelling of the inundation hazards at Conway Flat **is not warranted** as the footprint of potential inundation being limited to the small coastal stream and water course inlets.

7.3.4 Change in Annual Recurrence Interval

As well as water levels, future SLR will also increase the annual probability that the present day 1% AEP event will occur. As shown in Figure 7.3, the Annual Recurrence Interval (ARI) for the present day 1% AEP event magnitude (e.g. 2.95 m static water level) reduces from the current 100 years to 12-16 years by 2050, to 5-10 years by 2070, and to 1 year by 2120. Expressed another way, this magnitude event is five to ten times as likely to occur in any one year by 2050 and could become an annual occurrence by 2120 under both SLR scenarios.

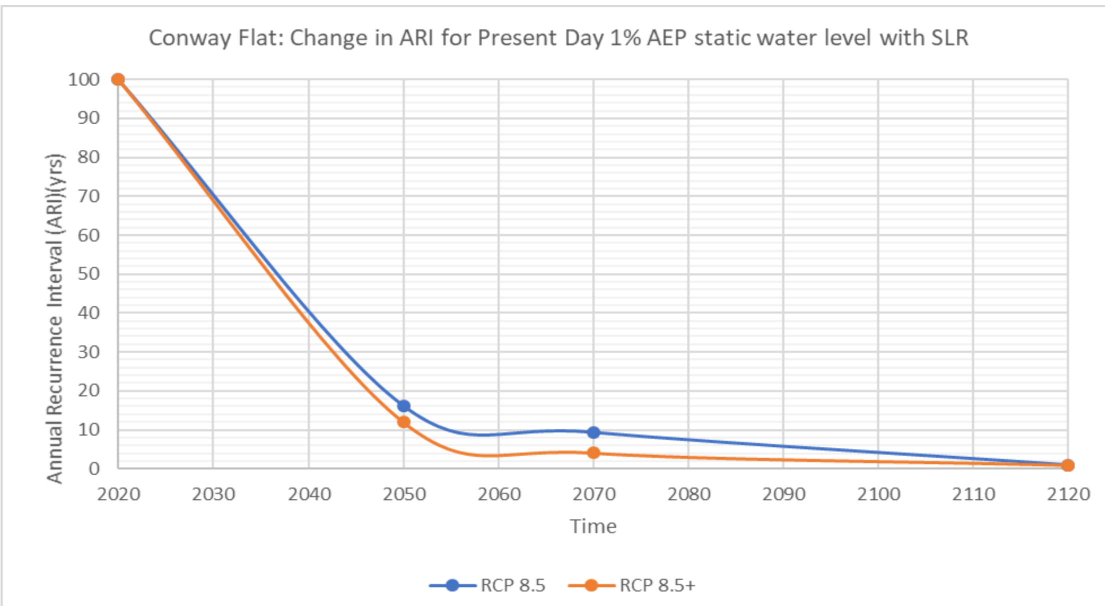

Figure 7.3: Effect of SLR on Annual Recurrence Interval for Present Day 1% AEP static water event at Conway Flat.

7.4 Rising Groundwater Hazard Assessment

7.4.1 Existing Groundwater Conditions

The Conway Flat Rd is located on the western edge of a more substantial alluvial coastal plain than Gore Bay, but is similarly bounded inland and underlain by Tertiary (Pliocene and Miocene) and Cretaceous Period sedimentary lithologies of Motunau Group, Eyre Group, and Pahu Terrain.

The coastal plain comprises beach gravels and sands and alluvium from the Conway River, and smaller drainages. There is little data indicating the thickness of the sediments. Well O32/0084 situated on the alluvial plain north of the Conway River at an elevation of approximately 11 m MSL indicates a depth of alluvium to 27.5 m below ground. It is not indicated if the hole bottomed out on Tertiary sediments. Depth to water at O32/0084 is approximately 7m below ground (4m MSL). In the southern area of the coastal plain well O32/0088, at an elevation of approximately 16m (MSL) indicates Quaternary gravel sand to 30 m depth. The hole was drilled to a depth of 120 m and intersected alternating clay and gravel layers. It is unclear whether these are analogous with the glacial and interglacial cycles of the Christchurch Aquifers. No significant yields were indicated on the bore log. No water level data is available for O32/0088.

The lower Conway River and coastal plain are mapped as semi-confined or unconfined aquifers on the Canterbury Maps Viewer and are likely to host moderate permeability, several orders of magnitude greater than the underlying sedimentary lithologies.

Groundwater flow within the alluvial aquifer will be towards the coast, and it is anticipated that the Conway River will be a source of groundwater recharge.

7.4.2 Adopted Parameters

The following parameters were adopted for incorporation in the AnAqSim groundwater model:

- Quaternary alluvial and beach deposits hydraulic conductivity assumed at 20 m/day;
- Tertiary formations simulated as unconfined/confined aquifer with hydraulic conductivity assumed at 1 m/day;
- Recharge rate – elevated recharge 219 mm/year applied to alluvium to account for recharge from Conway River, and 36.5 mm/year to Tertiary to reflect low permeability formation.
- Inland boundary conditions set to indicative regional water levels.

7.4.3 Rising Groundwater Mapping

Maps of depths to the indicative average shallow groundwater conditions at Conway Flat under present day and future sea levels in 100 years are presented in Appendix I. Note that since the RCP 8.5+ 100yr SLR scenario does not show groundwater depths being < 2m BGL at the settlement, there are not considered to be any significant impact of rising groundwater, so the intermediate 50-year scenario was not run. The predicted saline interface is shown in Figure 7.4.

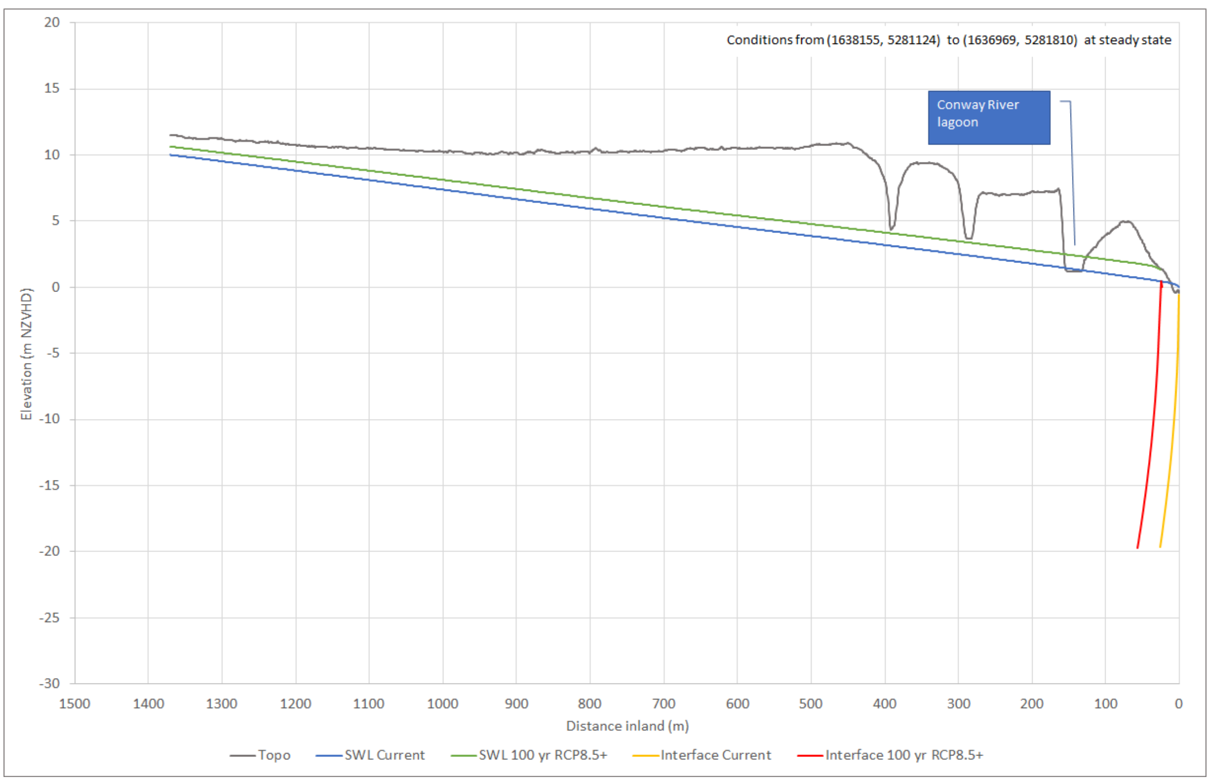

Figure 7.4: Conway Flats simulated water levels and saline interface with SLR.

7.4.4 Rising Groundwater Risk

Predicted groundwater depths along the coastal margin of Conway Flat Road are provided in Table 7.7. As indicated the road is not predicted to be significantly impacted by groundwater with water levels being 2.5 m BLG even with SLR over 100 years.

Table 7.7: Indicative average groundwater depths at critical infrastructure in Conway Flat (m BGL)

Infrastructure	Present Day (2020)	100-year (2120) RCP 8.5+
Road	2-5m	2-5m

8. Claverley

8.1 Settlement Description

As shown in Figure 8.1 below, Claverley is a small settlement of 13 dwellings and 27 residents[6], situated on the low northern river mouth terrace of the Conway River, approximately 2.5 km north of the river. The beach at Claverley is Mixed Sand and Gravel (MSG) with a high gravel storm ridge up to 7 m LVD and based on google earth images a steep nearshore step on which waves break in all conditions.

The key piece of infrastructure of interest to the council is Claverley Road which runs along the back of the beach to the north of the settlement and passes under a rail bridge approximately 500 m north of the settlement (Figure 8.2). The road at this underpass has been subjected to inundation and erosion previously.

Figure 8.1: Claverley settlement overview of settlement footprint and critical infrastructure.

[6] Taken from the New Zealand Census Data (2013) as provided by HDC.

Figure 8.2: Rail bridge over Claverley Road 500 m north of Claverley settlement (photo DTec, 2004)

8.2 Coastal Erosion Hazard Assessment

8.2.1 Historical Long-term Shoreline Movements

A previous analysis of shoreline stability at Claverley by DTec (2004) found that five independent lines of evidence all indicated that the shoreline that been accreting since 1897, with rates of advance in the order of 0.5m/yr to 0.6 m/yr being measured from aerial photograph analysis between 1950 and 2003.

For this assessment historical erosion was measured from five aerial photographs between 1950 and 2019 (see Table 2.1), and DSAS transects from 1 (north) to 21 (south), with Transects 12-19 representing the 350m of shoreline in front of the settlement and Transects 9-3 the section of Claverley Rd running behind the beach to the rail underpass. A map of the resulting shoreline positions and net change being presented in Appendix B, and trends of movements at selected transects being presented in Figure 8.3.

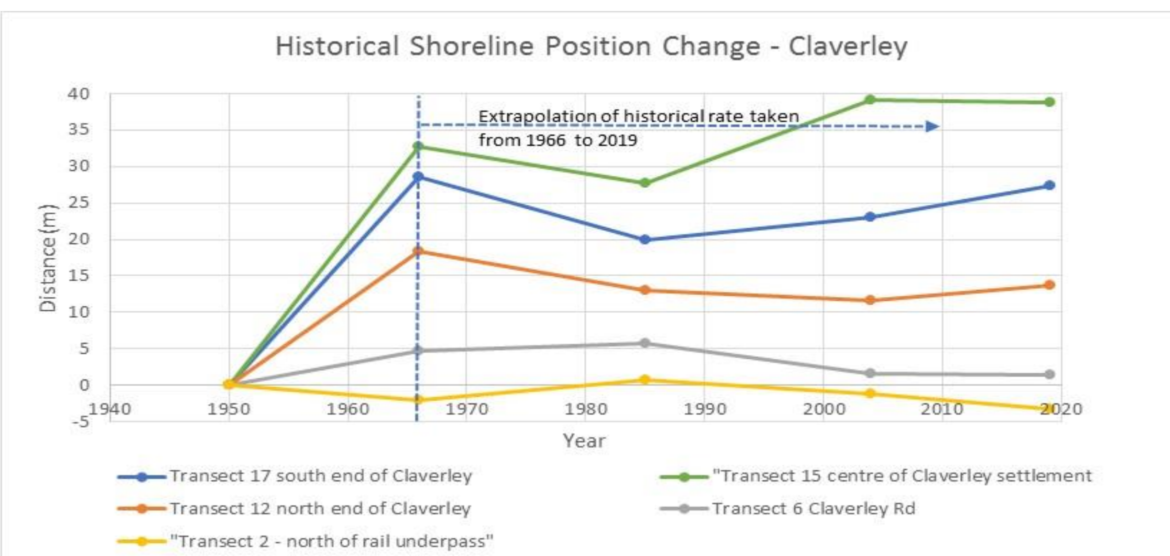

Figure 8.3: Historical shoreline position change for selected DSAS Transects at Claverley 1950-2019

As can be seen from Figure 8.3, all transects except those north of the rail underpass has experienced a net accretion since 1950, but with generally stable trends with small scale dynamic fluctuations since 1966. The reason for the change from relatively rapid net accretion in the 1950-1966 period to a more dynamic equilibrium is not clear, as there is no indication in reduction of sediment supply from the Conway River.

However, as a conservative approach to the extrapolation of historical rates for input into the determination of the PFSP position, only shoreline advance rates since 1966 have been used. The resulting projected shoreline advance distances from extrapolating these rates 30, 50 and 100 years into the future are presented in Table 8.1.

Table 8.1: Projected shoreline advances from extrapolation of rates from 1966 to 2019 for selected DSAS Transects at Claverley.

Scenario	30 years (2050)	50 years (2070)	100 years (2120)
Transect 17 (Southern end of settlement)	+2.4 m	+4.0 m	+8.0 m
Transect 15 (Centre of settlement)	+0.3 m	+0.5 m	+1.0 m
Transect 12 (Northern end of settlement)	-0.6 m	-1.0 m	-2.0 m
Transect 6 (Claverley Rd north of settlement)	-2.1 m	-3.5 m	-7.0 m
Transect 2 (north of rail underpass)	-0.6 m	-1.0 m	-2.0 m

8.2.2 Accelerated Sea Level Rise Effects

The effects of projected accelerated SLR on erosion of the beach at Claverley was calculated using the modified Bruun rate for MGS beaches as set out in Section 2.3.2 with closure depth at the base of the nearshore being assumed from nearshore surveys of this feature at Washdyke, Timaru to be 5 m (below MSL) with a 1:10 slope. Beach crest height from Environment Canterbury profile surveys (profile HCK9150) was set at 7.8 m (AMSL), which combined with the shallow closure depth and steep slopes resulted in minimal effects of accelerated SLR on future coastal erosion over 30, 50, and 100-year timeframes as presented in Table 8.2.

Table 8.2: Calculated erosion distance due to accelerated SLR at Claverley (Profile HCK9150).

Scenario	30 Year (2050)		50 Year (2070)		100 Year (2120)	
	SLR	Erosion Dist	SLR	Erosion Dist	SLR	Erosion dist
RCP8.5	+0.23 m	-1.6 m	+0.40 m	-2.9 m	+1.01 m	-8.0 m
RCP8.5+	+0.32 m	-2.6 m	+0.56 m	-4.6 m	+1.31 m	-11.2 m

8.2.3 Short-term Storm Effects

The short term storm effect measured from one Environment Canterbury profile north of the settlement showed that between 1997-2019, the maximum inter-survey change was -3.8 m at the landward beach toe, as seen in Table 8.3. Adopting the upper rounded limit of this observation, an arbitrary value of -4 m has been adopted as the short-term erosion component for the PFSP calculation for Claverley.

Table 8.3: Maximum short-term erosion measured by ECan beach profiles at Claverley.

Profile	Feature Measured	Maximum Inter-survey Change	Period of max change	Storm Notes (from ECan Storm register)
HCK9150	Landward beach Toe	-3.8 m	February 2010 - January 2011	2 events: May & Aug 2010

8.2.4 Projected Future Shoreline Positions (PFSP)

From the combination of the above information on the individual components, the resulting distances from the current shoreline to the PFSP under the RCP 8.5 and RCP 8.5+ scenarios at selected transects are presented below in Table 8.4. Full details of the components at all transects are presented in Appendix G and the ground position in relation to the settlement is shown in Appendix F.

The assessment shows that the future projected erosion distances are small at the Claverley settlement but to the extrapolation of the long-term accretion and the small accelerated SLR effect. The largest contribution to the position of the PFSP over the next 30 years to 50 years is the short-term storm erosion being up to or more than 50% of the total erosion distance. North of the settlement, the extrapolated long-term erosion contributes 10 –25% of the total erosion distance over the next 50 years, with accelerated SLR effects contributing 20-40%.

Table 8.4: Distances from current shoreline to PFSP at Claverley (Distances rounded to nearest metre).

Timeframe	30 years (2050)		50 years (2070)		100 years (2120)	
Scenario	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Transect 17 (Southern end of settlement)	-3 m	-4 m	-3 m	-5 m	-4 m	-7 m
Transect 15 (Centre of settlement)	-5 m	-6 m	-7 m	-8 m	-11 m	-14 m
Transect 12 (Northern end of settlement)	-6 m	-7 m	-8 m	-10 m	-14 m	-17 m
Transect 6 (Claverley Rd nth of settlement)	-8 m	-9 m	-10 m	-12 m	-19 m	-22 m
Transect 2 (north of rail underpass)	-6 m	-7 m	-8 m	-10 m	-14 m	-17 m

8.2.5 Coastal Erosion Risk

As can be seen from the mapping of the PFSP in Appendix F and Table 8.5 below, only two of the 13 properties in the Claverley settlement are projected to be at risk from coastal erosion over the next 30-50 years, increasing to eight without 100 years. For the critical infrastructure of Claverley Road, there is projected to be 150 m at risk within 50 years, and up to 250 m at risk within 100 years. The projections show that the rail bridge over Claverley Rd could be at risk from erosion processes within 30 years.

Table 8.5 Number of properties at Claverley and Claverley Rd lengths affected by coastal erosion in future SLR scenarios.

Timeframe	Total	30-year (2050)		50-year (2070)		100-year (2120)	
Scenario		RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Number of properties	13	2	2	2	2	8	8
Claverley Rd lengths		Rail bridge underpass	Rail bridge underpass	150 m	150 m	250 m	250 m

8.3 Coastal Inundation Hazard Assessment

8.3.1 Bathtub Model Results

Coastal inundation bathtub modelling maps for each settlement under current and future SLR scenarios are presented in Appendix H.

Under no scenario over the next 100 years is there any projected inundation from the bath tub modelling in the settlement or along Claverley Rd in a 1% AEP static water level scenario. When run-up overtopping is included, only small located areas north of the settlement in gullies draining to the beach are indicated as being potentially inundated in scenarios up to 100-year RCP8.5. For the 100-year RCP8.5+ SLR scenario, wave run-up overtopping during a 1% AEP storm event (projected to reach elevations in the order of 7.3 m LVD), have been modelled to potentially inundate parts of the settlement, however inundation depths have not been calculated.

8.3.2 Coastal Inundation Risk

Dwellings and Properties

As per the results above, no dwellings or properties intersect with the coastal inundation hazard footprint for 1% AEP static water level under all current or future SLR scenarios.

Critical Infrastructure

Claverley Road is not mapped as being at risk from coastal inundation under any SLR scenario.

8.3.3 Recommended Further Inundation Modelling

Given the results of the bathtub modelling, it is recommended that further hydrodynamic modelling of the inundation hazards at Claverley **is not warranted**.

8.3.4 Change in Annual Recurrence Interval

As well as water levels, future SLR will also increase the annual probability that the present day 1% AEP event will occur. As shown in Figure 8.2, the Annual Recurrence Interval (ARI) for the present day 1% AEP event magnitude (e.g. 3 m static water level) reduces from the current 100 years to 12-16 years by 2050, to 5-10 years by 2070, and to 1 year by 2120. Expressed another way, this magnitude event is five to ten times as likely to occur in any one year by 2050 and could become an annual occurrence by 2120 under both SLR scenarios.

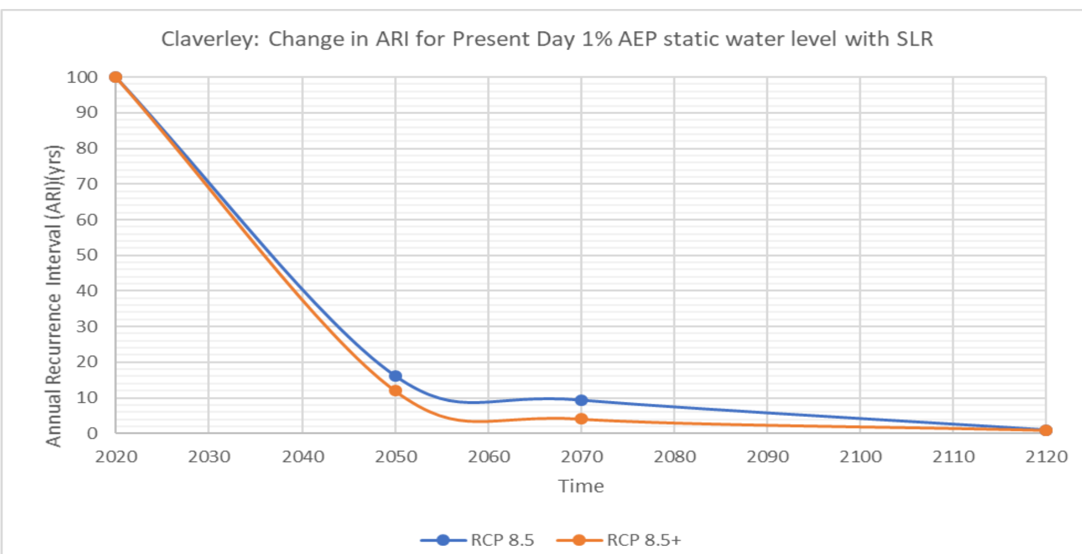

Figure 8.2: Effect of SLR on Annual Recurrence Interval for Present Day 1% AEP static water event at Claverley.

8.4 Rising Groundwater Hazard Assessment

8.4.1 Existing Groundwater Conditions

The Claverley settlement is located on a more substantial alluvial coastal plain, but similar to Gore Bay, are bounded inland and underlain by Tertiary (Pliocene and Miocene) and Cretaceous Period sedimentary lithologies of Motunau Group, Eyre Group, and Pahu Terrain.

The coastal plain comprises beach gravels and sands and alluvium from the Conway River, and smaller drainages. There is little data indicating the thickness of the sediments. Well O32/0084 situated on the alluvial plain north of the Conway River at an elevation of approximately 11 m MSL indicates a depth of alluvium to 27.5 m below ground. It is not indicated if the hole bottomed out on Tertiary sediments. Depth to water at O32/0084 is approximately 7m below ground (4m MSL). In the southern area of the coastal plain well O32/0088, at an elevation of approximately 16m (MSL) indicates Quaternary gravel sand to 30 m depth. The hole was drilled to a depth of 120 m and intersected alternating clay and gravel layers. It is unclear whether these are analogous with the glacial and interglacial cycles of the Christchurch Aquifers. No significant yields were indicated on the bore log. No water level data is available for O32/0088.

The lower Conway River and coastal plain are mapped as semi-confined or unconfined aquifers on the Canterbury Maps Viewer and are likely to host moderate permeability, several orders of magnitude greater than the underlying sedimentary lithologies.

Groundwater flow within the alluvial aquifer will be towards the coast, and it is anticipated that the Conway River will be a source of groundwater recharge.

8.4.2 Adopted Parameters

The following parameters were adopted for incorporation in the AnAqSim groundwater model:

- Quaternary alluvial and beach deposits hydraulic conductivity assumed at 20 m/day;
- Tertiary formations simulated as unconfined/confined aquifer with hydraulic conductivity assumed at 1 m/day;
- Recharge rate – elevated recharge 219 mm/year applied to alluvium to account for recharge from Conway River, and 36.5 mm/year to Tertiary to reflect low permeability formation; and
- Inland boundary conditions set to indicative regional water levels.

8.4.3 Rising Groundwater Mapping

Maps of depths to the indicative average shallow groundwater conditions at Claverley under present day sea levels and 100-year future SLR are presented in Appendix I. The predicted saline interface is shown in Figure 6.5.

Areas susceptible to groundwater level rise and potential groundwater flooding at Claverley are presented in Appendix I for both the current day and for a future 100-year scenario. Note that since the RCP 8.5+ 100yr SLR scenario does not show groundwater depths being < 2m BLG at the settlement, there are not considered to be any significant impact of rising groundwater, so the intermediate 50-year scenario was not run. The predicted saline interface is shown of Figure 8.3.

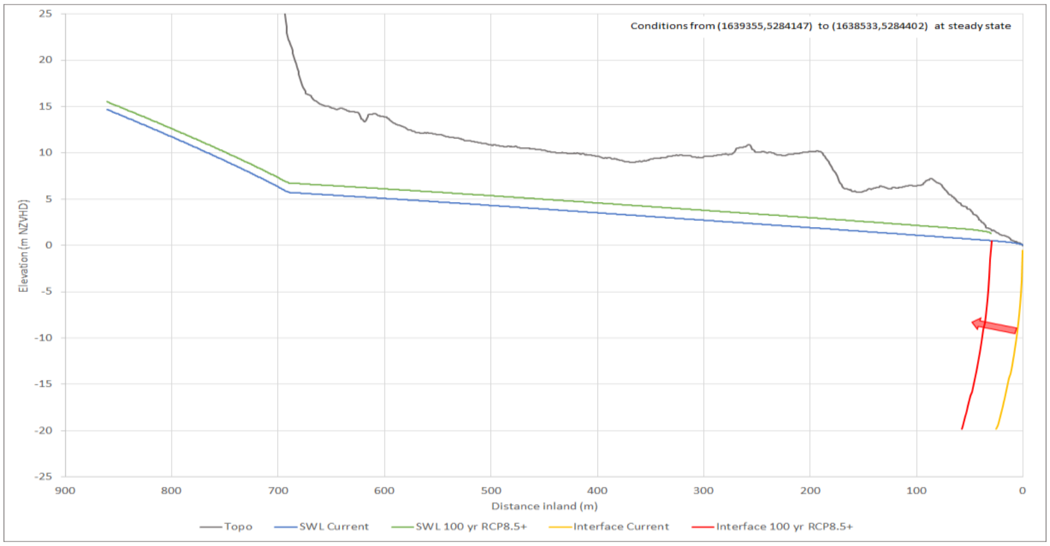

Figure 8.3: Claverley simulated water levels and saline interface with SLR.

8.4.4 Rising Groundwater Risk

As indicated above, no dwellings are projected to be at risk from rising groundwater (e.g. <0.5m BGL) under any of the SLR scenarios over the next 100 years.

Claverley Rd, as a piece of critical infrastructure is also not projected to be significantly impacted by rising groundwater levels with SLR, with the depths to groundwater along the road being in the order of 1-2 m in 100 years of projected rise as shown in Table 8.6.

Table 8.6: Indicative average groundwater depths along Claverley Road (m BGL).

Infrastructure	Present day (2020)	100-year RCP 8.5+(SLR=1.3 m)
Road	2-5m	1-2m

9. Conclusions and Recommendations

The coastal hazard assessment of the six coastal settlements within the Hurunui coastline has determined at a high-level which settlements would be most affected by coastal erosion, inundation and groundwater hazards in future events with SLR. A summary of the results is presented below, detailing the extent and potential magnitude of the hazard, as well as the risk to the settlements and critical infrastructure.

9.1 Coastal Erosion Hazard

9.1.1 Projected Future Shoreline Positions

A summary of the distances from the current shoreline to the Projected Future Shoreline Position (PFSP) are 30, 50 and 100-year SLR scenarios are presented below in Table 9.1. The results of this assessment indicate than all settlements are most likely to be subjected to erosion over all the timeframes considered, even the currently accreting shorelines at Leithfield Beach and Claverley, with a large range of erosion estimates at all settlements except Conway Flat and Claverley.

Table 9.1: Summary of distances to Projected Future Shoreline Positions (PFSP) for Hurunui coastal settlements assessed in this report.

Timeframe	30-year		50-year		100-year	
Scenario	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Leithfield Beach	-6 to -22 m	-21 to -39 m	-7 to -35 m	-25 to -54 m	-30 to -88 m	-66 to -123 m
Amberley Beach	-24 to -42 m	-27 to -45 m	-36 to 65 m	-41 to -70 m	-72 to -130 m	-81 to -140 m
Motunau	-22 to -40 m	-23 to -41 m	-34 to -62 m	-36 to -65 m	-66 to -123 m	-68 to -128 m
Gore Bay	-15 to -37 m	-25 to -46 m	-24 to -60 m	-41 to -76 m	-66 to -136 m	98m to-168 m
Conway Flat	-4 to -8 m	-5 to -8 m	-6 to -11 m	-6 to -13 m	-9 to -22 m	-10 to -24 m
Claverley	+3 to -8 m	-4 to -9 m	-3 to -10 m	-4 to -12 m	-5 to -12 m	-7 to -22 m

9.1.2 Coastal Erosion Risk

Properties

The intersection of the PFSP line and property boundaries was assessed to determine how many properties at each settlement would be at risk from coastal erosion with future SLR. These results are summarised in Table 9.2.

The results show that Amberley Beach, Motunau and Gore Bay are likely be the most affected by coastal erosion due to the close proximity of the settlements to the shorelines. In Motunau and Gore Bay, a number of dwellings will be affected in 30 years, and this incrementally increases as sea level rises to be 85% and 35% of the current dwellings at Gore Bay and Motunau respectively within 100 years under the RCP8.5+ SLR scenario. At Motunau the number of properties assessed as being at risk is likely to be an under estimate due to the lower river mouth terrace not being included in the erosion assessment. At Amberley Beach, no properties are projected to be at risk from coastal erosion within the 30-year time frame, but there are 15 mapped as being affected in 50 years, and 45 (33% of current total) within 100 years.

At Leithfield Beach the PFSP distances were in the same order of magnitude as at Amberley Beach, Motunau and Gore Bay, however there is a 200m vegetated back shore buffer between the settlement and the shoreline which is projected to provide protection for properties from coastal erosion for up to 100 years.

At Claverley, although the PFSP distances are much lower, 15% of the current properties in the settlement could be at risk from coastal erosion within 30 years, and up to 62 % are projected to be at risk within 100 years.

Table 9.2: Summary of the number of properties in each settlement likely to be affected by coastal erosion in RCP8.5 and RCP8.5+ sea level rise scenarios.

Timeframe	Total Properties	30-years (2050)		50-years (2070)		100-years (2120)	
Scenario		RCP 8.5	RCP8.5+	RCP8.5	RCP8.5+	RCP8.5	RCP8.5+
Sea Level Rise		+0.23 m	+0.32 m	+0.40 m	+0.56 m	+1.01 m	+1.31 m
Leithfield Beach	197	0	0	0	0	0	14
Amberley Beach	138	0	0	15	15	45	45
Motunau	132	>11	>11	>13	>13	>46	>46
Gore Bay	106	4	19	19	37	57	91
Conway Flat	NA	NA	NA	NA	NA	NA	NA
Claverley	13	2	2	2	2	8	8

Critical Infrastructure

Critical infrastructure at each settlement was identified by the Hurunui District Council and was included in the risk assessment.

At Leithfield Beach, the assessment identified that none of the critical infrastructure of a community water supply bore and two wet wells, would be likely to be affected by coastal erosion over the next 100 years, but the drain pipe outfall structure to the ocean could be affected by beach erosion within a 30-50 year period.

At Amberley Beach, the wet well and the waste water treatment pond were shown to not be affected by erosion over the next 100 years. However, the coastal section of Golf Links Rd and the inundation protection bund along the settlement frontage could be totally lost to erosion within 10-15 years within continued and ongoing intervention.

At Motunau, none of the critical infrastructure (two wet wells and a wastewater treatment plant) are projected to be at risk from coastal erosion over the next 100 years. However, it is important to note that the southern wet well on the low river terrace was not assessed for this hazard, and given its close proximity to the shoreline it will likely be affected by coastal erosion in the future.

At Gore Bay, although no critical infrastructure was required to be assessed, around 300 m of Cathedral Rd at the southern entrance to the settlement and around 170 m of the southern section of Gore Bay Rd are mapped as be at risk from coastal erosion by 2050, with almost all the roading network through the settlement affected by 2120.

For the 1.5km of Conway Flat Road along the coastline, around one-third is projected to be affected by coastal erosion by 2050, up to two-thirds by 2070, and all but 200 m by 2120.

At Claverley the assessment showed that the rail bridge over Claverley Rd 500 m north of settlement could be at risk from erosion processes within 30 years, 150 m for the road along the coast could be at risk within 50 years, and up to 250 m at risk with 100 years.

9.2 Coastal Inundation Hazard

9.2.1 Bathtub Model Results

A summary of the percentage of settlement coverage and average water depth for the bathtub modelling of a 1% AEP coastal inundation event is presented below in Table 9.3. The settlements potentially worst affected by this hazard were shown to be Leithfield Beach and Amberley Beach, with both settlements having coastal inlets with barrier beaches below the modelled static water levels, and low-lying topography over which this water could easily spread to inundate. However, it is noted that the bathtub modelling method produces very conservative results as it does not account for temporal variances of the event, or any hydrodynamic factors.

Table 9.3: Summary of the spatial extent and average depths of inundation in a 1% AEP storm static water level with RCP8.5 and RCP8.5+ SLR scenarios.

Settlement	Timeframe	Current day	30-year	50-year (2070)		100-year	
	Scenario		RCP8.5	RCP8.5	RCP8.5+	RCP8.5	RCP8.5+
	Sea Level Rise		**+0.23 m**	**+0.32 m**	**+0.40 m**	**+0.56 m**	**+1.01 m**
Leithfield Beach	Approx % of settlement inundated	99%	99%	99%	99%	99%	99%
	Average Depth	0.5m	0.8m	1.2m	1.4m	1.8m	2m
Amberley Beach	Approx % of settlement inundated	20%	33%	90%	95%	99%	99%
	Average Depth	0.1m	0.25m	0.3m	0.5m	0.8m	1.2m
Motunau	Approx % of settlement inundated	10%	10%	10%	10%	10%	10%
	Average Depth	0.69m	1.3m	1.8m	1.9m	2.2m	2.4m
Gore Bay	Approx % of settlement inundated	10%	10%	15%	15%	20%	20%
	Average Depth	0.3m	0.6m	0.7m	1m	1.2m	1.3m
Conway Flat	No settlement assessed						
Claverley	No inundation hazard at settlement						

At Leithfield Beach, the spatial extent of the coastal inundation hazard was modelled to potentially cover 99% of the settlement under all scenarios, with modelled average inundation depths across the settlement increasing from 0.5 m in current sea levels to 2 m for 100-year SLR under the RCP8.5+ scenario.

At Amberley Beach, with the current inundation bund elevations, the extent of inundation was modelled to be 20 to 30% of the settlement for current and 30-year sea levels respectively. However, by 2070 the extent of potential inundation was modelled to increase to 90% of the settlement, and 99% of the settlement under the 100-year SLR under the RCP8.5+ scenario. Modelled inundation depths for static water levels were less than at Leithfield Beach, being up 0.3 m for 30 years of SLR and up to 0.5 m for 50 years. However, wave run-up overtopping would be greater, and could increase inundation depths by up to 0.5 m for 1% AEP coastal storms with 50 years of SLR.

At Motunau, possible inundation under all scenarios is limited to around 10% of the settlement footprint located on the low river terrace, with depths of inundation under the bathtub modelling approach increasing from an average of 0.7 m for current sea levels to 1.8 m with SLR over the next 50 years.

At Gore Bay the northern part of the settlement footprint is susceptible to coastal inundation under all scenarios include current sea levels from over topping the low ridge in front of the combined mouths of Buxton Creek and the Jed River. The extent of potential inundation mapped for a 1% AEP static water level increases from 10% under current sea level conditions to 15% in 50 years SLR and 20% in 100 years with SLR. Modelled average inundation depths increased from 0.3 m for current sea levels to 1.3 m under the 100-year RCP8.5+ scenario. The northern end of Gore Bay is also susceptible wave run-up overtopping over the low beach barrier along Gore Bay Rd, which could add considerable inundation volume, increasing the inundation extent to cover around 35% of the total settlement under current and 30-year scenarios, and increasing inundation depths.

Along Conway Flat Rd, any potential inundation hazard is limited to the mouths of the numerous small streams and watercourses that discharge to the beach fronting the coastal cliffs. Under current day and 30-year SLR scenarios any inundation would be as a result of wave run-up overtopping the beach barrier fronting these streams and watercourses. Under the 50-year RCP 8.5 scenario the 1% AEP static water level could extend up small coastal inlets, and may affect approximately 20 m Conway Flat Rd at the southern end of the study area with 0.2 m water depth. In the 100-year SLR scenarios, the spatial extent of the inundation footprint continues to increase, and the potential depth across Conway Flat Rd could increase up to 0.8-1.0 m under the RCP8.5+ scenario.

At Claverley, no coastal inundation hazard was detected from the 1% AEP static water level modelling, with wave run-up overtopping only potentially effecting the settlement in the 100-year RCP8.5+ scenario.

9.2.2 Coastal Inundation Risk

Dwellings and Properties

A summary of the number of properties and dwellings within each settlement that intersect with the coastal inundation hazard footprint is presented below in Table 9.4.

The most affected settlements in terms of risk to properties and dwellings are Leithfield Beach and Amberley Beach, where even in the current day scenario 60% (Amberley Beach) to all or nearly all (Leithfield Beach) properties and dwellings intersect with the coastal inundation hazard footprint. For Amberley Beach this percentage increases to 80% under the 30-year SLR scenarios, and near 100% under the 50-year scenarios.

At Motunau, the at-risk properties and dwellings are limited to the lower river mouth terrace under all scenarios, being around 10% of the total settlement in all timeframes.

For Gore Bay, properties and dwellings at risk from static water level inundation are limited to around 3% of the total settlement located at the northern end along Gore Bay Rd to the Buxton Creek under current conditions, and only increases to around 8% under both 100-year SLR scenarios. However, the inclusion of wave run-up overtopping increases this inundation risk to around 10% for both total settlement properties and dwellings under current conditions, and up to 40% under 100-year SLR scenarios.

No properties or dwellings were assessed along Conway Flat Rd, and none were assessed as being at risk from 1% static water inundation at Claverley.

Table 9.4: Summary of dwellings and properties within assessed settlements at potential risk from coastal inundation in a 1% AEP storm static water level with RCP8.5 and RCP8.5+ SLR scenarios.

Settlement	Timeframe	Total	Current day	30-year	50-year		100-year	
	Scenario			RCP 8.5	RCP 8.5	RCP 8.5+	RCP 8.5	RCP 8.5+
Leithfield Beach	Dwellings	265	265	265	265	265	265	265
	Properties	197	191	191	192	193	195	196
Amberley Beach	Dwellings	108	65	88	106	108	108	108
	Properties	138	85	110	136	138	138	138
Motunau	Dwellings	131	12	12	12	12	12	13
	Properties	132	11	12	12	13	14	14
Gore Bay	Dwellings	92	2	3	7	8	8	8
	Properties	106	4	5	5	8	8	8
Conway Flat	Properties and dwellings not assessed							
Claverley	Dwellings	13	0	0	0	0	0	0
	Properties	13	0	0	0	0	0	0

Critical Infrastructure

The critical infrastructure identified by Hurunui District Council included wet wells, community water supply bores, wastewater treatment plants and ponds, and roads. A summary of the risk to identified critical infrastructure is presented below in Table 9.5.

The wastewater treatment ponds at Amberley Beach and plant at Motunau are not expected to be subjected to any coastal inundation over the next 100 years. All other critical infrastructure is assessed as being at some potential risk of coastal inundation within the next 100 years. The inundation depths at the wet wells and water supply bore are given from ground level at the structure, with the assumption that the head of the structures is at ground level. It is our understanding that any inundation of these structures will have an effect of the function and efficiency of the infrastructure. Inundation at roads will affect the evacuation route of residents to leave the settlement in the event of an emergency.

The community water supply bore and wet wells at Leithfield, and wet well on the lower river terrace at Motunau (south well) are modelled to be at potential risk of inundated during a 1% AEP storm event with current day sea levels, while the wet well at Amberley Beach and the north well at Motunau are modelled to be at risk with SLR within 30 years.

Of the critical roads assessed, Golf Links Rd at Amberley Beach is at risk of inundation during 1% AEP storm events under all scenarios including current day levels. Although depths are shown to be only in the order of 0.2 m with 30 years of SLR, the addition of run-up overtopping water and velocities is likely to create issues for vehicle access in storm events well before this time. For the roads assessed at Conway Flat and Claverley, only segments are mapped as being at risk of coastal inundation from 50 years on, with inundation depths not likely to be at issue to closer to 100 years.

Although not included as critical infrastructure, the northern entrance to Gore Bay via Gore Bay Rd is potentially at risk from inundation in under current day 1% AEP storm conditions with inundation depths up to 0.2 m and increasing to 1 m with 100 years of SLR. At the southern entrance to the settlement, parts of Cathedral Rd are also at risk from inundation by 1% AEP storm wave run-up overtopping under the 50 year RCP8.5+ scenario, and under both 100 year SLR scenarios.

Table 9.5: Summary of critical infrastructure in each coastal settlement at potential risk of coastal inundation in a 1% AEP static water level event with RCP8.5 and RCP8.5+ SLR scenarios.

Settlement	Infrastructure	Current day	30-year (2050)	50-year (2070)		100-year (2120)	
Scenario			RCP8.5	RCP8.5	PC8.5+	RCP8.5	PC8.5+
Leithfield Beach	Wet Well North	0.87m	1.2m	1.34m	1.5m	1.95m	2.28m
	Wet Well South	0.93m	1.2m	1.38m	1.59m	2.0m	2.29m
	Water Supply Bore	0.59m	0.87m	1.09m	1.2m	1.8m	2.27m
Amberley Beach	Wastewater Treatment Pond	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated
	Wet Well	Not inundated	0.16m	0.36m	0.49m	0.92m	1.2m
	Road	0.02m	0.21m	0.3m	0.5m	>0.6m	>1.2m
Motunau	Wastewater treatment pond	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated
	Wet Well North	Not inundated	0.3m	0.45m	0.61m	1.06m	1.36m
	Wet Well South	0.2m	0.89m	1.16m	1.31m	1.76m	2.06m
Gore Bay	No critical infrastructure assessed.						
Conway Flat	Road (% of total road affected)	0%	0%	2%	3%	3%	3%
	Road (average inundation depth)	Not inundated	Not inundated	0.2m	0.4m	0.8m	1.1m
Claverley	Road (% of total road affected)	0%	0%	0%	0%	0%	<1%
	Road (average inundation depth)	Not inundated	Not inundated	Not inundated	Not inundated	Not inundated	0.15m

9.2.3 Change in Annual Recurrence Interval

As well as water levels, future SLR will also increase the annual probability that the present day 1% AEP event will occur. The resulting change in Annual Recurrence Interval (ARI) for the present day 1% AEP event magnitude at each settlement with SLR is shown in Table 9.6. Within 30 years this magnitude water level is two to five as likely to occur in any one year, and within 50 years five to 15 times as likely to occur in any year. Within 100 years SLR under the RCP8.5+ scenario, this magnitude event would become an annual occurrence.

Table 9.6: Annual Recurrence Interval (ARI) of the current 1% AEP static water level event under RCP8.5 and RCP8.5+ SLR scenarios.

Settlement	30-year (2050)		50-year (2070)		100-year (2120)	
Scenario	RCP8.5	RCP8.5+	RCP8.5	RCP8.5+	RCP8.5	RCP8.5+
Leithfield Beach	50 yrs	40 yrs	27 yrs	15 yrs	5 yrs	1 yr
Amberley Beach	27 yrs	16 yrs	12 yrs	6 yrs	1 yr	1 yr
Motunau	57 yrs	50 yrs	32 yrs	20 yrs	4 yrs	2 yrs
Gore Bay	47-43 yrs	27-28 yrs	20-21 yrs	10-13 yrs	2.5 yrs	1 yr
Conway Flat and Claverley	43 yrs	27 yrs	20 yrs	10 years	2 yrs	1 yr

9.3 Rising Coastal Groundwater Hazard

The settlements most susceptible to groundwater level rise in future SLR scenarios are Leithfield Beach and Amberley Beach, due to the low-lying nature of the settlements and the shallow water tables.

In Leithfield Beach, at present significant areas of existing development and infrastructure are located in areas of indictive average shallow groundwater (<1m BGL). Under the RCP 8.5+ 50yr SLR scenario the majority of the settlement is predicted to have average groundwater levels shallower than 1m BGL, with areas shallower than 0.5m BGL encroaching on the settlement for the RCP 8.5+ 100yr SLR scenario.

In Amberley Beach, at present there very limited areas in the north west corner of the settlement with average shallow groundwater less than 1 m BLG, which changes little with projected SLR over the next 50 years. However, under the 1.3m SLR scenario over 100 years, the whole of the western margin of the settlement (e.g. west of Grierson Ave) is predicted to have average groundwater levels shallower than 1m BGL, with some areas shallower than 0.5m BGL in the northwest corner.

It is predicted that both Leithfield Beach and Amberley Beach settlements are susceptible to saline incursion in the unconfined aquifer.

At Motunau and Gore Bay, the majority of the settlements are elevated and not considered to be at any risk from future ground level rise scenarios. The main areas of risks at both settlements are near the river mouths where average ground water has the potential to rise to within 1 to 0.5 m of the ground surface under the 100-year RCP8.5+ SLR scenario.

Due to the high elevations of both Claverley and Conway Flat, it was determined that even under the 100-year RCP 8.5+ SLR scenarios the settlements were no hazards from a rise in shallow groundwater levels.

9.3.1 Coastal Groundwater Risk

Dwellings

Table 9.7 is a summary of the number of dwellings with depth to shallow groundwater intervals.

Leithfield Beach is predicted to be the most at-risk settlement in terms of dwellings impacts by groundwater rise, with an increase from 5 dwellings currently exposed to average groundwater less than 0.5m BGL to 112 dwellings exposed with a 1.3 m SLR within 100 years.

At Amberley Beach, the number of dwellings predicted to be impacted by groundwater shallower than 0.5m BGL increases from zero to 15 houses by 100 years with over 60% of dwellings in areas of groundwater shallower than 1m BGL, compared to 8% under the current scenario.

At the rest of the settlements, no dwellings are expected to have groundwater within 1 m of ground level even under the 100-year RCP8.5+ SLR scenario.

Table 9.7: Summary table of number of dwellings with brackets of groundwater depth from the ground in RCP8.5+ SLR scenarios.

		Depth to Groundwater (m BGL)			
Settlement	**SLR Scenario**	**≤ 0.5**	**0.5-1**	**1-2**	**> 2**
Leithfield Beach	**Current day**	5	132	128	0
	50 year/0.6m	16	193	56	0
	100 year/1.3m	112	130	23	0
Amberley Beach	**Current day**	0	9	83	16
	50 year/0.6m	1	30	77	0
	100 year/1.3m	15	51	42	0
Motunau	**Current day**	0	0	0	131
	50 year/0.6m	0	0	7	124
	100 year/1.3m	0	0	12	119
Gore Bay	**Current day**	0	0	0	92
	50 year/0.6m	0	0	1	91
	100 year/1.3m	0	0	7	85
Conway Flat	**No Dwellings Assessed**				
Claverley	**Current day**	0	0	0	13
	50 year/0.6m	0	0	0	13
	100 year/1.3m	0	0	0	13

Critical Infrastructure

The only settlement where critical infrastructure is predicted to be potentially at risk by rising groundwater levels is at Leithfield Beach, where levels are likely to increase from a depth of 1-2m BGL (present day) to 0.5 to 1m BGL at the northern wet well, and shallower than 0.5m BGL at the southern wet well. The remaining infrastructure identified at other settlements is not likely to be affected by groundwater rise.

9.4 Recommendations

9.4.1 Coastal Erosion

Continued on-going monitoring of shoreline changes in both position and profile is required to verify and validate the extrapolation of past long-term rates and the role of accelerated SLR in future rates of shoreline retreat. Hence it is recommended that Hurunui District continue to support Environment Canterbury in maintaining and enhancing their long-term coastal profile network. Required enhancements include surveying of nearshore profiles at composite and MSG beaches (e.g. Leithfield Beach, Amberley Beach, Gore Bay and Claverley) for input in geometric models of SLR effects.

There is currently very limited research surrounding what will happen in river mouth environments with SLR, as open coast models tend to over predict what may occur there due to the morphological differences in the foreshore and the influence of river borne sediment supplies. So currently there is no accepted robust methodology for determining change with SLR in these environments. Therefore, it is recommended that further research is required for the effects of SLR on coastal river mouth environments in order to define the erosion hazard on the lower river terrace at Motunau.

It is also recommended that more detailed three-dimensional numerical modelling of geomorphic shoreline response to SLR be considered at some stage over the next 10 years for Amberley Beach, Motunau and Gore Bay, which include inputs of wave and water level drivers, sediment transport, and coastline plan-shape.

9.4.2 Coastal Inundation

From the results of the bathtub modelling, it is recommended that further hydrodynamic modelling of the inundation hazards is warranted at Leithfield Beach, Amberley Beach and Gore Bay to better quantify the threshold for overtopping and inundation, the spatial extent and magnitude (e.g. inundation depths) of the hazard, and risks posed to the dwellings and critical infrastructure that can be inputted into decision making toward adaptive planning pathways.

Any future modelling should also incorporate the effect of future erosion and changes to beach topography on future inundation hazards. For example, the loss of the current inundation protection bund at Amberley Beach, or lowering of the storm ridge at Leithfield Beach, Gore Bay or Claverley would have a large impact on inundation volumes, extents and depths.

The risk assessment undertaken in this study was simplistic is that assets were only assessed based on their intersection with the hazard, and not the magnitude of the hazard which occurred at that asset. To further define the risk at dwellings and properties to coastal inundation, floor level data would be required in order to determine the effect of the hazard on the dwelling (e.g. whether the inundation reached the floor level of the house). Therefore, we recommend that to improve the risk assessment for community engagement, floor level data is included at Leithfield Beach, Amberley Beach and Gore Bay.

9.4.3 Rising Groundwater

The groundwater assessment shows that Leithfield Beach and Amberley Beach settlements are the most at risk from rising groundwater with SLR. While it is noted that there was minimal existing data on which to develop the groundwater model, the model outputs are considered a reasonable assessment of potential risk and correlate well with the conceptual hydrogeological understanding. Should further refinement of the assessment be required to increase confidence in the outcomes, then additional data will be required to be collected. This would include accurate survey and levelling of groundwater monitoring locations, collection of contemporary, high frequency water level data at Leithfield Beach, Amberley Beach and inland areas so that data can be used to validate or refine current modelling.

In order to further refine potential risk to the Leithfield Beach community water supply bore, we recommend a review of the test pumping data to assess if the data can be used to estimate distance to an offshore discharge point (potentially indicated as a hydraulic boundary) or through analysis of observed tidal responses (as are noted on the Environment Canterbury Well Search Database).

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