Studholme outfall: Coastal processes and hazards assessment

Prepared for Fonterra Limited

April 2015

Prepared by:
Murray Hicks
Jo Hoyle
Jo Bind

For any information regarding this report please contact:

Murray Hicks
River & Coastal Geomorphologist
Sediment Processes Group
+64 3 343 7872
m.hicks@niwa.co.nz

National Institute of Water & Atmospheric Research Ltd
10 Kyle Street
Riccarton
Christchurch 8011

Phone +64 3 348 8987

NIWA CLIENT REPORT No:	CHC2015-019
Report date:	April 2015
NIWA Project:	FCR15501

Contents

Tables

Figures

Reviewed by	Approved for release by
Richard Measures	Helen Rouse

Executive summary

This report presents results from a study of coastal processes and hazards on a section of the South Canterbury coastline, at Waimate Creek just south of Wainono Lagoon, where Fonterra Limited have proposed an ocean outfall for wastewater disposal related to planned expansion of their milk processing plant at Studholme. The overall aim of the study was to assess the effects of the outfall installation and operation on the local shore morphology and position, in the context of existing shoreline movement trends and as complicated by future sea-level rise (SLR) and climate change effects.

The scope of work covered an appreciation of the existing environment and an assessment of the environmental effects of the proposed project. The analysis of the existing environment included:

- a literature review;
- assessment of existing shoreline management along the project shore;
- an updated analysis of shoreline stability;
- field inspections of the proposed outfall site at the end of Meyer Road and of the Clandeboye outfall that was installed north of Timaru in 2005;
- assessment of sea flooding by wave overtopping and storm surge and assessment of tsunami hazard on the project; and
- assessment of effects on the shoreline, in the vicinity of the project, of predicted rises in sea level and changes in wave climate.

The assessment of project effects included:

- assessment of project effects on shore processes, stability, and flooding hazard during and post construction;
- advice/recommendations on outfall design and construction issues relating to coastal process; and
- mitigation and monitoring recommendations during and post construction.

The main findings are as follows:

- Historical surveys of beach profiles and the position of shoreline features at the pipeline crossing show that the barrier is currently approximately 5.6 m high, has a trend for long-term retreat ranging from 0.19 m/yr to 0.73 m/yr, and has a volume of around 220 m^3 per m shore length that varies largely due to losses and gains in foreshore volume associated with longshore transport. While the barrier height has decreased historically, this has occurred in approximately decadal jumps with long stable periods between.
- Environment Canterbury (ECan) coastal hazard mapping defines two Coastal Hazard Zones extending some 30 and 60 m landward of the barrier backshore toe position. These largely reflect the threat of ongoing barrier retreat at historically-observed rates

50 and 100 years into the future from 2005. Since the pipeline crosses these zones, the installation will be a discretionary activity requiring consent.

- Analysis of historical sea-flooding events in the Waihao-Wainono Lowlands area shows that barrier overwash or breaching has occurred about every 10 years on average but is usually confined to limited spans of barrier during any given event. Factors rendering the barrier more vulnerable to overwashing and breaching are a relatively low barrier height, low barrier gravel volume, more deeply buried substrate underneath the barrier, and a waterway adjacent to the backshore toe. The last substantial breaches of the barrier occurred in 2001-2002, when the backshore toe at Waimate Creek moved approximately 12 m landward.

- An approximately 10 year return period for barrier overtopping/overwashing was confirmed by analysis of sea level and wave hindcast records. The likelihood of an overtopping event occurring during a 6 month construction period at the shore crossing would be approximately 5%.

- Sediment budget estimates combined with calculations of longshore transport rates based on the wave hindcast record indicate a dominantly northward littoral drift averaging some 120,000 m^3/yr. Longshore transport rates show a seasonal pattern, with 5% exceedance monthly transport rates averaging 12,000 m^3/month over November-April and 18,000 m^3/month over May-October.

- The estimated return period of barrier overtopping by tsunami run-up at the outfall site ranges between 60 and 270 years. The shorter return period is a conservative, “maximum potential” estimate that has been adopted for evacuation planning, while the longer return period provides a more likely expectation. Using the latter, the likelihood of an overtopping tsunami during the 50 year project life is 17% and 0.2% over the construction phase.

- Future climate change could affect the project shoreline through several drivers, including SLR, altered wave and storm surge climate, and altered sediment budget. All of these can interact to influence barrier processes. How wave and storm surge climate might change, and associated net effects on the local sediment budget, remains uncertain and available guidance is vague, but future SLR scenarios recommended by Ministry for the Environment (MfE 2008) are more definitive and were used (after adjustment to the present) to assess potential SLR on barrier stability and retreat at the project site through its 50-year design life. The SLR scenarios assessed included a base rise of 0.26 m and a higher rise of 0.40 m between 2015 and 2065, at average rates of 5.2 and 8 mm/yr.

- Assuming the barrier at the project site to be adequately nourished with beach sediment, the expected response to SLR is an overtopping/overwash process that would both build the height of the barrier and roll it landward as sediment is transferred from the foreshore to the backshore. This produces only a few metres of retreat, which, when combined with the largest estimate of historical retreat rate, provides a total barrier retreat to 2065 of 41 m and 43 m for the two SLR scenarios, respectively. Alternatively, assuming that the barrier is in a state of sediment deficit, the calculated retreat by 2065 is 61 m for the lower scenario and 94 m for the higher scenario. In this case, since the observed historical retreat rates are of the order of those calculated from the observed historical SLR, it is assumed that these retreat figures capture the factors driving the historical retreat. This accelerated retreat would also be accompanied by more frequent overwashing events if the barrier height does not increase to match the rise in sea level.

- It is recommended that the project plans for the larger estimate of retreat associated with the lower SLR scenario (i.e., 61 m by 2065), but also has a contingency for a greater retreat (94 m by 2065) to cover the case of the higher MfE SLR scenario or an unfavourable change in wave climate. Specifically, this would mean placing the surge chamber at least 94 m back from the current position of the backshore toe of the barrier.
- Inspection of the shore at the site of the existing Clandeboye outfall, and of ECan's beach profile surveys there, showed no morphological signature of the pipeline. This provides confidence to the expectation that the proposed Studholme outfall will also have no long term morphological signature.
- The main issue relating to the installation phase of the outfall pipe at Waimate Creek is that the proposed coffer dam has the potential to intercept significant quantities of littoral drift on its southern side, which will cause near-field foreshore erosion on its northern side. The likely interception would be enough to render the downdrift barrier vulnerable to overtopping or breaching by common storm wave events, so it will be necessary to mitigate this by artificial bypassing of foreshore sediment. Shortening the coffer dam would likely reduce the amount of artificial bypassing required, but may increase wear and fatigue on the pipeline if it is left exposed on the low-tide step or lower foreshore.
- During the construction phase, the foreshore topography 200 m either side of the pipeline and coffer dam should be surveyed weekly to ensure that the artificial littoral drift bypassing limits any downdrift foreshore erosion. Any trend of downdrift erosion should be countered with accelerated bypassing. In addition, the beach levels immediately against the coffer dam should be checked after storm events, and any low areas near the ridge crest should be filled with beach gravel.
- Once installed and buried under the beach ridge, the pipeline will have no significant effects on shore processes. Even with a more extreme SLR scenario, the pipeline would only become exposed on the lower foreshore towards the end of its 50-year design life, and then would simply flex over the seabed.
- The potential for the refilled trench through the barrier becoming a weak point, vulnerable to breaching, will be mitigated by refilling the trench with beach-grade sediment stockpiled during the trench excavation stage. It is recommended that this fill is laid and compacted in layers that parallel the foreshore to reproduce the natural stratification and permeability characteristics of the barrier, and that the barrier is monitored for signs of seepage for some 5 years post-construction.
- A potential long-term issue is that upper bound estimates of barrier retreat associated with SLR would compel ECan to shift the Waihao Arm channel landward within a 50-year time frame. If this situation arose, it would be necessary to re-install a segment of

pipeline deep enough in the backshore that the re-located Waihao Arm channel could cross over it, and it would be necessary to have the surge chamber located at least 115 m back from the present barrier backshore-toe position to provide adequate space for a relocated channel. It is recommended that the pipeline design allow for this contingency.

- Long-term monitoring should include the height and position of the barrier at the pipeline and along the adjacent barrier (within ~ 500 m either side). This will confirm expectations of barrier retreat rates and warn of impending infilling of the Waihao Arm channel. It would be achieved by continuing to monitor ECan's Waimate Creek profile annually, surveying longshore variability in barrier crest height and backshore toe position at the same time, and maintaining a visual watch after coastal storms.

1 Introduction

1.1 Background

Fonterra Limited commissioned NIWA to undertake a study of coastal processes and hazards on a section of Waimate coastline just south of Wainono Lagoon. The study is required to inform the design and resource consent applications for a proposed ocean outfall for wastewater disposal related to planned expansion of Fonterra's milk processing plant at Studholme, South Canterbury. In regard to resource consenting, the overall aim is to assess the effects of the outfall installation and operation on the local shore morphology and position, in the context of existing shoreline movement trends and as complicated by future sea-level rise and climate change effects. This report presents the findings of this investigation.

1.2 Scope of work

The work scope provides an appreciation of the existing environment and an assessment of the environmental effects of the proposed project. The analysis of the existing environment includes:

- A literature review covering past investigations of the project span of coast;
- Assessment of existing shoreline management along the project shore, covering the planning context, Environment Canterbury's (ECan's) shore management and backshore drainage activities and plans, existing hazard zones and their basis, and the Regional Coastal Environment Plan;
- Updated analysis of shoreline stability, covering historical shoreline positions analysed from existing air photograph and LiDAR datasets, analysis of 3d change from repeat LiDAR surveys, and analysis of ECan coastal profile data for trends in position and beach height;
- A field visit to the proposed outfall site and also the site of an existing outfall at Clandeboye (covering site inspection, updated RTK-GPS surveys of barrier profile, crest-line, and backshore-line at the proposed site, and assessment of any post-construction effects at Clandeboye);
- Assessment of sea flooding by wave overtopping (covering a summary of historical sea-flooding events and an analysis of risk using LiDAR topography, wave data, and storm surge/sea-level data – the wave analysis will use refracted wave data from the ECan wave buoy; the sea-level analysis will use local port records);
- Assessment of tsunami hazard on project shore (based on recent regional NIWA modelling for ECan); and
- Assessment of effects on project shore of predicted rises in sea level and changes in wave-climate (covering shoreline retreat and flooding by over-topping).

The assessment of project effects covers:

- Assessment of project effects on shore processes, stability, and flooding hazard during and post construction;

- Advice/recommendations on outfall design and construction issues relating to coastal process; and
- Mitigation and monitoring recommendations during and post construction.

2 Study area

2.1 Location

The area of interest lies within the 'Wainono Lowland Coast' (after Kirk 1987), a 28 km stretch of coast between the Waihao River Outlet and the Otaio River (Figure 2-1). Todd (1988) defines this area as a distinct morphological unit comprising a sand and gravel beach ridge (or barrier) fronting low-lying hinterlands. This study is focused on the section of shore that lies between the Wainono Lagoon and the Waihao River 'boxed' outlet. This is referred to as the Waihao-Wainono coast. The proposed outfall location is sited opposite Waimate Creek, aligned with Meyers Road (Figure 2-2).

Figure 2-1: Map of the Wainono Lowland Coast showing the location of the proposed outfall.

Figure 2-2: Map of the Waihao-Wainono coast showing the location of the proposed outfall.

The Wainono Lagoon extends 2.5 km along the coast and has an area of 4.3 km^2 (at a water level of 1.2 m above MSL Lyttelton datum). The lagoon acts as a storage area for a catchment of 252 km^2 and does not have a direct opening to the sea; instead it drains via the Waihao Arm (often referred to as the 'Dead Arm'). This Waihao Arm meanders from the south east corner of the lagoon in a south westerly direction for 3.4 km and then runs behind the barrier for a further 4 km until reaching the mouth of the Waihao River. The Waihao River also does not have a natural opening to the sea. To combat the lack of a permanent opening, a box structure (commonly term the 'Waihao Box') was built in 1897 to provide an outlet for the river (Figure 2-3).

Figure 2-3: Photo of the Waihao Box outlet. The sea is to the left and the Waihao River is to the right of the barrier. From Stapleton (2005).

2.2 Geomorphic setting and history

In the late Pleistocene (up to around 12,000 years ago) there were large glaciers in the upper Waitaki catchment delivering abundant sediment to the Waitaki River. Sea level was approximately 120 m lower than its present level, and the river built a large alluvial fan that extended well seaward of the present coastline. Around 15,000 years ago the situation changed as sea level started to rise, flooding the Waitaki fan and the lower-lying area to its north, and initiating wave-driven erosion of the fan margin. At the same time, the supply of sediment delivered by the Waitaki River reduced as the retreating glaciers left moraine-dammed lakes that disconnected the river from many of its alpine sediment sources, and this reduced coastal supply would have exacerbated the erosion trend.

By around 7,000 years ago, when sea level stabilised at its current level, the shoreline at the Waitaki mouth was some 7 km seaward from its present position. At this time also, the Waihao-Wainono lowland, which occupies the area between the Waitaki Fan, the rising ground to the north of the Otaio River, and a terrace fronting the Waimate plains and downland, was initially an embayment of the sea. However, this embayment was soon spanned by a sandy-gravel spit fed by sediment from the south, supplied both by the Waitaki River and coastal erosion of the Waitaki Fan, and driven northwards by the prevailing southerly swell. The spit grew into the present beach ridge, while the isolated embayment has partially filled with sediment supplied from the Waihao River and smaller creeks, leaving Wainono Lagoon as a vestige of a once larger water body.

The present beach ridge now forms part of a continuous span of mixed-sand-and-gravel beach that extends from Oamaru to Pareora – partly as a beach lapping eroding cliffs cut into Pleistocene glacial outwash gravels (Waitaki Fan, both sides of the Waitaki River mouth) or wind-blown loess (notably from the Otaio River north and at Oamaru) and partly as a ridge fronting coastal lowland or lagoon. Over the last 7,000 years, while sea level has been relatively stable, the adjoining cliffs have retreated by coastal erosion and the beach ridge has also rolled landward to maintain alignment with its roots.

3 Literature review of existing environment

The following review summarises what is covered in the literature with regard to coastal processes and hazards at two scales: the broad span of coast encompassing the Waitaki fan and the Waihao-Wainono shore, and the immediate area of the proposed outfall. We start with an overview of the key pieces of work, then focus on topics of particular interest.

3.1 Overview of key previous research

3.1.1 Neale (1987)

Neale (1987) studied changes in the average volume of the barrier north of the Waitaki River and proposed that northward-migrating 'sediment slugs' caused the variability of sediment volume observed along the barrier. He thought the slugs were sourced from events such as Waitaki River floods or severe coastal storms that erode the cliffs and introduce new sediment to the beach. Neale estimated the average rate of slug migration to be 1.15 km/yr, and suggested that breaching or overtopping of the barrier would occur in areas of troughs between slugs.

3.1.2 Single (1992)

Single's PhD thesis study was focused on the stretch of coast from the Waihao Box to the Otaio River mouth. It examined the role of high wave energy events on long term coastal retreat, the causes of beach crest lowering, beach-ridge breaching mechanisms, and what conditions predispose the occurrence of breaching. Key factors determining beach response to high energy events were the foreshore slope, the presence and dimensions of intermediate berms, the pre-storm volume of sediment seaward of the ridge crest, and the foreshore sediment composition and structure.

3.1.3 Todd (2003)

Todd (2003) investigated coastal processes and hazards at Clandeboye, north of the Orari River, to assess the potential effects of a dairy-factory outfall at that location. His key findings and recommendations are summarised in Section 3.10 below.

3.1.4 Stapleton (2005)

Stapleton's (2005) MSc thesis study focused on the north Waitaki fan- Wainono coast, and, like Single's, addressed the relationship between beach ridge form and behaviour to improve understanding of why breaches tend to occur in some areas and not in others. Key new work was on the composition of the beach ridge, which was established by excavating trenches and sampling the surface, subsurface, and substrate sediment (where intact Pleistocene material was exposed). This trenching included two sites near the site of the proposed outfall. Stapleton concluded that past breach sites tended to have had steeper lower-foreshore slopes, but the importance of ridge composition to breaching was less conclusive.

3.1.5 Various work by Hicks

Hicks and co-authors investigated the Oamaru-Timaru coastline in regard to the effects on coastal sediment delivery and erosion of existing and proposed hydroelectric power operations on the Waitaki River (Hicks 2006; Hicks et al. 2002, 2006; Hicks & Todd 2003, 2005). Their work focussed on sediment budget assessment, and included measuring historic rates of coastal retreat using photogrammetry and LiDAR. They found that long-term shoreline shifts since 1864/5 show an overall trend for decreased erosion from south to north along the Waitaki coast, but with substantial variability superimposed and no clear, ongoing signal of the Waitaki River hydro-operations. They suggest that a dam effect is possibly yet to arrive, or, if it has arrived, it is sufficiently damped that it is hidden by the natural variability that occurs along this coast.

3.1.6 Dickson et al. (2009)

Dickson et al. (2009) examined variability in beach volume along the Waitaki coast from Oamaru to just north of Wainono Lagoon using ground penetrating radar to identify the boundary between beach sediments and the underlying Pleistocene alluvial fan sediments. The motivation was to determine the stock of beach sediment available to protect the base of the eroding sea cliffs. This study revealed a trend of northward-increasing beach volume, reflecting the progression from beaches in front of cliffs, to beaches that overtop low cliffs, to beach ridges fronting the Wainono Lowlands.

3.1.7 Gabites (2012)

Gabites (2012) presented results from ECan's coastal profile monitoring programme for the coastline south of Timaru for the period 1977 – 2011. This included 30 sites between the Waitaki River and Tuhawaiki Point (immediately south of Timaru), and examined trends in beach volume and horizontal shifts in the position of the ridge backshore toe, ridge crest, and the 1 m (above mean sea level) contour on the foreshore. The results of this study, updated for profiles close to the proposed outfall location, are presented in more detail in Sections 3.2, 3.5.2 and 4.3.

3.2 Historical shoreline movements

Historical shoreline shifts along the Waihao-Wainono shore have been measured using photogrammetry, LiDAR, and beach profile surveys.

Gibb (1978), using photogrammetric methods and cadastral maps, measured time-averaged retreat rates of 0.68 m/yr (1899-1963) at the Waihao River and 0.38 m/yr (1953-1977) at Wainono Lagoon. Kirk (1987) noted that the beach ridge fronting the 23 km span of shore from south of the Waihao River to north of Makikihi had shifted landward at an average rate of 0.5 m/yr historically.

Hicks and Todd (2005) undertook comprehensive mapping of shoreline shifts between Oamaru and Makikihi. Shoreline positions were extracted off cadastral surveys (1865) and by photogrammetry from aerial photographs (1943 and 1955 for adjoining segments of coast, 1977 and 2000 for whole of coast). On cliffed segments of the coast, the shoreline was defined as the cliff edge. On barrier-fronted lowland coasts, the shoreline was defined as the backshore toe of the beach barrier. Their results are shown on Figure 3-1. From this, the overall average erosion rates between 1865 and 2000 show a northward decline: from 0.6-0.7 m/yr on the south Waitaki fan, the rates reduce to around 0.5 m/yr on the north Waitaki fan then reduce further towards ~ 0.2 m/yr or less along the barrier-ridge shore further north past Wainono Lagoon. Note that the rates vary 'noisily' alongshore and between epochs, partly due to space-time episodicity of erosion events (or "bites") and partly due to errors in the photogrammetry (relating to image quality and interpretation of the shoreline position). These overall longshore patterns in shoreline behaviour are consistent with the geomorphic setting (Section 2.2), with the erosion of the Waitaki fan continuing a long-term response to the last post-glacial rise in sea level and reduction in river sediment load, and with wave energy focussing on the broad headland formed by the trimmed-back, old alluvial fan.

Figure 3-2 details the Hicks and Todd results for the Waihao-Wainono shore segment. This shows epoch-averaged rates of shoreline shift varying alongshore and between epochs generally between around zero m/yr and about 0.7 m/yr erosion. For the outfall location at Waimate Creek, average erosion rates varied between essentially zero (1943-1977) and approximately 0.5 m/yr (1977-2000).

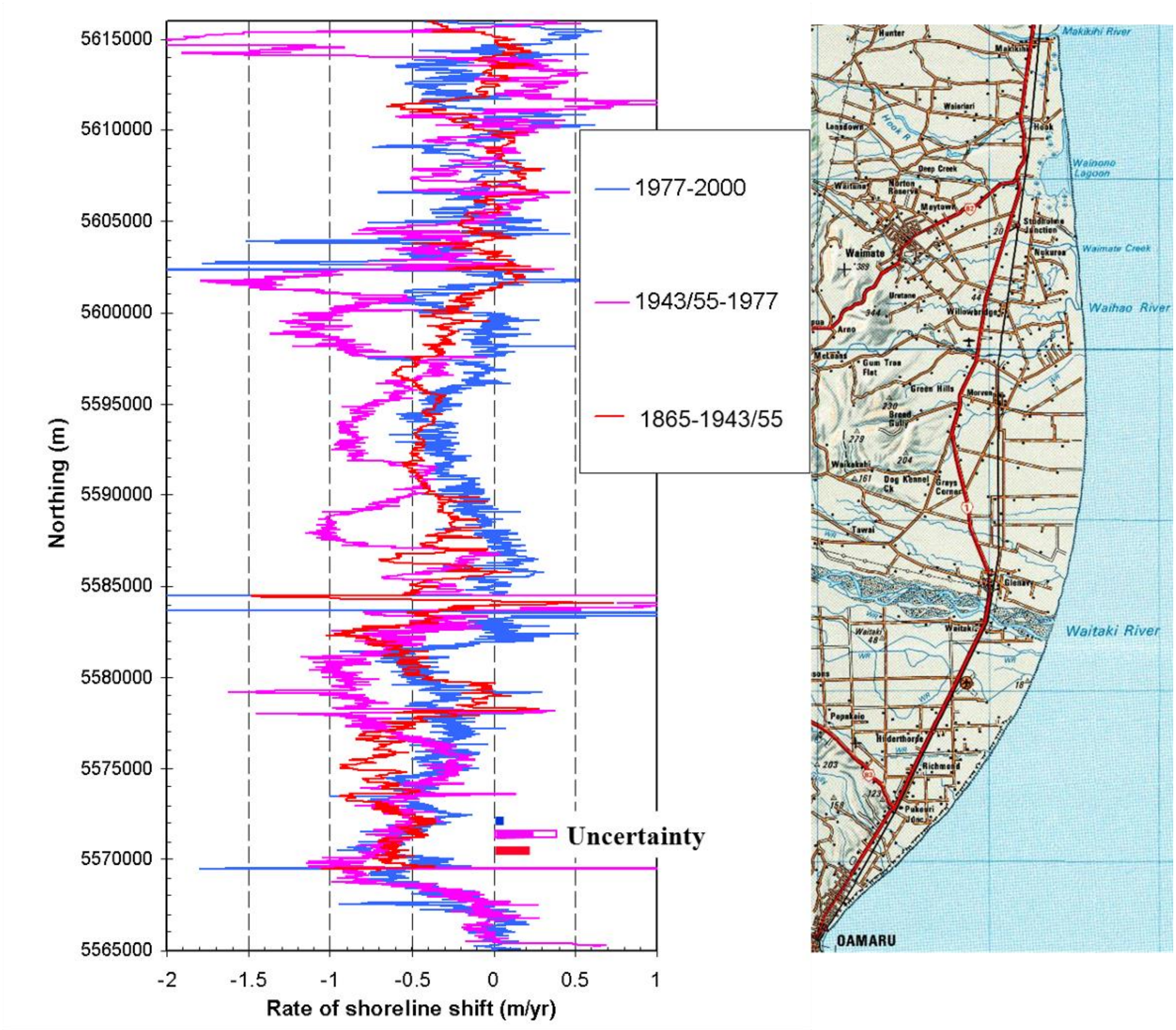


Figure 3-1: Average rates of shoreline shift over several historical epochs as determined by Hicks and Todd (2005). Negative values indicate retreat. Bars indicate the magnitude of uncertainty with each epoch.

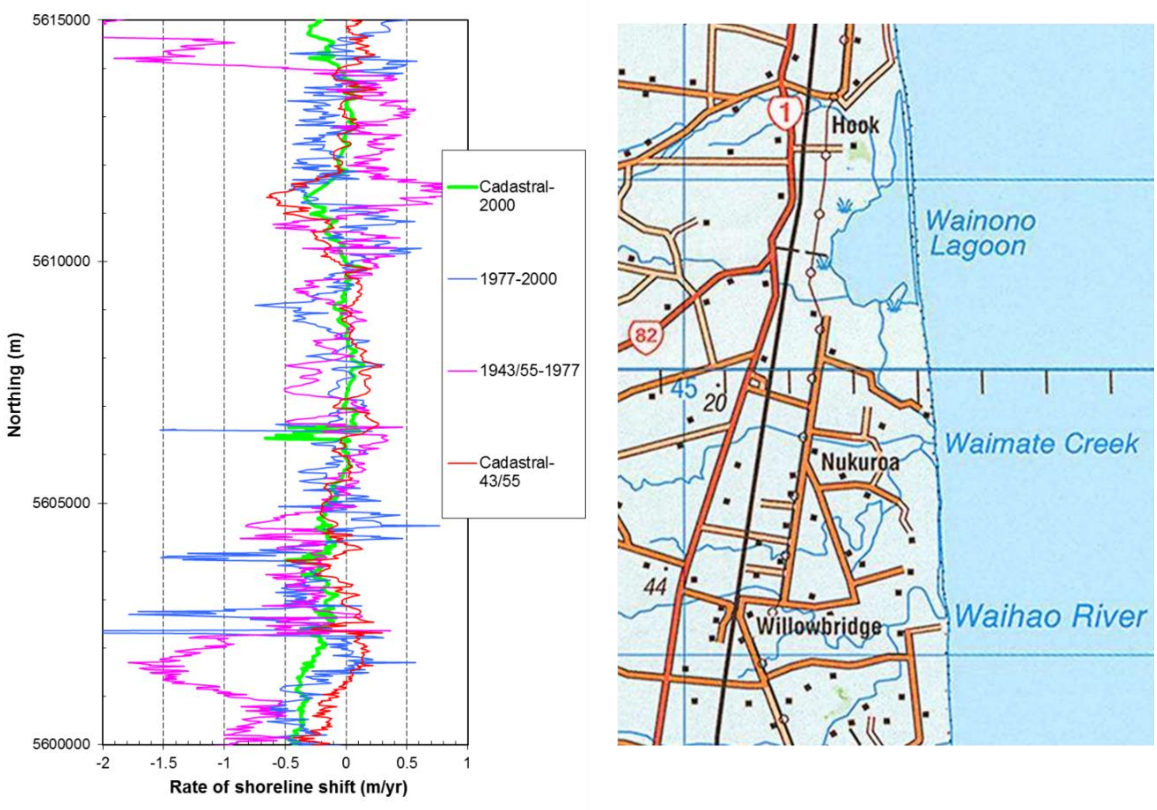


Figure 3-2: Detail of shoreline shift rates along Waihao-Wainono shore, from Hicks and Todd (2005). Proposed outfall location is opposite Waimate Creek.

Hicks and Bind (2013b) used repeat LiDAR surveys to measure shoreline shifts between Oamaru and Makikihi over the period 2004-2013. They defined the shoreline both as the backshore toe of the beach ridge (or the clifftop edge for cliffed segments of coast) and as the Mean Sea Level (MSL) contour along the beach face. They found that MSL definition was more reliable, since the backshore toe was often difficult to place on the LiDAR-derived digital elevation model. Their results (Figure 3-3) indicated MSL retreat rates varying from near zero to 1.4 m/yr along the Waihao-Wainono shore. Although the backshore toe positions suggested actual shore advance, they were considered unreliable. A more detailed examination of these LiDAR datasets within the project area is reported in Section 4.4.

ECan has monitored coastal profiles since 1977 at 30 sites along the South Canterbury coast. Three profiles cross the Waihao Arm and span the proposed outfall site. From south to north they are: S5800 (Lows Road), S5554 (Waimate Creek), and S5513 (Poingdestres Road). The Waimate Creek profile essentially overlays the proposed outfall alignment. Results for the 1985-2011 period at these three profiles are included in Gabites (2012), who derived rates of erosion by fitting linear regression trends to time-plots of excursion distance to the barrier backshore toe, ridge crest, and the 1 m above MSL contour on the foreshore (Table 3-1). This showed retreat rates ranging from 0.24 m/yr to 0.68 m/yr at Waimate Creek, depending on which feature is tracked. Retreat rates ranged from 0.67 to 1.69 m/yr at Lows Road (2.5 km further south) and from 0 to 0.59 m/yr at Poingdestres Road (0.6 km further north), suggesting a local trend for southwards-increasing erosion.

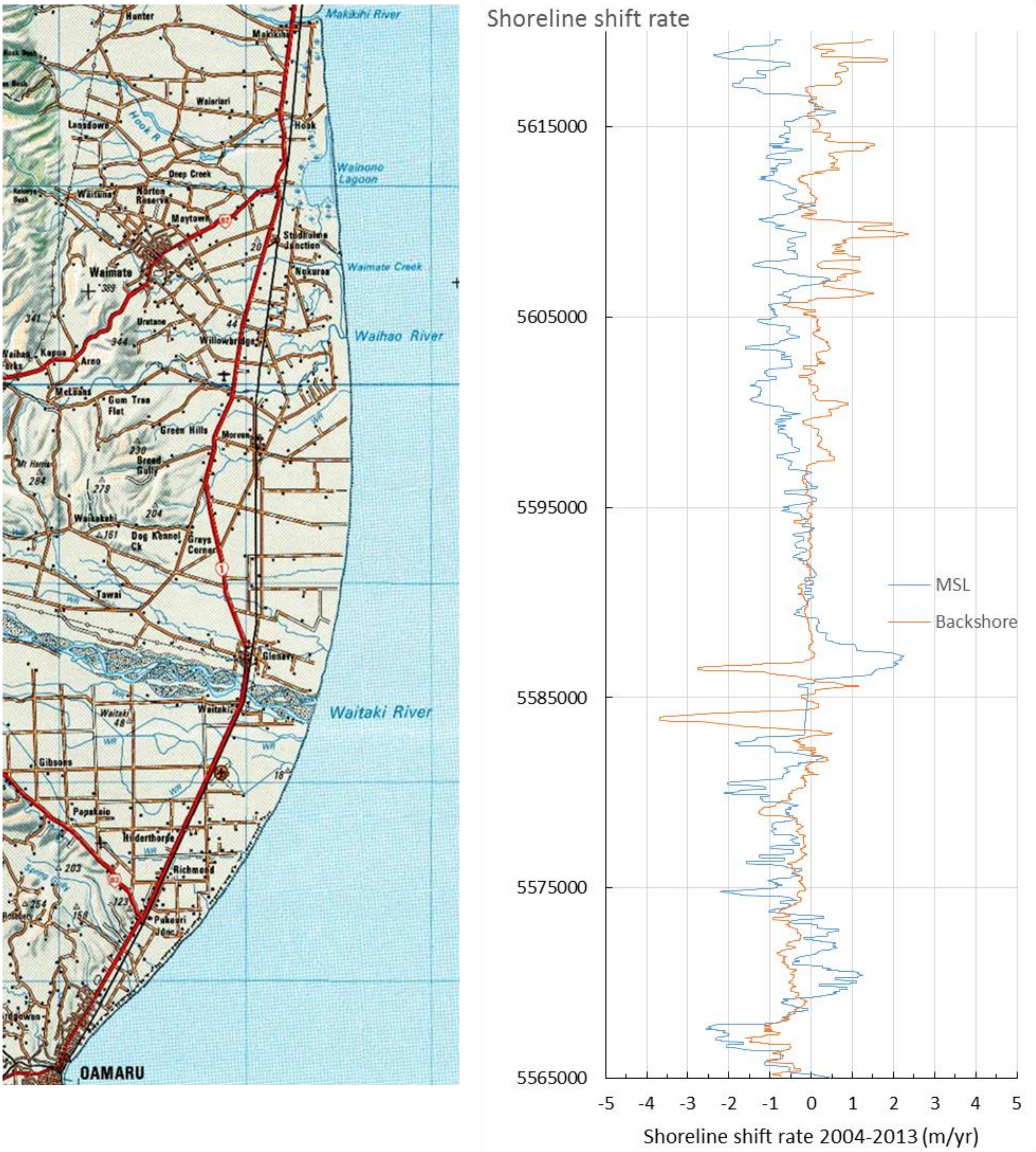


Figure 3-3: Shoreline shift rate determined from LiDAR surveys in 2004 and 2010. Note different patterns for shorelines defined by backshore ridge toe (or cliff edge) and MSL contour on foreshore.

These rates, and their spatial variability, are consistent with the range observed by the other investigations discussed above, and certainly provide the most robust indication of retreat rate at the immediate location of the outfall. Gabites' analysis is updated to 2014 using more recent beach survey data in Section 4.3.

What is clear is that over recent decades, retreat of the beach ridge along the Waihao-Wainono lowlands is not as rapid as the cliff retreat measured along the Waitaki fan, and it occurs in localised events associated with barrier breaching or washover by storm waves.

Orford et al. (1996) developed a conceptual model of the state of barriers similar to the Waihao-Wainono barrier, beginning with a *growth* domain (the initial spit growth), a *consolidation* domain (when the barrier moves into a drift-aligned position and develops an equilibrium height), and a *breakdown* domain, marked by *slow rollover* (with retreat predominantly by washover), *fast rollover* (when the barrier becomes less stable and more prone to breaching), and finally *dissolution* (if/when the barrier "dissolves" by a lack of sediment and frequent breaching) phases. The progression is typically driven by a long-term decline in sediment supply, such as when headland-trimming slowly reduces the breaker angle at the sediment source (in this case, the eroding cliffs of the Waitaki Fan). Stapleton (2005) considered the Waihao-Wainono barrier currently lies between the *consolidation* and *slow-rollover* states, as evidenced by observed rollover and ordered cross-shore sorting of sediment.

Table 3-1: Time trends of excursion distance to various barrier features at ECan profiles in the vicinity of the proposed outfall. From Gabites (2012).

Feature	Lows Road	Waimate Creek	Poingdestres Road
Beach (backshore) toe	-1.69 m/yr	-0.68 m/yr	0.00 m/yr
Beach crest	-0.67 m/yr	-0.34 m/yr	-0.59 m/yr
1 m amsl contour	-0.72 m/yr	-0.24 m/yr	0.00 m/yr

3.3 Sediment budget

An historical sediment budget for the Waitaki littoral cell, which extends from Cape Wainbrow at Oamaru to the port of Timaru, was developed by Hicks and Todd (2003) and updated by Hicks (2006). It accounts for beach sediment sources from eroding sea cliffs and the adjacent nearshore seabed[1] and from the Waitaki River and other smaller rivers further north, wave-driven longshore transport, and sediment losses by abrasion. Wave energy incident on this coast is predominantly from the southeast, driving a net northerly sediment transport.

These budget components are shown by coastal segment (or sub-cell) in Table 3-2 and are accumulated northward from Oamaru in Figure 3-4 (which also compares the net volumes that must be passing north as littoral drift against the computed longshore transport potential). It represents the pre-dam era for the Waitaki River's bedload supply. Hicks (2006) considered that this almost certainly has decreased, due mainly to the damped river flood regime under HEP operations.

[1] Cliff erosion volumes were computed using a shore height extending from cliff top to foreshore toe. This recognises that the shore retreat also consumes Pleistocene substrate below sea level. If this were not the case, then there would be a several km wide wave-cut terrace fronting the Waitaki fan cliffs.

Table 3-2: Beach material budget for sub-cells of the Waitaki littoral cell. The Waitaki River supply is the estimated pre-dam supply (from Hicks 2006).

Cell	Distance N of Waitaki (km)	Cliff supply m^3/yr	River supply m^3/yr	Abrasion losses in each cell m^3/yr	Passing north m^3/yr
South fan	-21.4 to -2.6	250151	0	-77948	172203
Waitaki mouth area	-2.6 to 3.2	41820	153000	-53530	313493
North fan	3.2 to 13.3	79656	0	-108623	284526
Wainono lowland	13.3 to 36.4	0	0	-160907	123620
Makikihi-Pareora	36.4 to 51.4	5600	17200	-52906	93514
Pareora-Timaru	51.4 to 63.9	0	0	-33887	59627
Sum/net		**371626**	**170200**	**-487800**	**59627**
% of total supply		69%	31%	89%	11%

Abrasion rates were calibrated so that the beach-material delivery at Timaru matched the average historical rate of accumulation on South Beach at Timaru (51,000-60,000 m^3/yr, Tierney 1977; Kirk 1987; Neale 1987).

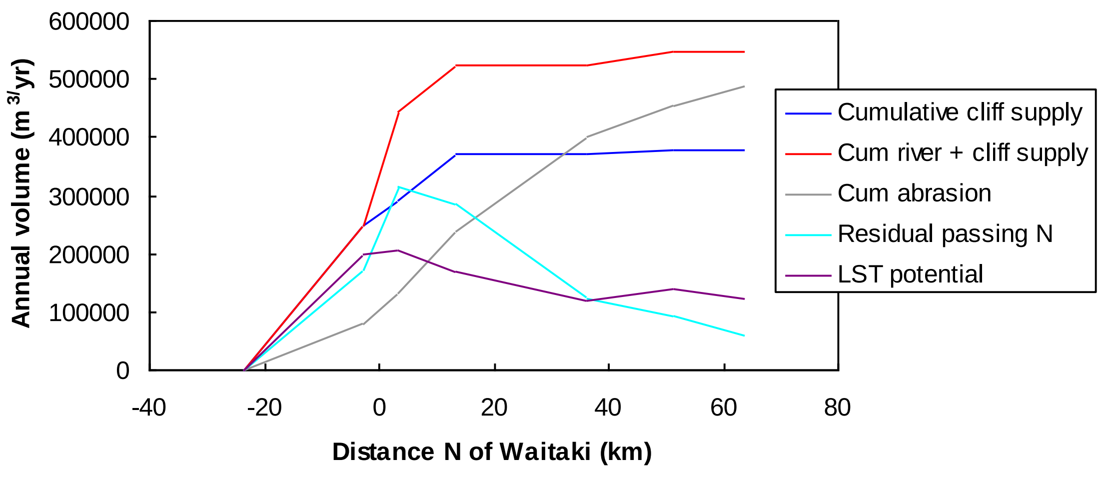


Figure 3-4: Northward-cumulative beach-material supplies from cliff and river sources and losses to abrasion, calculated longshore transport potential (LST), and actual longshore transport inferred from sediment budgeting (residual passing north) for Waitaki littoral cell. From Hicks (2006).

The longshore transport potential of the wave climate was based on the wave hindcast and refraction study of Gorman et al. (2002), which generated 20 years of wave record every km along the Canterbury coast. The CERC[2] (1985) longshore transport function was tuned to match the potential transport at the north end of the Wainono barrier with the inferred actual amount of beach material drifting north past there (approximately 120,000 m^3/yr). In the natural (i.e., no Waitaki hydro-electric power scheme effects) situation, rivers (mostly the Waitaki) would have supplied

[2] Coastal Engineering Research Center

approximately 31% of the total beach sediment supply, 66% of the cliff-erosion supply was from the South Waitaki fan, and 89% of the combined supply from rivers and cliff erosion would have been lost to abrasion by the time it arrived at Timaru. The pattern of potential longshore transport largely reflects the shore exposure to southerly wave energy. It peaks (as does the inferred actual transport) around about the Waitaki mouth where the shore faces SE, decays north along to Makikihi where the shore faces more to the ENE, then picks up again towards Timaru as the shoreline re-orientates to face SE again.

In regard to the Waihao-Wainono lowland sub-cell, sediment is supplied by littoral drift driven northward from the north Waitaki fan shore, while sediment is lost northwards past Makikihi and everywhere alongshore by abrasion. These losses mean that for the beach volume to remain stable, sediment must be accumulating due to the progressive northward reduction in wave-driven longshore transport potential. Indeed, Hicks and Todd (2003) concluded that the barrier volume fronting the Waihao-Wainono lowlands, despite landward translation, must have remained more-or-less constant over the past several thousand years, reasoning that if there was a sediment deficit the barrier would have ceased to exist (or conversely, if there was a sediment surplus, a series of accretionary ridges would be evident). Thus, it appears that this shore has developed an alignment whereby deposition associated with the northward-declining transport potential just offsets beach sediment losses due to abrasion.

3.4 Beach ridge (barrier) characteristics & processes

3.4.1 General morphological features

The Waihao-Wainono shore comprises a mixed sand and gravel (MSG) beach backed by a high MSG ridge or 'barrier'[3] that has its backshore slopes formed by wave washover processes.

McLean (1970) noted that in New Zealand MSG beaches all:

- contain a wide range of sediment sizes (sand to boulders);
- are derived from the same dominant rock type (greywacke);
- are backed by Pleistocene and Holocene alluvial plains and fans often covered by major rivers; and
- are exposed to the high energy waves of an East coast swell environment.

Kirk (1980) identified four major morphological zones of MSG beaches/barriers (Figure 3-5):

- a very steep nearshore face – from below waterline to the inner continental shelf, usually comprised of coarse gravels and cobbles standing at near their angle of repose;
- a distinct break point step (or low tide terrace) at the top of the nearshore face upon which waves break at all phases of the tide resulting in a very turbulent narrow surf zone comprised of gravel and cobble sediments;
- a steep foreshore - from the breakpoint step to the beach crest ridge - which is dominated by swash and backwash processes, is where most of the morphological

[3] Beach ridges are also referred to in the literature as barriers (Hesp and Short 1999; Forbes et al. 1995).

changes on mixed sand gravel beaches occur, and includes berms formed by distinct high tides or storms; and.

- the backshore – landward of the crest ridge, which (away from cliffed backshore) is formed from broad washover surfaces dominated by coarse gravels and cobbles.

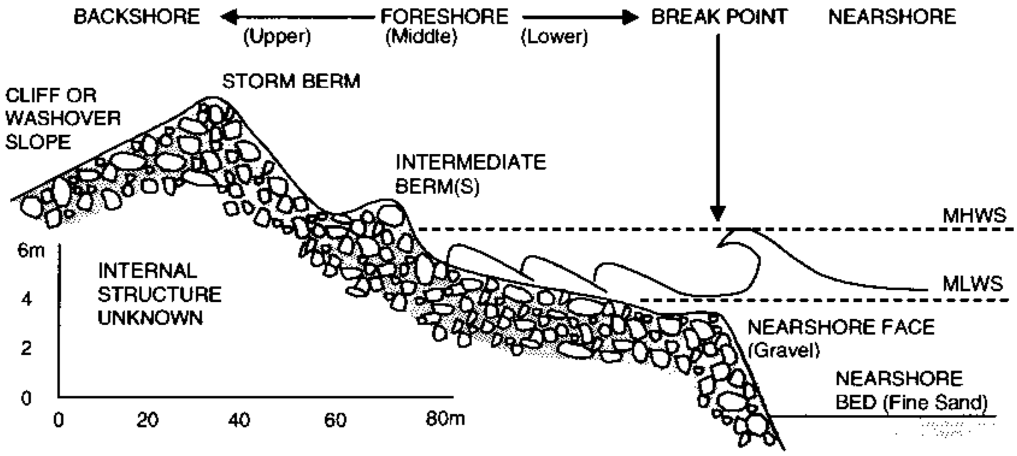


Figure 3-5: Typical morphology of a mixed sand and gravel beach profile. Updated from Kirk (1980) by Stapleton (2005).

3.4.2 Composition and substrate

Beach surface sediment size analysis along the South Canterbury coast has been carried out by several investigators. Hewson (1977) found that the composition of samples averaged 70% gravel and 30% sand, with mean grain sizes ranging from coarse sand (0.9 mm) to pebbles and cobbles (32 mm). Hewson (1977) identified across-shore trends in sediment size, with mean sediment sizes being largest on the backshore and decreasing to the upper foreshore. The mean size then increased slightly towards the recent storm berm and then decreased sharply towards the nearshore zone. Single (1992) identified a similar trend to that of Hewson, with both authors noting a greater spread in sediment size across-shore than any trends found along the coast. Hicks and Todd (2003) analysed surface grainsize data from 1977 and 1994, sampled at the same morphologic points across the beach profile. Although noting scatter between sites, they observed a northward-fining trend for both years.

Stapleton (2005) used an excavator to examine the internal composition of the barrier and to establish the profile of the underlying Pleistocene substrate at several sites between the north Waitaki Fan and Wainono Lagoon. She found that the substrate profile shows several longshore trends, including:

- the depth of the substrate is greater further north, due to the lower elevation of the hinterland and possibly also because the beaches are slightly higher;
- sites fronting Wainono Lagoon showed a distinctive peaked substrate profile; and
- breach sites have very steep foreshore substrate slopes.

Stapleton's interpretations of the substrate profiles, based on her excavations at the Lows Road, Waimate Creek, and Poingdestres Road, are shown in Figure 3-6. The continuous red lines were drawn with confidence, while the isolated red triangles show the depth of excavations that did not discover substrate. Stapleton's excavations into the barrier at Waimate Creek and Lows Road (down to about MSL) did not expose any substrate, so while it is clear that the beach deposits are at least 4-5 m thick under the ridge crest at Waimate Creek, the substrate profile was uncertain there. At Poingdestres Road, however, the substrate (as identified by clay-bound gravels) was detected and sloped-down seaward from the backshore toe of the ridge.

Dickson et al. (2009) used Ground Penetrating Radar (GPR) to probe the internal structure of the beach ridge at sites between Oamaru and the north side of Wainono Lagoon. A profile at Wainono Lagoon (Figure 3-7) indicated that the barrier sediments rested on back-barrier strata at a level of around -1 to 0 m relative to mean sea level, which was confirmed by Stapleton's excavations at that location. The ridge also showed a composite internal structure, formed from episodes of washover and foreshore erosion and accretion.

Stapleton (2005) also analysed the grainsize of samples from the surface and interior of the beach ridge at Waimate Creek. The results (Figure 3-8) showed surface armouring by cobbles, but the interior was dominated by sandy pea gravel (fining to granules at depth) and was typically poorly sorted, comprising strata variously enriched in pebbles or sand (Figure 3-9). This interior material is considered representative of the bulk of material transported by littoral drift, with the coarser surfaced layer developing on the upper beach over time by the winnowing and sorting action of swash.

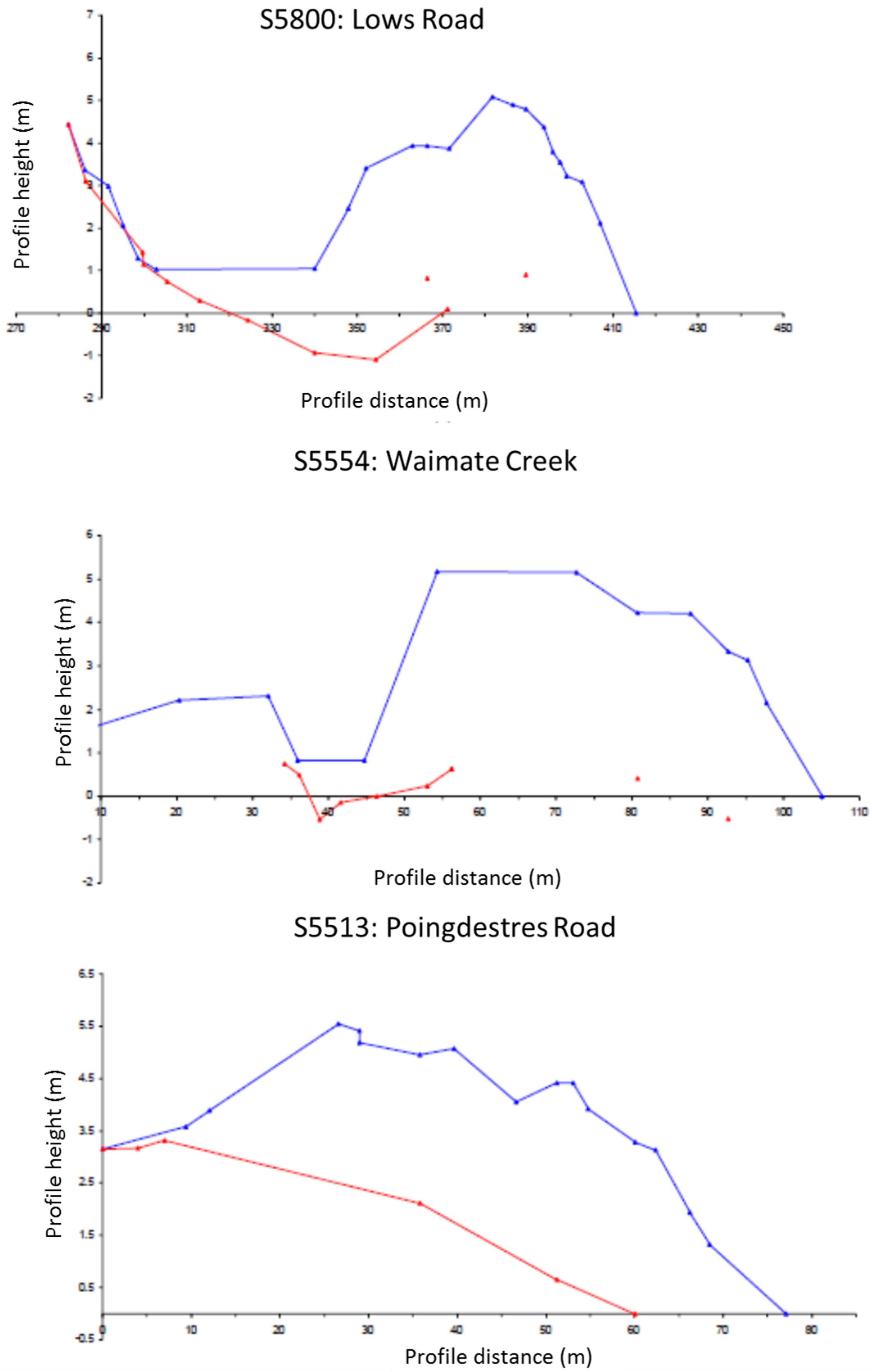


Figure 3-6: Interpolated profiles of substrate under beach ridge at Lows Road, Waimate Creek, and Poingdestres Road. Figure from Stapleton (2005). Red lines show observed substrate profile. Isolated red markers indicate excavations that did not reach substrate.

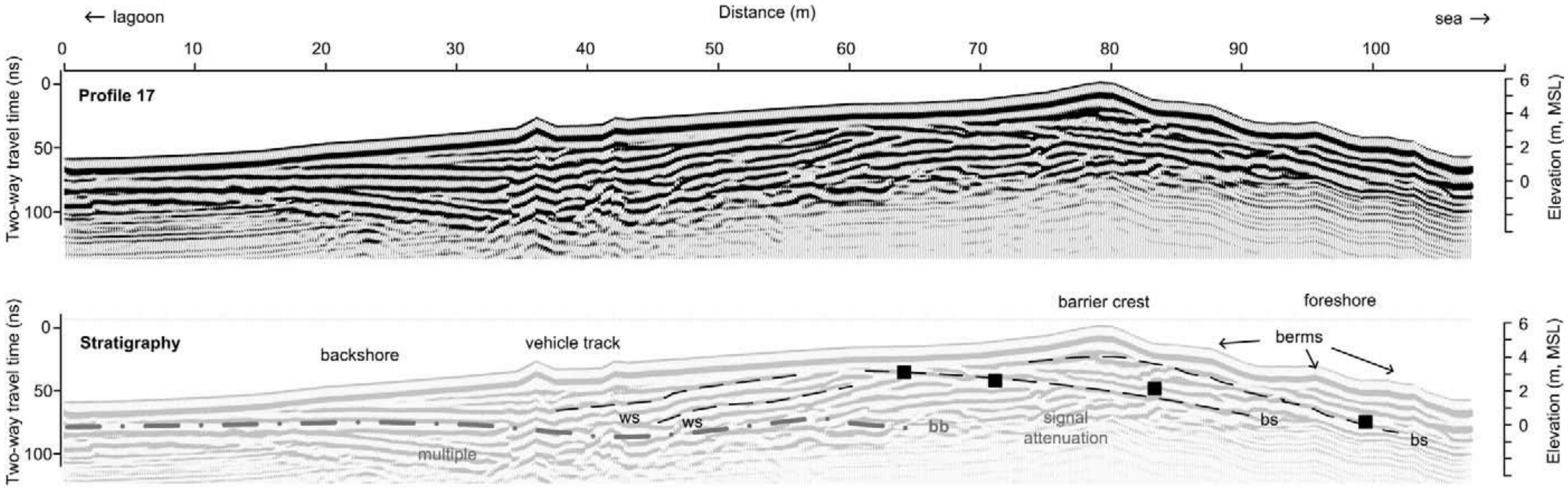


Figure 3-7: Barrier profile and internal structure at Wainono Lagoon obtained using Ground Penetrating Radar. bb = back-barrier sediment, ws = washover surface, bs = beach surface. Dash-dot line interprets position of base of beach sediment. From Dickson et al. (2009).

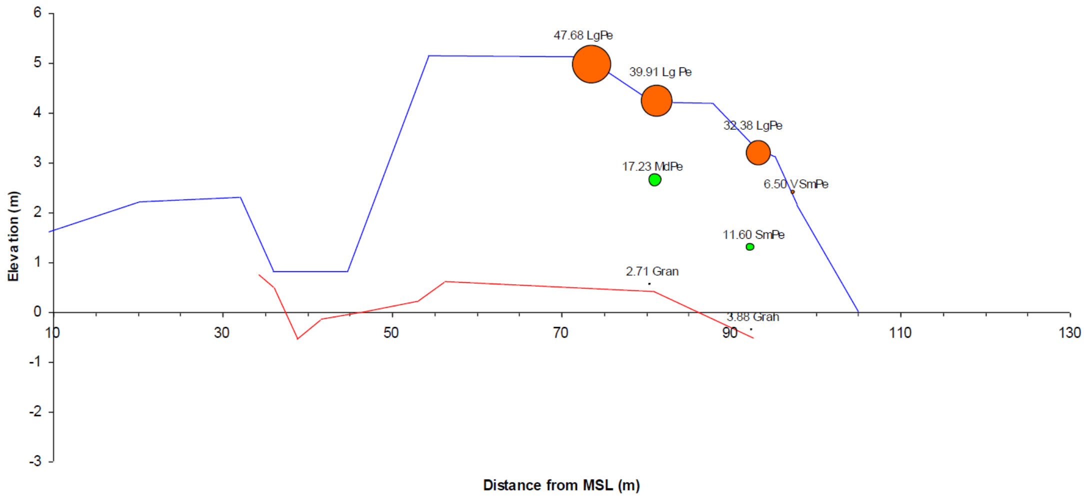


Figure 3-8: Surface (orange) and subsurface (green) median grainsize characteristics of the beach ridge at Waimate Creek. Grainsize (middle) are given in mm. Pe = pebbles, Gran = granules, Lg = large, Sm = small, Md = medium. Red line under ridge is estimated profile of substrate. From Stapleton (2005).

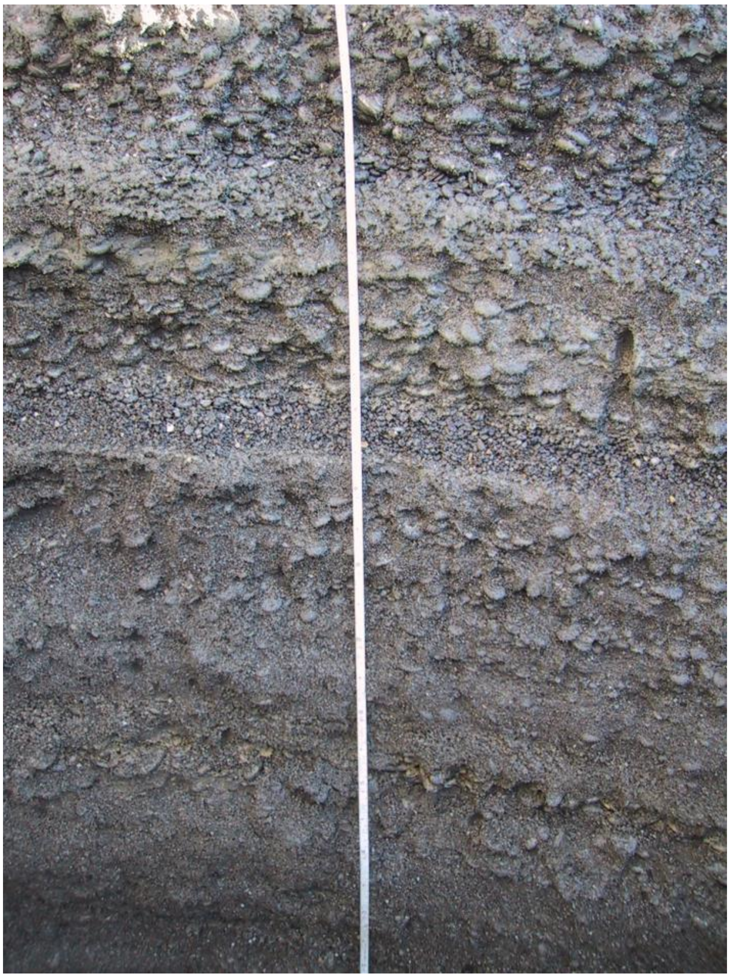


Figure 3-9: Interlayered sand and pebble-rich strata within the beach ridge at Waimate Creek. From Stapleton (2005).

3.5 Beach ridge space-time variability

3.5.1 Spatial variability

Stapleton (2005) undertook a detailed assessment of alongshore variation in beach ridge geometric characteristics for the Waihao-Wainono shore, including crest height and ridge volume above inferred substrate level. She found general trends for ridge height and foreshore and backshore slopes to decrease northward whilst ridge width and volume increased northward (albeit with local variability between adjacent sites).

For the barrier fronting the Waihao Arm, crest height ranged from 5.1 – 5.5 m above mean sea level (amsl) and the average width was 66 m. She noted that the Lows Road and Waimate Creek profiles both had relatively steep foreshore and backshore slopes as a result of breaching in 2002. Mechanical clearance of the Waihao Arm following the breaching had piled overwash material back up on the barrier at the repose angle. These two sites also had the greatest ridge volumes (254.72 m^3/m and 221.97 m^3/m respectively) of all the sites along the Waihao-Wainono barrier by virtue of their deep substrate. Stapleton considered this might be a factor rendering these sites more vulnerable to breaching.

Dickson et al. (2009) found similar longshore patterns of beach volume based on their GPR assessment, noting a substantial increase coinciding with the transition from cliff-front beaches to barrier shore. However, their estimates of beach volume in front of the Waihao Arm were closer to 400 m^3/m.

Hicks and Bind (2013b) used LiDAR data from 2004 and 2013 to obtain continuous plots of longshore variations in beach crest height, width, volume, and foreshore slope. They used the GPR results of Dickson et al. (2009) to estimate the substrate profile for their volume estimates. Their results (Figure 3-10) show:

- trends for crest height and foreshore slope to decrease both north and south of the Waitaki River mouth;
- relatively uniform beach width and volume in front of the cliffed shores but these increased northward along the Waihao-Wainono barrier shore; and
- these patterns persisted in both surveys, albeit with some general reduction in beach volume, width and slope between 2004 and 2013.

Along the barrier fronting the Waihao Arm (northing 5604000 – 5614000 on Figure 3-10) the crest height varied in the range 5.0-6.8 m and changed little between 2004 and 2013 except at two locations where washover appears to have occurred. The barrier volume varied from 150-300 m^3/m (which aligns more with the results of Singleton than Dickson et al.) and decreased by about 15-20% between 2004 and 2013. Crest height figures are updated in Section 4.

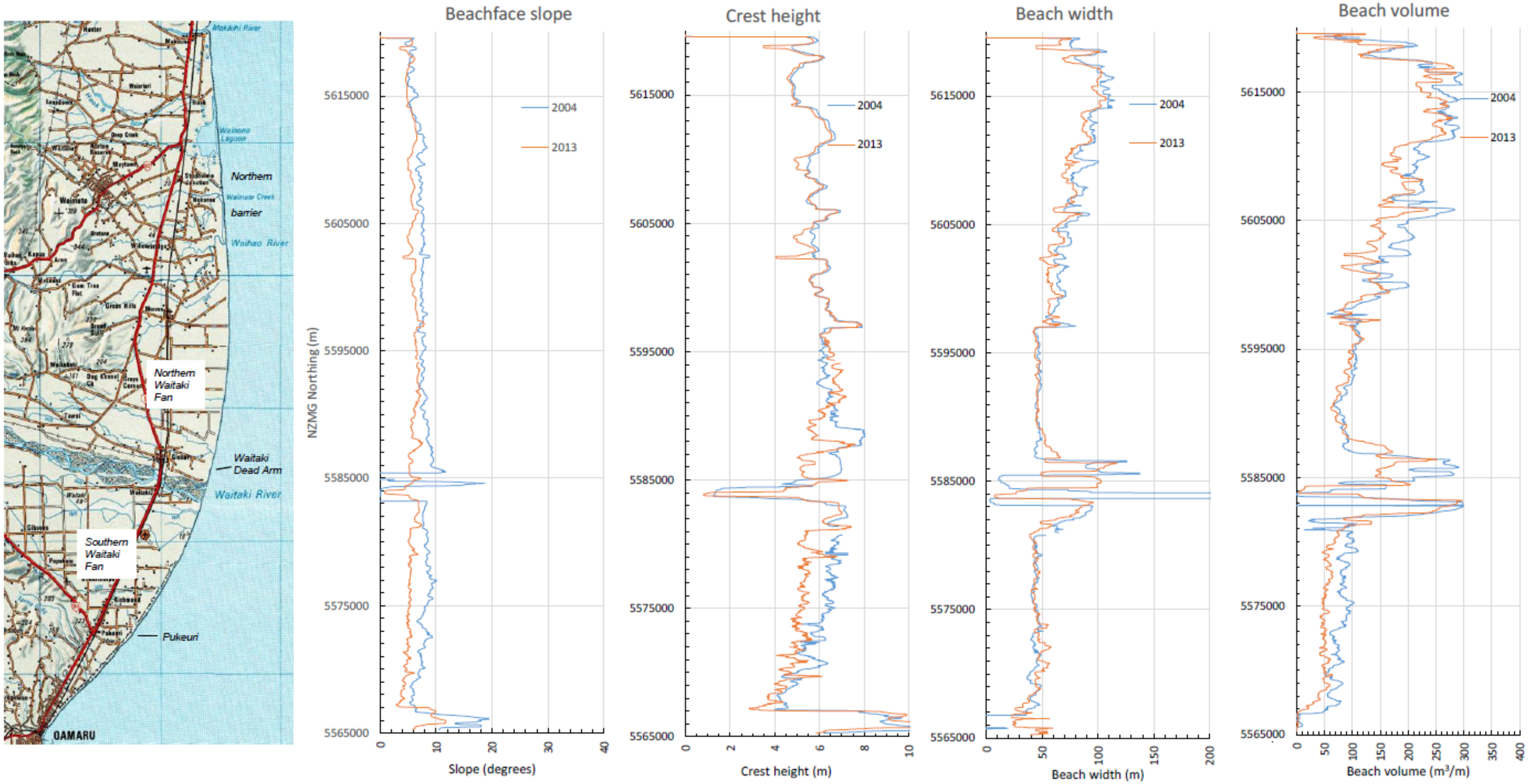


Figure 3-10: Beach characteristics from LiDAR surveys in 2004 and 2013. Beach-face slope averaged over foreshore between 4 and 1 m above MSL contours. Crest-height is peak height of barrier on barrier shores, height of toe of talus or cliff on cliffed shores. Beach width is distance from MSL line to barrier backshore toe or cliff toe. Volume is beach material volume per unit length of shore under beach width and above estimated substrate profile. From Hicks and Bind (2013b).

3.5.2 Temporal variability

While the barrier volume appears to be relatively constant over the long term (see Section 3.3), many sites display a large degree of shorter-term variation in barrier size and shape (Neale 1987; Single 1992; Hicks & Todd 2003; Stapleton 2005; Gabites 2012).

Neale (1987) believed that northward-migrating sediment 'slugs' were responsible for the variability of sediment volume along the barrier over space and time. Single (1992) tested Neale's concept of sediment slugs, confirming that variations in sediment volume occurred along the Waihao-Wainono coast, with the average difference in sediment volume between the slug crest and the slug trough being 50.5 m^3, which is close to the 45 m^3 that Neale calculated. Neale suggested that breaching or overtopping of the barrier would occur in areas of sediment troughs (sites with below average volume).

Hicks and Todd (2003) carried out a linear regression analysis of temporal trends in barrier width, height, and volume at ECan's profile sites. They found it difficult to see a persistent-alongshore, significant temporal trend in any of these features. Only at Poingdestres Road and the Wainono Lagoon sites were there significant trends of reducing barrier height.

Gabites' (2012) analysis of ECan's beach profiles provides an update of Hicks and Todd's 2003 analysis. Gabites' results on volume change from the three profiles closest to the proposed outfall site are given in Table 3-3. He defined beach volume three ways: (i) the volume above a surface connecting the backshore toe and the 1 m amsl contour on the foreshore; (ii) the portion of this volume seaward of the ridge crest; (iii) the volume above a surface connecting the survey start point in the backshore and the 1 m amsl contour on the foreshore. Over the 1977-2011 monitoring period both measures of total barrier volume (i.e., i and ii above) showed significant trends for increasing volume at Waimate Creek and Lows Road, whereas at Poingdestres Road no statistically significant change was concluded.

Table 3-3: Time trends of beach volume at ECan profiles in the vicinity of the proposed outfall. Refer to text for volume definitions. From Gabites (2012).

Volume definition	Lows Road	Waimate Creek	Poingdestres Road
Crest to 1 m amsl	0.00 m^3/yr	0.00 m^3/yr	2.04 m^3/yr
Toe to 1 m amsl	1.47 m^3/yr	1.74 m^3/yr	-0.72 m^3/yr
0.0 m to 1 m amsl	1.76 m^3/yr	1.63 m^3/yr	0.00 m^3/yr

It is of note that at both Waimate Creek and Lows Road, Gabites' whole-barrier volumes showed reducing trends up to November 2001, but between November 2001 and November 2002 the volume increased substantially before remaining quasi-stable. While this 'step' increase was apparently sufficient to create the significant overall trend for increasing volume, this result may be a 'statistical aberration' sensitive to a single large accretion event. Based on the data included here in Tables 3-1 and 3-3, Gabites concluded that the beaches at all three locations were in a state of 'landward translation' – i.e., they were retreating either with stable or increasing volumes.

An update on the analysis of trends at ECan's Waimate Creek profile is provided in Section 4.3.

3.6 Sea flooding

There have been a number of coastal inundation events recorded along the South Canterbury Coast since 1962. This sea flooding occurs through two different processes: beach overwashing and beach failure (or breaching). Overwashing occurs when the maximum wave height exceeds that of the barrier crest, while breaching occurs when a weak area in the barrier loses strength and fails, giving way to water passing through the barrier.

3.6.1 Beach overwashing

Beach overwashing occurs when wave run-up is greater than the height of the beach ridge. The swash, instead of returning to the sea as backwash, runs over the top of the ridge and down the backshore slope. This overwashing also results in the barrier being 'rolled over' in the landward direction, since the overwash cuts sediment from the ridge crest and moves it down the backshore where it is deposited on an apron (Stapleton 2005). Hicks and Todd (2003) describe four situations known to contribute to overwashing[4] (Table 3-4). From observations during storm events this type of process takes place over three to four hours during high tide and occurs for a maximum of two to three tidal cycles. Flooding from this type of event has been quite extensive, especially along the Makikihi to Otaio section of coast where the area behind the barrier is at particularly low elevations. Typically the barrier 'self-repairs' quickly when the sea conditions wane.

Table 3-4: Known conditions for overwashing and breaching events to occur. From Hicks & Todd (2003).

Inundation Processes Along the South Canterbury Coast	
Conditions for *Overwashing*	**Conditions for *Breaching***
High sea level (e.g. high tide) and large storm wave height	High sea level (e.g. high tide) and large storm wave height
Flat foreshore	Steep foreshore and narrow upper beach
Poorly sorted sediment distribution so that percolation into the beach is restricted	Well sorted coarse gravel layer on upper beach on top of a base of finer poorly sorted material
Long-duration event so that the width of the beach foreshore becomes saturated and limits percolation	Steep backshore slope

3.6.2 Barrier breaching

Breaching occurs when percolating wave run-up flows through to the backshore (typically where there is a narrow ridge crest). This collapses the backshore and locally reduces the barrier height, which, in turn, promotes overwash. Breaching generally has a greater effect on the surrounding lowland area than simple overwashing as the volume of water that passes through the breach site is greater than in an overwashing event (Hicks and Todd 2003).

Hicks and Todd (2003) describe four known conditions contributing to breaching (Table 3-4). This was investigated further by Stapleton (2005), who examining whether the nature of the subsurface material under the barrier affected breaching vulnerability. Previously, others (Neale 1987; Single 1992) had concluded that the greater the barrier volume (notably under the foreshore), the greater the amount of protection offered to the land behind the barrier and the less likelihood of a breach or overwashing event. Single (1992) observed that profiles with foreshore volumes over 130 m^3/m and

[4] Hicks and Todd (2003) termed this process overtopping. Here, we restrict the term overtopping to the case where waves just overtop the crest and build-up the crest rather than trimming it off.

widths greater than 35 m sustained less damage to the barrier crest than those with lesser dimensions, and used these findings to provide guidelines for managing beach sizes.

Contrary to Single's findings, Stapleton (2005) found that two of the four past breach sites along the Waihao-Wainono coast had the largest volumes of the 17 sites studied, with all four breach sites having a beach volume in excess of 150 m^3/m. Stapleton concluded that greater volume does not necessarily mean greater protection from breaching; rather, the major factors controlling where a breach may occur are the foreshore and backshore slopes and the depth to the underlying substrate. Stapleton also noted that the permeability of barrier sediments may also be a factor, as her excavations at the two breach sites indicated a lack of sand mixed in with the gravel. Stapleton also noted that all breaches occurred where the sites were backed by water (i.e., the Waihao Arm at Lows Road and Waimate Creek, and Wainono Lagoon at the Wainono North site), and she suggested that hinterland morphology and elevation were also likely to influence breaching.

3.6.3 Inundation history

Hicks and Todd (2003) provided a history of inundation events based on information in Todd (1988), Cope and Young (2001), and various ECan records. This history () shows spates of sea-flooding events recurring on an approximately decadal basis. It is of note that there have been no significant events since April 2002 (Justin Cope, ECan, pers. comm.).

It is also of note that while the longshore span of breaches and overtopping may be relatively limited (as evident by the span-length of low areas in the ridge crest line, e.g., Figure 3-10, and width of overwash aprons in the backshore - which are typically from a few tens of metres to several hundred metres long), the areas inundated can be substantial and slow to drain. This reflects that once flooded, the Waihao-Wainono backshore must drain through the Waihao Arm and the Waihao Box, with the Waihao Arm vulnerable in places to blockage by barrier-breaching and the Box prone to blocking by wave-deposited shingle.

3.7 Backshore engineering works

3.7.1 Stopbanks

The historical sea-flooding events along the low-lying Waihao-Wainono coast have spurred the construction of stopbanks and drains (Single and Hemmingsen 2000). Hicks and Todd (2003) provide the following summary:

- In the late 1940's and 1950's, stopbanks were constructed around the upper Waihao Arm from Wainono Lagoon to Waimate Creek to exclude the lagoon from the farmland on its southern and western sides.
- In 1968 following the 'Wahine' Storm, stopbanks were constructed south of the Makikihi River (750 m long) and north of Sinclair's Creek (750 m long).
- Following sea flooding from two severe coastal storms in 1974, stopbanks were constructed at Hook Swamp (2.5 km long) and south of the Waihao River (1 km long).
- Following two storms that caused sea flooding in 1977, a further 3.6 km of stopbank were constructed on the landward side of the Waihao Arm from Sir Charles Creek to the Waihao River, a 700 m section of stopbank to the north of Waimate Creek was

realigned as a result of coastal retreat, and banks were constructed on the west side of the lagoon.

- In 1984, as part of the Sinclair's Creek Control Scheme upgrade, the coastal stopbanks were extended south of the creek for 750 m to aid the discharge of water through the beach.
- Following coastal storms in 1992, stopbanks were constructed in the Kohika Stream-Otaio area.

Table 3-5: Summary of inundation events along the Wainono Lowlands, 1962-2002. From Hicks & Todd (2003).

Date	Area Affected	Inundation Location
April 1962	400 hectares	Flooded south of Timaru. Although no specific locations are given, it is assumed that some of this would have been along the Waihao-Wainono lowland as damage was reported to the Waihao Box.
April 1968	-	Areas along lowland coast shown in Benn (1988) as being flooded by the 'Wahine' storm.
April 1969	350 hectares	The Waihao Arm blocked by four breaches in the coastal ridge, resulting in flooding. Possibly up to 150 hectares, excluding Wainono Lagoon flooded from Makikihi to Morven, and another 200 hectares in Makikihi to Otaio area.
June 1974	2200 hectares	500 hectares south of Otaio River flooded, and some 1700 hectares experienced interference with drainage due to coastal drains being infilled with sediments
April/July 1977	-	Likely to have been some overtopping along lowland coast.
May 1985	-	Some small-scale breaching at Wainono Lagoon and Waihao Box. Waves removed 29% of upper beach volume.
July 1985	505 hectares	Total of 505 hectares flooded along the lowland coast. Breaches of the barrier ridge occurred at Sinclair's Creek, at Waimate Creek on the Waihao Arm, and Wainono Lagoon.
June 1992	10 hectares	Barrier ridge overtopped, but stopbanks prevented flooding of farmland at all locations except 10 hectares at Kohika stream.
July 2001	1148 hectares	Total of 1148 hectares flooded along the lowland coast. Breaches of the barrier ridge occurred at Wainono Lagoon, Waihao Arm, and at the Waihao Box.
April 2002	625 hectares	Total of 625 hectares flooded along the lowland coast. The location and scale of the inundated areas being similar to the July 1985 event.

The stopbanks have been constructed parallel to the coast and have not always been successful. Bank failure occurs when large volumes of water become trapped between the stopbank and the barrier. Once this water is trapped, drainage of floodwater is difficult. Coastal erosion has a part to play in bank failure, with sea-flood storage areas being reduced in size as the coast retreats landwards towards the banks and ultimately the banks are overwhelmed by the beach ridge.

3.7.2 Waihao Arm clearing

At the Waimate Creek profile site, due to the proximity of the Waihao Arm to the beach, the Waihao Arm has occasionally been blocked off by beach gravels washed over the ridge or as result of breaching. Such blockages contribute to extensive flooding to the properties surrounding Wainono Lagoon and the associated nearby waterways, so they are promptly cleared by excavator, with the material being deposited back onto the beach ridge (Gabites 2012). To reduce the number of blockages that occur in the Waimate Creek area, in 2013 a cut was made by ECan that opened a new channel some 30 m landward of the old Waihao Arm channel. ECan's strategy is to leave the bypassed segment of Waihao Arm channel to fill naturally.

3.7.3 Waihao Box clearing

While the Waihao Box was designed to provide an open mouth in times of normal flow and as a river flood structure during times of high flow (Hicks and Todd 2003), historically the box has been closed by wave-deposited shingle for around 50% of the time. When this occurs, water levels rise behind the box and flow in the Waihao Arm can reverse, flowing north toward Wainono Lagoon. Mechanical assistance has often been required to clear the beach sediment that has built up in front of the box so that the water can be released (Stapleton 2005). A recent (2014) upgrade of the structure was intended to allow it to self-clear, however, too little time has passed yet to assess how effective this has been (Sneddon et al. 2015).

3.8 Wave climate

Davies (1972) classified the South Canterbury wave environment as an 'east coast swell' type. The bulk of the wave energy dispersed on the coast is derived from the southern ocean where most storms are generated (Hicks et al. 2002). Wave refraction takes place along the South Canterbury coast, but because of the large approach angle typical of storm waves (e.g. 45-90 degrees), refraction is often not complete by the time the waves reach the shore. This results in an oblique wave break at the shore which drives longshore transport of beach sediment.

Stapleton (2005) analysed wave data at the 10 m isobath off Wainono Lagoon from the hindcast study of Gorman et al. (2002), which covered the period 1979-1998. This showed (Figure 3-11) waves arriving from the SE quadrant 80% of the time, the highest waves (2-4 m) arriving from the SSE, an overall average significant wave height of 1.09 m, and a seasonal pattern with monthly average heights peaking in June-July (at 1.35 m) and lowest in December-January (0.9 m).

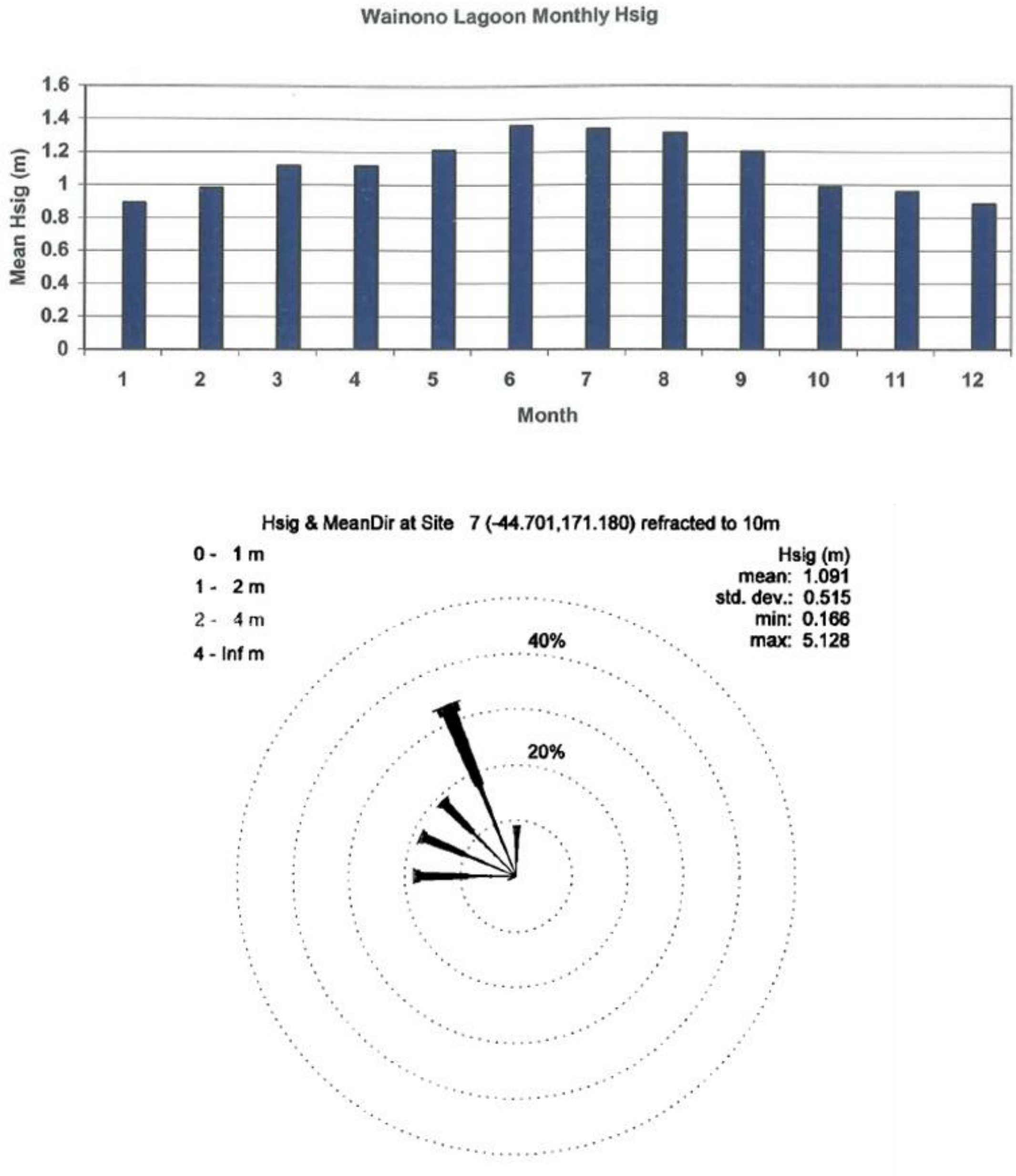


Figure 3-11: Wave climate at Wainono Lagoon, 1979-1998. Top plot shows monthly mean significant wave height; lower plot shows wave 'rose' (note: directions are the direction of wave travel). From Stapleton (2005) using data from Gorman et al. (2002).

Single (1992) proposed that episodes with breaking wave heights in excess of 2.5 m can be considered as high energy events, as waves of this magnitude produce run-up that affects over half the beach profile. These events are classified as either destructive, damaging/constructive, or damaging/erosive. Damaging events had wave run-up of 4.5 m amsl. The main differences between damaging and destructive events were that destructive events had higher storm surge levels and were of longer duration (over 20 hours).

3.8.1 Storm surge

Goring (2004) estimated extreme sea levels on the South Canterbury coast, using sea-level records at Green Island and Timaru as surrogate sites. Through analysis of the tide, storm surge, and mean sea level, Wainono Lagoon was considered to have a 2% annual exceedance probability (AEP) sea level of 1.74 m above MSL (Table 3-6).

Table 3-6: 2% AEP (50 y return period) sea levels reduced to MSL. Rising sea level assumed to be over 50 years at 1.8mm/yr. MLOS= mean level of the sea. From Goring (2004).

Item	Green Island	Timaru
Tide + Storm Surge	1.57	1.51
Monthly MLOS fluctuation	0.08	0.08
Present MLOS	0.00	0.03
Rising Sea Level	0.09	0.09
TOTAL m above MSL	1.74	1.71

3.9 Tsunami hazard

Several large tsunami that originated in South America have impacted the Canterbury coast in the past, including in 1868, 1877 and 1960.

Power (2013a, 2013b) estimated the maximum tsunami wave amplitude around the New Zealand coast for tsunami with return periods of 500 and 2,500 years. The assessment gives a distribution of possible maximum wave heights at the coast for different confidence levels, which takes into account uncertainty in the source parameters[5]. Power's tsunami hazard curves for the Wainono Lagoon area are provided in Figure 3-12. GNS Science recommends using the 2500 year return period at the 84 percentile confidence interval (1 standard deviation above mean) for evacuation planning.

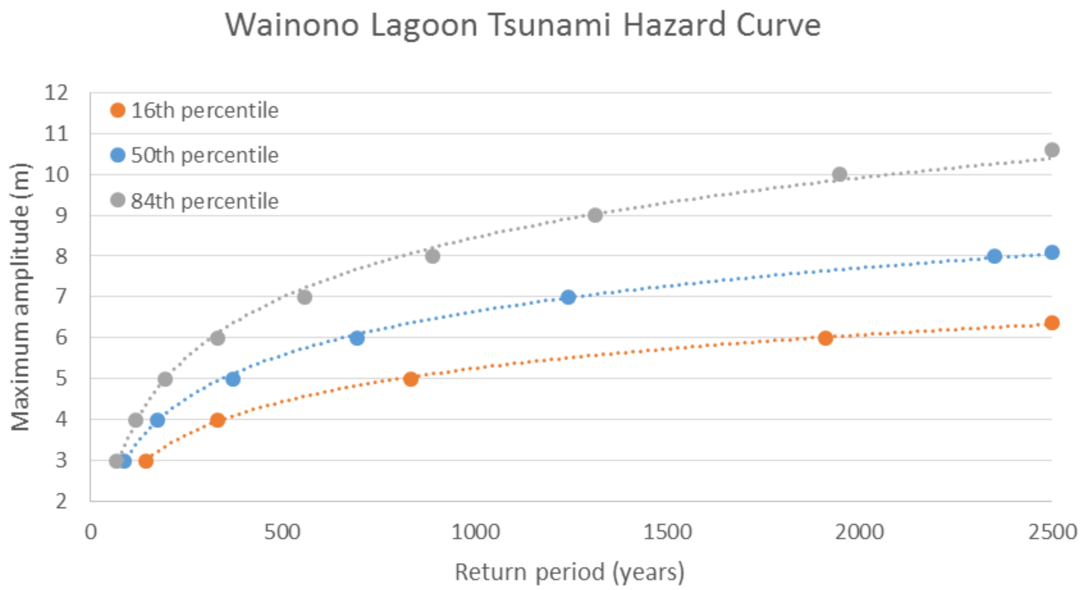


Figure 3-12: Tsunami hazard curve for Wainono Lagoon. Maximum wave amplitude curves are for 16th, 50th and 84th percentile confidence intervals. From Power (2013a).

[5] Deaggregation of the source of tsunami that would generate such an event in the Christchurch region identified south Peru / North Chile as a major source of the hazard (Lane et al., 2014). The magnitude of earthquake needed from this location to give the 2,500 return period wave heights at the Christchurch coast was Mw 9.485. Lane et al. (2014) used this extreme scenario to model the potential inundation caused by such a tsunami around key areas of the Canterbury coast; unfortunately the Lane et al. (2014) report did not include the Waihao-Wainono area.

The actual height of tsunami run-up on the shore depends on the shore and backshore characteristics. Maximum potential run-up is what would occur with no attenuation and if ideal topographic and bathymetric conditions existed for focussing and amplifying wave height - for example, in a steep, narrow bay (Fraser 2014). After Fraser (2014), the rule of thumb for calculating the maximum potential run-up, R (m), is twice the 84th percentile of probabilistic wave amplitude at the coast (as derived by Power 2013a and 2013b). It is of note that this rule-of-thumb for determining maximum potential run up, and the degree of inundation inland, is intentionally conservative as it is used to establish evacuation zones (Fraser 2014). Less conservative estimates are obtained by applying linear attenuation relationships that are dependent on the coastal features of the area of interest.

Using the conservative rule-of-thumb, Power's 84th percentile tsunami hazard curve for Wainono (Figure 3-12) can be converted into a tsunami run-up curve (Figure 3-13). By relating this curve to the height of the barrier we can estimate the return period event that would be required to overtop the barrier. The barrier at the proposed outfall site is approximately 5.6 m high, which Figure 3-13 indicates would be overtopped by approximately a 60 year return period tsunami. As this represents a worst case scenario, alternatively it would be possible to directly relate the 84th percentile tsunami hazard curve (Figure 3-12) to the barrier height assuming no run-up. This suggests that the barrier could be overtopped by tsunami waves generated by a ~270 year return period event. These two results provide bounds around the size of tsunami that could overtop the barrier along the Waihao-Wainono coast, however, the longer return period provides a more likely expectation. Using the latter, the likelihood of an overtopping tsunami is 17% during the 50 year project life and 0.2% during the estimated 6 month construction phase.

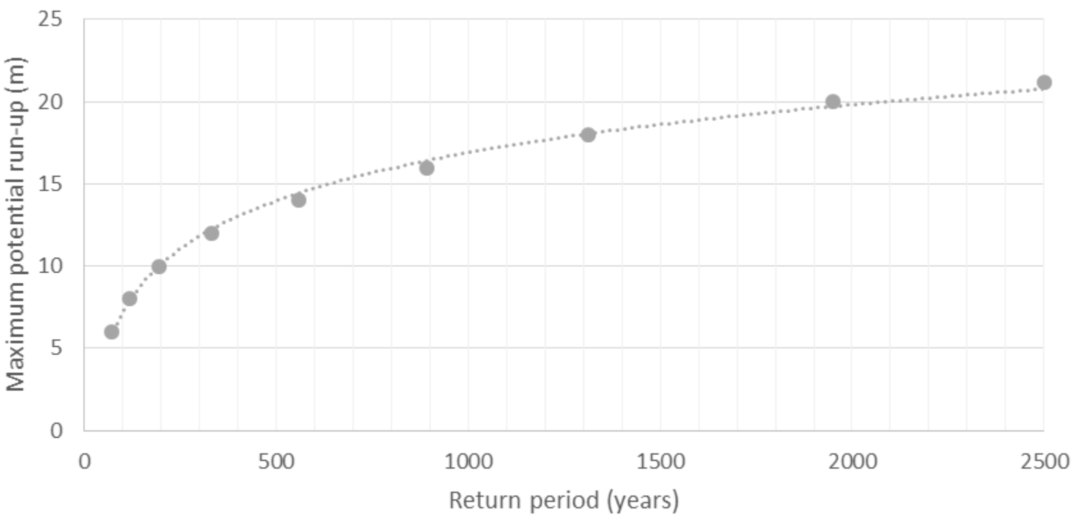


Figure 3-13: Tsunami run-up curve for Wainono Lagoon.

3.10 Clandeboye outfall site

Todd's 2003 study of the Clandeboye outfall area found it had a history of sea water inundation as a result of low ridge crest elevations and poor drainage at the old Orari River mouth lagoons. While previous studies had documented coastal erosion rates in the range of 0.5-1.2 m/yr, Todd's (2003) analysis of survey plans and aerial photographs showed that these high rates of erosion were associated with beach rollover and river-mouth channel-infilling in the period up to the mid 1950's, when the mouth of the Orari River was realigned to its current position. The mouth is now prevented from returning to its former position by lower river stopbanking and mouth training works. Aerial photographs indicate that the beach position has since remained relatively stable over the last 50 years (to date of report), with the ridge crest undergoing intermittent periods of slow erosion and then slow accretion.

Todd's (2003) analysis of ECan's annual beach profiles since 1981 confirmed that the ridge crest had experienced slow steady increases in elevation over most years, interrupted by abrupt drops in height by up to 0.2 m as a result of large storm events. The backshore of the beach had become more vegetated, suggesting that the ridge crest elevation had increased and that washover events had become less frequent. Nonetheless, foreshore retreat and steepening had occurred since the early to mid 1990's. Todd considered that continued steepening and loss of beach volume was likely to result in greater beach instability and reduced ability for the beach to act as an effective buffer to storm wave energy, further increasing its vulnerability to erosion. A similar acceleration was noted at profile monitoring sites south of the Orari River, and these trends did not appear to be driven by changes in storm frequency, sediment supply or swings in weather patterns.

Todd concluded that the worst case storm surge and tsunami scenarios should not result in any inundation risk at the Clandeboye outfall site, providing the ridge crest elevation remaining at least 6 m above MSL.

Todd made the following recommendations relating to the outfall:

- An erosion distance of 25 m should be applied to the beach position over the next 50 years to accommodate on-going retreat and the additional effect of sea level rise. For the 50 years beyond this (2050-2100), an additional 35 m should be set aside for erosion.
- A crest ridge height of at least 6 m above MSL should be maintained at the Clandeboye outfall site to prevent inundation from potential storm surge and tsunami and to prevent accelerated erosion from beach rollover. A beach renourishment programme was recommended to achieve this if this height could not be maintained naturally.
- Ongoing beach profile monitoring should be carried out to ensure that changes in beach height, slope and position are quickly identified, such that appropriate remedial measures can be rapidly implemented.

As detailed in Section 4.2, the Clandeboye site was visited for this study, and up-to-date ECan profile data were examined, to check on Todd's predictions and for any sign of local effects of the outfall on the shore processes and stability.

4 Updated/further analyses of existing environment

4.1 Field inspection and survey

The proposed outfall site was visited on 13 October 2014. The visit included a re-survey of the ECan shore profiles S5554 (Waimate Creek) and S5513 (Poingdestres Road) and surveys along the barrier crest, the beach-face along the 1 m above MSL contour, and the backshore toe of the barrier. These latter three survey lines extended some 450 m either side of the proposed outfall line, which approximately coincides with ECan profile S5554. This span of barrier was also inspected for signs of wave overwashing, while the composition of the bed and banks of the freshly-dug cut-off channel for the Waihao Arm were inspected. The surveying used a Trimble R10 RTK-GPs system, with a single-point calibration to the LINZ benchmark ACGF at SH1 at Studholme.

ECan profile S5554 was extended landward several 100 m to gain a good appreciation of the backshore levels and gradient (Figure 4-1, Figure 4-2). The backshore is approximately 2.60 m above MSL Lyttelton, with a shore-normal gradient of only 0.2 m/km (not significantly different from zero at 5% level). The banks and bed of the new Waihao Arm cut were formed in firm clay, except for isolated loose gravel lobes (Figure 4-3). The clay is inferred to be deposited from backshore freshwater floods, ponded behind the barrier/overflowing from the Waihao Arm, while the gravel lobes will have been from barrier overwash/breaching events. The base of the clay horizon was not exposed in the cut, but a log from a well located some 270 m inland from the barrier showed 2 m of soil-capped clay above a 'tight shingle'[6]. This shingle is inferred to be Pleistocene substrate, which probably extends under the beach ridge (Figure 4-1) – but was not intercepted by Stapleton's (2005) excavations.

The beach crest profile showed longshore variations in crest height, with the height reducing by about 1.3 m in a step northward from NZTM northing 5044465, about 120 m south of ECan profile S5554 (Figure 4-4). This step-down in crest height was matched by evidence of more frequent wave overtopping and overwashing, which are marked by lines of fresher woody debris, fresher backshore gravel (without a lichen cover), and occasional backshore slump features (compare Figure 4-5 and Figure 4-6). The outfall line (crest height 5.60 m) is in the area of lower crest height and so at present is more vulnerable to overtopping than is the shore further south. As discussed in the next section (Figure 4.7), there was a 'step' fall in crest height after 1992 at this location, from ~ 6.3 m on average to 5.7 m, and since 1977 there has been an overall trend for a decline in crest height of 0.033 m/yr.

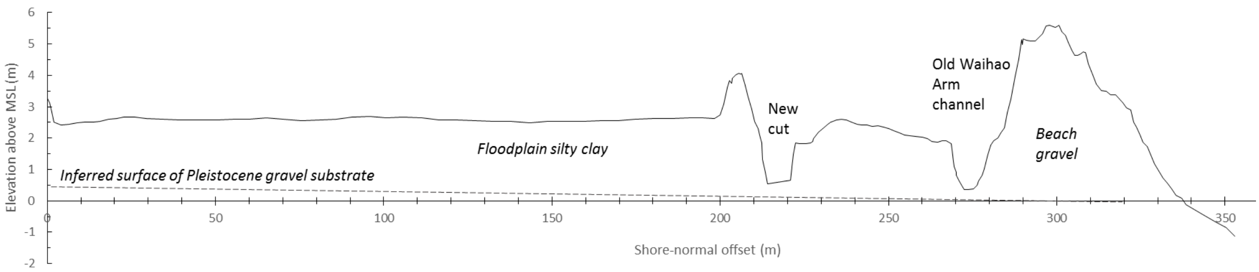


Figure 4-1: ECan profile S5554 at Waimate Creek surveyed 13 October 2014. Note new Waihao Arm channel cut 50 m landward of old channel. Dashed line shows inferred surface of backshore Pleistocene gravel substrate.

[6] Borelog for well J40/0714, drilled by Washington's Exploration Ltd at grid reference J40:6240-0587, ECan well records.

Figure 4-2: View inland along line of proposed outfall. Old, bypassed segment of Waihao Arm in foreground, with barrier-breach gravel deposit on both banks. New cut and Waihao Arm indicated by clay banks in mid-ground.

Figure 4-3: View seaward along proposed outfall line towards new Waihao Arm channel and barrier. ECan profile marker in foreground.

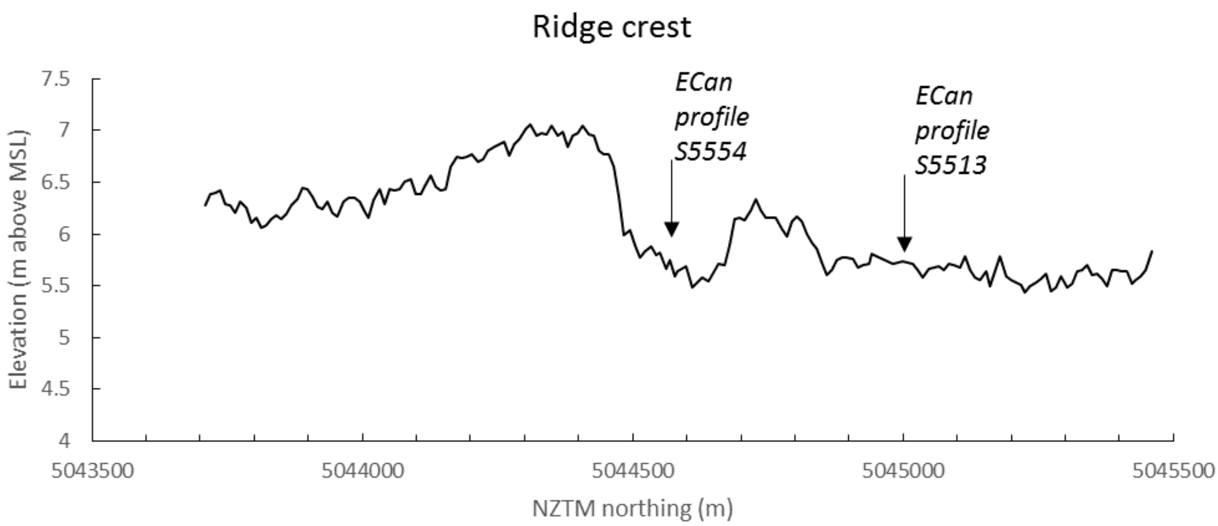


Figure 4-4: Beach ridge crest elevation along a 900 m span of shore centred on outfall location and ECan profile S5554. Surveyed 13 October 2014.

Figure 4-5: View south along barrier and old Waihao Arm channel at proposed outfall site. Note driftwood line showing extent of relatively recent wave overwash, steep backshore slope, and gravel riffle in channel formed in breach deposits. Near-field crest elevation is 5.5 m, rising to 7.0 m in distance.

Figure 4-6: Barrier backshore-slope 200 m south of proposed outfall location. Driftwood line and cobble colour shows extent of wave overwash in recent years. Lichens on cobbles on lower backshore-slope indicate older deposits. Ridge crest elevation is 7.0 m amsl. Photographed 13 October 2014.

4.2 Update on Clandeboye outfall site

The shore at the Clandeboye outfall was also inspected on 13 October 2014. There was no morphological signature of the buried pipeline. While there were signs of a recent episode of dune erosion along the shore (which had left a ‘scarp about 1 m high fronting the foredune), and evidence of sand deposited on the backshore from wave overtopping, these were general along the shore and not confined to the pipeline crossing point. Moreover, the beach contours passed the pipeline crossing with no deflection. While the erosion ‘scarp was deflected shoreward a few m at the crossing point, this appeared to be by chance since the erosion line was ‘scalloped’ all along the shore (Figure 4-7).

Figure 4-7: View north along the ridge crest at Clandeboye. Buried pipeline indicated by marker-post. Photographed 13 October 2014.

An ECan shore profile (RCN0952) coincides with the Clandeboye pipeline. The data from this was inspected for evidence of a change in shore behaviour following pipeline installation in 2004. Figure 4-8 shows the record of beach crest height and position, backshore toe position, and 1 m foreshore contour position at this profile.

These show overall trends for foreshore retreat of 0.39-0.64 m/yr (depending on the feature tracked), which are similar to the rates indicated by Todd (2003), minimal shift in the backshore toe position, and a broadly stable dune crest height (varying to within ± 0.25 m of the average). While there appears to have been a 2-3 m retreat in the foreshore features between 2005 (the year after the pipeline was installed) and the next survey in 2007, if anything this appears to have been a short-term effect either of construction activities or a storm event and did not persist.

Thus, it is concluded that there have been no significant long-term effects of the Clandeboye outfall on shore morphology or processes.

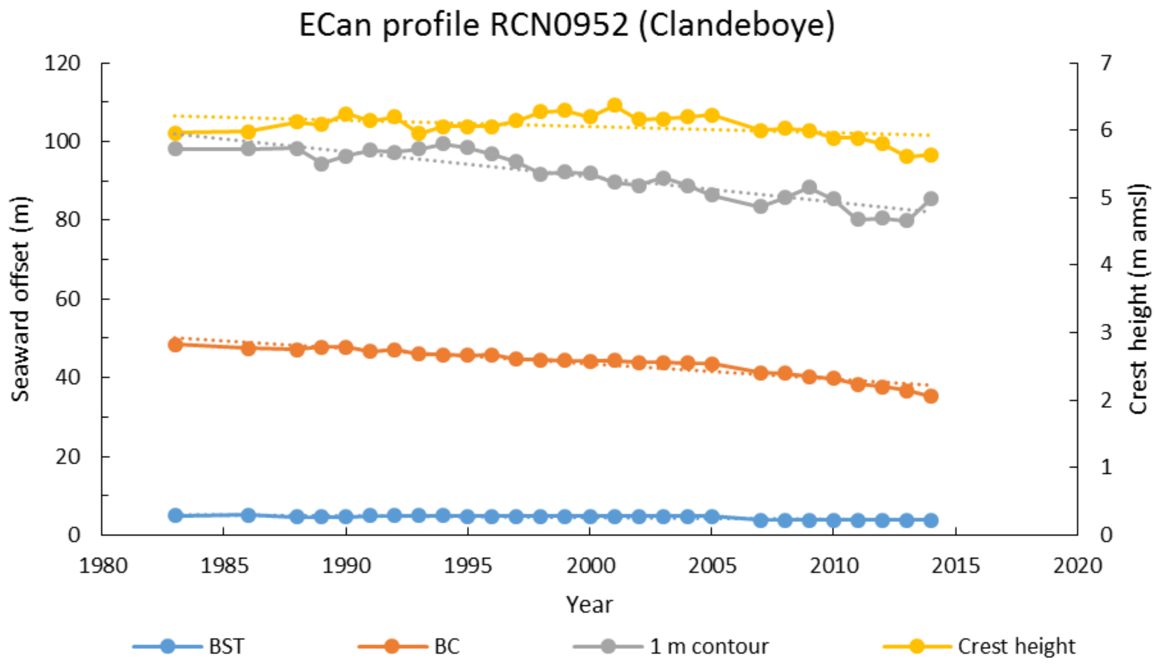


Figure 4-8: Records of beach crest (BC) height and position, and backshore toe (BST) position, and 1 m foreshore contour position at ECan profile RCN0592, along the line of the Clandeboye outfall pipe. Dotted lines show regression trend lines.

4.3 ECan coastal profile monitoring – update

Gabites' (2012) analysis of the ECan profile dataset extended up to the October 2011 survey. ECan kindly supplied data for the 2012 and 2013 surveys, while we resurveyed the Waimate Creek and Poingdestres Road profiles in October 2014. This section provides an updated assessment of the profile dataset at these two locations. We note that we have only used one data point per year, generally surveyed in October[7]. Time trends for key features are shown in Figure 4-9, Figure 4-11, and Table 4-1, while selected profiles are plotted in Figure 4-10 and Figure 4-12.

At Waimate Creek between 1977 and 2014 (Figure 4-9, Figure 4-10), the backshore toe of the beach ridge moved landward 19.5 m, with 12 m of this in association with the overwash events in 2001-2. Also, the backshore slope slumped sometime between October 2011 and October 2012. The foreshore went through erosion and recovery cycles. There appears to be a reasonable balance between the overall volumes lost from the foreshore between 1977 and 2014 and volumes gained on the backshore, confirming a rollover process. However, for individual episodes (e.g. 2001-2), there are imbalances between foreshore loss and backshore gain, which indicates the importance of longshore sediment transfers (e.g., an overwash event could transfer sediment to the backshore then the 'hole' on the foreshore could be re-filled by longshore transport).

The overall trends (Table 4-1) show retreat of the ridge features at 0.34-0.73 m/yr. These are slightly larger than the figures from Gabites (2012), possibly because Gabites appeared to use the full dataset between 1985 and 2011, with several surveys per year between 1985 and 1992 – which would have provided excess weight to the data from those years in the regression fit. In our analysis,

[7] From 1985 to 1992, ECan surveyed these profiles several times per year. Since then they have only been surveyed annually, typically in October.

we used data from 1977 and only used one value per year. We note also that the backshore toe trend rate is essentially set by the time elapsed since the last significant overwash event. Similarly, the overall trend for crest lowering by 0.033 m/yr was largely due to a step-drop in the crest height by ~ 0.6 m in 1992.

We suggest that for design purposes, it would be appropriate to be conservative and assume the largest retreat rate of 0.73 m/yr from backshore toe.

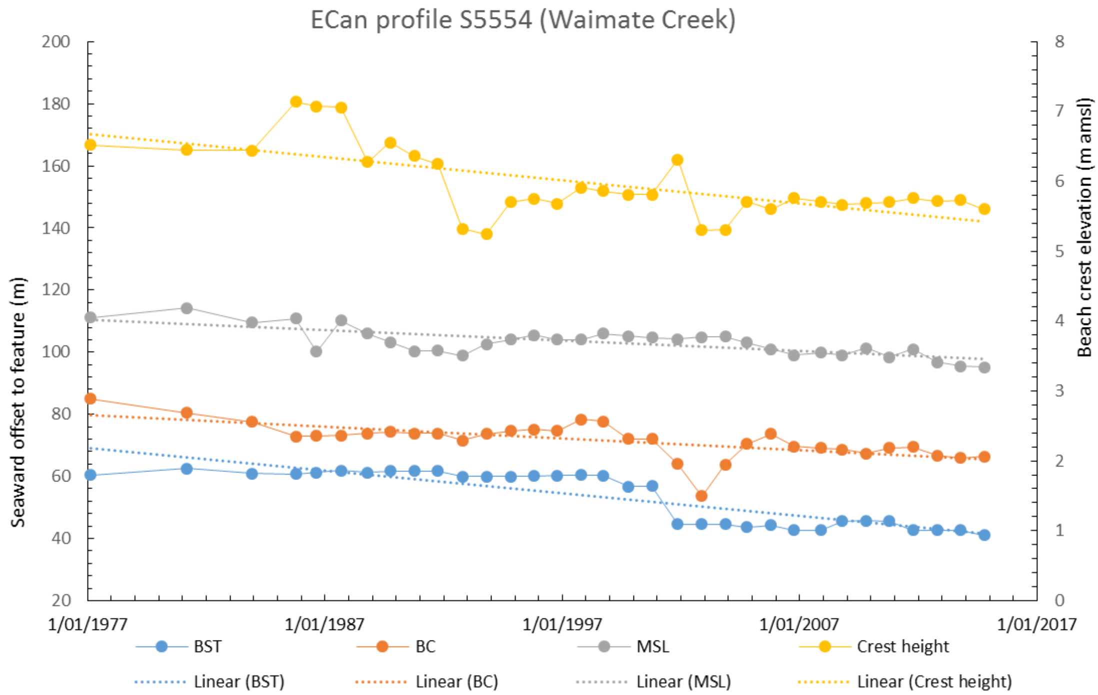


Figure 4-9: Records of seaward offsets to backshore toe (BST), beach crest (BC), and MSL contour on foreshore (MSL), plus beach crest elevation, for ECan profile S5554 at Waimate Creek, 1977-2014. Dotted lines show regression trend lines.

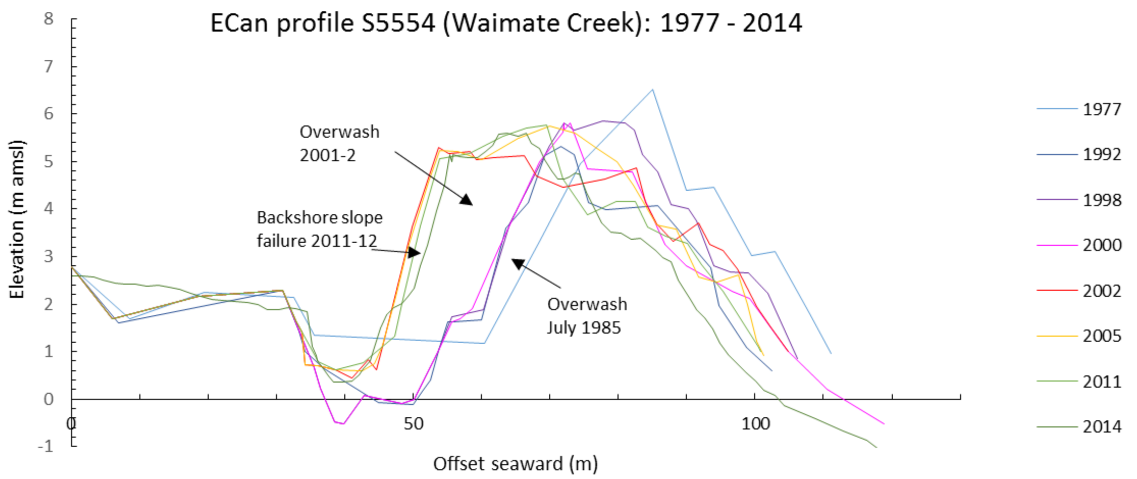


Figure 4-10: Selected surveys of ECan profile S5554 at Waimate Creek, 1977-2014. Backshore deposition/erosion areas associated with overwash/slope-failure events are arrowed.

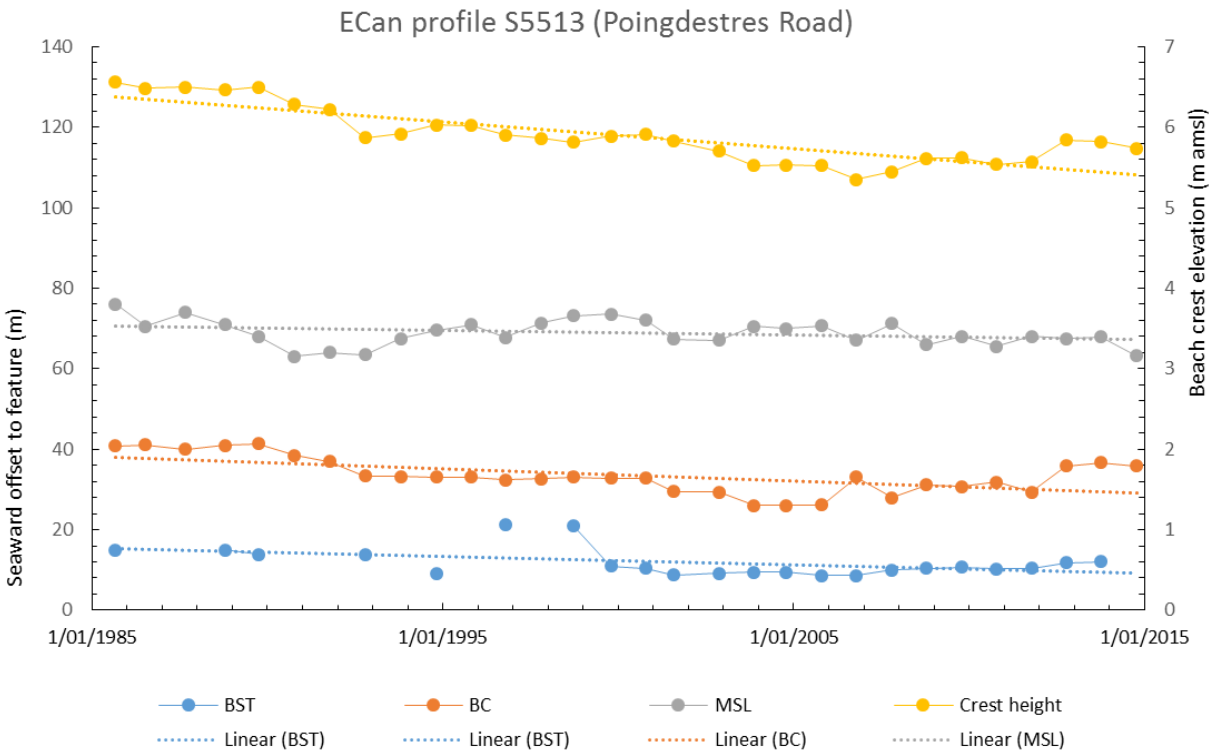


Figure 4-11: Seaward offsets to backshore toe (BST), beach crest (BC), and MSL contour on foreshore (MSL), plus beach crest elevation, for ECan profile S5513 at Poingdestres Road, 1977-2014. Dotted lines show regression trend lines.

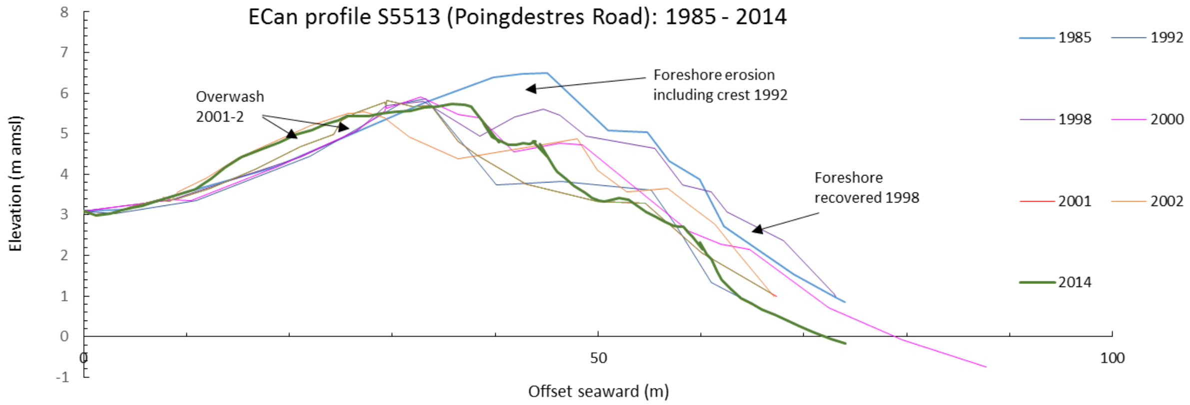


Figure 4-12: Selected surveys of ECan profile S5513 at Poingdestres Road, 1985-2014. Backshore deposition/erosion areas associated with overwash/slope-failure events are arrowed.

The changes at Poingdestres Road between 1985 and 2014 are shown in Figure 4-11 and Figure 4-12. While the backshore toe of the beach ridge moved landward by only a few m in association with overwash events in 2001-2, the foreshore went through erosion and recovery cycles and lost considerable volume overall, particularly during 1992. The much larger foreshore volume changes must be explained by longshore transfers of beach sediment rather than landward transfer over the beach ridge. The overall trends (Table 4-1) show retreat of the ridge features at 0.11-0.31 m/yr. It is of note that these figures differ from Gabites (2012), possibly because Gabites appeared to use the full dataset up to 2011, with several surveys per year between 1985 and 1992 – which would have provided excess weight to the data from those years in the regression fit. As at Waimate Creek, a 0.033 m/yr trend of falling crest height was observed at Poingdestres, and was caused largely by a step-drop in 1992.

Table 4-1: Time trends for seaward offset to backshore toe, beach crest, and 1 m amsl foreshore contour as well as crest height at ECan profiles at Waimate Creek and Poingdestres Road 1977-2014. Values in brackets are from Gabites' 2012 analysis of 1985-2011 data, as listed in Table 3-1.

Feature	Waimate Creek (m/yr)	Poingdestres Road (m/yr)
Backshore toe offset	-0.73 (-0.68)	-0.21 (0.00)
Beach crest offset	-0.37 (-0.34)	-0.31 (-0.59)
1 m amsl contour offset	-0.34 (-0.24)	-0.11 (0.00)
Beach crest height	-0.033	-0.033

4.4 Shoreline stability from LiDAR analysis

The project shore was covered in LiDAR surveys undertaken in 2004 and 2013 (Hicks 2005; Hicks and Bind 2013a). These surveys were analysed in this investigation to provide a 3-d assessment of morphological change and shift of the beach ridge over the intervening 9 years. A 2 km span of shore, centred on the outfall location, was analysed. This provides spatial context to assess how representative the ECan profiles are of the general behaviour of the beach ridge in the vicinity of the proposed outfall.

Net topographical change was mapped by generating digital elevation models (DEMs) from both surveys, then differencing these to produce a DEM of Difference (DoD). The DoD (Figure 4-13) shows a general drop in elevation of the beach ridge focussed on the foreshore, beginning just south of ECan profile S5513 at Poingdestres Road and generally intensifying to the south. Bands of erosion of various intensities along the foreshore indicate the work of multiple events. Only a thin 'thread' of deposition is apparent along the backshore side of the beach ridge, indicating only minor rollover. So, by-and-large, the LiDAR surveys indicate that this span of shore experienced a net volume loss to longshore transfer – as inferred from the analysis of the ECan beach profiles (Section 4.2).

North from Poingdestres Road, there is a more equal balance of erosion and deposition, more areas where the elevation did not change at all, and also a narrow band of rollover deposits on the backshore.

Overall, this analysis shows that in the outfall area, segments of the barrier experience phases of volume reduction by foreshore erosion while adjacent segments show relative stability. Because the foreshore erosion is not balanced by backshore deposition, the foreshore erosion appears to be due to longshore sediment transfer rather than landward rollover. This is consistent with Neale's (1987) migrating sediment and erosion waves.

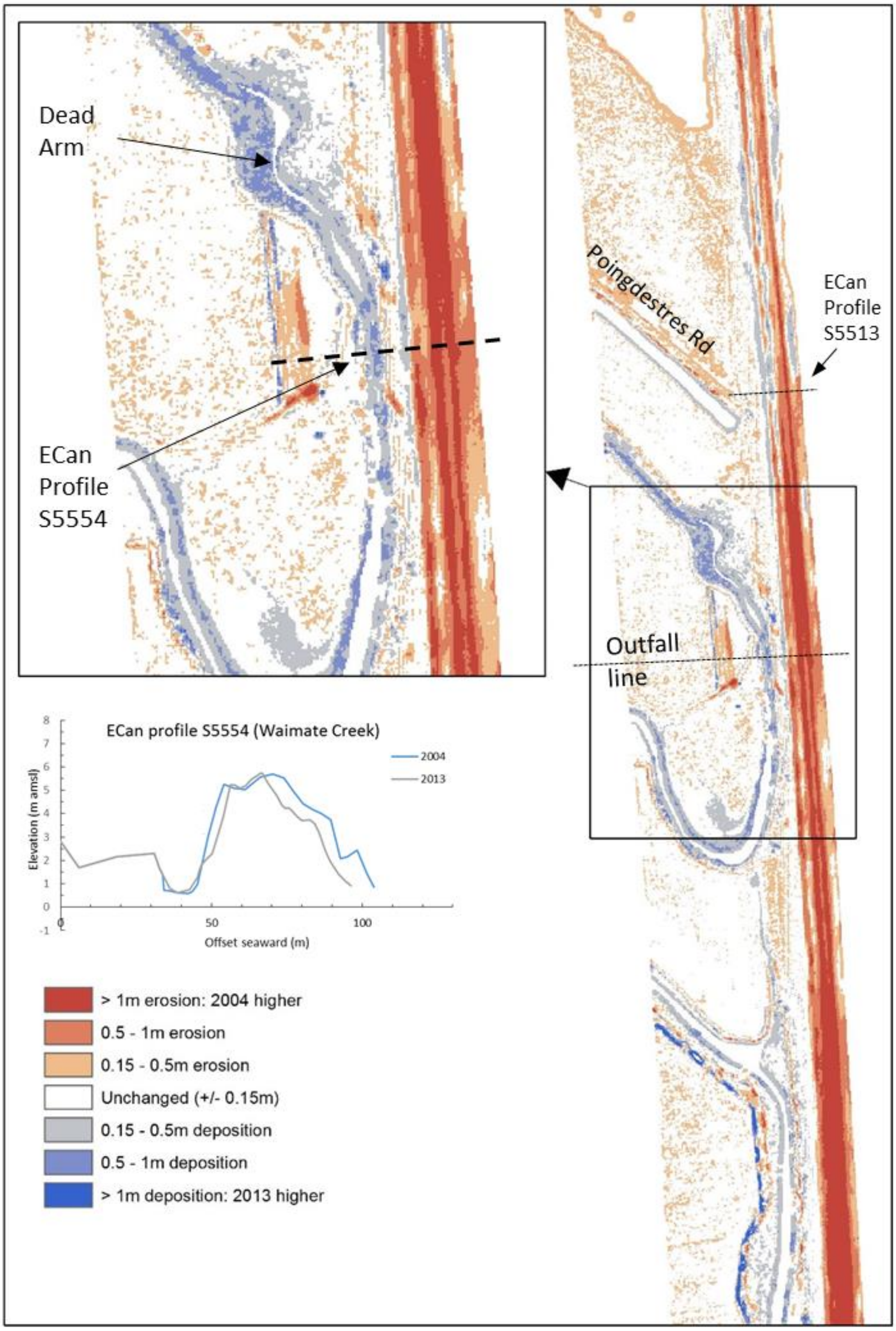


Figure 4-13: Changes in ground levels between 2004 and 2013 LiDAR surveys in area of proposed outfall. Red-orange shading indicates erosion, blue-grey shading indicates deposition. Apparent ground level changes in the Waihao Arm are due to different water levels. ECan profile S5554 surveys for 2004 and 2013 also plotted.

4.5 Updated mapping of shifts in backshore toe line

The 2004 and 2013 LiDAR surveys were also used to update maps of shift in backshore toe position. These were mapped manually from the 2004 and 2013 LiDAR DEMs at the point where the backshore slope reduced significantly, using the concurrent air photographs as additional context. These lines were also compared with backshore toe lines for 1943, 2000, and 1977 that were mapped by Hicks and Todd (2006) using photogrammetric stereo viewing. The results (Figure 4-14) show relatively small-scale and patchy landward shifts of the backshore toe in the project area, with little change between 2004 and 2013. This is consistent with overwash events of limited lateral extent before 2004, pushing shingle lobes into the backshore (red/orange lines on the figure).

In the vicinity of Waimate Creek, the retreat was 4 m between 1943 and 1977, 10 m between 1977 and 2004 (8 m associated with the 2001-2002 overwash events), and - 1 m from 2004 to 2013 (due to the slumping of the backshore slope, which advances the backshore toe seaward). These movements indicate between-survey rates of 0.11, 0.33, and effectively 0 m/yr, respectively, with an overall 1943-2013 retreat rate of 0.19 m/yr. We note that this is less than the 1977-2014 rate of 0.73 m/yr derived in Section 4.3 by regression analysis of the ECan profile data (but note also that the rate derived by either method depends substantially on the chance occurrence of a few significant events over the observation period). We note also that since 2002, when the barrier began blocking the Waihao Arm channel at this location, the barrier backshore slope has been affected by the channel maintenance work. With this, gravel excavated from the channel has been mounded up on the back of the barrier, building a slope steeper than the natural slope and effectively fixing the toe position. This could have locally and temporarily reduced the retreat rate shown in recent years. The regression result would be less sensitive to this. Also, we might expect to see backshore-toe retreat re-commencing now that ECan no longer maintain the old Waihao Arm channel.

Further south, towards Lows Road (at the southern end of the span of shore shown on Figure 4-14), significant landward shifts occurred between 1943 and 1977 and between 1977 and 2004. Opposite Lows Road the retreat was 6 m between 1943 and 1977, 32 m to 2004, and ~ zero to 2013 (indicating between-survey rates of 0.2, 1.2, and 0 m/yr, respectively). As at Waimate Creek, much (23 m) of the retreat between 1977 and 2004 occurred after 2000, in association with the overwashing events of 2001 and 2002 - which aligns reasonably well with Gabites' (2012) results from the ECan profile at Lows Road. The overall 1943-2013 retreat rate indicated at Lows Road is 0.51 m/yr. Also as at Waimate Creek, it is likely that the backshore toe position around Lows Road has been influenced over the past decade or so by channel maintenance work.

In summary, these results:

- Confirm the picture of barrier retreat occurring on an erratic basis in space and time, with limited spans of barrier rolling landward in association with wave washover and/or breaching events;
- Indicate relatively less retreat since the 1940's at the project site compared to further south (in the vicinity of Lows Road and on towards the Waihao Box);
- Show that the Waimate Creek profile appears to be reasonably representative of its immediately adjacent shore, at least at a longshore scale of around 100 m; and
- Show that retreat rates estimated by the change between two surveys tend to be less than those estimated from regression analysis that includes data from intermediate surveys. This may be because the most recently surveyed backshore toe positions have

been affected by Waihao Arm channel maintenance work, thus we recommend taking a conservative approach and using the regression results for design purposes.

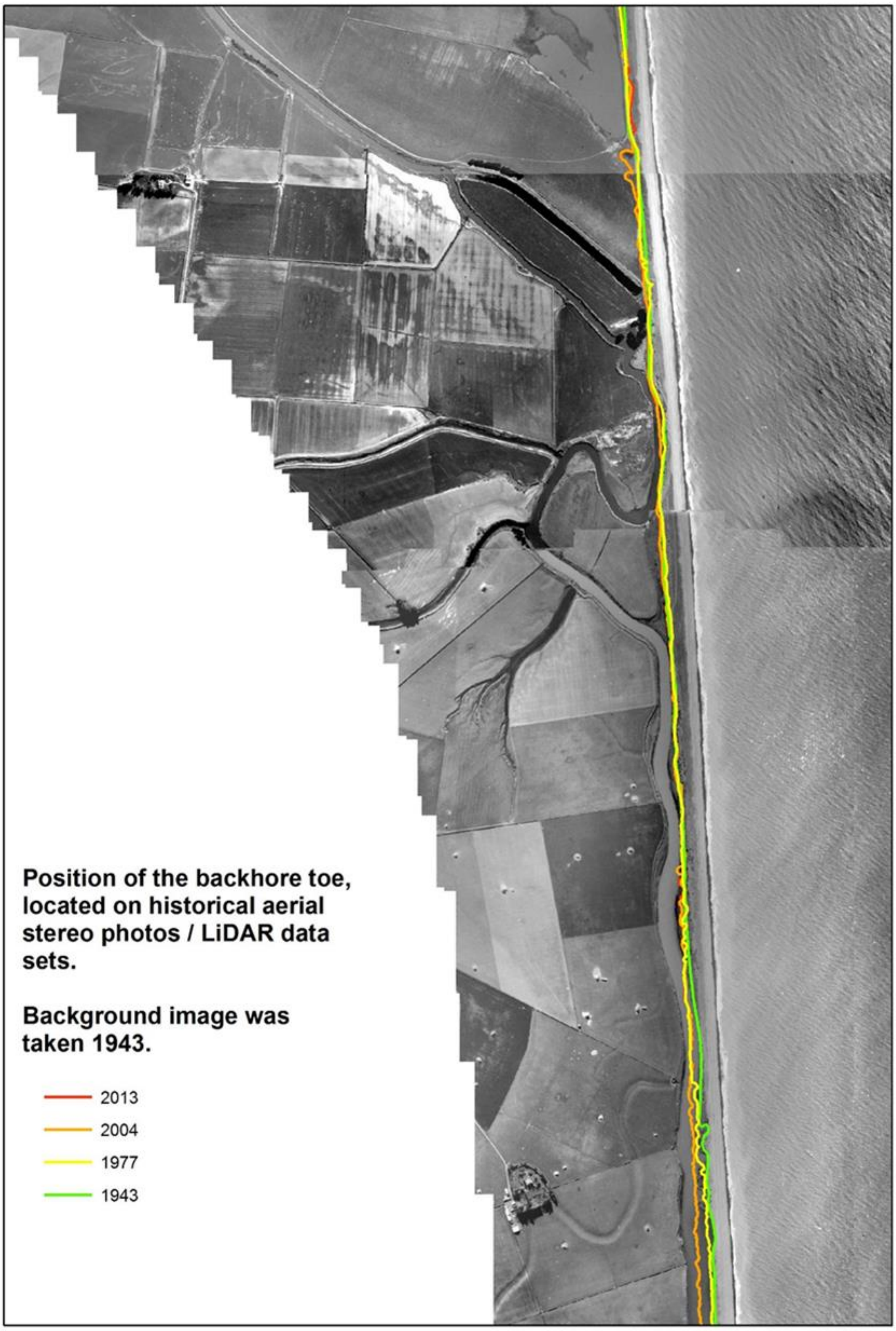


Figure 4-14: Positions of barrier backshore toe along the Waihao Arm from near Lows Road (bottom) to near Poingdestres Road (top), surveyed in 1943 and 1977 using photogrammetry and in 2004 and 2013 with LiDAR. Background image from 1943.

4.6 Coastal drainage management – ECan's plans for the Waihao Arm

In the past two years, ECan has had a bypass channel cut in the Waihao Arm where it crosses the alignment of the proposed outfall. This was done because the previous channel, which ran along the toe of the beach ridge, was being periodically infilled with overwashed shingle and this was compromising floodwater drainage. The new channel is some 60 m landward of the previous channel. At a meeting with an ECan drainage engineer (Leigh Griffiths) and coastal scientist (Justin Cope), they explained ECan had no plans to shift this bypass channel further landward from its present position for at least the next 30 years. They also noted that the old channel would be left to naturally block off as the barrier migrated landward. Assuming a barrier retreat rate of 0.73 m/yr as above, the barrier toe would only advance ~ 20 m toward this new channel over the next 30 years.

4.7 Wave and sea level analysis

4.7.1 Wave hindcast modelling

A wave hindcast record was generated for the proposed outfall site for use in calculating statistics on longshore transport rate and wave run-up and overtopping. The hindcast used the SWANN model developed by Gorman et al. (2002) for the Canterbury coast, and was run for this investigation by Richard Gorman using NIWA's High Speed Computing Facility in Wellington. 'Deep-water' wave conditions at the offshore boundary of the SWANN model (which extended from Wellington to Otago Peninsula) were provided by output from the ECFWM ERA-40 global wind model downscaled by a regional climate model. Hourly wind records from Oamaru airport were input to SWANN to add locally-generated seas to the wave energy crossing the offshore boundary. These two data sources provided a 19.5 year record, spanning the period 1 July 1980 to 31 December 1999[8]. The SWANN model spectrally transforms the wave field as it shoals and refracts across the shelf, and it provided output on spectral peak significant wave height, direction, and period at a station at 10.46 m depth below MSL, 4 km directly offshore of the project site[9]. Breaking wave conditions at the shore were then calculated assuming conservation of wave energy flux and using approximations appropriate for shores with straight and parallel bathymetry contours (which is the case with the project coast), as detailed in Appendix A.

Cross-checks of hindcast wave conditions with those observed by NIWA's wave buoy off Banks Peninsula showed reasonable agreement although sometimes peak wave energy was under-predicted when the model did not simulate rapidly changing pressure systems (R. Gorman, NIWA, pers. comm.). Thus, if anything, this approach may under-predict peak wave heights during some events.

4.7.2 Longshore transport

Longshore transport at the proposed outfall site was calculated using the wave hindcast records. The primary aim was to estimate statistics of monthly longshore transport rates to inform on what might be expected during the construction phase.

The longshore transport calculations used the approach followed by Gorman et al. (2002) as detailed in Appendix A. The transport function was calibrated so that the long term average net northward transport aligned with that estimated through sediment budgeting considerations (~120,000 m^3/yr,

[8] This hindcast modelling was essentially an update of the Gorman et al. (2002) modelling, using higher-quality global wind model output and using Oamaru airport rather than Christchurch airport as the source of nearshore wind data.
[9] At NZMG coordinates 2369000 5606200.

as in section 3.3). With this calibration, the annual net northward longshore transport ranged between 74,000 m^3/yr and 197,000 m^3/yr. Net northward transport was 87% of the gross transport.

Monthly net northward longshore transport for given exceedance percentiles are shown in Figure 4-15. There is a clear seasonal pattern, with higher transport more likely during the winter months, May through September. Lower transport rates are more likely in October through February. In these months, the transport has a 50% probability of exceeding ~7,000 m^3/month and should not exceed ~ 15,000 m^3/month. The maximum longshore transport for any month was 33,000 m^3 in August.

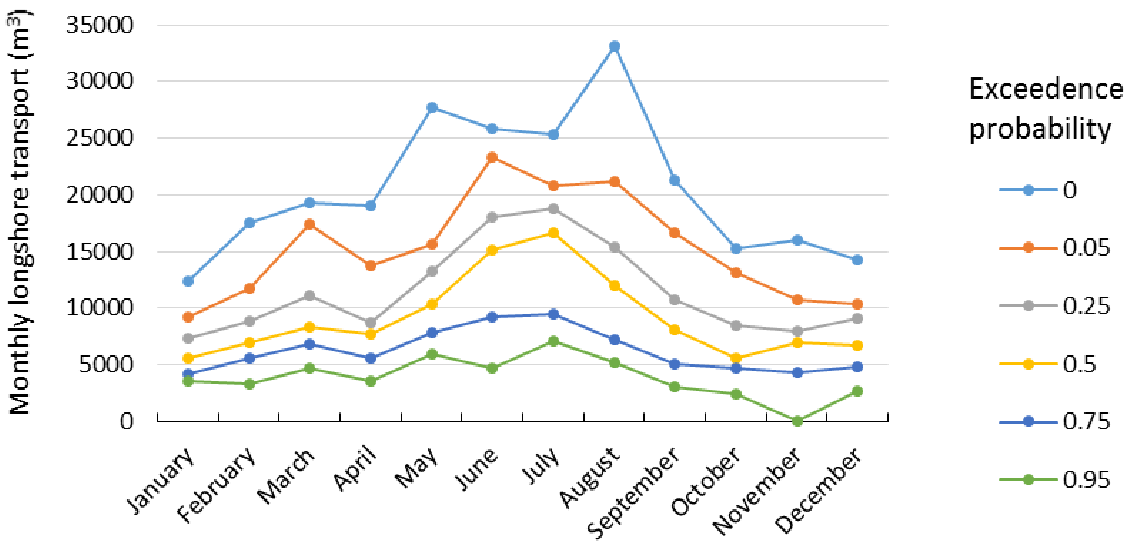


Figure 4-15: Monthly net northward longshore transport past project site at given exceedance percentiles. Based on 1980-1999 wave hindcast analysis.

4.7.3 Wave run-up and overtopping

We investigated the heights of extreme wave run-up to help inform on the return period at which storm waves overtop the barrier at the outfall site. This is of particular relevance to the risk of an overtopping event during the construction phase.

The absolute height of wave run-up on the barrier depends on the storm wave characteristics (height, period, approach angle), the shore characteristics (foreshore geometry, slope, composition), and the sea level that the waves ride in on. Our approach was to generate a 20 year record of hourly absolute run-up height using the output of the wave hindcast modelling described above (shoaled to the breakpoint) combined with a record of sea level.

The sea level combined several components[10]. The astronomic tide, regional oceanic influences, and the "inverse barometer" effect were incorporated in the base sea-level record. This was derived mainly from an edited version of the Port of Lyttelton tide-gauge records (supplied by NIWA Hamilton). Gaps in this record (mainly from mid-1988 through mid-1994) were filled with a synthetic record derived from the tidal constituents and barometric pressure records. This record[11] was

[10] Commonly, the "inverse barometer" effect, wind set-up, and wave set-up are collected together and termed "storm surge" – which gives the net change in sea level against the shore due to meteorological/oceanic events.

[11] We note that the maximum sea-level recorded at Lyttelton over this period was 1.79 m above MSL Lyttelton datum. This is 0.2 m higher than the 2% exceedance sea-level due to the tide and storm surge estimated at Timaru by Goring (2004) – as reproduced in Table 3-6 – thus from a conservative viewpoint we have chosen to work with the observed records.

advanced by 1 hour to allow for the approximately 1.5 hour tidal phase difference between Lyttelton and the proposed outfall site.

To this was added wind and wave set-up against the shore. Wind set-up was calculated using hourly Oamaru Airport wind records with the approach described in Sorenson (2006). This integrates the incremental set-up due to wind stress over the sea surface between a nominal, distant offshore point and the shore, and depends on the wind speed, wind direction with respect to the shore, and the inner shelf to foreshore geometry. The integration was commenced 10 km offshore and assumed an inner-shelf gradient of 6.6 m/km.

Wave set-up, S_w, was calculated using the following equation from CERC (1984):

$$S_w = 0.15h_b - g^{0.5} H_o^2 T / (64 \pi h_b^{0.66})$$

where h_b is the breaking wave depth (assumed twice the breaker height, as in appendix B), H_o is the deep water wave height (generated by inverse-shoaling the 10.64 m depth hindcast waves back to deep water conditions), T is the wave period, and g is the gravitational acceleration.

There are numerous formulae that estimate wave run-up on sandy beaches or against impermeable structures, but we struggled to find any reliable, "established" formulae/relations for shingle beaches. After testing several (Kirk 1975; Dawe 2006; Stockton et al. 2006; Schuttrumpf et al. 2010) against the run-up heights indicated from the historical overtopping record and from foreshore features at the study site (e.g. crest height), we found that the approach of Hughes (2005) developed for impermeable breakwaters, as applied by Shand et al. (2007) on a boulder beach near Raglan, produced sensible results. Hughes' equation is:

$$R_{2\%}/h = 4.4(\tan\alpha)^{0.7}[P_{mf}]^{0.5}r$$

where $R_{2\%}$ is the run-up height that is exceeded by 2% of all waves, h is the water depth at the foreshore toe, $\tan\alpha$ is the foreshore slope, P_{mf} is the wave momentum flux parameter (dependent on wave period, significant wave height at the foreshore toe, and h), and r is the roughness reduction factor. This factor takes a value of 1 for smooth impermeable shores, while Shand et al. found that a value of 0.5 fitted their measurements of run-up on a boulder beach. Thus we estimated r= 0.7 at an intermediate (less rough) value for a shingle beach. We added $R_{2\%}$ to the concurrent sea-level to obtain the absolute run-up height, $R_{a2\%}$.

The record of absolute run-up height so generated is shown in Figure 4-16. This shows absolute run-up peaks typically in the range 3.5-4.5 m (which aligns well with the range of storm-berm heights shown on the Waimate Creek profile, Figure 4-10) and just exceeding 5.6 m (the typical ridge crest height over recent years, Figure 4-9) only three times over the 20 year hindcast period (i.e., in 1985, 1992, and 1999). The historical record () confirms sea flooding events in the area in 1985 and 1992, which provides a reasonable validation of this analysis.

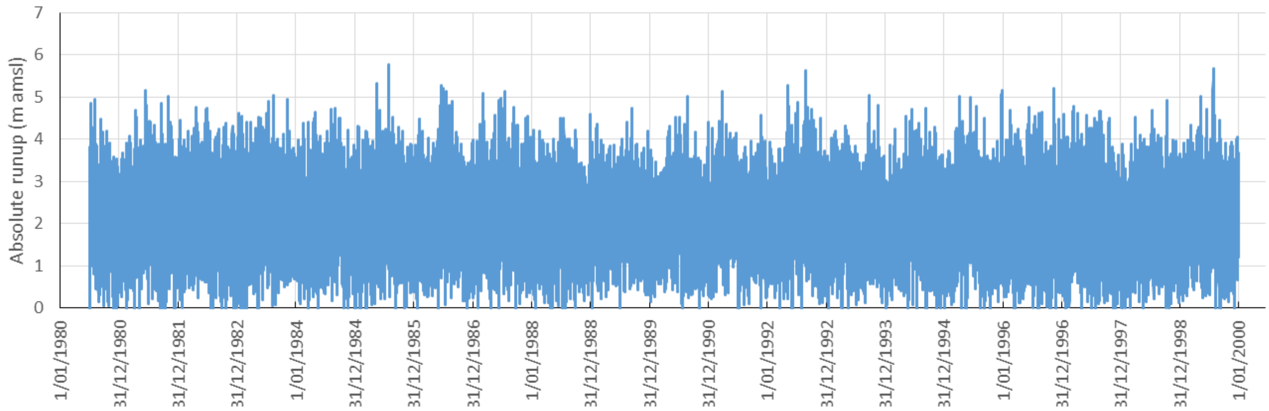


Figure 4-16: Absolute wave run-up, July 1980 – December 2000, derived from wave hindcast analysis and Lyttelton sea level record. Barrier crest at 5.6 m above mean sea level in October 2014.

Figure 4-17 plots absolute run-up annual maxima against return period, T (using the Weibull plotting formula wherein 1/T = k/(1+n), where k is event rank and n is the number of years of record). A regression fit to these data provided the relation:

$$R_{a2\%} = 0.3885 \log_e T + 4.7125$$

where $R_{a2\%}$ is the absolute 2% exceedance run-up height.

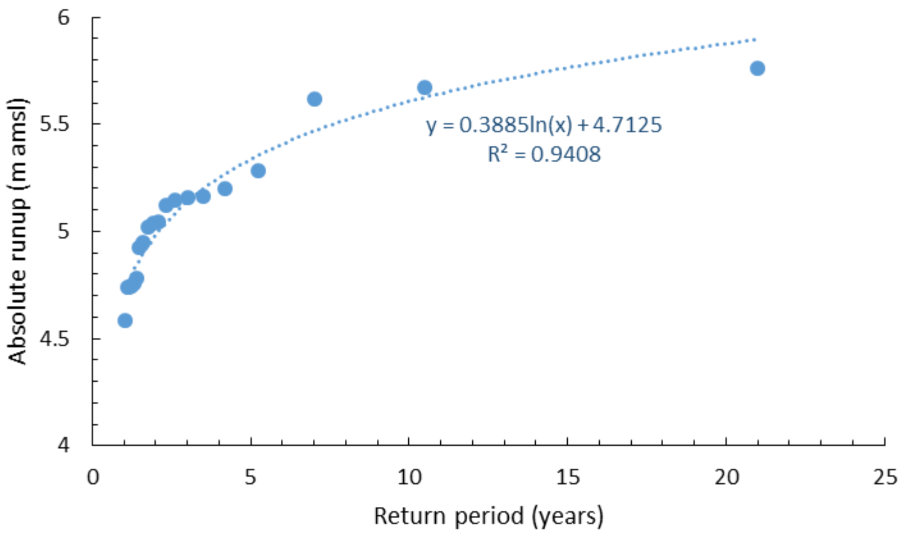


Figure 4-17: Absolute wave run-up (above msl) vs return period at Waimate Creek. Barrier crest at 5.7 m above mean sea level in October 2014.

This relation was used to estimate absolute run-up heights by return periods (Table 4-2). We adjusted these to the present (i.e., 2015) by allowing for a sea-level rise (SLR) of 0.05 m since 1990 (the mid-time of the hindcast analysis). We also estimated the absolute run-up heights at the end of the design life in 2065 allowing for future SLR at rates of 5.2 and 8 mm/yr (figures derived in Section 4.8.2). For example, we estimate the absolute run-up above MSL Lyttelton 1937 datum as 5.66 m for a 10 year event assuming the present mean level of the sea and as 5.71 m by 2065 assuming sea-level rises at 5.2 mm/yr from 2015 until then.

Considering the present, the analysis indicates that the 5.6 m high barrier should be overtopped about every 10 years – confirming the result from compilation of historical events (Section 3.6.3). With SLR, assuming no change in wave climate or barrier height, then by 2065 the overtopping interval would decrease to around 3.5-5 years, depending on whether the base or higher SLR scenario chosen. As discussed in Section 4.8, it is possible that the barrier will increase its height to match SLR, in which case the frequency of overtopping would not change (again assuming no future change in wave climate) – but it also may not.

Table 4-2 indicates that absolute run-up heights over 4.8 m have a high likelihood of occurring during the approximately 6 month period expected for laying the pipeline across the shore. However, the risk of an overtopping event (exceeding 5.6 m) during this period is low (at 5%).

Table 4-2: Absolute run-up elevation by return period at Waimate Creek. Run-up based on wave hindcast and sea level analysis over 1980-1999 adjusted to 2015 and to 2065 taking into account SLR scenarios of 5.2 mm/yr (lower scenario) and 8 mm/yr (higher scenario). Based on data from Section 4.8.2. Elevations are with respect to MSL Lyttelton 1937 datum.

Return period (yr)	Absolute run-up (m) at 2015	Absolute run-up (m) at 2065 for lower SLR scenario	Absolute run-up (m) at 2065 for higher SLR scenario
1	4.76	5.02	5.16
2.3	5.09	5.35	5.49
5	5.39	5.65	5.79
10	5.66	5.92	6.06
20	5.93	6.19	6.33
50	6.28	6.54	6.68
100	6.55	6.81	6.95

4.8 Climate change effects

4.8.1 Overview of climate change effects and guidance

Future climate change could affect the shore at the proposed outfall location through several drivers, including SLR, altered wave climate and storm surge magnitude, and altered sediment budget. All of these can interact to influence barrier processes. On its own, SLR will cause waves to break against the shore at a higher level. Other things staying the same, this will increase the likelihood of overtopping[12] events (which promote barrier height growth) but also overwash events (which lower the barrier and promote rollover – Orford et al. 1995). Similarly, any increase in the magnitude of storm surge will increase the likelihood of overtopping/overwashing events. Exactly how the barrier responds will depend on the relative frequencies of these events, the rate of SLR, the magnitude of storm surge change, and the state of the sediment budget. Changes in the wave climate can influence storm-wave frequency and also the supply of sediment by longshore drift through changes in wave height and approach angle (MfE 2008). The main supply of sediment for the project shore,

[12] An overtopping wave just laps over the barrier crest, depositing sediment on the crest and increasing its height. An overwashing wave is higher and more powerful, sweeping a slice of sediment off the crest and spreading it down the backshore slope, causing barrier 'rollover'.

from erosion of the cliffs fronting the Waitaki Fan to the south, could potentially increase from a higher sea-level (e.g., Forbes et al. 1995), higher rainfall permeating into the cliff sediment, and increased wave power.

Guidance on how sea level, storm surge, and wave climate might change in the future has been offered by MfE (2008). Future changes in storm surge are considered so uncertain that MfE (2008) recommend simply assuming that storm surge levels will rise in tandem with sea level.

With wave climate, a similar lack of knowledge meant MfE could offer little guidance except for some 'broad brush' assumptions for coasts with various aspects and exposure. Specifically for the eastern South Island coast south of Banks Peninsula, MfE recommends, for the period 2050-2100, assuming "a 10% increase in the westerly component of the deep-water wave climate" and "for nearshore wave modelling, assume also a 10% increase in the mean westerly wind component". For the Waihao-Wainono coast, this advice suggests an increase in the deep-water wave climate, since this is dominated by westerly systems in the Southern Ocean, but little effect on generation of onshore-directed nearshore waves, since a westerly wind blows offshore on this coast. Assuming that MfE's 10% increase in the wave climate refers to a 10% increase in wave energy flux, this suggests a 10% increase in longshore transport potential (which is directly proportional to wave energy flux – see Appendix A) and a 4% increase in average wave height (since energy flux is proportional to wave height to the power of 2.5 – see Appendix A). This signals a small increase in wave run-up height, which could slightly increase the frequency of barrier overtopping (if the barrier height does not adjust in response). In regard to effects of wave energy increase on the local sediment budget, a small increase in sediment losses to abrasion is expected but this will also be counterbalanced by an increased supply from littoral drift nourished by increased erosion of the Waitaki Fan cliffs updrift to the south. Thus, the net effect on the sediment budget is too uncertain to make reliable predictions, particularly given the uncertain basis of the MfE (2008) assumptions.

The more definitive guidance from MfE (2008) is around future SLR. In the following, therefore, we focus on assessing potential effects of SLR on barrier stability and retreat at the project site, but recognise that there is uncertainty on the net effects of all facets of future climate change, none more so than possible changes in wave climate.

Scenarios of future SLR associated with anthropogenic global warming have been recommended by MfE (2008) for planning purposes based on findings of the International Panel for Climate Change (IPCC). Specifically for a planning time-frame extending from 2060 to 2069 (which captures the 50 design-life of the Studholme outfall project from the present to 2065), MfE recommend (their Table 2.3) an allowance be made for a SLR of 0.31 m (above the 1980-1999 average sea level), while also considering the consequences of a SLR of 0.45 m.

It is noted that subsequent to the MfE (2008) guidelines, the latest New Zealand Coastal Policy Statement (NZCPS 2010) has issued a policy (Policy 24) that requires identification of coastal hazards, including the effects of climate change and SLR, over at least 100 years (i.e. out to 2115 from the present). Over that time-frame, the MfE guidance is to allow for a base rise of 0.5 m by 2099, a higher rise of at least 0.8 m by 2099, and SLR at a rate of 11 mm/yr from 2100.

For predicting shoreline retreat associated with SLR on barrier shores, the MfE Guidance notes qualitatively that:

- "Where there is a wide and well-nourished gravel barrier (i.e., sufficient sediment supply), the barrier will retreat slightly and increase in height in response to the rising sea level, increase in wave height or increase in the frequency or magnitude of extreme storms.

- Where the gravel barrier system has a net deficit in sediment supply, as is the case of many New Zealand gravel beaches (particularly on the west coast of the South Island), the barrier will experience an increased rate of retreat, or there may even be a breakdown of the gravel ridge. As most of these systems are recessional, future sea-level rise or increases in wave conditions will accelerate this present-day trend."

Beyond this qualitative advice, the complexity of coastal response to multiple factors means that only simple quantitative predictors are available, which MfE (2008) warn are more suited to providing broad estimates of relative erosion potential along a coastline rather than location-specific assessments of potential change, and their use in predicting future coastline position implies a level of certainty that is rarely justifiable. Bearing this in mind, we estimate the future shoreline position at the proposed outfall site for the MfE SLR guidance scenarios using two predictors: the first appropriate for the "well-nourished gravel barrier" situation (first bullet point above), the second assuming a sediment deficit situation. Before doing this, however, we "tune" the MfE guidance scenarios to the outfall project situation.

4.8.2 Project SLR scenarios

The MfE (2008) guidance recommends for the 2060-2069 timeframe that a base SLR of 0.31 m above the 1980-1999 average sea level be considered, as well as the consequences of a higher 0.45 m SLR. In the context of the present proposed Studholme outfall project, which has a 50 year design-life, we consider the expected rises in sea level between 2015 and 2065. Our analysis of the tide-gauge record from Lyttelton showed that between 1970 and 2012, sea level at Lyttelton increased at an average rate of 2.0 mm/yr. Assuming this rate, then a SLR of 0.05 m is indicated between 1990 (i.e., the middle year of MfE's 1980-1999 reference period) and 2015. Thus the remaining rises to consider between 2015 and 2065 are 0.26 m and 0.40 m, at average rates of 5.2 and 8 mm/yr.

Considering the NZCPS (2010) requirement to identify hazards at least 100 years into the future (i.e. to at least 2115), then again using the MfE (2008) SLR scenarios, the base SLR between 2015 and 2115 is 0.61 m (averaging 6.1 mm/yr), while the higher SLR is 0.91 m (averaging 9.1 mm/yr).

In the subsequent analysis the focus is on estimating shoreline retreats over the 50 year time frame associated with the project design life. However, estimates are also provided over the 100 year time frame to comply with the NZCPS (2010) and to forewarn of the erosion hazard should the project life be extended another 50 years.

4.8.3 Shore retreat for case of a well-nourished barrier

In the case of adequately-nourished, narrow-crested gravel barriers (which receive littoral drift sediment and are sometimes termed 'drift-aligned' barriers), the expected response to SLR is an overtopping/overwash process that both builds the height of the barrier and "rolls" it landward as sediment is transferred from the foreshore to the backshore (Shand et al. 2013). After Shand et al. (2013), a generalised "Bruun rule" type predictor equation for this situation is:

$$R = S\,(L + W)/(h_{fs} - h_{bs})$$

where R is the shore retreat, S is the sea-level rise, L is the barrier width seaward of its crest, W is the width landward of the crest, h_{fs} is the elevation difference from MSL to the foreshore toe, and h_{bs} is the elevation difference from MSL to the backshore toe (Figure 4-18).

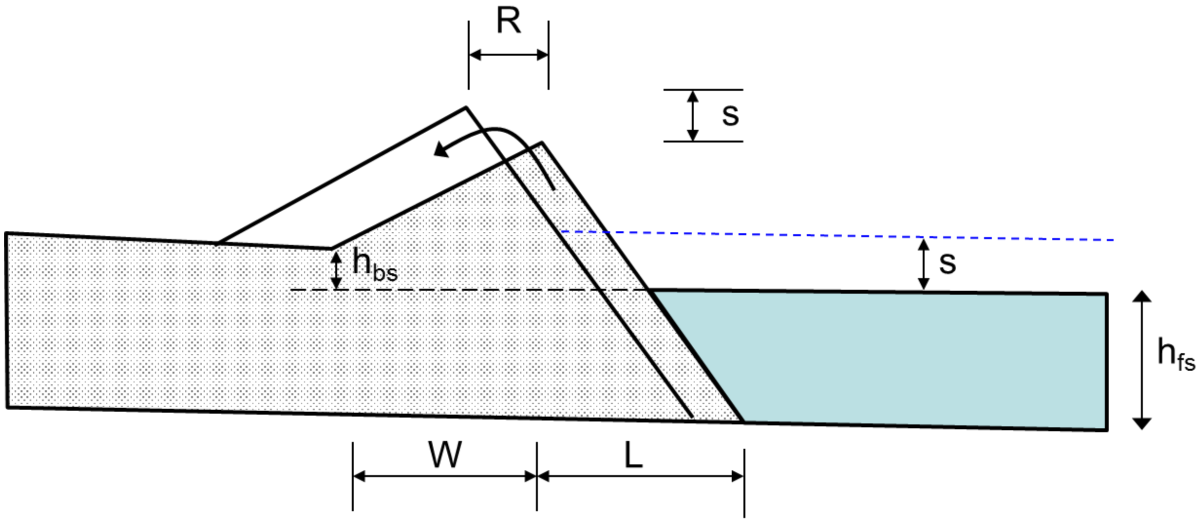


Figure 4-18: Schematic of a gravel barrier response to rising sea level. The barrier crest and backshore rises through overtopping and washover, nourished by sediment from the foreshore.

For the barrier at the outfall site, using data from the Waimate Creek profile surveyed in October 2014 and projecting the foreshore to a depth of 6 m (below MSL) to estimate its toe position, L = 77 m, W = 25 m, h_{bs} = -0.5 (at the invert of the old Waihao Arm channel), and h_{fs} = 6. For S = 0.26 and 0.4 m for the base and higher SLR to 2065, the derived retreats are 4.1 and 6.3 m, respectively (averaging 0.08 and 0.13 m/yr). Combining these results with the largest estimate of historical retreat rate derived previously (0.73 m/yr, Section 4.3) provides a total retreat over the next 50 years of 41 - 43 m.

Repeating these calculations for the next 100 years, with S = 0.61 and 0.91 m for the base and higher SLR scenarios respectively, the total retreat would be 83-87 m.

4.8.4 Shore retreat for a barrier in sediment deficit

Orford et al. (1995) address the case of SLR-driven retreat of swash-aligned gravel-dominated barriers, which are considered to be in a state of sediment deficit and have responded (or are in the process of responding) by aligning their shores normal to the dominant direction of wave approach. Orford et al. found a correlation between mesoscale (1-10 year time-frame) rates of barrier retreat and SLR (as determined from aerial photographs and tide gauges, respectively). They found that the rate of shore response to the 5-yearly average rate of sea-level change (R*, m retreat per mm SLR) depended inversely on barrier "inertia" (I), defined as the product of barrier volume (per unit shore length) and barrier height (between foreshore toe and crest), according to the relation:

$$R^* = 0.9575 - 0.000281\ I$$

with I ranging from approximately 400 to 2800 m^3.

Using Stapleton's (2005) volume of 221 m^3/m for the barrier volume at Waimate Creek, and using a foreshore crest height of 5.6 above MSL (based on our October 2014 profile survey) and foreshore toe at 6 m below MSL (as in Section 4.6.3 above), I = 2564 and thus R* = 0.236 m shoreline shift per mm SLR. This transforms to 50 year retreats of 61 and 94 m for SLRs of 0.26 and 0.4 m, respectively (at average rates of 1.22 and 1.88 m/yr). We propose these as upper-bound estimates for the project design life.

Repeating these calculations for the next 100 years, the estimated retreat would be 144 m and 215 m for the base and higher SLR scenarios (at average rates of 1.44 and 2.15 m/yr).

It is of note that Orford et al.'s relation predicts a retreat rate of 0.47 m/yr at Waimate Creek when the historical rate of SLR over the past century (2.0 mm/yr, from MfE 2008) is used. This lies between the 1943-2013 average retreat rate of 0.19 m/yr observed at Waimate Creek using the LiDAR and photogrammetry results and the 0.73 m/yr rate derived from the 1977-2013 regression analysis of the ECan profile data (Section 4.3). This suggests that much of the historical retreat observed along the Waihao-Wainono shore may be a response to SLR.

We investigated this further by relating quasi 5-yearly shifts in the backshore toe position at the Waimate Creek profile to the average rate of change in sea level at the Lyttelton tide gauge over the same periods (found by differencing the 5 year running mean sea level at the dates of the profile surveys). The results (Figure 4-19) show a very weak (only statistically significant at the 39% level) correlation, suggesting R* ~ 0.08. This is only about 1/3 the R* value estimated with the Orford et al relation and only suggests retreat of 20-31 m by 2065. Also, with a historical rate of SLR of 2 mm/yr, this R* = 0.08 indicates a retreat rate of 0.16 m/yr – which is close to our observed 1943-2013 average rate of 0.19 m/yr. In the end, however, the large amount of scatter on Figure 4-19 and the lack of a significant correlation with mesoscale sea-level fluctuations suggests that over this time-frame extreme wave events are the main driver of barrier retreat at Waimate Creek, and while this may be assisted by a rising sea level the R* values derived should be applied with caution.

Since observed historical retreat at Waimate Creek aligns reasonably with that expected due to historical SLR, it would be inappropriate to estimate future retreat from the sum of historical trends and future SLR.

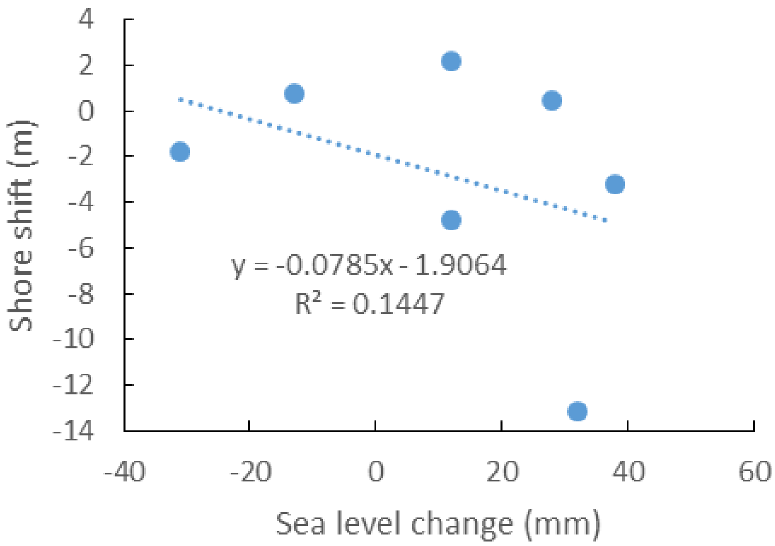


Figure 4-19: Relationship between 5-yearly shift in position of backshore toe at Waimate Creek profile and concurrent change in 5-yearly mean sea level at Lyttelton. Shore shift is positive seaward. Coefficient -0.0785 in regression relation is R* parameter of Orford et al. (1995).

4.8.5 Synthesis

Our previous assessment of the sediment budget of the Waihao-Wainono barrier suggested that in the long term, the barrier appeared to have developed an alignment that enabled it to transfer littoral drift northwards towards Timaru but also maintain its volume against losses to abrasion. Moreover, past studies have shown that since around the 1970's, at least, while the barrier volume has fluctuated (by ~ 50 m^3/m) it appears to have been more or less stable overall. Thus it appears to have had a reasonably balanced sediment budget (if it had not, it would have disappeared long ago). In this case, then, the expected response to SLR would be a barrier crest that increased in height on average in pace with SLR and retreated by only a few m over the project's 50-year lifetime (as in Section 4.8.3), and we should add this modest response to historical retreat rates in predicting future shore positions. This provides us with our lower-bound retreat estimate of 41 m by 2065.

However, there is also no doubt that historically the barrier has retreated at the same time as sea level has been rising at modest rates. Moreover, the historical retreat rates (which have varied alongshore and with time in response to the patchy processes effecting rollover) are of the order of those expected from a barrier of similar size in a supply-limited, swash-aligned situation (as per the Orford et al. approach). Quite possibly, this historical response reflects retreat at times of high storm waves and at places where the barrier is locally and temporarily in sediment deficit (which we know occurs). This being the case, it would be prudent to plan for the higher retreat extents estimated in Section 4.8.4, while also accepting that these could be less or more depending on how the wave climate might change.

Thus, it is recommended that the project plan for a retreat of 61 m by 2065, but also have a contingency for a greater retreat up to 94 m to cover the case of an upper-range rise in sea level or an unfavourable change in wave climate. This means locating the surge chamber at least 94 m back from the present position of the backshore toe of the barrier.

5 Planning instruments

The following section provides an outline of the planning instruments used on a national and regional basis with respect to coastal environments. These instruments are a series of policies and plans which set rules around activities permitted in a coastal environment in order to manage issues of concern, such as coastal erosion and inundation events along the Waihao-Wainono Coast.

5.1 New Zealand Coastal Policy Statement

The New Zealand Coastal Policy Statement (NZCPS 2010) is a national policy statement under the Resource Management Act 1991 (RMA). The purpose of the NZCPS is to outline policies that will help achieve the purpose of the RMA in relation to the coastal environment of New Zealand.

The NZCPS lists a number of particular challenges in promoting sustainable management and outlines several key issues faced by the coastal environment. One of these key issues is "continuing coastal erosion and other natural hazards that will be exacerbated by climate change and will increasingly threaten existing infrastructure, public access and other coastal values as well as private property".

The NZCPS then states a number of Objectives based on the issues faced by the coastal environment. Objective 5, for example, relates to the issue of coastal hazards. This objective is:

"To ensure that coastal hazard risks taking into account climate change, are managed by:

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

In order to meet the stated objectives the NZCPS then provides a series of detailed policies. For example, there are a number of policies relating to coastal hazards, including:

- Policy 24 – Identification of coastal hazards over at least 100 years;
- Policy 25 – Subdivision, use, and development in areas of coastal hazard risk;
- Policy 26 – Natural defences against coastal hazards; and
- Policy 27 – Strategies for protecting significant existing development from coastal hazard risk.

The NZCPS also states that regional policy statements, regional plans and district plans must give effect to the NZCPS.

5.2 Canterbury Regional Policy Statement

The Canterbury Regional Policy Statement (CRPS) sets the framework for resource management in Canterbury. It provides an overview of the significant resource management issues facing the region, and sets out objectives, policies and methods to address the region's resource management issues. Its goal is the integrated management of the region's natural and physical resources.

Chapter 8 of the CRPS covers the objectives and policies relating to the coastal environment and Chapter 11 covers those relating to natural hazards.

5.3 Regional Coastal Environmental Plan for the Canterbury Region

The purpose of the Canterbury Regional Coastal Environment Plan (RCEP; ECan 2005), which became operational in 2005, is to promote the sustainable management of the natural and physical resources of the Coastal Marine Area and the coastal environment and to promote the integrated management of that environment. In particular the RCEP sets out the issues relating to:

- protection and enhancement of the coast;
- water quality;
- controls on activities and structures; and
- coastal hazards.

The RCEP sets out objectives, policies and methods including rules to resolve these issues and to improve the coastal environment. This plan has regard to and is consistent with the CRPS and the NZCPS[13].

The RCEP specifies two issues for the South Canterbury Coast:

- coastal erosion that is occurring at a rapid rate, limiting use of the coastal strip and leading to the loss of wildlife habitats in coastal lagoons and wetlands; and
- industrial discharges causing localised reductions in water quality.

Chapter 9 of the RCEP for the Canterbury Region covers coastal hazards, including coastal erosion and sea water inundation. In considering a planning response to avoid or mitigate these hazards, Environment Canterbury has defined Hazard Zones along the regions coast. Two zones are defined:

- Hazard Zone 1 – is a line approximately parallel with the shoreline, set inland from the high water springs mark, which contains the current active beach system and land that is at risk from coastal erosion within 50 years of the RCEP being produced; and
- Hazard Zone 2 – is inland from Hazard Zone 1 and marks land that is at risk from coastal erosion in the period 50 to 100 years of the RCEP being produced.

The widths of the coastal hazard zones were based on understanding of coastal erosion rates (e.g. Benn 1988). Maps showing the hazard zones in the region of the proposed outfall is presented in Appendix B. At the proposed outfall location, Hazard Zones 1 and 2 extend ~ 30 m and ~60 m[14], respectively, behind the landward toe of the beach. We note that the Hazard Zone 2 line coincides with our upper-bound, 61 m by 2065 retreat distance estimated in Section 4.6.

The RCEP states a set of rules relating to Hazard Zones 1 and 2, including Permitted Activities (Rule 9.1), Discretionary Activities for which Discretion is Restricted (Rule 9.2)., and Prohibited Activities for which no resource consent shall be granted (Rules 9.3 and 9.4 for the respective Hazard Zones 1 and 2). The pipeline to the proposed ocean outfall will pass through Hazard Zones 1 and 2 and hence under Rule 9.2 of the RCEP the following activities associated with the proposal would be discretionary within Hazard Zones 1 and 2, and therefore require land-use consents for:

[13] The RCEP would have considered the NZCPS that was operative in 2005. The NZCPS has now been updated (NZCPS 2010).
[14] These figures are accurate only as far as the lines can be scaled off the maps in Appendix B.

- the erection of any structure;
- the disturbance of any vegetation within the active beach systems;
- the artificial adjustment of the beach profile within the active beach system;
- excavation and filling in volumes greater than 5 m^3 per 100 m^2 of land area; and
- the removal of sand, rocks, shingle or shell or other natural material from an active beach system in volumes greater than 5 m^3 within any 12 month period.

Under Rule 9.2 the discretion of the Regional Council is restricted to the following matters:

- whether the activity is likely to exacerbate coastal erosion; and
- whether the activity is likely to lead to adverse effects from natural hazards on any other property; and
- provision for the removal of any structure or parts of any structure that will be rendered unusable through coastal erosion.

We are not aware of activities associated with the proposed ocean outfall that would be prohibited in Hazard Zone 1 and 2 under rules 9.3 and 9.4 respectively of the RCEP.

6 Effects of the Studholme project

6.1 Proposed outfall design and construction

The design of the proposed outfall is illustrated in Figure 6-1 (Kristian Nelson, McConnell Dowell Constructors Ltd, pers. comm.). This shows the 0.6 m outside-diameter buried pipeline passing beneath the new Waihao Arm channel (at 2 m below the invert level) then sloping under the barrier to intersect the nearshore seabed at the beach toe (-6 m from MSL). From there, the pipeline will run across the seabed, supporting its own weight on the seabed, out to the discharge point at a depth of ~ 10 m. The pipe will be flexible. A surge chamber in the pipeline is expected to be located in the field west of the New Waihao Arm.

The pipeline will be laid through the barrier and into the nearshore by excavating a 3-4 m wide trench walled all around by sheet-piles and pumped dry (forming a coffer dam). The coffer dam will rise to a height of 6 m above MSL and extend some 150 m from the backshore toe of the barrier to beyond the foreshore toe. After laying the pipeline, the sheet-piling will be removed and the trench refilled with beach material that was stockpiled during its excavation. The pipeline laying across the shore zone is expected to require 4-7 months (nominally 6 months for risk-estimation purposes).

The pipeline will be laid under the new Waihao Arm channel first. During this stage, the new channel will be isolated by bunds while flows through the Waihao Arm will be routed through the old channel. Then, the new channel will be re-opened, the old channel bunded, and the pipe laid under the old channel.   All bunds will be removed after the pipeline had been laid.

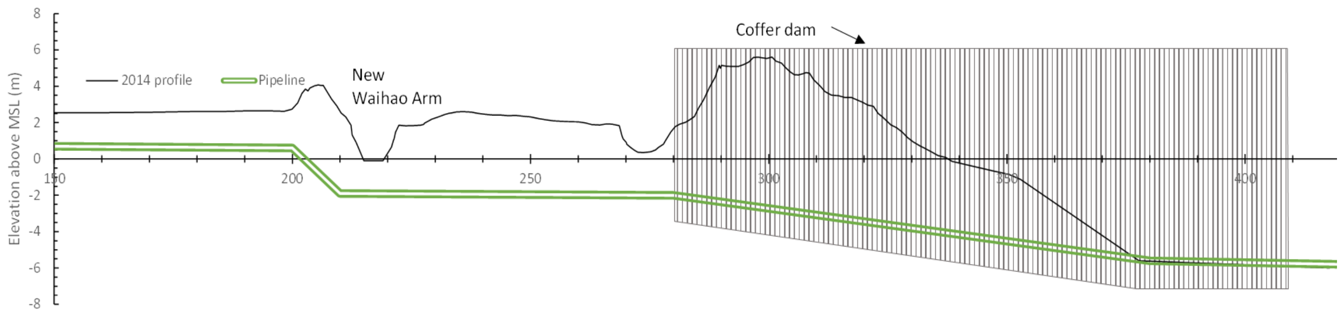


Figure 6-1: Proposed design pipeline profile where it passes under the Waihao Arm and beach ridge, along with Coffer dam profile.

6.2 Potential issues during construction phase

Potential issues during the construction phase include:

- Elevated risk of barrier breaching due to littoral drift interruption by coffer dam;
- Interruption of backshore drainage during pipe laying across Waihao Arm;
- Elevated risk of wave or tsunami overtopping at the trench location; and
- A large wave and storm-surge event or tsunami occurring during construction.

6.2.1 Littoral drift interruption

The sheet-pile coffer dam will provide a barrier to littoral drift along the foreshore, much like a groyne. This will cause a “fillet” of beach sediment (mainly fine gravel and coarse sand) to build-up on the updrift side, while the foreshore immediately downdrift will erode due to the interruption of its supply of beach material (Figure 6-2). Since the prevailing drift direction is northward (northward drift is 87% of gross drift), the build-up is expected on the southern side while erosion is expected on the northern side. However, the opposite situation is possible if a phase of waves from the north-easterly quarter were to occur.

We estimated the seaward and landward offsets of the shoreline updrift and downdrift of the coffer dam (X, m) associated with a given volume of accretion and erosion (V, m^3) using the equation:

$$X = (2V\tan\alpha_b/H)^{0.5}$$

where α_b is the drift-weighted average wave breaker angle (13.7 degrees, based on our longshore transport analysis of Section 4.7.2) and H is the shore profile height (set at 10 m, being the height from the foreshore toe, at 6 m below MSL, to the average height of storm berms on the foreshore at the Waimate Creek profile, at 4 m above MSL). The assumption is that the shore either side of the coffer dam will equilibrate at an orientation that renders a zero breaker angle and longshore transport potential. Using this equation with our estimates of monthly longshore transport rates (Figure 4-15), the shoreline offset X after one month is 18 m for a 50%-exceedance average summer drift rate of 7000 m^3/month, 24 m for a 5%-exceedance average summer drift rate of 12,000 m^3/month, and 30 m for a 5%-exceedance average winter drift rate of 18,000 m^3/month. The corresponding lengths of affected shore (L in Figure 6-2) range from 76 to 122 m either side. In the currently narrow, volume-depleted state of the barrier at the pipeline location (e.g., Figure 4-1), erosion offsets of these magnitudes on the downdrift side of the coffer dam could consume the ridge crest and significantly increase the risk of overwash and breaching.

This effect and risk could be contained by regular mechanical bypassing of the trapped littoral drift, either with an excavator or a pumping system, and with detailed monitoring of the foreshore topography adjacent to the coffer dam to ensure a significant downdrift erosion “bite” does not develop. This is detailed further in Section 6.4. Also, beach material excavated from the trench could be stockpiled on the northern, downdrift side to augment the bypassing.

This effect will stop as soon as the coffer dam is removed, which will restore the longshore connection of littoral drift. We also expect that foreshore processes will quickly (over a period of days to weeks, depending on wave conditions) smooth out any profile differences between the updrift and downdrift sides of the coffer dam position.

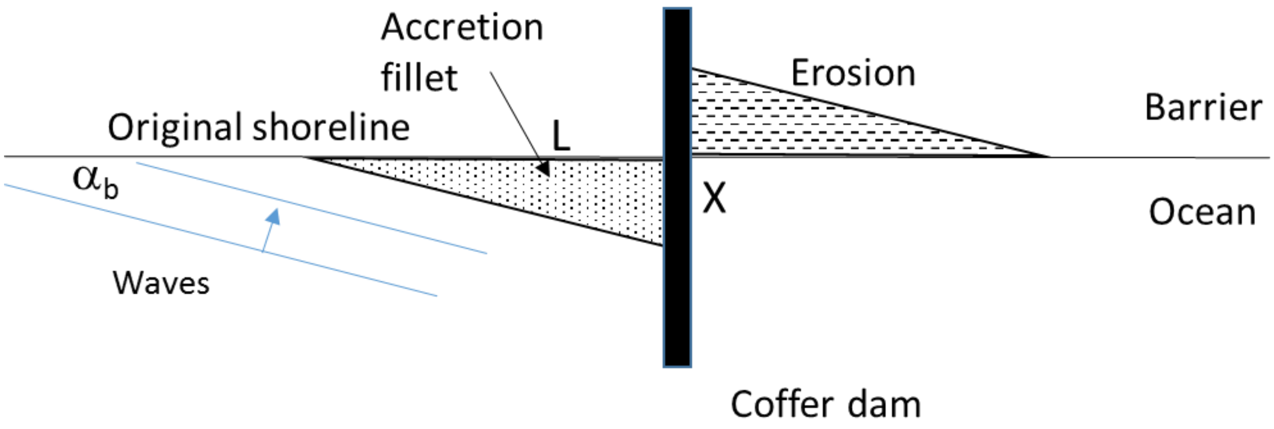


Figure 6-2: Schematic planform showing updrift accretion and downdrift erosion due to coffer dam.

6.2.2 Interruption of backshore drainage

This should not occur, since the old channel of the Waihao Arm will be kept open while the new channel is temporarily bunded for pipe-laying and vice-versa. The bunds on both old and new channels will be removed after pipe-laying. The old channel will likely need to be excavated from its present state to ensure it provides adequate conveyance while it is the sole channel.

6.2.3 Greater risk of wave or tsunami overtopping at trench

Potentially, the trench could be a low, weak point in the barrier through which storm waves or tsunami could surge and penetrate into the backshore. This will be avoided by the construction plan, which will see the trench side-walls and end-walls completed before any excavation occurs. The walls will rise to a height of 6 m, which is higher than the current barrier crest by 0.4 m.

6.2.4 Large coastal storm or tsunami occurs during construction

Potentially, a coastal storm could occur during the construction phase that would be large enough to overwash, possibly breach, the barrier even without any construction activity. Based on the above, it is not expected that the construction would exacerbate any backshore flooding that would have occurred anyway, but there could be consequences for the construction operations. Assuming a 10-year recurrence interval for such events occurring somewhere along the Waihao-Wainono shore (Section 3.6.3), the likelihood of this occurring in any 6 month construction phase at the project site is less than 5%. While this risk is low, it would nonetheless be appropriate for Fonterra to have a contingency plan for managing such an event during the construction phase. This plan could simply involve removing the workforce and machinery from exposed positions on the barrier if an event was forecast, and repairing any structural damage as need be after the event to ensure no lingering weak-point in the barrier.

Similarly with an overtopping tsunami, it is estimated (Section 3.9) that these have a return period of 60-300 years, indicating a likelihood of occurrence in any 6 month construction period of 0.17-0.83%. Again, while this risk is very low, it would be prudent to include this in the contingency planning for the construction period.

6.3 Potential post-construction and longer-term issues

After pipeline installation under the barrier has been completed and the trench refilled with the beach material stockpiled during its excavation, the barrier will have been restored to its natural profile and alignment and the pipeline is not expected to have any significant impact on foreshore processes. Indeed, this has been confirmed by behaviour of the shore at Clandeboye, where no morphological effects of the buried pipeline have been observed.

The one potential short-term post-construction issue relates to the refilling of the trench, which potentially could leave a more porous and permeable 3-4 m wide 'slice' of barrier that is more prone to breaching during wave overtopping events. The risk of this occurring would be minimised by refilling the trench with beach-grade sediment (as is planned), and by compacting this in layers that parallel the foreshore – which aims to reproduce the natural stratification (e.g. as shown in Figure 3-9) and encourage percolating swash to flow seaward rather than landward. Using the stockpiled excavated material should ensure that the same size sediment is returned to the barrier and this will also be roughly in the sequence that it was removed. It is considered unlikely that the excavation will encounter a clay base under the barrier (e.g. see sections 3.4.2 and 4.1) – if any is encountered at all it is likely to be a clay-bound gravel. However, if additional backfill beach material is required, it could be taken from one of the nearby backshore washover aprons – such as to the immediate north of Poingdestres Road. The total volume of excavation/refill material is estimated at 1600 m^3 (i.e. a 3.5 m wide barrier slice 460 m^2 in area – as per Figure 6-1).

Another minor effect is that the 1 m high pipeline on the nearshore seabed may collect a drift of sand on its southern side. This should have no significant effect on the foreshore, however, because on mixed-sand-and-gravel shores (e.g., Figure 3-5), the coarse, beach-grade material is held against the foreshore by the asymmetry of the approaching waves (which have more intense shoreward orbital velocities than seaward velocities). Thus, there is minimal sediment exchange between the nearshore and foreshore and the longshore transport of beach material is confined to the foreshore.

Based on these points, even if future climate change may increase the risk of backshore inundation through more frequent barrier overwashing or breaching events, we take the view that the pipeline, once built and buried, will have no effect on this increased risk.

The potentially significant longer-term issues relate to coastal retreat exacerbated by rising sea level. This could:

- expose the pipeline on the foreshore; and/or
- create a conflict with future repositioning of the Waihao Arm channel.

A potential scenario is that the barrier rolls far enough landward that the pipeline is exposed on the foreshore. In such a scenario, the pipeline may potentially begin functioning as a groyne, promoting local erosion downdrift, and it may become structurally weak by having its foundation removed. In Section 4.8.5 we estimate barrier retreat over the 50 year design-life of the project (i.e., 2015 to 2065), including the effects of SLR, at somewhere between 41 and 61 m with MfE's (2008) lower guidance figure for SLR and 43-94 m for MfE's higher guidance figure. As shown in Figure 6-3, with these the pipeline would be exposed progressively further up the lower foreshore but not above the low-tide step. In such a situation, the pipe will flex (by design) and drape on the lower foreshore, thus the structural concern is not an issue. Moreover, since the bulk of the longshore transport on mixed sand and gravel beaches is driven by swash in the intertidal zone, we do not anticipate any significant effect of such an exposed pipe on littoral drift continuity.

Another scenario is that the retreat is large enough to force ECan to relocate the new Waihao Arm channel further landward to avoid it infilling with backshore gravel. This infilling would start to occur when the barrier backshore toe has migrated 55 m from its present location (Figure 6-3) – which lies within the range of estimates of retreat for both of the SLR scenarios. A shallow-buried pipeline would compromise this Waihao Arm relocation. Options include burying the pipeline deeper to landward now or accepting a condition that the pipeline would be re-laid deeper, and the surge chamber relocated westward, should the Waihao Arm channel need relocating within the design life of the project. With the former option, the location of the surge chamber and the pipeline plunge-point would need to be at offset 170 m on Figure 6-3 to leave enough space for a relocated channel on the landward side of the barrier with the upper bound SLR and barrier retreat scenario. With the latter option, it would be important to design the pipeline to facilitate a future relaying of the affected segment, including relocation of the surge chamber.

We conclude that by burying the pipeline to the level of the foreshore toe as far back landward as proposed, the direct effects of coastal retreat exacerbated by sea level rise on the pipeline will be avoided over its design life. However, the presence of the pipeline may compromise the ability of ECan to relocate the Waihao Arm channel further landward – unless the pipeline can be re-laid deeper as need be.

This would become an important issue should future consideration be given to extending the design life of the project beyond 2065.

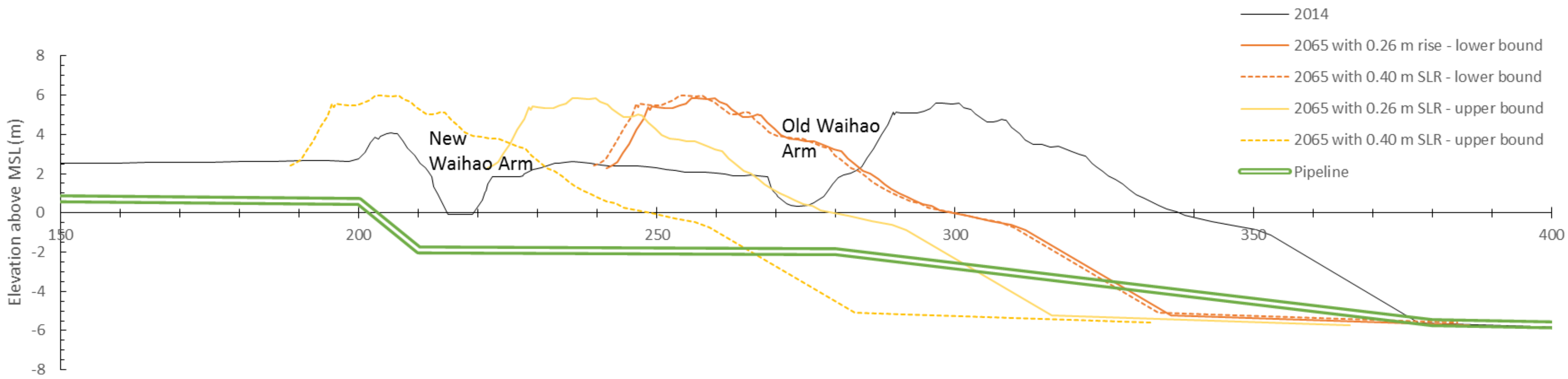


Figure 6-3: Pipeline overlaid on shore profile at Waimate Creek, as surveyed in 2014 and translated landward with different scenarios of barrier retreat.

6.4 Mitigation and monitoring

The coastal hazard issues associated with the pipeline crossing the shore, as discussed above, are summarised in Table 6-1, along with requirements for mitigation and monitoring. We note that some hazards are already avoided in the current proposal design.

The significant outstanding hazard during the construction phase relates to beach erosion on the northern side of the coffer dam due to its groyne effect. Left unmitigated, this will likely be enough to locally lower and narrow the barrier to the point where the risk of overwash or breaching is substantially increased. Thus it is recommended that beach sediment be bypassed across the coffer dam while it is in place. The expected bypassing rates required are in the range 7,000 – 12,000 m^3/yr during summer months (233 – 400 m^3/day), and higher in the winter. This could be undertaken with an excavator on the coffer dam or by pumping a fluidized slurry of beach sediment.

A partial alternative could be to shorten the coffer dam, terminating it at or just below the step that separates the upper and lower foreshore (at offset 350 m on Figure 6-1). This would allow some bypassing of littoral drift around the dam, in association with storm-related cross-shore exchanges of sediment between the upper foreshore and the step (offshore during storms, shoreward in the post-storm recovery phase), and by swash action once the updrift shore had built out to the end of the dam. From Figure 6-1, the step (in October 2014) was some 15 m seaward of the MSL contour. A 15 m seaward shift of the updrift shore is likely to produce a 15 m landward shift of the downdrift shore, which would still begin to compromise the downdrift beach crest height. Thus, this would only be a partial mitigation, and would still require a facility for artificial bypassing, albeit at rates less than discussed above. A consequence of a shorter coffer dam, however, would be that the pipeline would have to emerge on the shoreface just below the step, from where it would drape down the lower shoreface. This would expose it to increased agitation and wear by breaking waves and abrasion by mobile gravel, particularly during storm wave conditions.

Monitoring of the foreshore topography should be undertaken to ensure that an erosion 'bite' does not develop downdrift. This should extend at least 200 m south and north of the coffer dam and should at least survey the positions of index contours on the foreshore – we suggest the 1 m and 3 m above MSL contours using an RTK-GPS system. Alternatively, full 3d topography could be collected with a terrestrial laser scanner mounted on the coffer dam. Given the potential for rapid retreat with inadequate bypassing, we recommend these surveys be undertaken weekly. Trends of progressing downdrift erosion should be responded to with accelerated bypassing. Surveying updrift will inform on the littoral accumulation rates.

Another possible effect during the construction period is for swash to erode a narrow runnel (i.e. groove) against the sheet piling during storm wave events, which may create a small weak point for overtopping. This should be checked after storm events occurring during the construction period and any near-crest runnels should be filled.

The risk of post-construction barrier-breaching through the refilled trench will be minimised by refilling the trench with beach-grade sediment and compacting this in layers that parallel the foreshore (to promote seaward percolation of swash). The barrier backshore at the pipeline should be monitored for visual evidence of seepage associated with wave overtopping events for the first five years (which should be long enough to experience several overtopping events and for any seepage issues to become apparent). In the unlikely event of seepage being observed through the refilled trench material, then permeability could be reduced by grouting.

Table 6-1: Coastal hazards anticipated during construction phase and long term, their risk in current proposal, recommended mitigation and monitoring.

Hazard	Period	Risk	Mitigation	Monitoring
Backshore flooding by storm wave overwash and breaching	Construction phase	Low	Management plan involving moving workforce and machinery to safety during events, repairing any damage to barrier and works after events	None
Backshore flooding by tsunami	Construction phase	Very low	Management plan as above	None
Backshore flooding through trench	Construction phase	Avoided	Trench walled all round to 6 m above MSL	Inspection of beach level beside sheet-piling after storms
Impaired backshore drainage by blocking old Waihao Arm channel	Construction phase	Nil	ECan's new Waihao Arm channel designed to replace old channel	None
Downdrift erosion and backshore flooding by groyne effect	Construction phase	High	Regular beach sediment bypassing while coffer dam in place; possibly shortening coffer dam	Weekly survey of 1 and 3 m above MSL contours (or full upper foreshore topography) within 200 m of coffer dam
Barrier breaching at re-filled trench	Post-construction	Low	Refill trench with stockpiled beach sediment from excavation; fill and compact parallel to foreshore; grout if signs of seepage	Monitor backshore at pipe location for signs of seepage after overtopping run-up events
Pipeline exposed on foreshore by barrier retreat, affecting pipeline integrity littoral drift	Long term	Low	Pipe designed to be flexible	None
Pipeline compromises landward relocation of Waihao Arm channel	Long term	Low but possible	Bury pipe deeper to landward now, or have a condition to bury deeper as need arises	Annual monitoring of shore profile above pipeline and height and position of adjacent shore features (e.g., crest, backshore toe)

In the potential long-term case of the barrier retreating into the new Waihao Arm channel, the mitigation options are to either bury the pipeline deeper to landward now or to re-lay it deeper should the need arise. This requires that the position of the backshore toe of the barrier is monitored regularly. Continuing to monitor ECan's Waimate Creek profile annually should serve this purpose (as would simple visual monitoring). It would be useful also to, at the same time, survey the position and height of key barrier features (beach crest, backshore toe) within ~ 500 m of the pipeline to detect any potential instances where the pipeline may be outflanked by more rapid barrier retreat. This could be done simply with RTK GPS in conjunction with the profile survey, perhaps under an arrangement with ECan.

7 Conclusions

1. The proposed wastewater outfall pipeline from the Studholme dairy factory expansion would pass under the mixed-sand-and-gravel barrier approximately opposite Waimate Creek at the end of Meyer Road. Historical surveys of beach profiles and the position of shoreline features at this location show that the barrier is currently approximately 5.6 m high, has a trend for long-term retreat ranging from 0.19 m/yr to 0.73 m/yr (depending on the period of observation and the type of analysis), and has a volume of around 220 m^3/m shore length that varies by around ± 50 m^3/m largely due to losses and gains in foreshore volume associated with longshore transport. While the barrier height at the outfall site has decreased historically, this has occurred in approximately decadal jumps with long stable periods between; moreover the barrier is higher and has had a relatively stable height within several hundred metres to the north and south.

2. ECan coastal hazard mapping defines two Coastal Hazard Zones extending some 30 and 60 m landward of the barrier backshore toe position. These largely reflect the threat of ongoing barrier retreat at historically-observed rates 50 and 100 years into the future from 2005. Since the pipeline crosses these zones, the installation will be a discretionary activity requiring consent.

3. Analysis of historical sea-flooding events in the Waihao-Wainono Lowlands area shows that barrier overwash or breaching occurs about every 10 years on average but is usually confined to limited spans of barrier during any given event. Factors rendering the barrier more vulnerable to overwashing and breaching are a relatively low height, low volume, a deep buried substrate, and a waterway adjacent to the backshore toe. The last substantial breaches of the barrier occurred in 2001 and 2002, when the backshore toe at Waimate Creek moved approximately 12 m landward.

4. An analysis of extreme wave run-up height against the barrier that combined a 20 year record of sea level with a wave hindcast and refraction analysis confirmed that barrier overtopping/overwashing events at the outfall site should re-occur about every 10 years on average. The likelihood of an overtopping event occurring during a 6 month construction period is approximately 5%.

5. Sediment budget estimates combined with calculations of longshore transport rates based on the 20-year wave hindcast record indicate a dominantly northward littoral drift averaging some 120,000 m^3/yr. Longshore transport rates show a seasonal pattern, with higher rates on average in the winter (May-October). 5% exceedance monthly transport rates are 12,000 m^3/month over November-April and 18,000 m^3/month over May-October.

6. The estimated return period of barrier overtopping by tsunami run-up at the outfall site ranges between 60 and 270 years. The shorter return period is a conservative, “maximum potential” estimate that has been adopted for evacuation planning, and is based on the 84th percentile of the range of tsunami heights estimated with various assumptions and also assumes no attenuation of tsunami energy during the run-up process. The longer return period provides a more likely expectation. Using that, the

likelihood of an overtopping tsunami is 17% during the 50 year project life and 0.2% over the construction phase.

7. Future climate change could affect the project shore through several drivers, including sea-level rise (SLR), altered wave and storm surge climate, and altered sediment budget. All of these can interact to influence barrier processes but definitive guidance on how these factors might change in the future has only been provided for SLR. Thus MfE's (2008) recommended future SLR scenarios were used (after adjustment to the present) to assess potential SLR on barrier stability and retreat at the project site through its 50-year design life, but it is noted that there is considerable uncertainty on the net effects of all facets of future climate change, none more so than possible changes in wave climate and associated effects on the local coastal sediment budget. The SLR scenarios assessed through the 50 year project design life, between 2015 and 2065, included a base rise of 0.26 m and a higher rise of 0.40 m (at average rates of 5.2 and 8 mm/yr, respectively).

8. Assuming the barrier at the project site to be adequately nourished with beach sediment, the expected response to SLR is an overtopping/overwash process that would both build the height of the barrier and roll it landward as sediment is transferred from the foreshore to the backshore. This produces only a few metres of retreat, which, when combined with the largest estimate of historical retreat rate, provides a total barrier retreat to 2065 of 41 m and 43 m for the two SLR scenarios, respectively. Alternatively, assuming that the barrier is in a state of sediment deficit, the calculated retreat by 2065 is 61 m for the lower scenario and 94 m for the higher scenario. In this case, since the observed historical retreat rates are of the order of those calculated from the observed historical SLR, it is assumed that these retreat figures capture the factors driving the historical retreat.

9. It is recommended that the project plan for the higher estimate of retreat associated with the lower SLR scenario (i.e., 61 m by 2065), but also have a contingency for a greater retreat (94 m by 2065) to cover the case of the higher SLR scenario or an unfavourable change in wave climate. This has implications for the location of the surge chamber (which should be located at least 94 m back from the present barrier backshore-toe position) and the depth profile of the buried pipeline.

10. Inspection of the shore at the site of the Clandeboye outfall, and of ECan's beach profile surveys there, showed no morphological signature of the Clandeboye pipeline. This lends confidence to predictions that the pipeline at Studholme will similarly have no morphological signature.

11. The main issue relating to the installation phase of the outfall pipe at Waimate Creek is that while the coffer dam is in place it has the potential to intercept significant quantities of littoral drift on its southern side, which will cause near-field foreshore erosion on its northern side. The likely interception would be enough to render the downdrift barrier vulnerable to overtopping or breaching by common storm wave events. It will be necessary to mitigate this by artificial bypassing of foreshore sediment. Shortening the coffer dam would likely reduce the amount of artificial bypassing required, but may increase wear and fatigue on the pipeline if it is left exposed on the low-tide step or lower foreshore.

12. During the construction phase, the foreshore topography 200 m either side of the pipeline and coffer dam should be surveyed weekly to ensure that the artificial littoral drift bypassing contains any downdrift foreshore erosion. Any trend of progressing downdrift erosion should be countered with accelerated bypassing. Surveying updrift will inform on the littoral accumulation rates. In addition, the beach levels immediately against the coffer dam should be checked after storm events, and any low areas near the ridge crest should be filled with beach gravel.

13. Once installed and buried under the beach ridge, with the temporary coffer dam removed and the barrier reinstated to its current height and composition, the pipeline will have no significant effects on shore processes. Even with a more extreme SLR scenario, the pipeline would only become exposed on the lower foreshore towards the end of its 50-year design life, and then would simply flex over the seabed.

14. The potential for the refilled trench through the barrier becoming a weak point, vulnerable to breaching, will be mitigated by refilling the trench with beach-grade sediment stockpiled from the excavation stage. It is recommended that this fill is laid and compacted in layers that parallel the foreshore to reproduce the natural stratification and permeability characteristics of the barrier.

15. A potential long-term issue is that upper bound estimates of barrier retreat associated with SLR might compel ECan to shift the Waihao Arm channel landward within a 50-year time frame. If this situation arose, it would be necessary to re-install a segment of pipeline deep enough in the backshore that the re-located Waihao Arm channel could cross over it. It would also be necessary to locate the surge chamber at least 115 m back from the present barrier backshore-toe to provide adequate space for a relocated channel. Thus the pipeline design should allow for this contingency, especially if there is the possibility that the design life may be extended in the future.

16. The only long-term monitoring recommended is to monitor the height and position of the barrier at the pipeline and along the adjacent barrier (within 500 m either side). This will confirm expectations of barrier retreat rates and warn of impending infilling of the Waihao Arm channel. It would be achieved by continuing to monitor ECan’s Waimate Creek profile annually, surveying longshore variability in barrier crest height and backshore toe position at the same time, and maintaining a visual watch after coastal storms.

8 Acknowledgements

Bruce Gabites (ECan) kindly made available the data from beach profile monitoring for the span of shore of interest. ECan agreed to the use of LiDAR data co-funded by ECan and Meridian Energy Ltd. Meridian also agreed to use of LiDAR data and results from previous studies of the area of interest. ECan drainage engineers (Leigh Griffiths, Bruce Scarlett) kindly described their issues and plans relating to coastal drainage in the Waihao-Wainono area. Jo Stapleton provided a copy of her Master's thesis. Marianne Gadsby (ECan) and Emily Lane (NIWA) provided information on the tsunami hazard. Richard Gorman (NIWA) provided the wave hindcast modelling records, Benjamin Robinson (NIWA) provided the Lyttelton sea-level records, and Emily Lane (NIWA) provided the synthetic historical sea-level records.

9 References

Benn, J.L. (1988): Coastal hazard maps: Waitaki River to Rakaia River. *SCCB Publication No. 56.*

CERC (1984): Shore Protection Manual. US Army Corps of Engineers, Coastal Engineering Research Centre, Vicksburg, Mississippi.

Cope, J.; Young, B. (2001): Report on South Canterbury coastal flooding event, 19-22 July 2001. *ECan Technical Report* No. U01/101, 37pp.

Davies, J.L. (1972): *Geographical variation in Coastal Development.* Edinburgh, Oliver and Boyd, 240pp.

Dawe, I. (2006): Longshore sediment transport on a mixed sand and gravel lakeshore. PhD thesis in geography, University of Canterbury.

Dickson, M.E., Bristow, C.S., Hicks, D.M., Jol, H., Stapleton, J., Todd, D. (2009): Beach volume and an eroding sand-gravel coast determined using ground penetrating radar. *Journal of Coastal Research*. 25 (5): 1149-1159.

ECan (2005): Canterbury Regional Coastal Environment Plan. Environment Canterbury, Christchurch, 2005.

Forbes, D.L., Orford, J.D., Carter, R.W.G., Shaw, J. and Jennings, S.C. (1995): Morphodynamic evolution, self-organisation, and instability of coarse-clastic barriers on paraglacial coasts. *Marine Geology*. 126:63-85.

Fraser, S.A. (2014): An ArcGIS/Python tool for mapping indicative tsunami evacuation zones defined by a geospatially-calculated attenuation rule. *GNS Science Report* 2014.

Gabites, B. (2012): A summary of Environment Canterbury's coastal Environment monitoring programme for the coastline south of Timaru – 1977 to 2011. Report No. R12/116.

Gibb, J. (1978): Rates of coastal erosion and accretion in New Zealand. *NZ Journal of Marine & Freshwater Research* 12: 429-456.

Gibb, J., Adams, J. (1982): A Sediment Budget for the East Coast between Oamaru and Banks Peninsula, South Island, New Zealand. *New Zealand Journal of Geology and Geophysics*. 25:335-352.

Goring, D. (2004): *Extreme Sea Levels in South Canterbury*, Unpublished Report for Meridian Energy Limited.

Gorman, R., Hicks, M., & Walsh, J. (2002): Canterbury open-coast wave refraction and longshore transport study. NIWA Client Report ENC02509 prepared for Environment Canterbury, June 2002.

Hesp, P.A. and Short, A.D. (1999): Barrier morphodynamics. In A.D Short (Ed.), *Beach and Shoreface Dynamics*, (pp.307-333). Chichester: Wiley.

Hewson, P.A. (1977): Coastal erosion and beach dynamics in South Canterbury-North Otago. Unpublished Thesis (M.A., Geography) University of Canterbury, Christchurch, New Zealand.

Hicks, D.M. (2006): North Bank Tunnel Concept – Water Consents: River Geomorphology, Sediment Transport, Coastal Processes and Flood Hazard Management. *Report for Meridian Energy Ltd.* NIWA Client Report CHC2006-090.

Hicks, D.M. and Todd, D. (2003): Project Aqua: Coastal and river mouth effects-Supplementary Report. *Report for Meridian Energy Ltd.* NIWA Client Report CHC2003-022.

Hicks, D.M. and Todd, D. (2005): Historical changes in shoreline position and beach characteristics on the Waitaki coast. *Report for Meridian Energy Ltd.* NIWA Client Report, 2005.

Hicks, D.M., Single, M., Hall, R.J. (2006): Geomorphic character, controls, processes and history of the Waitaki Coast – a primer. *Report for Meridian Energy Ltd.* NIWA Client Report CHC2006-130.

Hicks, D.M, Wild, M, Todd, D. (2002): *Project Aqua: Coastal and river mouth effects*. *Report for Meridian Energy Ltd.* NIWA Client Report CHC01/112.

Hicks, D.M. (2005): September 2004 LiDAR survey of Waitaki to Makikihi coast. NIWA Client Report, CHC2005-072, prepared for Environment Canterbury and Meridian Energy Ltd, July 2005.

Hicks, D.M., Bind, J. (2013a): May 2013 LiDAR survey of Oamaru to Makikihi coast. NIWA Client Report, CHC2013-100, prepared for Meridian Energy Ltd, September 2013.

Hicks, D.M., Bind, J. (2013b): Changes in shoreline features along the Oamaru to Makikihi coast from LiDAR surveys in 2004 and 2013. NIWA Client Report, CHC2013-125, prepared for Meridian Energy Ltd, November 2013.

Hughes, S.A. (2005): Estimating irregular wave run-up on rough impermeable slopes. US Army Corps of Engineers, ERDC/CHL CHETN-III-70, July 2005.

Kajiura, K., (1983): Some Statistics Related to Observed Tsunami Heights along the Coast of Japan. In *Tsunamis – Their Science and Engineering*, edited by K. Iida and T. Iwasaki, 131–45. Tokyo: Terra. Sci. Publ.

Kajiura, K. (1986): Height Distribution of the Tsunami Generated by the Nihonkai-Chubu (Japan Sea Central Region) Earthquake. *Science of Tsunami Hazards* 4 (1): 3–14.

Kirk, R.M. (1975): Aspects of surf and run-up processes on mixed sand and gravel beaches. *Geografiska Annuler* 57A: 117-133.

Kirk, R.M. (1980): Mixed sand and gravel beaches: morphology processes and sediments. Processes in Physical Geography. 4(2):189-210.

Kirk, R.M., (1987): Coastal Erosion in South Canterbury-North Otago-an overview. A report to the South Canterbury Catchment Board and Regional Water Board. *SCCB Publication No. 52.*

Lane, E., Kohout, A., Chiaverini, A., Arnold, J. (2014): Updated inundation modelling in Canterbury from a South American tsunami. *Report for Environment Canterbury*. Report No. R14/78.

McLean, R.F. (1970): Variations in Grain-size sorting on two Kaikoura Beaches. *Journal of Marine and Freshwater Research* 4(2): 141-164.

Ministry for the Environment (2008): Coastal Hazards and Climate Change. A Guidance Manual for Local Government in New Zealand. 2nd edition. Revised by Ramsay, D, and Bell, R. (NIWA). Prepared for Ministry for the Environment. viii+127 p.

Neale, D.M (1987): *Longshore Sediment Transport in a Mixed Sand and Gravel Foreshore, South Canterbury*. Unpublished Thesis (M.Sc. Geography), University of Canterbury, Christchurch, N.Z. 243pp.

New Zealand Coastal Policy Statement (2010). Department of Conservation, Wellington.

Orford, J.D., Carter, R.W.G., McKenna, J. et al. (1995): The relationship between mesoscale sea-level rise and the retreat rate of swash-aligned gravel-dominated coastal barriers. Marine Geology 124: 177-86.

Orford, J.D., Carter, R.W.G., Jennings, S.C. (1996): Control domains and morphological phases in gravel-dominated coastal barriers of Nova Scotia. *Journal of Coastal Research* 12: 589-605.

Power, W.L. (2013a): Tsunami hazard curves and deaggregation plots for 20 km coastal sections, derived from the 2013 National Tsunami Hazard Model. GNS Science Consultancy Report 2013/59.

Power, W.L. (2013b): Review of Tsunami Hazard in New Zealand (2013 Update), GNS Science Consultancy Report 2013/131.

Power, W.L., Downes, G.L., Stirling, M. (2007): Estimation of Tsunami Hazard in New Zealand due to South American Earthquakes. *Pure and Applied Geophysics* 164 (2-3): 547–64.

Schuttrumpf, H., van der Meer, J., Kortenhaus, A., Bruce, T., Franco, L. (2010): Wave run-up and wave overtopping at armoured rubble slopes and mounds. Chapter 15 in Handbook of coastal and ocean engineering, Young C. Kim (Ed.): 383-409.

Shand, T.D., Reinen-Hamill, R., Shand, R.D., Viljoen, W. (2007): Empirical modelling of run-up and overtopping at a boulder beach, Raglan, New Zealand. Proceedings of the 18th Australasian Ports and Coasts Conference, Melbourne, Australia

Shand, T., Shand, R., Reinen-Hamill, R., Carley, J., Cox, R. (2013): A review of shoreline response models to changes in sea level. Proceedings, 2013 Coasts and Ports Australasian Conference, 11-13 September, Sydney, Australia.

Single, M.B. (1992): *High Energy Coastal Processes on Mixed Sand and Gravel Beaches*. Unpublished Thesis (Ph.D Geography), University of Canterbury, Christchurch, New Zealand.

Single, M., Hemmingsen, M. (2000): Mixed sand and gravel barrier beaches of South Canterbury, New Zealand, *in* Packham J.R. et al*., Ecology and geomorphology of coastal shingle*. Otley, Westbury: 261-276.

Sneddon, R., Dunmore, R., Barter, P., Clement, D., Melville, D., Kelly, D., Elvines, D. (2015): Ecological investigations and effects of the proposed Fonterra Studholme treated wastewater outfall, 2015. Cawthron Report No. 2666. Prepared for Fonterra Cooperative Group Ltd.

Sorenson, R.M. (2006): *Basic coastal engineering*, 3rd Edition. Springer.

Stockton, H.F., Holman, R.A., Howd, P.A., Sallenger, A.H. Jr. (2006): Empirical parameterization of set-up, swash and run-up. *Coastal Engineering* 53: 573-588.

Stapleton, J. (2005): Form and function of the Waihao-Wainono Barrier, South Canterbury. MSc thesis, Department of Geography, University of Canterbury, Christchurch, New Zealand.

Tierney, B.W. (1977): Coastal changes around the Port of Timaru, New Zealand. *NZ Geographer* 33: 80-83.

Todd, D.J. (1988): Annotated Coastal Bibliography of South Canterbury. *SCCB Publication No. 57.*

Todd, D.J. (2001): Tsunami and storm surge assessment for Timaru District Council Lifelines project. *DTec Report for Environment Canterbury.* ECan Pub No. U01/5.

Todd, D.J. (2003): Coastal processes and hazards assessment: Clandeboye, north of the Orari River, South Canterbury. Report for IFBS Ltd.

Appendix A Longshore transport calculation

Longshore sediment transport at the project site was calculated using the hindcast wave data on the 10.46 m isobath with the approach of Gorman et al (2002). This used a set of relationships from CERC (1984).

For a single-component sinusoidal wave of height H travelling with group velocity C_g, the energy transport (i.e. the energy crossing unit length of wave front per unit time) is a vector quantity directed normal to the wave crest, with magnitude

$$F = \frac{1}{8} \rho g C_g H^2 \tag{1}$$

where ρ ~ 1025 kg/m^3 is the density of seawater and g = 9.81 m/s^2 is the gravitational acceleration.

For shallow water of depth h, the group velocity has magnitude

$$C_g \approx \sqrt{gh} \tag{2}$$

and the wave height on breaking is proportional to the depth, i.e.

$$H_b \approx \gamma h_b, \tag{3}$$

with a constant of proportionality γ= 0.5. Hence the (conserved) magnitude of the energy transport at both the 10.46 m isobath and at the breakpoint is

$$F_b = F \approx \frac{\rho g^{1.5}}{8\gamma^{0.5}} H_b^{2.5}. \tag{4}$$

The SWAN modelling procedure provides an estimate of wave parameters on the 10 m isobath, but does not have sufficient resolution to directly compute wave transformation to the breaker zone. However, an estimate of wave height and direction at the breakpoint can be made by assuming that the seabed has simple topography shoreward of the 10.46 isobath, with shore-parallel depth contours. Then conservation of energy flux F can be used to find H_b and h_b through Equations (3) and (4), while conservation of the longshore component of wavenumber gives

$$\frac{\sin \alpha_b}{\sin \alpha} = \frac{k}{k_b} \approx \sqrt{\frac{h_b}{h}} \tag{5}$$

where α and α_b are the angles between the wave propagation direction and the shore-normal direction at the 10.46 m isobath and at the breakpoint, respectively, and h = 10 m and h_b are the corresponding depths. Equivalently, the wave crests are at an angle α_b to the beach.

The energy transport at the breakpoint has longshore and onshore components $F\sin\alpha_b$ and $F\cos\alpha_b$ respectively. The flux of *longshore directed* wave energy across a *shore-parallel* line is then given by the longshore flux factor

$$P_{ls} = F \cos \alpha_b \sin \alpha_b = \tfrac{1}{2} F \sin 2\alpha_b \approx \frac{\rho g^{1.5}}{16\gamma^{0.5}} H_b^{2.5} \sin 2\alpha_b \tag{6}$$

while the flux of *onshore-directed* wave energy across a *shore-parallel* line is

$$P_{os} = F \cos^2 \alpha_b \approx \frac{\rho g^{1.5}}{8\gamma^{0.5}} H_b^{2.5} \cos^2 \alpha_b. \tag{7}$$

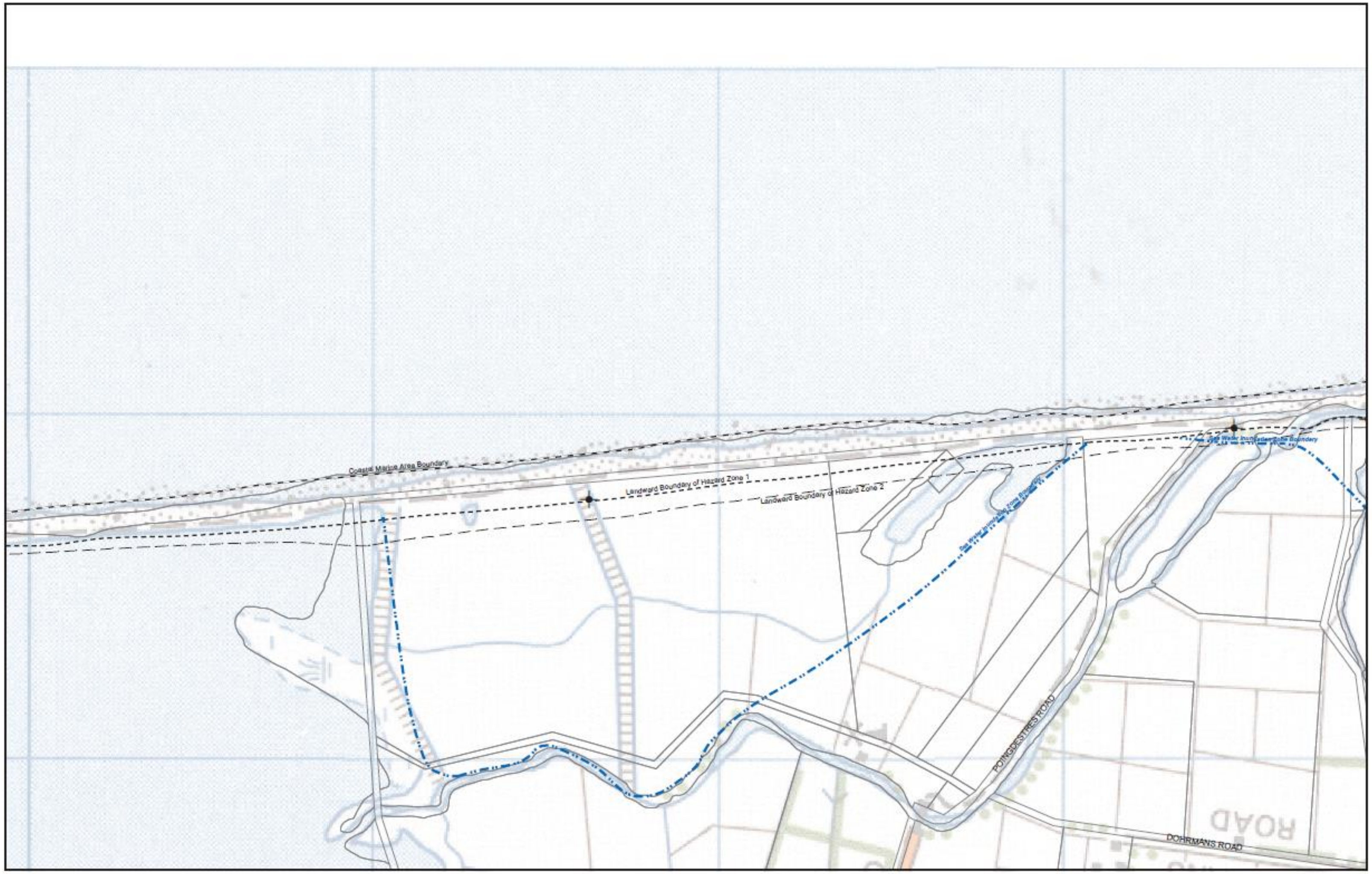
The longshore sediment flux can be approximated by an empirical relationship

$$Q = fKP_{ls} \tag{8}$$

where f = 3308 $m^2.s^3/kg/yr$ = 1.05×10^{-4} $m^2.s^2/kg$ is an empirical constant, while K is a dimensionless efficiency factor dependent on sediment properties.

For this investigation, K was assigned a value of 0.0224 so that the long-term average net northward transport aligned with that estimated for this location from sediment budgeting considerations (~ 120,000 m^3/yr, as detailed in section 3.3). The shoreline orientation was set at 4 degrees West of North. This was the average shoreline trend over a 5 km span of shore centred on the outfall site.

Appendix B Coastal Hazard Zone Maps



Coastal Hazard Zone Map: Waimate 7

- Erosion Determination Points
- Sea Water Inundation Zone Boundary
- Coastal Marine Area Boundary**
- Landward Boundary of Hazard Zone 1
- Landward Boundary of Hazard Zone 2
- Territorial Authority Landward Boundary

**The Coastal Marine Area Boundary (CMA) along the line of Mean High Water Springs (MHWS) is indicative only.

Cadastral information sourced from Land Information New Zealand data. CROWN COPYRIGHT RESERVED.

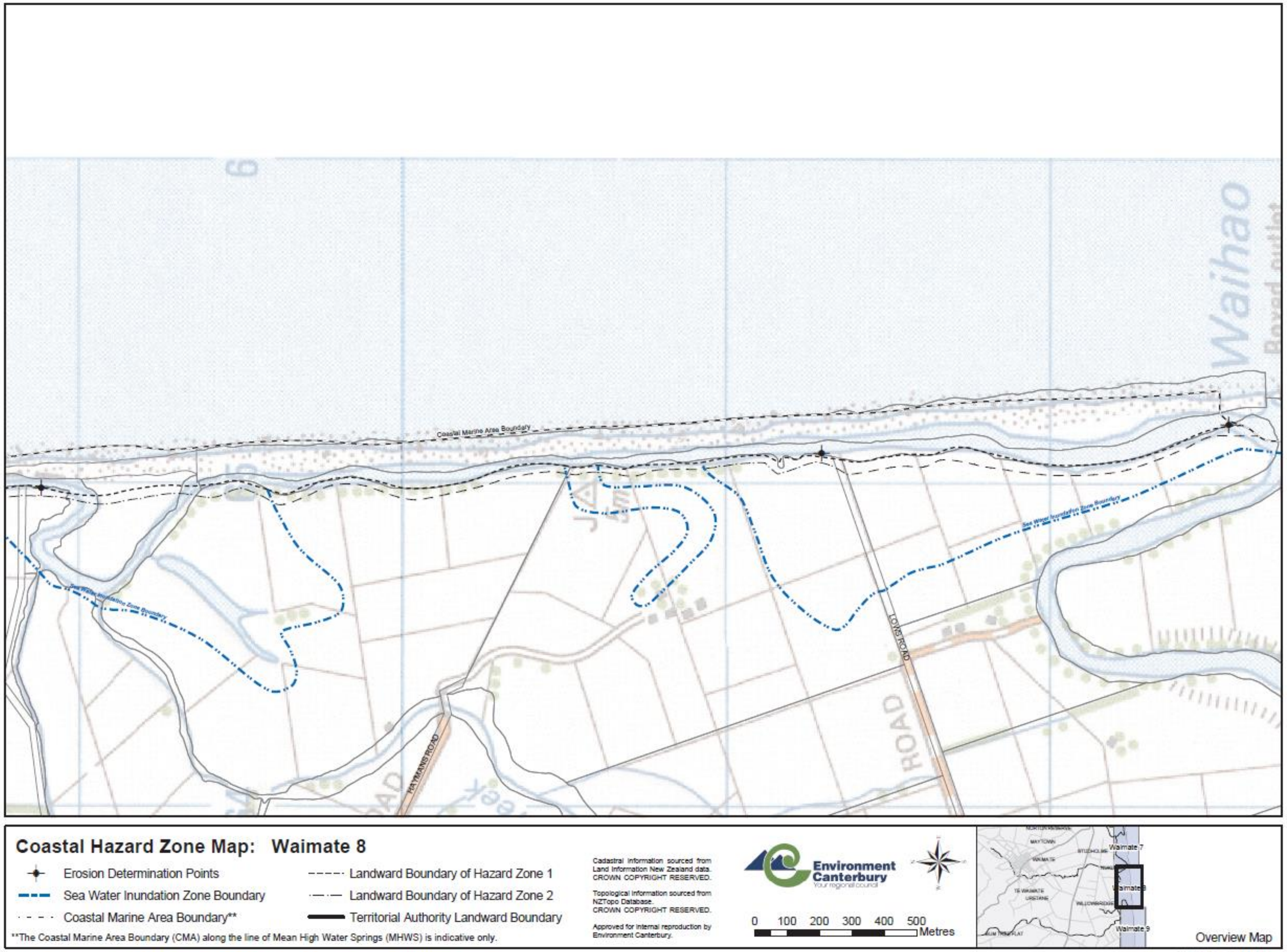
Topological information sourced from NZTopo Database. CROWN COPYRIGHT RESERVED.

Approved for internal reproduction by Environment Canterbury.

Environment Canterbury
Your regional council

0 100 200 300 400 500 Metres

Overview Map


Waihao
Coastal Marine Area Boundary
Sea Water Inundation Zone Boundary
LOWS ROAD
ROAD
HAYMANS ROAD
Coastal Hazard Zone Map: Waimate 8
Erosion Determination Points
Sea Water Inundation Zone Boundary
Coastal Marine Area Boundary**
Landward Boundary of Hazard Zone 1
Landward Boundary of Hazard Zone 2
Territorial Authority Landward Boundary
**The Coastal Marine Area Boundary (CMA) along the line of Mean High Water Springs (MHWS) is indicative only.
Cadastral information sourced from Land Information New Zealand data. CROWN COPYRIGHT RESERVED.
Topological information sourced from NZTopo Database. CROWN COPYRIGHT RESERVED.
Approved for internal reproduction by Environment Canterbury.
Environment Canterbury
Your regional council
0 100 200 300 400 500
Metres
Waimate 7
Waimate 8
Waimate 9
Overview Map