Molyneux Bay and Clutha Delta Morphology Investigation

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IS328700-NM-RPT-0001-1 | 1

July 09, 2021

Otago Regional Council

Project Name

Project No: IS328700
Document Title: Molyneux Bay and Clutha Delta Morphology Investigation
Document No.: IS328700-NM-RPT-0001-1
Revision: 1
Document Status: Final
Date: July 09, 2021
Client Name: Otago Regional Council
Client No: Client Reference
Project Manager: Kate Murray
Author: Kate MacDonald
File Name: IS328700-NM-RPT-0001-1 Molyneux Bay and Clutha Delta Morphology Investigation

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Revision	Date	Description	Author	Reviewed	Approved
0	15/12/2020	Report for External Peer Review	KM	DT	CR
1	09/07/2021	External Review Comments Addressed	KM	DT	CR

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Executive Summary

In May 2020, Otago Regional Council (ORC) commissioned Jacobs to undertake a coastal morphology and climate change investigation of the Molyneux Bay – Clutha Delta area. The purpose of this investigation is to help guide decision making regarding management of this coastline and delta area, including the mouth training wall structures, floodbanks and coastal drainage channels of the Lower Clutha Flood Protection and Drainage Scheme (LCFPDS). The investigation included an assessment of coastal erosion over the next 30 to 50 years with sea level rise and climate change, and for coastal inundation 0.2m sea level increments from 0.2m to 1.0m plus an extreme scenario of 1.35m rise. The impact that these hazards on LCFPDS infrastructure was also assessed.

Coastal Erosion

For the coastal erosion hazard assessment, a ‘probabilistic’ approach to manage uncertainty in the data inputs was used to determine the Potential Future Shoreline Positions (PFSP), calculated from the extrapolation of contemporary erosion trends and the estimated effect of sea-level rise (SLR) over 30 and 50 year timeframes.

A major component of the erosion hazard assessment was understanding trends and patterns of shoreline movements, particularly the effect of river mouth location, migration and training structures on these patterns. The conclusion from this analysis, covering shoreline change since 1946, included:

- Over the total period from 1946 to 2020 there is a trend of greatest erosion in southern Molyneux Bay at average rates in the order of -4m/yr reducing in a northward direction through the centre of the bay between the river mouths, which reverts to long-term accretion to the north of the Matau Mouth. This trend is constant with net northerly sediment transport and potentially greater supply from the Matau Branch than the Koau Branch.

- For southern Molyneux Bay (e.g. south of the Koau Mouth), the analysis revealed that erosion rates have increased significantly since the construction of Koau Mouth training walls in the early 1980’s. Prior to this time the shoreline fluctuated between erosion and accretion, with an apparent relationship between the mouth position and whether this section of beach was in a state erosion or accretion – for example erosion in north mouth migration 1962-1972, and accretion in south mouth migration 1972-1982. In contrast, for the periods covered by aerial images following the construction of the training wall (1997-2020), trends of shoreline movement over the whole southern cell have been consistently erosional with average net retreat rates in the order of -10m/yr at the southern end of the beach reducing to -6 to -7.5m/yr closer to the Koau Mouth.

- The mechanism of this post training wall acceleration in erosion over the whole southern cell appears to be a negative feedback loop of increased beach rollover lowering dune crest elevations, which in turns allows more frequent rollover and therefore further increased erosion. This is a common shoreline response for beaches lacking sediment supply that are rolling back onto wetlands, lagoons or low hinterlands. This reduced sediment supply is considered to be due to a combination of the following possible causes:

    1. A lagged response to the reduced supply from the Clutha River since the construction of the Roxburgh Dam (1957).
    2. Lack of significant floods in the Clutha River in the 20 years from November 1999 to December 2019.
    3. The presence of southern training wall preventing southern migration of the river mouth, therefore removing any significant periodic sediment inputs to the southern beaches.

4. The alignment of the training walls directing the ebb tide jet further offshore and into the more northerly sediment transport pathway.

- In the central Molyneux Bay cell (e.g. between the river mouths), the rates of retreat over the total 70+ year period are lower (range -1.57 north of the Koau Mouth to -0.3m/yr south of the Matau Mouth), however have gone through periods of erosion and accretion both pre and post the construction of the Koau Mouth training walls. Although mouth migrations of both mouths prior to the construction of the mouth trainings structures have influenced shoreline movements in the adjacent sub-cells during some time intervals, erosion rates have generally been similar across both pre and post mouth training construction periods.

- For northern Molyneux Bay, the historical shoreline trend for this has been in the range +0.6 to +0.8m/yr, which has generally been consistent over the total period 70+ year period. It is therefore considered that the shoreline movements along this section of coast are not influenced by river mouth orientation or migration, or any of the mouth training structures.

Due to the significant change in erosion rates south of the Koau Mouth in the post training wall period, which the presence of the southern wall is considered to have a substantial influence on, the extrapolation of historical shoreline movements for this section of Molyneux Bay was undertaken applying the following scenarios:

- For the 30 years to 2050, the wall is assumed to remain in place, and the extrapolation is at the higher 1997-2020 post wall rates.

- For the 20 years between 2050 and 2070, one scenario of continued extrapolation at the higher 1997-2020 rates under the assumption that the wall continues to be maintained and in its current position, and a second scenario of the wall being removed, and erosion assumed to slow and return to the 1946-1982 pre-wall rates.

For the central and northern sections of Molyneux Bay, the erosion rate over the total 1946 to 2020 period was used in the extrapolation.

Projected Future Shoreline Positions

For the calculation of Projected Future Shoreline Positions (PFSP), over the 30- and 50-year timeframes, the erosional impact of sea level rise was added to the extrapolation of contemporary erosion rates. The impact of sea level rise on future erosion was calculated using a modified Bruun rate for overtopping on sand beaches for rise of 0.2m by 2050 and both 0.4m and 0.6m by 2070. Short term storm events were estimated from SBEACH-32 numerical modelling simulating beach responses to extreme storm events (50, 100, 200-year ARI), however these were not included in the total PFSP distances. The resulting estimates of the total erosion distances to PFSP over each time period and the potential effects on the infrastructure of the LCFPDS are as follows:

By 2050

- Southern Molyneux Bay: Greatest shoreline retreat at the southern limit of the bay, with the 'most likely' erosion of -320m from the present-day position, and a 5% chance of an additional 150m retreat from this position. The predicted erosion distances decrease in a northward direction to in the order of -220m for the section immediately south of the Koau Mouth, with a 5% chance that erosion could be 120m greater. The erosion results indicate that 85-90% of the predicted retreat will be due to the extrapolation of contemporary erosion rates since the construction of the southern training wall, with sea level rise increasing erosion by around 25m over the 30-year period.

- Central Molyneux Bay: Estimated erosion distances continue to reduce to the north, being on average -65m on the north-20m side of the Koau Mouth, with a 5% chance that erosion distances could be an additional 40m landward of this position. Sea level rise is estimated to contribute 25-30% of the predicted erosion. At the northern end of the cell, estimated erosion distances are most likely to be in the order of -20m, with a 5% chance that the PFSP could be an additional 25m landward this position. Up to 90% of the predicted erosion is due to sea level rise.

- Northern Molyneux Bay: Due to the extrapolation of the contemporary shoreline change, the 30-year PFSP for the area north of the Matau Mouth is 'most likely' to be accretionary, with the shoreline likely to be in a similar or more seaward position than the present day shoreline.

By 2070

- Southern Molyneux Bay: The projected future shoreline position for this cell is heavily influenced by whether the southern training wall is maintained or not, with less sensitivity around the increase in SLR. In both SLR scenarios where the wall is maintained, erosion distances are greatest in the southern end of the cell, in the order of -540 to -570m, and decrease in a northward direction to -380 to -400m for the sub-cell south of the mouth training wall. There is a 5% chance that erosion distances could an additional 245m at the southern end of the cell, and 195m at the northern end of the cell. The results indicate that returning to 1946-1982 erosion rates by removing the southern training wall could reduce erosion distances by the order of 150 to 180m at the southern end of the cell, and by 40m across the middle of the cell. However, the removal of the wall is predicted to increase the erosion distance immediately south of the training wall by approximately 45m.

  The average difference in erosion distances between the two SLR scenarios is in the order of 15-25m, with the effect of SLR contributing 10-15% of the total erosion distance in the 0.4m rise scenario, increasing to 15-20% for the 0.6m rise scenario.

- Central Molyneux Bay: Erosion distances to the PFSP decrease in a northward direction being in the order of 110m to 130m at the south end of the cell (5% chance that could be an addition 67m of erosion), reducing to 40 to 55 m at the northern end (5% chance that could be an additional 40m). The average erosion difference between the two SLR scenarios was estimated to in the order of 15m, with the relative contribution of this effect on the total erosion increasing from 25% (0.4 m SLR) to 40% (0.6m SLR) in the southern sub-cell, to 78% (0.4m SLR) to 92% (0.6m SLR) in the northern sub-cell.

- Northern Molyneux Bay: Most areas of shoreline in this cell continue to accrete with 0.4m of SLR, but overturn to being erosional with 0.6m of SLR. For example, at the southern end of the cell, PFSP distances are +10.8m with 0.4m SLR, and -1m with 0.6m SLR. Due to uncertainty there is a 5% chance that there could be an additional -14m of erosion.

Potential Effects on the Infrastructure of the LCFPDS

By 2050 around 150-170m of the Koau Southern Training Wall structure could be exposed to direct wave energy if the wall remains in its current form. Hence there will be an increase in costs to maintain this structure in its current form. The Puerua Diversion outlet within the wall would also be exposed, the drainage along the diversion channel would be blocked by the beach adjacent to the outlet and compromised further south. A 500m section of floodbanks at the end of Port Molyneux Road is also likely to be compromised by the shoreline retreat over this time period, with a 5% chance that the whole length of banks along the Puerua Diversion are similarly affected by the erosion. Along the central bay between the river mouths, the predicted shoreline positions are located within the current high back dune environment, indicating that the ability of these dunes to provide protection against coastal inundation could be compromised within this time period. At the northern end of the cell the predicted erosion would place the beach very close to the position of the Matau River training wall, indicating that the beach may roll over the top of this structure exposing the front to damage from open coast processes.

Under all 2070 (e.g 50 year) scenarios, the impacts on the LCFPDS infrastructure is the same; the shoreline being landward of the Puerua River Diversion channel and floodbanks over their total length and the southern training predicted to be outflanked from behind with the shoreline predicted to most likely be located on the main river channel landward of the wall. In the central bay, the shoreline is predicted to be landward of the Matau Mouth training line, further exposing the structure to open coast erosion processes. For the very unlikely 5% PFSP, the drainage capacity of the northern end of the Inch Clutha Diversion channel would also be compromised. Along the centre of the cell the PFSP indicates that the current high back dunes are lost to erosion suggesting that the vulnerability to coastal inundation along this section of coast would increase.

Coastal Inundation

For the coastal inundation hazard assessment, a two stage approach was proposed, where stage one used a 'bathtub inundation' approach to determine the likelihood of overtopping of the LCFPDS floodbanks with SLR over 30, 50 and 100 year timeframes, and a stage two involving hydrodynamic modeling to confirm inundation extents and depths where stage one determined that overtopping was likely to occur. This report only presents the results of the stage one assessment and recommendations on the need to move to stage two modelling.

The results of the stage one inundation assessment can be summarised as the following:

- Some small sections along the Port Molyneux Floodbank are below the current day 100-year ARI high water level with wave set up (2.31 m DVD). With SLR, around 70% is below the 100-year ARI with rise of 0.2 m (e.g. by 2050), and almost the entire length (95%) is likely to be overtopped in the same frequency event with + 0.4 m SLR.
- The left bank of the Puerua Deviation Floodbank is above the present day 100-year ARI high water level with wave set-up, while the right bank downstream of Kaka Point Road has two low points that could be overtopped in this magnitude event with current sea levels. Approximately 200m of left bank near the Puerua River Mouth would be overtopped in the 100 year ARI with 0.4m of SLR, while at the mouths of the Puerua River and the Koau Branch the crest levels are below 2.83m and are likely to be overtopped with the 100-year ARI + 0.6m SLR. On the Puerua River, upstream of the Kaka Point Road Bridge, overtopping by the 100-year ARI high water level with 0.2m of SLR is likely to occur on the right bank, and with 0.4m of rise on the left hand bank up to Wix Road.
- On the Inch Clutha Floodbank, crest levels would not be overtopped in present day 100-year ARI high water levels, or with SLR up to 0.4m. With 0.6m of SLR, the floodbank elevation is exceeded by the 100-year ARI high water level at four locations, and with 0.8m of SLR, the floodbank elevation is exceeded for around 70% of its length for this frequency event, increasing to 100% with 1.0m of SLR. However, the dune field east of the floodway includes an extensive width with elevations > 3m, hence the source of any bank overtopping would have to be via the lower river channels and the Inch Clutha Bypass floodway, rather than overtopping the beach.
- The left bank of the Koau Branch Floodbank below where the Inch Clutha Floodbank joins could be overtopped 100-year ARI high water level + 0.6m SLR. Upstream of the Inch Clutha Floodbank both left and right banks could be overtopped up to 800m up the river channel from the river mouth by the 100-year high water level + 0.8m SLR, up to 1000m up the river channel with 1.0m of SLR, and up to 1350-1450m up the river channel with 1.35m of SLR.
- There are low points on the right bank of the Matau Branch which could be overtopped by the the100-year ARI high water level + 0.6m SLR (including river mouth / inlet wave set up). Much of the right bank crest levels are below the 100-year ARI +1.0m SLR high water level. Along much of the left bank there is no

formal floodbank protection, but the higher ground level provides protection from coastal inundation. The left bank at the mouth section has lower parts that will be overtopped by high water levels above the 100-year ARI high water level with 1.0m of SLR. The left bank Summer Hill section will be overtopped by the 100-year ARI high water level + 0.6m SLR.

It was recommended that ORC undertakes stage two hydrodynamic modelling to better understand the extent and wider effects of the flooding across the LCFPDS structures, however this should be delayed until ORC are able to have further consideration around erosion management and the possible consequences for the LCFPDS assets so that these decisions can be accounted for in the next stage of the inundation modelling.

Important note about your report

The sole purpose of this report and the associated services performed by Jacobs is to undertake a coastal inundation and erosion morphology assessment of the Molyneux Bay - Clutha Delta area in accordance with the scope of services set out in the contract between Jacobs and Otago Regional Council ('the Client'). That scope of services, as described in this report, was developed with the Client.

In preparing this report, Jacobs has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by the Client and/or from other sources. Except as otherwise stated in the report, Jacobs has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

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1. Introduction

1.1 Project Background and Scope

In May 2020, Otago Regional Council (ORC) commissioned Jacobs to undertake a coastal morphology and climate change investigation of the Molyneux Bay – Clutha Delta area (Figure 1.1). The purpose of this investigation is to help guide decision making regarding management of this coastline and delta area, including the mouth training wall structures, floodbanks and coastal drainage channels of the Lower Clutha Flood Protection and Drainage Scheme (LCFPDS), which ORC are responsible for. The investigation included an assessment of coastal erosion and coastal inundation hazards over the next 30 to 50 years with sea level rise and climate change, and the impact that these hazards will have on the coastline and the LCFPDS infrastructure over these timeframes. This assessment follows a number of ORC reports over the last few years that have identified a potential issue with sea level rise impacts for the on-going maintenance and efficiency of the LCFPDS infrastructure (e.g. Hornblow, 2106a & 2016b; Bloor, 2017). The purpose of this assessment is to try and quantify, and where possible, qualify the magnitudes and timeframes for these issues.

For the coastal erosion hazard assessment, a 'probabilistic' approach to manage uncertainty in the data inputs was used to determine the Potential Future Shoreline Positions (PFSP), calculated from the extrapolation of contemporary erosion trends and the effect of sea-level rise (SLR) over 30 and 50 year timeframes. This was followed by assessment of the impact of these shoreline positions on the maintenance requirements, efficiency, and potential useful lifetime of the current river mouth training structures and floodbanks of the LCFPDS. The original scope of the project also involved conducting the assessment over a 100 year timeframe, however, in undertaking the erosion assessment it became clear that projected shoreline position beyond 50 years were very uncertain and are going to be totally influenced by decisions around the future of the mouth training walls and Puerua Diversion Drainage Channel. As a result, the scope was altered to determine PFSP under two scenarios for the 50 year timeframe; one with the Koau Mouth Southern Training Wall maintained for the total 50 year period, and the other for this structure being removed after 30 years. As per the project scope, the resulting PFSP's have been mapped in GIS and are included in Appendix A.

For the coastal inundation hazard assessment, a two stage approach was proposed, where stage one used a 'bathtub inundation' approach to determine the likelihood of overtopping of the LCFPDS floodbanks with SLR over 30, 50 and 100 year timeframes, and a stage two involving hydrodynamic modeling to confirm inundation extents and depths where stage one determined that overtopping was likely to occur. This report presents the results to the stage one assessment and recommendations on the need to move to stage two modelling.

In assessing the impacts of coastal erosion and shoreline morphology changes on the infrastructure of the LCFPDS, the assessment has considered at a high level a range of possible mitigation options for dealing with these impacts, and the consequences of these options on the efficiency of the LCFPDS to provide flood control and drainage for the Clutha Delta. The assessment recommends further investigations are required to provide more certainty around the relative merits and consequences of these options.

Finally, the assessment provides recommendations on an ongoing coastal monitoring program for the area to assess the future changes in coastal morphology and shoreline position for the purpose of determining trigger points for decisions on further action on the LCFPDS infrastructure.

Figure 1.1: Outlines of study area extents, with inundation (green) including the lower Clutha Delta, and the erosion assessment (orange) being limited to the coastline between Kaka Point to the north of the Matau River mouth.

1.2 Timeframes and Sea-Level Rise Scenarios

The original project scope was for the assessment to consider sea level rise (SLR) for both erosion and inundation assessments over 30, 50- and 100-year timeframes. However, as stated above, the erosion assessment was limited to 50-year timeframes due to the uncertainty with the decisions around the future of the mouth training walls beyond this timeframe and the influence those decisions will have on the shoreline position. Therefore, the erosion assessment is limited to SLR over a 50-year timeframe, while the stage one coastal inundation assessment considers SLR over 30, 50- and 100-year timeframes.

Within these timeframes, a number of SLR incremental scenarios have been assessed. Table 1.1 shows how these increments compare to the RCP[1] SLR scenarios for New Zealand presented in MfE (2017) *Coastal Hazards and Climate Change Guidance for Local Government*. As can be seen from the Table 1.1, the increments used in the assessment cover the range of SLR magnitudes from the MfE (2017) scenarios. It is noted that MfE (2017) states that all RCP scenarios have the same likelihood of occurrence, therefore all the increments also have the same likelihood of occurrence.

Table 1.1.: SLR projections used in this assessment, compared to projections from MfE (2017)

Time frame and Year	SLR applied in Molyneux Bay Assessment	NZ RCP2.6*M* (Median)	NZ RCP4.5*M* (Median)	NZ RCP8.5*M* (Median)	NZ RCP8.5 *H+* (83rd Percentile of RCP8.5)
30 years (2050)	0.2 m	0.23 m	0.24 m	0.28 m	0.37 m
50 years (2070)	0.4 m	0.32 m	0.36 m	0.45 m	0.61 m
	0.6 m				
100 years (2120)	0.6 m	0.55 m	0.67 m	1.06 m	1.36 m
	0.8 m				
	1.0 m				
	1.35 m				

For the coastal erosion assessment, the above increments of SLR have been discounted by the contemporary rate of rise over the last 50 years (taken as an average of +2 mm/yr) to avoid double accounting of the theoretical effect of contemporary rates that are already included in the extrapolation of the historical coastal erosion. Therefore, the calculation of erosion due to SLR are for the acceleration in rate of rise above the contemporary rate.

Vertical land movement (VLM) from tectonics can be a factor which over a 30, 50- and 100-year timeframe influences the relative level of sea level rise. However, VLM does not appear to be a significant contributor in the Molyneux Bay area. Based on previous assessments (Bishop and Turnball, 1996; Barrell et al, 1998; Litchfield and Lian, 2004), Beavan (2012) reported the Molyneux Bay coastline as being stable, with long term tectonic movement reported to be 0 mm/yr. Therefore, for this assessment a factor of VLM has not been included in the SLR component.

1.3 Report Structure

This report is structured as follows:

- Section 2 outlines the coastal processes occurring in the Molyneux Bay – Clutha Delta area;

[1] RCP: Representative Concentration Pathways, based on global population and carbon emissions. From IPCC (2014) AR5 assessment.

- Section 3 outlines the methodologies employed for the coastal erosion and inundation assessments;
- Section 4 presents the results of the coastal erosion assessment and the impact on LCFPDS structures;
- Section 5 presents the results of the Stage one coastal inundation assessment and the impact on LCFPDS structures; and
- Section 6 summarises the findings of the investigation and provides recommendations on future investigations and ongoing monitoring programs.

2. Molyneux Bay-Clutha Delta Coastal Processes

The coastal processes influencing shoreline behaviour throughout the study area are a combination of waves, tidal hydrodynamics, floods and wind-driven currents around a complex coastline of headlands, reefs and trained entrances. The behaviour of the Clutha River and its two river mouths, the Koau and the Matau have a major influence on the shoreline behaviour. The mouth entrances and tidal deltas at these mouths are shaped by a number of processes, including: ocean waves transporting marine sand onshore, longshore sand transport along the beaches, tidal movement of sand in the lower reaches of the river, and flood events delivering fluvial material to the coastal zone. Therefore, any changes in these river mouth entrances will impact on the incident waves, sediment transport and shoreline position and alignment.

2.1 Geomorphologic Setting

The area of coastline assessed in this investigation is an 11 km stretch of Molyneux Bay orientated in a NE-SW direction from Kaka Point in the south to Summer Hill north of the Matau Mouth of the Clutha River (Figure 1.1). The majority of the coastline is the eastern edge of the coastal plain or delta of the Clutha River, which is the largest in the Otago Region, draining a catchment of over 20,000 km^2, with a mean average discharge at Balclutha of 570 m^3/s and recorded flood flows peaking at over 5000 m^3/s at Balclutha.

The most recent estimate of natural total sediment yield of the Clutha River at Balclutha was by Hicks et al (2000) as being 2.39 Mt/yr, of which 0.99 Mt/yr, equivalent to 550,000 m^3/yr, is likely to be sand and gravel that could theoretical contribute to the coastal sediment budget. However, this yield has been reduced as a result of the construction of hydro-electric power generation dams at Roxburgh and Clyde on the upper catchment over the last 60 years (see Section 2.2.1). Prior to the construction of these dams, the Clutha River was the largest supplier of sand/gravel to the East Otago coastal sediment budget.

As shown in Figure 1.1, the bifurcation of the lower Clutha River into two branches occurs near the Balclutha settlement – the southern Koau Branch and the northern Matau Branch. Although the formation of the Koau Branch is considered to have only occurred about 500 years ago (Acheson, 1968), deposition on the large low lying coastal plain of approximately 100 km^2 is considered to have been occurring in flood events over the last 6,500 years of the Holocene (Smith, 2007). Previous work on the coastal plain morphology suggests that the plain is 15m thick (Carter, 1986) and therefore approximately 1500 million m^3 of sediment has accumulated across this plain over the last 6500 years, of which an estimated 35% is sand, giving an historical average sand accumulation rate of around 80,800 m^3/yr (Smith, 2007). It is noted that this accumulation has not been occurring in contemporary times due to the construction in the 1950's of the stopbanks of the Lower Clutha Flood Protection and Drainage Scheme (LCFPDS) dramatically reducing flood flows across the plain.

The continental shelf off Molyneux Bay averages 30 km wide and has a very gently sloping profile (about 1:200 in general) to an outer edge in the order of 120 to 150 m water depth. Glacial melting some 12,000 years ago produced huge volumes of sand and gravel, the remains of which blanket the outer Otago shelf today. Much of this relict material is coarse shelly gravel. Transgression of the sea across the shelf was episodic but relentless from 18,000 to 6,500 years ago, when present sea level was reached (Gibb, 1986). The inner shelf is covered with a large modern (e.g. Holocene) sand wedge supplied by northward transported sand bypassing Nugget Point and from the Clutha River. This wedge is widest (17 km) and deepest (34 m) in Molyneux Bay and thins to the north (Carter et al, 1985, Carter 1986).

Carter et al. (1985) describes the modern sand along the littoral zone of the East Otago coast as being predominantly quartz feldspar sand of Haast Schist origin supplied by the Clutha River, which is generally very fine to fine sized (0.2-0.3mm) and well to very well sorted, with somewhat finer sediments occurring in the eddy deposits at Molyneux Bay. However, Bloor (2017) noted that sediments in Molyneux Bay are generally coarser, being medium to coarse sand, which is consistent with the Clutha River being the source of sands transported north along the East Otago coast. Future offshore on the upper part of the sand wedge, the Clutha sands are mixed with approximately quantities of Foveaux Strait-Western Province derived heavy minerals transported past Nugget Point by the Southland Current (Carter, 1986).

Early survey maps (e.g. Figure 2.1 from Kettle (1852)) shows a common mouth at the southern limit of Molyneux Bay for both the Koau and the Matau branches, with the lower river flowing parallel with the coastline for approximately 6 km. This mouth opening was large enough to be used as a harbour (Port Molyneux) in the early days of European occupation. However, the largest recorded Clutha flood on record in September 1878 forged a new mouth for the Matau Branch at its contemporary position near Summer Hill and the mouth of the Koau Mouth migrated north to occupy a number of positions between Port Molyneux and its current position approximately 1 km north of the former combined mouth position. Training structures were placed at both river mouths in the 1970's and 1980's to fix the mouth locations and reduce migration to both the north and south (see Section 2.2.2).

Figure 2.1: Historical survey map of Molyneux Bay, highlighting the common mouth of the Koau Branch and the Matau Branch located at Port Molyneux, the southern limit of Molyneux Bay (Kettle, 1852, supplied by ORC).

The shoreline along the Clutha Delta comprises of low vegetated sand dunes. The lowest section of beach elevation occurs near the southern end of delta, being in the order of 4 to 4.5 m DVD1958 with the beach showing evidence of frequent wave overtopping and sediment rollover (Figure 2.2 and 2.3) into the coastal lagoons and wetlands of the former Clutha mouth channel (Figure 2.4, Port Molyneux and the Puerua Diversion). Along the Inch Clutha shoreline between the Koau and Matau River mouths, primary dune crests are higher – in the order of 5-6 m, with a higher secondary dune in the order of 7-10 m (DVD1958) elevation located at the back of the beach environment. The beach dunes to the north of the Matau Mouth are higher again, being in the order of 6-6.5 m (DVD1958), but in contrast are backed by high (e.g. up to 25 m) former marine terrace (Figure 2.5) deposited in the last interglacial period and uplifted to the west of the Akatore Fault (Barrell et al.,1998). This terrace, known as Summer Hill immediately north of the Matau Mouth, extends north for a further 20 km to the mouth of the Tokomairiro River. To the south of the study area, the shoreline comprises of high bedrock sandstone cliffs of the northern Catlins with small pocket beaches. As shown in Figure 1.1, Nugget Point is a major headland along this shoreline, located approximately 8 km south of the study area.

Figure 2.2: Port Molyneux, looking north towards the Koau and Matau River mouths February 2020 (Supplied by ORC).

Figure 2.3: Facing South towards Kaka Point from the Koau Mouth Training Wall (Supplied by ORC, 2016).

Figure 2.4: Coastal wetlands and lagoons of the former Clutha mouth channel and the Puerua River Diversion, Port Molyneux (Supplied by ORC, 2020).

Figure 2.5: Low sand beaches with high vegetated former interglacial terrace behind, Summer Hill north of Matau River Mouth (Supplied by ORC, 2016).

2.2 Anthropogenic Changes to the Catchment and Clutha River Mouth

2.2.1 Catchment Changes

Over the last 60 years, the Clutha River has been dammed at two locations, the Roxburgh Dam (opened in 1957) located 100 km upstream of the Koau and Matau River Mouths, and the Clyde Dam (opened in 1992) located approximately 50 km further upstream. Trapping of sediment behind these dams is estimated by Hicks et al (2000) to have resulted in a 95% decrease in downstream sediment yield. Opus (2000) concluded that there had not been any post-damming increase in supply from downstream of the dams to compensate for these losses, therefore the resulting average annual contribution of sand and gravel to coastal budget post-damming was estimated by Tonkin & Taylor (2000) at around 42,100 m^3/yr. Smith (2007) noted that the earlier estimates of Carter (1986) resulted in a higher post-damming average annual supply of 69,400 m^3/yr due to a higher pre-damming load (594,400 M^3/yr) and a lower damming reduction of 88%.

Tonkin & Taylor (2000) suggest that the estimated pre-damming sand and gravel yields at Balclutha given could have been inflated due to land clearance gold mining activities in the mid to late 1800's, with yields potentially being up to 50% higher than pre-European settlement times. As a result, there would be corresponding higher sediment supply to the coast, but with time lags on delivery. Since bedload sediment transport distances in flood events are not known, it cannot be guaranteed that sand and gravel entrained in these events actually passed into the lower catchment before the construction of the Roxburgh Dam.

In the 1950s the Lower Clutha Flood Protection and Drainage Scheme (LCFPDS) was designed and implemented by the Otago Catchment Board, which is now owned and maintained by the ORC. The scheme consists of 100km of floodbanks, 200km of contour and drainage channels, tide gate structures, pumping stations, and a series of river protection works including the mouth training structures discussed below. The major components of the LCFDS for this study are shown in Figure 2.6.

Figure 2.6: Key components of the LCFPDS for this study.

2.2.2 River Mouth Changes

The modified flow regime as a result of hydro-power generation was also considered to be a factor in more frequent river mouth offsetting and partial blockage by small sand bar formations (Opus, 2000), resulting in impacts on drainage and flood risk. As a result, following physical modelling of the mouth processes by the Otago Catchment Board (OCB), the mouth training works were constructed to alleviate this problem.

At the Koau Mouth (southern mouth), a northern rip-rap armour training wall was constructed in 1973 to restrict the northerly offset of the channel (Bloor, 1973). However, this construction date is inconsistent with the aerial photography record, in which the February 1972 imagery shows this wall being present. Following the construction, there was a significant period of accretion on the northern side of the wall, where a previously offset mouth of the Koau was then infilled, and the mouth of the Koau mitigated south of its present-day position. At the time of the second largest flood on Clutha (October 1978, 4500 m^3/s at Balclutha) the mouth was around 1 km south and was considered a contributing factor in the extensive inundation area that occurred (Hornblow, 2016a). By 1982, the mouth had migrated a further 1 km south. To address this issue, a 350 m southern rip-rap armour training wall was constructed in 1983-1984 (Figure 2.7A) to restrict the mouth opening to its current location and hence reduce the potential for backwater flooding effects. The seaward end of this southern wall included a 50 m “bullnose” section constructed in a NE direction tangentially to the rest of the training wall to prevent southeasterly waves diffracting around the end of the wall back into the river mouth and eroding beach material (Figure 2.7B).

Figure 2.7: South training wall at the Koau River mouth. Photo A (left) - Under construction in May 1983; Photo B (right) – In 2016 from google earth imagery. Note the end of the northern wall can also be seen (Source: Bloor, 2017).

During recent times this southern wall has required ongoing maintenance, largely due to the impact of wave energy displacing armour rock (Figure 2.8) and settlement from the washing out of sand foundation material (GeoSolve, 2017). Between 2009-2014 maintenance was carried out five times, which largely included retrieving displaced rip-rap from the intertidal zone with up to 25 m of the seaward end of "Bullnose" having been lost due to erosion and relocation processes (Bloor, 2017). Although the training walls play a vital role in keeping the Koau River Mouth open and reducing upstream flooding, future maintenance needs to take into account the high rates of coastal erosion being experienced to the south of the wall.

Figure 2.8: Koau River Mouth training wall (north) following a storm when it was badly damaged by the removal of rip rap by high sea levels and large waves, exposing the training line to further damage (Bloor, 2017).

Physical modelling was also undertaken at the Matau Mouth, resulting in a 650 m training line aligned parallel to the shoreline being constructed to the south of the Matau Mouth in 1987 (Figure 2.9) to control any mouth offsets to the south which result in mouth blockage and significantly elevated flood levels upstream. The works were designed to maintain the mouth in a northern position against the uplifted former marine terrace at Summer Hill and included a short (40 m) rock groyne at the base of the raised terrace to restrict northern mouth mitigation along the cliff line (Figure 2.10). From time to time, the southern training line has not been successful, and it has been necessary to artificially relocate the mouth to the desired northern position with earthmoving equipment (e.g. 1992-93, 2017-18) (Johnstone, 1993 & GeoSolve, 2018). As a result, the end of southern training line was extended east in 1993 and 2018 as shown in Figure 2.11.

Figure 2.9: Training line at the south of the Matau Mouth (Jacobs, 2020)

Figure 2.10: Matau River mouth looking south from Summer Hill showing the southern training line and the small northern rock groyne (Supplied by ORC, 2020).

Figure 2.11: Matau River Mouth extensions of the southern training line in 1993 (yellow), and 2018 (red) (Source: GeoSolve, 2018).

2.3 Sediment Transport

Sediment transport along the East Otago Coast is predominantly northward due to combination of the Southland current, and the dominant southerly wave direction. This dominant northward transport is supported by the shape and extent of the nearshore sand wedge, and the spread of quartz feldspathic Clutha sand along beaches and dunes north toward Dunedin. Carter (1986) gives an estimate of the sediment bedload transport (e.g. sand) across the middle continental shelf (e.g. 50-70 m water depth) from south of Nugget Point as being in the order of 222,000 m^3/yr over the Holocene (conversion rate 1.8 m^3/t), of which 50% is shown in his sediment budget as being transported in shallower water on the inner shelf of Molyneux Bay (e.g. < 50 m water depth). However, based on the morphology of the sandstone bedrock shoreline and reefs of the southern Catlins coast, it is considered that this material is not transported close to shore, with the alongshore beach transport from Kaka Point to Molyneux Bay assumed to be zero. It is considered that this inner shelf transport does not supply sediment to the southern section of Molyneux Bay south of the Koau Mouth but does add to the Clutha supply to beaches in the northern section of the bay. Further north, net northward transport on the inner shelf was estimated to be in the order of 611,000 m^3/yr between the Toko and Taieri Rivers (Carter, 1986).

Recent work by eCoast (Atkin et al., 2020a) presents 40 years of offshore wave hindcast from a point approximately 23 km east of Nugget Point showing the wave approach to be dominated by southwest to southerly approach directions (approximately 75-80% of time), which will be refracted by shoaling to approach the shore as south-east to easterly swell. Due to the orientation of the southern bay shoreline, the more southerly swells and waves will result in northward transport. However, around 20-25% of the time there are south-east to north-east wave approach directions, which will result in southerly transport.

Based on the 40 year wave hindcast, recent longshore sediment transport modelling reported by Atkin et al., (2020b) found mean net northward sediment transport potential along the South Otago Coast (Nugget Point (Tokata) to Cape Saunders (Kaimata)) in the order of 34,500 m^3/yr with a maximum annual transport of 248,460 m^3/yr. However, at a number of locations, including around the Koau Mouth, the modelling indicated a reversal of a net northern transport potential, with it being noted that most of these locations co-located with, or were updrift of headlands (e.g. Nugget Point/Kaka Point). The modelled gross and net transport potential for Molyneux Bay from Atkin et al (2020b) is shown in Figure 2.12, demonstrating net transport potential in the order of 50,000 m^3/yr at the southern limit of the bay, reducing towards the Koau Mouth with a small reversal on the north side, before increasing to be in the order of 80,000 to 90,000 m^3/yr around the Matau Mouth. Evidence of periods of southward net transport occurred when there was a southward mouth offset at both the Koau and Matau River Mouths. Since the mouth training walls were built in the 1970's and 1980s, erosion rates have accelerated on the southern side of the Koau Mouth Training Walls, suggesting that before this time, the periodic southern discharge from the river mouth and the counter-eddy played an important role in transporting sediment to the south of the Koau Mouth.

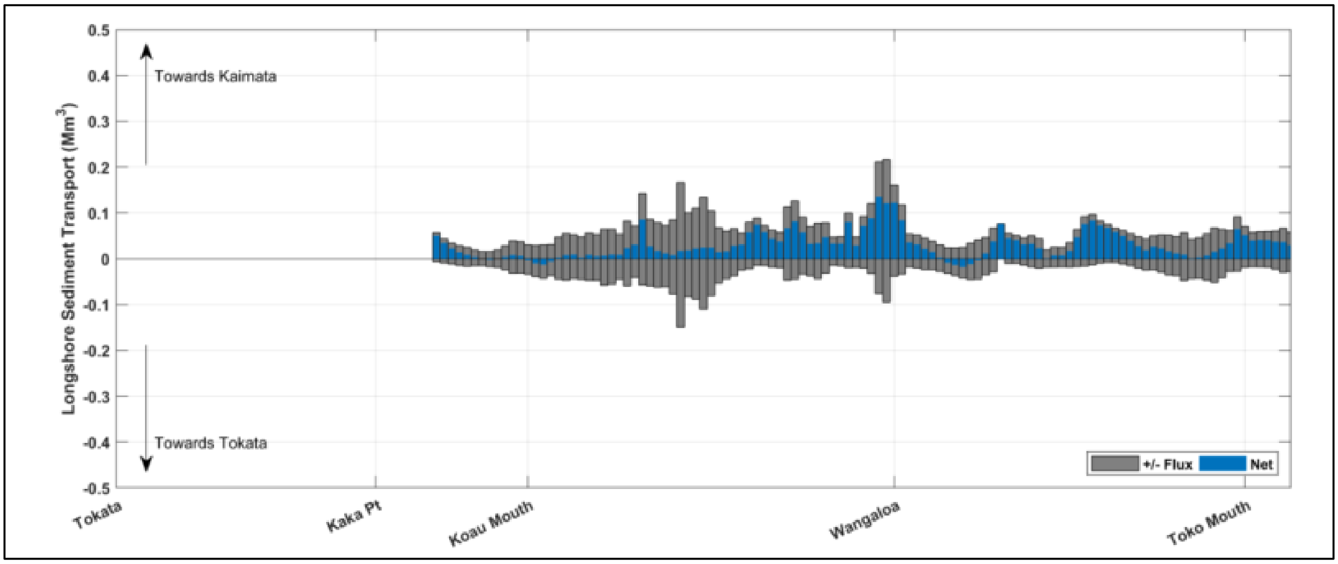


Figure 2.12: Modelled longshore sediment transport potential from waves from Nugget Point (Tokata) to the Toko River Mouth from Atkin et al. (2020b).

A second consideration with sediment transport and supply to the local beaches is "jetting effect" of discharge from the Clutha River to the ebb tide delta and the nearshore wedge. During moderate to small flood conditions, the discharge jet would be relatively weak, resulting in the sediment discharge to be to the ebb tide delta relatively close to shore, and therefore available to be rapidly redistributed back to beaches close to the mouth position. In historical times of a single mouth outlet this supply would have been to beaches at the southern end of the bay, which could of continued periodically with the southern migrated Koau Mouth positions prior to the construction of the southern Koau Training Wall in 1983. The success of this training wall in preventing further episodes of southern mouth migration has limited the cross shore supply from the ebb tide delta to the southern beaches away from mouth. Southern migration of the Matau Mouth would similarly have supplied the local beach between the river mouths from the ebb tide delta of this mouth in these moderate to small flood conditions.

Conversely, during larger flood events the discharge jet deposits the fluvial supply further offshore in deeper water, which is more likely to be transported north away from the local beaches, or possibily is deposited beyond closure depth and therefore lost to the inshore sediment budget.

Sediment is also transported across-shore during coastal storm events, when the low barrier beach ridge to the south of the Koau Mouth appears to be frequently overtopped, resulting in sediment rollover on the backshore, as can be seen in a high sea event in Figure 2.13. Evidence and consequences of this overtopping process at the Koau Mouth can be seen in Figure 2.14, where wave overtopping has pushed sediment through the crib, and buried the crib in the backshore. Once the beach sediment has been moved to this backshore position, it can only be relocated back on the crest berm or foreshore in small volumes due to aeolian processes, or as a result of on-going coastal retreat.

Figure 2.13: Waves overtopping the beach moving sediment into the backshore in a high sea event in 2009, looking SW from the Koau River Mouth (Bloor, 2017).

Figure 2.14: Evidence of wave overtopping south of the Koau Mouth southern training wall (Jacobs, 2020).

2.4 Sediment Budget

There have been multiple past reports on the sediment budget on the wider Molyneux Bay area, especially with concerns of the effects of damming the Clutha River and how this would impact the contribution of fluvial sediment sources reaching the shoreline (Carter 1986, Tonkin & Taylor, 2000; Todd, 2002; Smith, 2007). The sediment budget is a conceptual idea of the sediment sources, sinks and storage within a coastal cell in which sediment moves alongshore, onshore, and offshore. When sources exceed or balance the sinks, the balance is in credit and shoreline could be expected to prograde. Conversely, when sinks exceed the sources, the budget is in deficit and shoreline retreat would be expected to occur. The major sediment sources in the Molyneux Bay area include longshore transport, onshore transport (from the nearshore sand wedge), and fluvial sources. Sediment can be stored on the continental shelf, in the dune systems, and in spits/bar formations. Sediment can be removed from the system by up-catchment damming, dredging/mining, and offshore/alongshore transport.

As indicated above in Section 2.2.1, the Clutha River is a major supplier of sediment for the East Otago coastal sediment budget, with around 39% of the total catchment load being sand and gravel that is theoretically available to coastal budget. Hicks et al (2000) estimated that sand and gravel transport at Balclutha prior to the Roxburgh Dam was in the order of 550,000 m^3/yr (using conversion of 1.8 t/m^3). However, deposition on the Clutha Delta, plus the trapping effects of the Roxburgh Dam reducing downstream sediment yield by an estimated 95%, reduces the post-damming supply under current conditions to an estimated average of 42,100 m^3/yr of sand and gravel (Tonkin & Taylor, 2000). Due to time lags in the river bedload transport system, it is estimated that this sediment input is likely to decrease to around 29,400 m^3/yr over the next 100 years as the full effects of dam trapping (both Roxburgh and Clyde) plus due to channel deposition as a result of sea level rise. Carter (1986) noted that a reduction in supply of this magnitude would affect one or both of the sediment transport rate or the growth of the nearshore sand wedge.

Sediment is stored in Molyneux Bay and further north to the Otago Peninsula (Dunedin) in both onshore beaches, dunes, and spits, as well as in the large nearshore sand wedge on the inner continental shelf. The volume of sand contained in onshore coastal storage was calculated by Carter (1986) to be in the order of 160 million m^3, which equates to an average accumulation of 22,200 m^3/yr over the last 6,500 years of relative sea level stability. Smith (2007) calculated a larger volume of 228 million m^3, at an average annual rate of 35,000 m^3/yr.

As indicated in Section 2.1, the nearshore sand wedge is at widest and thickest in southern Molyneux Bay, due to the contribution of the Clutha River inputs, and thins in a northward direction towards the Taieri River (Figure 2.14). The large volume south of the current Clutha mouths is relict deposits from former river deltas when the combined mouth was located at the southern end of Molyneux Bay. The volume of Holocene sand accumulated in the nearshore wedge was calculated by Carter (1986) to be 7.2×10^9 m^3, which when allowing for compaction, averages out to be in the order of 750,000 m^3/yr over the last 9,600 years of the Holocene, and accounting for nearly half of the total bedload input to the sediment budget. Smith (2007) calculated a larger nearshore storage volume with an equivalent rate of 847,000 m^3/yr. It can also be inferred from Figure 2.14 that around half of wedge volume in southern Molyneux Bay is located in water depths of less than 20 m, therefore was considered by Tonkin & Taylor (2000) as being likely to be less than the depth of closure for transport back towards the beaches by wave processes. However, the recent modelling by Atkin et al (2020) calculated that closure depth in southern Molyneux Bay was a maximum of around 8-9 m water depth, hence return of this sediment to the beaches in this area by wave transport is likely to be much less than previously thought.

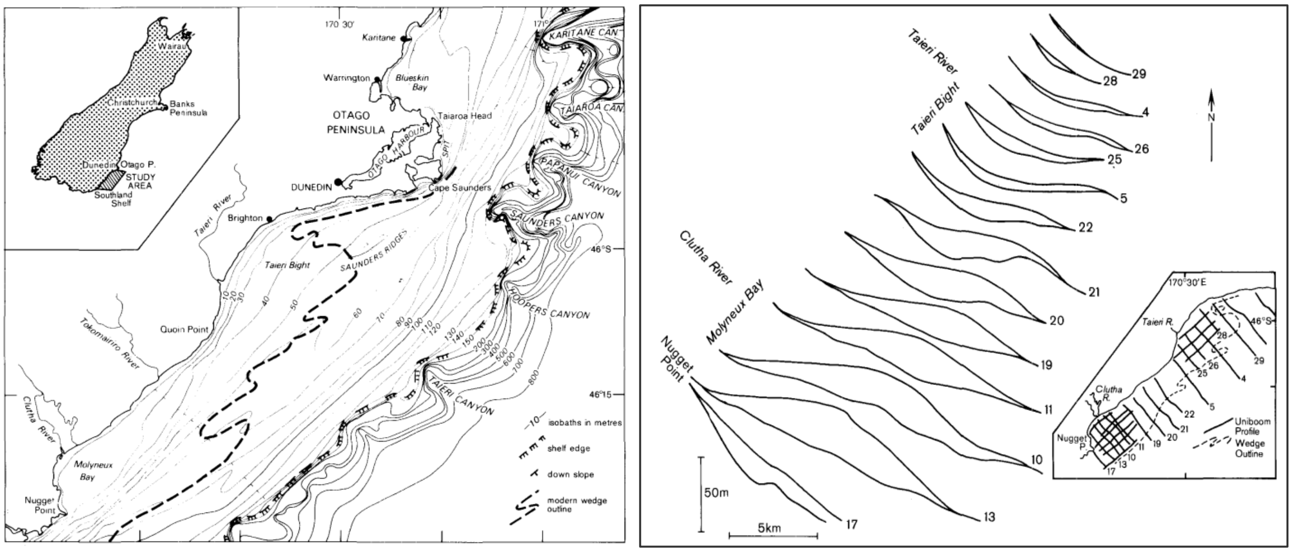


Figure 2.14: Location and extent of the modern sand nearshore wedge (Source from Carter, 1986).

While these onshore and nearshore storage volumes are greater than the Clutha River supply over a historical time period (e.g. last 100 years including damming), they represent accumulation over a much longer timeframe (6,500 to 9,600 years), when fluvial supply rates were likely to be considerably higher, and the nearshore wedge includes an estimated 180,000 m^3/yr longshore supply from south of Nugget Point. However, the majority of this supply is to the mid and outer shelf, with supply to the inner littoral section of the wedge being in the order of 55,500 m^3/yr (Smith 2007). While contemporary Clutha supply to nearshore wedge is much reduced, the storage volume is vast and the coastal processes indicate there is a clear transport path for all sized material from the Clutha River out onto the inner section of the nearshore wedge for storage, with transport back to the beaches by wave-driven currents in suitable conditions. Hence, the supply of modern sand to the regional beaches was considered to be dependent on the wave climate rather than variations in the supply rate (Tonkin & Taylor, 2000; Todd 2002). However, recent local variations in supply appear to have occurred due to the anthropogenic changes at the river mouths discussed in Section 2.2, particular for the shoreline cell south of the Koau Mouth.

2.5 Historical Shoreline Movements

Todd (2002) considered the following three alternative hypotheses for the relationship between the sediment budget and shoreline movements.

1. The normal view of sediment budgets is that all of the Clutha River supply is transported along the beaches, hence any changes in river sediment supply will only be addressed in the sediment budget by changes in the shoreline position. Under this hypothesis, in pre-European times the South Otago beaches would have been advancing at rates of around 1.1 m/yr, the surplus supply as a result of the gold mining activity in the Clutha Catchment would have produced shoreline advances in the order of 2.5 m/yr in the early 1900's, and the current supply deficit since the 1950's construction of the Roxburgh Dam would result in shoreline retreat in the order of 0.4 m/yr. Todd (2002) considered that this is an overly simplistic view that does not recognise the sediment transfers which occur between vast store of sand in the offshore wedge and the beaches.

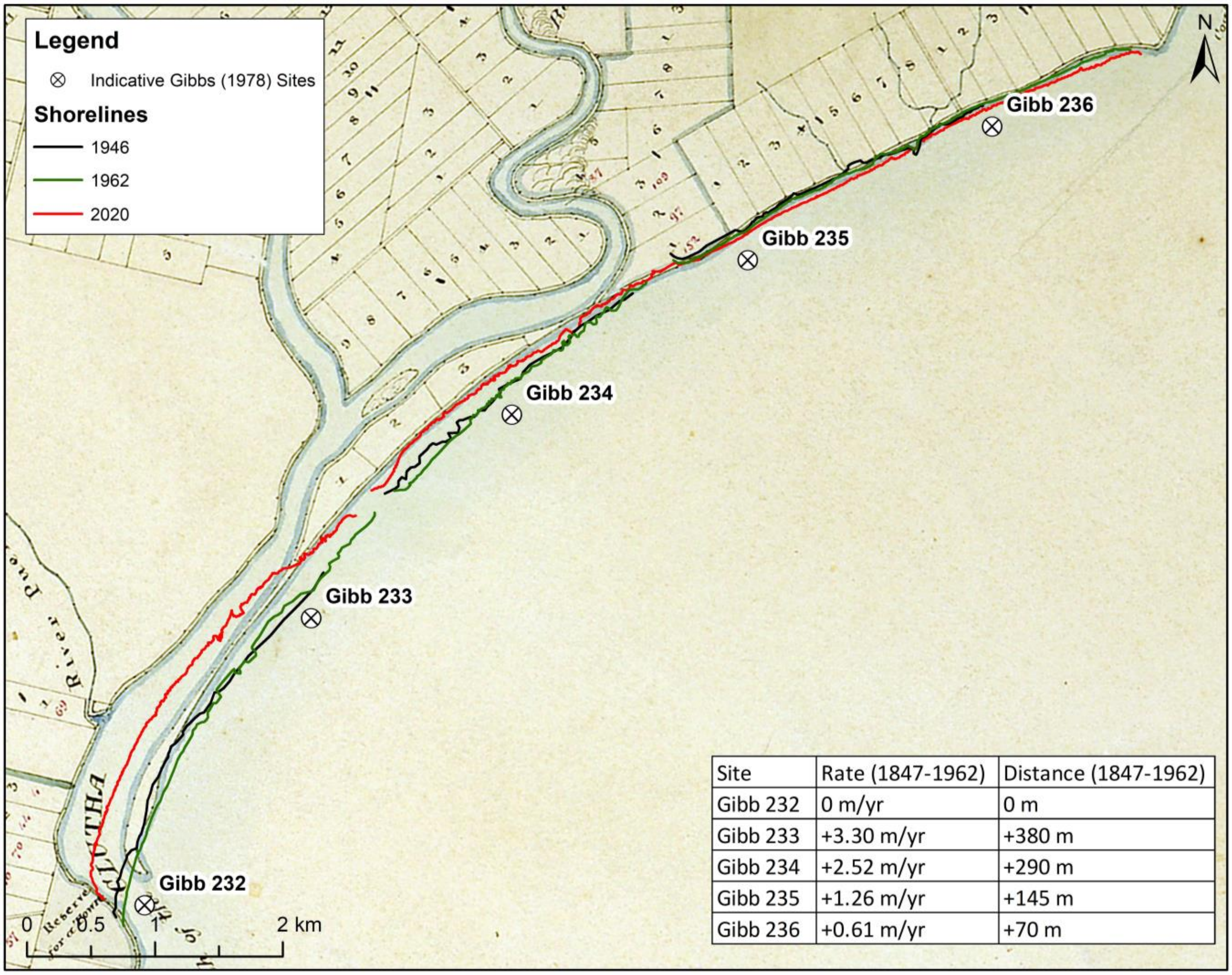
2. Due to there being an adequate nearshore supply of sand from the nearshore wedge to the beach, the changes in shoreline position along the South Otago Coast will be in response to changes in the transport potential rather than changes in the river sediment supply. Therefore, the beaches of South

Otago should go through periods of accretion and erosion in relation to short- and medium-term fluctuations in the transport potential by waves, but overall will be in a state of long-term equilibrium.

3. There is a constant relationship between the amounts of sediment stored in the nearshore and the onshore storage areas that is preserved with changes in supply and is therefore independent of the transport potential. Under this hypothesis, in pre European times the beaches would have been advancing at a rate of less than 0.1 m/yr, the additional supply during the gold mining era would have resulted in an increase in advance to rates of 0.15 m/yr, and the current supply deficit would result in very slow shoreline retreat at mean rates in the order of 0.03 m/yr. This hypothesis can be discounted in that it does not recognise any of the coastal processes operating in supply, transport and transfer of sediment.

The earliest reported analysis of coastal erosion in Molyneux Bay is from Gibb (1978), who included the five sites shown in Figure 2.15 in his nationwide study of contemporary shoreline movements. Except for site 232 at the southern limit of Molyneux Bay, the analysis is limited to a single 115 year time period comparison of shoreline from cadastral map in 1847 to aerial photograph in 1962 (site 232 also having a second period from 1962 to 1977), being a period prior to the effect of the Roxburgh Dam on sediment supply. The results of the analysis were that over the 115 year period (1847-1962) site 232 displayed stability (e.g. nil retreat or advance) and the other four sites all displaying shoreline advance at decreasing rates in a northward direction from +3.30 m/yr (+380 m) immediately south of the Koau Mouth to +0.61 m/yr (+70 m) 2.5 km north of the Matau Mouth. While there are questions regarding the accuracy of the analysis due uncertainty of the shoreline reference used in the 1847 cadastral, the resolution and clarity of the 1962 aerial images, and the accuracy with which the two data-sets could be geo-referenced, Tonkin & Taylor (2000) indicated that this accretion pattern showed the influence of the full Clutha River sediment supply to the local coast. However, the single time period of the analysis has no consideration of the temporal changes over the period associated with the splitting and migration of mouth positions within the time period, which would have influenced the results at the sites of 232 and 233 south of the Koau Mouth and possibly at Site 234 between the Koau and Matau Mouths. Therefore, only sites 235 and 236 north of the Matau Mouth are likely to not be influenced by mouth migration and therefore likely to be reliable indicators of net trends in shoreline movements for this section of Molyneux Bay in the pre-Clutha damming period.

Figure 2.15 shows the shoreline as shown in Kettle (1852), which is assumed to be similar to the 1847 position used in the Gibb (1978) analysis, and some of the shorelines mapped for this study, including the 1962 shoreline also used in the Gibb analysis. The results for sites 233 and 234 show shoreline advance, but at distances in the order of 100-150 m less, while sites 235 & 236 north of the Matau Mouth indicate shoreline stability between 1852 and 1962. This indicates the previously stated limitations around the accuracy of using cadastral shoreline positions for this type of analysis.



Site	Rate (1847-1962)	Distance (1847-1962)
Gibb 232	0 m/yr	0 m
Gibb 233	+3.30 m/yr	+380 m
Gibb 234	+2.52 m/yr	+290 m
Gibb 235	+1.26 m/yr	+145 m
Gibb 236	+0.61 m/yr	+70 m

Figure 2.15: Gibb (1978) shoreline change analysis sites and historical shoreline positions from Kettle (1852), 1946, 1962 and 2020 from this study.

Tonkin & Taylor (2000) analysed the South Otago Coast shoreline movements from the Clutha River using cadastral plans between 1847 and 1886, and aerial photographs to determine changes pre and post damming time periods. However, no sites within Molyneux Bay were included in pre-dam period. For the eleven sites analysed from Wangaloa to Karitane the results did not reveal any overall pattern of temporal changes in shoreline movement, with no widespread trend of increase in coastal erosion or reduction in shoreline advance rates since the construction of the Roxburgh Dam. In fact, the results suggested the reverse, that overall, the South Otago shoreline has been advancing at more rapid rates since the 1950's. Further analysis was undertaken to test whether the lack of relationship between river sediment supply and shoreline movement may be due to the time lags in the supply of material from catchment changes and river flood events to the coast. While the results indicated that erosion on the South Otago coastline became more prevalent in the 1980-1990 period compared to the 1960-1970 period, the role of weather and variations in wave energy in these trends could not be ruled out. From more analysis of shoreline change at three sites north of Molyneux Bay (Toko Mouth, Waldronville and Sandfly Bay), Atkin et al (2020) concluded that there is a relationship between the construction of the dams and a reduction in the progradation rates and/or coastal erosion. It noted that the analysis of the pre-damming period is limited to since 1946, and that Atkin also concluded that the timescales of impacts of the dams on the sediment transport system are unknown.

DTec (2002) undertook an analysis of historical shoreline movements south of the Clutha River to Nugget Point. The conclusion from the analysis was that for the Koau Mouth to Port Molyneux Beach, the nil erosion from 1847 to 1962 reported by Gibb (1978) was likely to mask a number of shoreline fluctuations in response to river mouth location, river flood events and coastal storm events, and that the -80 m retreat recorded between 1962 to 1977 was not representative of the current conditions with the Koau Mouth Training Wall in place. For a further 14 sites along the sandstone bedrock coast north of Nugget Point the analysis indicated that there was no evidence of sediment accumulation prior to construction of the Roxburgh Dam, and that the shoreline had been very stable since 1959.

More recently ORC have undertaken further analysis of shoreline movements in Molyneux Bay between 1946 and 2016 based on aerial photographs over this period (Williams & Goldsmith 2014; Hornblow, 2016a). The results from these studies for the coastal cell from Kaka Point to the Koau Mouth are presented in Figure 2.16, which shows that over the past 70 years this section of shoreline has eroded at an average rate of -4 m/yr. However, as with the previous assessments, the results indicate that the position of the river mouth along the coastline has had a significant influence on spatial variability in shoreline movement recorded in different time periods. In general, prior to the Koau Mouth Southern Training Wall construction in the mid 1980's, the erosion rate was significantly lower due to sediment supply to the south from the occasional southern migration of the Koau Mouth. Since the construction of the training wall, this average rate has increased to -5 to -10 m/yr with rates of -10 to -20 m/yr over the most recent period of 2006 to 2013. As pointed out by Hornblow (2016a), these high erosion rates are likely to have a number of implications for the LCFPDS structures in the near future with accelerated with sea-level rise and increasing wave attack on the structures as the beaches retreat.

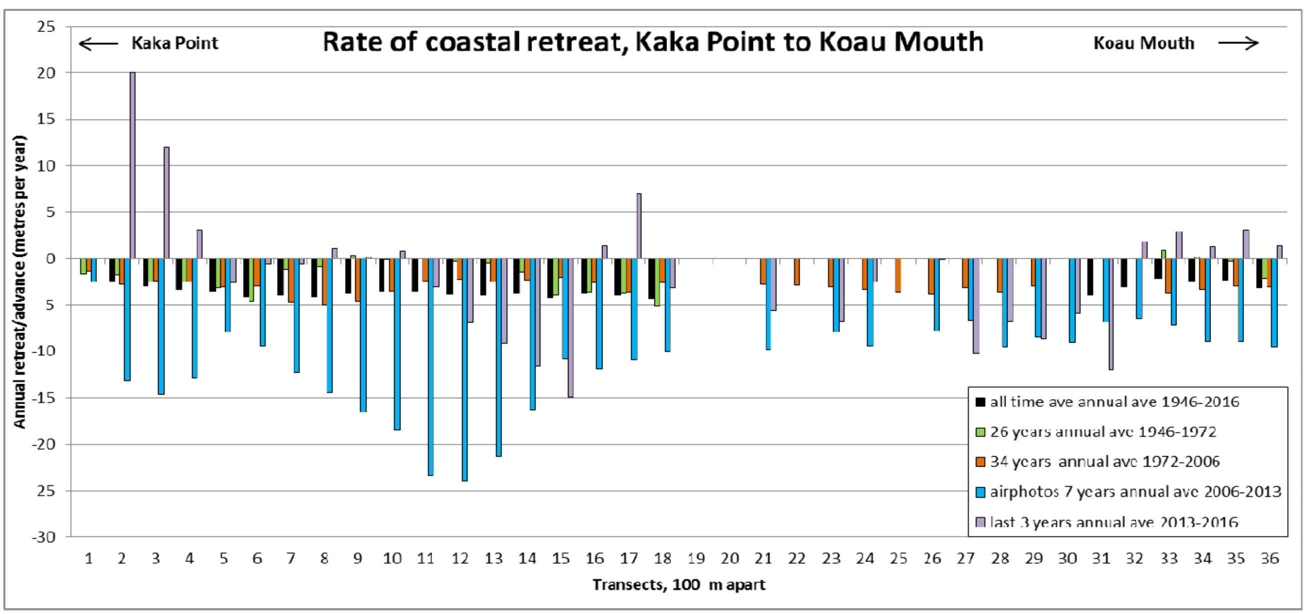


Figure 2.16: Rates of shoreline movements from Kaka Point to the Koau river mouth measured from aerial photographs at 100m transects. (source Hornblow, 2016a).

North of the Koau Mouth, the analysis indicated that the long-term trend (as shown in Figure 2.17) was for lower rates of erosion toward the north and reverting to a stable position approaching the Matau Mouth. However, rates of recent retreat (e.g. 2006 to 2016) have been high for up to 1200m immediately north of the Koau Mouth with erosion distances of up to 70 m.

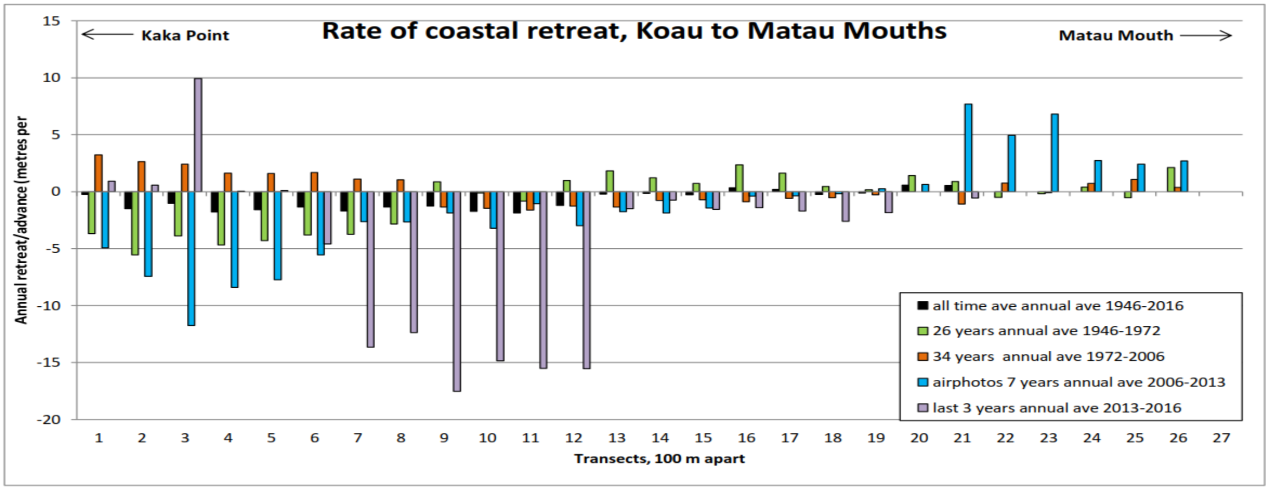


Figure 2.17: Rates of shoreline movements from the Koau River mouth to the Matau River mouth measured from aerial photographs at 100m transects. (source Hornblow, 2016a).

3. Methodology

3.1 Literature Review

Relevant literature from ORC investigations and previous studies on the coastal environment in the Molyneux Bay area were reviewed for information regarding the historical erosion and flooding events, the coastal processes in the area, and the construction of LCFPDS structures near the coast. Key literature reviewed as a part of this investigation is included in the references at the back of the report.

3.2 Data Collation

The following data was collated from various sources to inform the GIS modelling as well as to ground truth the model results. In addition to the imagery used for modelling, ORC also provided GIS files of the positions and elevations of the LCPFDS, including floodbanks, outfall structures, and pump stations.

Data was projected in Dunedin Vertical Datum1958 (DVD58) and a horizontal projection of NZTM2000.

3.2.1 LiDAR Imagery

LiDAR imagery was obtained from ORC and used for both erosion and inundation modelling purposes. The most recent LiDAR data set used in this study was flown by LANDPRO from 16-18 January 2020 for ORC. A Digital Elevation Model (DEM) across the whole study coastline was generated from this data capture and used in this assessment.

Previous LiDAR data along five specified coastal transects (MB1-MB5) from 2004 and 2013 were also used to compare historical profiles along the coastline, however, it is noted that the 2004 data could only be used for one transect (MB5), as the datasets for the other transects only provided data for the backshore environment. The location of these transects is presented in Figure 3.1. These LiDAR transects also align to the bathymetric transects collected in 2013 as described in Section 3.2.3.

It is noted that another LiDAR survey in 2009 has not been used due to issues with the projection and alignment of the data with other LiDAR datasets.

3.2.2 Aerial Imagery

Aerial imagery was collated from ORC, Retrolens and LINZ Online Data Service. The following eight imagery dates were used in the analysis:

- 16-18 January 2020
- 10 February 2014
- 26 March 2006
- January 1997
- 16 April 1982
- 14 February 1972
- 28 February 1962
- 26 February 1946

These imagery dates give a total time period of 74 years for analysis of shoreline change, with time intervals of six to sixteen years between shoreline fixes.

Figure 3.1: Location of ORC profile surveys (MB1-MB5), bathymetric transects, and beach profiles (P1-P11) created from 2020 Lidar used for this study.

3.2.3 Bathymetry Data

Offshore survey data was collected by IXSurvey on 24-25 February 2014 using a multibeam echo sounder to collect a bathymetric survey data for eight offshore transect lines in Molyneux Bay, with seven shown in Figure 3.1 above. The five transects perpendicular to the Molyneux Bay shoreline align with the onshore profiles MB1-MB5, which were surveyed at the same time. Comparisons of the bathymetric profiles across these transect lines are presented in Figure 3.2. Although unfortunately the bathymetric data does not cover the nearshore area from the shore to the -7 m contour, the general seabed slope across this area can be interpolated from the intersection with the beach profiles. From this approach, the greater extent and volume of the nearshore sand wedge at the southern profiles (MB1, MB2, MB3) located at the relic and contemporary mouths of the Clutha River are noticeable in Figure 3.2.

3.2.4 Beach Profile Data

The five beach profiles from the 2013 LiDAR (MB1-MB5) were compared to profiles extracted at the same locations from the 2020 LiDAR to give data on the magnitude of recent shoreline profile changes occurring along Molyneux Bay. These temporal profile comparisons are presented in Appendix A. Spatial comparisons of the five profiles extracted from the 2020 LiDAR are shown in Figure 3.2.

As shown in Figure 3.1, an additional eleven profiles (P1-P11) were extracted from 2020 LiDAR data along the shoreline to give a greater representation of shoreline cells to get profile dimensions for input into the modified Bruun model to assess the erosion effects with SLR. These profiles are presented in Appendix B.

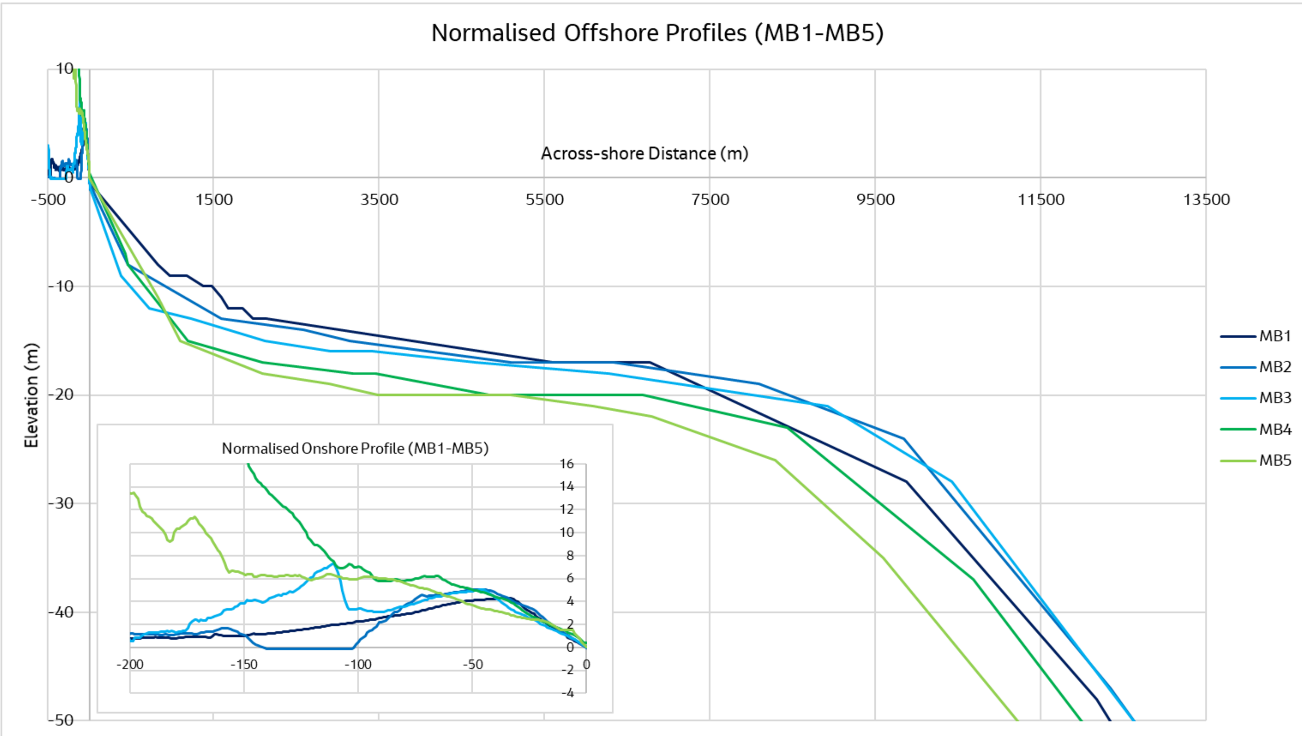


Figure 3.2: Normalised nearshore bathymetry from 2014 multibeam survey and beach profiles from 2020 LiDAR survey. All profiles normalised to zero distance at MSL position.

3.2.5 Sea Level and Wave Data

NIWA were subcontracted to provide extreme event distributions for sea level and waves required for inputs into the short-term storm effects and inundation assessments. The base sea level data was from the 16-year record (2002 to 2018) of total high-water levels at the open coast Green Island, Dunedin site. This data contained both the tidal and storm surge components. For deep water waves, the base data was extracted from the 40 year global hindcast (JRA55 covering 1955-2019) for the closest cell to the Clutha Mouth (cell resolution 1.25 x 1.25 degrees), calibrated by the differences between the appropriate hindcast cell and Banks Peninsula deep water wave buoy for the period 1999-2019. Then the Heffernan and Tawn joint probability method was applied to each of the two timeseries, and a 10,000-year simulation was run from which the extreme event distributions were calculated. The resulting joint probability extreme event return periods for storm tide level, storm surge, deep water significant wave height and peak wave period are given in Table 3.1.

For extreme water levels arriving at shore, open coast wave set-up was calculated at each of the beach profile locations (MB1-MB5) using the above water levels, deep water wave conditions and the combined beach-bathymetry profile in a simple numerical wave shoaling model from Deltares (2016). The resulting wave set-up and total extreme water levels at shore for 50, 100 and 200-year ARI's used for input in the short-term erosion assessment and Stage 1 inundation modelling are presented in Table 3.2. For comparison the total water level at shore for the Clutha area from NIWA (2008) is also included in Table 3.2.

Table 3.1: Joint Probability Extreme event return periods for high water level, storm surge and deep-water wave characteristics. Data provided by NIWA

	Return Periods (ARI)					
	10 Years	20 Years	50 Years	100 Years	200 Years	500 Years
P50$^{(1)}$ Current Storm Tide Level (DVD1958)	1.52	1.57	1.62	1.65	1.68	1.73
P50 Storm surge component (m)	0.47	0.50	0.55	0.58	0.61	0.66
Current Deep water Hs (m)	8.11	8.42	8.80	8.99	9.19	9.39
Current Deep water Tp (sec)	20.94	21.48	22.16	22.58	23.08	23.62
Notes:	(1) P50 is the 50th percentile current storm tide level combined with current wave/storm surge climate, hence is considered the 'most likely storm tide level'. The 95th percentile current storm tide level was also calculated as the 'very unlikely' extreme level, which was only 1 cm higher that the 'likely' level at 100, 200, and 500 year ARI's.					

Table 3.2: Calculated open coast wave set-up and resulting total high-water levels at shore

	Return Periods (ARI)								
	50 Years			100 Years			200 Years		
P50 Current Storm Tide Level (DVD1958)	1.62			1.65			1.68		
Beach-bathymetric profile location	MB1, MB5	MB2, MB4	MB3	MB1, MB5	MB2, MB4	MB3	MB1, MB5	MB2, MB4	MB3
Open coast wave set-up (m)	0.65	0.57	0.58	0.66	0.58	0.59	0.67	0.59	0.60
Resulting total high water at shore (e.g. includes wave set-up) (DVD1958)	2.27	2.19	2.20	2.31	2.23	2.24	2.35	2.27	2.28
Total high water level at shore from NIWA (2008) (DVD 1958)	2.19			2.25			2.36		

For sea level rise, the projected increments of rise were simply added to the water levels, while for future wave projections the NIWA WASP[2] model projections for 2070-2100 assuming the IPCC AR4 scenario B2 was used. These projections were for extreme deep-water significant wave heights to increase by around 1 m, which would add in the order of 5cm to the wave set-up in addition to the sea level rise.

3.3 Calculation of the Projected Future Shoreline Positions

3.3.1 Projected Future Shoreline Position Formula

This erosion hazard assessment applied an integrated approach using commonly accepted coastal erosion hazard formula to estimate the Projected Future Shoreline Position (PFSP) based on extrapolation of past shoreline movements, geometric models and empirical relationships to calculate the impact of sea level rise (SLR), and knowledge of the local coastal processes and geomorphological responses. As a result, along some sections of the coast, the shoreline projections obtained from the hazard formula have been modified due to

[2] WASP: Wave and Storm-surge Projection

shoreline plan–shape and process considerations, particularly where there are small erosional pockets where there have been 'blow-out's with sand blowing through into the backshore.

The following coastal erosion hazard formula has been used to calculate the erosion distances from the current shoreline position to the PFSP. This formula meets the requirements of NZCPS Policy 24 (Department of Conservation, 2010):

PFSP = (LT X T) + SL

Where:
T = Time frame (i.e. 30 and 50 years for this assessment for reasons set out in section 1.1)
LT = Rate of long-term shoreline movement
SL = Erosion due to accelerated SLR over the selected time frames

It is common to also include short-term storm erosion in the erosion hazard formula for defining land-use hazard zoning or building set-back. This term acts as a safety factor in the hazard zone calculation for ensure that developments or land-uses located behind hazard zone within the planning timeframe are not adversely impacted by an extreme erosion event occurring at or near the end of the planning timeframe, such that it would not be accounted for in the extrapolation of the average long-term rates. This is particularly important for dynamic beach systems which experience periods of both erosion and accretion. But, since this assessment is for asset management purposes of the LCFPDS infrastructure rather than for land-use or building control purposes, and due to a lack of data on beach responses to storm events creating large uncertainty in appropriate magnitude of short-term effect to include, this component has not be included in the PFSP. However, an estimate of the potential short-term erosion affects has still be provided based on the results of SBEACH numerical simulation modelling. Further details on this modelling methodology are described in Section 3.4, and the results are presented in Section 4.3.

3.3.2 Timeframes and Sea-Level Rise Scenarios

As indicated in Section 1.2, timeframes of 30 and 50 years have been applied to the calculation of the erosion distances to the PFSP. As shown in Table 1.1, within these timeframes, incremental SLR scenarios of 0.2m by 2050, and 0.4m and 0.6m by 2070 have been modelled, which covers the range of SLR magnitudes from the MfE (2017) scenarios for these timeframes.

As also indicated in Section 1.2, these projected future magnitudes of SLR have been discounted by the contemporary rate of rise over the last 50 years (taken as an average of +2 mm/yr) to avoid double accounting of the theoretical effect of contemporary rates that are already included in the extrapolation of the historical coastal erosion. Therefore, the calculation of erosion due to SLR is for the acceleration in rate of rise above the contemporary rate. With consideration for the presence and effect of the LCFPDS mouth training structures on shoreline movements, the following timeframes and scenarios were assessed:

- 30 years (e.g. to 2050) with SLR of 0.2 m under the assumption that the southern training wall is maintained in its current length and condition such that post training wall erosion rates continue;
- 50 years (e.g. to 2070) with SLR of 0.4 m and 0.6 m under the assumption that the southern training wall is maintained in its current length and condition such that post training wall erosion rates continue for this entire period; and
- 50 years (e.g. to 2070) with SLR of 0.4 m and 0.6 m under the assumption that after 30 years (2050) the southern training wall has been removed, and pre-wall construction erosion rates apply for 2050-2070 period due to reinitiation of sediment supply south of the Koau Mouth.

3.3.3 Extrapolation of Long-Term Historical Shoreline Movements

The inclusion of the long-term historical shoreline movement in the PFSP calculation is to ensure that appropriate previous trends driven by the relationship between wave climate, sediment supply, and sediment transport in each geomorphic cell are accounted for in future projections. For this assessment, it is assumed the same coastal processes will continue over the 50-year timeframe, including wave and wind climate, sediment supply and transport.

Historical shoreline positions used in the assessment are limited to those determined from the eight aerial imagery dates over the 74-year period between 1946 and 2020, as given in Section 3.2.2. We have excluded the use of earlier cadastral shoreline positions due to the limitations described in Section 2.5, and due to uncertainty of their relevance for extrapolation into the future.

These images were geo-referenced using common features and shorelines were digitised in ArcGIS, with shorelines being defined by either the vegetation line along the foredune or the position of back of the beach where vegetation was absent (e.g. rollover slopes south of Koau Mouth, within the river mouth). The GIS-based DSAS (Digital Shoreline Analysis Systems) tool was then used to calculate the net shoreline movements and long-term historical rates at transects located at 50m intervals, creating 221 transects over the 11km of shoreline. The digitised shoreline positions from the aerial imagery and the location of the DSAS transects are shown in Appendix C.

The long-term erosion rate from linear regression (LRR) of the erosion distances calculated by the DSAS was used as the historical trend component of the PFSP equation when the R^2 value for the transect was high, indicating a strong linear trend. The 95% confidence interval calculated in the DSAS analysis is used as the minimum and maximum values to form the long-term rate distribution for input into the monte carlo simulation for assigning probabilities of the erosion distribution (see Section 3.3.6).

From the results of the DSAS analysis, the rates across the total period were analysed as well as the rates both pre (1946-1972) and post (1982-2020) construction of the Koau Mouth Southern Training Wall, as presented in Table 3.3. The results of this preliminary historical erosion assessment (see Section 4.1.1) indicated a significant change in erosion rate south of this mouth since the construction of the wall. For the remainder of Molyneux Bay shoreline north of the Koau Mouth, there was minimal change between rates pre and post-wall construction, and there is a stronger and less variable linear relationship between erosion distance and time (as expressed by the R^2 values for each transect) over the total record than when the record is split into two time periods. A comparison of the DSAS results for this study, and previous studies across the same geographic area is presented in Table 3.3. Therefore, for this section of shoreline we can be more confident about extrapolating these rates into the future. The resulting historical rates used for extrapolation in each scenario are detailed below in Table 3.4, and the raw probability distributions used in the PFSP calculations are presented in Appendix G.

Table 3.3: Summary of shoreline change rates from this study and previous assessments.

		South Koau Mouth	Central Molyneux Bay	North Molyneux Bay
Source	**Timeframe**	Rate	Rate	Rate
Gibb (1978)	1847-1962	+3.30 m/yr	+2.52 m/yr	+1.26 to +0.61 m/yr
Hornblow (2016)	2006-2016	-10 to -6.9 m/yr	-5.6 to +1.9 m/yr	
	1946-2016	-4 m/yr		
Jacobs (This study)	1946-2020	-4.08 to -3.87 m/yr	-1.57 to -0.3 m/yr	+0.6 to+ 0.81 m/yr
	1946-1982 (Pre-Koau training wall)	-5.04 to -0.94 m/yr	-3.52 to -2.71 m/yr	+0.94 to +0.98 m/yr
	1997-2020 (Post-Koau training wall)	-9.73 to -6.34 m/yr	-4.45 to +0.84 m/yr	+0.89 to +0.83 m/yr

Table 3.4: Extrapolation of linear regression erosion rate (LRR) from DSAS analysis based on future assumptions around the southern training wall.

Scenario	South of Koau River Mouth	North of Koau River Mouth
30-year (2050) with southern training wall maintained	Extrapolation of post-construction LRR (1997-2020)	Extrapolation of total record LRR
50-year (2070) with southern training wall maintained	Extrapolation of post-construction LRR (1997-2020)	Extrapolation of total record LRR
50-year (2070) with southern training wall removed in 2050	Extrapolation of post-construction LRR (1997-2020) to 2050, then extrapolation of the pre-construction rate (1946-1982) for the period between 2050-2070.	Extrapolation of total record LRR

3.3.4 Limitations to Calculating Long-Term Historical Shoreline Change

The following limitations have been identified with the methods used to calculate the long-term historical shoreline change:

- The long-term rate is dependent on the accuracy of the georeferencing of the imagery. Georeferencing was more difficult for earlier imagery (1946-1982) due to the image quality and the lack of development of land in rural areas limiting common features and structures.
- The poor image quality on earlier imagery also increased the uncertainty in the digitising of the shoreline due to difficulty in identifying the features used to determine shoreline position. For sections of coastline where the shoreline could not be determined, a shoreline was not digitised from that image, resulting in gaps in the analysis for several transects for that period.
- At river mouths, the dynamic nature of the shoreline feature used (i.e. back of beach) means that the resulting long-term rate may not be a reflection of the trend of the general coastline in that area. For river mouth environments, shorelines were still digitised across the back of the barrier, and it was determined in the mapping phase of the assessment if the resulting PFSP lines required adjustment to fit shoreline plan-shape considerations. This was particularly an issue for pre-construction of the training walls at the Koau Mouth when the shoreline was prone to periodic migration.
- The assumption that the historical rate of shoreline movement will continue at the same linear rate in the future, including that the current plan shape of the bay with be retained. This assumption was tested by undertaking an assessment of how well the current shoreline south of the Koau Mouth fits a theoretical log-spiral plan shape in the lee of Kaka Point and what the consequences of future shoreline development within this plan shape (see Section 3.3.8 for methods and Section 4.4.2 for results).
- The assumption that the effect of the southern training wall on coastal processes and therefore erosion rate will continue in the same way over at least the next 30 years.

3.3.5 Erosion Impacts of Accelerated Sea Level Rise

The beaches along the Molyneux Bay shoreline are primarily low sand beaches prone to sediment rollover due to frequent wave overtopping. The resulting future erosion effect from the acceleration of SLR is calculated at each of the DSAS transects using a Modified Bruun Rule approach (Rosati et al., 2013). The general ‘Bruun Rule’ approach (Bruun, 1962, 1988) is widely used in international literature and is recommended in the MfE (2017) guidance as an appropriate method for sand beaches. The model involves the assumptions of conservation of an equilibrium profile shape with the volume eroded seaward from the beach being that required to raise the nearshore profile out to the closure depth for cross-shore sediment transport by the same vertical magnitude as the magnitude of SLR. Therefore, the resulting horizontal shoreline retreat is dependent on the beach-nearshore slope from the dune crest to the closure depth.

The modified Bruun Rule approach (Rosati et al., 2013) further accounts for sediment losses landward due to dune overtopping processes, which are dominant along the southern Molyneux Bay coastline. The use of this method along the Molyneux Bay southern and central shoreline is appropriate as it still accounts for sediment being taken from the onshore beach profile to build the elevation of the nearshore profile at the same pace as SLR, while also giving consideration to the fact that there will be losses of volume into the backshore due to overwash of the low lying beach profile. The frequency of overtopping events at these low elevation profiles will increase over time with SLR, and therefore it is important to account for this additional erosion process from SLR. A sensitivity analysis indicated that the inclusion of this overwash/rollover factor results in up to an additional -2.5 m of horizontal shoreline erosion over all timeframes and SLR scenarios assessed in this study. The modified Bruun formula from Rosati et al (2013) is expressed as:

$$Retreat = S\,\frac{L + \frac{V}{S}}{(h + d)}$$

Where:
S is SLR
h is the dune crest height
d is the closure depth
L is distance from dune crest to closure depth
V is sediment roll-over volume in units of per metre length of beach

The components for the Modified Bruun equation were determined by the following:

- Dune crest height above MSL was obtained from the beach profiles extracted from the LiDAR data. Maximum and minimum elevations were calculated based on the elevation difference between the three ORC historical profiles (2009, 2013, and 2020) associated with each profile. For example, if the height of the dune crest at P1 was 4.2, and the range of the dune crest at its associated MB1 profile was 3.9 to 4.5 m (giving a range of 0.6 m), then the minimum dune crest at P1 was 3.6 m and the maximum was 4.8 m.
- Closure depth was calculated using Halliemeirs equation, using the inner and outer depths as the maximum and minimum, and median point between these two as the mean. For this equation, the storm wave input was taken as the mean of the maximum annual significant wave height calculated from the 15 year of hindcast wave data at Green Island as supplied by NIWA, which gave value of H_s = 2.6 m. The outer Halliemeirs closure depth (maximum) calculated in this study was 7.8 m, which is in the same order as the maximum 8-9m closure depth presented by Atkin et al (2020).
- Distance from the dune crest to the closure depth was based on the width of the beach from the onshore profile (P1-P11) combined with the distance from MSL to the closure depths based on the ORC bathymetry data.
- Sediment roll-over volume was calculated from the backshore volume differences between the 2013 and 2020 ORC profiles. A schematic example of these calculations for profile MB1 is shown below in Figure 3.3.
- Where there was no sediment roll over, V was equal to 0 and the normal Bruun Rule was applied.

The raw probability distribution inputs for each SLR scenario used in the PFSP calculations are presented in Appendix G.

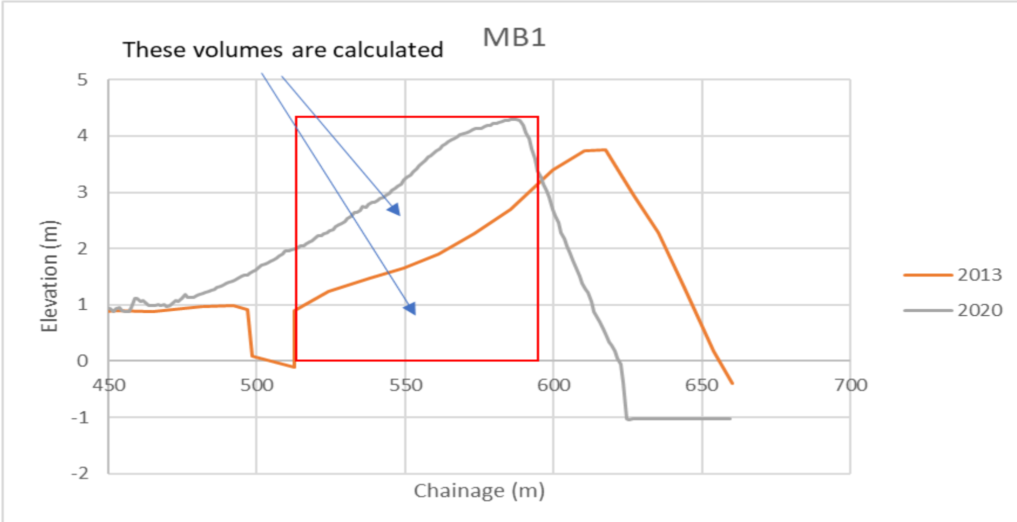


Figure 3.3: Explanatory diagram showing the areas used to calculated volume overtopping.

Limitations of the adjusted Bruun Rule are well documented, including the following relevant points for Molyneux Bay:

- It assumes only a two-dimensional cross-shore sediment movement, hence it does not include consideration of longshore sediment transport input/ losses, or plan shape controls (e.g. headlands);
- It is only applicable to equilibrium beach profiles;
- It is difficult to accurately determine a closure depth for offshore sediment transport; and
- It does not account for morphological change of the beach profile going forward, i.e. loss of volume lowering the dune crest. This model assumes the same beach profile shape and elevation will be maintained into the future.

3.3.6 Probabilistic Approach

A probabilistic approach manages the uncertainty surrounding the data inputs and the results obtained from the methods used to define each of the components of the PFSP calculation.

The probability calculations involved using the mathematical software MATLAB R2019b to run a 'Monte Carlo' simulation where for each transect and SLR scenario, 10,000 realisations of the PFSP lines were made by combining random values from each of the long-term (LT) and SLR erosion distributions. To run the 'Monte Carlo' simulations, a triangular distribution was assumed for each component of the PFSP at each transect using minimum, mean, and maximum values obtained or assumed from the data. This was considered the best distribution to apply given the limited nature of data available to define the uncertainty in the value of these components. Sensitivity testing between a normal and a triangular distribution of the LT erosion rates showed that there was minimal difference between the two distributions.

The resulting distribution of the PFSP realisations shows the range of where the projected shoreline will be in relation to the present-day shoreline, and what the probability is that the erosion could extend beyond a given distance. Figure 3.4 below shows the PFSP distribution output for a single transect, where the bars represent the number of realisations from the 10,000 trials.

In this assessment, we present the 50th percentile and the 95th percentile of 10,000 random observations of the PFSP erosion distance for each timeframe and SLR scenario. The 50th percentile represents a 'most likely' magnitude of erosion, in which there is a 50% chance that erosion will extend beyond this position and 50% that it will be less than this position. This position is the mid position of a 'about as likely as not' range of positions following the terminology of MfE (2017) (e.g. 33-66% probability of occurrence range). It is denoted in the mapping as the P50 position for each timeframe and SLR scenario.

The 95th percentile statistically represents the erosion distance that has only a 5% probability of being exceeded therefore is referred to as being a 'very unlikely' magnitude of erosion, and is denoted in the mapping as the P5 position for each timeframe and SLR scenario. Our reason for using the P5 notation in the mapping is that in describing the results, it is easier in the context of extreme conditions to explain that there is a 5% chance of the position being exceeded, rather than a 95% chance that it will not be exceeded.

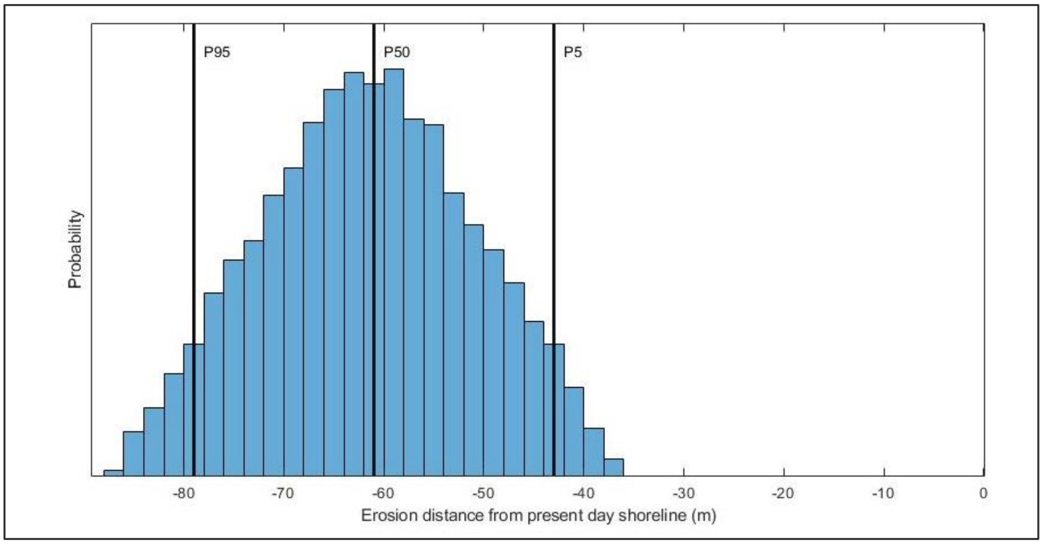


Figure 3.4: Example of a single transect and the probability of PFSP distances from the present-day shoreline. The bars represent the 10,000 realizations of the projected future shoreline position made from drawing random LT and SLR from their respective distributions.

3.3.7 Mapping of PFSP

Mapping of the PFSP at each DSAS transect was undertaken in ArcGIS, using the Offset Line tool to create lines offset from the 2020 shoreline based on the P50 and P5 values from the 'Monte Carlo' simulation for each SLR scenario. Once the lines were plotted in ArcGIS, these were smoothed by removing points along the line which created segments of discontinuity along the future shoreline that were not supported by geomorphic plan shape considerations[3] based on knowledge of the coastal processes. This primarily occurred at localised 'beach blow-out' areas created by localised dune rollover.

Overview maps of the Molyneux Bay shoreline with PFSP lines for 2050 and 2070 are presented in Appendix D. Shapefiles of these lines were also presented with this report to ORC.

3.3.8 Logarithmic-Spiral Plan Shape Assessment

An assessment was also undertaken to determine the role of a theoretical logarithmic spiral plan shape in the lee of headland formed by the southern Catlins in determining the future shoreline position for southern Molyneux Bay cell to the south of the Koau Mouth.

[3] Geomorphic plan shape considerations: The removal of steps in the shoreline plan shape that do not occur in nature.

Beaches located in the lee of a headland and subjected to a predominant direction of wave attack around the headland characteristically have a seaward-concave plan shape resulting from erosion caused by refraction, diffraction and reflection of waves into the shallow zone behind the headland (Yasso, 1965). The gradual decrease in curvature of the bay with increasing distance from the headland reflects the decreasing influence of the headland on the shore, with the distance-decay function being curvilinear that can be represented by logarithmic-spiral curves, with the increasing radius of plan curvature with distance from the headland approximating a log-spiral curve (Yasso, 1965).

The equations used to generate the log spiral curve and test for 'goodness of fit' of the curve are included in Appendix E. For this assessment, the origin of the log-spiral curve that best fits the 2020 shoreline was determined, then the same origin was used for testing the 'goodness of fit' and anomalies of a log-spiral curve to the 2050 PFSP shape as a test of where the PFSP may over or under estimate the shoreline erosion. The results of this shoreline plan shape assessment are presented in Appendix F.

3.4 Short-Term Storm Erosion

In this assessment, due to the lack of historical data on inshore waves, beach storm responses in the study area, the numerical simulation model SBEACH-32 was used to provide indicative and relative shoreline movements in theoretical extreme storm event. Three storm ARI simulations of 200 years, 100 years, and 50 years were run for each of the five 'MB' profiles, with the input deep water wave being from the NIWA data presented in Table 3.1. These waves were run as arriving perpendicular to the shore at constant values for a 48 hours storm duration. Maximum storm tide elevations during the events were also taken from those provided by NIWA in Table 3.1, with a spring tide range super-imposed to simulate water level variations over the 48-hour storm duration.

The erosion results from the SBEACH-32 modelling are presented in Section 4.3. However, since there is no site-specific wave data or regular beach profile monitoring programme at Molyneux Bay, the results of the modelling could not be calibrated. It is also noted that the use of SBEACH modelling is limited on beaches which are susceptible to overwash, such as the low-lying beaches south of the Koau mouth. The model itself does not have good mechanisms for addressing overwash and resulting beach lowering. The use of constant wave input at the maximum levels was considered most likely to over predict erosion, however, observation of the simulation indicates that it appears to underpredict the anticipated amount of wave overtopping of the low dune profiles at the southern end of the bay, which would under predict the resulting erosion for these sites.

3.5 Coastal Inundation Hazard Assessment

As indicated in Section 1.1, a two stage approach was proposed for the coastal inundation assessment, where stage one used a 'bathtub inundation' approach to determine the likelihood of overtopping of the LCFPDS floodbanks with SLR over a 100 year timeframe, and stage two involving hydrodynamic modelling to confirm inundation extents and depths where stage one determined that overtopping was likely to occur. This report presents a summary of the methods and results of the stage one assessment and makes a recommendation to ORC whether to proceed with stage two modelling. The complete methods and results of this assessment were presented to ORC prior to the completion of this report, and are presented in Appendix F.

3.5.1 Stage 1 Inundation Assessment

For this assessment, the surveyed crest levels of the LCFPDS floodbanks parallel to the shoreline and in the lower river channels as shown in Figure 2.6 were compared to the current and future 100 and 200 year ARI sea levels to identify locations where the crest would be overtopped by these levels. The following information was used in the assessment:

- Otago Regional Council provided a Floodbank Crest Level Survey carried out by Terramark 27th October and 17th November 2017. Crest Levels were provided in Otago Metric Datum Height as well as NZVD16 with

Local Vertical Datum correction factors supplied. For the analysis all levels were converted to DVD 1958, for which MSL has an elevation of 0.114 m.

- The current total high-water levels at shore presented in Table 3.2, being comprised of the 'most likely' (e.g. P50) storm tide levels provided by NIWA (Table 3.1) and the open coast wave set-up calculated for this project.
- Coastal water levels in the river mouths being the combination of the NIWA storm tide levels in Table 3.1, and wave set-up inside the river mouths calculated as being 23% of the storm tide (Irish & Canizares (2009).
- Increments of SLR as given in Table 1.1.

There is no allowance for freeboard in this assessment. Overtopping or breaching could therefore occur at water levels that do not reach the crest of the stopbank, depending on the structure of the bank (for breaching) and wind conditions (for local wave overtopping).

A potential issue in addition to sea level rise is the subsidence of the stopbanks over time on soft alluvial sediments, which could lower the crest height. At present, current survey datasets (LiDAR, crest surveys) are not of the accuracy or duration to allow for any assessment of the level of subsidence that has occurred since the construction of the stopbanks as a result of settling on alluvial sediments.

4. Coastal Erosion Assessment Results

4.1 Long-term Historical Shoreline Movements

4.1.1 Assessment of Past Rates of Movement

The plots of historical shoreline position from aerial photography and the DSAS results of long-term rates of shoreline movement over the total 74 years from 1946 to 2020 are presented in Appendix C. The total change in shoreline position over this period is presented in Table 4.1, with the shoreline divided into the following three coastal cells:

1) Southern Molyneux Bay - South of the Koau Mouth
2) Central Molyneux Bay - Koau Mouth to the Matau Mouth
3) Northern Molyneux Bay - North of the Matau Mouth

For the analysis of shoreline movements, each coastal cell was further divided in sub-cells with alongshore lengths of 0.5km to 1.5km based on the obvious breaks in shoreline behaviour from the historical mapping. For example, sub cells are defined north and south of each river mouth based on positions of the mouth openings.

Table 4.1: Summary of the rate of change in shoreline position from DSAS linear regression (LR) analysis for 0.5 to 1.5 km sub-cells of shoreline for total record (1946-2020), pre and post Koau Mouth South Training Wall construction.

Shoreline Cell	Sub-cell DSAS Transects	Average Total Erosion/Accretion Distance (1946-2020) (m)	Average Total Record (1946-2020) LR rate (m/yr)	Average Pre-Koau south training wall (1946-1982) LR rate (m/yr) (1)	Average Post-Koau south training wall (1997-2020) rate (m/yr) (1)
Southern Molyneux Bay - South of Koau Mouth	1 – 20	-264.1	-4.08	-0.94 (R^2 0.12) EPR:-0.26	-9.73
	21 – 40	-278.3	-4.10	-1.25 (R^2 0.15) EPR: -0.44	-9.22
	41 – 66	-290.6	-3.87	-5.04	-6.34
	67-75 (mouth influence)	Imagery does not cover total time period		Imagery does not cover total time period	-7.49
Central Molyneux Bay - Koau Mouth to Matau Mouth	82 – 95 (mouth influence)	-130.3	-1.57	-3.52 (R^2 0.45) EPR: -2.71	-4.45
	96 – 122	-89.8	-0.97	-1.73 (R^2 0.49) EPR: -1.92	-1.71 (R^2 0.44) EPR: -2.04
	123-135 (mouth influence)	-57.6	-0.30	Imagery does not cover total time period	+0.84 (R^2 0.31) +0.37
Northern Molyneux Bay - North of Matau Mouth	140 – 150 (mouth influence)	+57.8	+0.60	+0.94	+0.89
	161 – 180	+64.4	+0.81	+0.98	+0.83

Shoreline Cell	Sub-cell DSAS Transects	Average Total Erosion/Accretion Distance (1946-2020) (m)	Average Total Record (1946-2020) LR rate (m/yr)	Average Pre-Koau south training wall (1946-1982) LR rate (m/yr) (1)	Average Post-Koau south training wall (1997-2020) rate (m/yr) (1)
	181 - 207	Imagery does not cover total time period		Imagery does not cover total time period	+0.45 (R^2 0.47) EPR: -0.45
Note: (1) For sub-cells with $R^2 < 0.5$, the End Point Rate (EPR) is also presented as the Linear Regression (LR) rate is not considered to represent a trend.					

The key trend from the total erosion distances and rates in Table 4.1 is that the greatest erosion has occurred in southern Molyneux Bay (in order of -4 m/yr), with a constantly reducing rate through the centre of the bay between the river mouths, which reverts to long-term accretion at the northern end of the bay. This trend is constant with net northerly sediment transport and potentially greater supply from the Matau River Branch than the Koau Branch. It is notable that the greatest variation in net erosion rate occurs across the Koau Mouth.

Based on these results, further analysis of the rates of historical shoreline change was undertaken to determine if there were significant changes in the temporal trends associated with the construction of the river mouth training walls. Of particular interest was the effect of the southern training wall at the Koau Mouth on the erosion rates in southern Molyneux Bay. For this analysis, the period covered by the aerial photographs were split into a pre training wall period (1946-1982) and post wall period (1997-2020)[4]. A summary of these results is presented in Table 4.1 and are graphed for each transect in Figure 4.1. The results are discussed in more detail for the three shoreline cells below. For reference in the discussion, a time series of historical images of the Koau Mouth from 1946-2020 is presented in Figure 4.2, the relative longshore position of the mouth presented in Figure 4.3, and 1982 and 1997 images of the Matau Mouth presented in Figure 4.5.

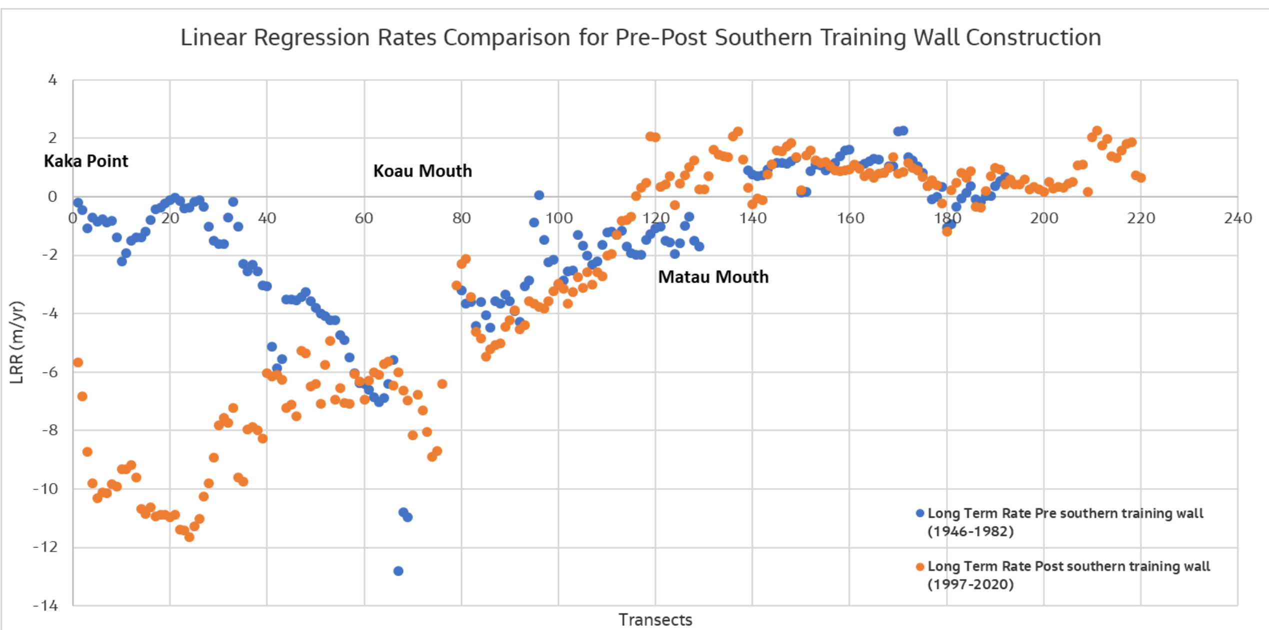
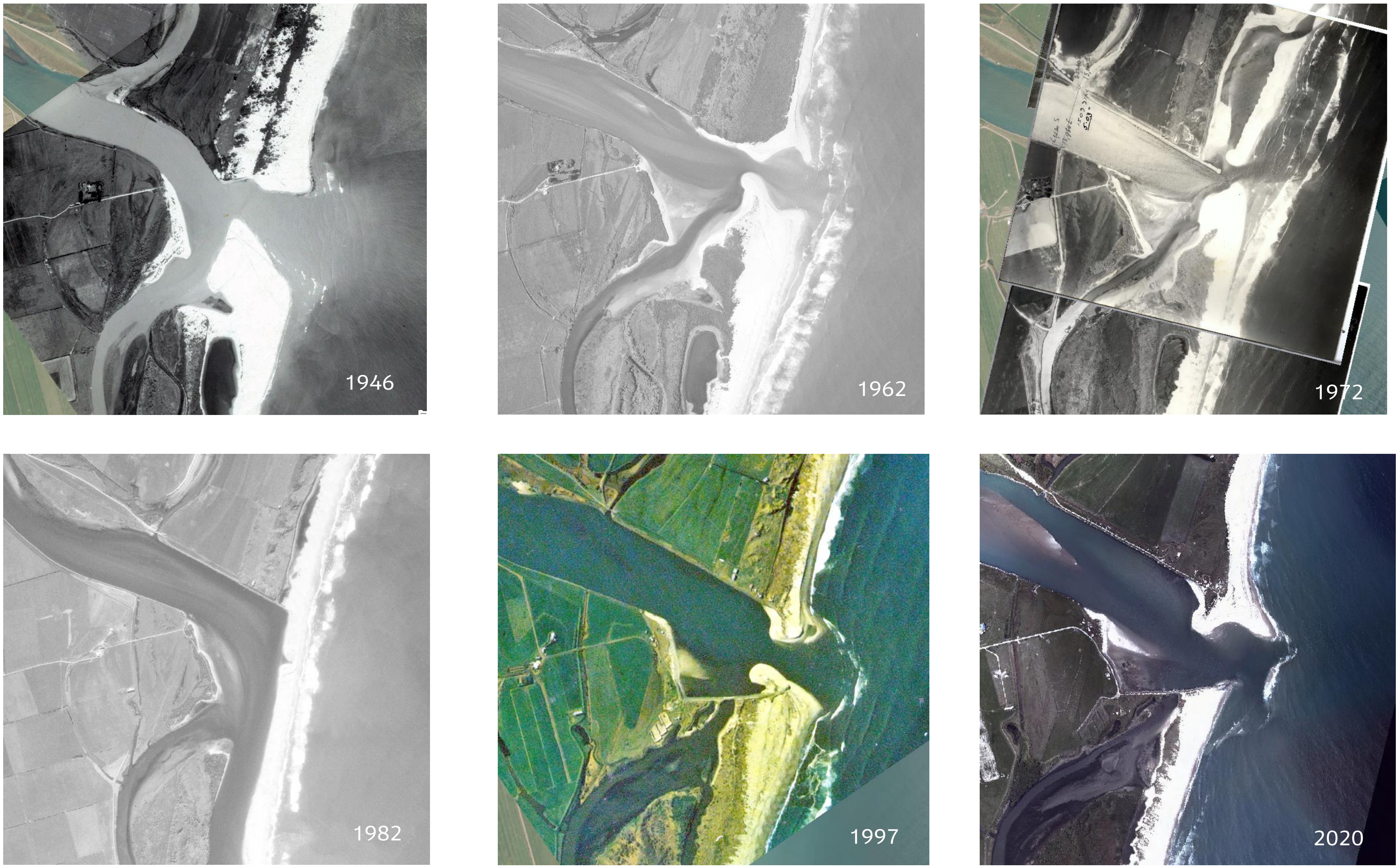


Figure 4.1: Comparison of Linear Regression Rates at all transects pre (1946-1982) and post (1997-2020) southern training wall construction, indicating significant changes to the LRR at transects 1-75 south of the Koau river mouth, and no significant change at transects north of the Koau Mouth.

[4] The southern training wall was constructed in 1983-84, therefore the 1997 is the first aerial photograph with the wall in place.

Figure 4.2: Time series of historical images of the Koau Mouth from 1946-2020, showing the presence of the northern training wall in 1972, and the southern training wall by 1997.



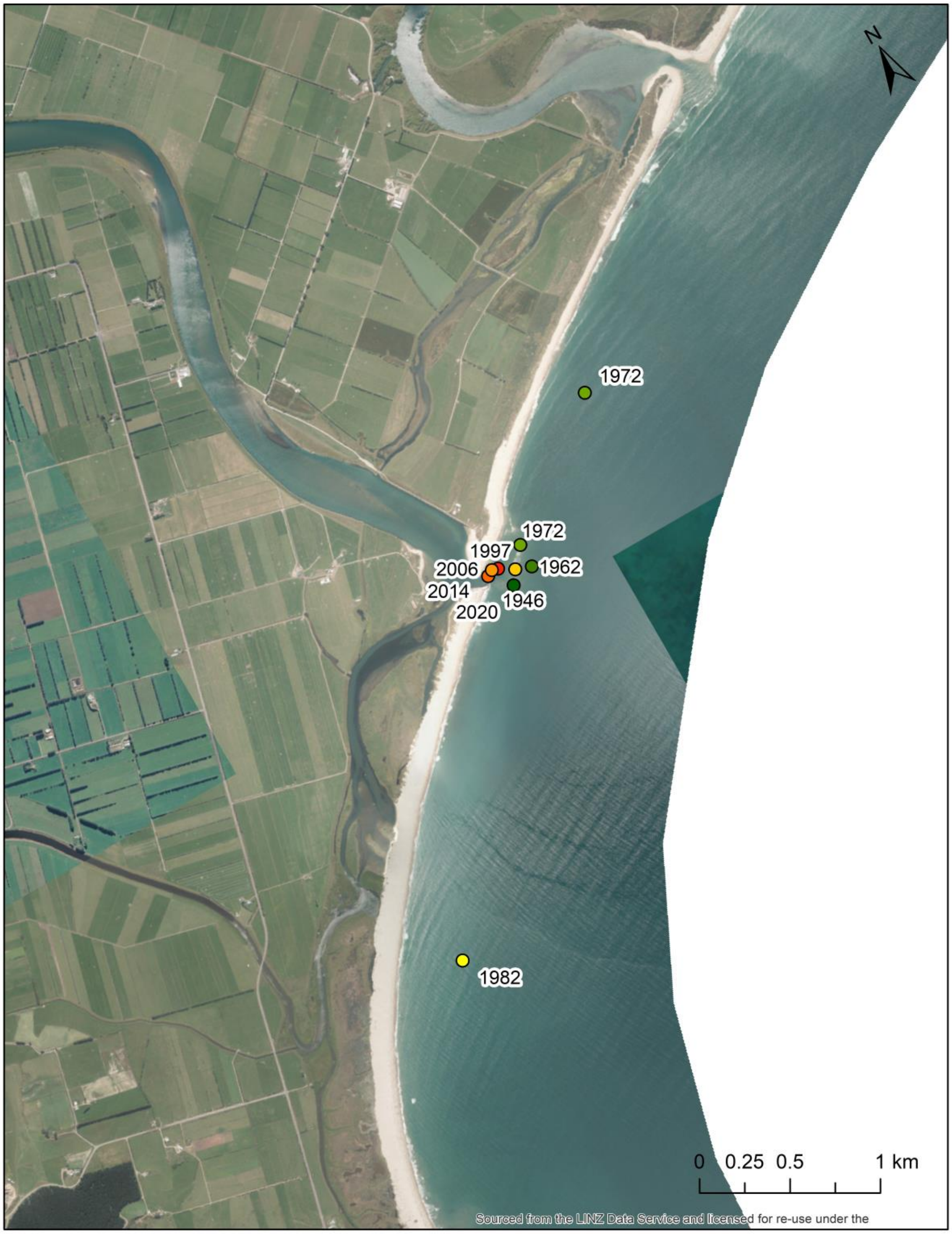


Figure 4.3: Koau River Mouth longshore positions from aerial photographs 1946-2020. Note the 1972 image shows two river mouth openings to the ocean.

4.1.1.1 Southern Molyneux Bay - South of the Koau Mouth

Table 4.1 and Figure 4.1 indicate that this southern cell has been in a state of long-term erosion over the total 70+ year period, which was consistent with the findings of Williams & Goldsmith (2014) and Hornblow (2016a) as presented in Section 2.5. However, the analysis shows that the average rate of retreat within this coastal cell has changed significantly over different time periods.

Pre-south Koau Mouth Training Wall Period (1946-1982)

Prior to construction of the Koau Mouth southern training wall (1983-84), movements in the southern two sub cells (transects 1-40) fluctuated between erosion and accretion, with accretion in 1946-1962 period (+4 m/yr), followed by high rates of erosion in the 1962-1972 period (e.g. -10 to -12 m/yr), and then switch back to accretion in the 1972-1982 period (+3 to +4 m/yr). As noted in Table 4.1, as a result of these fluctuating trends in movement, the LLR is not a good indicator of retreat rate (e.g. r^2=0.12- 0.15), hence the End Point Rate (EPR) of -0.26 to -0.44 m/yr is considered to be a better indicator of pre-mouth training wall shoreline movements for the southern parts of the cell (e.g. transects 1-40).

From Figure 4.3, there appears to be a relationship between mouth position and whether the southern shoreline was in a state of erosion or accretion. Erosion in the middle 1962-1972 period coincided with a northward mouth offset, and from the imagery the erosion appears to be in response to episodes of dune overtopping and rollover removing crest/backshore vegetation. This is a common beach response to coastal storm events for beaches lacking sediment supply that are rolling back onto wetlands, lagoons or low hinterlands.

The dune vegetation line advance in 1972-1982 appears to be in response to increased beach sediment supply associated with a southerly mouth position over this period, with the mouth by 1982 being located at transect 32-33, 2km south of its current position (e.g. total southward migration of around 3km southward over the 10 year period). It is also noted that the largest flood since 1878 occurred during this period, being October 1978 with a flood peak of 4580 m^3/s at Balclutha. As shown in Figure 4.4, the mouth was directed to the south at this time, therefore the injection of the flood bedload of sand and gravel would also be to the south to nourish this southern section of the beach system.

Along the centre to northern parts of the cell (transects 41-66), the shoreline was retreating over all three time periods prior to 1982, with the average erosion over total pre-training wall period (1946-1982) considerably higher for this part of the cell (LLR = -5 m/yr, R^2= 0.82). These net higher rates are likely to have been influenced by the southward migration of the river mouth channel through this part of the cell in the 1972-1982 period resulting in average retreat at -8.8 m/yr at a time when the more southern parts of the cell were accreting.

Figure 4.4: October 1978 flood event on the Clutha Delta showing a southward orientated Koau Mouth (Source: Hornblow, 2016a)

South Training Wall Construction Period (1982-1997)

For the period including the southern Koau Mouth training wall construction (1982-1997), the trends were reversed, with the southern part of the cell (transects 1-40) eroding at average rates of -2.6 to -4.2 m/yr, considered to be likely in response to reduced river supply again, while the transects in the centre of the cell (transects 41-66), accreting by 2.50 m/yr, are most likely in response to infilling of the former mouth channels following mouth stabilisation and possible positive impacts of shoreline adjustments from trapping of northward moving sediment against the training wall. As shown in the 1997 image in Figure 4.2, the beach immediately south of the wall at this time was well seaward of the end of the wall and offset more to the east than the beach to the north of the mouth, again inferring a dominant northward transport over this period, and sand bypassing the southern wall via the inner edge of the ebb tide delta, to be either deposited in the mouth channel as spits on both the southern and northern side of the channel, or on the beach north of the stabilised mouth position.

Post South Training Wall Period (1997-2020)

In contrast, for the periods following the construction of the southern training wall (1997-2020), trends of shoreline movement over the whole cell have been consistently erosional throughout the all-time intervals between aerial photographs (1997, 2006, 2014, 2020). Net retreat rates over the 23-year period have increased compared to the 1946-1982 period, however rates over the last 6 years (2014-2020) have been lower than in the preceding 1997-2006 and 2006-2014 time periods. Over the total 23 year period the highest rates at the southern end of the cell (Transects 1-40) in the order of -10 m/yr. Closer to the Koau Mouth (transects 41-75), the post training wall net erosion rates are lower, in the order of -6 to -7.5 m/yr, however, for transects 41-66 this is still an increase from the pre-construction rate.

The mechanism of this post wall acceleration in erosion over the whole cell appears to be a negative feedback loop of increased beach rollover lowering dune crest elevations, which in turns allows more frequent rollover and therefore further increased erosion. As indicated above, this is a common shoreline response for beaches lacking sediment supply that are rolling back onto wetlands, lagoons or low hinterlands. There are four combined possible causes of the reduced sediment supply to these southern beaches.

1) A lagged response to the reduced supply from the Clutha River since the construction of the Roxburgh Dam (1957).
2) Lack of significant floods in the Clutha River in the 20 years from November 1999 to December 2019. As reported by Hornblow (2016a), the only other decades in the past 150 years that did not experience events greater that 2000 m^3/s were the 1880's and 1890's.
3) The presence of southern training wall preventing southern migration of the river mouth, therefore removing any significant periodic sediment inputs to the southern beaches.
4) The alignment of the training walls directing the ebb tide jet further offshore and into the more northerly sediment transport pathway.

The spatial pattern of retreat, being greatest in the southern to central section of the cell, is constant with the wave refraction pattern and lack of beach/nearshore longshore supply around the headlands of the northern Catlins (e.g. Nugget Point and Kaka Point), therefore supply is limited to periods of southerly counter drift and southward by-passing of the training wall and offshore from the nearshore wedge. The lower erosion rates for the beaches closest to the Koau Mouth reflect that they can receive some sediment redirected onshore from the current ebb tide delta and the trapping against the training wall of northward transported material.

Based on an analysis of beach profile change from 2013-2020 for profiles MB1 and MB2, this area of shoreline has an average beach cross sectional volume (e.g. above the msl contour) loss of -2 m^3/m per year associated with an average crest retreat rate from the profiles of -4 m/yr. Therefore, an average volume loss ratio of 0.5 $m^3/m/yr$ per 1 m of horizontal retreat can be applied to the erosion distance over the 23 year period post southern training wall construction (-8 m/yr from Table 4.1) across the total length of shoreline south of the Koau mouth (3600 m), to obtain an indicative net volume loss in the order of-14,400 m^3/yr from this section of shoreline to offshore and longshore transport processes. Despite the high horizontal erosion distances along this section of shoreline, these indicative volume losses are small (e.g. compare to the shoreline between the Koau and Matau mouths) due to the large influence of beach rollover in the erosion processes, which retains cross-section volume in the beach profile as the shoreline retreats landward. It is also noted that this net volume loss rate is much less that the 50,000 m^3/yr longshore transport rate calculated by Atkin et al (2020b) for this section of the shoreline.

4.1.1.2 Central Molyneux Bay - Koau Mouth to Matau Mouth

In relation to the southern cell, the results in Table 4.1 and Figure 4.1 show the following two important differences for this central cell.

1) The net rates of retreat over the total 70+ year period are lower and show a spatial trend of reducing retreat in a northerly direction. This result is consistent with the dominance of net northerly longshore transport over the majority of this cell as shown in Figure 2.12 from Atkin et al (2020b), hence this cell regularly receives sediment supplied by the Clutha River.

2) The rates of shoreline movement have been generally similar across both pre and post Koau training wall periods. However, these masks shorter phases of erosion and accretion, which at least for the 850m

immediately north of the Koau Mouth (Transects 82-96) appear to have been influenced by the position river mouth.

Between 1946 and 1962, the average rate of shoreline change across this sub cell was advancing at +3.7 m/yr, which from the 1962 image in Figure 4.1 was due to the development of dense vegetation on the dunes, most likely accompanied by increase in dune size due to aeolian processes. This was followed by rapid erosion up to at least 1972 (130-190 m retreat at average of -16.4 m/yr) associated with the northern migration of the river mouth channel to around transect 98-99 (Figure 4.3). As noted above, and also shown in Figure 4.3, by 1982 the mouth had migrated up to 3km south with the former northern mouth channel being infilled and re-vegetated, which shows as accretion over the 1972-1982 (+0.7 m/yr) and 1982 to 1997 (+4.2 m/yr) periods. In the 1997 image the seaward end of the northern wall has been buried by accumulated beach sediment and is located approximately at the vegetation line.

Post 1997, this section of shoreline has been erosional over all time periods, with average net retreat in the order of 100m up to 2020 (average -4.45 m/yr). As with the southern cell, the greatest retreat occurred in the 2006 to 2014 period. It is noted that although these post training wall retreat rates are high, they are only around 60% of those on the immediate south side of the Koau Mouth, and around 50% of those experienced at the southern end of the bay.

Although the central sub-cell (Transect 96-122) is not directly influenced by the presence of migrating mouth channels from either the Koau or Matau Mouth, this sub-cell also experienced fluctuations between shoreline retreat and advance in the time-periods both prior and post the construction of the training walls. Maximum erosion rates of -6.59 m/yr occurred in the 1972-1982 period when the Koau Mouth was located to the south, and -9.14 m/yr in the recent 2014-2020 period. Maximum accretion rates of +1.98 m/yr occurred in the 1982-1997 period, corresponding to the periods of southward Matau Mouth discharge in 1982 (Figure 4.5) and 1992-93 (Johnstone, 1993), as well as the construction of the Matau Mouth training wall in 1987. However, overall erosion has been the dominant process, with net retreat rates from 1946 to 2020 being -0.97 m/yr. The DSAS produced similar LLR and EPR net erosion rates of over both the pre training wall (e.g. 1946-1982) and post wall (e.g. 1997-2020) periods, however due to the fluctuations between erosion and accretion periods giving poor R^2 values (< 0.5) for both periods, the EPR 's of -1.92 m/yr and -2.04 m/yr are considered more appropriate for extrapolation into the future.

As indicated in Table 4.1, for the northern sub-cell where there is evidence from the aerial photographs of direct influence of periods of southward offset Matau Mouth, there is insufficient transects with data to determine trends of the pre-training works period from the 1946 to1982. For the post-works period (1997-2020), net accretion over the 1997-2014 period, was largely wiped out by erosion at average rates of -8.7 m/yr over the recent 2014-2020 period to leave a small net shoreline advance averaging 8.5m across the transects in this sub-cell. This acceleration of erosion rates in the 2014-2020 period is similar to that experienced in the rest of central Molyneux Bay and is therefore not considered to be overly influenced by the southward mouth migration in 2017-2018. Although Table 4.1 notes total retreat of over 50m over the total 1946-2020 period, this is from only 8 transects that cover the total period, with the results being dominated by the 50m of erosion in the 2014-2020 period.

Based on an analysis of beach profile change from 2013-2020 for profile MB3, this area of shoreline has average beach cross sectional volume (e.g. above the msl contour) loss of -27 m^3/m per year associated with an average dune erosion rate of -19 m/yr,. Therefore, an average volume loss ratio of 10 $m^3/m/yr$ per 1 m of horizontal erosion can be applied to the erosion distance over the 23 year period (3.1 m/yr from Table 4.1) across the total length of shoreline between the Koau and Matau mouths (2700 m), to obtain an indicative net volume loss in the order of -83,500 m^3/yr from this section of shoreline to offshore and longshore transport processes. This volume is significantly higher than volume losses south of the Koau mouth as the higher dune elevations limits beach rollover processes along this section of shoreline, and therefore sediment is being lost to longshore and offshore transport, rather than being retained in the beach profile.

Figure 4.5: Matau mouth in 1982 (left) where the river mouth has breached the shoreline south of its present position, and 1997 (right) where the river mouth training structure has been put in place.

4.1.1.3 Northern Molyneux Bay - North of the Matau Mouth

The section of shoreline north of the Matau Mouth is covered by transects 139 – 220, with data being available as across pre and post training wall periods up to transect 180. As shown in Table 4.1, the historical shoreline trend for all sub-cells of the shoreline along this section of coastline has been of long term accretion, with rates in the range +0.6 to +0.8 m/yr, and Figure 4. below indicates that the rates of shoreline advance have generally been consistent over the total period. It is therefore considered that the shoreline movements along this section of coast are not influenced by river mouth orientation or migration, or any of the mouth training structures. It is also noted that the net rate over the 74 years are similar to the shoreline advance rates reported by Gibb (1978) for the 1847 to 1962 period.

Based on an analysis of beach profile change from 2013-2020 for profile MB4 and MB5, this area of shoreline has an average beach cross section (e.g above the MSL contour) volume gain of 1 to 9 m^3/m per year associated with an average upper beach accretion distance of 14 to 30 m/yr. Therefore, an average volume gain ratio of 0.2 to 5 m^3/m per 1 m of horizontal shoreline advance can be applied to the total accretion distance over the 23 year period (+0.4 to +0.9 m/yr from Table 4.1) across the total length of shoreline north of the Matau mouth (4000 m) to give an indictive a net volume gain in the order of +2,600 m^3/yr. Whilst the northern area of the bay has a net positive supply of sediment, this accretion volume is considerably less than volume of sediment being lost from the southern and central areas of the bay, indicating that across the whole Molyneux Bay area, there is a net loss of sediment to longshore and offshore sediment transport. These findings are consistent with the sediment transport modelling presented by Atkin et al (2020) in Figure 2.12.

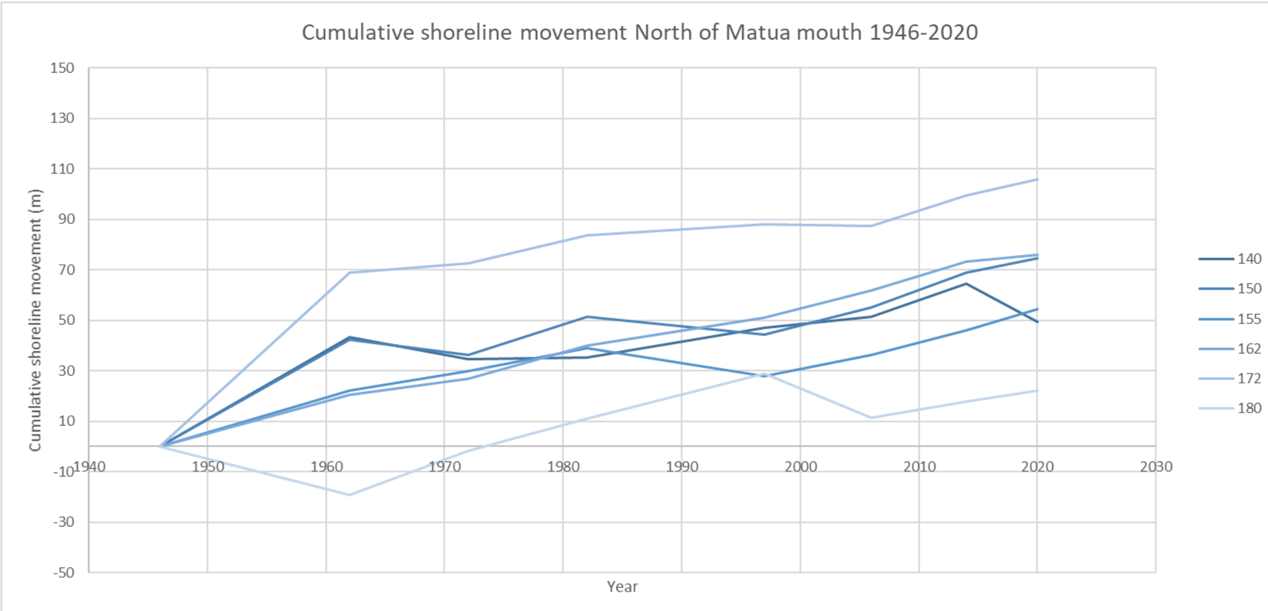


Figure 4.6: Cumulative shoreline movement of selected transects north of the Matau Mouth over the 74 year period of 1946-2020.

4.1.2 Extrapolation of Long-Term Rates

As set out in Table 3.3 (Section 3.3.3.), the significant change in erosion rates south of the Koau Mouth in the post training wall period, which the presence of the southern wall is considered to have a substantial influence on, are taken into account in the extrapolation of historical shoreline movements for this section of Molyneux Bay in the following scenarios:

- For the 30 years to 2050, the wall is assumed to remain in place, and the extrapolation is at the higher 1997-2020 post wall rates.

- For the 20 years between 2050 and 2070, one scenario of continued extrapolation at the higher 1997-2020 rates under the assumption that the wall continues to be maintained and in its current position, and a second scenario of the wall being removed and erosion assumed to slow and return to the 1946-1982 pre-wall rates. Note for the transects 1-40, the erosion rate used in the extrapolation is the EPR rather than the LLR rate due to the poor R^2 values from the linear regression.

It is noted that these rates for the 1997-2020 may be conservatively high due to lack of significant flood events over the majority of this period, which may not be repeated in future periods, however any lagged effect of the Roxburgh Dam on sediment supply will continue into the future. It is also considered that the extrapolated shoreline positions along this section of coastline are likely to be conservative as the predictions did not take into account that the beach is rolling back into wetlands and the drainage channels of the Puerua Diversion, therefore beach crest elevations are likely to reduce as the weight of beach sediment compresses the soft sediments in the water bodies. Such crest lowering associated with long-term erosion from overtopping of the sand and gravel barrier at Washdyke Lagoon, Timaru is well documented. This factor is considered to be accounted for in the uncertainty range for the extrapolations.

For the remainder of Molyneux Bay shoreline north of the Koau Mouth, there was minimal change between rates pre and post-wall construction, therefore rates from the total 1946 - 2020 period were considered appropriate, including the area immediately north of the Koau Mouth directly influenced by mouth migration prior to the construction of the training walls.

The resulting extrapolated erosion distances for the 2050 and 2070 are presented below in Table 4.. These erosion distances are purely for the extrapolation of the most appropriate historical rates of shoreline movement, and do not take into account the effects of SLR into the future.

The extrapolation statistics presented in Table 4.2 include:

- The 'Mean', being the average of the mean extrapolated erosion distances from all transect within the sub-cell;
- The 'Max at an Individual Transect', being the maximum retreat (or minimum advance) of all the transect 'means' within the shoreline section; and
- The 'Average Upper 95% Confidence Level' being the average of all upper 95% confidence levels for this defined group of transects, demonstrating the average uncertainty of the long-term extrapolation within the section of shoreline.

The results are presented in Table 4.2 as averages across DSAS transects relating to the eleven profile transects extracted from the 2020 LiDAR surveys (e.g. P1, P2 P3 etc.).

Table 4.2: Extrapolation of long-term erosion rates and resulting projected erosion distances at Molyneux Bay, with 2070 scenarios accounting for the presence and removal of the southern training wall.

Shoreline Area	DSAS Transects (profile)	Statistic	Erosion rate extrapolated to 2050 with wall maintained (m)	Erosion rate extrapolated to 2070 with wall maintained (m)	Erosion rate extrapolated to 2070 without wall maintained (m)
Southern Molyneux Bay: South of Koau Mouth	**1 – 20 (P1)**	Mean	-291	-492	-314
		Max erosion/ min accretion at individual transect	-332	-553	-340
		Avg Upper 95% Confidence Level	-509	-849	-713
	21 – 40 (P2)	Mean	-274	-456	-302
		Max erosion/ min accretion at individual transect	-332	-554	-338
		Avg Upper 95% Confidence Level	-424	-707	-650
	41 – 60 (P3)	Mean	-194	-323	-283
		Max erosion/ min accretion at individual transect	-214	-357	-313
		Avg Upper 95% Confidence Level	-264	-439	-460
	61 – 78 (P4)	Mean	-199	-332	-375
		Max erosion/ min accretion at individual transect	-219	-365	-432
		Avg Upper 95% Confidence Level	-371	-618	-1507

Shoreline Area	DSAS Transects (profile)	Statistic	Erosion rate extrapolated to 2050 with wall maintained (m)	Erosion rate extrapolated to 2070 with wall maintained (m)	Erosion rate extrapolated to 2070 without wall maintained (m)
Central Molyneux Bay: Koau Mouth to Matau Mouth	**79 – 96 (P5)**	Mean	-46	-77	
		Max erosion/ min accretion at individual transect	-52	-87	
		Avg Upper 95% Confidence Level	-105	-174	
	97 – 115 (P6)	Mean	-34	-56	
		Max erosion/ min accretion at individual transect	-51	-85	
		Avg Upper 95% Confidence Level	-67	-111	
	116-135 (P7)	Mean	-3	-5	
		Max erosion/ min accretion at individual transect	-15	-26	
		Avg Upper 95% Confidence Level	-38	-64	
Northern Molyneux Bay: North of Matau Mouth	**139 – 160 (P8)**	Mean	+21	+35	
		Max erosion/ min accretion at individual transect	+17	+29	
		Avg Upper 95% Confidence Level	+9	+15	
	161 – 180 (P9)	Mean	+25	+41	
		Max erosion/ min accretion at individual transect	+12	+20	
		Avg Upper 95% Confidence Level	+16	+27	
	181 – 200 (P10)	Mean	+13	+22	
		Max erosion/ min accretion at individual transect	+7	+12	
		Avg Upper 95% Confidence Level	+2	+3	
	201 – 221 (P11)	Mean	+22	+36	
		Max erosion/ min accretion at individual transect	+16	+26	
		Avg Upper 95% Confidence Level	+4	+7	

4.1.2.1 Southern Molyneux Bay - South of the Koau Mouth

The extrapolations of the long-term historical rates south of the Koau Mouth show large erosion distances with high degrees of uncertainty. Averaged extrapolated erosion distances over the next 30 years range between 200-300m, with the greatest erosion expected at the very southern limit of Molyneux Bay (-332 m). The results of this analysis indicate that there is high uncertainty in the extrapolation over a 30-year period, with a 5% chance that the shoreline could retreat an additional 50-250 m than the 'most likely' shoreline position.

Over the next 50 years, under the assumption that the southern training wall is maintained in its current position, erosion distances increase to range between 325-500m. There is also high uncertainty in the extrapolations of these rates over the next 50 years, where there is a 5% chance that the shoreline could be between 110-350m landward of the 'most likely' position, with the greatest uncertainty occurring on across transects 1-20 at the southern limit of the bay.

Over the next 50 years, under the assumption that the wall is removed in 2050, erosion distances are less at the southern end between transects 1-40 by in the order of 150-180m compared to maintaining the wall. Across transects 41-60, on average erosion distances are 50m less with the wall removed, however over transects 61-78, the erosion distances increase with the wall removed to be in the order of 40m greater than when the wall is maintained. The uncertainty around this scenario of the wall being removed is greater than the scenario when the wall has been maintained. Over transects 1-40, there is a 5% chance that the shoreline could be landward an additional 350-400m than the 'most likely' shoreline position. At transects 61-78, there is very high uncertainty around the position of the shoreline under this scenario, where the shoreline could be up to 1.1km landward of the 'most likely' shoreline position, therefore in this area we have very low confidence in predicting the range of shoreline over the next 50 years if the southern training wall is removed.

4.1.2.2 Central Molyneux Bay - Koau Mouth to Matau Mouth

For the section of shoreline between the Koau Mouth and the Matau Mouth, the extrapolated future erosion rates are significantly less than those south of the Koau Mouth, and there is greater certainty around the predicted extrapolation into the future.

From transects 79-96, which experienced periodic mouth migration until the construction of the northern training wall in the 1970's, the extrapolated erosion distances are the highest within the cell, and the degree of uncertainty the greatest. Over 30 years, the extrapolated erosion distances for this sub-cell are on average -46m, with a 5% chance that the shoreline could retreat an additional 60m from the 'most likely' shoreline position. For the 50-year scenarios (same under both scenarios of southern wall retained and removed), the average 'most likely' extrapolated erosion distances are -87m, with a 5% chance that there could be up to an addition 100m of retreat.

Extrapolated erosion distances decrease in a northward direction with transects 97-115 on average most likely to erode -34m over a 30-year period, with a 5% chance that the shoreline could erode landward an additional -35m. For the 50-year scenario, the extrapolated erosion distance for this sub cell is on average -56m landward of the current shoreline position, with a 5% chance that the shoreline could erode an additional 55m.

For the northern sub-cell (transects 116 to 135), the average extrapolated most likely erosion is -3m, decreasing from -16m at transect 116 to accretion of +22m on the south side of the Matau Mouth , and a 5% chance that the shoreline could erode landward an additional -35m. For the 50 year scenario, the average extrapolated erosion distance is only -5m with the range being from -26m at transect 116 to +21m at transect 135, and a 5% chance that the shoreline could retreat an additional -60m from the 'most likely' shoreline position.

4.1.2.3 North of the Koau Mouth

For the section of shoreline north of the Matau Mouth (Transects 139-207), the long term historical accretionary rates mean that when extrapolated into the future there is continued accretion of the shoreline. On average, this shoreline is most likely to accrete on average between 13 to 25m over the next 30 years, and 22 to 41m over the next 50 years. There is some uncertainty around the extrapolation of these rates, however the results show that it is very unlikely that this section of shoreline will turn erosional. There is a 5% chance that this shoreline would accrete 9-18m less over the next 30 years, and between 19-29m less over the next 50 years.

4.2 Erosion Effect of Accelerated Sea Level Rise

The effect of SLR was calculated using the modified Bruun rule for overtopping on sand beaches. A summary of the mean and upper 95% confidence level for predicted erosion distances under the SLR scenarios for 2050 and 2070 averaged across the transects relating to each of the eleven profile transects extracted from the 2020 LiDAR surveys is presented below in Table 4..

Table 4.3: Predicted erosion from future SLR at the eleven profile transects located along the Molyneux Bay shoreline.

Transect (Profile)	Statistic	Erosion Distance 0.2m SLR by 2050	Erosion Distance 0.4m SLR by 2070	Erosion Distance 0.6m SLR by 2070
1 – 20 (P1)	Mean	**-26.4 m**	**-51.4 m**	**-76.4 m**
	Upper 95% Level	-28.8 m	-55.7 m	-82.6 m
21 – 40 (P2)	Mean	**-25.8 m**	**-50.2 m**	**-74.5 m**
	Upper 95% Level	-27.9 m	-54.1 m	-80.2 m
41-60 (P3)	Mean	**-24.8 m**	**-48.9 m**	**-73.1 m**
	Upper 95% Level	-27.1 m	-53.4 m	-79.8 m
61-78 (P4)	Mean	**-23.8 m**	**-47 m**	**-70.2 m**
	Upper 95% Level	-25.9 m	-51.1 m	-76.3 m
79-96 (P5)	Mean	**-18.2 m**	**-33.8 m**	**-49.4 m**
	Upper 95% Level	-21.3 m	-39.5 m	-57.6 m
97-115 (P6)	Mean	**-16.7 m**	**-31 m**	**-45.4 m**
	Upper 95% Level	-19.2 m	-35.5 m	-51.9 m
116-136 (P7)	Mean	**-17.8 m**	**-33.2 m**	**-48.5 m**
	Upper 95% Level	-20.9 m	-38.6 m	-56.3 m
139-160 (P8)	Mean	**-12.1 m**	**-24.3 m**	**-36.4 m**
	Upper 95% Level	-12.6 m	-25.1 m	-37.7 m
161-180 (P9)	Mean	**-11.6 m**	**-23.3 m**	**-34.9 m**
	Upper 95% Level	-12 m	-24 m	-35.9 m
181-200 (P10)	Mean	**-11.8 m**	**-23.6 m**	**-35.3 m**
	Upper 95% Level	-12.2 m	-24.3 m	-36.5 m
201-221 (P11)	Mean	**-15.7 m**	**-31.5 m**	**-47.2 m**
	Upper 95% Level	-16.7 m	-33.3 m	-50 m

The results indicate that the effect of the SLR will be greater at the southern end of Molyneux Bay than the north as a result of the flatter nearshore slopes of the southern transects (P1, P2 and P3) compared to the slightly steeper profiles at the northern end of Molyneux Bay (P8 to P11).

For all the sites, the effect of SLR over the next 30 years with 0.2m of SLR is likely to be in the range of -11 to -26m, with lower erosion distances tending to be in the northern transects of the bay (-11 to -16m).

Over a 50 year period with 0.4m of SLR, the range of effect from SLR increases with transects south of the Koau Mouth likely to experience in the order of -50m of erosion from the impact of SLR, while transects north of the Koau Mouth range from -24 to -34m of likely erosion. Over a 50-year period with 0.6m of SLR, the erosion distances increase by around 50%. Transects at the south of the bay (P1 to P4) are predicted to experience erosion in the order of -70 to -75m, while transects at the northern end of the bay (P8-P11) are predicted to experience erosion in the range of -35 to -50m.

The uncertainty around the effect of SLR are calculated to be less than 7m across all profile transects and sea level rise scenarios. However, this low uncertainty is a result of the lack of data available in Molyneux Bay to determine extremes for input into the Bruun Rule model, and in reality, the uncertainties are likely to be much higher. For example, the use of the maximum closure depths from Atkin et al (2020b) would increase the uncertainty.

4.3 Short Term Storm Effects

As set out in Section 3.1, the SBEACH-32 numerical simulation model was used to give an indication of beach response in a large storm event. The resulting beach crest erosion distances from the modelling of simulated 1 in 50, 1 in 100, and 1 in 200 year ARI events on the five surveyed beach profiles (MB1-MB5) are presented in Table 4.4 and morphological responses for each beach cell are further discussed below. Due to the limitations of the modelling and the input data, these results are considered to be a conservative indicative estimate of how the beach morphology could change in these large events.

Table 4.4: Indicative beach crest erosion from SBEACH modelling of large storm events.

	Storm		
Profile	**1 in 50 year ARI**	**1 in 100 year ARI**	**1 in 200 year ARI**
MB1	-15 m	-20 m	-24 m
MB2	-21 m	-22 m	-24 m
MB3	-14 m	-35 m	-35 m
MB4	-13 m	-15 m	-17 m
MB5	-2 m	-2 m	-2 m

4.3.1.1 South of the Koau Mouth (MB1 and MB2)

The results of the SBeach storm simulation of profiles MB1 and MB2 showed a crest retreat in the range of 15m to 24m from wave overtopping and sediment rollover processes in the extreme storm events simulated. The upper beach (around the 4-5m contour) narrowed on both profiles, however, neither indicated a significant reduction in crest elevation as a result of overtopping, or the deposition of rollover volumes into the water bodies and wetlands located behind the beach profiles. These are processes observed and likely to occur during overtopping events, hence is considered to be under-represented in the model responses. As noted above, crest lowering associated with long-term erosion from overtopping of the sand and gravel barrier at Washdyke Lagoon, Timaru, is well documented and a similar response is considered to most likely be occurring at southern Molyneux Bay. As a result, the erosional response to storm events are likely to increase in the future.

4.3.1.2 Koau Mouth to Matau Mouth (MB3)

The results of the SBeach simulation on profile MB3, located between the Koau and the Matau Mouths indicated that in a 1 in 50-year storm there would be steepening and narrowing of the foredune crest to an elevation of around 5.5m (DVD). There would only be a small amount of overtopping over washing sediment, and the crest would retreat in the order of 15m. In the larger simulated events (1 in 100 year, 1 in 200 year events), it shows the foredune being overtopped, resulting in the landward retreat in the order of 35m, however the elevation of the secondary dune remains around 7m (DVD), indicating that the higher landward dune structure is not likely to be affected by these extreme events with the current foredune profile, and is likely to continue to provide good protection for the hinterland behind (including the Inch Clutha Diversion Channel) should the current foredune be lost to long-term erosion. However, as with the southern sites, the simulation is considered to underrepresent rollover volumes.

4.3.1.3 North of the Matau Mouth (MB4 and MB5)

Beach profiles MB4 and MB5, north of the Matau Mouth, have crest elevations around 6m (DVD) and are backed by high cliffs (around 25m). The results from the SBeach simulation indicate that the beach crest elevations are unlikely to be overtopped in the storm events modelled, however sediment on the front of the beach profile will be removed seaward into the nearshore, resulting in a flattening of both foreshore and nearshore slope. At profile MB4, this results in an upper beach scarp forming around the 4m contour, and retreat of the beach crest in the order of 13 to 17m for all simulated storm events. At profile MB5, the foreshore slope is flattened with the removal of a small foreshore berm. Upper beach erosion is small in the order of -2m in all three simulated storm events.

4.4 Projected Future Shoreline Scenarios

The calculated PFSPs averaged over the length of coast represented by profiles P1 to P11 are presented below in Table 4.5, and are presented spatially in Appendix D, with the P50 position representing the 'most likely' position and the 'P5' representing the 'very unlikely' landward position under 0.2m, 0.4m and 0.6m of SLR over 30 and 50 year periods. It is noted that there is also a corresponding very unlikely 5% erosion position for which the erosion distances will be the same magnitude less than the 'most likely' position. However, these potential 'best case' positions are not plotted in the Appendix D maps and are not discussed in this report.

The 50-year scenarios represent two increments of SLR (0.4 and 0.6 m) as well as accounting for the future of the southern training wall, with both the removal and maintenance of the wall assessed under both SLR increments.

Table 4.5: Calculated average erosion distances (m) to the 'most likely' (P50) and 'very unlikely' (P5) PFSP at Molyneux Bay.

Shoreline Area	Transect (Profile)	PFSP 2050 (0.2m SLR)		PFSP 2070 (0.4m SLR) with wall maintained		PFSP 2070 (0.4m SLR) without wall maintained		PFSP 2070 (0.6m SLR) with wall maintained		PFSP 2070 (0.6m SLR) without wall maintained	
		P50	P5	P50	P5	P50	P5	P50	P5	P50	P5
	1-20 (P1)	-319.8	-467.5	-543.3	-788.1	-366.1	-638.2	-568.6	-812.5	-390.2	-663.7
	21-40 (P2)	-299.6	-402.5	-506.7	-677.3	-352.7	-591.0	-531.2	-702.3	-377.1	-614.8

South of the Koau mouth	**41-60 (P3)**	-218.7	-266.4	-372.3	-451.7	-332.4	-453.7	-396.3	-476.1	-356.8	-477.8
	61-78 (P4)	-223.1	-340.4	-379.8	-575.0	-424.4	-1198.7	-402.9	-598.4	-448.5	-1220.9
Koau mouth to Matau mouth	**79-96 (P5)**	-64.6	-104.6	-111.3	-177.8	-111.3	-177.8	-127.2	-194.1	-127.2	-194.1
	97-115 (P6)	-50.7	-73.4	-87.9	-125.6	-87.9	-125.6	-102.4	-140.1	-102.4	-140.1
	116-136 (P7)	-21.0	-45.0	-38.5	-78.4	-38.5	-78.4	-53.8	-93.9	-53.8	-93.9
North of the Matau mouth	**139-160 (P8)**	+8.9	+0.8	+10.8	-2.7	+10.8	-2.7	-1.0	-14.6	-1.0	-14.6
	161-180 (P9)	+13.3	+7.3	+18.3	+8.3	+18.3	+8.3	+6.8	-3.2	+6.8	-3.2
	181-200 (P10)	+0.9	-6.8	-2.5	-15.3	-2.5	-15.3	-14.6	-27.4	-14.6	-27.4
	201-221 (P11)	+5.8	-6.2	+4.3	-15.6	+4.3	-15.6	-11.6	-31.5	-11.6	-31.5

4.4.1 30-Year (2050) Projected Future Shoreline Position

4.4.1.1 Southern Molyneux Bay - South of the Koau Mouth

The PFSP in 2050 with 0.2 m of SLR for the area of shoreline south of the Koau Mouth shows that the greatest amount of future erosion is predicted to be at the southern limit of Molyneux Bay (transects 1-20), where the shoreline is 'most likely' to be -320m from the present day shoreline, with a 5% chance that the shoreline could be a further 150m landward from this position. The predicted erosion distances decrease in a northward direction, with retreat distances to the PFSP at transects 21-40 being in the order of -300m and transects 41-78 being in the order of -220m from current position. As indicated from the earlier results, 85-90% of the predicted erosion is due to the extrapolation of contemporary erosion rates since the construction of the southern training wall. There is also reduced uncertainty around the PFSP in a northward direction, with a 5% chance that the shoreline could be 100m further landward across transects 21-40; and 45m further landward across transects 41-60. The uncertainty then increases at transects approaching the Koau Mouth, being -120m to the 5% probability position across transects 61-78.

The results indicate that over the next 30 years, the retreat of the shoreline is likely to have a significant effect on the exposure of the Koau Mouth southern training wall and on the drainage capacity of the Puerua Diversion channel, plus some potential impacts of the flood stopbanks along this channel. These effects are further examined in Section 4.5.1.

4.4.1.2 Central Molyneux Bay - Koau Mouth to Matau Mouth

For the area of shoreline between the Koau Mouth and the Matau Mouth, the PFSP distances continue to reduce in a northward position. For the southern sub-cell (transects 79-96), the most likely PFSP is on average -65m landward of the present-day shoreline, with a 5% chance that erosion distances could be an additional 40m landward of this position. As shown in Appendix D, this reduced erosion results in a discontinuity of shoreline position across the Koau Mouth, which is a factor of the training wall structures.

In the middle of the cell (transects 97-115), the most likely PFSP is on average -51m landward of the present day shoreline, reducing to -21m for transects 116-135, which would put the shoreline landward of the Matau

Mouth Training Wall. For both of these sub-cells there is a 5% chance that the PFSP could be an additional 25m landward of the 'most likely' position.

4.4.1.3 Northern Molyneux Bay - North of the Matau Mouth

Due to the extrapolation of the contemporary shoreline change, the 30-year PFSP for the area of shoreline north of the Matau Mouth is 'most likely' to be accretionary, with the shoreline likely to be in a similar or more seaward position than the present day shoreline. On average, across transects 139-160 immediately north of the Matau Mouth, the 30-year shoreline is 'most likely' to be 9m seaward of its current position; reducing to 13m seaward across transects 161-180, and 1m seaward across transects 181-200. The uncertainty along this cell is lower than the shoreline in the central and the southern of the bay, with a 5% chance that the shoreline will accrete 6-8m less than its 'most likely' shoreline position across transects 139-180, hence shoreline stability. For transects the most northern sub cell (transects 181-200), there is a 5% chance that the shoreline will erode in the order of up to -7m over the next 30 years, changing its prediction from being stable/accretionary to slowly eroding.

4.4.2 30 Year PFSP – Log Spiral Assessment

As shown in Figure 4.7, apart from the very southern limit of the beach, the current shoreline of southern Molyneux Bay conforms well to a log-spiral shape in the lee of a headland, therefore the transition of this plan shape over a 30 year period to 2050 can be used to potentially validate the results of the PFSP predictions.

Figure 4.7: Theoretical log-spiral plan shape fitting for southern Molyneux Bay compared to calculated PFSP over a 30-year period to 2050.

The results, as also shown in Figure 4.7 indicate that the predicted PFSP over the 30-year period are likely to be conservatively large at the southern end of Molyneux Bay (e.g. transect 1-15) by a maximum of around 200m, crossing over to under-predict erosion north of transect 28, with the under-prediction being in the order of 200m close to the Koau Mouth. It is noted that these differences in the PFSP from the predicted log-spiral position are greater than the 5% uncertainty at either end of this beach cell.

4.4.3 50-Year (2070) Projected Future Shoreline Position

4.4.3.1 Southern Molyneux Bay - South of the Koau Mouth

The projected future shoreline position for the shoreline cell south of the Koau Mouth is heavily influenced by the future presence of the southern training wall (i.e. the extrapolation of the long term rate) with less sensitivity around the increase in SLR in the various scenarios.

As seen in the results presented in Table 4.5, the average difference between the two SLR scenarios is in the order of 15-25m, indicating that future shoreline position is not as sensitive to the increase in SLR as it is to the extrapolation of the long term rate. Of the total PFSP, the effect of SLR contributes 10-15% of the total erosion distance in the 0.4m SLR scenario, which increases to 15-20% for the 0.6m SLR scenario. In both SLR scenarios where the wall is maintained, erosion distances are greatest in the southern end of the cell, in the order of -540 to -570m, and decrease in a northward direction to -380 to -400m south of the southern training wall. There is a 5% chance that erosion distances could an additional 245m at the southern end of the cell, and 195m at the northern end of the cell.

In the two SLR scenarios where the wall has been removed in 2050, and pre-wall construction rates have been extrapolated from 2050, PFSP erosion distances reduce everywhere relative to the scenario where the wall is maintained, except for across transects 61-78 immediately south of the training wall. At the southern end of the cell (transects 1-40) erosion distances decrease to being in the order of -350 to -390m and are in the order of -330-360m across transects 41-60. These distances increase to -425 to -450m south of the southern training wall (transects 61-78). This increase is likely a response of the higher erosion rate at this section of coastline due to breaching of the river mouth to the south when it was not restricted by the presence of the training wall, in which these rates are accounted for in the pre-construction historical rates. There is a 5% chance that erosion distances could be an additional 240-270 m at the southern end of the cell (transects 1-40), and 120m across transects 41-60. There is large uncertainty around the PFSP immediately south of the training wall (transects 61-78), where there is a 5% chance erosion distances could be an additional 775m.

The above results indicate that by removing the wall, this could reduce erosion distances by the order of 150 to 180m at the southern end of the cell (transects 1-40), and by 40m across transects 41-60. However, the removal of the wall is predicted to increase the erosion distance across transects immediately south of the training wall by approximately 45m.

4.4.3.2 Central Molyneux Bay - Koau Mouth to Matau Mouth

For the section of shoreline north of the Koau Mouth, the PFSP distances in both scenarios are the same, as the extrapolation of the historical long term rate was taken from the total record given that on average there was no difference between pre and post wall construction.

As seen in the results presented in Table 4.5, the average erosion difference between the two SLR scenarios is in the order of 15m. PFSP erosion distances in the central cell decrease in a northward direction. As presented in

Table 4.5, across both SLR scenarios, erosion distances at the southern end of the cell (transects 79-96) are in the order of 111m to 127m, with a 5% chance that erosion distances could be an addition 67m. Across the centre of the cell (transects 97-115) PFSP erosion distances are in the order of 88 to 102m, with a 5% chance that erosion distances could be an additional 38m. At the northern end of the cell (transects 116-136), PFSP erosion distances are 38 to 54m, with a 5% chance that erosion distance could be an additional 40m.

The relative contribution of the effect of SLR with the PFSP erosion distance increases in a northward trend along the shoreline of the central cell. Across cells immediately north of the Koau mouth (transects 79-96), the relative contribution of SLR is 25-40% over both scenarios. This increases to 35-50% over both scenarios for transects 97-115. The SLR contribution is greatest across transects 116-136 where it makes up 78% of the PFSP erosion distance in the 0.4m SLR scenario, increasing to 92% with 0.6m SLR. The northward trending increase in SLR contribution is a result of the corresponding decrease in the extrapolation of the historical erosion rate in the same direction.

4.4.3.3 Northern Molyneux Bay - North of the Matau Mouth

For the section of shoreline north of the Matau Mouth, the PFSP distances in both scenarios are the same, as the extrapolation of the historical long term rate was taken from the total record given that on average there was no difference between pre and post wall construction.

As seen in the results presented in Table 4.5, the average difference between the two SLR scenarios is in the order of 11-16m. Due to the historical accretionary trend along this section of shoreline, PFSP distances that are landward of the present day shoreline are completely dependent on the effect of SLR, in which the estimated effect of SLR has overturned the beach from being accretionary to erosional. Where PFSP are seaward of the present-day shoreline, the effect of SLR will reduce the rate at which the beach is accreting in the future.

Along the northern cell, most areas of shoreline continue to accrete with 0.4 m of SLR, but overturn to being erosional with 0.6 m of SLR. At the southern end (transects 139-160), PFSP distances are +10.8m with 0.4m SLR, and -1m with 0.6m SLR, with a 5% chance that there could be an additional 14m of erosion. Across transects 161-180, under both SLR scenarios the PFSP distances are 7-18m of accretion, with a 5% chance that there could be an additional 10m of erosion, only overturning the shoreline to being erosional in the 0.6m SLR scenario, but remaining accretionary for the 0.4m SLR scenario. Across transects 181-200, the PFSP distances are erosional, with -2 to -15m of erosion on average over both SLR scenarios, with a 5% chance that erosion distances could be an additional 13m. Across transects 201-211, the PFSP distances are on average accretional (+4m) with 0.4m of SLR, and erosional (-12m) with 0.6m of SLR, with a 5% chance that there could be an additional 20m of erosion.

4.5 Effects of Projected Future Shoreline Positions on LCFPDS

4.5.1 Southern Molyneux Bay - South of Koau Mouth

As shown in the Appendix D maps, the PFSP in 30 years will have implications on the southern training wall structure, with the back of the beach most likely to be located at the very landward extent of the present section of wall perpendicular to the coast, implying that around 150-170m of the wall structure could be exposed to direct wave energy if the wall remains in its current form and position. The Puerua Diversion outlet within the wall would also be exposed, and the drainage along the diversion channel would be blocked by the beach adjacent to the outlet and compromised further south. A 500m section of flood stopbanks at the end of Port Molyneux Road is likely to be compromised by the shoreline retreat over this time period, with a 5% chance that the whole length of banks along the Puerua Diversion are similarly affected by.

In the 50-year scenarios, the impacts on the LCFPDS infrastructure is the same under both SLR scenarios with and without the training wall removal. The predicted shoreline is landward of the Puerua River Diversion channel and stopbanks over their total length from Kaka Point, which will cause substantial drainage and flood protection issues on the southern part of the delta. Under all scenarios, the southern training wall is predicted to be outflanked from behind, with the shoreline predicted to most likely be located on the main river channel landward of the wall.

4.5.2 Central Molyneux Bay – Koau Mouth to Matau Mouth

The 30-year PFSP is located very close to the position of the Matau River training wall, indicating that the beach may roll over the top of this structure exposing the front to damage from open coast processes (e.g. wave attack). Along the remainder of the cell the PFSP does not impact any floodbank structures along the Inch Clutha diversion channel, however, the predicted shoreline positions are located within the current high back dune environment, indicating that the ability of these dunes to provide protection against coastal inundation could be compromised within this time period.

Over the 50-year scenarios, the PFSP is shown in Appendix D to be landward of the Matau Mouth training line, further exposing the structure to open coast erosion processes. For the very unlikely 5% PFSP, the drainage capacity of the northern end of the Inch Clutha Diversion Channel would also be compromised. At the southern end of the cell, the PFSP is located along the river stopbank rather than the northern training wall, but well seaward (e.g. greater than 500m) of the Inch Clutha channel outlet structure. Over the rest of the cell, the PFSP indicates that the current high back dunes are lost to erosion over this time period, suggesting that the vulnerability to coastal inundation along this section of coast would increase.

4.5.3 Northern Molyneux Bay – north of Matau Mouth

For this area, the only potential effect of the PFSP on LCFPDS infrastructure for this cell is the potential erosion of the short rock groyne located at the base of the raised terrace on the north side of the mouth. This could occur within the 30-50-year timeframe.

4.6 Coastline Management Options Assessment

4.6.1 Management Options

Table 4.6 below is a high-level assessment of possible coastline management options that ORC could employ to address the coastal erosion issues and/or consequences for the LCFPDS infrastructure identified in this report as likely to occur over the next 30-50 years. The assessment includes consideration of the strengths and weaknesses of each management option.

Table 4.6: Potential coastline management options for Koau River Mouth erosion hazard mitigation and asset management.

Option	Strength	Weakness/ Consequences
Do nothing	• No additional costs.	• Existing training wall maintenance costs likely to increase. • Ongoing shoreline erosion with the shortest time to total loss of training wall function and compromise of Puerua Diversion drainage.

Option	Strength	Weakness/ Consequences
Back beach push up (sand sourced from back dunes)	• Slows shoreline recession by adding sediment from back dunes to active beach system. • Buys time for decision on training wall future. • Incremental implementation possible. • Increased protection against coastal inundation (can mitigate wave over wash). • Slows loss of drainage capacity of Puerua Diversion. • Low additional cost.	• Does not address sediment supply deficit or sea level rise (SLR) impacts and therefore does not provide long-term solution. • Limited service life (probably 10-20 years; dependent on volume of sand that can be sourced from back dunes). • Does not directly address maintenance requirements on training wall.
Sediment bypass to south of Koau River Mouth.	• Slows shoreline recession along shoreline between Kaka Point and Koau River Mouth. • Buys time for decision on training wall future. • Slows loss of drainage capacity of Puerua Diversion.	• Increases shoreline recession along shoreline between Koau and Matau River entrances. • Does not address sediment supply deficit or SLR impacts and therefore does not provide long-term solution. • Not directly address maintenance requirements on training wall. • Substantial ongoing cost.
Beach nourishment (sand sourced from storage on the nearshore sand wedge)	• Offsets shoreline recession by adding sediment from the nearshore wedge. • Long-term, adaptive solution. • Buys time for decision on training wall future. • Slows loss of drainage capacity of Puerua Diversion.	• For best result will also require sand placement on the upper beach to increase crest elevation hence reduce erosion by rollover. • Will require ongoing works (cost will increase over time) • Does not directly address maintenance requirements on training wall. • High capital and ongoing cost.
Relocate Puerua Diversion channel landward	• Ensures drainage from this catchment is not compromised by shoreline recession processes. • Reduces likelihood of dune breaching during fluvial flood events.	• Potential land ownership issues. • Does not address coastal erosion issues including loss of training wall function. • High cost.
Create new mouth for Puerua River	• Could improve flood drainage from this catchment. • Ensures drainage from this catchment is not compromised by shoreline recession processes. • Reduces likelihood of dune breaching during fluvial flood events.	• Effectiveness of mouth training structure uncertain. • Does not address coastal erosion issues including loss of training wall function. • High cost as would require new training structures to maintain open outfall to ocean.
Progressive shortening of Koau Training Walls as south coast erodes back	• Reduces wall maintenance costs. • Reduces shoreline recession rate to south of Koau River Mouth as groyne effect on longshore sediment transport past entrance is reduced.	• Progressively higher risk of outflanking and channel migration, leading to entrance instabilities and associated erosion episodes, increased catchment flooding and possible water quality issues. • Increases shoreline recession along shoreline between Koau and Matau River entrances.
Realignment of Koau Training Walls to allow more sediment supply to south	• Potential retains effectiveness in preventing mouth migration and coastal inundation. • Reduces wall maintenance costs. • Reduces shoreline recession rate to south of Koau River Mouth as groyne.	• Increases shoreline recession along shoreline between Koau and Matau River entrances. • High capital cost.

4.6.2 Recommended Investigations to Determine Feasibility of the Options

All of the above management options require some combination of coastal process, entrance dynamics, and/or flood impacts investigations to develop and evaluate the options to the detail required to make informed decisions on the feasibility of the options going forward. The following Table (4.7) outlines the range of investigations required for each of the options, with further details on the investigations being presented below the Table.

Some of the options would also require data from ongoing monitoring of beach morphology and position to track sediment volumes and trends in shoreline movements to determine when trigger conditions for action are met (e.g. beach push -up, sand by-passing, renourishment), or to monitor the effectiveness of infrastructure options (shortening or re-aligning Koau Mouth Training Walls). Therefore, this type of monitoring is considered a necessary requirement regardless of the management option employed, with recommended monitoring practices provided in Section 6.2.

Table 4.7: Recommended investigations for each coastline management option

	Management Options						
Investigations Required	**Beach Push-Ups**	**Sediment Bypass from North of Koau Mouth**	**Beach Renourish-ment**	**Relocation Puerua Diversion Channel**	**Create new Mouth for Puerua River**	**Shortening of the Koau Training walls**	**Realignment of the Koau Training Walls**
Establish wave, wind and sea water levels conditions at the site.	✓	✓	✓		✓	✓	✓
Storm erosion modelling and wave overtopping assessment to determine a design dune profile for avoidance of excessive wave overtopping and breaching of the dunes (e.g. XBeach modelling).	✓	✓	✓				
Littoral transport study to assess the longshore transport regime along the embayment.	✓	✓	✓		✓	✓	✓
Assessment of the aeolian (windblown) sediment transport regime of the upper beach and its potential influence on the stability of the dune system	✓	✓	✓				
Coastal evolution modelling to assess the influence of sand bypassing/ renourishment		✓	✓			✓	✓

operations and training walls on littoral transport regime and shoreline position.							
Flood impacts study to assess the effects of channel relocation and /or river mouth configuration on flood behaviour throughout the lower Clutha River system.				✓	✓	✓	✓
Entrance dynamics study to estimate the likely requirements to maintain an entrance.					✓	✓	✓

Future details of the possible investigations' packages are outlined below:

- Wave, wind and sea water levels conditions: This will involve using 40 year hindcast deep water wave climates from regional/national models (e.g. NOAA WAVEWATCH III) as the boundary input conditions into wave SWAN modelling to establish a time series of nearshore waves and extreme nearshore wave statistics at various locations along the embayment. More detailed bathymetry than is currently available for shoreward of the -7 m contour will be required for the nearshore wave modelling. Calibration of the wave modelling is difficult due to the absence of wave bouy measurements in the area for the period covered by the wave hindcasts. The 40 year hindcast meteorological data can be used to model storm surge and extreme storm tide water levels, using data from the Green Island sea level recorder for calibration. Joint probability of the wave and storm tide can then be undertaken using appropriate methods (e.g RJPM from Tawn and Vassie, 1991) and software (JOIN-SEA developed by HR Wallingford & Lancaster University).
- Storm erosion modelling and wave overtopping assessment: This would involve inputting a timeseries of the extreme nearshore wave and water level conditions into beach storm erosion models such as SBEACH or XBEACH to determine the morphological adjustments to the beach profile as a result of the storm event. This will provide insights into design profiles, trigger conditions, volumes and frequency of the beach push-up, by-passing, and renourishment options to avoid excessive wave overtopping and breaching of the dunes.
- Littoral transport study to assess the longshore transport regime into and along the embayment: This will involve using the outputs of the nearshore wave modelling as inputs into longshore transport modelling along with detailed analysis of beach profile/bathymetric surveys pre- and post-coastal storm and river flood events to determine net and gross littoral transport, seasonal trends and the cross shore distribution of transport.
- Assessment of the aeolian (windblown) sediment transport regime of the upper beach and its potential influence on the stability of the dune system. This will require sediment sampling of the beach and dune system to determine the percentage of the sediment size distribution able to be transported by this process, and hence the effectiveness of plantings to trap windblown sediment. Sediment size

distributions could be determined by Digital Grain Size (DGS) analysis, a fast and effective way of collecting this data in the field, or through more traditional lab analysis of sediment samples (e.g. sieving or settling tube).

- Coastal evolution modelling: This modelling would use the outputs of the wave and sediment transport modelling in standard 1D numerical shoreline evolution modelling (e.g GENESIS, LITPACK, UNIBEST) to assess the influence of sand bypassing, renourishment of the presence/alignment of the Koau Training Walls on littoral transport regime and shoreline plan shape. These models will require calibration against historical shoreline change from aerial photographs.
- Flood impacts study to assess the effects of channel relocation on flood behaviour throughout the lower Clutha River system. This will require the application of a suitable flood model of the lower Clutha River system.
- Entrance dynamics study to estimate the likely requirements to maintain the new entrance. This will include sediment transport modelling of entrance area to assess sediment fluxes in and around the entrance and estimate the sedimentation regime of the entrance. The sediment transport model (2D process model) will need to be capable of accurately resolving the key processes that influence transport of sediments through the entrance, including waves, currents (driven by tides, floods, waves and winds) and bathymetric change.

4.6.3 Indicative Costs for Non LCFPDS Infrastructure Options

The information presented in this section is sourced from our experience and contacts in both New Zealand and Australia. It is noted that the costs given are very indicative to give relative orders of magnitude of costs for the different options. They would require considerable future investigations and refinement before being used for any budgeting purposes.

4.6.3.1 Beach Push-ups

The cost of beach push-up to increase beach crest elevations to the south of the Koau Mouth and therefore to reduce erosion by rollover processes, is influenced by:

- The volume of sediment to be relocated to the crest;
- The type and number of machinery used in the operation; and
- The frequency at which the treatment is repeated.

Based on the beach profiles, water levels, and wave climate, a push-up volume of 20 m^3/m has been assumed on the basis of increasing the beach crest elevation by 1 m height over a 20 m crest width. Therefore, for the 3.5 km of beach south of the Koau Mouth, the net volume required to be moved would be in the order of 70,000 m^3 for an initial treatment of the whole length of the beach.

From discussions with a local contractor, the indicative price for equipment hire (bulldozer, loader) to move this volume of material and shape the upper beach profile to design was estimated to be in the order of \$6.50/m^3. This is similar to the \$5/m^3 given by Carley and Cox (2017) for Australian examples of beach push-ups. Therefore, the total price for this initial treatment is in the order of \$455,000, not including any consent charges, matting or planting. It was also estimated that the time required to undertake this work would be in the order of 12 days assuming that the work can be done continuously (i.e. not affected by wildlife breeding seasons etc).

The frequency at which this activity would be required is not known with any certainty, but assuming that it the exercise was repeated on a 5 yearly basis, the cost over a nominal 30 year period for comparison with "whole of life" cost of other options would be in the order of $2.73 million. In reality, these costs are most likely to be on the low side, as more frequent overtopping with SRL is likely to require larger volumes to be relocated over longer time frames.

4.6.3.2 Sand Bypassing

Assisted sand bypassing of river and inlet mouths is not common in New Zealand, however there are a number of examples on both the east and west coast of Australia. Carley and Cox (2017) give an indicative cost of sand bypassing in Australia as in the order of $10/m^3. However, the capital and operating cost of a permanent sand bypassing system is strongly influenced by;

- The scope and objectives of the system;
- The natural environment that the system will be operating in;
- The transfer distance; and
- The accessibility to services such as power.

The scope and objectives of a sand bypassing system to manage sand transport around the mouth of the Koau River is largely unknown, and as such it is not possible to provide an accurate estimate of the costs for a permanent sand bypassing solution in Molyneux Bay.

Instead, Jacobs undertook a case study review to assess the costs of a number of sand bypassing systems that are currently in operation in Australia. Table 4.8 provide a summary of the assessed facilities.

Table 4.8: Summary of operational costs of selected sand bypassing schemes in Australia

Name/Location	Sand transfer rate (m3/year)	Method	Environment	Cost
Dawesville Cut / Mandurah entrance (WA, Australia)	~185,000 m^3/year	Slurry track and pipelines	Exposed ocean environment	Total Annual Expenditure: NZ$ 1.1million
Tweed Sand Passing facility (NSW, Australia)	~550,000 m^3/year (includes approx. 120,000 m3/year of dredging)	Permanent jetty-mount sand transfer system with pipeline network	Highly exposed ocean environment	Total Annual Expenditure: NZ$ 8.5million

Noosa Main Beach Sand Recycling System (QLD, Australia)	~40,000 m^3/year	Sand shifter and pipelines	Moderately exposed ocean environment	Total Annual Expenditure: NZ$ 0.7million

Based on the net northward longshore transport volumes south of the Koau Mouth from Atkin et al (2020b) (e.g. estimate 50,000 m^3/yr), the scale of sand transfers required to match northward loses are in the order of those at Noosa. Therefore, the costs from this example are considered to be most relevant for a similar bypass scheme at the Koau Mouth. This cost over a nominal 30-year period for comparison with "whole of life" cost of other options would be in the order of $21 million.

However, if we consider the net volume loss from profile comparison presented in section 4.1.1.1 (-14,400 m^3/yr), the bypassing cost would be less, but not by a pro-rata ratio to the above costs for by-passing 50,000 m^3/yr.

4.6.3.3 Beach Renourishment

The cost of beach renourishment is influenced by several factors, including:

- The project size – larger scale projects able achieve greater economies of scale;
- The availability and size of the dredgers, and where they are mobilized from (larger international dredgers will have higher mobilisation;
- The bathymetry of the borrow area and the distance between the borrow and target sites;
- The grain size of the nourishment material – coarser material causes greater equipment wear and tear, which is passed on to the customer;
- The exposure of the site – can limit the type and size of dredger to be used, and the number of days unable to operate within a dredge campaign; and
- The method of placement – bottom dumping, rainbowing or pumping ashore and shaping.

Carley and Cox (2017) give the range of costs for offshore dredging and placement for nourishment as between $20 and $50 per cubic metre. However, costs of past renourishment of Auckland beaches (Mission Bay and Kohimarama) provided by Auckland Council were in the order of $100/$m^3$ for sand dredged from the Pakiri nearshore, barged to site (80 km barge distance), and pumped ashore.

Dunedin City Council have also undertaken renourishment of Ocean Beach Dunedin (2007 and 2016) using sand dredged from Otago Harbour entrance channel near Taiaroa Head, deposited on the T & U wharf in the city and trucked to Ocean Beach for placement. Port Otago dredge costs in 2016 (including ORC royalty) were in order $15/$m^3$. Applying this source of sand supply to Southern Molyneux Bay would require trucking from the Dunedin wharf (approximately 100 km travel distance), at an estimated cost in the order of $50/m3. Therefore, the total cost of supply from this source/ method is in the order of $65/$m^3$, plus the cost of placement into appropriate profile shape, which for the sake of this exercise, is assumed to be similar to the beach push-up option. Hence, the total cost of supply, transport, and placement of an initial volume of 60,000 m^3 to cover a potential four years of loss to offshore/longshore (e.g. annual loss rate in order of -15,000 m^3/yr from Section 4.1.1.1) would be in the order of $4.5 million. If this was repeated on a five-yearly basis, the comparable whole of life cost over 30 years would indicatively be in the order of $27 million. There are also logistical issues with this supply option, as the supply of 60,000 m^3 would involve 100 dredge loads (600 m^3 capacity), which since the wharf can store multiple loads would take approximately a year to deliver, and would involve over a total of 3300 truck and trailer movements (18 m^3 capacity) over a route which is understood to not be heavy vehicle permitted route.

The alternative supply source as outlined in Table 4.6, is dredging the nearshore sand wedge in Southern Molyneux Bay and either pumping or rainbowing ashore, from where it can either be placed to increase beach heights, or allow waves to move naturally onto the beach profile. Under the second of these options, crest height is unlikely to be naturally increased to reduce erosion by rollover, therefore reducing the efficiency of the renourishment. The Port of Otago dredge cannot undertake this dredging, as Molyneux Bay is outside of their Maritime NZ operating limit and the dredge does not have the capacity to rainbow ashore. Inquiries have indicated that that the private commercial dredge used in the Auckland beach renourishments has capacity to undertake such dredging, with an indicative estimated supply cost of \$100/m^3, excluding consenting costs for coastal permits to extract and place the sand. Under this scenario, based on an initial supply of 60,000 m^3 to cover a potential four years of loss to offshore/longshore (e.g. annual loss rate in order of -15,000 m^3/yr from Section 4.1.1.1) would be in the order of \$6 million, with additional placement costs to move to beach crest being higher that indicated in the beach push up option (e.g. \$455,000) due to greater push distance up the foreshore. If this dredging/placement was repeated on a five-yearly basis, the comparable whole of life cost over 30 years would indicatively be in the order of \$39 million.

There is no reported gravel/sand build up on the bed of the lower Clutha River, except for the small sand spits at the Koau Mouth, therefore this option of supply has not been considered further.

5. Coastal Inundation Assessment Results

5.1 Summary of Stage One Inundation Assessment Results

As noted in Section 3.5, the results of the Stage One inundation assessment are summarised below, and presented in full in Appendix F.

The results of the stage one inundation assessment are based on simple comparison of surveyed crests with present and future sea levels. It is noted that the difference between the 100-year and 200-year ARI high water levels is only 0.04m, which is much lower than the range of SLR scenarios for different timeframes up to 100 years (e.g. 0.2m by 2050, up to maximum of 1.35m by 2120). Therefore, this summary focus on the 100-year ARI results. Figure 5.1 below shows the extents of the floodbanks considered in this assessment and discussed in the following sections.

5.1.1 Port Molyneux Floodbank

Some small sections of the Port Molyneux Floodbank are below the current day 100-year ARI high water level with wave set up (2.31m). With SLR, around 70% are below the 100-year ARI with rise of 0.2m (e.g. by 2050), and almost the entire length (95%) is likely to be overtopped in the same frequency event with 0.4m SLR (e.g. 2.71m with wave set-up).

5.1.2 Puerua Deviation Floodbank

All left banks of the Puerua Deviation Floodbank are above the present day 100-year ARI high water level with wave set-up, while the right bank downstream of Kaka Point Road has two low points that could be overtopped in this magnitude event with current sea levels. Approximately 200m of left bank near the Puerua River Mouth would be overtopped in the 100-year ARI with 0.4 m of SLR (2.63m, DVD 1958). At the mouths of the Puerua River and the Koau Branch the crest levels are below 2.83m and are likely to be overtopped with the 100-year ARI with 0.6m SLR. On the Puerua River, upstream of the Kaka Point Road Bridge, overtopping by the 100-year ARI high water level including river mouth / inlet wave set up with 0.2m of SLR is likely to occur on the right bank, and with 0.4m of rise on the left hand bank up to Wix Road.

5.1.3 Inch Clutha Floodbank

Crest levels along the Inch Clutha Floodbank would not be overtopped in present day 100-year ARI high water levels, or with SLR up to 0.4m. With 0.6m of SLR, the floodbank elevation is exceeded by the 100-year ARI high water level including wave set up at four locations. With 0.8m of SLR, the floodbank elevation is exceeded for around 70% of its length for this frequency event, increasing to 100% with 1.0m of SLR. However, the dune field east of the floodway includes an extensive width with elevations greater than 3m, hence the source of any bank overtopping would have to be via the lower river channels and the Inch Clutha Bypass Floodway, rather than overtopping the beach.

5.1.4 Koau Branch

The left bank floodbank along the Koau Branch below where the Inch Clutha Floodbank joins could be overtopped by the 100-year ARI high water level with 0.6m SLR (including river mouth/inlet wave set up). Upstream of the Inch Clutha Floodbank, both left and right banks downstream of chainage 800 (Figure 5.2) could be overtopped by the 100-year high water level with 0.8m SLR, below 1000m chainage with 1.0m of SLR and below chainage 1350-1450m with 1.35m of SLR.

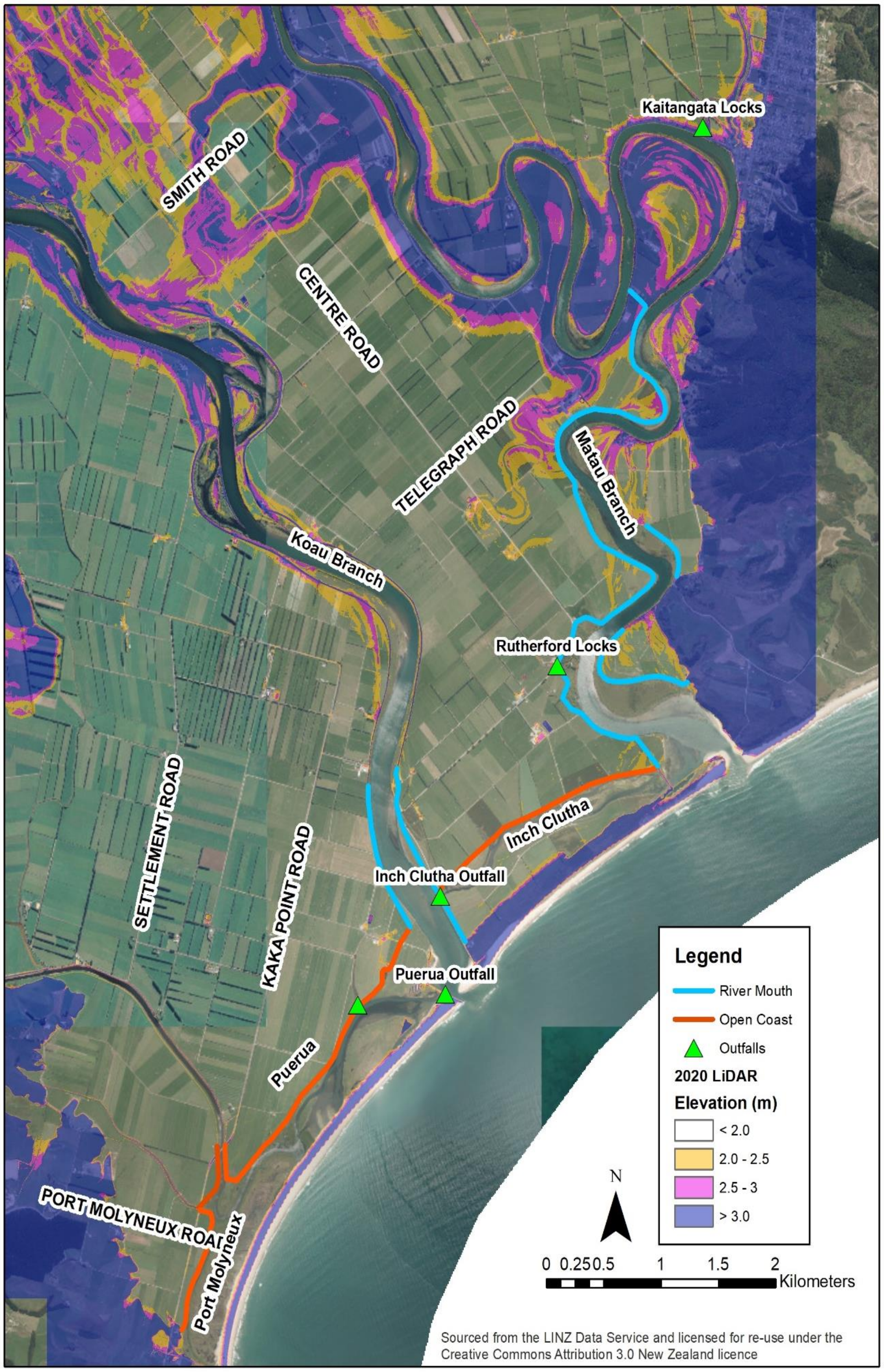


Figure 5.1: LCFPDS structures considered in the inundation assessment.

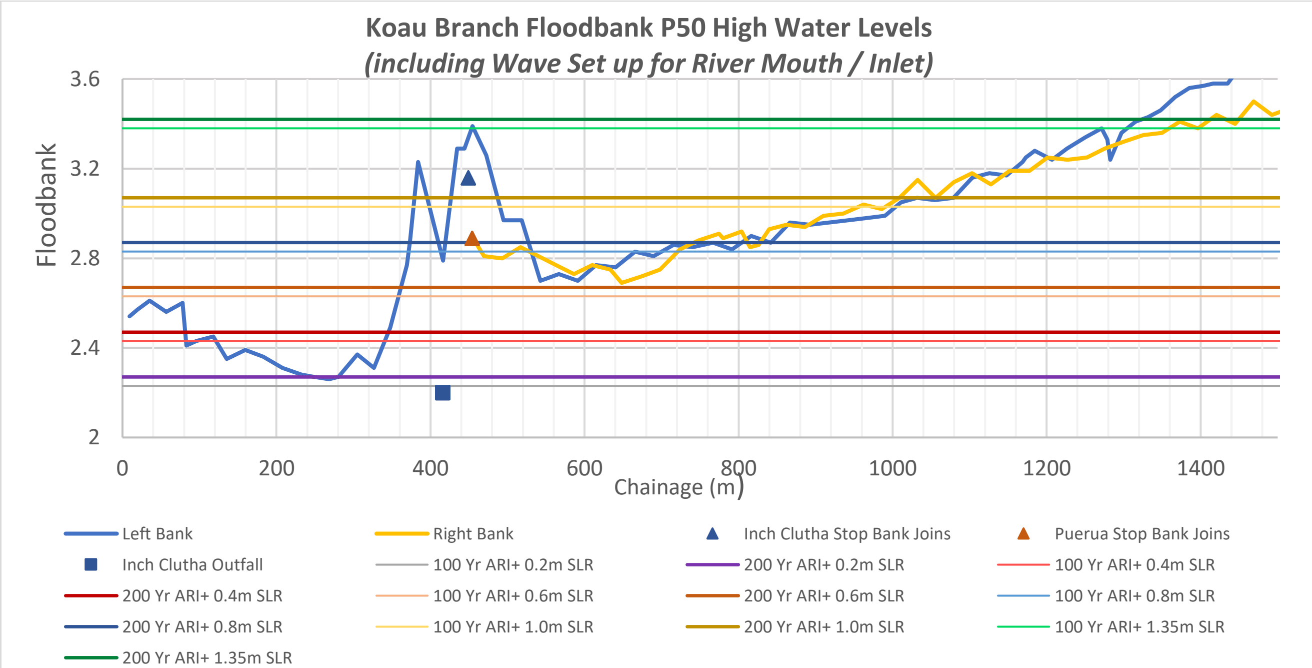


Figure 5.2: Koau Floodbank P50 inundation levels (including wave set up for river mouth/inlet) for 100 year and 200 year return periods.

5.1.5 Matau Branch

The right bank on the Matau Branch has low points at Rutherford Locks and chainage 3100 m (Figure 5.3) that could be overtopped by the the100-year ARI high water level with 0.6m SLR (including river mouth / inlet wave set up). Below chainage 4800m much of the right bank crest levels are below the 100-year ARI with 1.0m SLR high water level. Along much of the left bank there is no formal floodbank protection, but the higher ground level provides protection from coastal inundation. The left bank at the mouth section has parts that could be overtopped by high water levels above the 100-year ARI high water level with 1.0m of SLR (including river mouth/inlet wave set up). The left bank Summer Hill section could be overtopped by the 100-year ARI high water level with 0.6m SLR (including river mouth/inlet wave set up) of 2.67m.

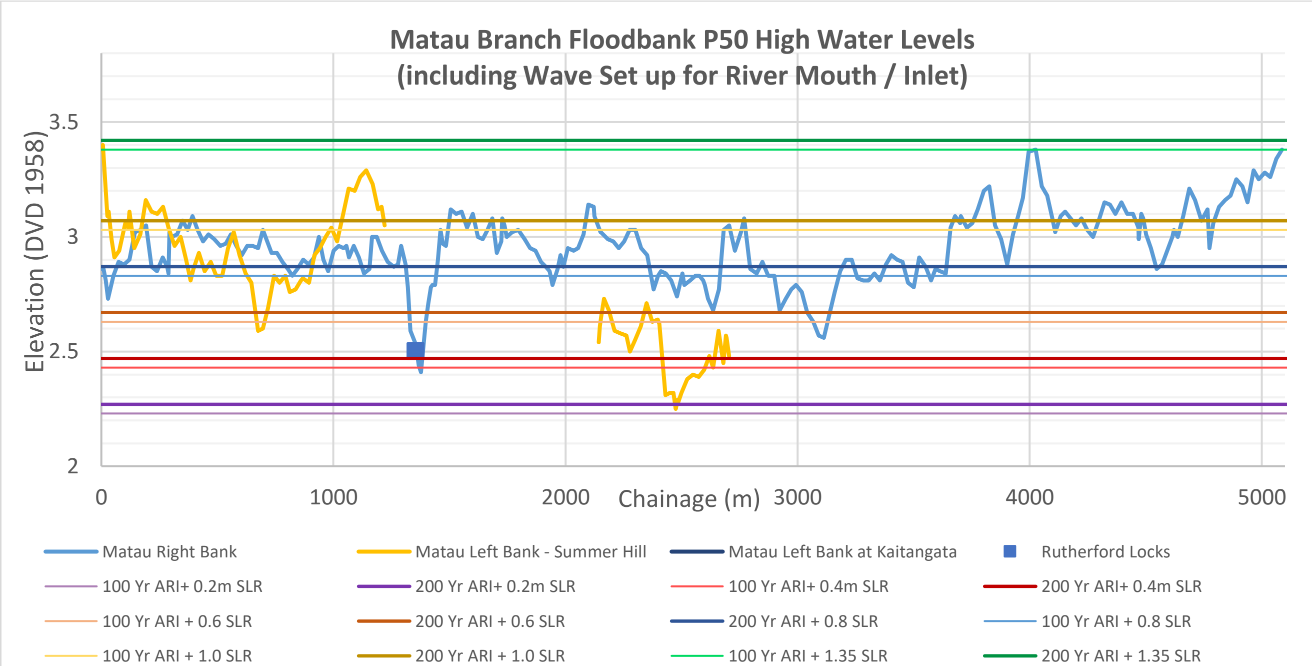


Figure 5.3: Matau Floodbank P50 inundation levels (including wave set up for river mouth/ inlet) for 100 year and 200 year return periods.

5.2 Recommendation for Stage Two Inundation Assessment

The Stage One inundation assessment shows that overtopping is likely for the Port Molyneux floodbanks by the 100-year ARI high water level with 0.2m SLR with wave set up (e.g. within 30 years), and for the Puerua and Inch Clutha Floodbanks with 0.4m SLR (e.g. within 50 years). The more protected river mouths are at risk of overtopping in the 100 year ARI with SLR from 0.6m to 1.35m (including river mouth / inlet wave set up), which are not likely to occur in at least the next 50 to 100 years.

On this basis Jacobs have recommended that the stage two hydrodynamic model inundation assessment be carried out, with the events to be modelled being agreed with ORC. The difference between the 100-year and 200-year ARI high water levels is only 0.04m, while the range of SLR is from 0.2m to 1.35m, suggesting that the sea level range has a much greater influence on the frequency and extent of future inundation. Therefore, the modelling is likely to show very little difference between 100- and 200-year high water levels and hence the following recommended scenarios are primarily limited to 100-year ARI high water levels including the appropriate wave setup allowances.

The projected shoreline positions presented in Section 4 are limited to SLR of up to 0.6m over 50 years, and do not include the scenarios of higher SLR over longer periods due to the uncertainty with the future and influence of the mouth training walls on shoreline position beyond this time frame, particularly for the southern Koau Mouth Training Wall. Hence, our recommendation is to model to following events and shoreline positions:

- 100-Year ARI with 0.2m SLR and projected 2050 shoreline position.
- 100-Year ARI with 0.4m SLR and projected 2070 shoreline position.
- 100-Year ARI with 0.6m SLR and projected 2070 shoreline position.
- 50-Year ARI with 0.6m SLR and projected 2070 shoreline position.

Following the presentation of these results and recommendations to ORC, and a discussion of the value of Stage Two Inundation assessment at this time, it was agreed by ORC and Jacobs to delay stage two modelling until ORC are able to have further consideration around erosion management and the possible consequences for the LCFPDS assets so that these decisions can be accounted for in the next stage of the inundation modelling.

6. Conclusions and Monitoring Recommendations

The purpose of this coastal morphology and climate change investigation is to help guide decision making regarding management of the Molyneux Bay – Clutha Delta coastline, including the river mouth training wall structures, floodbanks and coastal drainage channels of the LCFPDS. To achieve this purpose, the investigation included an assessment of coastal erosion and coastal inundation hazards over the next 30 to 50 years with sea level rise and climate change, the potential impact of these hazards on the LCFPDS infrastructure over these timeframes, and a high level options assessment for dealing with the consequences of coastal erosion hazards on this infrastructure.

6.1.1 Coastal Erosion

A major component of the coastal erosion hazard assessment was understanding trends and patterns of shoreline movements, and particularly the effect of river mouth location, migration and training structures on these patterns. The conclusions from this analysis, covering shoreline change since 1946, include:

- Over the total period from 1946 to 2020 there is a trend of greatest erosion in southern Molyneux Bay at average rates in the order of -4m/yr reducing in a northward direction through to the centre of the bay between the river mouths, which reverts to long-term accretionary trend to the north of the Matau Mouth. This trend is constant with net northerly sediment transport and potentially greater supply from the Matau Branch than the Koau Branch.

- For southern Molyneux Bay (e.g. south of the Koau Mouth), the analysis revealed that erosion rates have increased significantly since the construction of Koau Mouth Training Walls in the early 1980's. Prior to this time the shoreline fluctuated between erosion and accretion, with an apparent relationship between the mouth position and whether this section of beach was in a state erosion or accretion – for example, erosion in the north corresponded to mouth migration in 1962-1972, and accretion in south corresponded to mouth migration between 1972-1982. In contrast, for the periods covered by aerial images following the construction of the training wall (1997-2020), trends of shoreline movement over the whole southern cell have been consistently erosional with average net retreat rates in the order of -10m/yr at the southern end of the beach, reducing to -6 to -7.5m/yr closer to the Koau Mouth.

- The mechanism of this post training wall acceleration in erosion over the whole southern cell appears to be a negative feedback loop of increased beach rollover lowering dune crest elevations, which in turns allows more frequent rollover and therefore further increased erosion. This is a common shoreline response for beaches lacking sediment supply that are rolling back onto wetlands, lagoons or low hinterlands. This reduced sediment supply is considered to be due to a combination of the following possible causes:
    - A lagged response to the reduced supply from the Clutha River since the construction of the Roxburgh Dam (1957).
    - Lack of significant floods in the Clutha River in the 20 years from November 1999 to December 2019.
    - The presence of southern training wall preventing southern migration of the river mouth, therefore removing any significant periodic sediment inputs to the southern beaches.
    - The alignment of the training walls directing the ebb tide jet further offshore and into the more northerly sediment transport pathway.

- In the central Molyneux Bay cell (e.g. between the river mouths), the rates of retreat over the total 70+ year period are lower (range -1.57 north of the Koau Mouth to -0.3 m/yr south of the Matau Mouth), however have gone through periods of erosion and accretion both pre and post the construction of the Koau Mouth Training Walls. Although the migration of both mouths prior to the construction of the mouth trainings structures have influenced shoreline movements in the adjacent sub-cells during some time intervals, erosion rates have generally been similar across both pre and post mouth training construction periods.

- For northern Molyneux Bay, the historical shoreline trend for this has been in the range +0.6 to +0.8 m/yr, which has generally been consistent over the total period 70+ year period. It is therefore considered that the shoreline movements along this section of coast are not influenced by river mouth orientation or migration, or any of the mouth training structures.

Due to the significant change in erosion rates south of the Koau Mouth in the post training wall period, which the presence of the southern wall is considered to have a substantial influence on, the extrapolation of historical shoreline movements for this section of Molyneux Bay was undertaken applying the following scenarios:

- For the 30 years to 2050, the wall is assumed to remain in place, and the extrapolation is at the higher 1997-2020 post wall rates.

- For the 20 years between 2050 and 2070, one scenario of continued extrapolation at the higher 1997-2020 rates under the assumption that the wall continues to be maintained and in its current position, and a second scenario of the wall being removed and erosion assumed to slow and return to the 1946-1982 pre-wall rates.

For the central and northern sections of Molyneux Bay, the erosion rate over the total 1946 to 2020 period was used in the extrapolation.

6.1.2 Projected Future Shoreline Positions

For the calculation of Projected Future Shoreline Positions (PFSP), over the 30- and 50-year timeframes, the erosion impact of sea level rise was added to the extrapolation of contemporary erosion rates. The impact of sea level rise on future erosion was calculated using a modified Bruun rate for overtopping on sand beaches for rise of 0.2 m by 2050 and both 0.4 m and 0.6 m by 2070. Short term storm events were estimated from SBEACH-32 numerical modelling simulating beach responses to extreme storm events (50, 100, 200-year ARI), however these were not included in the PFSP erosion distances. The resulting estimates of the total erosion distances to PFSP over each time period and the potential effects on the infrastructure of the LCFPDS are as follows:

- By 2050: Greatest shoreline retreat at the southern limit of the bay, with the 'most likely' erosion of -320m from the present-day position, and a 5% chance of an additional 150m retreat from this position. The predicted erosion distances decrease in a northward direction to in the order of -220m for the section immediately south of the Koau Mouth, (5% chance that erosion could be 120m greater), -65 m on the north side (5% chance that could be 40m greater), and down to -20m immediate south of the Matau Mouth (5% chance could be 20m greater retreat). For the southern section of the bay, 85-90% of the predicted retreat will be due to the extrapolation of contemporary erosion rates since the construction of the southern training wall, while of the central cell, SLR contributes 25-30% at the southern end increasing to 90% at the northern end. In the northern bay, due to the extrapolation of the contemporary shoreline change, shoreline is most likely to remain accretionary, with the shoreline in a similar or more seaward position than the present-day position.

- By 2070 the projected future shoreline position in southern Molyneux Bay is heavily influenced by whether the Southern Koau Training Wall is maintained or not, with less sensitivity around the increase in SLR. In both SLR scenarios where the wall is maintained, erosion distances are in the order of -540 to -570m at the southern end of the bay (5% chance could be 245m more), and decreasing in a northward direction to -380 to -400m south of the mouth training wall (5% chance could be 195m more). Returning to 1946-1982 erosion rates by removing the southern training wall could reduce these erosion distances by the order of 150 to 180m at the southern end of the bay, and by approximately 45m south of the Koau Mouth. The average difference in erosion distances between the two SLR scenarios is in the order of 15-25m in the southern bay, with the effect of SLR contributing 10-15% of the total erosion distance in the 0.4m rise scenario, increasing to 15-20% for the 0.6m rise scenario. Erosion distances continue to decrease in a northward direction around the central coastal cell between the two river mouths, reducing to -40 to -55m at the Matau Mouth (5% chance that could be an additional 40m). The contribution of SLR on the total erosion increases from 25% (0.4 m SLR) to 40% (0.6 m SLR) at the Koau Mouth, to 78% (0.4 m SLR) to 92% (0.6 m SLR) at the Matau Mouth. In the northern bay most areas of shoreline continue to accrete with 0.4m of SLR, but overturn to being erosional with 0.6m of SLR.

6.1.3 Potential Effects on the Infrastructure of the LCFPDS

By 2050 around 150-170m of the Koau Southern Training Wall structure could be exposed to direct wave energy if the wall remains in its current form. Hence there will be an increase in costs to maintain this structure in its current form. The Puerua Diversion Outlet within the wall would also be exposed, the drainage along the diversion channel would be blocked by the beach adjacent to the outlet and compromised further south. A 500m section of floodbanks at the end of Port Molyneux Road is also likely to be compromised by the shoreline retreat over this time period, with a 5% chance that the whole length of banks along the Puerua Diversion are similarly affected by the erosion. Along the central bay between the river mouths, the predicted shoreline positions are located within the current high back dune environment, indicating that the ability of these dunes to provide protection against coastal inundation could be compromised within this time period. At the northern end of the cell the predicted erosion would place the beach very close to the position of the Matau River training wall, indicating that the beach may roll over the top of this structure exposing the front to damage from open coast processes.

Under all 2070 (e.g. 50 year) scenarios, the impacts on the LCFPDS infrastructure is the same; the shoreline being landward of the Puerua River Diversion channel and stopbanks over their total length and the southern training predicted to be outflanked from behind with the shoreline predicted to most likely be located on the main river channel landward of the wall. In the central bay, the shoreline is predicted to be landward of the Matau Mouth training line, further exposing the structure to open coast erosion processes. For the very unlikely 5% PFSP, the drainage capacity of the northern end of the Inch Clutha Diversion Channel would also be compromised. Along the centre of the cell the PFSP indicates that the current high back dunes are lost to erosion suggesting that the vulnerability to coastal inundation along this section of coast would increase.

6.1.4 Coastline Management Options

A number of possible coastline management options that ORC could employ to address coastal erosion issues and/or consequences for the LCFPDS infrastructure by 2050 were identified, which included a high-level assessment of the strengths and weaknesses of each option. The options which cover a range of soft engineering, hard engineering, and waterway diversions include:

- Do nothing

- Back beach push up – Raising the crest elevation of the beach in southern Molyneux Bay by pushing up sand sourced from the back of the beach
- Sediment bypass to the south of the Koau Mouth
- Beach nourishment – Adding sediment to the beach profile in southern Molyneux Bay which is sourced from the nearshore wedge
- Relocate the Puerua Diversion channel landward
- Create a new mouth for the Puerua River

6.1.5 Coastal Inundation

The stage one coastal inundation assessment identified the following floodbank areas that could be potentially be overtopped with various increments of sea level rise in 1% AEP (e.g. 100-year ARI) coastal storm events.

- For SLR of 0.2m (e.g. by 2050), 70% of the Port Molyneux floodbanks could be overtopped.
- For SLR of 0.4m (e.g. by 2070), up to 95% of the Port Molyneux floodbanks could be overtopped along with approximately 200m of the Puerua Diversion banks and 65% of the Koau Branch left bank (below Inch Clutha outfall) and low spots on the both banks of the lower Matau Branch.
- For a higher SLR of 0.6 m by 2070, overtopping would increase to 100% of the Port Molyneux, Puerua Diversion banks, and the Koau Branch left bank below the Inch Clutha outfall. An addition five locations on the Inch Clutha floodbanks could also be overtopped.
- For SLR of 1m (e.g. by 2120) would result in 1000m of the right floodbank on the lower Koau Branch being overtopped, along with considerable lengths of both floodbanks of the lower Matau Branch.

As a result of the stage one assessment, it is recommended that ORC undertakes stage two hydrodynamic modelling to better understand the extent and wider effects of the flooding across the LCFPDS structures, however this should be delayed until ORC are able to have further consideration around erosion management and the possible consequences for the LCFPDS assets so that these decisions can be accounted for in the next stage of the inundation modelling.

6.2 Monitoring Recommendations

The findings of this report are based on a limited data set of shoreline movements and geomorphologic change for the Molyneux Bay area, including historical imagery, LiDAR data, and limited historical beach profiles. In order to help better inform future decision making and morphological modelling, it is recommended that ORC implement an ongoing monitoring programme for the Molyneux Bay area, especially for the area of shoreline south of the Matau River Mouth. Collecting useful and accurate data on shoreline change will help better inform any future investigations assessing future changes in coastal morphology and will help ORC determine trigger points for decisions on further action on LCFPDS infrastructure.

There are several different methods of shoreline morphology data collection which will be useful for informing future investigations and decision making. Ideally, this data collection needs to inform the position, elevation and volume of the beach at regular intervals (e.g. annually, bi-annually) as well as pre and post significant storm events if possible. When this is done over long period of time, it can help inform longer term patterns and trends as well as the magnitude of short-term fluctuations. The collection of this data can be undertaken in different ways, which come at varying costs depending on the resources ORC have access to (e.g. GNSS surveying equipment, drones). The possible methods, along with their advantages and limitations are set out in Table 6.1.

Table 6.1: Possible shoreline morphology monitoring methods

Method	Notes	Advantages	Limitations
Aerial Imagery Collection	Used to inform the relative position the beach compared with historical data collected by the same method. Data collection historically by plane, but more recently by UAV (e.g. drone).	Easy to compare to historical data	Trends to be piece-meal in frequency, with not being easy or cheap to undertake more regularly. Limited to analysis of position data and does not allow collection of data on morphological or volume change.
LiDAR Data Collection	Can be used to inform the elevation and volume of the beach. It can be collected by plane or UAV, with the use of a UAV likely to provide higher resolution and be carried out at a lower cost, depending on the equipment available	Two-dimensional data collection allowing analysis of geomorphological change over time. Beach profiles and volumes can be taken retrospectively from this data capture.	Trends to be piece-meal in frequency, with not being easy or cheap to undertake more regularly. The accuracy of the measurements is limited to the resolution of the data capture. Possible limitations on accuracy of elevation data around vegetation.
Beach Survey Profiles	Preferably carried out using GNSS survey equipment. ORC have five beach profile monitoring sites within the bay; however these have not been monitored at regular and frequent intervals.	Two-dimensional data collection allowing analysis of geomorphological change over time. Can give a very accurate measurement of the position and elevation of the beach profile. Can be rapidly immobilized to undertake pre and post event surveys.	Limited spatial coverage, with assumed representativeness of profiles. No ability to retrospectively add new profiles from past surveys.
Topographic Survey	Involves using GNSS for survey on foot or by beach vehicle, or by data capture from UAV simultaneously recorded with aerial imagery. Can help inform the position, volume and elevation of the beach by capturing surface	Two-dimensional data collection allowing analysis of geomorphological change over time. Beach profiles and volumes can be taken retrospectively from this data capture.	Can be time consuming if surveying by foot. If using UAV, limitations of how low down the beach data can be captured.

	elevations of the beach over the entire shoreline.	Can give a very accurate measurement of the position and elevation of the beach profile. Can be rapidly immobilised to undertake pre and post event surveys.	

Of these methods, we recommend that ORC undertake topographic surveys of the Molyneux Bay area using a UAV (e.g. drone) to monitor the shoreline and coastal structures. It is recommended that the topographic surveys are carried out annually at approximately the same time each year to avoid any seasonal variation in beach condition. In addition to annual surveys, it is also recommended that additional topographic surveys be undertaken following significant coastal storm events in order to capture the magnitude any short-term effects on beach position and morphology. By using this method, it allows for data to retrospectively be taken out of the dataset (e.g. beach profiles and volumes) to inform future analysis. It is a method that can collect a large amount of data over a considerable area in a short amount of time, in a format which can be used to comparably analyse various aspects of the beach morphology at any location in the future.

In addition to monitoring of the onshore profile, an updated bathymetric survey is recommended to assist in understanding in the nearshore/subaerial system. This information would also assist any future modelling of coastal evolution and longshore transport processes.

For future inundation modelling, and as noted in Section 3.5.1, a potential issue in addition to sea level rise is the subsidence of the stopbanks over time on soft alluvial sediments, which could lower the crest height. At present, current survey datasets (LiDAR, crest surveys) are not of the accuracy or duration to allow for any assessment of the level of subsidence that has occurred since the construction of the stopbanks as a result of settling on alluvial sediments. It is recommended that resurveying of crest levels along the stopbanks continue to allow for this analysis to be undertaken and incorporated into future floodbank assessments.

7. References

Atkin E., O'Neil S., and Mead S. (2020a) South Otago coast sediment transport: Literature Summary. eCoast Report for Dunedin City Council. 32 pps

Atkin E., O'Neil S., and Mead S. (2020b) South Otago coast sediment transport: Numerical modelling eCoast Report for Dunedin City Council. 34 pps

Barrell D. J. A. et al (1998) Quaternary fans and terraces of coastal Otago, New Zealand. IGNS Science Report 98/11.

Beavan R. J. (2012) Vertical land movement around the New Zealand coastline: implications for sea-level rise. GNS Science Report 2012/29.

Bishop, D. G. and Turnbull, I. M. (compilers) (1996). Geology of the Dunedin area, Institute of Geological and Nuclear Sciences 1:250,000 geological map 21.

Bloor, H (2017). Analysis of the Koau Branch mouth and Molyneux Bay coastline – Future options for the training line. Internal ORC Memorandum. 23 pps

Bruun P. (1962) Sea level rise as a cause of shore erosion. *Journal of Waterways and Harbors Division*, ASCE 88, 117–130.

Bruun, P. (1988) The Bruun Rule of erosion by sea level rise: a discussion on large-scale two- and three-dimensional usages. *Journal of Coastal Research* 4, 627–648.

Carley J.T. and Cox R.J (2017) Guidelines for sand nourishment: Science and synthesis for NSW. NSW office of Environment and Hertigate's coastal Processes ans Response Node – Technical Report. WRL Resewrach Report 263.

DoC (2010) New Zealand Coastal Policy statements. Department of Conservation.

DTec (2002) Coastal Erosion Issues South of the Clutha River mouth. Report for Contact Energy. 27pps

Carter, R.M., Carter L., Williams J.J., and Landis C.A. (1985). Modern and relict sedimentation on the South Otago continental shelf. New Zealand Oceanographic Institute Memoir 93, 43 pp.

Carter L (1986) A budget for modern Holocene Sediment on the South Otago continental shelf. *New Zealand Journal Marine & Freshwater Research 20:* 665-676

GeoSolve (2017) Koau Mouth – southern Training Works. Letter report to ORC. 6pps

GeoSolve (2018) Clutha River/ Mata-au Matau Mouth. Letter report to ORC. 7pps

Gibb J.G. (1978) Rates of Coastal Erosion and Accretion in New Zealand; *NZ Journal of Marine and Freshwater Research, 12 (4): pp 429-56*

Hicks, D.M., Walsh, J.M. and Duncan M.J., (2000) Clutha River Sediment budget. Report prepared for Contact Energy Limited. 79pps

Hornblow, S (2016a). Natural Hazards on the Clutha Delta. Report Prepared for Otago Regional Council. 137 pps.

Hornblow, S (2016b). 2016 report on shoreline retreat at Molyneux Bay and threat to Lower Clutha Flood Protection and Drainage Scheme. Internal ORC report. 23pps.

Irish J.L & Canizares R. (2009) Storm-wave flow through tidal inlets and its influence on bay flooding. *Jl Waterways, Port, Ocean Eng., 2009*, 135 (2): 52-60

Johnstone N.P. (1993). Matau Mouth (Clutha River) offset. Internal ORC Report 3 pps.

Kettle C.H. (1852) Otago – General map of the settlement rural sections. Supplied by ORC.

Litchfield, N. J. and O. B. Lian (2004) Luminescence age estimates of Pleistocene marine terrace and alluvial fan sediments associated with tectonic activity along coastal Otago, New Zealand, *New Zealand Journal or Geology and Geophysics,* 47, 29-37.

NIWA (2008) Otago Regional Council storm surge modelling study. Report for ORC. 164pps

OPUS (2000) Lower Clutha Sediment and Erosion Issues – Review of Hydrological, hydraulic and Geomorphological Aspects. Report for Contact Energy

Smith, A (2007). Marine Sedimentation and Coastal Processes on the Otago Coast. Report Prepared for Otago Regional Council. 43 pps.

Rosati J.D., Dean R.G., Walton T.L. (2013) The modified Bruun rule extended for landward transport. *US Army Research.* Paper 219. 12pps

Tawn, J. A.; Vassie, J. M. (1991) Recent improvements in the joint probability method for estimating extreme sea levels. In Tidal Hydrodynamics, B. B. Parker (ed). John Wiley & Sons, pp 813:828.

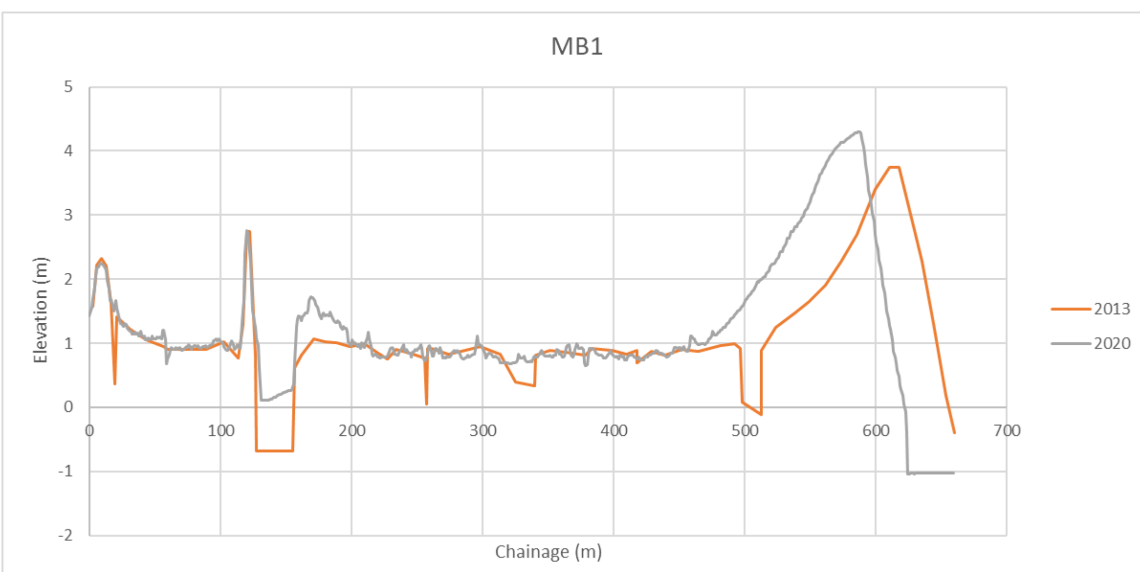
Tonkin and Taylor (2000). Clutha Sediment Budget Report for Contact Energy. Ref Number 5027 6.002. 80pps

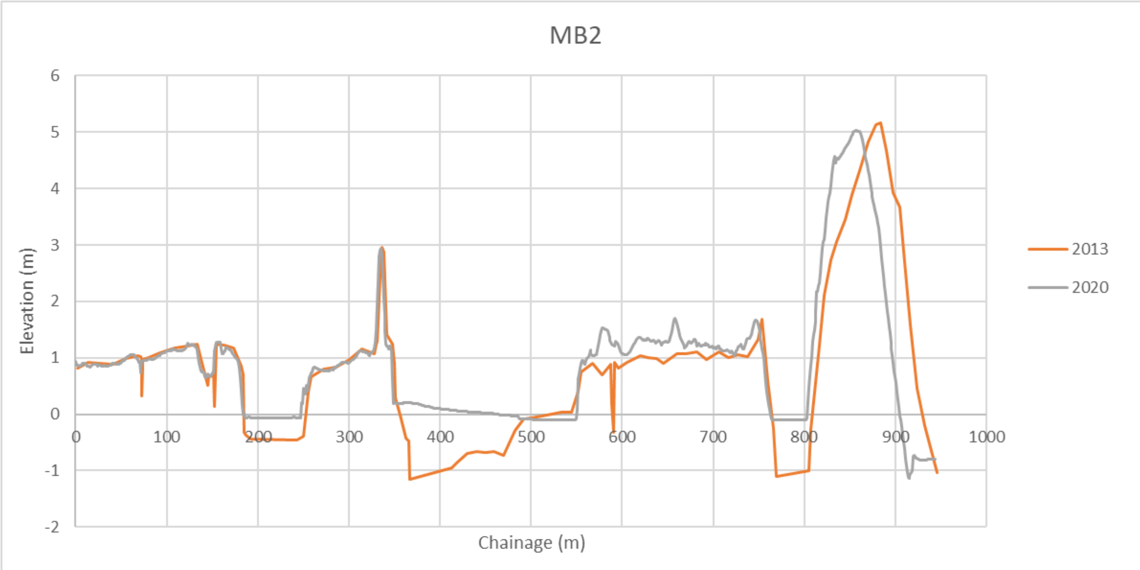
Todd, D (2002). Statement of Evidence for Contact Energy Limited Resource Consent Hearing on the Roxburgh Dam. 46 pps

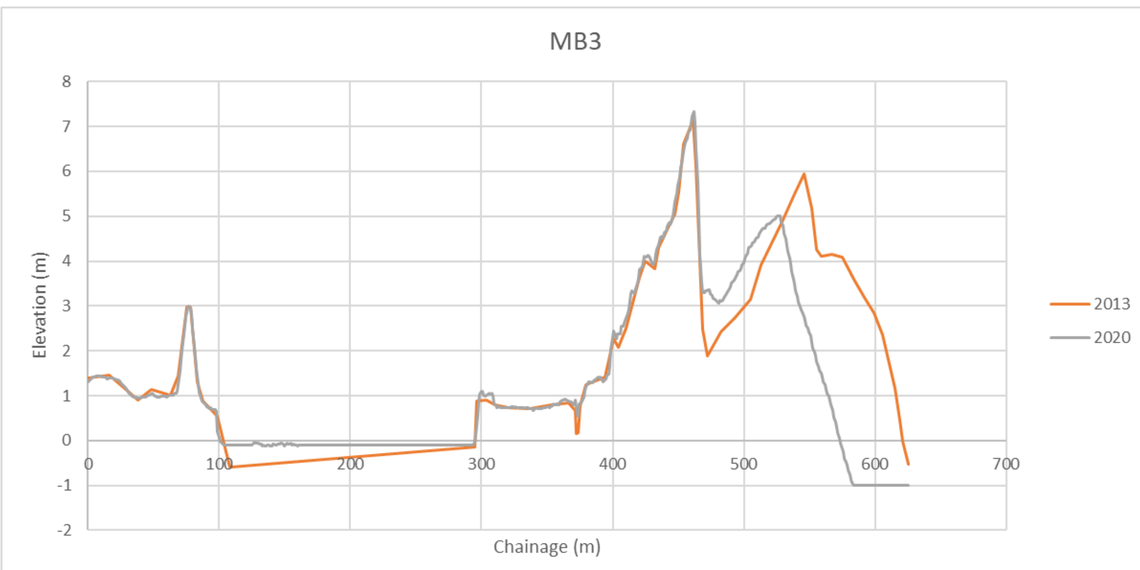
Williams J. & Goldsmith M. (2014) Coastal morphology of South Otago: Nugget Point to Chrystalls Beach. Orc report. 64pps.

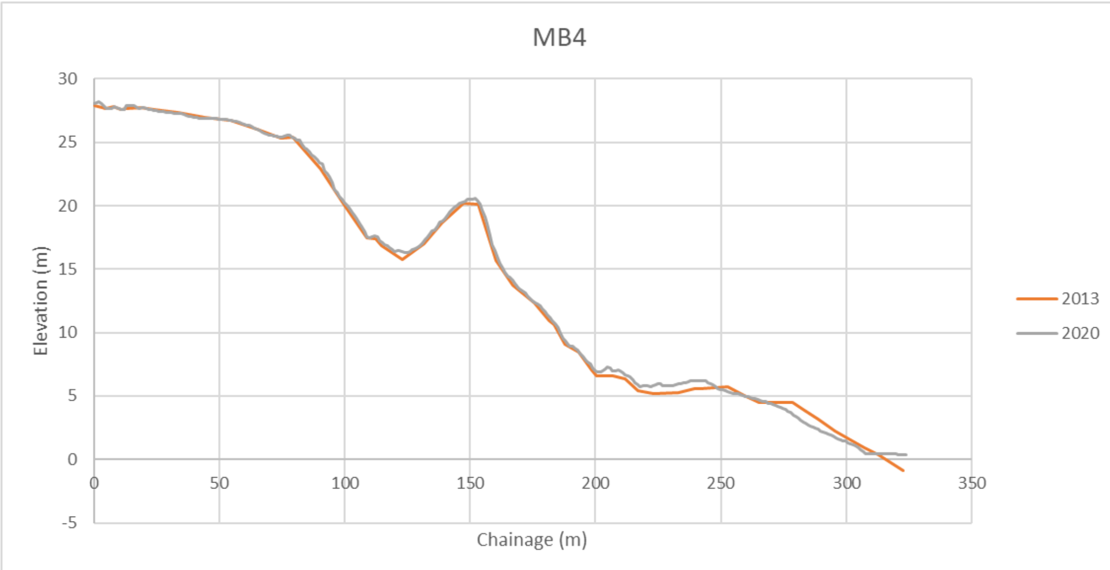
Appendix A. Transects from 2004, 2013 & 2020 LiDAR

Figure 1: Location of Transects from 2004 and 2013 Lidar, the position of transects used in this study and bathymetry data points.




MB2
Elevation (m)
Chainage (m)
2013
2020


MB3
Elevation (m)
Chainage (m)
2013
2020


MB4
Elevation (m)
Chainage (m)
30
25
20
15
10
5
0
-5
0
50
100
150
200
250
300
350
2013
2020


MB5
Elevation (m)
Chainage (m)
30
25
20
15
10
5
0
-5
0
50
100
150
200
250
300
350
400
450
2004
2013
2020

Appendix B. Representative Beach Profiles from 2020 LiDAR

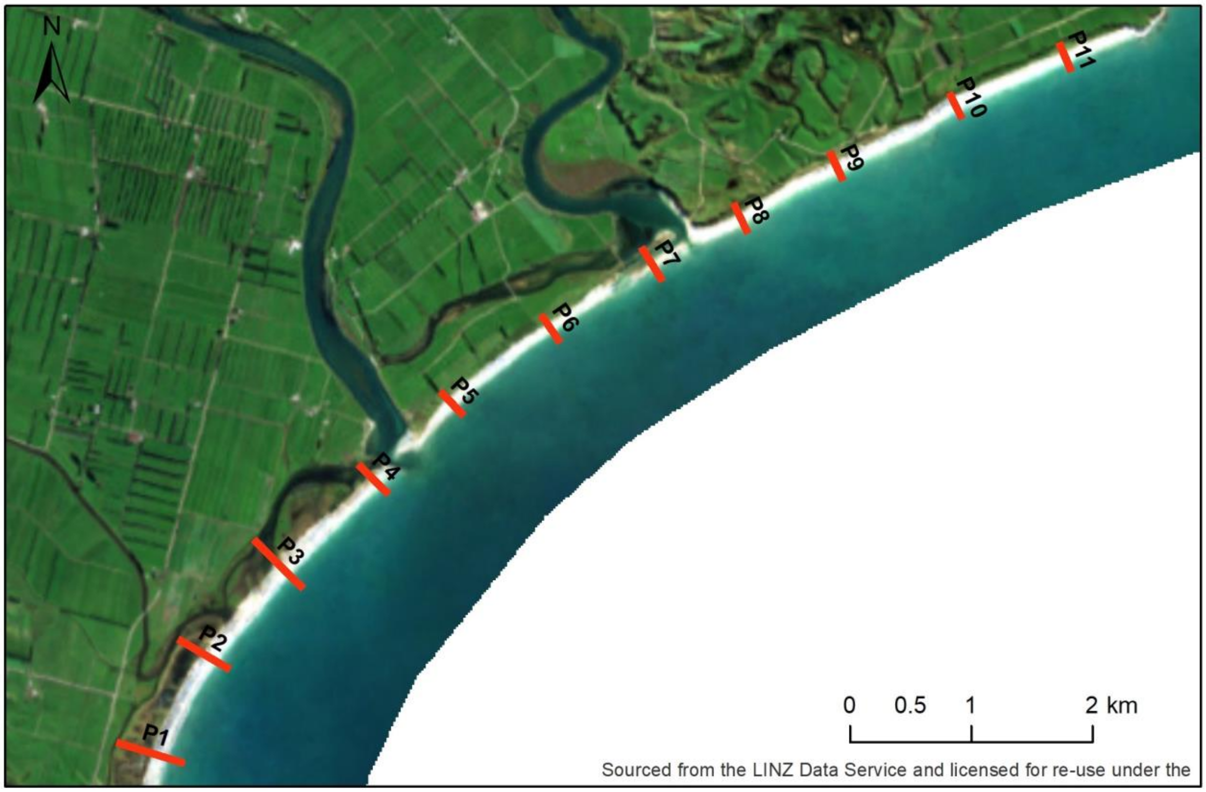
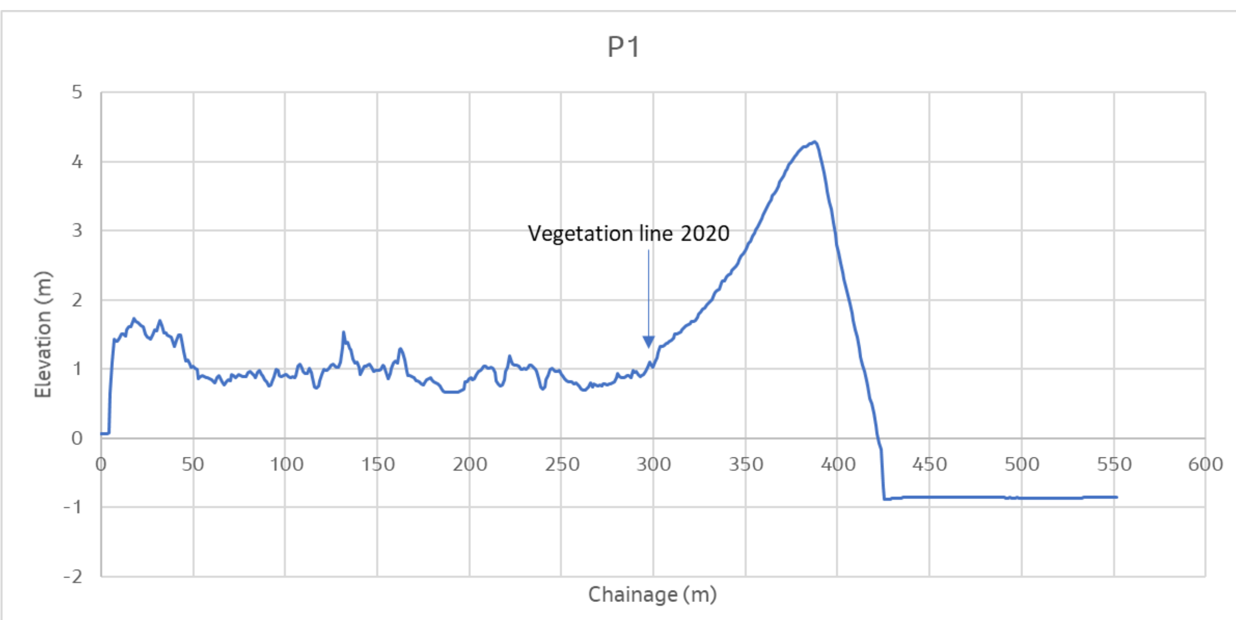
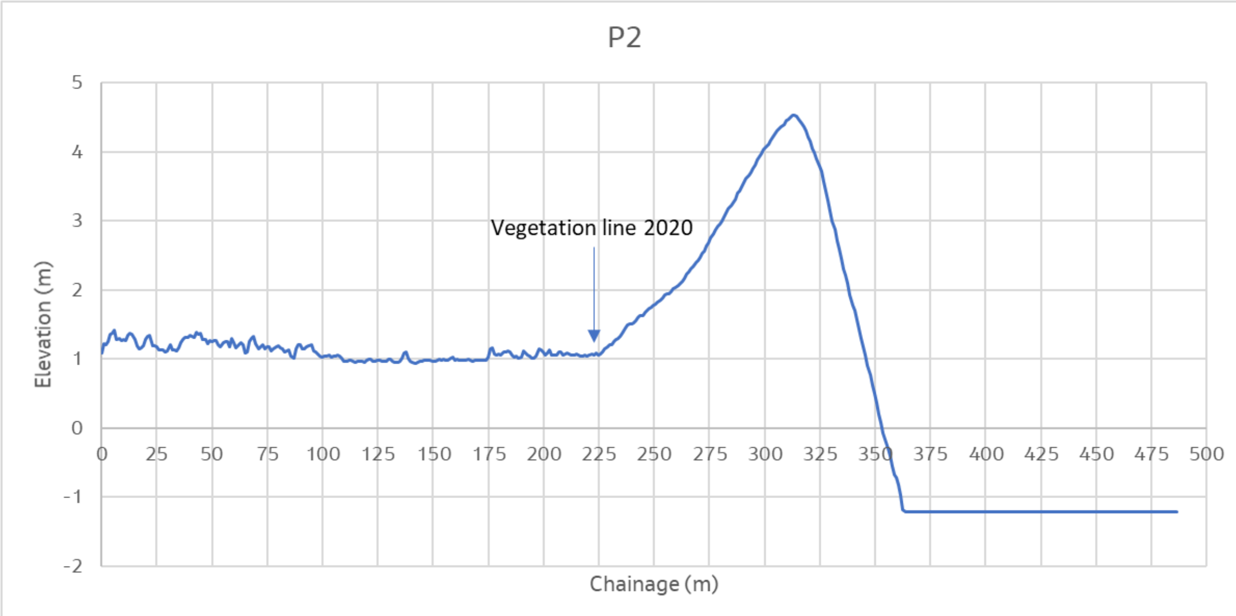
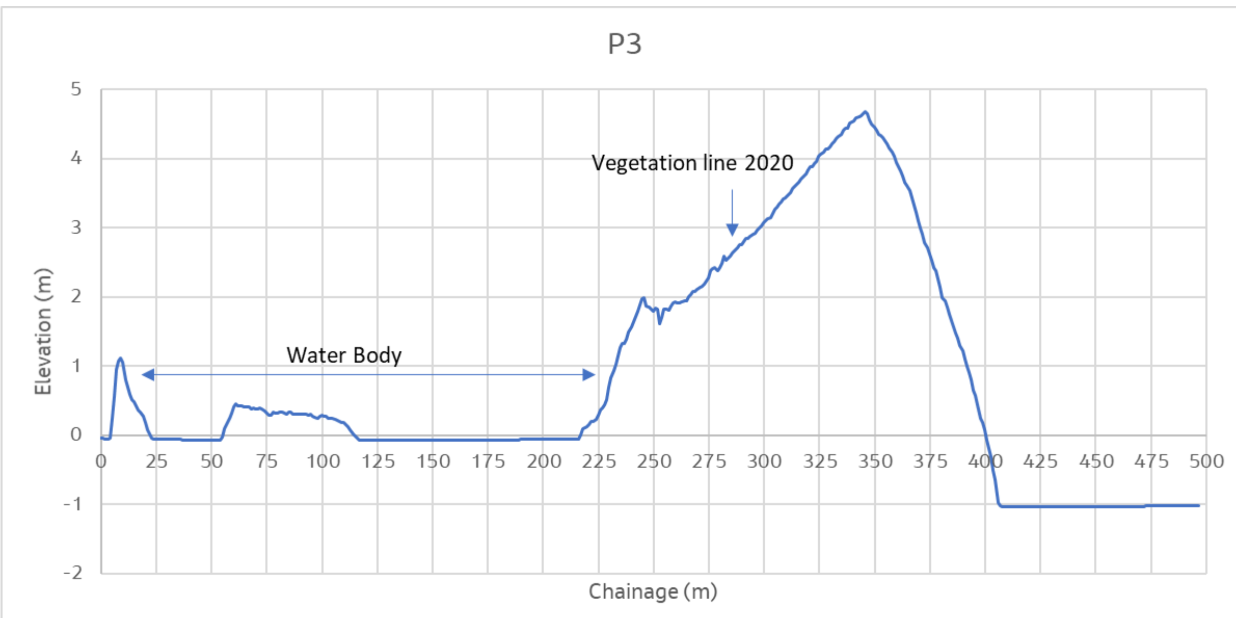
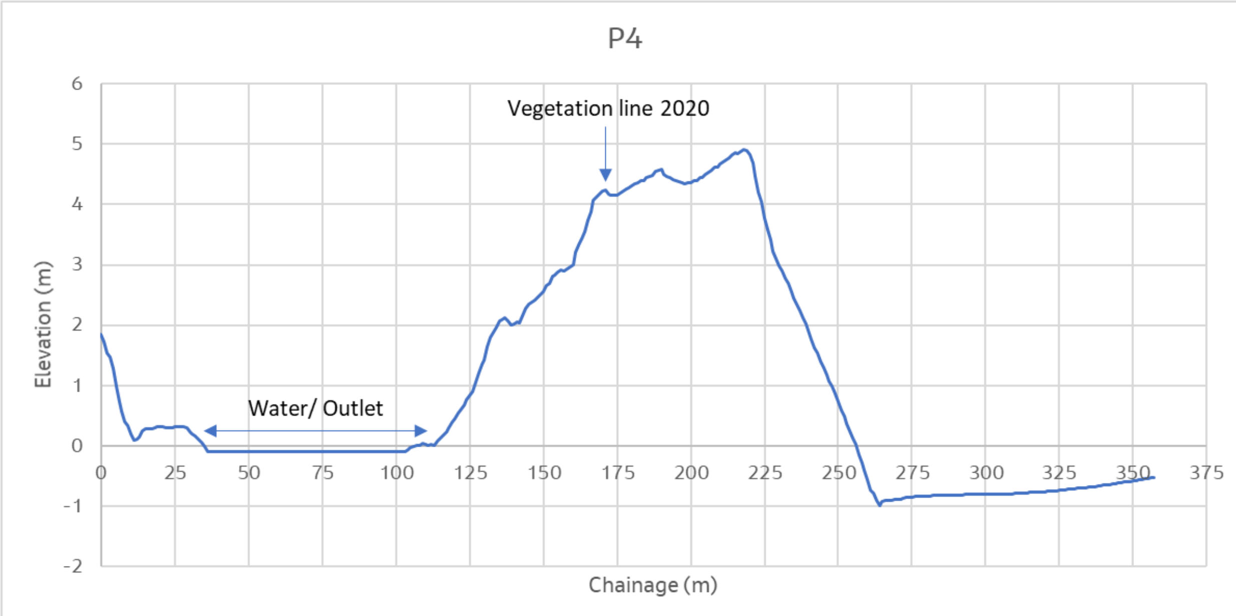


Figure 1: Location of beach transects taken from 2020 LiDAR

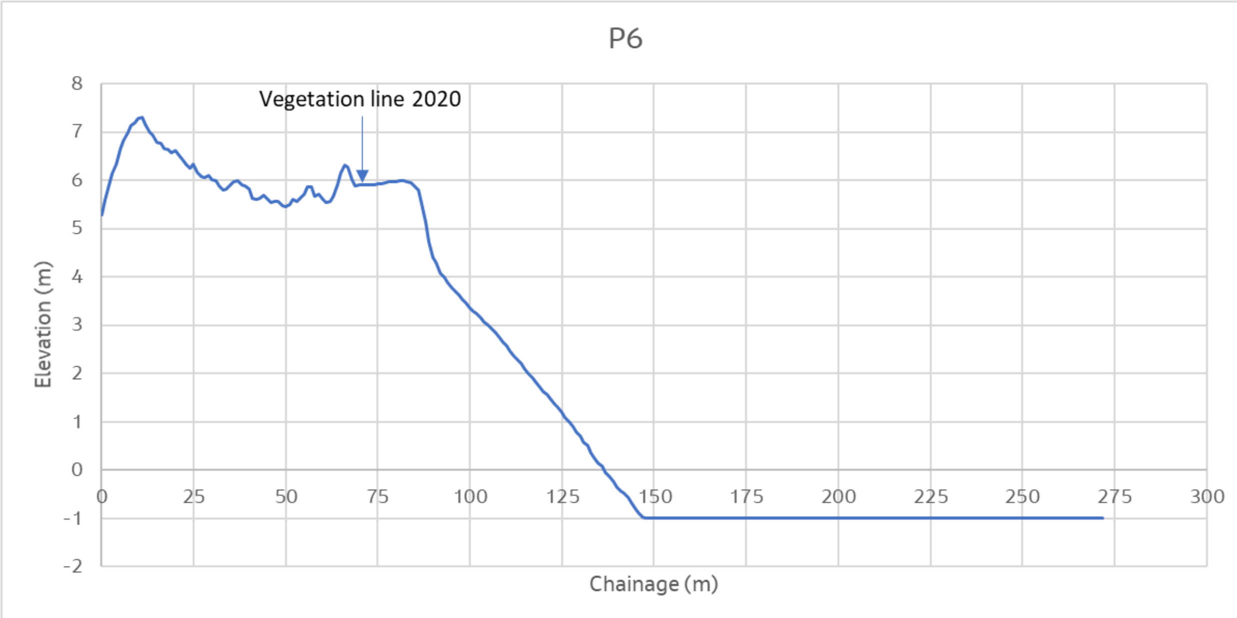


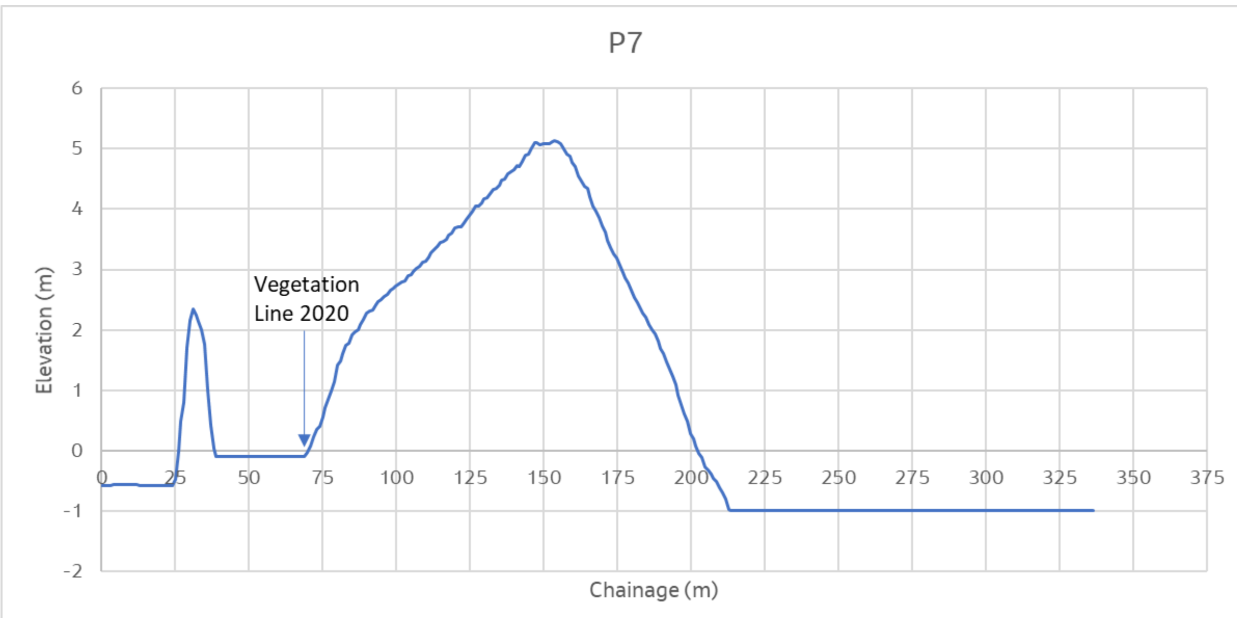

P2
Vegetation line 2020
Elevation (m)
Chainage (m)
5
4
3
2
1
0
-1
-2
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375
400
425
450
475
500

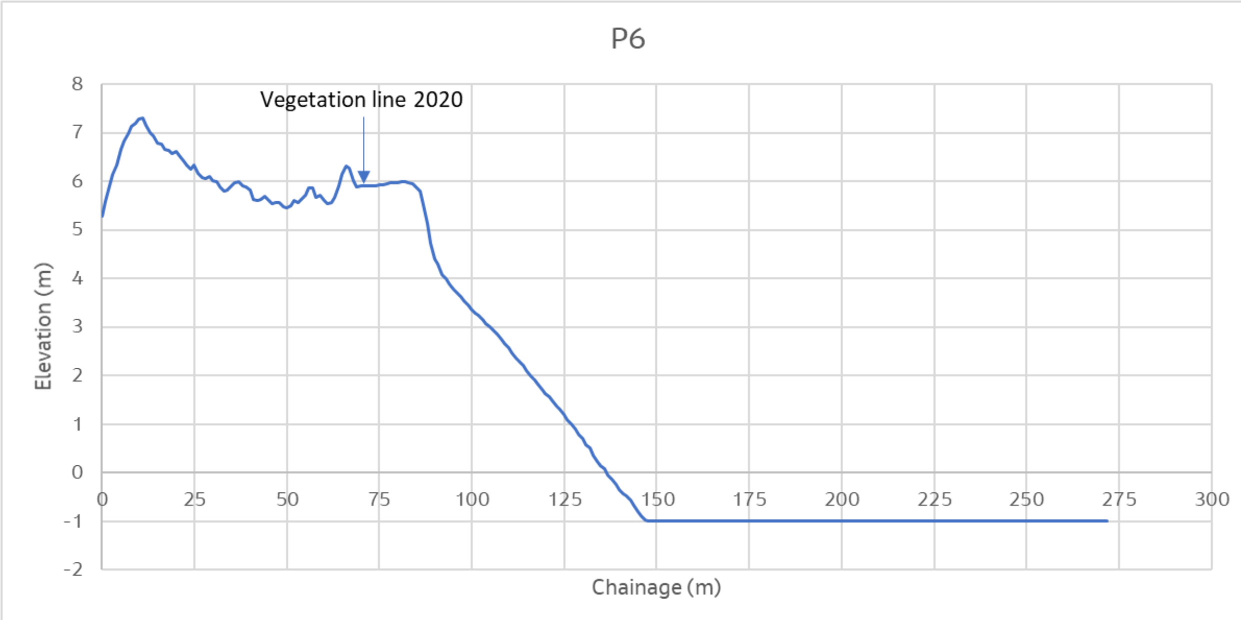

P3
Vegetation line 2020
Water Body
Elevation (m)
Chainage (m)
5
4
3
2
1
0
-1
-2
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375
400
425
450
475
500

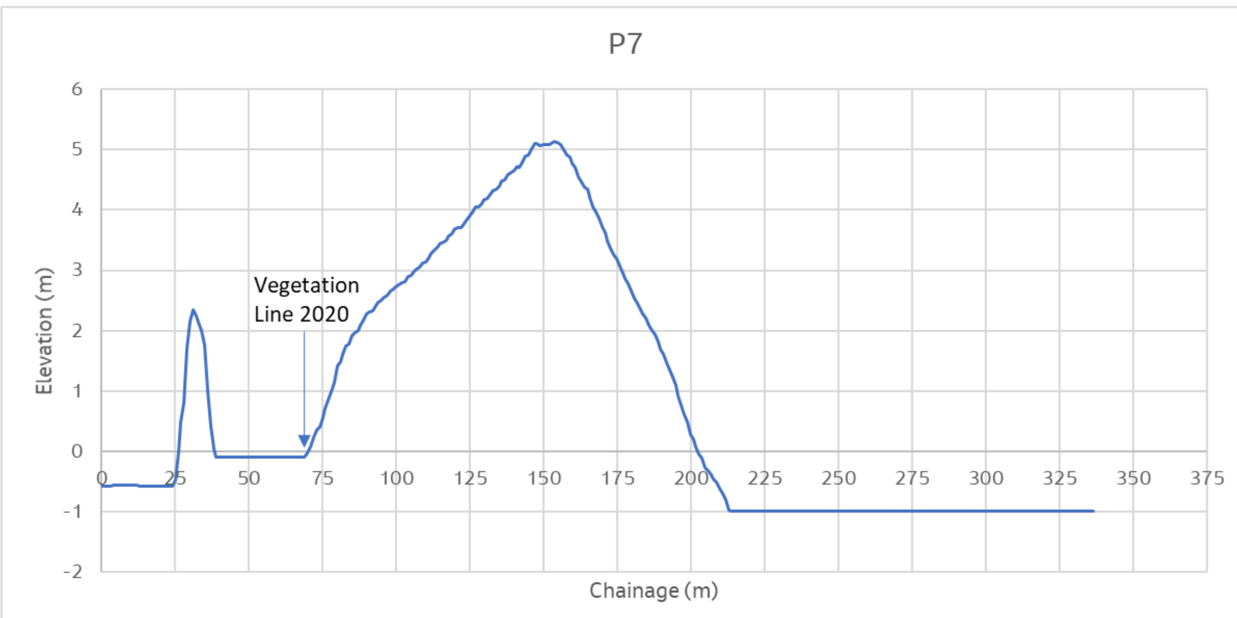

P4
Vegetation line 2020
Water/ Outlet
Elevation (m)
Chainage (m)
6
5
4
3
2
1
0
-1
-2
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375

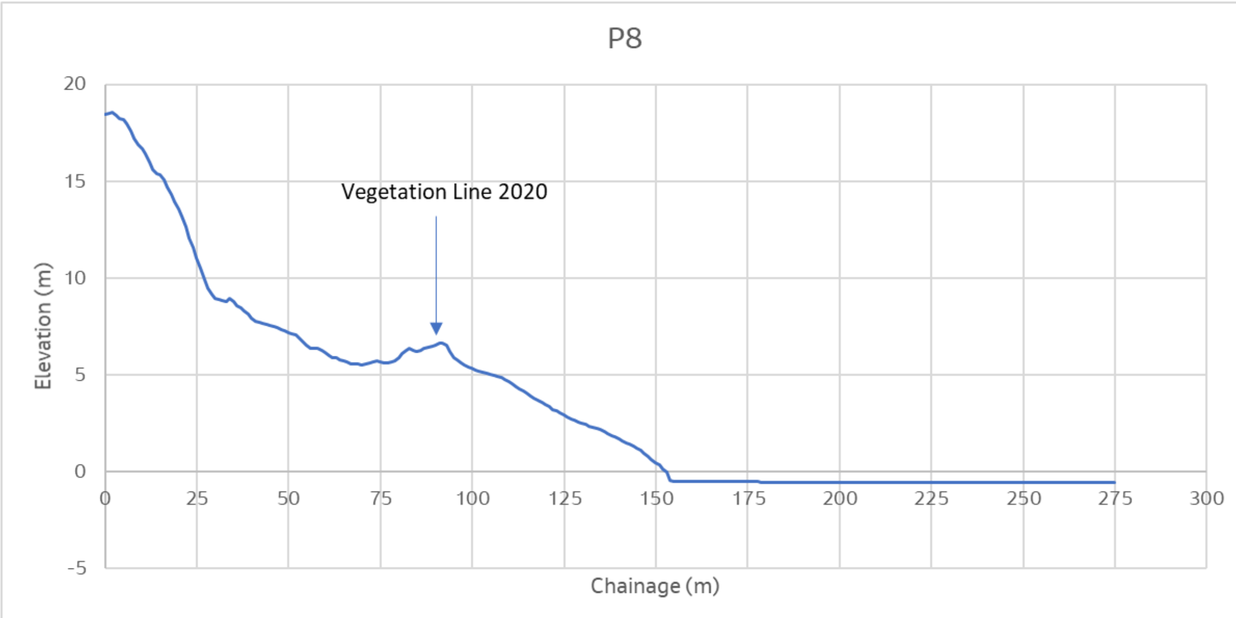

P5
Vegetation line 2020
Elevation (m)
Chainage (m)
8
7
6
5
4
3
2
1
0
-1
-2
0
25
50
75
100
125
150
175
200
225
250
275
300

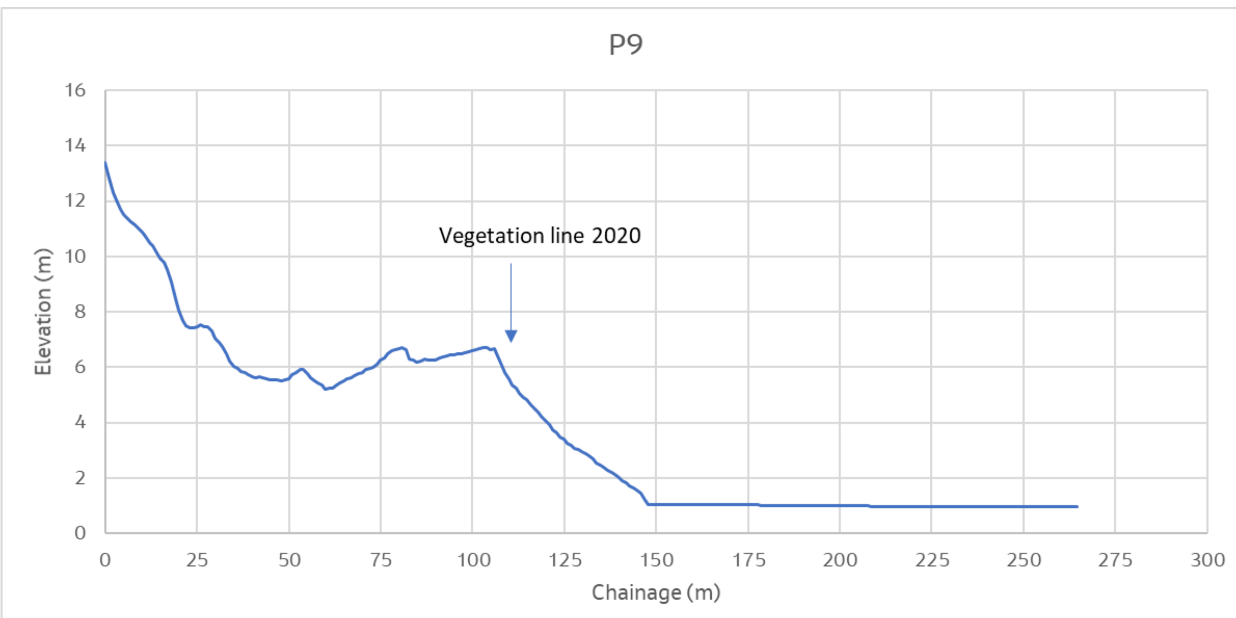

P6
Vegetation line 2020
Elevation (m)
Chainage (m)
8
7
6
5
4
3
2
1
0
-1
-2
0
25
50
75
100
125
150
175
200
225
250
275
300


P7
Vegetation
Line 2020
Elevation (m)
Chainage (m)
6
5
4
3
2
1
0
-1
-2
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375


P6
Vegetation line 2020
Elevation (m)
Chainage (m)
8
7
6
5
4
3
2
1
0
-1
-2
0
25
50
75
100
125
150
175
200
225
250
275
300


P7
Vegetation
Line 2020
Elevation (m)
Chainage (m)
6
5
4
3
2
1
0
-1
-2
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375


P8
Vegetation Line 2020
Elevation (m)
Chainage (m)
20
15
10
5
0
-5
0
25
50
75
100
125
150
175
200
225
250
275
300


P9
Vegetation line 2020
Elevation (m)
Chainage (m)
16
14
12
10
8
6
4
2
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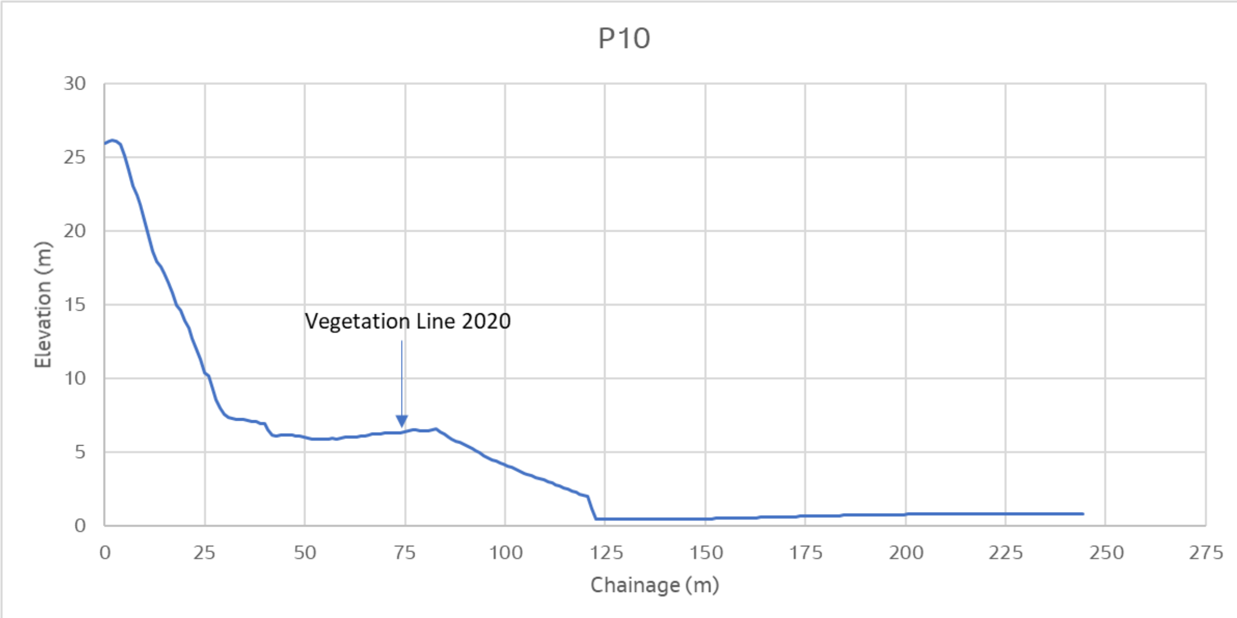
25
50
75
100
125
150
175
200
225
250
275
300


P10
Vegetation Line 2020
Elevation (m)
Chainage (m)
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20
15
10
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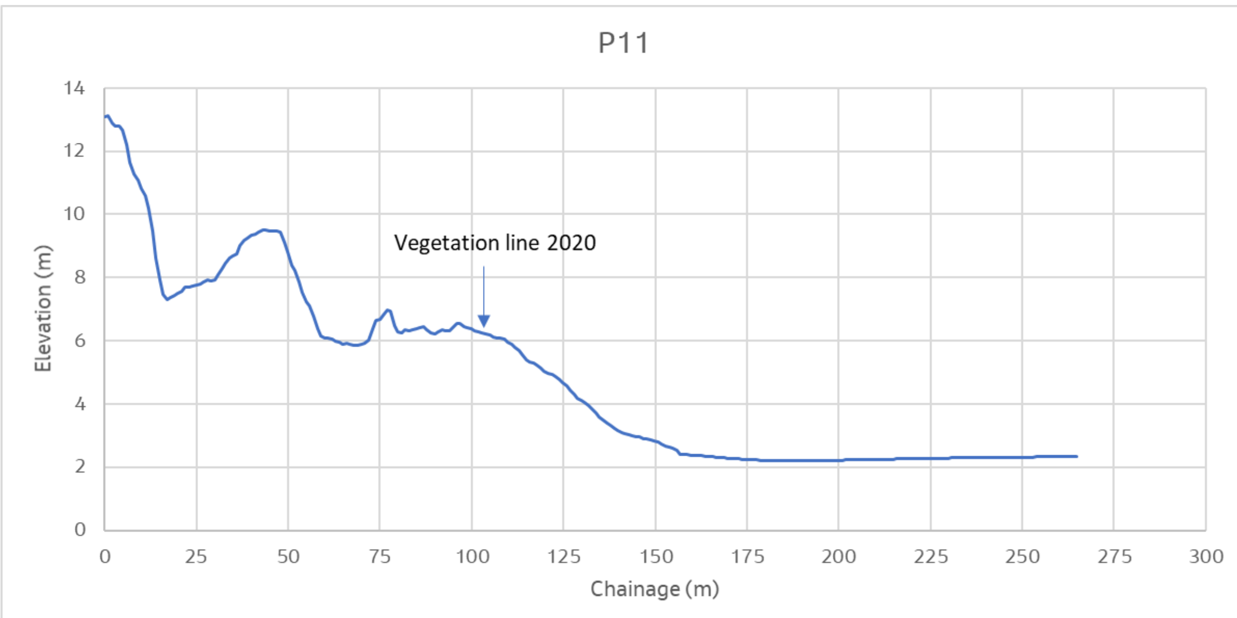
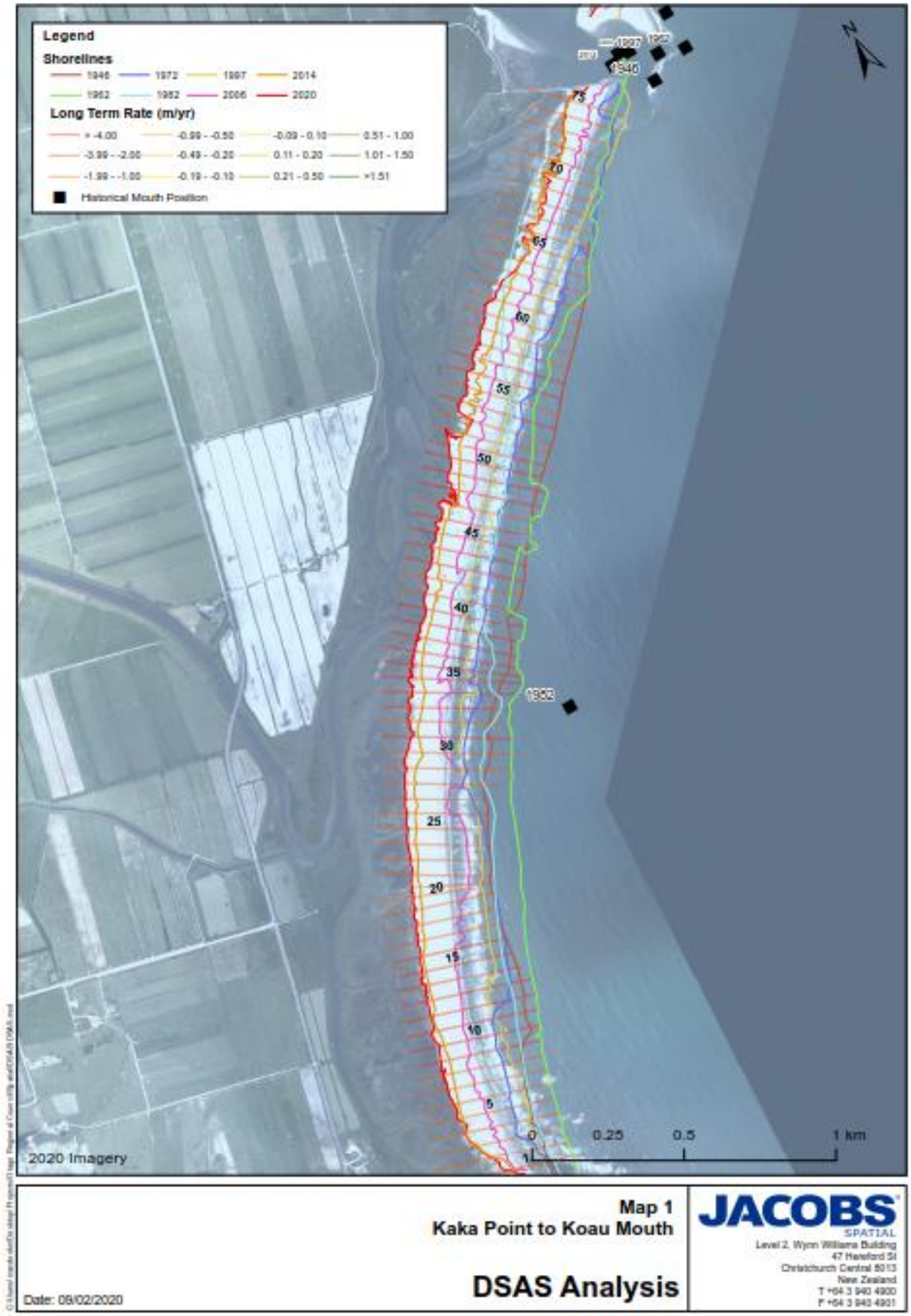
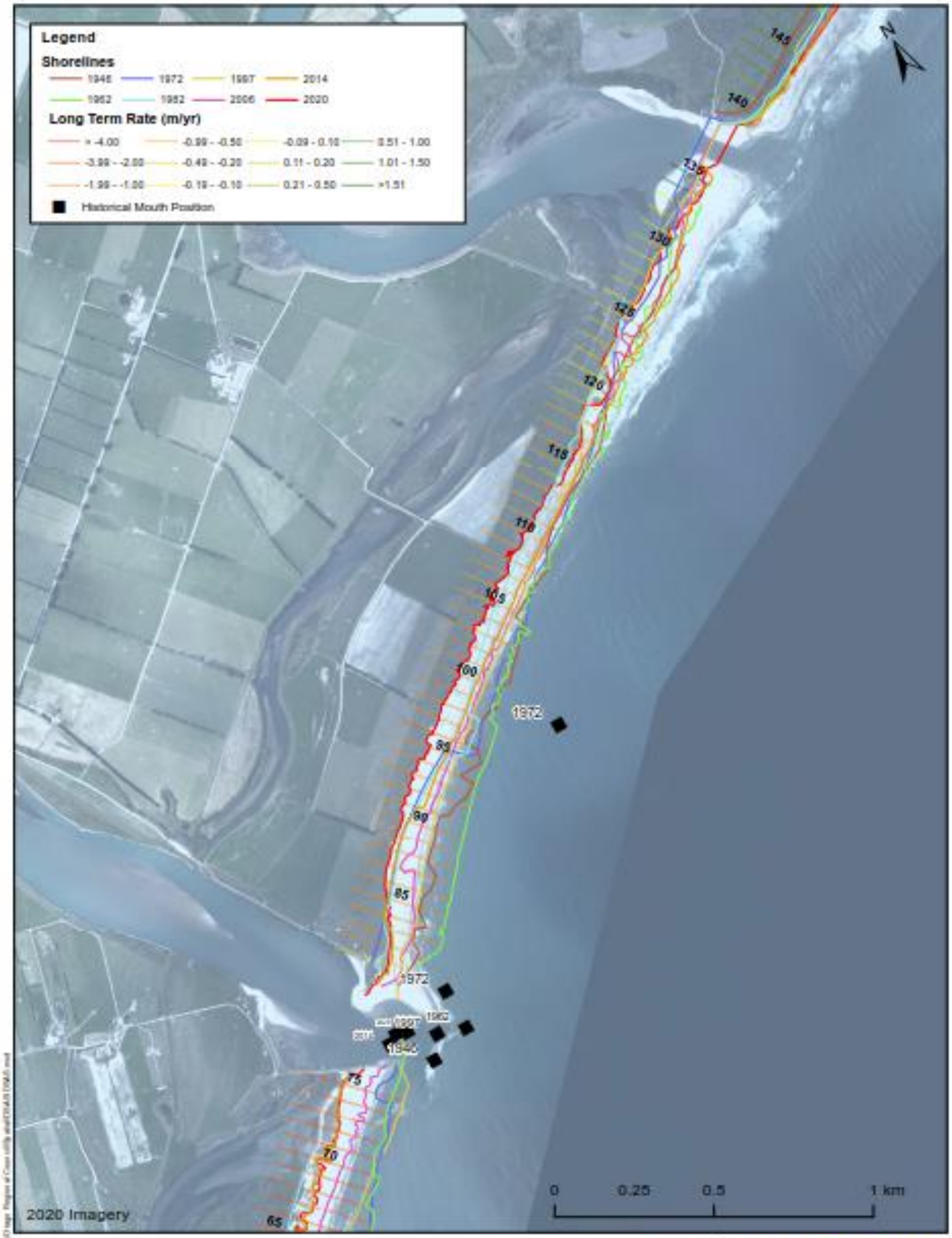
25
50
75
100
125
150
175
200
225
250
275


P11
Vegetation line 2020
Elevation (m)
Chainage (m)
14
12
10
8
6
4
2
0
0
25
50
75
100
125
150
175
200
225
250
275
300

Appendix C. Historical Shoreline Positions and DSAS Transect Locations


Legend
Shorelines
1946
1972
1997
2014
1962
1982
2006
2020
Long Term Rate (m/yr)
< -4.00
-0.99 - -0.50
-0.09 - 0.10
0.51 - 1.00
-3.99 - -2.00
-0.49 - -0.20
0.11 - 0.20
1.01 - 1.50
-1.99 - -1.00
-0.19 - -0.10
0.21 - 0.50
>1.51
Historical Mouth Position
1997
1962
1946
1982
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
1
0
0.25
0.5
1 km
2020 Imagery
Map 1
Kaka Point to Koau Mouth
DSAS Analysis
Date: 09/02/2020
JACOBS
SPATIAL
Level 2, Wynn Williams Building
47 Hereford St
Christchurch Central 8013
New Zealand
T +64 3 940 4900
F +64 3 940 4901

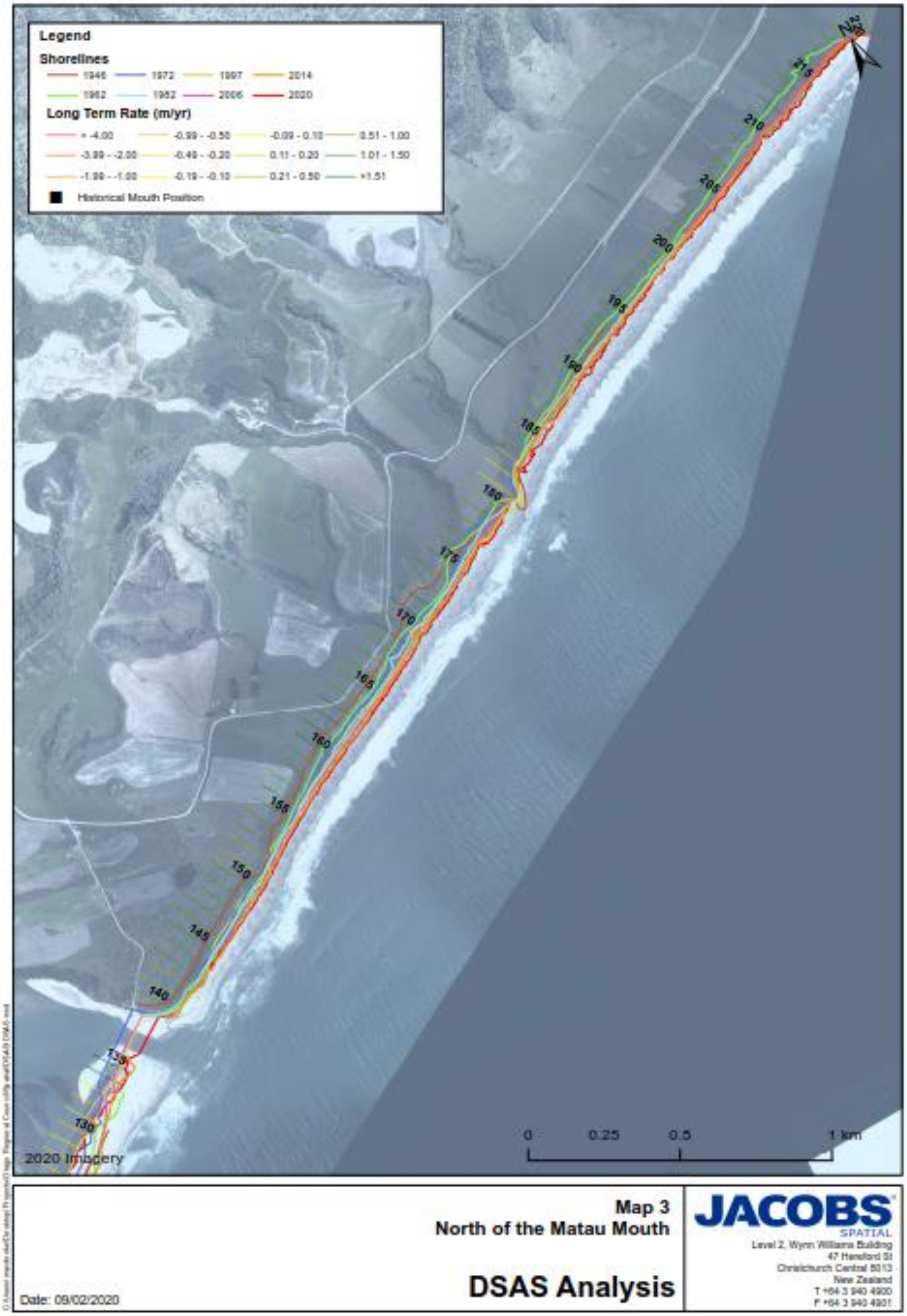


Map 2
Koau Mouth to Matau Mouth

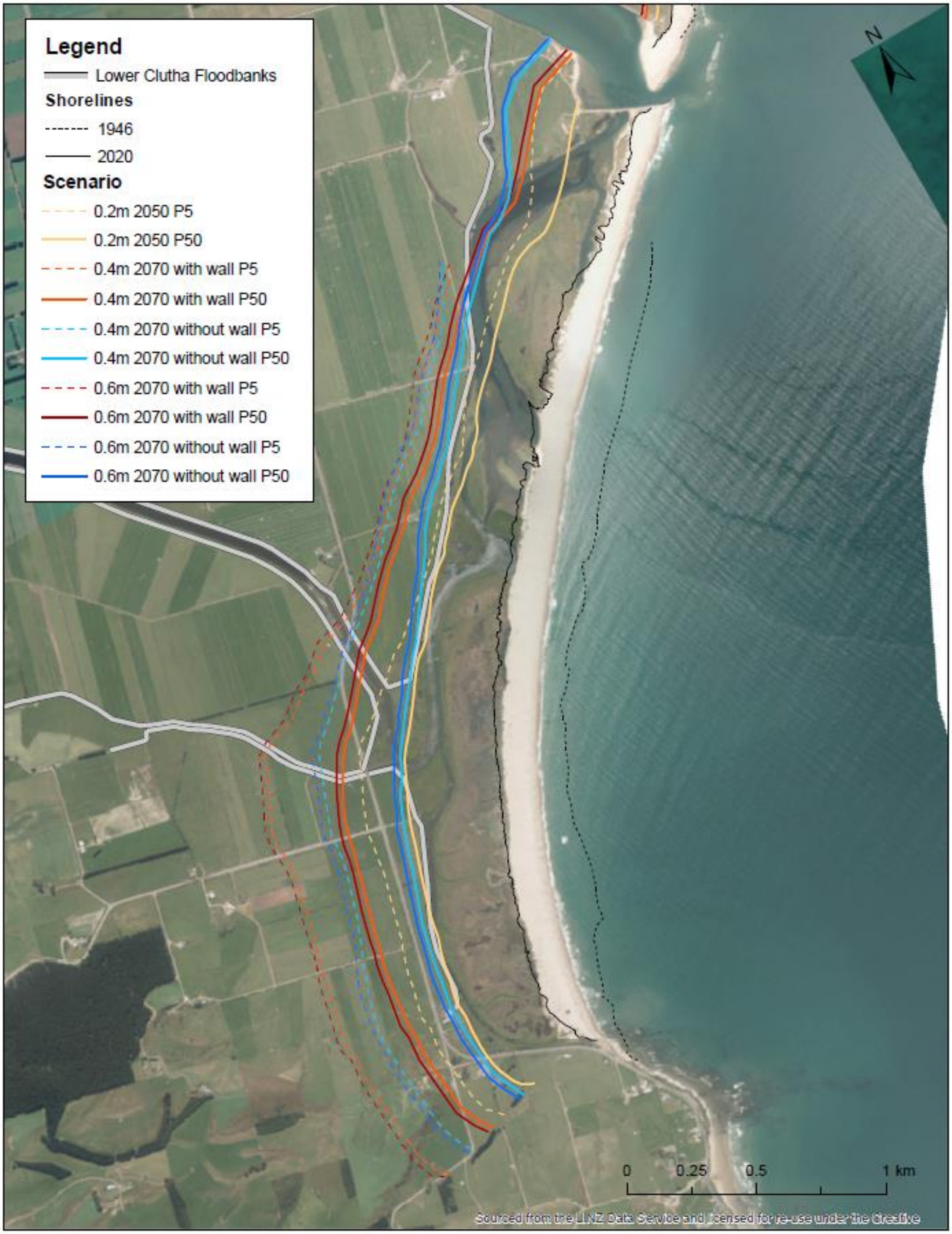
DSAS Analysis

Date: 09/02/2020

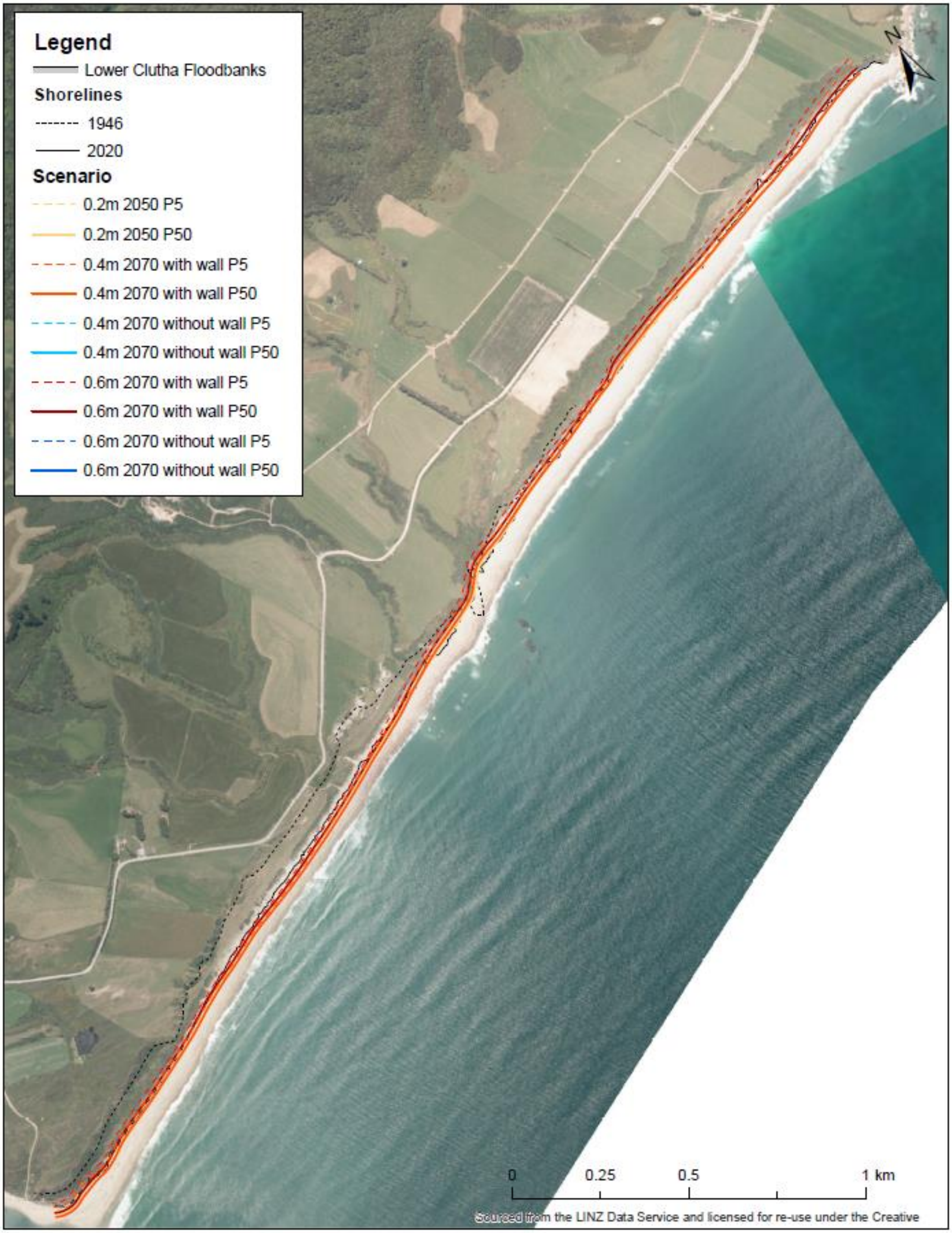
JACOBS SPATIAL
Level 2, Wynn Williams Building
47 Hereford St
Christchurch Central 8013
New Zealand
T +64 3 940 4900
F +64 3 940 4901


Legend
Shorelines
1946
1972
1997
2014
1962
1982
2006
2020
Long Term Rate (m/yr)
< -4.00
-3.99 - -2.00
-1.99 - -1.00
-0.99 - -0.50
-0.49 - -0.20
-0.19 - -0.10
-0.09 - 0.10
0.11 - 0.20
0.21 - 0.50
0.51 - 1.00
1.01 - 1.50
>1.51
Historical Mouth Position
2020 Imagery
0
0.25
0.5
1 km
Map 3
North of the Matau Mouth
DSAS Analysis
Date: 09/02/2020
JACOBS
SPATIAL
Level 2, Wynn Williams Building
47 Hereford St
Christchurch Central 8013
New Zealand
T +64 3 940 4900
F +64 3 940 4901

Appendix D. Projected Future Shoreline Positions for 2050 and 2070


Legend
Lower Clutha Floodbanks
Shorelines
1946
2020
Scenario
0.2m 2050 P5
0.2m 2050 P50
0.4m 2070 with wall P5
0.4m 2070 with wall P50
0.4m 2070 without wall P5
0.4m 2070 without wall P50
0.6m 2070 with wall P5
0.6m 2070 with wall P50
0.6m 2070 without wall P5
0.6m 2070 without wall P50
N
0
0.25
0.5
1 km
Sourced from the LINZ Data Service and licensed for re-use under the Creative


Legend
Lower Clutha Floodbanks
Shorelines
1946
2020
Scenario
0.2m 2050 P5
0.2m 2050 P50
0.4m 2070 with wall P5
0.4m 2070 with wall P50
0.4m 2070 without wall P5
0.4m 2070 without wall P50
0.6m 2070 with wall P5
0.6m 2070 with wall P50
0.6m 2070 without wall P5
0.6m 2070 without wall P50
N
0
0.25
0.5
1 km
Sourced from the LINZ Data Service and licensed for re-use under the Creative


Legend
Lower Clutha Floodbanks
Shorelines
1946
2020
Scenario
0.2m 2050 P5
0.2m 2050 P50
0.4m 2070 with wall P5
0.4m 2070 with wall P50
0.4m 2070 without wall P5
0.4m 2070 without wall P50
0.6m 2070 with wall P5
0.6m 2070 with wall P50
0.6m 2070 without wall P5
0.6m 2070 without wall P50
N
0
0.25
0.5
1 km
Sourced from the LINZ Data Service and licensed for re-use under the Creative

Appendix E. Logarithmic Spiral Curve Fitting Equations

The equation for a logarithmic spiral curve is generally given in polar coordinates, with the radius *r* being a function of the angle θ and the general form of the equation being

$$r = r_o e^{\theta \cot\alpha}$$

where as shown in the figure below,

r is a radius from the log-spiral centre to any point on the curve,

r_o is the radius from the log-spiral centre to the log-spiral origin,

θ is the angle between *r* and r_o measured from the log-spiral centre, and

α is the spiral angle, defined as the angle between the radius vector and the tangent to the curve at any point. For any curve, α is constant.

Appendix F. Stage One Inundation Assessment Memo (Jacobs, 2020)

Jacobs

Memorandum

Level 8, 1 Grey Street,
PO Box 10-283
Wellington, 6143
New Zealand
T +64 4 473 4265
F +64 4 473 3369
www.jacobs.com

Subject	**Inundation Assessment Stage One**	**Project Name**	Clutha Delta Coastal Hazards Assessment
Attention	Tim van Woerden, ORC	**Project No.**	IS328700
From	Kristin Stokes		
Date	September 1st, 2020		
Copies to	Kate Murray, Derek Todd, Craig Redmond		

Jacobs was engaged by Otago Regional Council to carry out a coastal morphology and climate change investigation for the Molyneux Bay – Clutha Delta area. This investigation will guide decision making regarding management of this coastline and delta area. As part of this investigation Jacobs were to assess the extent and magnitude of potential coastal inundation on the Lower Clutha Delta with future sea level rise (SLR). Our proposal was to carry out the inundation assessment in two stages:

- Stage One to assess whether the floodbanks are likely to be overtopped and whether further dynamic modelling of the overtopping is needed to assess the potential extent of coastal inundation.
- Stage Two using a hydrodynamic model assessment to estimate the extents and depths of inundation for those scenarios where overtopping is predicted to occur. Stage Two will only be carried out if the Stage One assessment concludes this is necessary.

This memo reports the findings of the Stage One inundation assessment and provides a recommendation on whether to proceed with the Stage Two hydrodynamic model approach.

1.1 Methodology

We used surveyed crest levels and compared these to a range of current and future annual recurrence interval (ARI) storm tide levels to identify locations where the crest is predicted to be overtopped. The following information was used:

- Otago Regional Council provided a Floodbank Crest Level Survey carried out by Terramark. The survey was carried out between 27th October and 17th November 2017. Crest Levels were provided in Otago Metric Datum Height as well as NZVD16 with Local Vertical datum correction factors.
- NIWA provided return high level water levels from the Green Island sea level gauge and wave data for the nearest hindcast cell to the Clutha River mouth scaled from the Banks Peninsula wave buoy. Jacobs used this data to calculate sea levels for a range of return periods, and with open coast and river mouth wave set ups. Further details of this analysis are provided in our draft report *Molyneux Bay and Clutha Delta Morphology Investigation – Rev B Final Draft.*
- Analysis of coastal morphology and climate change was carried out using a 'probabilistic' approach to determine the Projected Future Shoreline Positions (PFSP) with SLR. Figure 4 and Figure 5 in

Appendix A show the PFSP for locations along the coast. Further details of this analysis are provided in our draft report *Molyneux Bay and Clutha Delta Morphology Investigation – Rev B Final Draft.*

- All levels provided in this memo are in terms of DVD 1958. Mean Sea Level is 0.114m (DVD 1958).
- Unless otherwise stated the high water levels presented are P50 'most likely' levels

Surveyed crest levels were compared to a range of sea levels to determine current and future inundation potential. We used the P50 sea levels which are considered a 'most likely' level for a given return period. Where banks were not predicted to be overtopped in these 'most likely scenarios we also tested a P95 return period; these levels are considered 'very unlikely to be exceeded'.

The amount of SLR considered for each floodbank was informed by the assessment of future shoreline position. For example, as shown in Figure 4 (Appendix A), the potential future shoreline position approaches the Port Molyneux Floodbank location within 30 years and so the 50- and 90-year SLR projections were not considered.

1.2 Sea Level

Current and future high-water levels for a range of return periods and with future SLR are provided in Table 1. Details of the analysis are provided in our draft report *Molyneux Bay and Clutha Delta Morphology Investigation – Rev B final Draft*. There is greater range of SLR scenarios considered further in the future and there is overlap in the SLR scenarios considered between 50- and 100-years in the future.

Table 1: High water Return Levels (excluding wave set up) for P50 ('most likely') with possible Sea Level Rise for 2030, 2070 and 2120. No wave set up is included.

Timeframe and Year	Sea Level Rise (m)	Return Period (years)		
		10	100	200
Present Day	N/A	1.521	1.651	1.684
30 Years (2030)	0.2	1.721	1.851	1.884
50 Years (2070)	0.4	1.921	2.051	2.084
	0.6	2.121	2.251	2.284
100 Years (2120)	0.6	2.121	2.251	2.284
	0.8	2.321	2.451	2.484
	1.0	2.521	2.651	2.684
	1.35	2.871	3.001	3.034

Our analysis also considered wave set up. Some of the floodbanks are parallel to the coast, so in large events will also be exposed to further elevated water levels due to wave set up. Wave set up in the river mouths tends to be lower and so high water levels in the rivers will tend to be lower and we have estimated these water levels separately. The floodbanks assessed against open coast wave run up are shown in Figure 2. Open coast wave runup was considered for five transect locations along the coast. Two locations MB4 and MB5 are north of the Matau Branch where the ground rises steeply inland and so coastal inundation is not an issue.

- MB1 is located to the south of the Puerua River Deviation's southernmost extent.

- MB2 is located just south of the Puerua Outfall.
- MB3 is located between the Koau and Matau Branches of the Clutha River.

Figure 1: Location of ORC profile surveys (MB1-MB5), bathymetric transects, and beach profiles (P1-P11) created from 2020 Lidar used for this study

The water levels at each of the three transect locations are shown in Table 2.

Table 2: 100-year Return Period sea levels for open coast including wave set up

Year	Sea Level Rise (m)	Transect Location		
		MB1	MB2	MB3
Present Day	N/A	2.31	2.23	2.24
2030	0.2	2.51	2.43	2.44
2070	0.4	2.71	2.63	2.64
2070	0.6	2.91	2.83	2.84
2120	0.6	2.91	2.83	2.84
2120	0.8	3.11	3.03	3.04
2120	1.0	3.31	3.23	3.24

Year	Sea Level Rise (m)	Transect Location		
		MB1	MB2	MB3
2120	1.35	3.66	3.58	3.59

River mouths, inlets and the associated floodbanks are protected from the full wave set up, this is reflected in lower wave set up values being used for these locations. The values projected are shown in Table 3.

Table 3: Sea Levels including Wave Set Up Values (most likely) for River Mouths and Inlets

Year	Sea Level Rise (m)	100-year Return Period	200-year Return Period
Present Day	-	2.03	2.07
2030	0.2	2.23	2.27
2070	0.4	2.43	2.47
2070	0.6	2.63	2.67
2120	0.6	2.63	2.67
2120	0.8	2.83	2.87
2120	1.0	3.03	3.07
2120	1.35	3.38	3.42

1.3 Overtopping Assessment

The overtopping assessment is presented in Sections 1.4 and 1.5 below. Figure 2 shows the extents of the floodbanks considered and the allowances for setup applied in each section. Figure 3 shows the surveyed crest points for all the stopbanks considered, colour coded by the 100- year ARI high water levels and SLR for which they are exceeded. A detailed map and surveyed crest profile for each floodbank is provided in Appendix B. The profiles include the 100- and 200- year ARI water levels for a range of SLRs. For the inland river floodbanks their inland extent is graphed only until they are above the projected extreme high water level range. The difference between a 100- year and 200- year event is 0.04m which is much less than the differences with projected SLR.

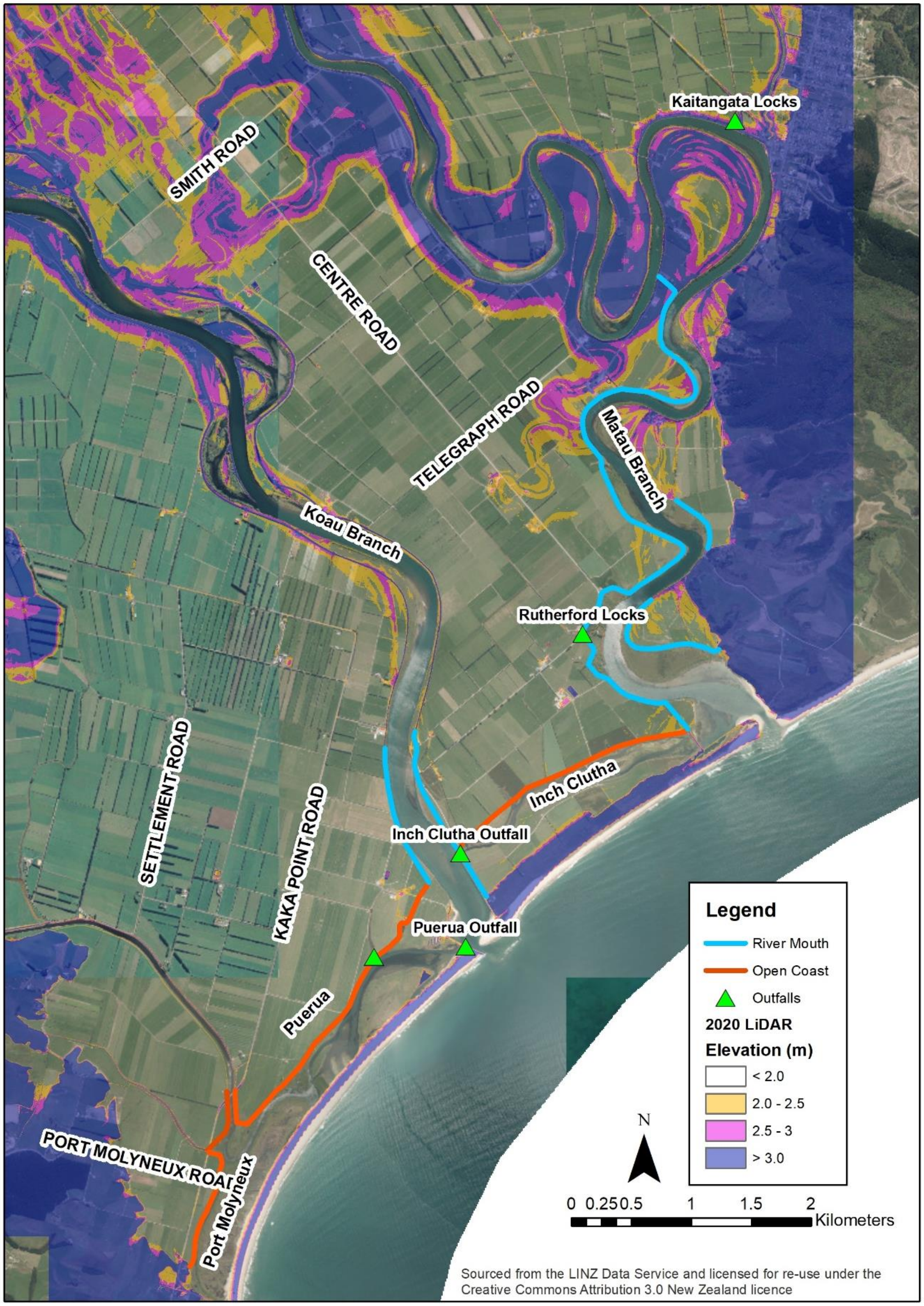


Figure 2: Open Coast and River Mouth/Inlet Floodbank locations

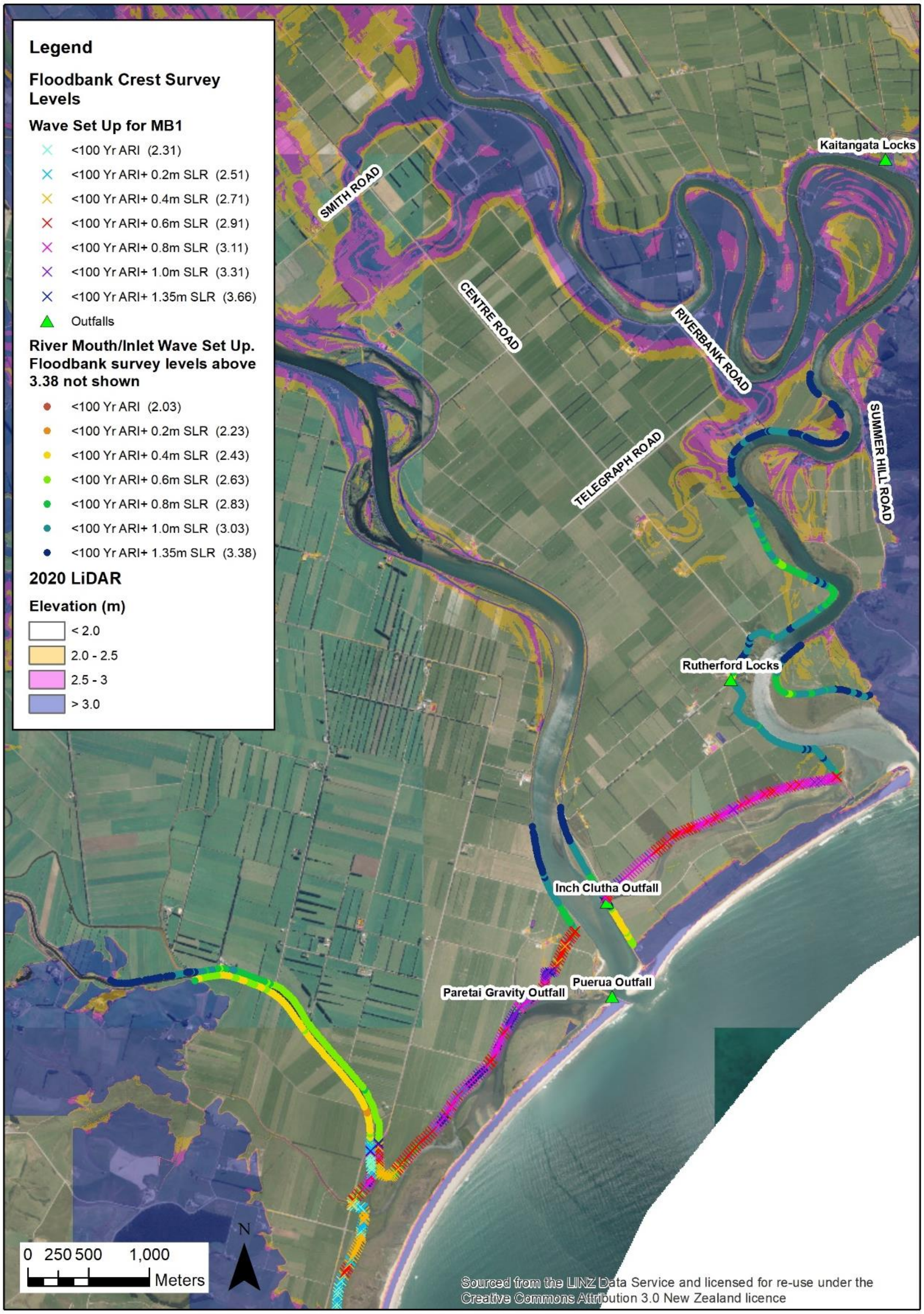


Figure 3: Surveyed Floodbank Crest Levels below 100 year ARI / SLR combination. Wave set up is included for open coast and river mouth / inlet as appropriate.

1.4 Open Coast Floodbank Assessment

The Port Molyneux, Puerua and Inch Clutha Floodbanks were assessed using the open coast levels provided in Table 2. Figure 2 shows that the northernmost Inch Clutha section of open coastal floodbank, between the Koau and Matau Branches is protected by coastal dunes that are higher than 3m, while the more southern Inch Clutha and Puerua floodbanks are less protected.

1.4.1 Port Molyneux Floodbank

The Port Molyneux floodbank is relatively low with a maximum surveyed crest elevation of 2.75m and parts of the floodbank below 2.5m elevation (DVD1958). The 100-year and 200-year ARI high water levels for current day and the crest alignment with chainage are shown in Figure 6 (Appendix B), with surveyed crest levels coloured according to the lowest ARI / SLR combination that they could be overtopped being displayed in Figure 7. As shown in these figures, the present 100-year ARI high water level with open coast wave set up at MB1 is 2.31m, which includes an estimated wave set up of 0.66m, would overtop the Port Molyneux floodbank in four locations (e.g. around chainage 120, 220-250, 780, and 1000-1080). However, with 0.2m SLR, these locations could be overtopped in a 10-year ARI event and around 70% of the total floodbank length could be overtopped in a 100-year ARI event (e.g. within 30 years). With 0.4m SLR, which could occur within 50 years, the length of overtopping in a 100-year ARI event increases to around 95% (e.g. over 1000m),

Figure 4 (Appendix A) shows the PFSP for 30- and 50-years' time for Southern Molyneux Bay. The figure shows that the shoreline position is projected to move rapidly inland, such that in 30 years' time the shoreline is projected to be very close to (0.2m SLR, P50 position) or beyond (0.2m SLR, P5 position) the current Port Molyneux floodbank alignment. Over a 50-year period the shoreline is projected to be landward of the current floodbank alignment under all Koau Mouth training wall and SLR scenarios (0.4 and 0.6m rise). Therefore, the likelihood of these banks providing flood protection beyond 30-50 years will most likely be compromised by coastal erosion as well as by bank elevation.

1.4.2 Puerua Deviation and Puerua River Floodbanks

As shown in Figure 8 (Appendix B), the Puerua Deviation left bank crest level varies between 3.5m and 2.5m; hence it is above the 100- year ARI current day high water level including wave set-up at MB2 of 2.23m (DVD1958). An approximate 200m section near the Puerua River Mouth (e.g. chainage 2600-2800m) would be overtopped in the 100 year ARI with 0.4m of SLR (2.63m, DVD 1958) while at the mouths of the Puerua River and the Koau Branch, the crest levels are below 2.83m and are likely to be overtopped with the 100-year ARI + 0.6m SLR (Figures 8 and 9).

The short section of right bank downstream of Kaka Point Road has two low points that could be overtopped in a current day 100-year ARI high water level including open coast wave set-up at MB2 of 2.23m (DVD1958).

From Figure 3, it can be seem that upstream of the Kaka Point Road Bridge, both the left and right floodbanks of the Puerua River are likely to be overtopped by the 100-year ARI high water level including river mouth / inlet wave set up with 0.2m of SLR on the right bank and with 0.4m of SLR on the left hand bank up to Wix Road.

From Figure 4 (Appendix A) the PFSP for 30-years' time along the Puerua Deviation is not projected to be as far inland as at Port Molyneux but is still projected to be very close to the floodbank alignment over large lengths of bank. Over a 50-year period the shoreline is projected to be landward of the current floodbank alignment under all scenario's, except for a 500m section from chainage 700m to 1200m. Therefore, the likelihood of the Puerua Deviation providing flood protection beyond 30-50 years will most likely be compromised by coastal erosion as well as by bank elevation.

1.4.3 Inch Clutha Floodbank

As shown in Figure 10 (Appendix B), the Inch Clutha Floodbank is located along the landward side of the Inch Clutha Bypass Floodway that joins the Koau and Matau Branch river mouths. Crest levels along the floodbank are relatively uniform, mostly ranging between 2.8m and 3.2m, therefore would not be overtopped in 100-year ARI high water levels, or with SLR up to 0.4m. With 0.6m of SLR, the floodbank elevation is exceeded by the 100-year ARI high water level including wave set up at 5 locations (at 80m, 600-800m, 900-980m, 1170m and 2200m). With 0.8m of SLR, the elevation of this floodbank is exceeded for around 80% of its length in the 100-year ARI high water level and is exceeded over its total length with 1.0m of SLR (Figure 11). However, as shown in Figure 2 and Figure 10, the dune field east of the floodway includes an extensive width with elevations greater than 3m, hence the source of any bank overtopping would have to be via the lower river channels and the Inch Clutha Bypass Floodway, rather than overtopping the beach.

As shown in Figure 5 (Appendix A) the shoreline is much more stable along this section of coast and the PFSP in 30- and 50-years' time is likely to remain on the seaward side of the bypass floodway channel.

1.5 River Mouth and Inlet Floodbank Assessment

The Koau Branch and Matau Branch Floodbanks were assessed using the river mouth and inlet wave set up levels, provided in Table 3.

1.5.1 Koau Branch Floodbanks

The Koau Branch left floodbank crest level extends to the dunes adjacent to the river mouth, with an outfall for the Inch Clutha channel at approximately chainage 400m. As shown in Figure 12, this floodbank rises to a high point of 3.4m where it is joined by the Inch Clutha floodbank at about chainage 450m. As shown in Figures 12 and 13 (Appendix B), around 65% of the length of this floodbank downstream of the Inch Clutha outfall lies below the 100-year ARI high water level + 0.4m SLR, and 100% lies below the same frequency event with 0.6m SLR. At 0.6m SLR, around 65% of the length could be overtopped in a 10-year ARI event. Upstream of the Inch Clutha outfall the left floodbank crest level rises gradually from about 2.8m at chainage 530m to 3.4m at chainage 1350m. For this section of floodbank, the crest would not be overtopped until sea level rose by >0.6m, with below 800m chainage likely to be overtopped by 100-year ARI high water with 0.8m of SLR, below 1000m chainage with 1.0m of SLR, and below chainage 1350m with 1.35m of SLR.

The Koau Branch right floodbank begins where the Puerua Deviation floodbank joins from the south west at chainage 400m with a crest level above the 100-year ARI + 0.8m SLR (2.83m). As shown in Figure 12, the right bank crest levels are very similar to the left bank crest levels from about chainage 500m, with the floodbank crest level rising gradually from about 2.8m at chainage 450m to 3.4m at chainage 1350m. Similar to the left bank, the right bank could be overtopped below chainage 800m in the 100-year ARI high water level + 0.8m SLR, with 1.0m of SLR below 1000m chainage, and with 1.35m of SLR below chainage 1450 (Figure 13).

1.5.2 Matau Branch Floodbanks

The Matau Branch right floodbank begins where it meets the Inch Clutha Floodbank. The level at that point is approximately 2.8m. As shown in Figure 14 (Appendix B), the survey shows the crest level fluctuates more than the Koau Branch floodbanks. There is a low point at the Rutherford Locks at chainage 1350m, and another low point at approximately chainage 3100m; these low points could be overtopped by the100-year ARI high water level + 0.6m SLR (including river mouth / inlet wave set up). As shown in Figure 15, downstream of Rutherford Locks the crest levels are largely below the 100-year ARI high water level with 1.0m of SLR. Between chainage 1450m and 5000m the crest level fluctuates mostly between 2.8m and 3.4m, apart from the low point at chainage 3100m mentioned above. Much of this section could be overtopped in the 100-year ARI + 1.0m SLR high water level at all chainages below 5200m and all could be overtopped by the 200-year ARI + 1.35m SLR of 3.42m. Above chainage 5200m the crest level rises above 3.4m and remains above the 200-year ARI + 1.35m SLR upstream of this point. Figure 14 shows that above chainage 5000m the LiDAR ground levels are mostly above 3m, providing additional protection from overtopping due to high sea levels.

As shown in Figure 14, the ground on the left bank of the Matau Branch between the branch mouth and Kaitangata township rises into hills, with only a few sections of low-lying land where floodbanks are present. While there are only short sections of formal flood banks, high-ground above 3m provides protection from coastal inundation. The Matau Branch left floodbank section opposite the Rutherford Lock (referred to as the At Mouth Section) is between 2.8m and 3m, so parts could be overtopped by high water levels above the 100-year ARI high water level with 1.0m of SLR. There is a low point at chainage 750m which will be overtopped by the 100-year ARI high water level + 0.6m SLR (including river mouth / inlet wave set up) of 2.67m.

The short Summer Hill Section of left floodbank between chainage 2150m and 2700m is between 2.25m and 2.65m, hence could be overtopped by the 100-year ARI high water level + 0.6m SLR of 2.67m.

The next section of left floodbank is near the Kaitangata township, which is above the highest flood level considered of 200-year ARI high water level + 1.35m of SLR (3.42m), apart from one low point adjacent to Water Street between Dartmouth Street and Berry Street, which could result in inundation of the road and the triangular shaped area bounded by Water Street, Eddystone Street and Berry Street. LiDAR shows that other properties are likely to be above the inundation level.

1.6 Summary

A summary of our findings for the Stage One assessment for the main floodbanks is provided below. These findings are based on simple comparison of surveyed crests with present and future sea levels. It is noted that the difference between the 100-year and 200-year ARI high water levels is only 0.04m, which is much lower than the range of SLR scenarios for different timeframes up to 100 years (e.g. 0.2m by 2050, up to maximum of 1.35m by 2120). Therefore, this summary focus on the 100-year ARI results.

- Port Molyneux Floodbank – some small sections of the floodbank are below the current day 100-year ARI high water level with wave set up (2.31m). With SLR, around 70% are below the 100-year ARI with rise of 0.2m (e.g. by 2050), and almost the entire length (95%) are likely to be overtopped in the same frequency event with + 0.4m SLR (e.g. 2.71m with wave set-up).
- Puerua Deviation Floodbank – all left banks are above the present day 100-year ARI high water level with wave set-up, while the right bank downstream of Kaka Point Road has two low points that could be overtopped in this magnitude event with current sea levels. Approximately 200m of left bank near the Puerua River Mouth would be overtopped in the 100 year ARI with 0.4m of SLR (2.63m, DVD 1958) while at the mouths of the Puerua River and the Koau Branch the crest levels are below 2.83m and are likely to be overtopped with the 100-year ARI + 0.6m SLR On the Puerua River, upstream of the Kaka Point Road Bridge, overtopping by the 100-year ARI high water level including river mouth / inlet wave set up with 0.2m of SLR is likely to occur on the right bank, and with 0.4m of rise on the left hand bank up to Wix Road.
- Inch Clutha Floodbank – crest levels along this floodway would not be overtopped in present day 100-year ARI high water levels, or with SLR up to 0.4m. With 0.6m of SLR, the floodbank elevation is exceeded by the 100-year ARI high water level including wave set up at 4 locations, with 0.8m of SLR, the floodbank elevation is exceeded for around 70% of its length for this frequency event increasing to 100% with 1.0m of SLR. However, the dune field east of the floodway includes an extensive width with elevations > 3m, hence the source of any bank overtopping would have to be via the lower river channels and the Inch Clutha Bypass floodway, rather than overtopping the beach.
- Koau Branch - left bank floodbank below where the Inch Clutha floodbank joins could be overtopped 100-year ARI high water level + 0.6m SLR (including river mouth / inlet wave set up). Upstream of the Inch Clutha floodbank both left and right banks downstream of chainage 800 could be overtopped by the 100-year high water level + 0.8m SLR, below 1000m chainage with 1.0m of SLR and below chainage 1350-1450m with 1.35m of SLR.
- Matau Branch – right bank low points at Rutherford Locks and chainage 3100m are overtopped by the the100-year ARI high water level + 0.6m SLR (including river mouth / inlet wave set up). Below chainage 48000m much of the right bank crest levels are below the 100-year ARI + 1.0m SLR high water level. Along much of the left bank there is no formal floodbank protection, but the higher ground level provides protection from coastal inundation. The left bank At Mouth section has parts that will be overtopped by high water levels above the 100-year ARI high water level with 1.0m of SLR (including river mouth / inlet wave set up). The left bank Summer Hill Section will be overtopped by the 100-year ARI high water level + 0.6m SLR (including river mouth / inlet wave set up) of 2.67m.

1.7 Recommendations

This Stage One inundation assessment shows that overtopping is likely for the Port Molyneux floodbanks by the100-year ARI high water level + 0.2m SLR with wave set up (e.g. within 30 years), and for the Puerua and Inch Clutha floodbanks with +0.4m SLR (e.g. within 50 years). The more protected river mouths are at risk of overtopping in the 100-year ARI with SLR from 0.6m to 1.35m (including river mouth / inlet wave set up), which are not likely to occur until 50-100 years in the future.

On this basis we recommend that the Stage Two hydrodynamic model inundation assessment be carried out, with the events to be modelled being agreed with Otago Regional Council. However, we have further recommended that the modelling should consider the following high water scenarios In making these recommendations, we note that the difference between the 100-year and 200-year ARI high water levels is only 0.04m, while the range of SLR is from 0.2m to 1.35m so the sea level range has a much greater influence on the frequency and extent of future inundation. Therefore, the modelling is likely to show very little difference between 100- and 200-year high water levels and the recommended scenarios are primarily limited to 100-year ARI high water levels including the appropriate wave setup allowances.

We further note that the projected shoreline positions are limited to SLR of up to +0.6m over 50 years, and do not include the scenarios of higher SLR over longer periods due to the uncertainty with the future and influence of the mouth training walls on shoreline position beyond this timeframe, particularly for the southern Koau mouth training wall. Hence, our recommendation is to model to following events and shoreline positions:

- 100-Year ARI with 0.2m SLR and projected 2050 shoreline position.
- 100-Year ARI with 0.4m SLR and projected 2070 shoreline position.
- 100-Year ARI with 0.6m SLR and projected 2070 shoreline position.
- 50-Year ARI with 0.6m SLR and projected 2070 shoreline position.

For all scenarios, the river flow condition to be included in the models would be as agreed with Otago Regional Council.

Appendix A: Projected Future Shoreline Positions (from *Molyneux Bay and Clutha Delta Morphology Investigation – Rev B Final Draft)*

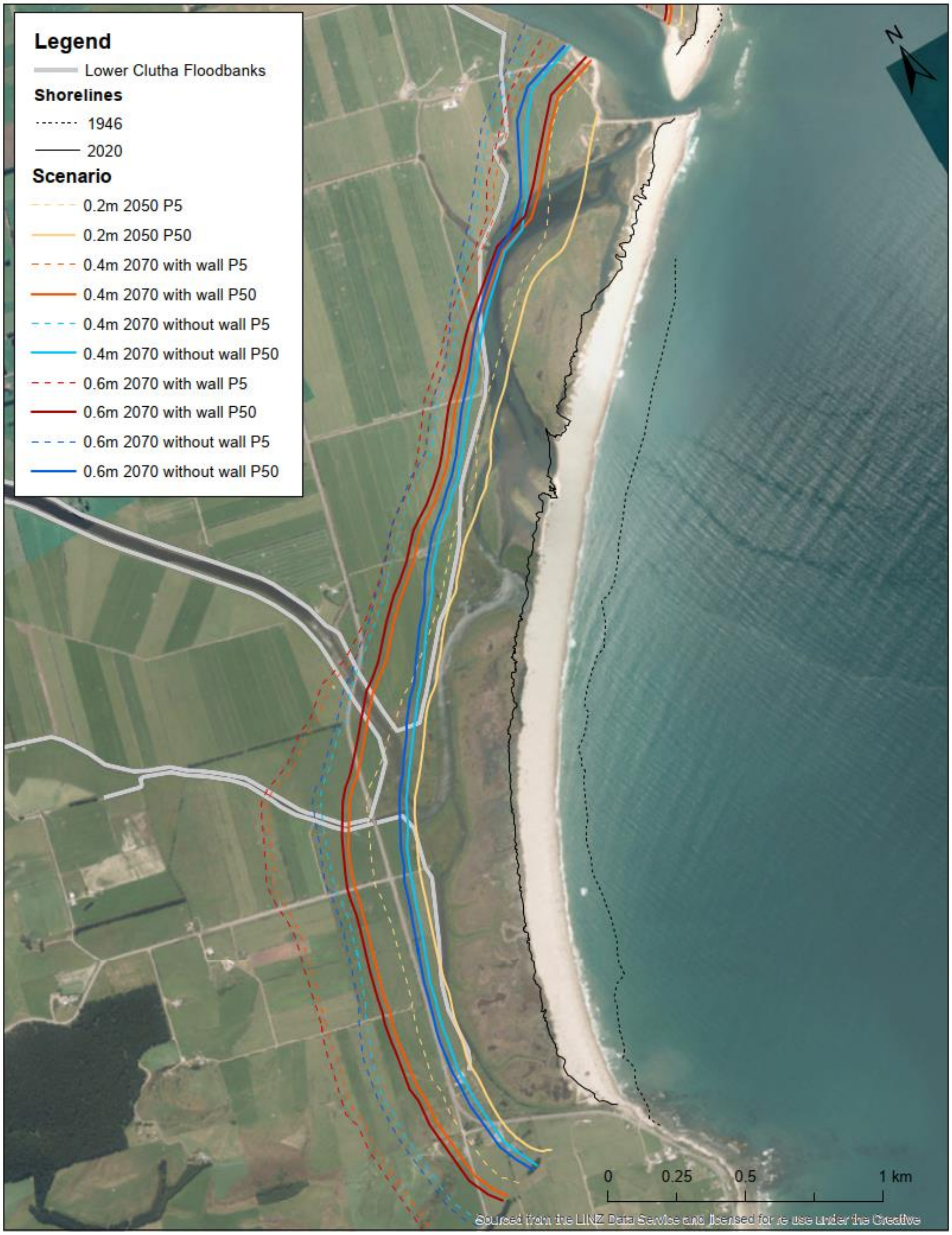
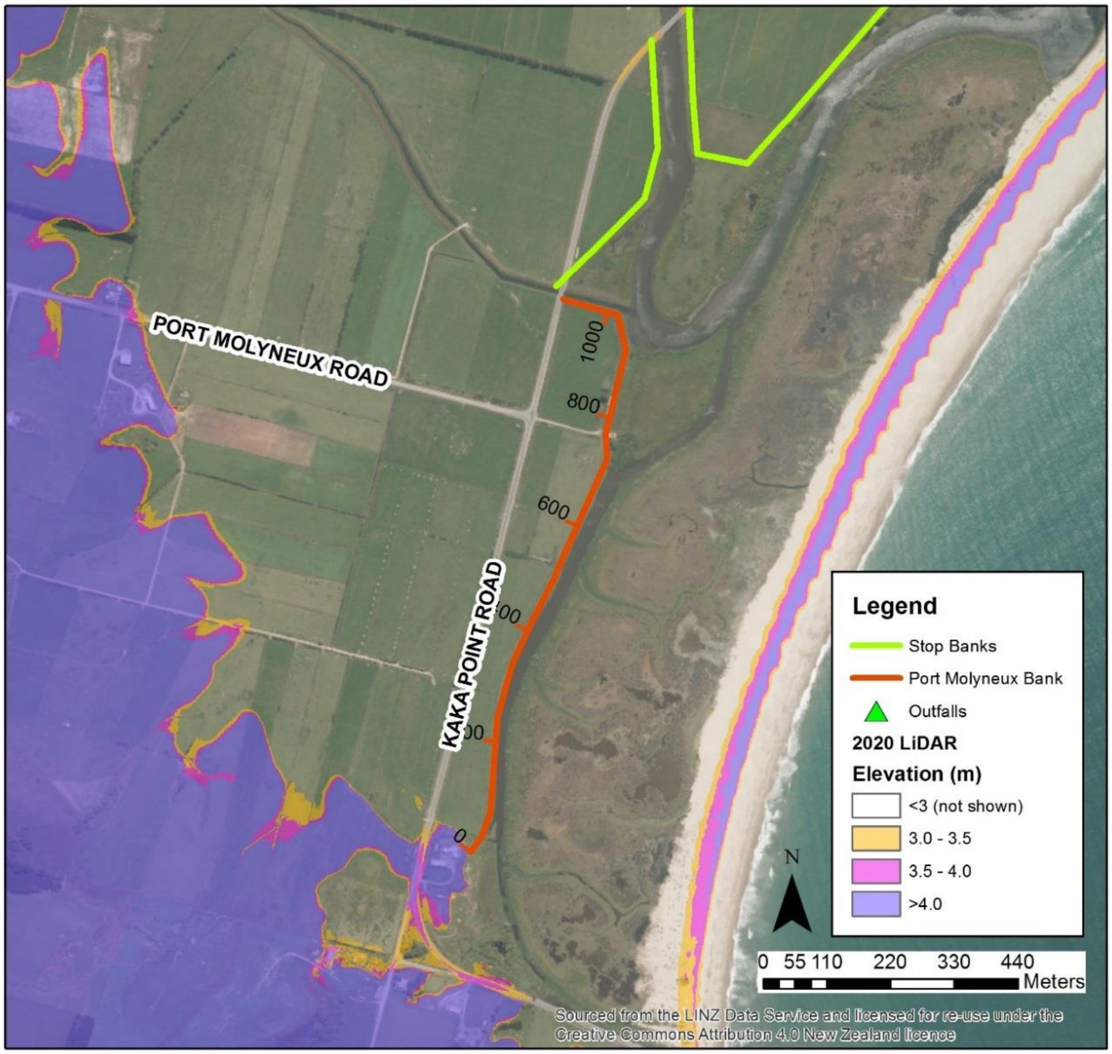


Figure 4: Port Molyneux to Puerua PFSP for P5 and P50 with and without training wall

Figure 5: Koau to Matau Branches PFSP for P5 and P50 with and without training wall

Appendix B: Map of floodbank crest elevations and ARI /SLR combinations where overtopping would occur under 'Bathtub modelling'



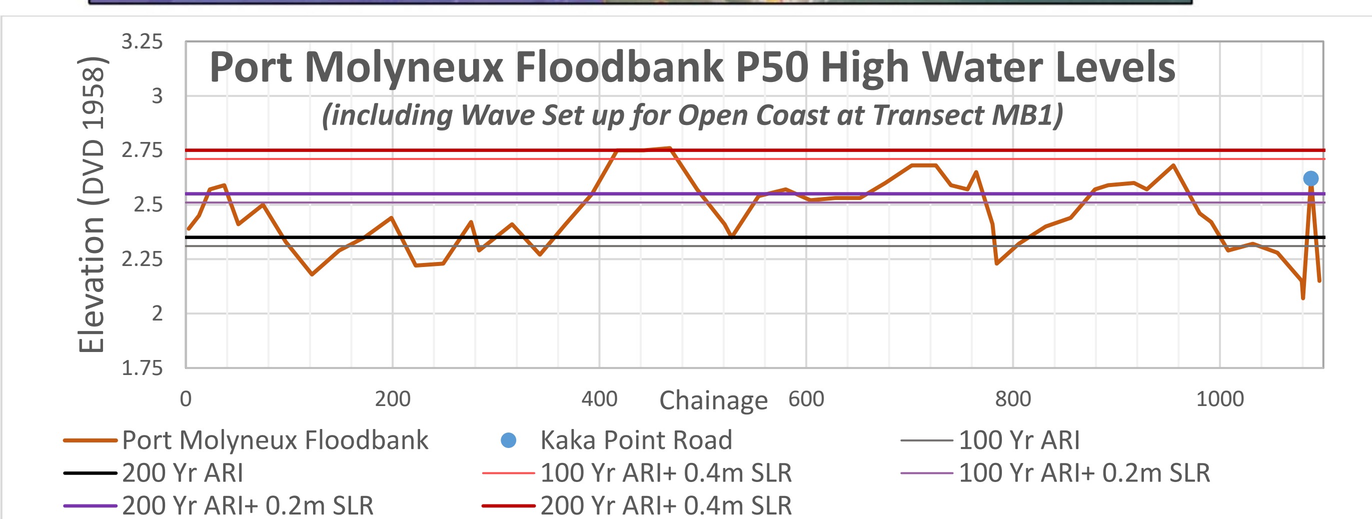


Figure 6: Port Molyneux Floodbank P50 inundation levels (including wave set up for open coast at MB1) for 100 year and 200 year Return Periods

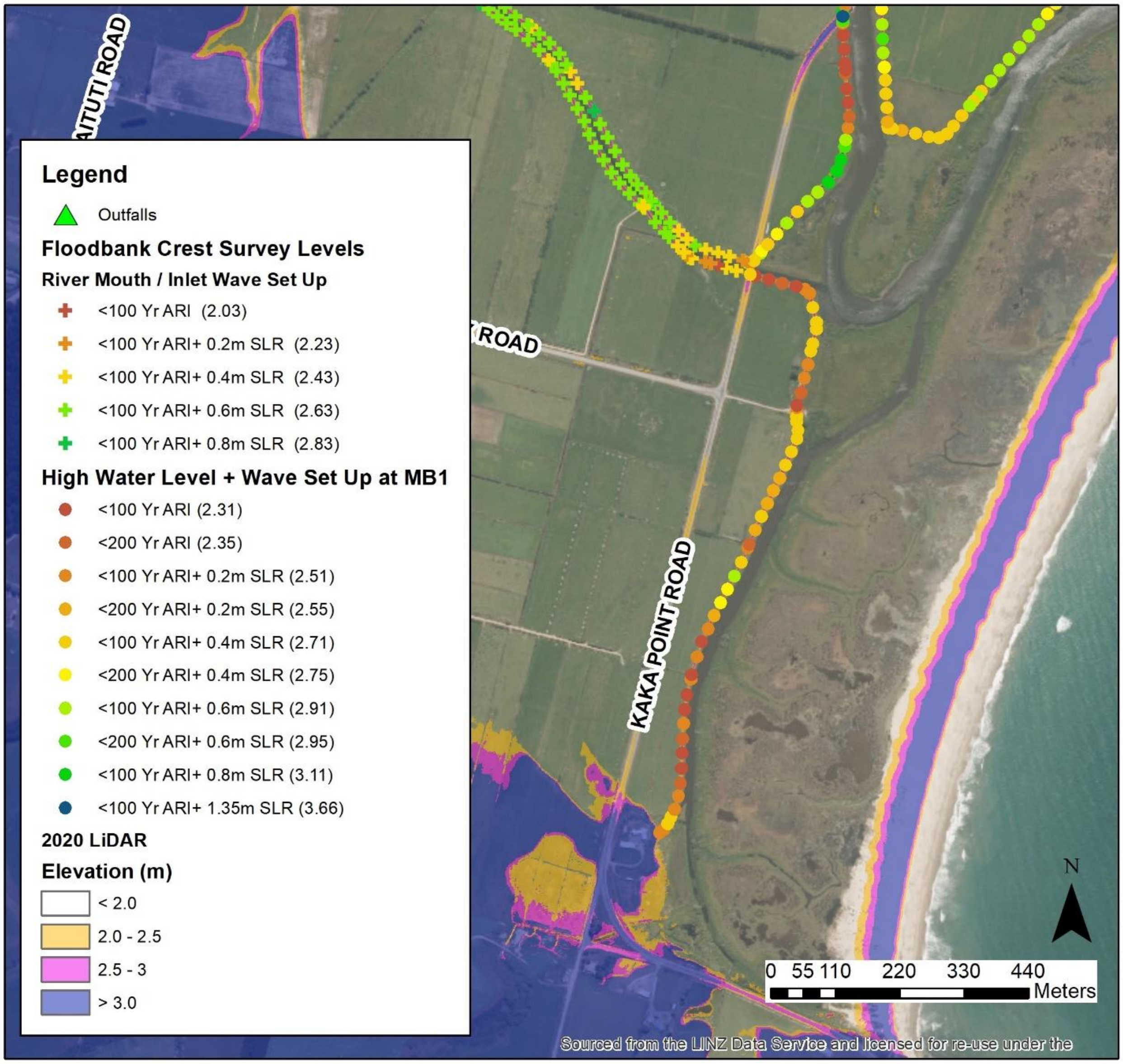


Figure 7: Port Molyneux Crest Levels (including wave set up for open coast and river mouth) showing ARI / SLR combination where overtopping occurs

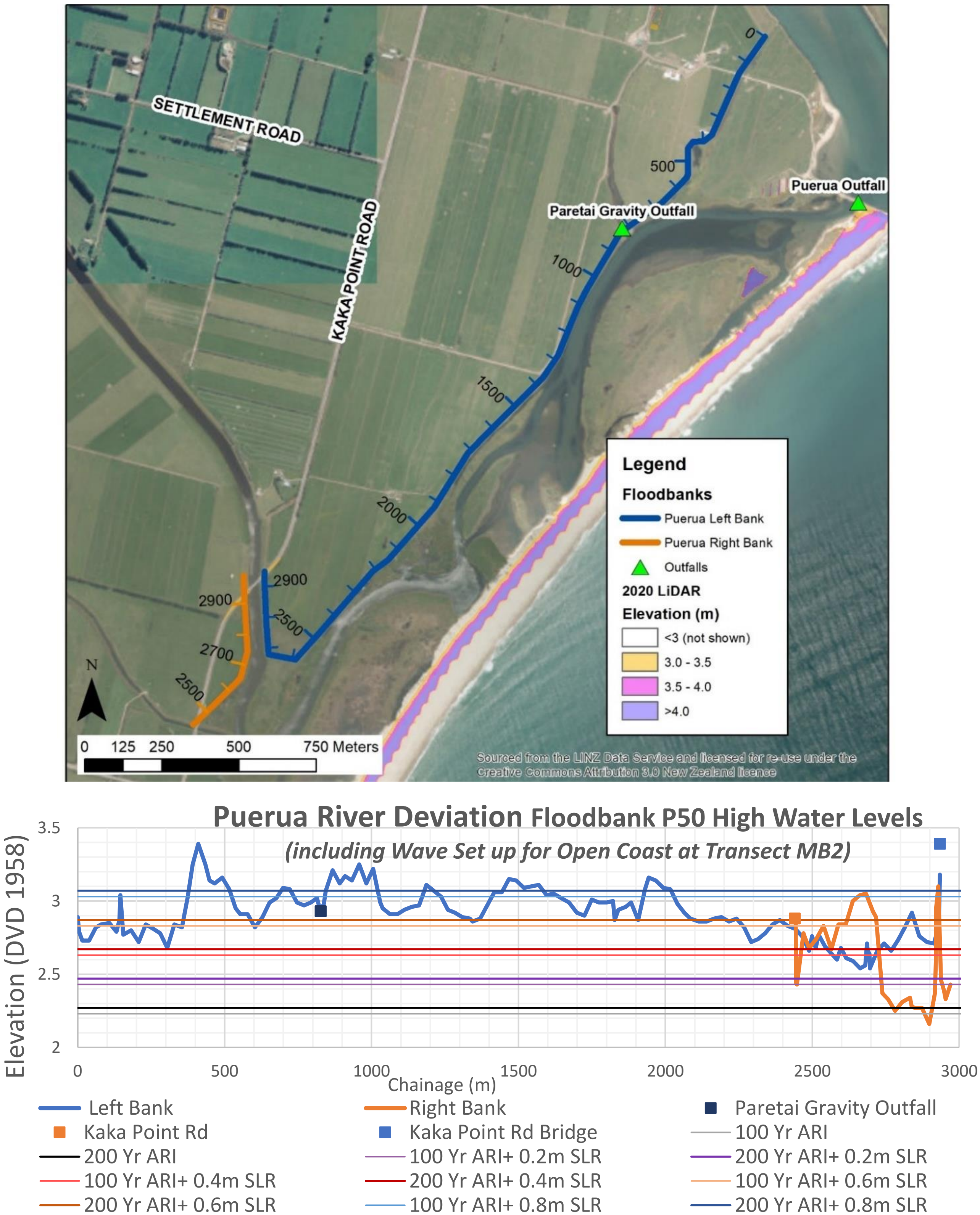


Figure 8: Puerua Floodbank P50 inundation levels (including wave set up for open coast at MB2) for 100 year and 200 year return periods

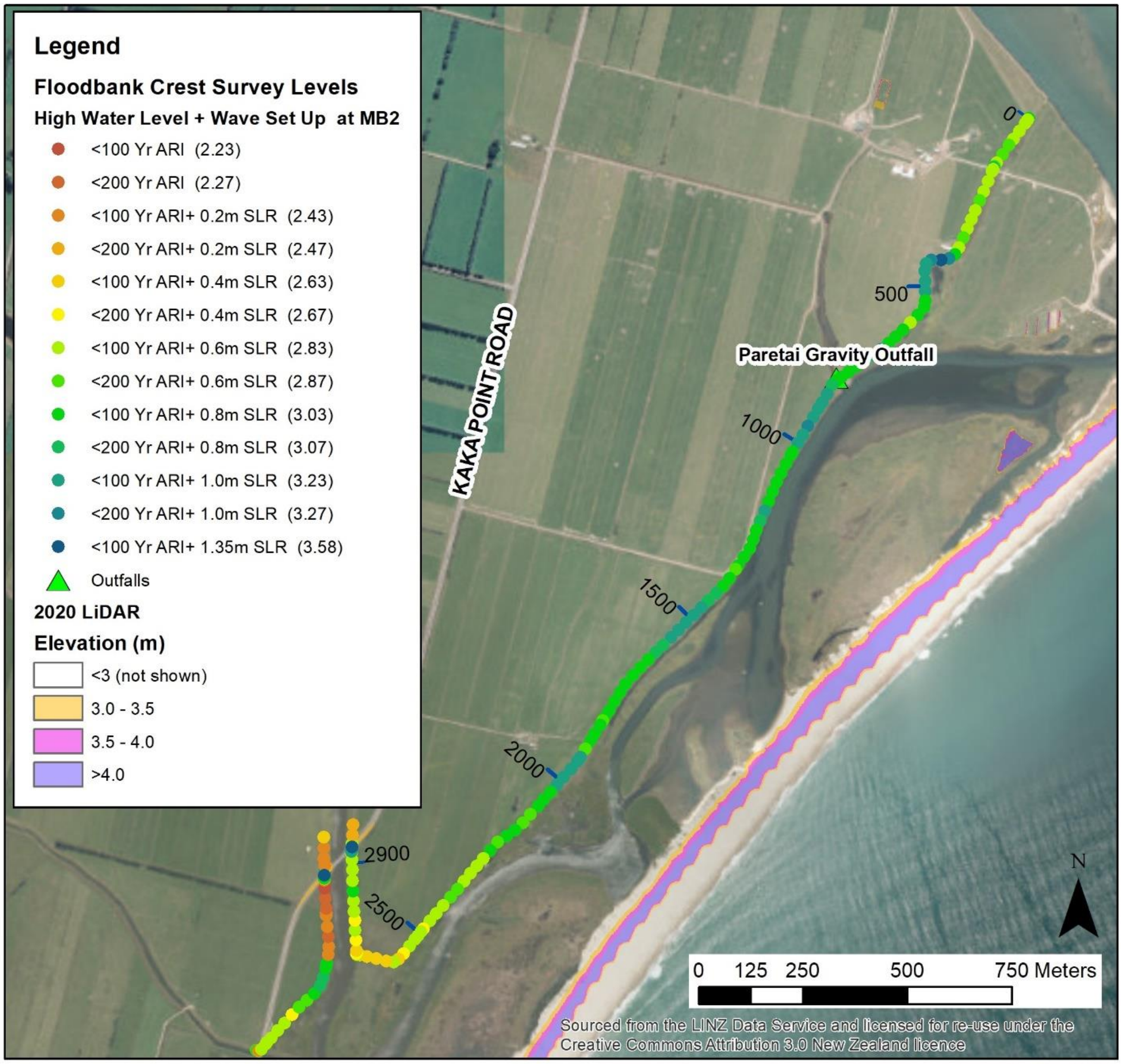
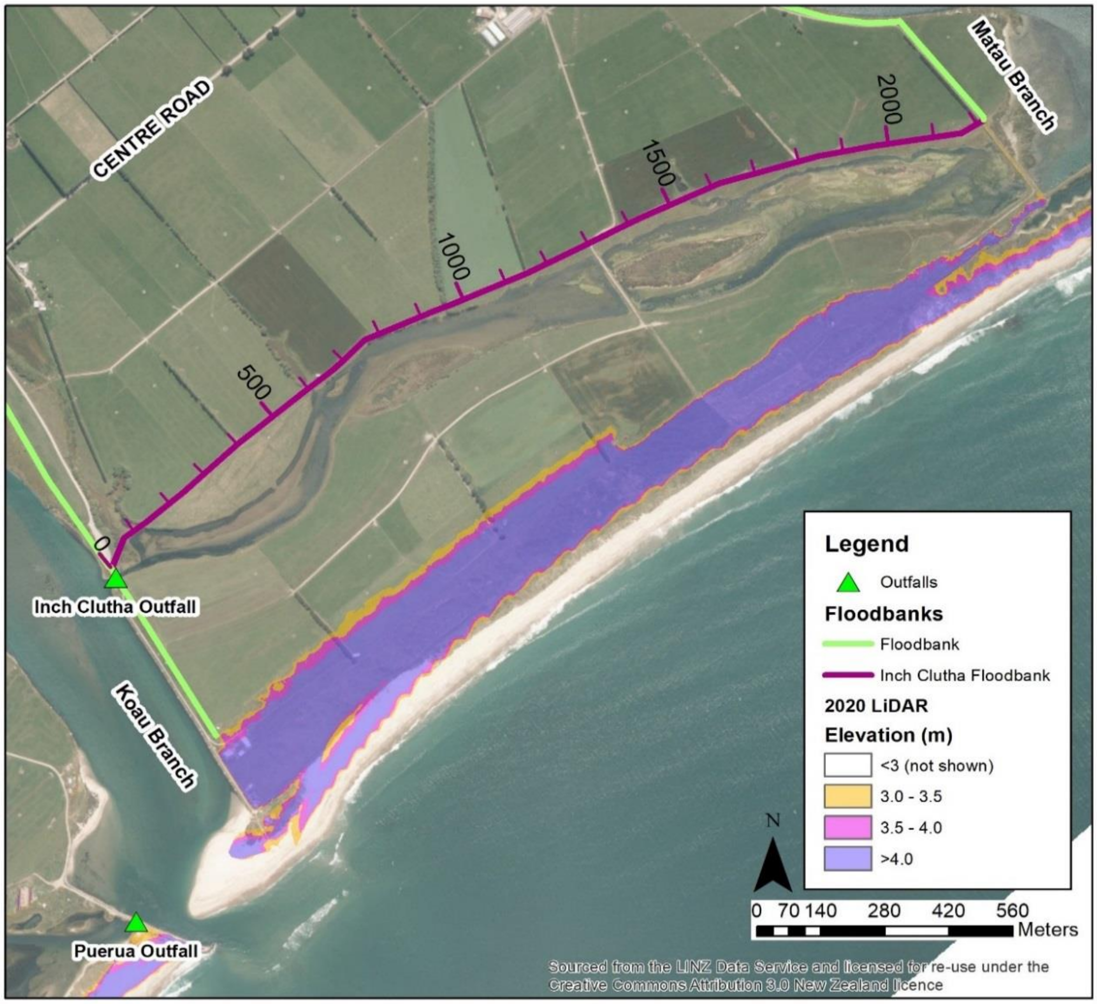


Figure 9: Puerua Floodbank Crest Levels (including wave set up for open coast at transect MB2) showing ARI / SLR combination where overtopping occurs



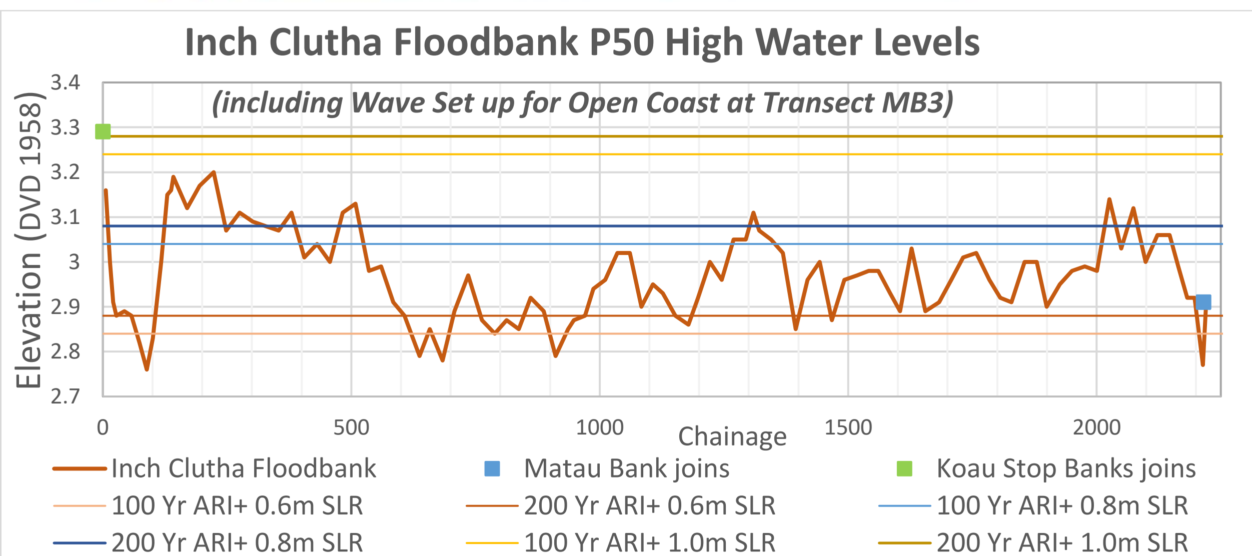


Figure 10: Inch Clutha Floodbank P50 inundation levels (including wave set up for open coast at transect MB3) for 100 year and 200 year return periods

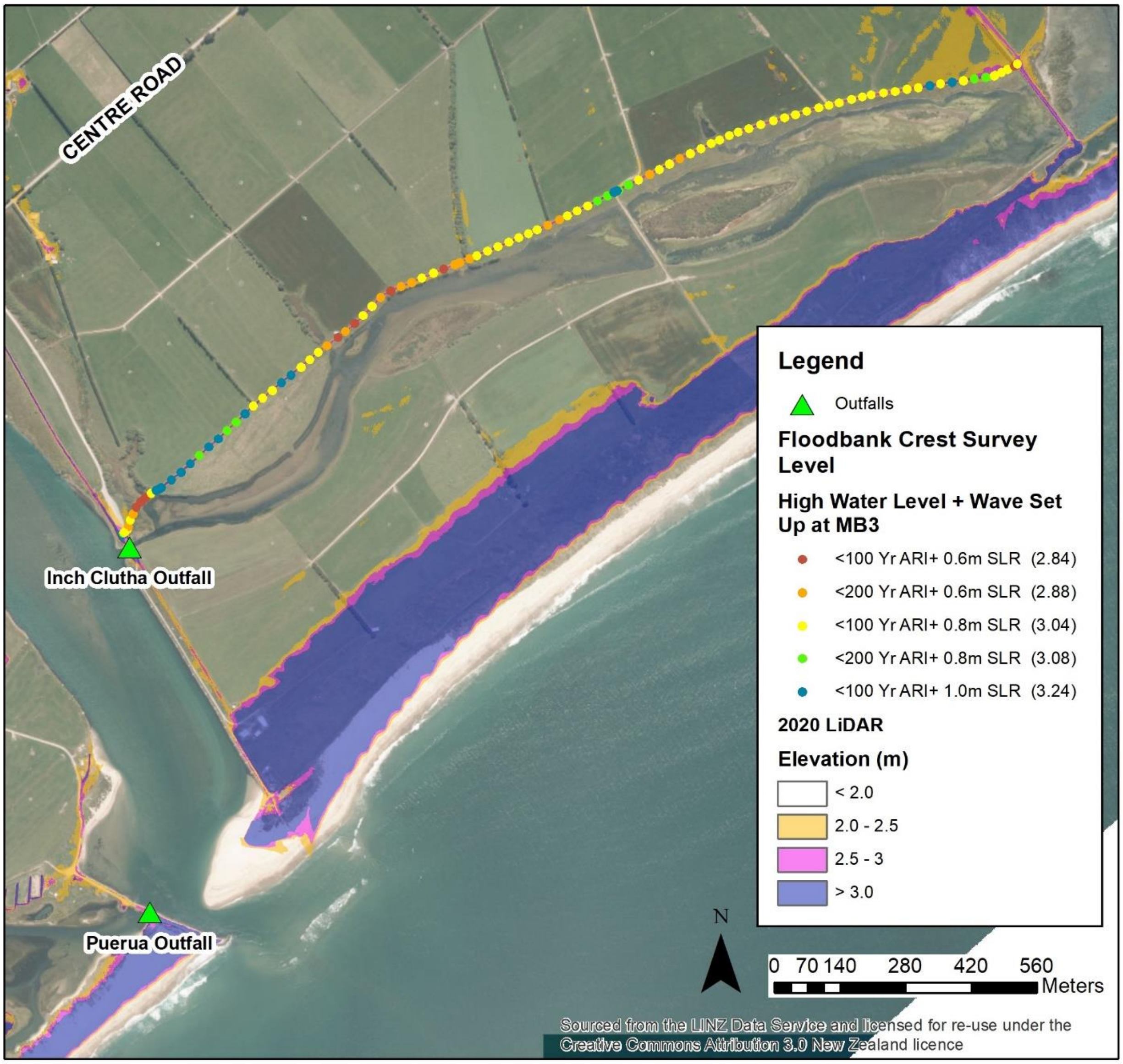
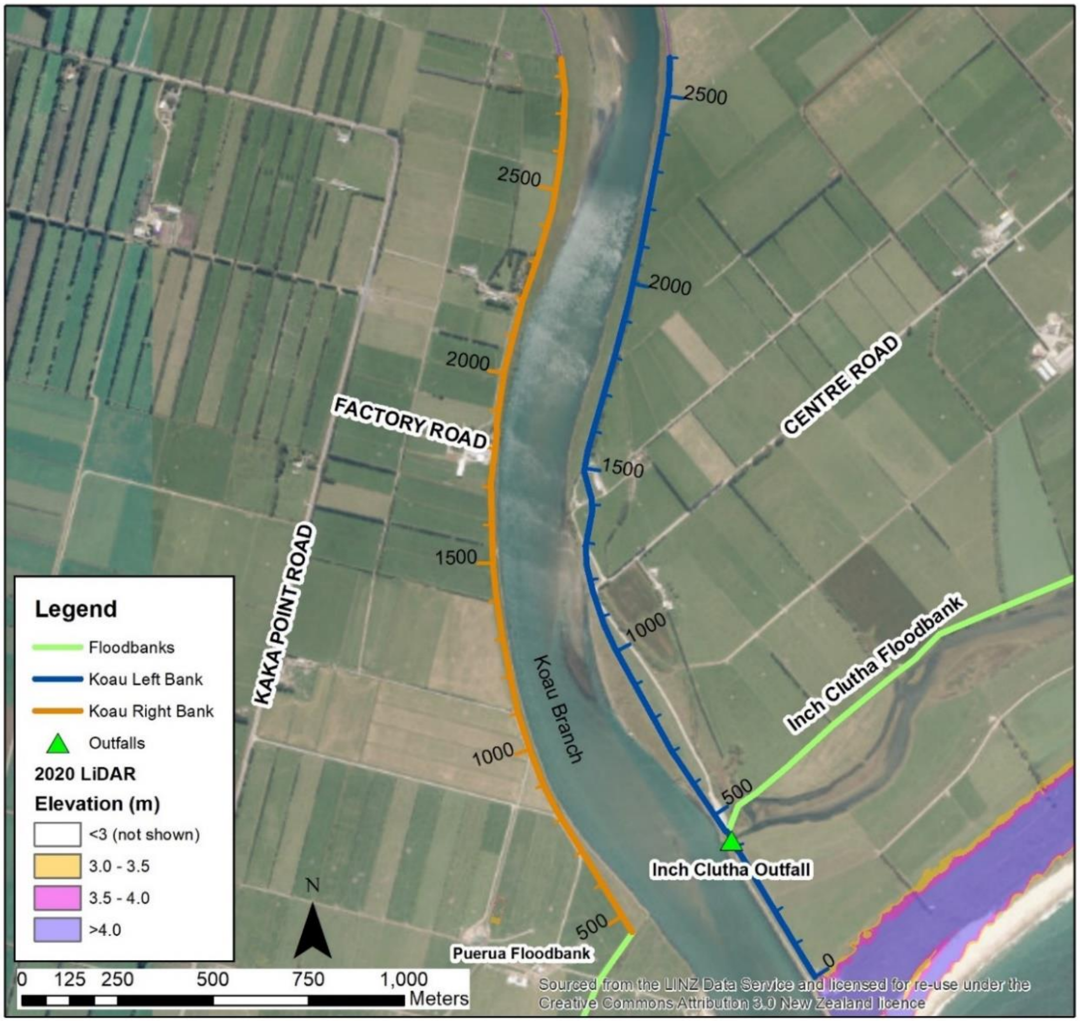


Figure 11: Inch Clutha Floodbank Crest Levels (including wave set up for open coast at transect MB3) showing ARI / SLR combination where overtopping occurs



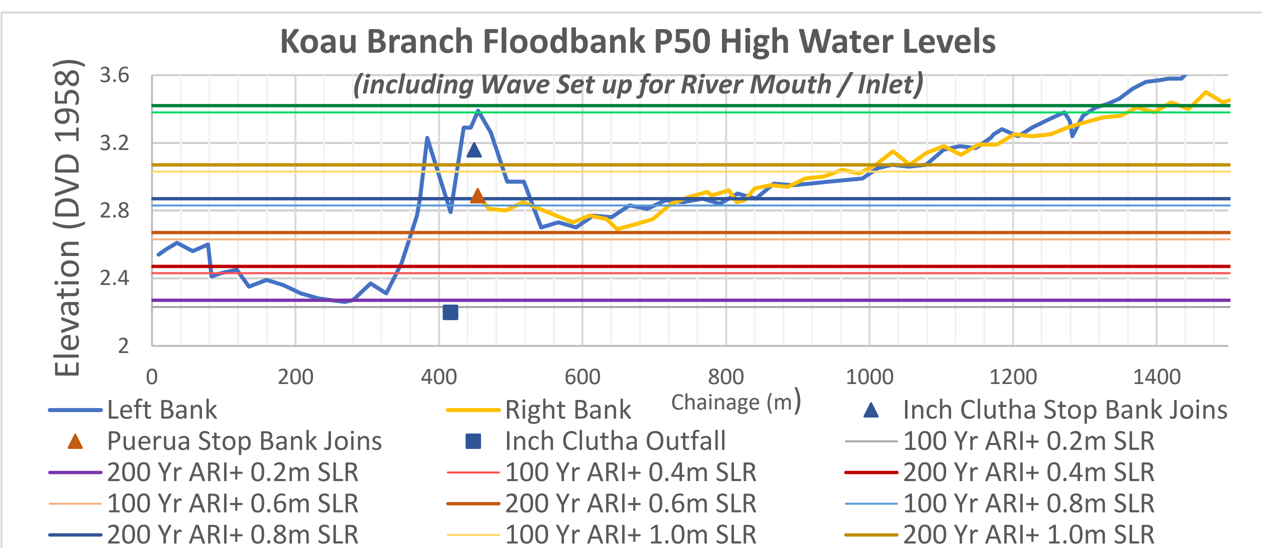


Figure 12: Koau Floodbank P50 inundation levels (including wave set up for river mouth / inlet) for 100 year and 200 year Return Periods

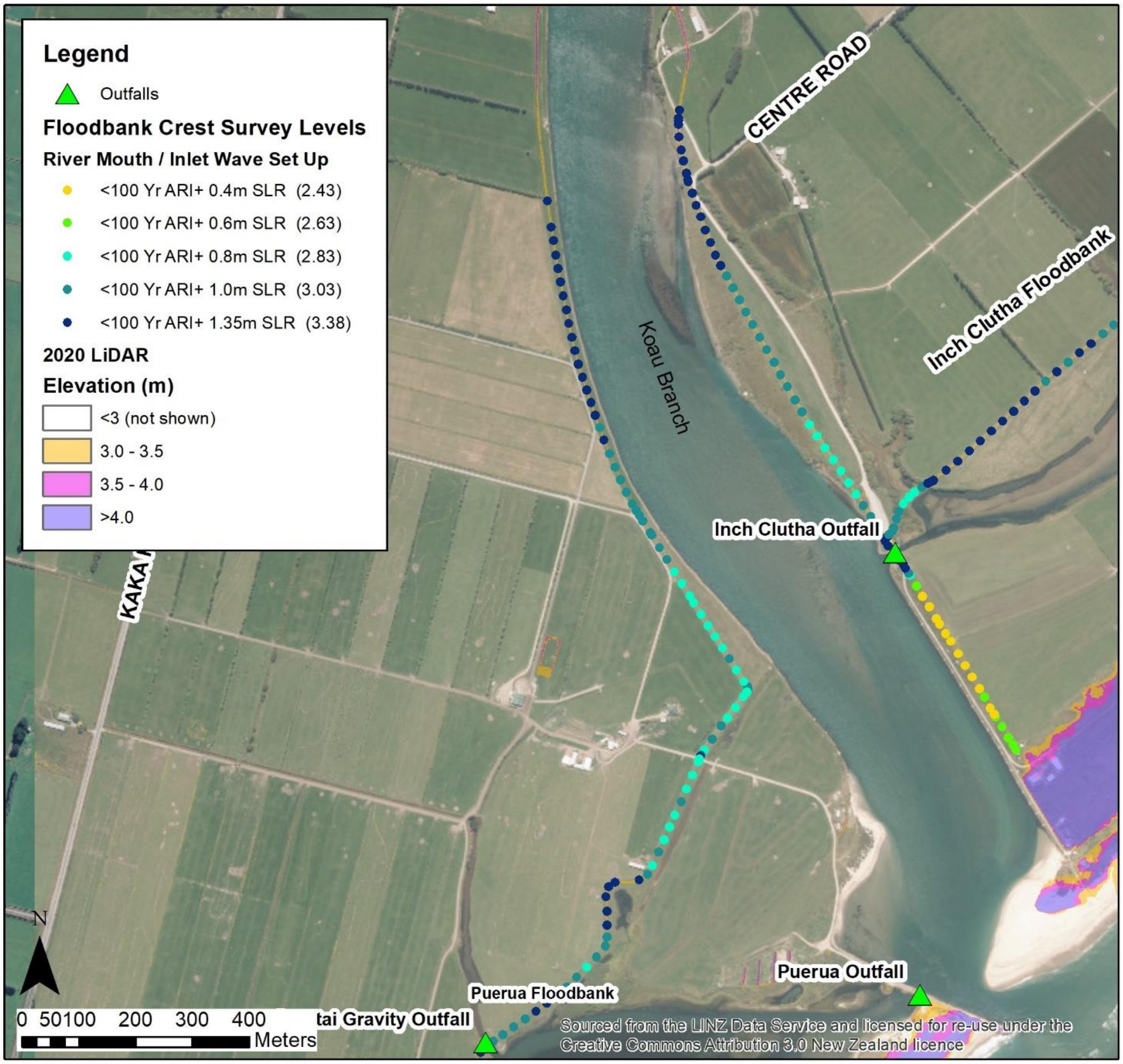
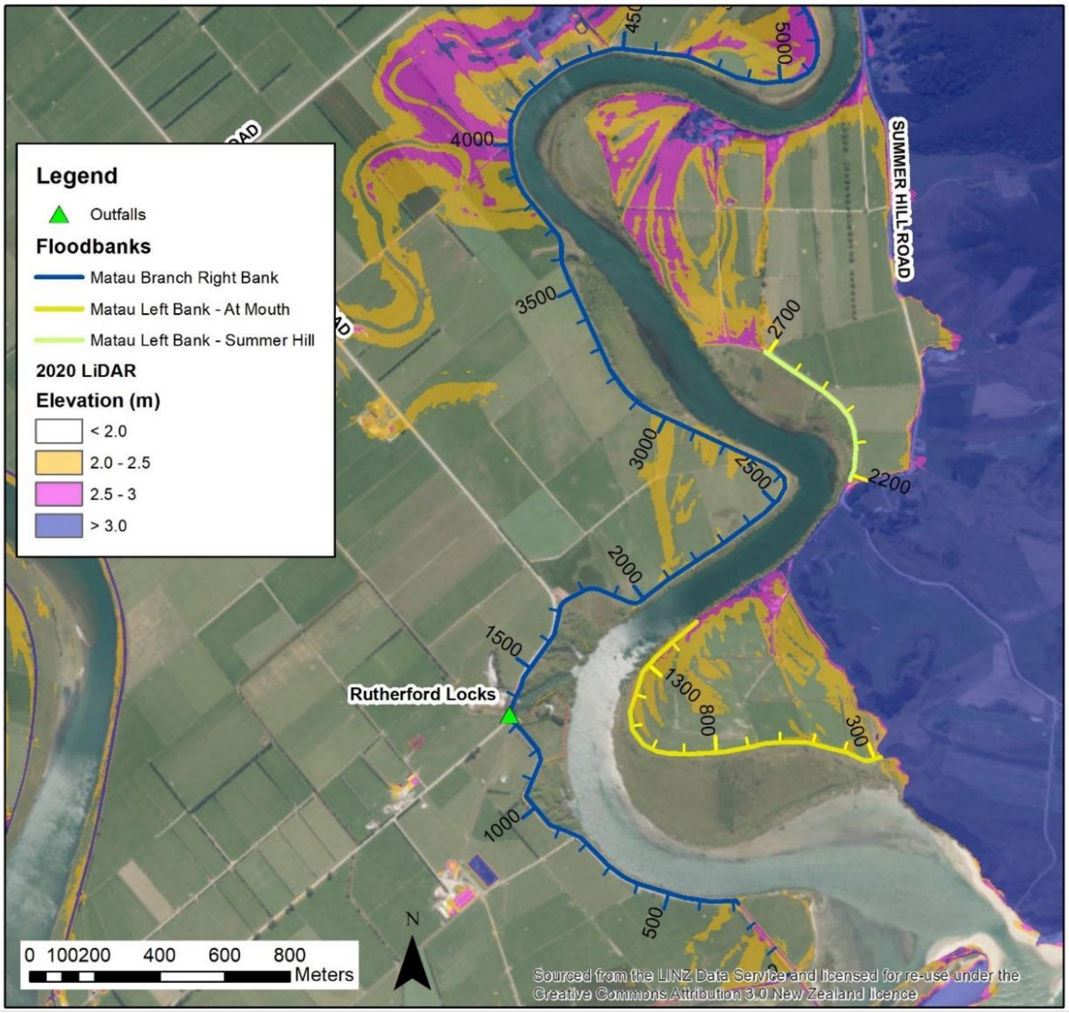


Figure 13: Koau Floodbank Crest Levels (including wave set up for river mouth / inlet) showing ARI / SLR combination where overtopping occurs



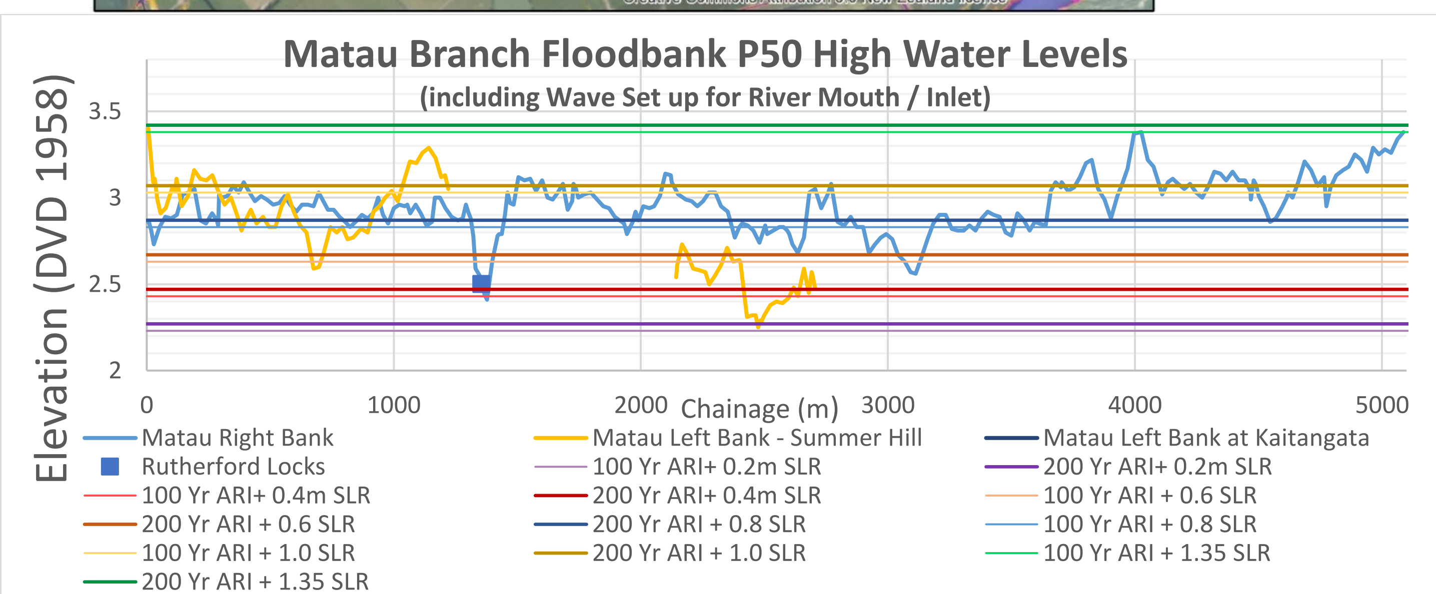


Figure 14: Matau Floodbank P50 inundation levels (including wave set up for river mouth / inlet) for 100 year and 200 year Return Periods

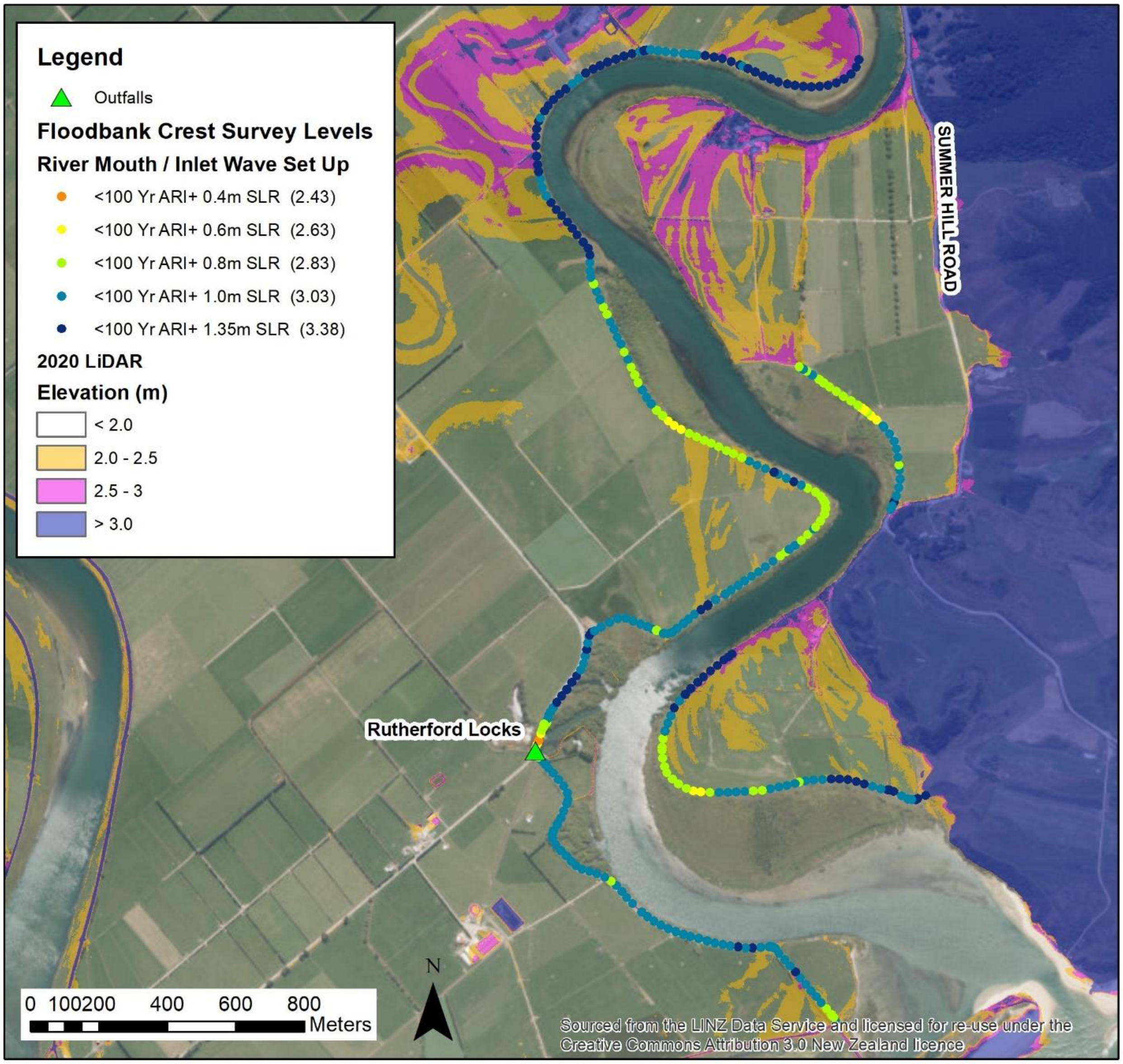


Figure 15: Matau Floodbank Crest Levels (not including wave set up) showing ARI / SLR combination where overtopping occurs

Appendix G. Probability Distribution Inputs

Table G.1 Probability Distribution Inputs.

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
1	-469.5	-170.1	-45.2	-782.4	-428.8	-75.3	-688.7	-270.9	146.9	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
2	-480.7	-264.0	-47.2	-801.2	-439.9	-78.6	-701.5	-278.1	145.3	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
3	-487.3	-267.8	-48.3	-812.1	-446.3	-80.5	-709.1	-282.2	144.6	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
4	-486.9	-271.1	-55.3	-811.5	-451.8	-92.2	-712.6	-287.0	138.6	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
5	-484.7	-272.0	-59.3	-807.9	-453.3	-98.8	-714.4	-290.8	132.9	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
6	-481.2	-272.7	-64.2	-802.0	-454.5	-107.0	-714.1	-293.3	127.6	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
7	-499.4	-282.3	-65.1	-832.4	-470.5	-108.5	-732.5	-305.2	122.1	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
8	-501.1	-289.8	-78.5	-835.2	-483.0	-130.8	-730.1	-314.5	101.2	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
9	-505.8	-295.1	-84.5	-843.0	-491.9	-140.8	-725.2	-320.3	84.5	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
10	-504.1	-298.0	-91.9	-840.2	-496.7	-153.1	-714.7	-324.0	66.6	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
11	-504.4	-298.8	-93.3	-840.6	-498.0	-155.5	-706.8	-324.7	57.4	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
12	-501.1	-301.0	-100.9	-835.1	-501.7	-168.2	-697.9	-326.3	45.4	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
13	-497.1	-303.0	-109.0	-828.5	-505.0	-181.6	-690.1	-327.4	35.3	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
14	-495.5	-305.9	-116.2	-825.9	-509.8	-193.7	-684.4	-329.2	26.1	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
15	-508.4	-308.7	-109.0	-847.3	-514.5	-181.7	-690.3	-329.7	31.0	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
16	-522.1	-312.9	-103.8	-870.1	-521.5	-173.0	-696.8	-329.9	37.0	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
17	-540.5	-318.5	-96.5	-900.9	-530.9	-160.9	-708.1	-332.2	43.7	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
18	-556.7	-324.6	-92.6	-927.8	-541.0	-154.3	-727.1	-336.3	54.5	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
19	-575.8	-330.2	-84.6	-959.6	-550.3	-141.0	-750.9	-340.0	70.9	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
20	-581.2	-331.8	-82.3	-968.7	-553.0	-137.2	-760.1	-339.4	81.2	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
21	-579.4	-332.2	-85.1	-965.6	-553.7	-141.8	-762.4	-338.0	86.4	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
22	-569.9	-331.2	-92.5	-949.9	-552.0	-154.2	-758.4	-336.1	86.1	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
23	-580.9	-328.2	-75.5	-968.2	-547.0	-125.8	-778.9	-334.2	110.5	-28.8	-26.4	-24.5	-55.7	-51.4	-47.8	-82.6	-76.4	-71.0
24	-585.1	-322.9	-60.7	-975.2	-538.1	-101.1	-789.5	-330.9	127.8	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
25	-567.1	-314.6	-62.1	-945.1	-524.3	-103.5	-779.4	-325.1	129.2	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
26	-537.4	-305.3	-73.2	-895.7	-508.9	-122.0	-757.0	-318.5	120.0	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
27	-507.3	-296.8	-86.2	-845.5	-494.6	-143.7	-730.2	-311.2	107.8	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
28	-470.5	-285.4	-100.3	-784.1	-475.7	-167.2	-693.7	-299.9	93.8	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
29	-443.5	-280.5	-117.5	-739.1	-467.5	-195.8	-668.1	-296.1	75.8	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
30	-417.8	-275.3	-132.9	-696.4	-458.9	-221.5	-652.5	-294.5	63.6	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
31	-389.0	-266.3	-143.6	-648.4	-443.8	-239.3	-632.7	-289.7	53.2	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
32	-368.9	-257.7	-146.5	-614.9	-429.5	-244.2	-616.4	-285.2	46.0	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
33	-358.9	-251.6	-144.3	-598.2	-419.3	-240.5	-608.4	-283.1	42.2	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
34	-334.1	-247.4	-160.7	-556.8	-412.3	-267.8	-579.5	-282.5	14.4	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
35	-305.6	-239.5	-173.5	-509.3	-399.2	-289.1	-550.9	-277.5	-4.1	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
36	-295.5	-234.9	-174.3	-492.5	-391.5	-290.5	-535.5	-279.3	-23.2	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
37	-294.8	-230.9	-167.1	-491.3	-384.9	-278.5	-532.0	-283.1	-34.2	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
38	-291.6	-226.9	-162.1	-486.0	-378.1	-270.2	-527.6	-287.9	-48.1	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
39	-293.2	-226.9	-160.6	-488.7	-378.2	-267.7	-525.0	-294.0	-63.0	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
40	-288.4	-220.1	-151.8	-480.6	-366.8	-253.0	-520.8	-291.6	-62.5	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
41	-283.4	-213.9	-144.4	-472.4	-356.5	-240.7	-506.1	-287.8	-69.4	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
42	-282.6	-206.6	-130.6	-471.0	-344.3	-217.7	-497.0	-282.0	-67.0	-27.9	-25.8	-24.0	-54.1	-50.2	-46.8	-80.2	-74.5	-69.6
43	-274.9	-199.7	-124.6	-458.1	-332.9	-207.6	-489.5	-276.9	-64.2	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
44	-263.8	-195.7	-127.5	-439.6	-326.1	-212.5	-480.1	-274.7	-69.2	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
45	-250.6	-190.6	-130.5	-417.7	-317.6	-217.5	-458.3	-271.0	-83.7	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
46	-251.8	-193.4	-135.1	-419.6	-322.4	-225.1	-446.7	-275.5	-104.3	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
47	-260.7	-192.4	-124.0	-434.5	-320.6	-206.6	-446.4	-272.6	-98.7	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
48	-263.8	-189.2	-114.5	-439.6	-315.3	-190.9	-438.1	-266.4	-94.7	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
49	-268.7	-191.0	-113.4	-447.8	-318.4	-189.0	-433.3	-265.8	-98.4	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
50	-269.9	-189.1	-108.3	-449.9	-315.2	-180.5	-431.5	-266.1	-100.8	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
51	-265.3	-188.9	-112.6	-442.2	-314.9	-187.6	-422.9	-268.5	-114.1	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
52	-261.9	-187.8	-113.8	-436.5	-313.0	-189.6	-420.7	-270.9	-121.1	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
53	-255.1	-190.0	-124.9	-425.1	-316.7	-208.2	-417.0	-277.8	-138.6	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
54	-256.2	-192.7	-129.1	-427.0	-321.1	-215.1	-418.1	-286.1	-154.2	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
55	-261.1	-193.8	-126.6	-435.1	-323.0	-211.0	-426.6	-292.4	-158.2	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
56	-260.2	-193.6	-126.9	-433.6	-322.6	-211.5	-443.9	-297.3	-150.7	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
57	-257.5	-190.6	-123.8	-429.1	-317.7	-206.3	-464.4	-299.5	-134.7	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
58	-257.3	-191.5	-125.7	-428.9	-319.2	-209.5	-491.5	-305.8	-120.0	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
59	-260.3	-193.7	-127.1	-433.8	-322.8	-211.9	-522.0	-312.7	-103.4	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
60	-265.3	-190.2	-115.1	-442.2	-317.0	-191.8	-554.9	-313.2	-71.5	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
61	-275.0	-190.0	-105.0	-458.3	-316.6	-175.0	-598.0	-314.5	-31.1	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
62	-288.8	-187.1	-85.5	-481.3	-311.9	-142.5	-775.5	-326.1	123.4	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
63	-301.6	-185.9	-70.1	-502.7	-309.8	-116.9	-957.6	-334.4	288.8	-27.1	-24.8	-23.0	-53.4	-48.9	-45.5	-79.8	-73.1	-67.9
64	-320.2	-188.3	-56.5	-533.6	-313.9	-94.1	-1119.0	-345.8	427.3	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
65	-339.7	-193.3	-46.9	-566.2	-322.2	-78.2	-1194.7	-353.8	487.1	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
66	-353.3	-192.8	-32.4	-588.8	-321.4	-54.0	-1270.4	-357.0	556.4	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
67	-374.7	-195.6	-16.5	-624.5	-326.0	-27.5	-1365.2	-363.8	637.7	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
68	-398.3	-201.2	-4.0	-663.9	-335.3	-6.7	-1476.9	-373.8	729.4	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
69	-415.9	-208.8	-1.7	-693.2	-348.0	-2.9	-1606.0	-386.8	832.3	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
70	-430.6	-216.9	-3.3	-717.6	-361.5	-5.5	-1776.9	-403.0	970.8	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
71	-435.1	-219.0	-2.9	-725.2	-365.0	-4.8	-2015.9	-419.6	1176.7	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
72	-417.3	-201.8	13.8	-695.6	-336.3	22.9	-2365.7	-432.1	1501.6	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
73	-420.4	-204.0	12.4	-700.6	-340.0	20.7	-2356.5	-421.5	1513.5	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5
74	-420.6	-204.6	11.4	-700.9	-340.9	19.1	-2220.4	-424.0	1372.4	-25.9	-23.8	-22.2	-51.1	-47.0	-43.8	-76.3	-70.2	-65.5

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
81	-123.1	-47.6	28.0	-205.2	-79.3	46.7	-205.2	-79.3	46.7	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
82	-107.1	-32.6	41.8	-178.5	-54.4	69.7	-178.5	-54.4	69.7	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
83	-99.3	-37.8	23.8	-165.5	-62.9	39.6	-165.5	-62.9	39.6	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
84	-100.7	-38.8	23.1	-167.8	-64.7	38.5	-167.8	-64.7	38.5	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
85	-101.3	-39.4	22.4	-168.9	-65.7	37.4	-168.9	-65.7	37.4	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
86	-105.2	-41.6	22.0	-175.4	-69.3	36.7	-175.4	-69.3	36.7	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
87	-109.0	-45.6	17.8	-181.6	-76.0	29.6	-181.6	-76.0	29.6	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
88	-111.4	-48.1	15.2	-185.7	-80.2	25.3	-185.7	-80.2	25.3	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
89	-111.4	-49.7	12.0	-185.6	-82.8	20.0	-185.6	-82.8	20.0	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
90	-109.7	-50.6	8.6	-182.8	-84.3	14.3	-182.8	-84.3	14.3	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
91	-107.9	-52.2	3.5	-179.8	-87.0	5.9	-179.8	-87.0	5.9	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
92	-104.8	-51.8	1.2	-174.7	-86.3	2.0	-174.7	-86.3	2.0	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
93	-100.9	-50.9	-0.9	-168.2	-84.9	-1.5	-168.2	-84.9	-1.5	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
94	-97.4	-50.9	-4.3	-162.4	-84.8	-7.1	-162.4	-84.8	-7.1	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
95	-93.9	-50.5	-7.2	-156.5	-84.2	-12.0	-156.5	-84.2	-12.0	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
96	-91.3	-50.8	-10.3	-152.1	-84.7	-17.2	-152.1	-84.7	-17.2	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
97	-88.9	-50.7	-12.5	-148.2	-84.5	-20.9	-148.2	-84.5	-20.9	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
98	-86.9	-50.3	-13.7	-144.8	-83.8	-22.8	-144.8	-83.8	-22.8	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
99	-84.5	-49.1	-13.6	-140.9	-81.8	-22.7	-140.9	-81.8	-22.7	-21.3	-18.2	-16.0	-39.5	-33.8	-29.9	-57.6	-49.4	-43.7
100	-81.1	-47.1	-13.1	-135.1	-78.5	-21.8	-135.1	-78.5	-21.8	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
101	-78.1	-44.8	-11.5	-130.2	-74.7	-19.2	-130.2	-74.7	-19.2	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
102	-75.4	-41.8	-8.3	-125.6	-69.7	-13.8	-125.6	-69.7	-13.8	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
103	-73.3	-40.1	-7.0	-122.1	-66.9	-11.7	-122.1	-66.9	-11.7	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
104	-71.7	-38.5	-5.2	-119.5	-64.1	-8.7	-119.5	-64.1	-8.7	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
105	-70.0	-36.1	-2.2	-116.6	-60.1	-3.6	-116.6	-60.1	-3.6	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
106	-67.6	-33.5	0.6	-112.6	-55.8	1.0	-112.6	-55.8	1.0	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
107	-64.9	-30.6	3.7	-108.1	-51.0	6.1	-108.1	-51.0	6.1	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
108	-61.7	-27.7	6.2	-102.8	-46.2	10.4	-102.8	-46.2	10.4	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
109	-57.4	-24.8	7.7	-95.6	-41.4	12.9	-95.6	-41.4	12.9	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
110	-53.4	-22.2	9.0	-89.0	-37.0	15.0	-89.0	-37.0	15.0	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
111	-52.0	-21.5	9.1	-86.7	-35.8	15.2	-86.7	-35.8	15.2	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
112	-50.7	-20.8	9.2	-84.5	-34.6	15.3	-84.5	-34.6	15.3	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
113	-50.6	-20.8	9.0	-84.4	-34.7	15.0	-84.4	-34.7	15.0	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
114	-48.8	-19.3	10.2	-81.4	-32.2	17.0	-81.4	-32.2	17.0	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
115	-47.1	-17.3	12.6	-78.5	-28.8	21.0	-78.5	-28.8	21.0	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
116	-45.1	-15.5	14.1	-75.1	-25.8	23.5	-75.1	-25.8	23.5	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
117	-44.4	-14.8	14.9	-74.0	-24.6	24.8	-74.0	-24.6	24.8	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
118	-44.2	-14.0	16.2	-73.6	-23.3	27.0	-73.6	-23.3	27.0	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
119	-44.3	-13.4	17.5	-73.9	-22.3	29.2	-73.9	-22.3	29.2	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
120	-45.4	-13.1	19.2	-75.7	-21.9	32.0	-75.7	-21.9	32.0	-19.2	-16.7	-14.9	-35.5	-31.0	-27.8	-51.9	-45.4	-40.7
121	-45.6	-12.2	21.2	-76.0	-20.3	35.3	-76.0	-20.3	35.3	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
122	-46.2	-11.8	22.7	-77.0	-19.6	37.9	-77.0	-19.6	37.9	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
123	-46.4	-10.9	24.6	-77.4	-18.2	41.0	-77.4	-18.2	41.0	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
124	-45.2	-8.7	27.8	-75.3	-14.5	46.3	-75.3	-14.5	46.3	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
125	-44.7	-8.2	28.4	-74.5	-13.6	47.3	-74.5	-13.6	47.3	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
126	-43.6	-7.6	28.4	-72.7	-12.7	47.4	-72.7	-12.7	47.4	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
127	-44.3	-7.8	28.6	-73.8	-13.0	47.7	-73.8	-13.0	47.7	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
128	-45.2	-7.1	31.0	-75.3	-11.8	51.7	-75.3	-11.8	51.7	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
129	-38.3	-2.1	34.1	-63.8	-3.5	56.8	-63.8	-3.5	56.8	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
130	-33.3	3.0	39.3	-55.5	5.0	65.5	-55.5	5.0	65.5	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
131	-29.3	7.5	44.4	-48.9	12.5	74.0	-48.9	12.5	74.0	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
132	-25.9	12.3	50.6	-43.2	20.5	84.3	-43.2	20.5	84.3	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
133	-21.4	18.0	57.5	-35.7	30.0	95.8	-35.7	30.0	95.8	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
134	-17.8	20.3	58.5	-29.6	33.9	97.5	-29.6	33.9	97.5	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
135	-13.1	22.4	57.9	-21.8	37.3	96.5	-21.8	37.3	96.5	-20.9	-17.8	-15.8	-38.6	-33.2	-29.4	-56.3	-48.5	-43.1
140	1.0	26.5	52.0	1.7	44.2	86.6	1.7	44.2	86.6	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
141	0.8	24.1	47.4	1.3	40.1	79.0	1.3	40.1	79.0	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
142	1.7	22.3	42.9	2.8	37.1	71.5	2.8	37.1	71.5	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
143	2.8	19.8	36.8	4.6	33.0	61.4	4.6	33.0	61.4	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
144	3.6	17.2	30.9	6.0	28.7	51.5	6.0	28.7	51.5	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
145	5.6	18.1	30.5	9.3	30.1	50.9	9.3	30.1	50.9	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
146	5.4	17.4	29.5	9.0	29.0	49.1	9.0	29.0	49.1	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
147	5.5	17.4	29.3	9.2	29.0	48.8	9.2	29.0	48.8	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
148	5.9	17.7	29.5	9.8	29.5	49.1	9.8	29.5	49.1	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
149	6.4	18.0	29.7	10.6	30.0	49.5	10.6	30.0	49.5	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
150	6.5	18.1	29.6	10.9	30.1	49.3	10.9	30.1	49.3	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
151	7.4	18.2	28.9	12.3	30.3	48.2	12.3	30.3	48.2	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
152	8.5	18.4	28.3	14.1	30.6	47.1	14.1	30.6	47.1	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
153	9.8	18.9	28.0	16.4	31.5	46.6	16.4	31.5	46.6	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
154	11.4	19.7	28.0	19.0	32.8	46.7	19.0	32.8	46.7	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
155	13.1	20.8	28.4	21.9	34.6	47.3	21.9	34.6	47.3	-12.6	-12.1	-11.3	-25.1	-24.3	-22.6	-37.7	-36.4	-33.9
156	14.4	21.7	29.0	24.0	36.2	48.4	24.0	36.2	48.4	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
157	16.6	23.5	30.4	27.7	39.2	50.6	27.7	39.2	50.6	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
158	19.0	25.3	31.6	31.7	42.2	52.7	31.7	42.2	52.7	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
159	20.4	26.1	31.8	34.0	43.5	53.0	34.0	43.5	53.0	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
160	21.4	26.7	32.0	35.7	44.5	53.3	35.7	44.5	53.3	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
161	22.4	27.5	32.6	37.4	45.8	54.3	37.4	45.8	54.3	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
162	23.1	28.1	33.2	38.5	46.9	55.3	38.5	46.9	55.3	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
163	23.7	28.8	33.9	39.5	48.0	56.5	39.5	48.0	56.5	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
164	24.2	29.3	34.4	40.4	48.8	57.3	40.4	48.8	57.3	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
165	23.7	29.0	34.4	39.5	48.4	57.3	39.5	48.4	57.3	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
166	23.2	29.4	35.5	38.7	49.0	59.2	38.7	49.0	59.2	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
167	22.0	29.2	36.4	36.7	48.7	60.6	36.7	48.7	60.6	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
168	20.7	28.7	36.6	34.5	47.8	61.0	34.5	47.8	61.0	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
169	19.6	28.0	36.4	32.7	46.7	60.7	32.7	46.7	60.7	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
170	19.2	28.1	37.0	32.0	46.8	61.6	32.0	46.8	61.6	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
171	18.8	28.1	37.5	31.3	46.9	62.5	31.3	46.9	62.5	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
172	17.9	27.4	36.9	29.9	45.7	61.5	29.9	45.7	61.5	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
173	16.4	26.8	37.1	27.4	44.6	61.8	27.4	44.6	61.8	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
174	15.0	25.7	36.5	25.0	42.9	60.9	25.0	42.9	60.9	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
175	12.8	24.1	35.5	21.3	40.2	59.2	21.3	40.2	59.2	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
176	8.6	20.7	32.8	14.3	34.5	54.6	14.3	34.5	54.6	-12.0	-11.6	-10.9	-24.0	-23.3	-21.8	-35.9	-34.9	-32.7
177	5.3	17.3	29.3	8.9	28.9	48.9	8.9	28.9	48.9	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
178	3.0	14.7	26.4	5.0	24.5	44.0	5.0	24.5	44.0	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
179	1.6	13.2	24.8	2.6	22.0	41.3	2.6	22.0	41.3	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
180	0.3	12.2	24.2	0.5	20.4	40.3	0.5	20.4	40.3	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
181	-0.9	11.1	23.2	-1.5	18.5	38.6	-1.5	18.5	38.6	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
182	-2.9	9.7	22.2	-4.9	16.1	37.0	-4.9	16.1	37.0	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
183	-4.7	8.7	22.1	-7.8	14.5	36.9	-7.8	14.5	36.9	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
184	-5.4	7.9	21.1	-9.0	13.1	35.2	-9.0	13.1	35.2	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
185	-6.1	7.3	20.7	-10.1	12.2	34.5	-10.1	12.2	34.5	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
186	-5.8	7.7	21.1	-9.6	12.8	35.2	-9.6	12.8	35.2	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
187	-2.6	10.4	23.4	-4.3	17.4	39.0	-4.3	17.4	39.0	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
188	-0.4	12.1	24.6	-0.6	20.2	41.0	-0.6	20.2	41.0	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
189	1.5	14.0	26.4	2.5	23.3	44.0	2.5	23.3	44.0	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
190	2.6	14.8	26.9	4.3	24.6	44.9	4.3	24.6	44.9	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
191	3.4	15.0	26.6	5.6	25.0	44.4	5.6	25.0	44.4	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
192	3.5	15.1	26.6	5.9	25.1	44.3	5.9	25.1	44.3	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
193	4.6	15.2	25.9	7.6	25.4	43.2	7.6	25.4	43.2	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
194	6.0	15.9	25.7	10.0	26.5	42.9	10.0	26.5	42.9	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
195	6.8	16.0	25.3	11.3	26.7	42.2	11.3	26.7	42.2	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
196	7.3	16.0	24.7	12.1	26.7	41.2	12.1	26.7	41.2	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
197	7.2	16.0	24.9	12.0	26.7	41.5	12.0	26.7	41.5	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
198	7.2	16.0	24.7	12.0	26.6	41.2	12.0	26.6	41.2	-12.2	-11.8	-11.0	-24.3	-23.6	-22.0	-36.5	-35.3	-33.1
199	6.8	15.8	24.8	11.3	26.3	41.3	11.3	26.3	41.3	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
200	6.9	15.4	23.9	11.5	25.7	39.9	11.5	25.7	39.9	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
201	6.6	15.7	24.7	11.0	26.1	41.2	11.0	26.1	41.2	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
202	6.3	16.0	25.7	10.5	26.7	42.8	10.5	26.7	42.8	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
203	6.5	16.5	26.5	10.8	27.5	44.2	10.8	27.5	44.2	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
204	7.1	17.3	27.4	11.9	28.8	45.7	11.9	28.8	45.7	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
205	7.2	17.4	27.6	12.0	29.0	46.0	12.0	29.0	46.0	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
206	7.3	18.9	30.6	12.1	31.5	51.0	12.1	31.5	51.0	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
207	7.4	20.8	34.2	12.3	34.6	57.0	12.3	34.6	57.0	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
208	8.0	21.9	35.8	13.3	36.5	59.6	13.3	36.5	59.6	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
209	7.4	22.8	38.2	12.3	38.0	63.6	12.3	38.0	63.6	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1

	Extrapolation of Long Term Rate									SLR Scenario								
	30 Year (m)			50 Year - With Wall			50 year - Without Wall			0.2m SLR (2050)			0.4m SLR (2070)			0.6m SLR (2070)		
Transect	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound	Mean	Lower Bound
210	9.6	24.4	39.1	16.0	40.6	65.2	16.0	40.6	65.2	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
211	10.6	25.2	39.9	17.6	42.0	66.5	17.6	42.0	66.5	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
212	10.7	25.7	40.7	17.8	42.8	67.8	17.8	42.8	67.8	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
213	9.3	25.4	41.6	15.5	42.4	69.3	15.5	42.4	69.3	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
214	6.5	24.5	42.4	10.9	40.8	70.6	10.9	40.8	70.6	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
215	5.1	24.6	44.0	8.5	41.0	73.4	8.5	41.0	73.4	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
216	-1.0	24.9	50.8	-1.6	41.5	84.6	-1.6	41.5	84.6	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
217	-1.7	24.5	50.7	-2.8	40.9	84.5	-2.8	40.9	84.5	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
218	-2.6	23.7	50.0	-4.3	39.5	83.3	-4.3	39.5	83.3	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
219	-4.2	23.0	50.1	-7.0	38.3	83.5	-7.0	38.3	83.5	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
220	-5.0	22.3	49.6	-8.4	37.1	82.6	-8.4	37.1	82.6	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1
221	-10.7	20.3	51.3	-17.8	33.8	85.5	-17.8	33.8	85.5	-16.7	-15.7	-15.4	-33.3	-31.5	-30.7	-50.0	-47.2	-46.1