Table 4.9: Adopted component values for the Christchurch open coast shoreline

Cell		1	2	3	4	5	6	7	8	9	10	11	12	13	14
Chainage, km (from Waimakariri River mouth)		0 to 1.2	1.2 to 4.5	4.5 to 7	7 to 8	8 to 10	10 to 11.7	11.7 to 12	12 to 13	13 to 13.6	13.6 to 15.1	15.1 to16.5	16.5 to 18	18 to 19	10 to 20.5
Morphology		Dune (Spit end)	Dune	Dune	Dune	Dune	Dune	Dune	Dune	Dune	Dune	Dune	Dune	Dune	Dune (Spit end)
Short-term (m)	mu (mean)	-11	-5.9	-5.9	-5.9	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-5.5
	sigma (shape parameter)	7	4.7	4.7	4.7	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	1.86
Dune (m above toe)	Lower	1	4	3.5	3	4.5	4	0.5	4	1	4	3	2.5	1.8	2
	Mode	2	5	4	4	5.5	5	0.8	5	1.5	5	3.5	3	2	3
	Upper	3	7.5	5	5	6.5	7	1	6	2	6	4.5	5	3	4
Stable angle (deg)	Lower	30	30	30	30	30	30	30	30	30	30	30	30	30	30
	Mode	32	32	32	32	32	32	32	32	32	32	32	32	32	32
	Upper	34	34	34	34	34	34	34	34	34	34	34	34	34	34
Long-term (m) -ve erosion +ve accretion	Lower	0.6	0.30	0.30	0.18	0.12	0.20	0.20	0.20	0.40	0.30	0.40	0.47	0.70	0.7
	Mode	0	0.25	0.25	0.16	0.08	0.10	0.10	0.10	0.25	0.20	0.30	0.45	0.65	0.2
	Upper	-0.6	0.18	0.18	0.14	0.00	-0.04	-0.04	-0.04	0.10	0.10	0.20	0.40	0.60	-0.1
Closure slope	Lower	0.060	0.060	0.040	0.050	0.050	0.030	0.030	0.06	0.06	0.07	0.05	0.05	0.05	0.050
	Mode	0.020	0.020	0.029	0.025	0.025	0.019	0.019	0.019	0.015	0.015	0.015	0.015	0.016	0.016
	Upper	0.019	0.019	0.024	0.021	0.021	0.017	0.017	0.017	0.014	0.014	0.012	0.012	0.014	0.014

4.2 Sumner

Sumner beach is a northeast facing shoreline located at the southern extent of Pegasus Bay. The northern end of the shoreline is influenced by the Avon-Heathcote estuary inlet while the southern end is bound by Sumner Headland. Cave Rock is a basalt outcrop that extends into the sea and acts as a natural groyne blocking some of the sediment transport into Sumner Bay on the eastern side. On the western side of Cave Rock is Clifton Beach which is largely influenced by dynamics of the Avon-Heathcote estuary inlet-delta system. Sumner settlement has been well established since 1880 and there has since been shoreline modifications with numerous seawalls constructed.

4.2.1 Cell splits

The shoreline has been split into three coastal cells (Figure 4.15). Cell 27 is within Clifton Bay at the eastern edge of the Avon-Heathcote estuary inlet. The beach shoreline is protected by a rock revetment. Cell 28 is an unprotected beach shoreline within Clifton Bay, on the north-western side of Cave Rock. Both Cells 27 and 29 have been classified as Class 1 structures (Figure 4.16) (refer to Section 3.1.5).

Figure 4.15: Overview of cell splits along the Sumner shoreline.

4.2.2 Short term component (ST)

There are six beach profiles along the Sumner shoreline, one of them is in front of the revetment near Shag Rock, two of them are along the natural dune within Clifton Bay and three of them are along the revetment within Sumner Bay.

For the areas with Class 1 structures, the current hazard is defined as the immediate hazard if the structure were to fail. This is assessed based on the structure height and stable angle of repose (see Section 4.2.3.

The short-term component along Clifton Beach been assessed using the same beach profile analysis method as adopted for the Christchurch open coast (Section 4.1.2). Figure 4.17 shows profile

CCC0190 within Cell 28 and the regression analysis at the 2.5 m RL contour. While the beach width seaward of the dune tends to show large fluctuations (up to 100 m) in response to changes in the inlet delta, the dune toe (2.5 m RL contour) shows relatively small fluctuations (up to 6 m). It is likely that the wide beach provides a buffer against significant storm cut along the dune toe.

Figure 4.16: Site photos (taken August 2020) for the Sumner shoreline. (Left) unprotected dunes along Clifton Beach (Cell 28), (right) rock revetment along Sumner Bay with Sumner Headland in the background (Cell 29).

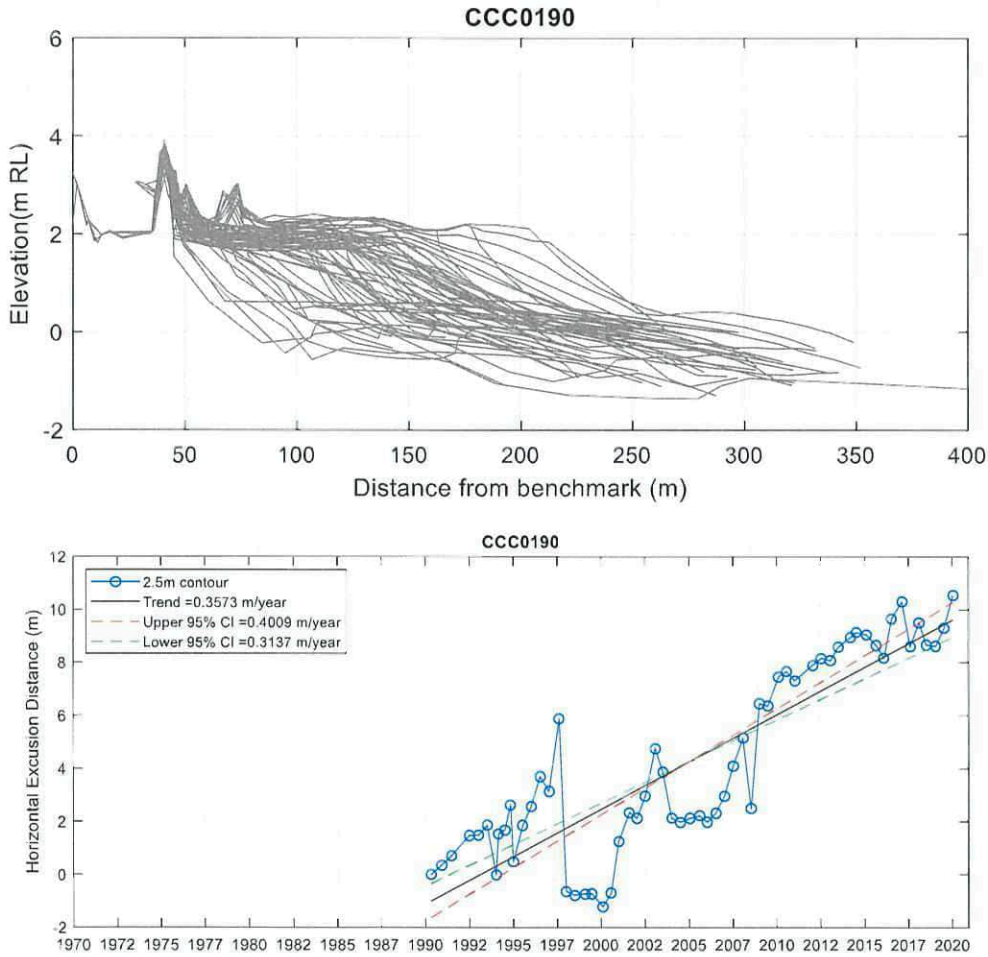

Figure 4.17: Beach profiles and regression plot for profile CCC0190 within Cell 28, Sumner.

As with the Christchurch open coast, the measured inter-survey storm cut distances have been assessed using an Extreme Value Analysis (EVA). Extreme value distributions for profile CC0190 (Cell 28) are shown in Figure 4.18. The mean storm cut at the 2.5 m RL contour Cell 28 is less than 1 m (Table 4.10).

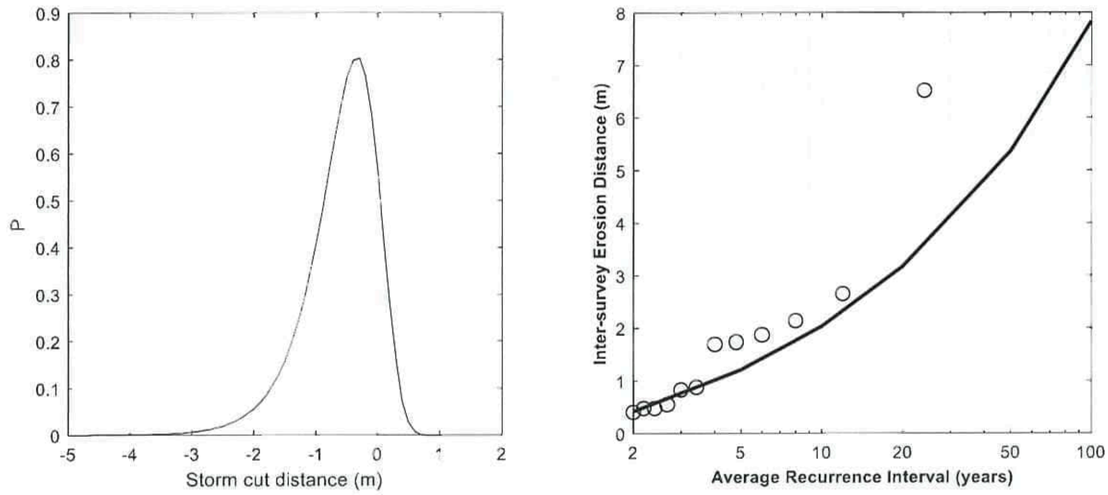

Figure 4.18: Example of extreme value distribution and curve for inter-survey storm cut distances at Clifton Beach (profile CC00190).

Table 4.10: Summary of extreme value distributions for inter-survey storm cut distances along Clifton Beach

Cell	Mean inter-survey storm cut (μ) (m)	Shape parameter (σ)[1]	Resultant 100 year ARI storm cut (m)
28	-0.34	0.5	-8

[1] Shape parameter describes the shape of the distribution (e.g., a larger shape parameter results in a wider distribution).

4.2.3 Dune stability (DS)

Dune stability for the Sumner coast has been assessed as described in Section 4.1.3. Parameter bounds are defined based on the variation in dune/structure height within the coastal cell and potential range in stable angle of repose (Table 4.11 and Table 4.12). The stable angle of repose for Cell 28 is based on the angle of repose for dune sand, while the stable angle of repose within Cells 27 and 29 is based on an assumed angle of repose for fill material behind the structure.

Table 4.11: Dune stability component values for Sumner Beach

Cells	Dune stability component values		
	Lower (degrees)	Mode (degrees)	Upper (degrees)
27	18	22	26.6
28	30	32	34
29	18	22	26.6

Table 4.12: Dune and structure height component values

Cell	Dune/structure height component values		
	Lower (m)	Mode (m)	Upper (m)
27[1]	2.5	2.8	3
28	0.5	1	2
29[1]	1	2	3

[1] Height of Class 1 structure.

4.2.4 Long-term component (LT)

It is apparent that Clifton Beach (Cell 28) has undergone periods of erosion and accretion which generally are related to changes in the adjacent inlet delta. Thompson (1994) found that periods of erosion at Southshore tend to correspond to accretion at Clifton Beach and *vice versa*. Findlay and Kirk (1988) state the main ebb channel from the estuary historically flowed south-east past Shag Rock and changed to its current position during 1938. The change in channel position is likely to have contributed to the extensive infilling of Clifton Beach between 1927 and 1950s.

Historical shoreline data (from aerial imagery) indicates Clifton Beach (Cell 28) has experienced long-term accretion, with up to 20 m accretion since the 1940s (Figure 4.19). The beach profile data also shows some fluctuations with overall accretion at the 2.5 m RL contour at an average rate of 0.36 m/year (Figure 4.17).

Hicks et al (2018a) noted that the phase of accretion since 2011 may be associated with effects from the earthquakes. Following the earthquake there was a reduction in the tidal prism and subsequently a reduced volume on both the ebb and flood tidal deltas at the inlet entrance. This reduction in delta size has potentially resulted in a surplus of sand being supplied to the adjacent shoreline and hence the period of accretion following 2011 (Figure 4.17).

As with the distal end of the Southshore Spit, there is high uncertainty in future erosion rates along the shoreline adjacent to the tidal inlet (see Section 4.1.4). SLR may result in an increased tidal prism within the Avon-Heathcote Estuary which would lead to widening of the tidal inlet and increased erosion along the adjacent shoreline. Quantification of this is, however, beyond the scope of this assessment.

Long-term rates adopted for the Clifton Beach are summarised in Table 4.13.

Figure 4.19: Historical shorelines along Clifton Beach.

Table 4.13: Adopted long-term component values for Sumner beach

Cell	Long-term rate (m/yr)[1]		
	Upper	Mode	Lower
28	0.4	0.2	0.1

[1] +ve values are accretion and -ve values are erosion.

4.2.5 Response to sea level rise (SLR)

The shoreline response to sea level rise has been assessed based on the Bruun model described in Section 0. Wave climate parameters and resultant closure depths are summarised in Table 4.14.

Table 4.14: Inner and outer profile closure depth estimates derived from Hallermeier's definitions with wave parameters sourced from the MetOcean wave hindcast

Cell	Significant wave height, $H_{S,12h}$ (m)	Wave period, $T_{p,12h}$ (s)	Inner closure depth, dl (m)	Outer closure depth, di (m)	Slope		
					Lower	Mode	Upper
28	3.01	10.06	7.11	10.67	0.014	0.016	0.014

4.2.6 Summary of components

Adopted component values for the Sumner shoreline are summarised in Table 4.15.

4.2.7 Uncertainties

Key uncertainties in the erosion hazard assessment along the Sumner shoreline include:

- Condition and design life of structures along the 'significantly modified shoreline'.
- Tidal inlet response to SLR and the subsequent effects on the long term trends along the adjacent shoreline at Clifton Beach.

Table 4.15: Adopted component values for the Sumner shoreline

Cell		27	28	29
Chainage, km from Waimakariri River mouth		35.6 to 36	36 to 36.6	36.6 to 37.8
Morphology		Class 1 structure	Dune	Class 1 structure
Geology		Anthropic deposits	Dune deposit	Anthropic deposits
Short-term (m)	mu (mean)	N/A	-0.34	N/A
	sigma (shape parameter)		0.5	
Dune (m above toe)	Lower	2.5	0.5	1
	Mode	2.8	1	2
	Upper	3.0	2	3
Stable angle (deg)	Lower	18	30	18
	Mode	22	32	22
	Upper	26.6	34	26.6
Long-term (m) -ve erosion +ve accretion	Lower	N/A	0.4	N/A
	Mode		0.2	
	Upper		0.1	
Closure slope	Lower		0.014	
	Mode		0.016	
	Upper		0.046	

4.3 Taylors Mistake

Taylors Mistake/Te Onepoto is a small, northeast-facing, pocket beach on the southern side of Sumner Head. The embayment is bound by volcanic cliffs on either side. The beach comprises fine sand with a relatively flat profile (approximately 1(V):20(H)). The beach at the northern end includes a slightly narrower dune with the surf club and at the southern end there is a wider dune system that has infilled a historical stream channel/lagoon (Figure 4.20). There is still an ephemeral stream channel which discharges onto the coast under high rainfall events.

Figure 4.20: Site photos (taken August 2020) along Taylors Mistake. (Top left) Oblique photo of southern end of shoreline (top right) fencing along dunes at northern end of shoreline, (bottom left) southern end of beach, (bottom right) historical stream mouth.

4.3.1 Cell splits

Taylors Mistake beach has been classified into one coastal cell, approximately 350 m long (Figure 4.21).

Figure 4.21: Overview of cell extent along the Taylors Mistake shoreline.

4.3.2 Short term component

For Taylors Mistake, the majority of the beach has natural dunes and therefore, the short-term component has been assessed based on the same approach as the Christchurch open coast, using the inter-survey horizontal excursion distance of the dune toe, measured from beach profiles (see Section 4.1.2).

Extreme Value Analysis (EVA) has been completed based on the measured inter-survey storm cut distances at each beach profile. The distribution from profile BPN8010 has been adopted as shown in Figure 4.22. The adopted extreme value distribution (mean and shape parameter values) for Taylors Mistake are summarised in Table 4.16. The 100 year ARI storm cut distance is also included for context.

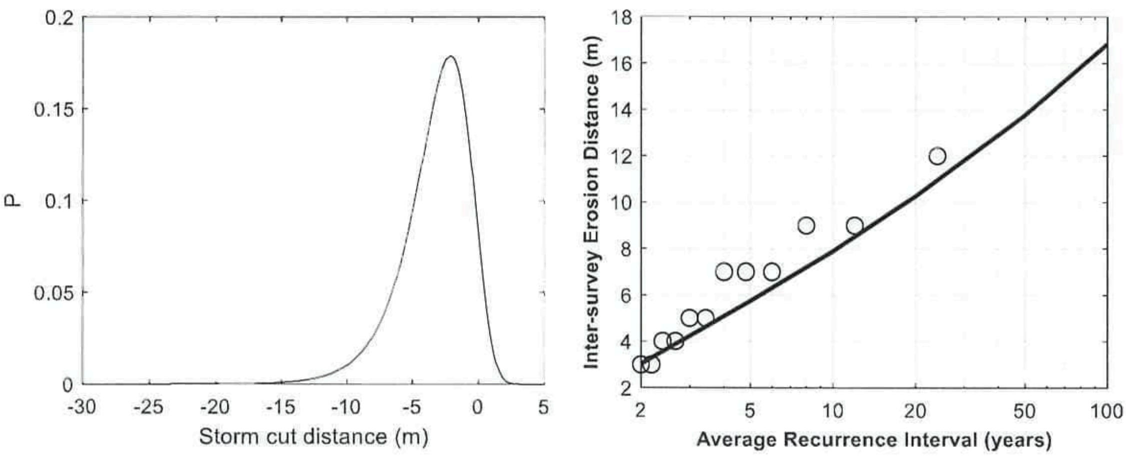

Figure 4.22: Extreme value distribution for inter-survey storm cut distances along Taylors Mistake.

Table 4.16: Summary of extreme value distributions for inter-survey storm cut distances at Taylors Mistake

Cell	Mean inter-survey storm cut (μ) (m)	Shape parameter (σ)[1]	Resultant 100 year ARI storm cut (m)
30	-6.5	1.8	-17

[1] Shape parameter describes the shape of the distribution (e.g., a larger shape parameter results in a wider distribution).

4.3.3 Dune stability (DS)

Dune stability for the Taylors Mistake has been assessed as described in Section 4.1.3. Parameter bounds are defined based on the variation in dune height within the coastal cell and potential range in stable angle of repose (Table 4.17 and Table 4.18).

Table 4.17: Dune stability component values for Taylors Mistake

Cell	Dune stability component values		
	Lower (degrees)	Mode (degrees)	Upper (degrees)
30	30	32	34

Table 4.18: Dune height component values for Taylors Mistake

Cell	Dune height component values		
	Lower (m)	Mode (m)	Upper (m)
30	0.8	1.1	1.5

4.3.4 Long-term trends (LT)

The long-term trends have been assessed based on historical shoreline data and beach profiles (Figure 4.23 and Figure 4.24). The beach profiles show that majority of the beach has been relatively stable with a slight erosion trend, except at profile BPN7975, at the southernmost end where there has been an accretion trend (Figure 4.23). The accretion trend at BPN7975 is not likely to be representative of the remainder of the beach as it is influenced partially be the stream and infilling that has occurred. The historical shorelines show that the southern end has infilled since at least 1974, with some fluctuations due to the ephemeral stream which discharges onto the coast under high rainfall events (Figure 4.23 and Figure 4.25).

Overall, since 1990 the beach has generally shown a slight erosion trend, ranging from -0.04 m/yr to -0.23 m/yr (Figure 4.23). Based on changes in historic shorelines from aerial photographs, and variation in mean regression rates measured from the beach profile data, the adopted long-term rates range from 0.2 m/year to -0.2 m/year (Table 4.19).

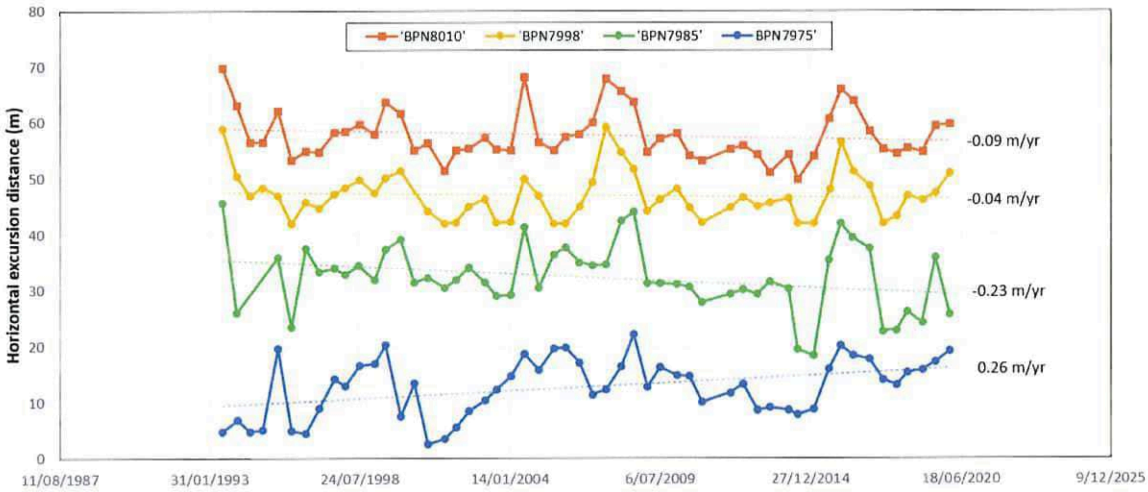

Figure 4.23: Regression analysis of beach profiles along Taylors Mistake.

Figure 4.24: Historic shorelines for Taylors Mistake.

Figure 4.25: Google Earth photos showing the ephemeral stream discharging onto the beach in September 2010, infilling by February 2016 and again discharging during August 2019.

Table 4.19: Adopted long-term rates along Taylors Mistake

Cell	Long-term rate (m/yr)		
	Lower	Mode	Upper
30	-0.2	-0.05	0.2

NOTE: Positive values are accretion and negative values are erosion.

4.3.5 Response to sea level rise (SLR)

The shoreline response to sea level rise has been assessed based on the Bruun model described in Section 0. Wave climate parameters and resultant closure depths are summarised in Table 4.20

Table 4.20: Inner and outer profile closure depth estimates derived from Hallermeier's definitions with wave parameters sourced from the MetOcean wave hindcast

Cell	Profile	Significant wave height, $H_{s,12hr}$ (m)	Wave period, $T_{p,12hr}$ (s)	Inner closure depth, dl (m)	Outer closure depth, di (m)	Slope[1]		
						Lower	Mode	Upper
30	BPN7998	3.02	8.12	6.8	10.2	0.016	0.017	0.076

[1] Average profile based on beach profile dataset. Offshore profile interpolated based on LINZ contour data.

4.3.6 Summary of components

Adopted component values for Taylors Mistake are summarised in Table 4.21.

4.3.7 Uncertainties

Key uncertainties in the erosion hazard assessment for the Taylors Mistake shoreline include:

- The influence of the ephemeral stream on short-term storm cut and long term trends.
- Future sediment supply and subsequently the long term accretion rates.

Table 4.21: Adopted component values for Taylors Mistake

Cell		30
Chainage, km from Waimakariri River mouth		40.6 to 41.1
Morphology		Dune
Geology		Dune deposit
Short-term (m)	mu (mean)	-6.5
	sigma (shape parameter)	1.8
Dune (m above toe)	Lower	0.8
	Mode	1.1
	Upper	1.5
Stable angle (deg)	Lower	30
	Mode	32
	Upper	34
Long-term (m) -ve erosion +ve accretion	Lower	-0.2
	Mode	-0.05
	Upper	0.2
Closure slope	Lower	0.016
	Mode	0.017
	Upper	0.076

4.4 Avon-Heathcote estuary

The Avon-Heathcote Estuary is a shallow intertidal estuary on the eastern side of Christchurch City. The Avon River flows into the northeastern corner and the Heathcote River into the southwestern corner. Their combined catchments give the estuary a total catchment area of 200 km^2 (MacPherson, 1978).

The estuary has a short inlet connection with the sea at the southern end and is partially enclosed by the 4 km long Southshore Spit. The estuary is on a coastal plain which consists of Late Quaternary terrestrial and estuarine gravels, sands, peats and mud. At the southern margin is the volcanic rock of Banks Peninsula. The suburbs which boarder the western side of the estuary were extensive swamplands until European settlement in the 1850s. The estuary has naturally infilled over time however early urbanisation led to a rapid increase of fine sediment to the estuary, particularly between 1850 and 1875 (MacPherson, 1978).

The southern margin of the estuary has undergone significant modification with construction of sea walls, causeways and reclamation. Historically, there was a vegetated flat island (Skylark Island) off the eastern end of McCormacks Bay. Erosion of the island began immediately after construction of the McCormacks Bay causeway in 1907 and by 1920 the island was reduced to mudflats (Findlay, 1988). Other modifications include the construction of various public and private seawalls along Southshore, Main Road, Beachville Road and Humphreys Drive. Findlay (1988) state the Beachville Road seawall was constructed in 1933.

4.4.1 Cell splits

The Avon-Heathcote estuary has been split into 12 cells (Figure 4.26). The eastern margin of the estuary is characterised by low-lying, unconsolidated shoreline with ad-hoc structures along sections (Figure 4.27). Sections of the shoreline, particularly at the northern end (i.e., Cell 20) are fronted with salt marsh vegetation. The western margin is characterised by unconsolidated estuary deposits and includes the Bromley oxidation ponds which comprise anthropic fill material along the shoreline. The southern end of the estuary (Cells 25 and 26) is classified as 'significantly modified shoreline' (see Section 3.1.5), comprising the causeway, some reclamation and various protection structures since the early 1900's (Figure 4.27).

Figure 4.26: Overview of cell extents around the Avon-Heathcote estuary shoreline.

Figure 4.27: Site photos (taken August 2020) around the Avon-Heathcote estuary shoreline. (Top left) natural unconsolidated shoreline on the western side of Southshore spit (Cell 15), (top right) rip rap along the unconsolidated shoreline near Penguin Street (Cell 17), (centre left) eroded shoreline near South New Brighton Park (Cell 19), (centre right) gravel shoreline near Windsurfer's Reserve (cell 24), (bottom left) protected bank near Humphreys Drive (Cell 25), (bottom right) protected bank near Beachville Road (Cell 26).

4.4.2 Short term component (ST)

The short term storm cut component along the estuary shoreline has been assessed based on the convolution method developed by Kriebel & Dean (1993). The method considers beach profile equilibrium response to storm events. The method includes initial beach geometry, peak nearshore water level and breaking wave height to determine the maximum potential erosion that would be achieved if the beach could respond to equilibrium (Figure 4.28 and Equation 4.4). Due to the method using equilibrium profiles the storm cut distances are conservative as it is not restricted to the storm event duration. However, as resulting storm cut distances are relatively small, the approach is considered acceptable.

$$R_\infty = \frac{S(x_b - \frac{h_b}{m})}{B + h_b - \frac{S}{2}} \tag{4.4}$$

Where:

S	=	Water level rise.
X_b	=	Distance to breaking location.
h_b	=	Breaking depth.
m	=	(Linear) beach slope.
B	=	Berm height above the initial water level.

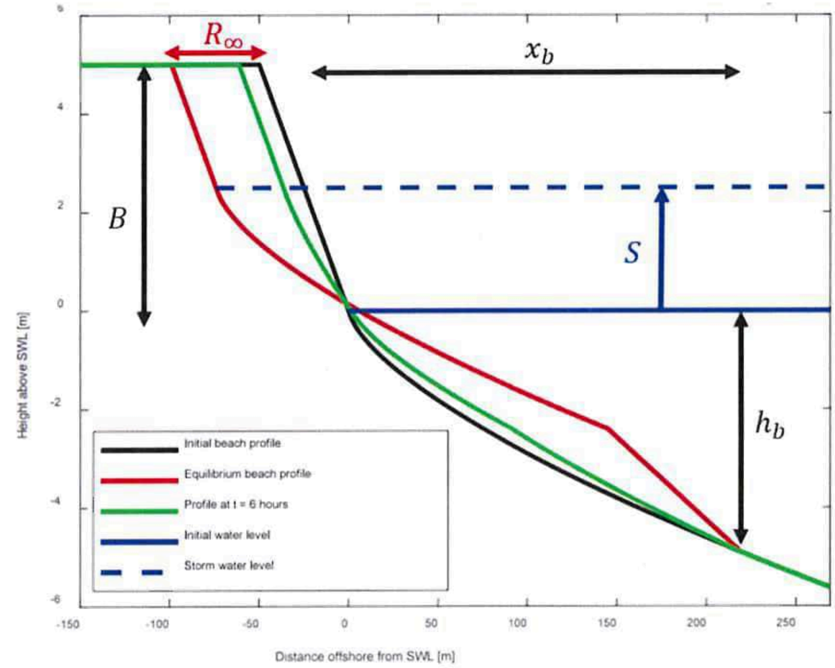

Figure 4.28: Schematic showing the beach storm response based on Kriebel and Dean (1993).

The short term storm cut has been assessed for a range of different storm tide levels and wave heights (Table 4.22). Assessed storm tide levels are based on the 1 year, 10 year and 100 year ARI water levels from the Bridge Street tide gauge. Wave heights are based on the wave height range simulated in the SWAN model (see Appendix A). Based on the LiDAR data a representative profile with an assumed berm elevation of 1.5 m NZVD-16 and an upper slope of 5(H):1(V) has been adopted for assessing the short term along the unconsolidated shoreline within the Avon-Heathcote estuary. Results indicate the short-term component ranges from 1 to 5 m (Table 4.22 and Figure 4.29).

Table 4.22: Summary of storm tide and wave heights used to assess storm cut along the estuary shoreline

	Storm tide level (m NZVD)[1]	Breaking wave height[2] (m)	Storm cut (m)
Lower bound	1.33	0.4	1
Mode	1.59	0.6	3
Upper bound	1.89	0.8	5

[1] Based on 1 year, 10 year and 100 year storm tide levels within the Avon-Heathcote estuary.

[2] Based on SWAN model outputs, for the average 1 year, 10 year and 100 year ARI wind speeds see Appendix B).

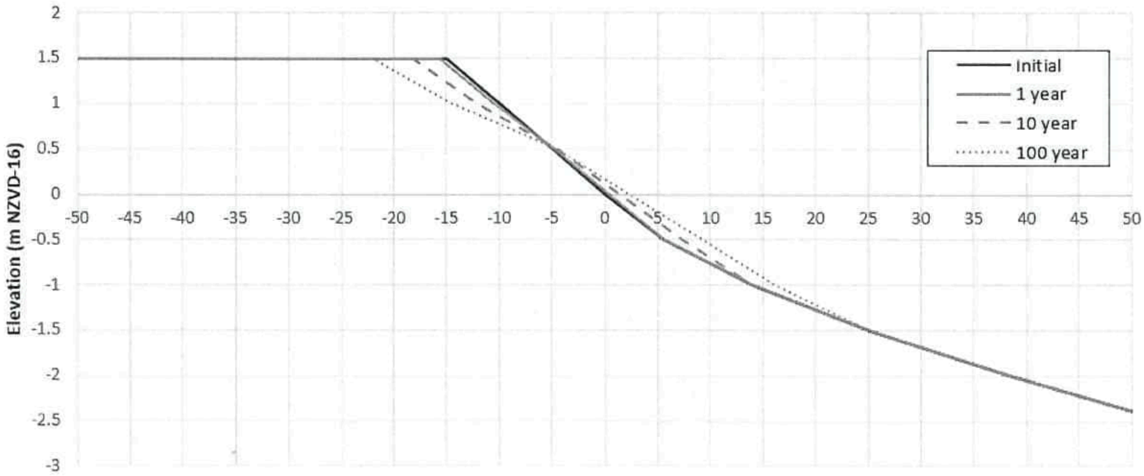

Figure 4.29: Example of beach response for the estuary shoreline under different storm conditions.

4.4.3 Long-term trends (LT)

The long-term component has been assessed based on regression analysis of historic shorelines derived from aerial photographs. Due to tree coverage and marsh vegetation, it is difficult to accurately identify the shoreline position along the entire site, particularly in the earlier historic aerials. Subsequently, the long-term trends have been assessed along several representative transects around the estuary (Figure 4.30).

Figure 4.30: Location of transects used to assess the long term trends around the Avon-Heathcote estuary.

4.4.3.1 Impact of the 2011 Canterbury Earthquakes

The effect of the 2011 Canterbury earthquake sequence (CES) has been considered. There are areas where there has been significant erosion of estuary vegetation due to land subsidence following the quakes. The significant loss of vegetation is not likely to be indicative of the long-term trends but instead shows the instantaneous response to subsidence. Therefore, long term rates have been based on the pre-quake trends.

Figure 4.31 provides an example along the Southshore shoreline where there has been increased shoreline erosion following the quake. The long-term trend pre-quake was -0.16 m/yr while the long-term including the earthquake induced erosion is -0.22 m/yr. Cells 16 to 24 typically show an increased erosion rate including post-quake (i.e. 1941 to 2020) compared with pre-quake (i.e. 1941 to 2011) (Table 4.23). It is uncertain whether the increased erosion rate will continue or whether the shoreline has reached an equilibrium state.

There is also uncertainty in how the areas of shoreline where there is salt marsh will adjust once the salt marsh vegetation is eroded away. It is possible that the shoreline landward of the salt marsh will erode at a slower rate compared to the erosion rate measured for the salt marsh vegetation.

However, in some areas the loss of vegetation may increase the shoreline exposure and subsequently the erosion rate.

Based on these uncertainties adopted long-term rates are based on the pre-quake trends and the transects less influenced by salt marsh vegetation (i.e. AH5). Adopted parameter bounds for long term component within each cell around the Avon-Heathcote estuary are presented in Table 4.24.

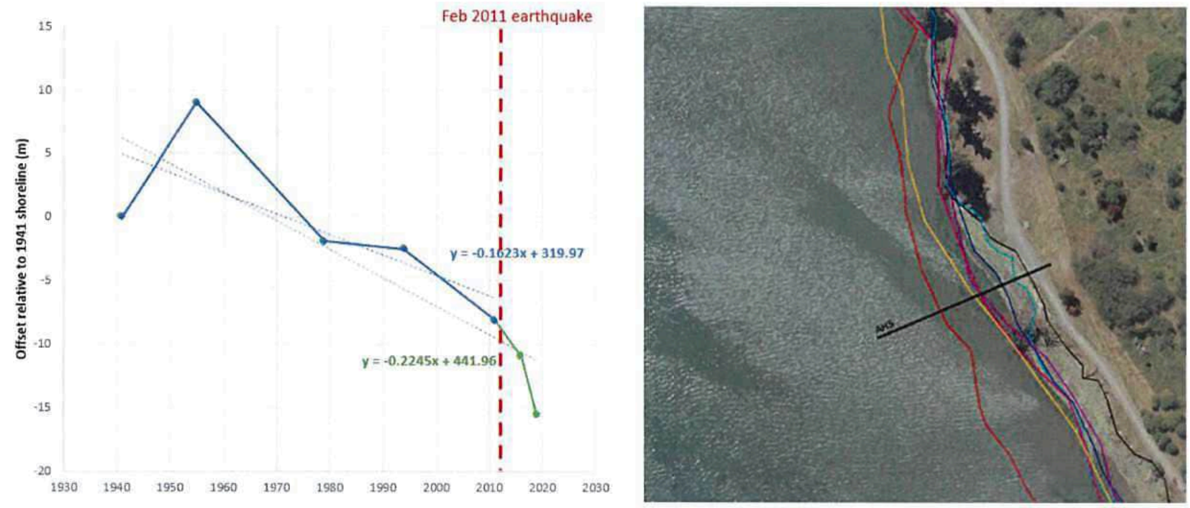

Figure 4.31: Example of long-term trends assessed using historic shoreline data within the Avon Heathcote Estuary. Profile is AH5 within Cell 19.

Table 4.23: Summary of regression rates measured from aerial imagery along each transect

Cell	Transect	Average regression rate (m/yr)[1,2]	
		1941 to 2020	1941 to 2011 (pre-quake)
15	AH1	+0.37	+0.44
	AH1a	+0.14	+0.17
16	AH2	+0.08	+0.07
17	AH3	-0.34	-0.17
	AH4	-0.45	-0.40
	AH4a	-0.12	-0.16
19	AH5	-0.22	-0.16
	AH5a	-0.08	-0.07
20	AH6	-0.18	-0.18
21	AH7	-0.30	-0.03
	AH8	-0.15	-0.11
23	AH9	-0.13	-0.13
24	AH10	-0.01	0

[1] +ve values are accretion and -ve values are erosion.

[2] Long-term rates for AH2 to AH5 are potentially influenced by shoreline protection works.

4.4.4 Dune/bank stability (DS)

Dune and bank stability around the Avon-Heathcote estuary has been assessed as described in Section 4.1.3. Parameter bounds are defined based on the variation in dune/bank height within the coastal cell and potential range in stable angle of repose. Adopted dune/bank heights and stable angles are shown in Table 4.24.

4.4.5 Response to sea level rise (SLR)

The estuary shoreline typically comprises either silty sand, fine sand, shell or mixed sand and gravel on the upper beach face with a wide intertidal zone and no extensive dune system. Due to this variation between the composition of the upper beach and the intertidal flats, the estuary shoreline is expected to behave differently to sandy beaches in response to a rise in mean sea level. The effect of sea level rise on estuarine type shorelines can be highly variable and complex and will depend on the interrelationship between:

- Backshore topography and geology.
- Sediment supply and storage.
- The wave energy acting on the shoreline.

While estuaries tend to be areas of sediment deposition, it is expected that future sea level rise will be greater than the rate of sedimentation and therefore there will be an increase in water depth across the estuary. The greater water depth will allow greater wave heights to act on the shoreline and subsequently increase the erosion potential. However, as it is a lower energy environment, erosion is likely to occur more episodically and more slowly than a more energetic open coast environment.

The traditional Bruun Rule, developed for open coast uniform sandy beaches that extend down beyond where waves can influence the seabed, does not directly apply for estuarine beaches where the upper beach is a markedly different composition from the intertidal areas. However, a modified equilibrium beach concept that assumes that the upper beach profile is likely to respond to increasing sea level rise with an upward and landward translation over time was accepted by the peer review panel (Kenderdine et al., 2016) as appropriate in this setting and was applied for harbour environments by T+T (2017). The landward translation of the beach profile (SL) can be defined as a function of sea level rise (SLR) and the upper beach slope (tanα). The upper beach slope above the intersection of the beach and the fronting intertidal flats has been adopted for each cell. The equilibrium profile method relationship is given in Equation 4.5.

$$SL = \frac{SLR}{\tan\alpha} \tag{4.5}$$

Where:

SLR	=	Increase in sea level rise (m) for the areas where the present height of beach above MHWS is higher than projected sea level rise increase; or the height of the beach above MHWS where the beach is lower than the sea level rise value.
tanα	=	Average slope of the upper beach.

In low energy environments there is likely to be insufficient energy to reform the beach crest to match the increase in sea level and subsequently once sea levels exceed the crest, inundation becomes the more significant controlling factor. Therefore, the maximum potential extent of SLR induced erosion for low-lying beach areas is assumed to be controlled by the crest height above the MHWS. Where the beach crest is higher than the projected sea level rise, the sea level value has been used. This means that when sea level exceeds the crest height and inundation occurs, there is no additional increase in erosion of the present-day shoreline. This method approximately follows the method by Komar et al. (1999), with the MHWS adopted as the dune-toe level.

The land subsidence that occurred during and following the 2011 earthquake provides an example of instantaneous sea level rise and the subsequently shoreline response. Figure 4.32 shows a cross-section along the South Brighton shoreline where the beach face slope is approximately 10%. Between February 2011 and December 2011, the LiDAR indicates the land subsided by approximately 0.25 m which is in line with the findings from Orchard (2020). The subsidence was equivalent to 0.25 m SLR and based on the LiDAR resulted in approximately a 2.5 m landward shift in shoreline position (Figure 4.32). This example of instantaneous SLR demonstrates that Equation 3-5 is appropriate for estimating the estuary shoreline response to future SLR. The adopted upper beach slopes for cells around the Avon-Heathcote estuary are shown in Table 4.24.

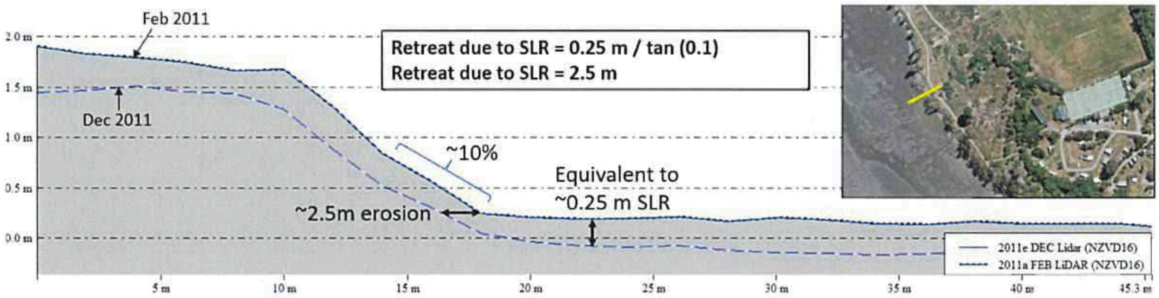

Figure 4.32: Example of shoreline response to SLR within the Avon-Heathcote Estuary.

4.4.6 Summary of components

Adopted component values for the Avon-Heathcote estuary are shown in Table 4.24.

4.4.7 Uncertainties

Key uncertainties in the erosion hazard assessment for the Avon-Heathcote estuary include:

- Long term erosion rates in absence of the protection structures, particularly along the southern margin.
- Long-term rates following the CES. Adopted rates are based on the pre-quake rates which could be non-conservative.
- Condition and design life of structures along the 'significantly modified shoreline'.
- Short term storm response. The Kriebel and Dean (1993) method does not account for different sediment types across the profile or response to short-duration events.
- Shoreline response to SLR. Estuaries are areas of deposition and infilling so if sedimentation rates are high, the shoreline may adjust and keep pace, depending on the rate of SLR.
- The effect of modified hydrodynamics and sediment transport regimes due to changed bed levels following the earthquake.

Table 4.24: Summary of adopted component values for the Avon-Heathcote estuary

Cell		15	16	17	18	19	20	21	22	23	24	25	26
Chainage, km from Waimakariri River mouth		20.5 to 21	21 to 21.5	21.5 to 22.8	22.8 to 23.2	23.2 to 24.25	24.25 to 25.5	25.5 to 26.6	26.6 to 28.35	28.35 to 29	29 to 30	30 to 31.1	31.1 to 35.6
Morphology		Harbour beach	Harbour beach	Harbour beach	Harbour beach	Harbour beach	Harbour beach	Harbour beach	Harbour beach	Harbour beach	Harbour beach	Class 1 Structure	Class 1 Structure
Geology		Dune deposit	Dune deposit	Dune deposit	Dune deposit	Dune deposit	Dune deposit	Anthropic deposits	Anthropic deposits	Anthropic deposits	Estuarine deposit	Anthropic deposits	Anthropic deposits
Short-term (m)	Lower	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1	N/A	N/A
	Mode	-3	-3	-3	-3	-3	-3	-3	-3	-3	-3		
	Upper	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5		
Dune (m above toe)	Lower	1	0.8	1.1	1.2	0.6	0.8	1.5	1.5	1.8	0.8	1.8	3
	Mode	1.6	1	1.3	1.5	0.8	1	2	2.5	2	1	1.9	3.5
	Upper	2.3	1.5	1.8	2	1.5	2	3.5	3	2.5	1.5	2.0	4
Stable angle (deg)	Lower	30	30	30	30	30	30	30	30	30	30	18	18
	Mode	32	32	32	32	32	32	32	32	32	32	22	22
	Upper	34	34	34	34	34	34	34	34	34	34	26.6	26.6
Long-term (m) -ve erosion +ve accretion	Lower	0	0	-0.18	-0.18	-0.18	-0.18	-0.18	-0.18	-0.15	-0.15	N/A	N/A
	Mode	0.15	0.05	-0.15	-0.15	-0.15	-0.15	-0.15	-0.15	-0.13	-0.13		
	Upper	0.30	0.08	-0.12	-0.12	-0.07	-0.07	-0.05	-0.05	0	0		
Closure slope	Lower	0.05	0.05	0.05	0.05	0.05	0.05	0.08	0.08	0.08	0.05	N/A	N/A
	Mode	0.06	0.06	0.06	0.06	0.08	0.08	0.1	0.1	0.1	0.06		
	Upper	0.08	0.1	0.1	0.1	0.1	0.12	0.12	0.12	0.12	0.08		

4.5 Banks Peninsula harbours (detailed sites)

Banks Peninsula comprises two large Miocene composite volcanic cones where the central areas have collapsed and been eroded. Subsequent drowning by the sea has resulted in the formation of Lyttleton and Akaroa Harbours. The present-day harbour morphologies are the product of weathering and marine incision of the crater remnants over millions of years (Hart, 2009). The heads of both harbours are characterised by shallow intertidal flats which have gradually infilled with the predominantly fine-grained loess and volcanic sediment runoff from their surrounding catchments.

Lyttelton Harbour (Whakaraupō) is on the northern side of the Peninsula and is a 15 km long, rock-walled inlet with an average width of approximately 2 km. The steep rocky slopes descend to a near-flat seabed with a maximum depth of 15.5 m below MLWS. The upper harbour comprises three bay, Governor's Bay, Head of the Bay and Charteris Bay separated by peninsulas and Quail Island.

Lyttelton Harbour also includes the Port of Lyttelton which was constructed between 1863 and 1876. Large scale dredging has occurred since 1876 and historically dredged sediment was deposited at Camp Bay, Little Port Cooper and Gollans Bay. Since 1990 the dredged sediment has been deposited on the northern side of the harbour inlet (Livingston, Breeze and Mechanics Bay) (Hart, 2013).

Akaroa Harbour is on the southern side of Banks Peninsula and is approximately 17 km long with an average width of 2 to 3 km. The upper harbour is surrounded by a radial pattern of hills and valleys while the lower harbour shoreline is dominated by steep cliffs of basalt and andesite rock. Maximum water depths at the harbour entrance are 25 m and reduce to 10 m along the southern 9.5 km. The bays in the upper harbour (e.g. Duvauchelle and Takamatua) are predominantly sandy silt with very shallow intertidal flats and shore platforms. Bays in the middle section (e.g. Tikao and Akaroa) are mostly sand with some gravel and the southern bay (e.g. Wainui) comprises gravel.

Sites with detailed assessments include the harbour beach and bank shorelines where substantial development has occurred, including Corsair Bay, Cass Bay, Rapaki Bay, Charteris Bay, Hays Bay, Purau, Akaroa, Takamatua Bay, Duvauchelle Bay and Wainui (Figure 4.33 and Figure 4.34). As stated in Section 3.4.3, there is limited data available to assess these sites with a full probabilistic, detailed approach and therefore generic assumptions have been made around several of the parameter bounds, resulting in a quasi-probabilistic approach.

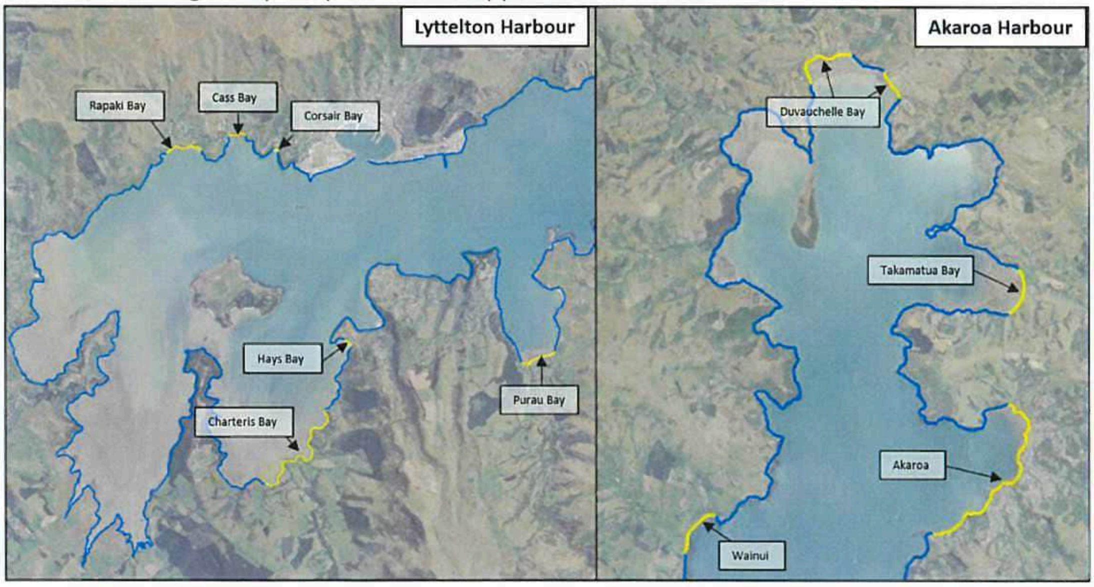

Figure 4.33: Overview map of detailed sites (yellow line) within the harbours.

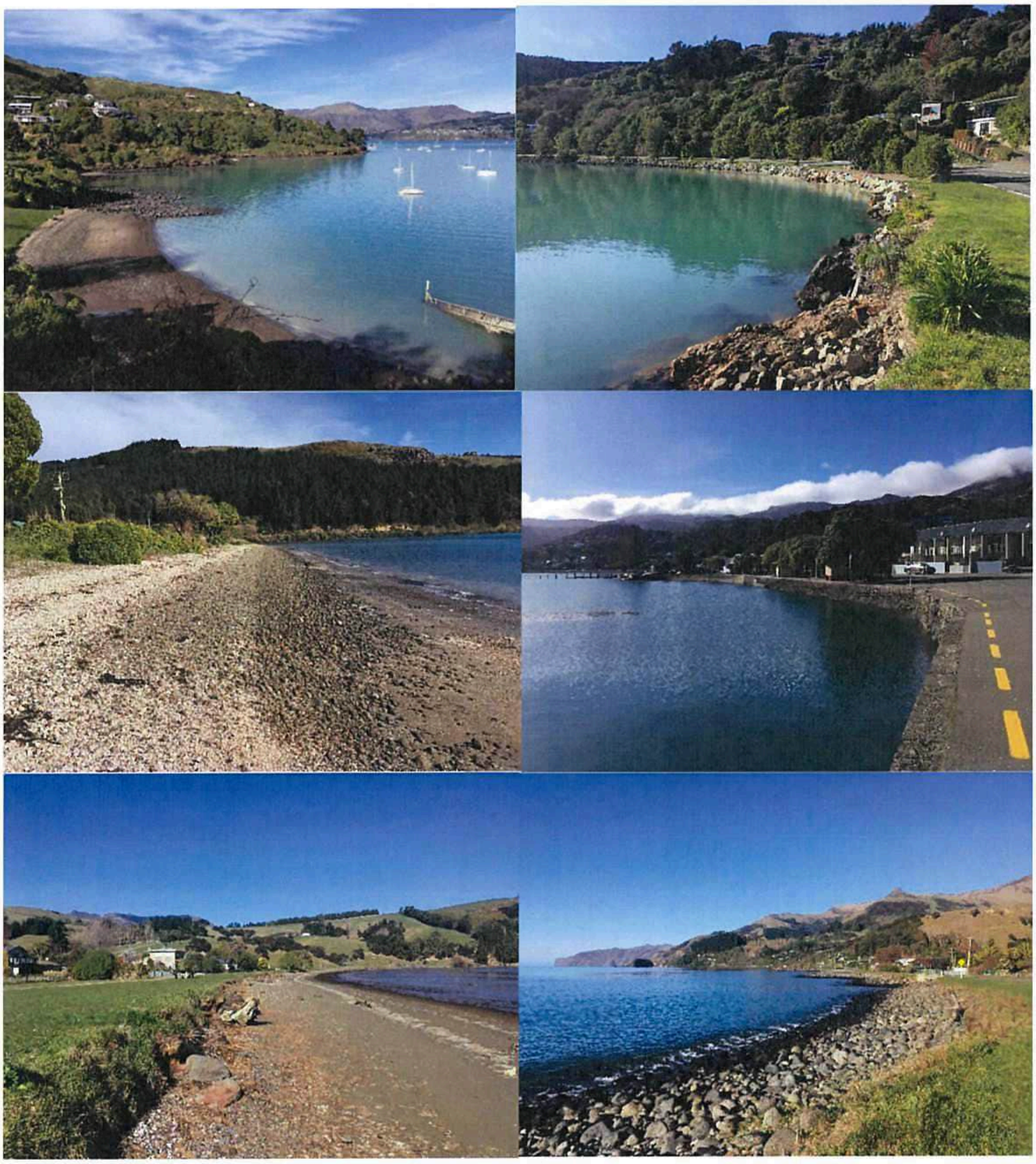

Figure 4.34: Site photos (taken August 2020) along the Lyttelton and Akaroa harbour shorelines. (Top left) Cass Bay, (top right) Charteris Bay, (centre left) Purau Bay, (centre right) Akaroa (bottom left) Takamatua Bay, (bottom right) Wainui.

4.5.1 Short term component (ST)

The storm term component for the harbour beaches has been assessed using the same Kriebel & Dean (1993) method as adopted for the ST component within the Avon-Heathcote estuary (see Section 4.4.2). The storm tide levels and wave heights used to assess the short term component within Lyttelton and Akaroa Harbours are shown in Table 4.25 . Based on LiDAR, a range of different berm elevations with an upper slope of 5(H):1(V) has been assessed. Results indicate the short term component ranges from -2 to -8 m.

While there is some variation in the exposure of the harbour beaches, most of the sites are fronted with tidal flats which dissipate wave energy and therefore the depth-limited wave heights at each site are similar. Subsequently, the short term parameter bounds for all harbour sites has been assumed the same. For the consolidated banks the short term component is not applicable as the banks behave differently to the unconsolidated beaches (see Section 3.1.3).

Table 4.25: Summary of storm tide and wave heights used to assess storm cut on the beaches within Lyttelton and Akaroa Harbours

	Lyttelton storm tide level (m NZVD)[1]	Akaroa storm tide level (m NZVD)[1]	Breaking wave height[2] (m)	Storm cut (m)
Lower bound	1.45	1.69	0.8	-2
Mode	1.67	1.90	1.2	-4
Upper bound	1.88	2.12	1.5	-6

[1] Based on 1 year, 10 year and 100 year storm tide levels.

[2] Based on SWAN model outputs for the average 1 year, 10 year and 100 year ARI wind speeds (see Appendix A).

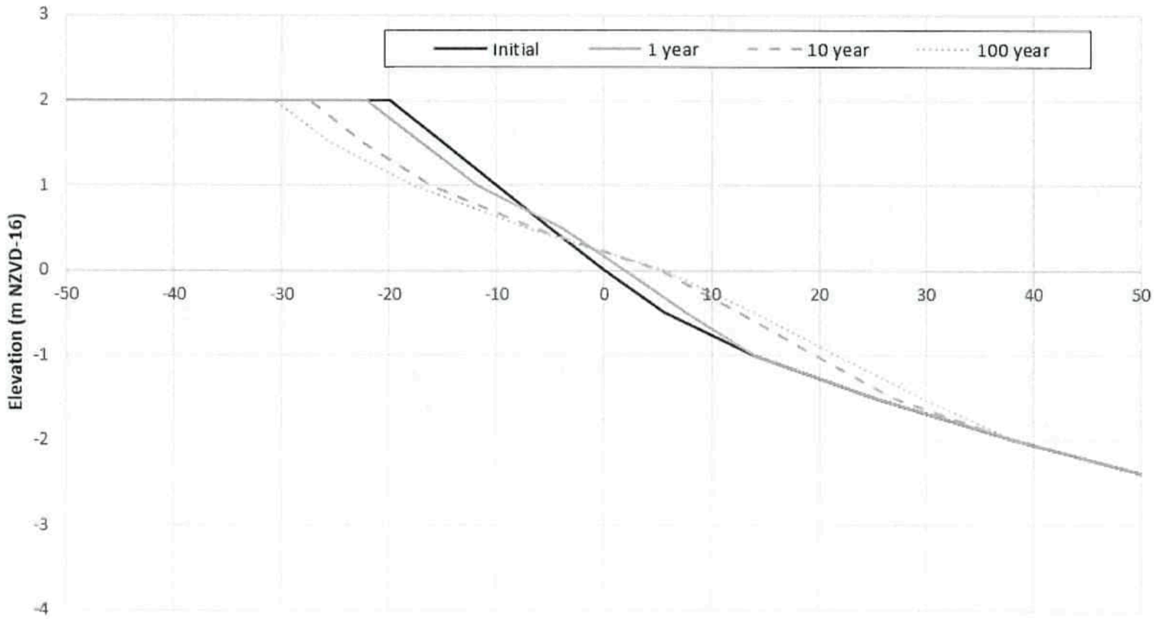

Figure 4.35: Example of beach response under different storm conditions within Lyttelton Harbour.

4.5.2 Long-term trends (LT)

Long-term trends within the harbour sites have been assessed based on analysis of historic aerial photographs. For most of the harbour sites the earliest historic aerial available is 1970. A significant portion of the beach and bank shoreline within Lyttelton and Akaroa Harbours has some form of protection structure along the toe and subsequently, there are limited areas available to measure natural long-term rates in absence of the structures. Due to this limitation, long-term rates have been assessed at discrete locations on unprotected shorelines Figure 4.36 provides an example of LT trends measured along transects at the unprotected shoreline within Charteris Bay and Allandale.

In areas where protection structures exist it is difficult to determine long-term rates, however, in absence of the structures, erosion is likely to occur and hence the structure exists. Site observations show evidence of scour and overtopping around structures which also implies that in absence of the structure, shoreline retreat is likely to occur (Figure 4.37). Similarly, some of the unprotected shorelines show minimal erosion in the historic aerials, however based on site observations there is evidence of undercutting and erosion, for example at Takamatua (Figure 4.38). In contrast, there are some sites which appear stable from the site observations and as expected show minimal movement in the historic aerials, for example Purau Bay and Cass Bay (Figure 4.39).

Subsequently adopted LT rates have been based on a combination of site observations and historic shoreline analysis. The shoreline analysis shows the highest rate of erosion occurring along the harbour beach shoreline within Charteris Bay, where the average rate of regression is -0.17 m/year since 1970 (Figure 4.36). Majority of the other unprotected harbour shorelines show lower erosion rates around 0 to -0.07 m/year, for example at Allandale (Figure 4.36).

For the bays where there are protection structures or evidence of active erosion the adopted LT rate ranges from -0.01 to -0.07 m/yr. These rates are based on the LT erosion measured at unprotected harbour sites. For the shorelines that appear stable and show no measurable erosion, a lower bound of 0 m/year has been adopted. The more stable shorelines tend to be within Lyttelton Harbour, such as Purau Bay and Hays Bay, whereas the detailed Akaroa sites tend to show more evidence of erosion. Adopted long-term trends for each cell are shown in Table 4.26 and Table 4.27.

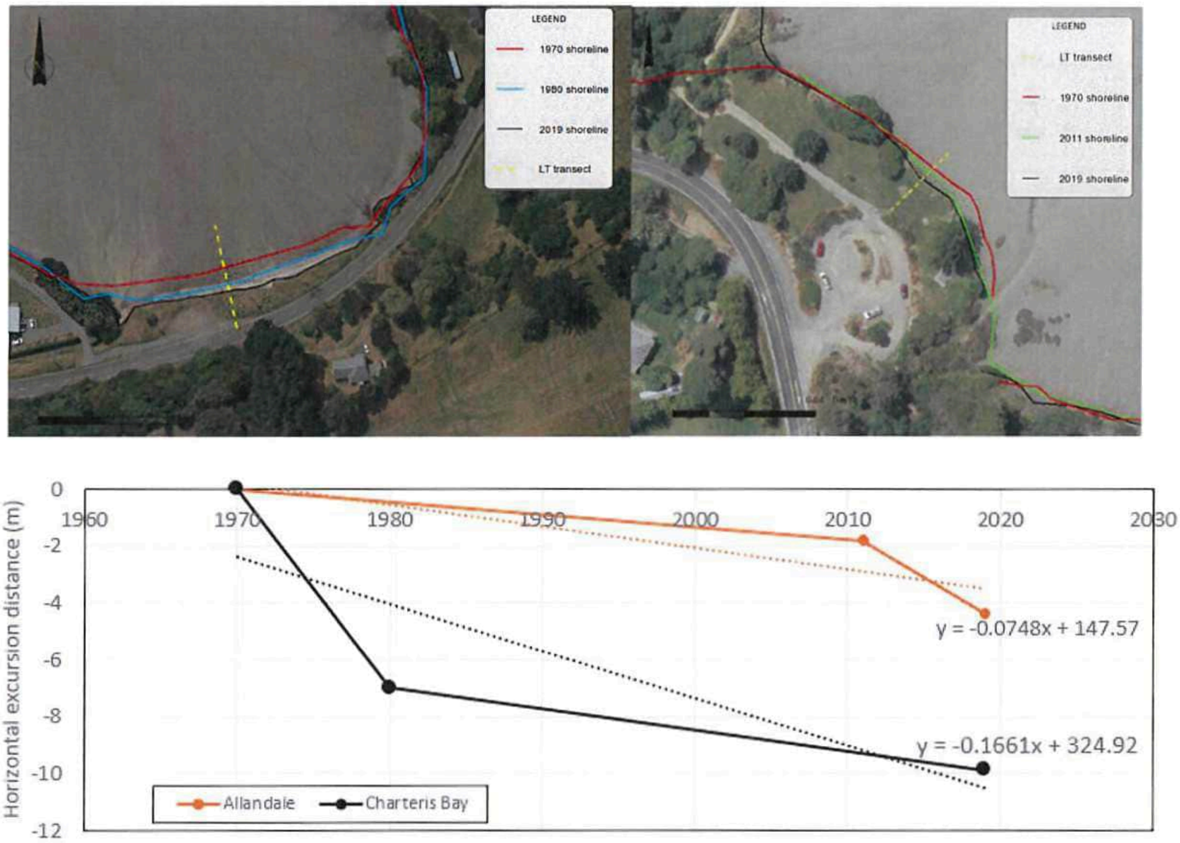

Figure 4.36: Example of shoreline analysis along transects at Charteris Bay (left) and Allandale (right).

Figure 4.37: Example of protection structures with evidence of scour and overtopping (Duvauchelle Bay).

Figure 4.38: Example of unprotected eroding harbour shorelines. (Left) Takamatua Bay, (right) Charteris Bay.

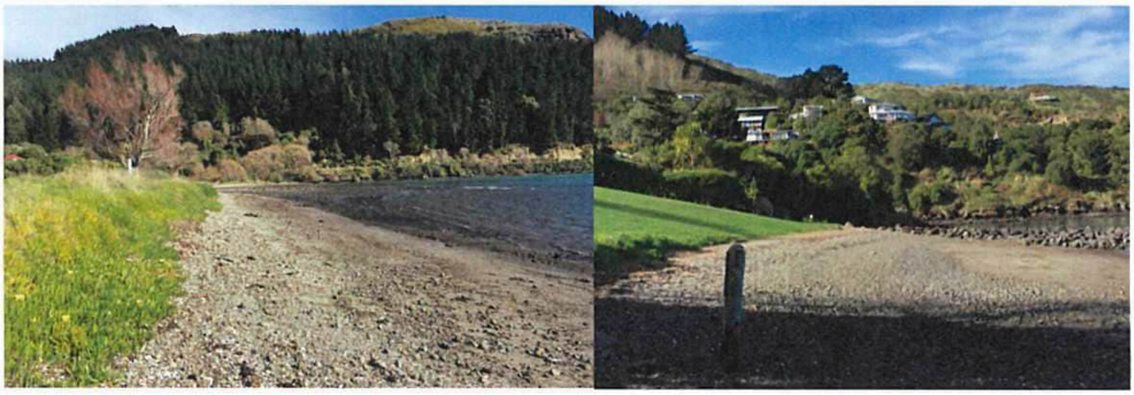

Figure 4.39: Example of stable unprotected harbour shorelines. (Left) Parau Bay, (right) Cass Bay.

4.5.3 Dune and bank stability (DS)

Dune and bank stability for the detailed sites around Lyttelton and Akaroa Harbours has been assessed as described in Section 4.1.3. Parameter bounds are defined based on the variation in dune/bank height within the coastal cell and potential range in stable angle of repose, which for beach sand is between 1(V):1.7(H) to 1(V):1.5(H) and for consolidated banks is assumed between 1(V):2(H) to 1(V):3(H). The slope stability for the underlying geology of the consolidated banks has not been included within this assessment, however based on the range of existing slopes, 1(V):2(H) to 1(V):3(H) is appropriate. Adopted dune/bank heights and stable angles for each cell are shown in Table 4.26 and Table 4.27.

4.5.4 Response to sea level rise (SLR)

4.5.4.1 Harbour beaches

The beaches within Lyttelton and Akaroa Harbours consist of either silty sand, fine sand, shell or mixed sand and gravel and have a wide intertidal zone with no extensive dune system. Majority of the terrestrial sediments supplied to the beach areas are from the catchment via the streams that discharge to the coast and, to a lesser degree, from erosion from the cliff coasts adjacent. Therefore, they are expected to behave differently to sandy beaches in response to a rise in mean sea level. The same method as adopted for the SLR response within the Avon-Heathcote estuary has been adopted for the beaches within Lyttelton and Akaroa Harbours (see Section 4.4.5). Adopted slope values are summarised in Table 4.26 and Table 4.27.

4.5.4.2 Harbour banks

There are several detailed assessment sites within the harbours which include consolidated banks. These sites include Corsair Bay, Cass Bay, Rapaki Bay and part of Duvauchelle Bay. While shorelines like Corsair Bay and Cass Bay have sandy sediment along the foreshore, the backshore is characterised by consolidated material. These consolidated banks are likely to respond differently to SLR compared with the beaches.

Sea level rise may increase the amount of wave energy able to propagate over a fronting platform or beach to reach a bank/cliff toe, removing talus more effectively and increasing the potential for hydraulic processes to affect erosion and recession. However, in some locations, the existence of a talus will provide self-armouring, and may slow bank recession due to waves.

Aston et al. (2011) propose a generalised expression for future recession rates of cliff coastlines shown in Equation 4-6 where *LT* is the background erosion rate and S_1 is the historic rate of SLR, S_2 the rate of future SLR and *m* is the coefficient, determined by the response system (sea level rise response factor),

$$SL = LT\left(\frac{S_2}{S_1}\right)^m \qquad (4\text{-}6)$$

An instantaneous response (m = 1) is where the rate of future recession is proportional to the increase in SLR. An instant response is typical of unconsolidated or weakly consolidated shorelines. No feedback (m = 0) indicates that wave influence is negligible and weathering dominates. The most likely response of consolidated shorelines is a negative/damped feedback system (m = 0.5), where rates of recession are slowed by development of a shore platform or fronting beach.

For the banks within the harbours a SLR response factor (m) ranging from 0.3 to 0.5 has been adopted. This is in line with what was used by T+T (2019) for the embankments within Tauranga Harbour which are likely to have similar erosion susceptibility as the harbour banks within Lyttelton and Akaroa Harbours

4.5.5 Summary of components

A summary of the adopted component values for detailed sites within Lyttleton Harbour and Akaroa Harbour is provided in Table 4.26 and Table 4.27.

4.5.6 Uncertainties

While some of the harbour sites have been completed at a detailed, quasi-probabilistic level, there is limited data availability and subsequently some key uncertainties exist:

- Long term erosion rates in absence of the protection structures, particularly within Corsair Bay, Charteris Bay, Duvauchelle and Wainui.
- Condition and design life of structures along the 'significantly modified shoreline' at Lyttelton Port and Akaroa township.
- Short term storm response. The Kriebel and Dean (1993) method does not account for different sediment types across the profile or response to short-duration events.
- Underlying geology and slope stability along the banks and hard cliffs.
- Shoreline response to SLR. Estuaries/harbours are areas of deposition and infilling so if sedimentation rates are high, the shoreline may adjust and keep pace, depending on the rate of SLR.

Table 4.26: Summary of adopted component values for the detailed sites within Lyttelton Harbour

Site		Lyttelton Port	Corsair Bay	Cass Bay	Cass Bay	Rapaki Bay	Rapaki Bay	Charteris Bay	Charteris Bay	Hays Bay	Purau
Cell		**31**	**32**	**33**	**34**	**35**	**36**	**42**	**43**	**44**	**45**
Chainage, km from Waimakariri River mouth		53.2 to 58.1 km	58.1 to 58.2 km	59 to 59.2 km	59.2 to 59.3 km	60.7 to 60.3 km	60.3 to 60.6 km	89.3 to 90.6 km	90.6 to 91.5 km	92.8 to 92.95 km	100.7 to 101.3 km
Morphology		Class 1 structure	Bank	Bank	Bank	Bank	Bank	Harbour beach	Bank	Harbour beach	Harbour beach
Geology		Anthropic deposits	Andesite	Andesite	Andesite	Loess	Loess	Estuarine deposit	Sandstone	Sand	Alluvial fan
Short-term (m)	Lower	N/A	N/A	N/A	N/A	N/A	N/A	-2	N/A	-2	-2
	Mode							-4		-4	-4
	Upper							-6		-6	-6
Dune (m above toe)	Lower	4.5	2	6	2	1.2	4	0.6	1.5	0.4	0.5
	Mode	5	2.5	9	3	1.5	5	0.8	2	0.5	1
	Upper	6	3.5	10	4	1.6	6	1	3	0.8	1.5
Stable angle (deg)	Lower	18	18	18	18	18	18	30	18	30	30
	Mode	22	22	22	22	22	22	32	22	32	32
	Upper	27	27	27	27	27	27	34	27	34	34
Long-term (m) -ve erosion +ve accretion	Lower	N/A	-0.07	-0.07	-0.07	-0.07	-0.07	-0.17	-0.07	-0.07	-0.07
	Mode		-0.02	-0.02	-0.02	-0.02	-0.02	-0.08	-0.02	-0.02	-0.02
	Upper		0	0	0	0	0	-0.05	0	0	0
Closure slope[1]/SLR factor[2]	Lower	N/A	0.3[2]	0.3[2]	0.3[2]	0.3[2]	0.3[2]	0.05[1]	0.3[2]	0.07[1]	0.06[1]
	Mode		0.4[2]	0.4[2]	0.4[2]	0.4[2]	0.4[2]	0.06[1]	0.4[2]	0.08[1]	0.08[1]
	Upper		0.5[2]	0.5[2]	0.5[2]	0.5[2]	0.5[2]	0.08[1]	0.5[2]	0.09[1]	0.09[1]

[1] Closure slope applicable for the harbour beach morphology
[2]SLR factor applicable for the bank morphology

Table 4.27: Summary of adopted component values for the detailed sites within Akaroa Harbour

Site		Akaroa township	Akaroa township	Akaroa township	Akaroa township	Akaroa north	Childrens Bay	Takamatua Bay	Takamatua Bay	Duvauchelle	Duvauchelle	Duvauchelle	Wainui
Cell		69	70	71	72	73	74	75	76	79	80	81	91
Chainage, km from Waimakariri River mouth		310.1 to 311.5 km	311.5 to 312.1 km	312.1 to 312.8 km	312.8 to 313.6km	313.6 to 314 km	314 to 314.1km	319.3 to 320 km	320 to 320.9 km	328 to 328.5 km	328.5 to 329 km	329 to 330.1 km	344 to 344.9 km
Morphology		Class 1 structure	Class 1 structure	Class 1 structure	Class 1 structure	Bank	Bank	Bank	Beach	Beach	Bank	Beach	Beach
Geology		Alluvial fan	Alluvial fan	Alluvial fan	Alluvial fan	Loess	Loess	Loess	Beach deposit	Alluvial fan	Loess	Alluvial fan	Gravel
Short-term (m)	Lower	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	-2	N/A	-2	-2
	Mode									-4		-4	-4
	Upper									-6		-6	-6
Dune (m above toe)	Lower	2	2	1.5	1.5	5	0.4	2.5	1	0.7	2	0.3	2.7
	Mode	3	2.5	1.8	1.8	6	0.5	4	1.5	0.8	2.2	0.5	3.5
	Upper	5	3	2	2	8	1	6	2.5	1	2.5	1.5	4
Stable angle (deg)	Lower	18	18	18	18	18	30	18	18	30	18	30	30
	Mode	22	22	22	22	22	32	22	22	32	22	32	32
	Upper	26.6	26.6	26.6	26.6	26.6	34	26.6	26.6	34	27	34	34
Long-term (m) -ve erosion +ve accretion	Lower	N/A	N/A	N/A	N/A	-0.07	-0.07	-0.07	-0.07	-0.07	-0.07	-0.07	-0.07
	Mode					-0.02	-0.02	-0.02	-0.02	-0.02	-0.02	-0.02	-0.02
	Upper					-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01
Closure slope[1]/ SLR factor[2]	Lower	N/A	N/A	N/A	N/A	0.3[2]	0.06[1]	0.3[2]	0.3[2]	0.06[1]	0.3[2]	0.06[1]	0.08[1]
	Mode					0.4[2]	0.07[1]	0.4[2]	0.4[2]	0.07[1]	0.4[2]	0.07[1]	0.09[1]
	Upper					0.5[2]	0.08[1]	0.5[2]	0.5[2]	0.08[1]	0.5[2]	0.08[1]	0.10[1]

[1] Closure slope applicable for the Harbour beach morphology and SLR factor applicable for the bank morphology.

[2]SLR factor applicable for the bank morphology

4.6 Banks Peninsula (regional sites)

Banks Peninsula is characterised by a radial pattern of drowned valleys and near-vertical plunging cliffs that terminate long sloping interfluves separating small bay-head beaches (Figure 4.40). The numerous bays of Banks Peninsula have formed as a result of flooding of the valleys by rising seas at the termination of the Pleistocene. Most of these embayments have filled with sediment composed of fine silts and clays, which were originally of aeolian or marine provenance. The beaches around Banks Peninsula vary from small pocket beaches with a mixture of sand gravel to more exposed fine sand beaches (Figure 4.41). Along many of the beaches the landward boundary is characterised by steep cliffs and banks (Dingwall, 1974). Figure 4.42 provides examples of the bank shorelines along undeveloped parts of Lyttelton and Akaroa harbours.

The Banks Peninsula sites have been completed at a regional hazard screening level with upper bound values adopted for each component.

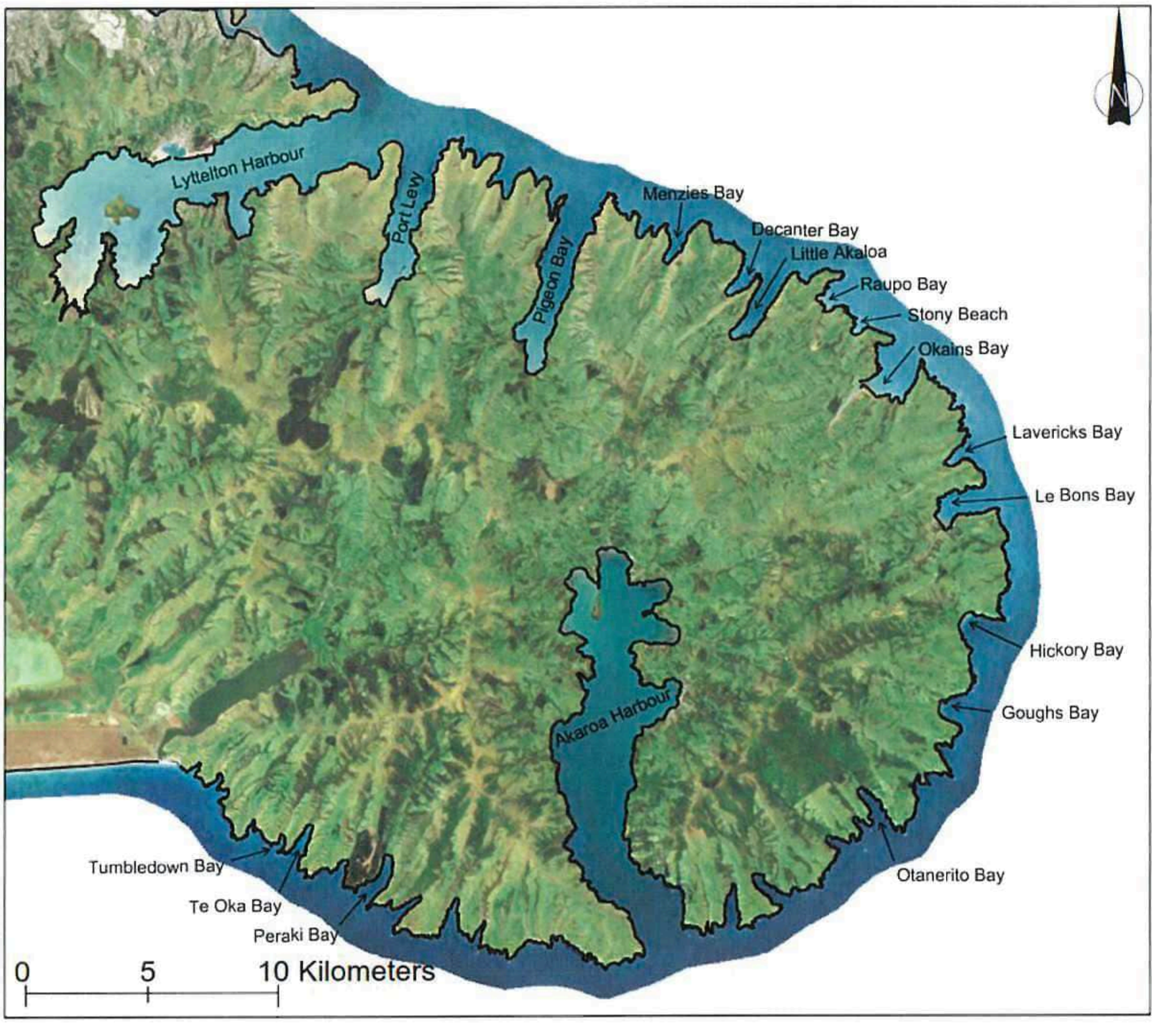

Figure 4.40: Overview of the bays and harbours around Banks Peninsula.

Figure 4.41: Examples of beach shorelines around Banks Peninsula. (Top left) Te Oka Bay, (top right) Okains Bay, (bottom left) Tumbledown Bay, (bottom right) French Farm Bay within Akaroa Harbour.

Figure 4.42: Examples of bank shorelines within the harbours around Banks Peninsula. (Left) Barry's Bay, Akaroa Harbour, (right) Ohinetahi, Lyttelton Harbour.

4.6.1 Short term component (ST)

There is limited data available to assess the short term storm cut along the Banks Peninsula beaches. Dingwall (1974) describes the bay-head beaches as generally stable or prograding, with the largest progradation in the north-eastern bays, such as Okains Bay. Beach surveying completed by Dingwall (1974) indicates storm cut up to 20 m at Le Bons and Hickory Bay.

For the regional screening assessment, generic storm cut distances have been adopted based on the beach exposure (i.e. a different distance will be adopted for sheltered and exposed beaches). The beaches have been broadly classified into 3 levels of exposure. Based on the level of exposure different short term values have been adopted as shown in Table 4.28. The storm cut on sheltered beaches is equivalent to the upper bound value for the detailed harbour beach sites. Sheltered beaches are those such as within the harbours and Port Levy. The storm cut for the exposed beaches is based on the findings from Dingwall (1974) and is approximately equivalent to the 100-year ARI storm cut on the Christchurch open coast. Exposed beaches are those such as Hickory and Le Bons Bay. Moderate exposure beaches include those within small bays such as Tumbledown Bay. It is assumed that storm cut along the moderately exposed beaches is between the sheltered and exposed beach storm cut distances. Adopted short term values for each beach are provided in Table 4.29.

Table 4.28: Adopted short term values for different beach exposures around Banks Peninsula

Exposure	Adopted short term component (m)
Sheltered	-6
Moderate	-12
Exposed	-20

4.6.2 Dune and bank stability (DS)

Dune and bank stability has been assessed as described in Section 4.1.3. Dune and bank heights have been measured from the 2018 DEM. The upper bound heights measured within each cell have been adopted with an upper bound stable angle of repose. Adopted heights and stable slopes for each cell are shown in Table 4.29.

4.6.3 Long term component (LT)

The long term component has been assessed using end-point regression analysis from two shorelines (earliest and most recent shorelines available from aerial photographs). The maximum long term rate erosion rate identified within each coastal cell has been adopted.

The historic aerials show majority of the beaches are stable or accreting. Figure 4.43 shows an example of the long term accretion measured at Okains Bay. Dingwall (1974) also found the Banks Peninsula beaches to be either stable or accreting with the highest rate of sediment accumulation occurring within Okains Bay and a slightly lower rate at Le Bons bay. Multiple dune ridges within these bays also indicate periods of rapid coastal progradation.

The sediment is predominately derived from erosion and river supply along the South and mid Canterbury coast with northwards net sediment transport. The accretional trends are also consistent with the apparent accretion trends along south Pegasus Bay and Kaitorete Spit.

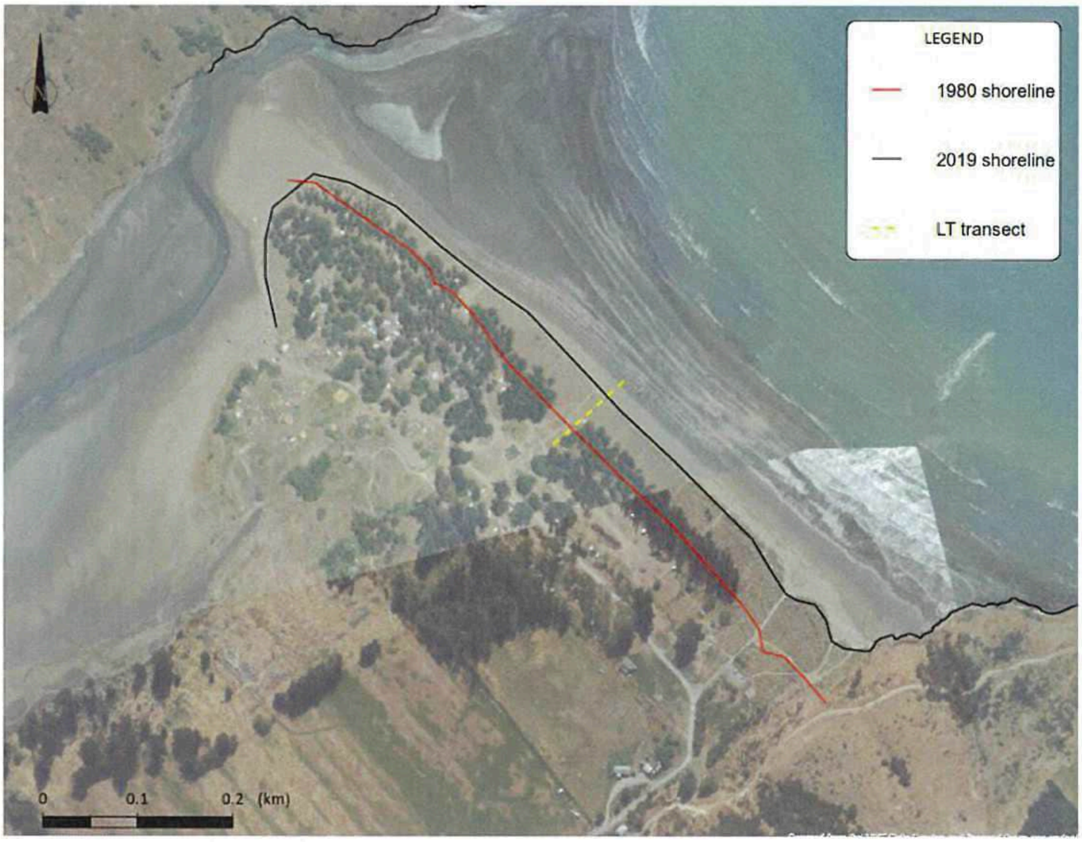

Figure 4.43: Example of shorelines used to assess long term trends in Okains Bay. Approximately 23 m of accretion measured between 1980 and 2019.

4.6.4 Response to SLR (SLR)

4.6.4.1 Beach response

For the sheltered beaches fronted by tidal flats, the SLR response has been assessed based on the modified Bruun Rule (see Section 4.4.5), using the upper beach slope. For the more exposed sandy beaches around Banks Peninsula, the standard Bruun model is more applicable with exchange occurring between the closure depth (see Section 0). However, due to limited wave statistics around the Peninsula it is difficult to define the closure depths for each beach. Subsequently, an assumed the closure slopes have been assumed the same as Taylors Mistake (0.02) which is a pocket beach with similar exposure as the Banks Peninsula beaches. Adopted slopes are included in Table 4.29.

4.6.4.2 Bank response

The bank response to SLR has been assessed based on the same response model outlined in Section 4.5.4.2 . An upper bound SLR response factor (*m*) of 0.5 has been adopted. This is in line with what was used by T+T (2019) for the embankments within Tauranga Harbour which are likely to have similar erosion susceptibility as the harbour banks around Lyttelton and Akaroa Harbours.

4.6.5 Cliff instability

Cliffs around Banks Peninsula are predominately basalt and andesite with greywacke-derived loess which forms an extensive mantle over the peninsula. The exposed cliffs around the edge of the Peninsula are near vertical with shore platforms in places (Figure 4.44). The cliffs within the bays tend to sit at lower angles and extend over 300 m high in places.

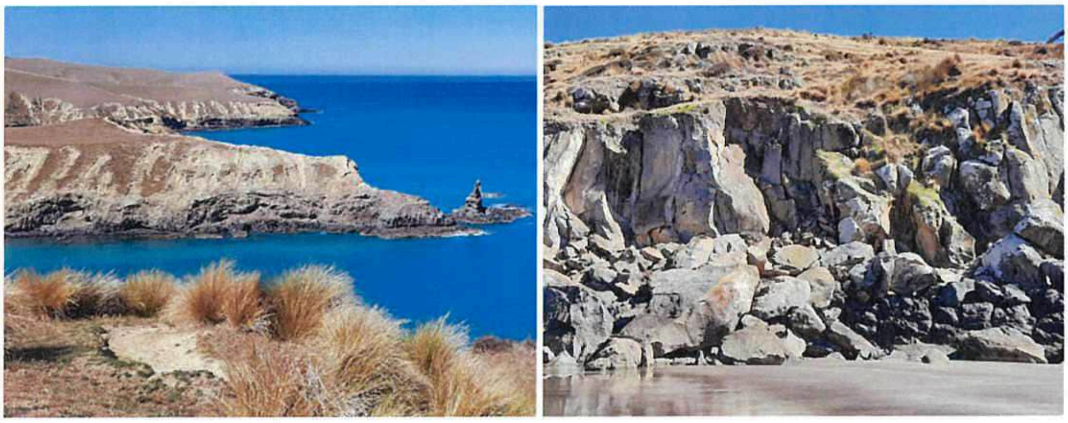

Figure 4.44: Example of cliff shorelines around Banks Peninsula.

The majority of the coastal cliffs around Banks Peninsula have existing slopes steeper than 1(H):1(V). The upper slopes which extend to over 500 m elevation tend to sit at a much lower angle, however the processes on these slopes are not substantially driven by coastal dynamics but instead are subject to more general slope instability hazard. This hazard is already identified and managed via the "Remainder of Port Hills and Banks Peninsula Slope Instability Management Area" in the Christchurch District Plan.

For coastlines where a "Cliff Collapse Management Area" or "Mass Movement Management Area" (either Class 1 or 2) is mapped in the Christchurch District Plan (e.g. Whitewash Head), this area has been used to define the width of the cliff instability component. The rationale for adopting this existing information rather than applying a separate regional screening analysis is that these management areas incorporate extensive site-specific geotechnical investigation, analysis of a range of trigger mechanisms and peer review which far exceeds the detail which is possible at a regional scale.

Where cliff collapse or mass movement management areas are not defined (i.e. remainder of Banks Peninsula), a simplified 3-step method based on the 2018 DEM has been used to define the width of the cliff instability component:

1. Where the slope is identified as being equal to or steeper than 1(H):1(V), the cliff slope has been identified as potentially unstable due to coastal processes (refer Figure 4.45).
2. A 20 m wide setback has been applied beyond the top of the steep cliff slope. This setback accounts for the physical scale of potential cliff failure mechanisms for typical cliff heights around Banks Peninsula. It also reflects the precision limitations involved in defining the top of the cliff at this regional scale. This 20 m setback value is at the upper end of the range of cliff retreat distances observed in the Canterbury Earthquakes, and of a similar scale to the width of the cliff collapse management areas defined in the district plan.
3. Where the coastal cliff edge is flatter than 1(H):1(V), a 30 m wide setback has been applied from the coastal edge. The 30 m setback is based on the average setback distance calculated for harbour beaches and banks for the 2130 1.5 m SLR scenario.

Historic aerial photographs indicate the long-term toe erosion of the Banks Peninsula cliffs is minimal and due to the scale and nature of the cliff assessment, it is not suitable to differentiate between current and future ASCE with different SLR scenarios therefore a single future ASCE is defined.

While this method is not a detailed cliff projection method, it is suitable for a regional coastal hazard screening assessment. It is emphasised that cliff collapse hazard is highly dependent on site-specific details and a can include a range of potential triggers in addition to coastal processes. These details cannot be incorporated into this regional-scale assessment. If more detailed information is required about the cliff instability hazard for a specific location (e.g. as part of proposed development or hazard management activities in future) then a site-specific assessment should be undertaken, which may indicate that the hazard area is narrower or wider than mapped in this regional assessment.

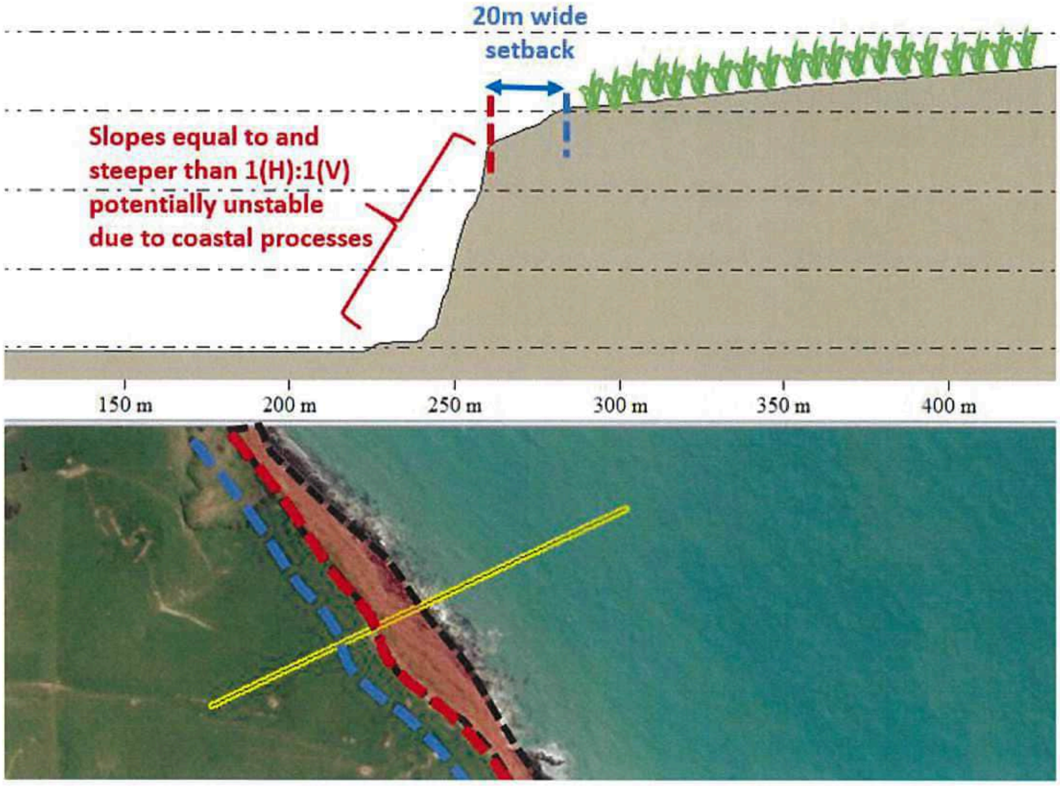

Figure 4.45: Example of cliff ASCE.

4.6.6 Summary of components

Adopted component values for the beaches and banks within regional hazard screening sites around Banks Peninsula are presented in Table 4.29.

4.6.7 Uncertainties

The regional hazard screening assessment includes a few uncertainties as outlined below:

- Long term erosion rates in absence of the protection structures.
- Short term storm response.
- Underlying geology and slope stability along the banks and hard cliffs.
- Shoreline response to SLR. Estuaries/harbours are areas of deposition and infilling so if sedimentation rates are high, the shoreline may adjust and keep pace, depending on the rate of SLR.
- Shoreline response to SLR on the outer Banks Peninsula beaches where there is limited data on offshore profiles and wave climate.

Table 4.29: Summary of adopted component values for the regional screening assessment beach and bank sites

Cell	Site	Chainage	Morph	Geology	Short-term (m)	Dune/bank height (m)	Stable Angle (deg)	Long-term (m/yr)	SLR component	
									Closure Slope	Factor
37	Sandy Beach Rd	64.7 to 67.1 km	Bank	Andesite	N/A	2	18	-0.07	N/A	0.5
38	Allandale	67.1 to 67.5 km	Beach	Alluvial fan	-6	1	30	-0.07	0.1	N/A
39	Teddington (low-lying)	73 to 81.3 km	Beach	Estuarine deposit	-6	0.8	30	-0.05	0.1	N/A
40	Moepuku (low-lying)	81.9 to 82.3 km	Bank	Loess	N/A	1.5	18	-0.05	N/A	0.5
41	Charteris Bay (west)	88.2 to 88.8 km	Bank	Sandstone	N/A	1.5	18	-0.07	N/A	0.5
46	Port Levy	122.1 to 123 km	Bank	Young alluvial fan	N/A	0.8	18	-0.07	N/A	0.5
47	Port Levy	123.5 to 123.7 km	Bank	Young alluvial fan	N/A	0.8	18	-0.07	N/A	0.5
48	Port Levy (Puari)	125 to 125.8 km	Beach	Young alluvial fan	-6	0.5	30	-0.05	0.1	N/A
49	Port Levy	126.1 to 126.8 km	Beach	Young alluvial fan	-6	1	30	-0.05	0.1	N/A
50	Holmes Bay	156.6 to 157.5 km	Beach	Young beach deposit	-6	1	30	-0.07	0.1	N/A
51	Pigeon Bay (south)	159.4 to 160.1 km	Beach	Young beach deposit	-6	2	30	-0.07	0.1	N/A
52	Pigeon Bay	160.4 to 161.2 km	Beach	Young alluvial fan	-6	2	30	-0.07	0.1	N/A
53	Menzies Bay	178 to 178.1 km	Beach	Young alluvial fan	-12	2	30	-0.07	0.1	N/A
54	Decanter Bay	186.5 to 186.7	Beach	Young alluvial fan	-12	1	30	-0.07	0.1	N/A
55	Little Akaloa	192.1 to 192.3 km	Beach	Young alluvial fan	-12	2.5	30	-0.07	0.1	N/A
56	Little Akaloa	192.3 to 192.5 km	Beach	Young alluvial fan	-12	3.5	30	-0.07	0.1	N/A
57	Raupo Bay	202 to 202.4 km	Beach	Young alluvial fan	-12	1	30	0	0.02	N/A
58	Raupo Bay	202.9 to 202.1 km	Beach	Young alluvial fan	-12	1	30	0	0.02	N/A
59	Stony beach	206.9 to 207.3 km	Beach	Young alluvial fan	-12	2.5	30	0	0.02	N/A
60	Okains Bay	213.3 to 214.3 km	Beach	Young beach deposit	-20	1	30	1	0.02	N/A
61	Lavericks	227.7 to 228.2 km	Beach	Young alluvial fan	-20	2	30	0	0.02	N/A
62	Le Bons Bay	235.6 to 236.1 km	Beach	Young beach deposit	-20	2	30	0.3	0.02	N/A
63	Le Bons Bay	236.1 to 236.5 km	Beach	Young beach deposit	-20	1	30	0.1	0.02	N/A

Table 4.29 (continued): Summary of adopted component values for the regional screening assessment beach and bank sites

Cell	Site	Chainage	Morph	Geology	Short-term (m)	Dune/bank height (m)	Stable Angle (deg)	Long-term (m/yr)	SLR component	
									Closure Slope	Factor
64	Hickory	247.8 to 248.5km	Beach	Young alluvial fan	-20	3	30	0	0.02	N/A
65	Goughs Bay	254 to 254.5 km	Beach	Young beach deposit	-20	2	30	0	0.02	N/A
66	Otanerito Bay	270.2 to 270.5 km	Beach	Young alluvial fan	-12	2	30	0	0.02	N/A
67	The Kaik	308.1 to 308.4 km	Bank	Loess	N/A	2	18	-0.07	N/A	0.5
68	Akaroa south	310.1 to 310.8 km	Bank	Loess	N/A	3	18	-0.07	N/A	0.5
77	Robinsons Bay south	324.3 to 324.6 km	Bank	Loess	N/A	4	18	-0.07	N/A	0.5
78	Robinsons Bay	324.6 to 325.6 km	Beach	Young alluvial fan	-6	1.5	30	-0.07	0.1	N/A
82	Barrys Bay (landfill)	332.1 to 332.4 km	Bank	Loess	N/A	4.5	18	-0.07	N/A	0.5
83	Barrys Bay	332.4 to 332.7 km	Bank	Loess	N/A	1.5	18	-0.07	N/A	0.5
84	Barrys Bay	332.7 to 333 km	Bank	Loess	N/A	2.5	18	-0.07	N/A	0.5
85	Barrys Bay	333 to 333.6 km	Beach	Young alluvial fan	-6	0.9	30	-0.05	0.1	N/A
86	Barrys Bay (south)	333.6 to 334.6 km	Bank	Loess	N/A	4	18	-0.07	N/A	0.5
87	French farm bay (boat houses)	335.5 to 335.8 km	Bank	Loess	N/A	2	18	-0.07	N/A	0.5
88	French farm bay	335.8 to 336.5 km	Beach	Young alluvial fan	-6	1.5	30	-0.05	0.1	N/A
89	Tikao Bay	340.9 to 341.1 km	Bank	Basalt	N/A	2	18	-0.07	N/A	0.5
90	Tikao Bay	341.4 to 341.8 km	Bank	Loess	N/A	3.5	18	-0.07	N/A	0.5
92	Wainui south	344.9 to 346.2 km	Bank	Loess	N/A	4	18	-0.07	N/A	0.5
93	Peraki Bay	390.2 to 390.6 km	Beach	Young alluvial fan	-12	2	30	0	0.02	N/A
94	Te Oka Bay	401.3 to 401.5 km	Beach	Young alluvial fan	-12	3	30	0	0.02	N/A
95	Tumbledown Bay	404.6 to 405 km	Beach	Young alluvial fan	-12	3	30	0	0.02	N/A

4.7 Kaitorete Spit

The Kaitorete Spit is located on the southern side of Banks Peninsula on the Canterbury Bight. The spit extends for approximately 26 km and is over 2 km wide at its widest extent. The sediment forming the spit is predominately gravel (Figure 4.46).

The barrier spit is understood to have existed for the last 8,000 years. The barrier initially developed as a spit extending north from near the Rakaia River mouth, during the sea level rise in the Late Pleistocene and Early Holocene. During the mid-Holocene the spit extended to Banks Peninsula creating a barrier lake complex behind the spit. Since the end of the Holocene transgression the whole coastline has been relatively stable (Soons et al., 1997).

Figure 4.46: Site photos (taken August 2020) near Birdlings Flat at the northern extent of Kaitorete Spit.

4.7.1 Cell splits

The Kaitorete Spit has been split into 5 cells (Figure 4.47). The cell split is largely based on the variation in long term trends along the shoreline.

Figure 4.47: Overview of cell splits and profile locations along Kaitorete Spit.

4.7.2 Short term component (ST)

The short term component along Kaitorete Spit has been assessed using the beach profile dataset. Figure 4.48 provides an example of the beach profiles measured at ECE3755 near Birdlings Flat. Based on visual inspection the berm toe along Kaitorete Spit is estimated to be around 6 m RL.

The short-term component has been quantified using statistical analysis of the inter-survey storm cut distances. The inter-survey storm cut distance is the landward horizontal retreat distance measured between two consecutive surveys (Figure 4.48).

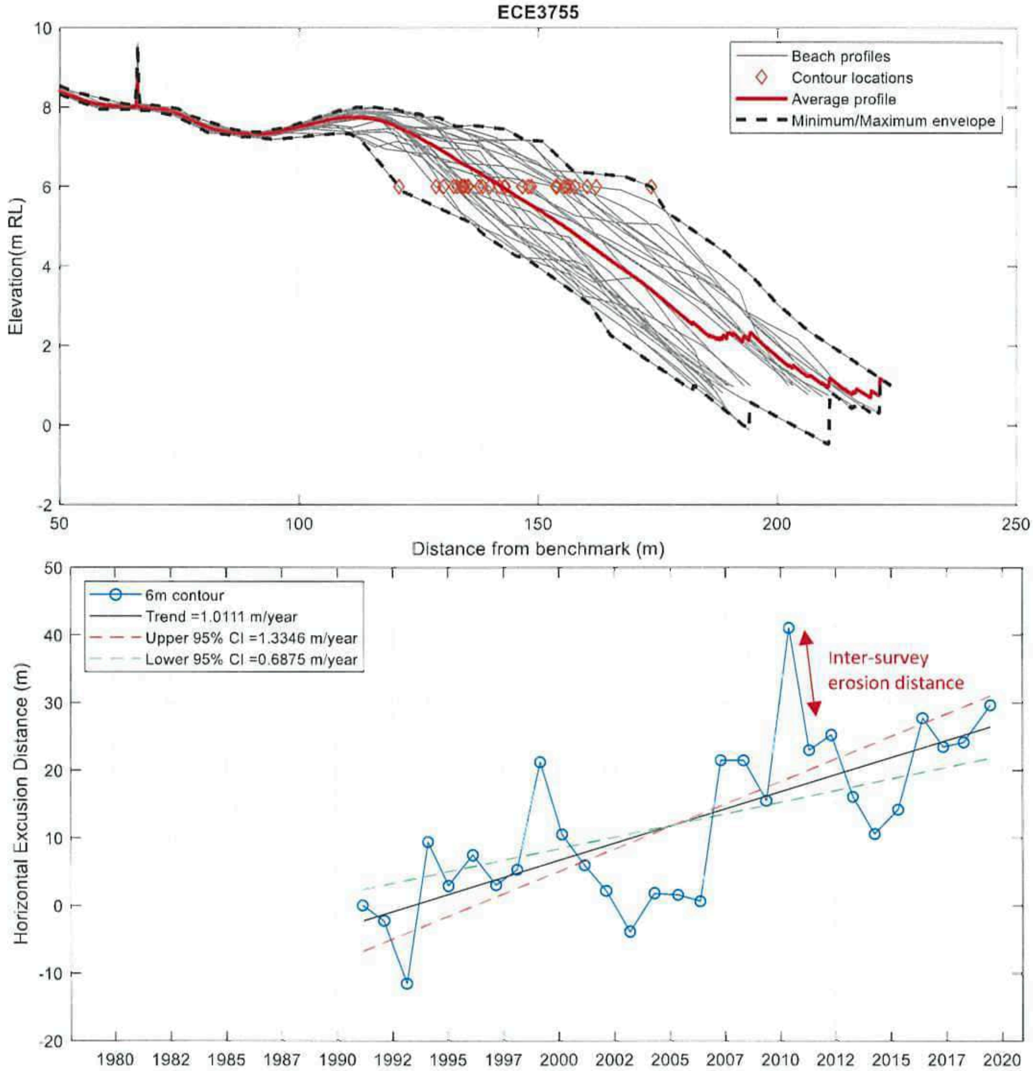

Figure 4.48: Example of beach profile ECE3755 used to assess short term erosion along the Kaitorete Spit.

Based on Extreme Value Analysis (EVA), using the inter-survey erosion distances for each beach profile, the 100 year ARI inter-survey storm cut distance ranges from 5 to 20 m. An example of the extreme value curve for profile ECE3800 is shown in Figure 4.49. For this assessment 20 m storm cut has been adopted for all cells along the spit.

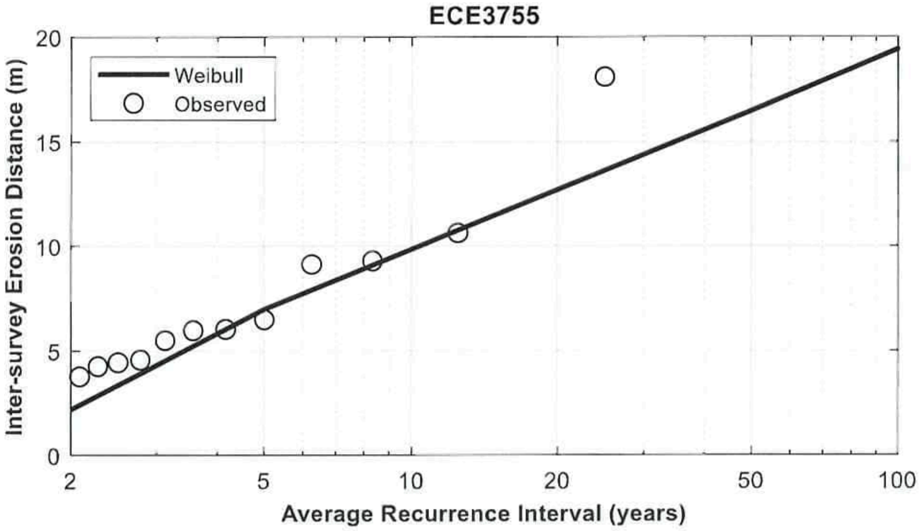

Figure 4.49: Example of extreme value curve for inter-survey storm cut distances at profile ECE3755 on Kaitorete Spit.

4.7.3 Long-term trends (LT)

Long-term shoreline changes have been assessed using a combination of beach profiles and historic shorelines. Linear regression analysis has been completed for both datasets and comparisons made to infer the long-term rates within each coastal cell.

The beach profiles include data from 1991 to 2019 and show a trend of erosion at the southern end of the spit and accretion at the northern end. Figure 4.50 shows erosion up to -0.6 m/year at the southern end and accretion over 1 m/year at the northern end. These profile trends are consistent with both the historic shorelines (Figure 4.51) and the findings from Measures et al (2014) and Cope (2018).

The shoreline retreat appears to greatest at ECE1620 (approximately 6 km north of Taumutu). North of ECE1620, the rates of retreat reduce until there is a complete switch to shoreline accretion around profile ECE2995. The accretion increases along the northern 10 km of the spit towards Birdlings Flat. Cope (2018) describes this transition between shoreline erosion and shoreline progradation as a hinge point where clockwise shoreline rotation is continuing to occur. The spit has formed through longshore drift northwards from the Rakaia River. The shoreline at Rakaia mouth has eroded and slowly changed shoreline angle which has reduced the sediment transport and resulted in the hinge point migrating north (Measures et al., 2014).

The maximum long term rate erosion rate identified within each coastal cell has been adopted and is shown in Table 4.30.

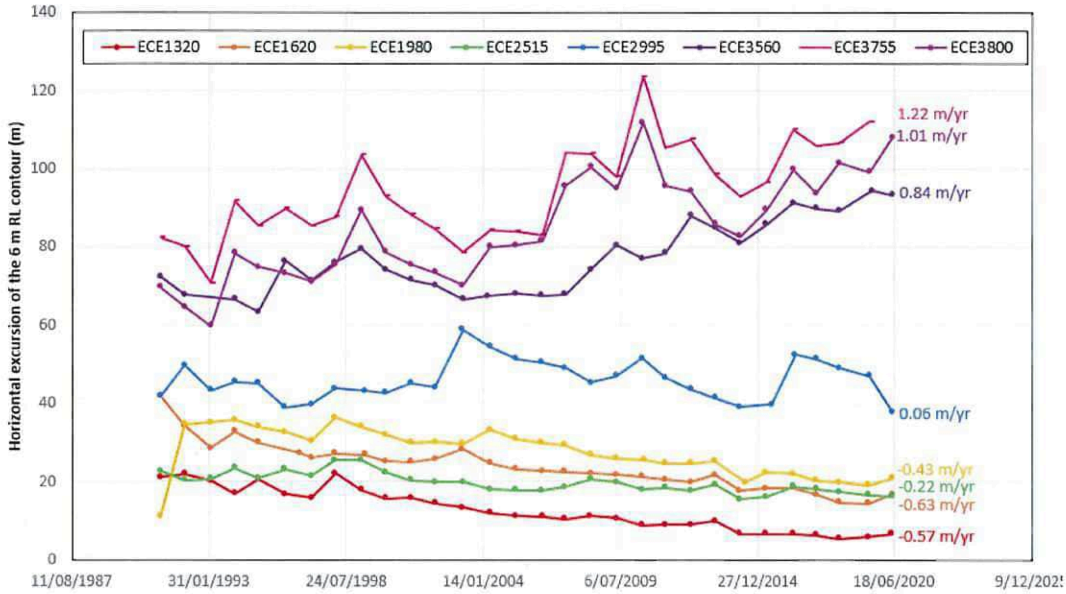

Figure 4.50: Horizontal excursion distances measured at the berm toe for profiles along Kaitorete Spit.

Figure 4.51: Historic shorelines along the northern end of Kaitorete Spit showing up to 80 m accretion since 1980.

4.7.4 Response to sea level rise (SLR)

Shoreline response to SLR along Kaitorete Spit has been assessed using a modification of the Bruun rule (Equation 4.7 and Figure 4.52). Instead of adopting closure depths based on offshore wave heights, the closure depth along mixed sand gravel beaches has been assumed to be equivalent to the beach step.

The beach step marks the lower extent of the active beach. Typically, on sand gravel beaches the gravel portion of the shoreface rarely extends below the low tide mark (Shulmeister and Jennings, 2009). As there is limited offshore survey data to determine the location of the beach step along Kaitorete Spit, the beach step has been assumed 5 m below MSL and approximately 60 m offshore from the MSL contour. This is consistent with the estimate made by Measures et al (2014) (i.e. -5.5 m LVD37 at Taumutu).

$$Retreat\ due\ to\ SLR = SLR\frac{L}{h} \qquad (4.7)$$

Where:

SLR = SLR (m).

L = Horizontal distance from beach step to the berm crest.

h = height of the berm above the beach step.

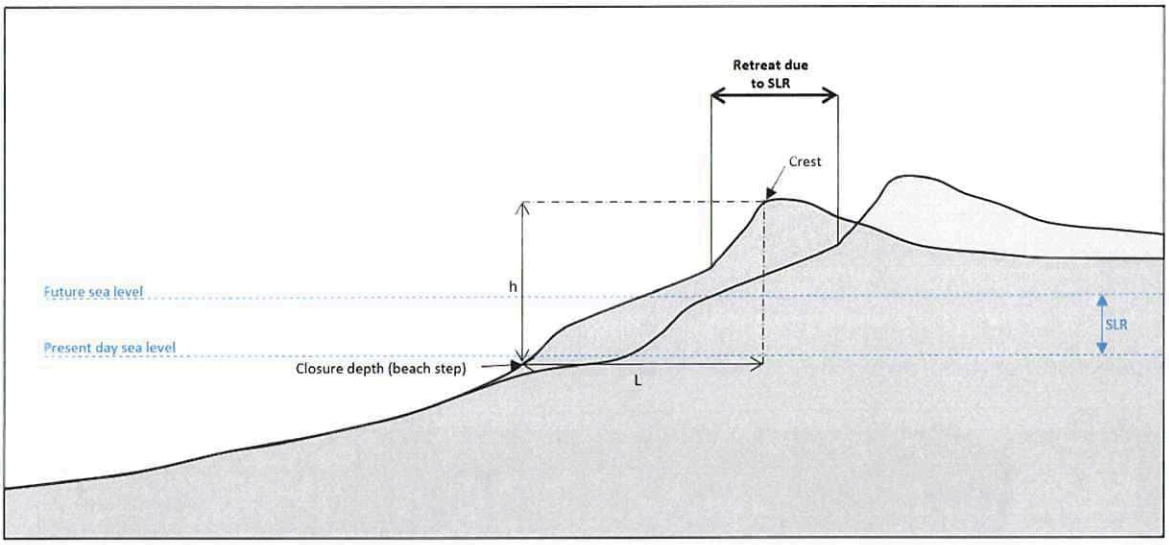

Figure 4.52: Conceptual diagram of SLR response along mixed sand gravel beach.

4.7.5 Summary of components

Adopted component values for the Kaitorete Spit are shown in Table 4.30.

4.7.6 Uncertainties

Key uncertainties in the erosion hazard assessment for Kaitorete Spit include:

- Future sediment supply from the Rakaia River.
- Offshore profile and subsequently the shoreline response to SLR.

Table 4.30: Summary of adopted components along the Kaitorete Spit

Cell	Name	Chainage (km)	Morphology	Geology	Short-term (m)	Berm height (m above toe, 6 m RL)	Stable Angle (deg)	Long-term (m/yr)	SLR response slope
96	Birdlings Flat	419 to 424.5	Gravel beacn	Gravel	-20	2.5	30	0.8	0.07
97	Kaitorete Spit	424.5 to 432	Gravel beach	Gravel	-20	2.5	30	0.06	0.07
98	Kaitorete Spit	432 to 438.5	Gravel beach	Gravel	-20	6	30	-0.4	0.07
99	Kaitorete Spit	438.5 to 442	Gravel beach	Gravel	-20	6	30	-0.6	0.07
100	Kaitorete Spit	442 to 446	Gravel beach	Gravel	-20	0.5	30	-0.6	0.07

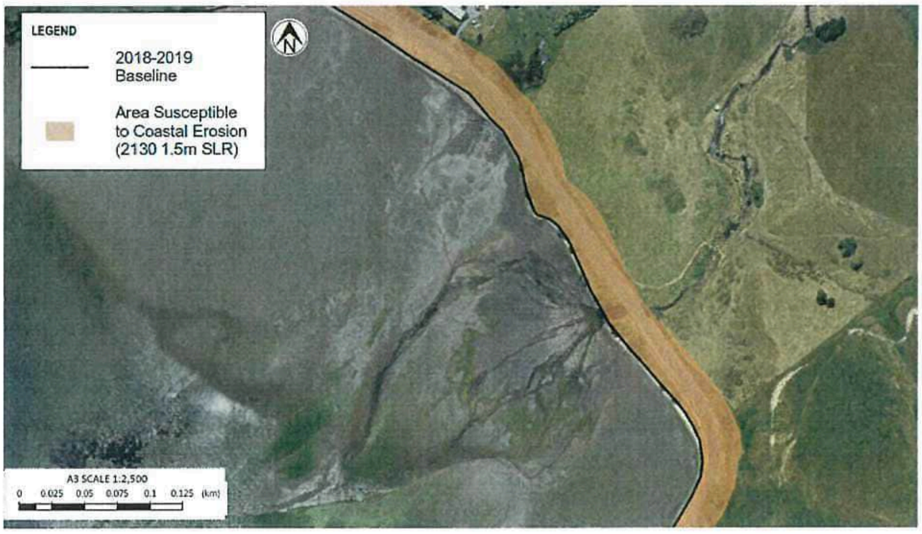

Figure 3.13: Example ASCE map for the regional hazard screening sites.

3.6.2 Detailed hazard assessment maps

For the detailed sites where ASCE have been assessed probabilistically, a raster-based mapping approach has been adopted. The rasters comprise of 1 m grid cells which include information on exceedance probability in every grid cell and show the full probabilistic range of the resulting ASCE across shore. An example of the raster map is shown in Figure 3.14.

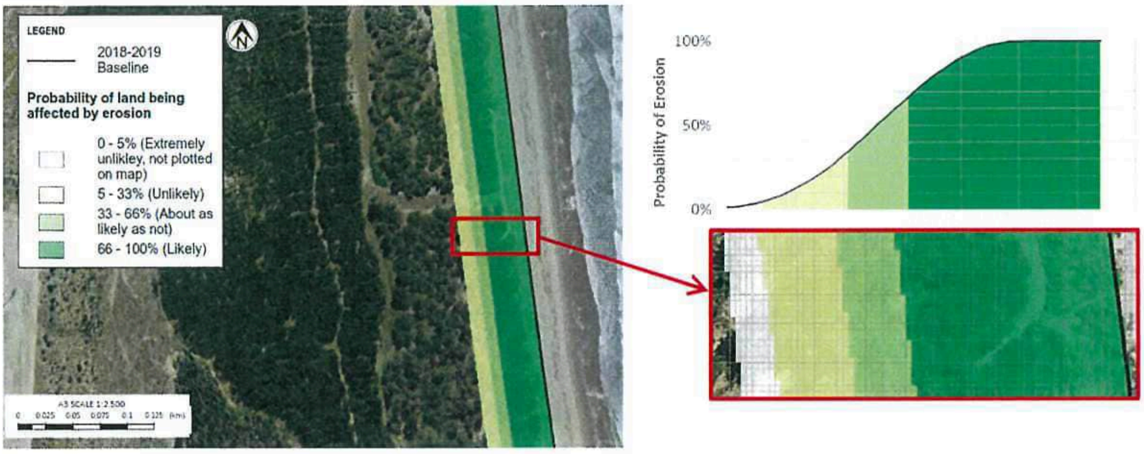

Figure 3.14: Example of raster ASCE mapped for a detailed site showing spatial extent of erosion and corresponding probabilities of occurrence.

5 Coastal erosion results

For each coastal cell, the relevant components influencing the ASCE have been combined as described in Section 3.1.1. The following section provides an overview of the results for each area. Erosion distances are summarised within the following tables, which for detailed sites, include the P66% and P5% ASCE and for the regional screening sites, include the single ASCE distance for each scenario. The P66% represents the distance at which there is 66% probability of the shoreline eroding beyond and can be considered a likely scenario. The P5% represents the distance at which there is 5% probability of the shoreline eroding beyond and can be considered as the extent to which it is possible but very unlikely for the shoreline to retreat to (P5% is taken as the middle of the IPCC (2013) "very unlikely" range of 0 - 10% probability). The P5% ASCE from the detailed scale assessments and the ASCE from the regional scale assessments are approximately equivalent, representing the 'upper end' erosion distances.

Coastal erosion maps, including the full probabilistic results for each of the detailed sites is available on the website viewer (refer Section 1.3). Overview maps which show the variation in erosion distances across the district are provided in Appendix E.

5.1 Christchurch open coast

The P66% and P5% ASCE distances along the Christchurch open coast are presented in Table 5.1. The current ASCE is dominated by the short-term storm erosion and tends to be largest towards the north where storm cut was found to be slightly larger. The short-term storm response also dominates the future ASCE under the shorter timeframes (i.e. 2050), accounting for over 50% of the total ASCE distance in most cells.

Seawalls are present within Cells 7 and 9, however these structures have not been accounted for within the assessment. The ASCE within these cells represents the hazard in absence of the structures, and therefore, while these structures remain functional, the ASCE is likely to be an overprediction.

The long-term trends are a key factor influencing the variation in the future ASCE along the Christchurch open coast. Most of the shoreline has historically shown long-term accretion trends due to the sediment supply from the Waimakariri River. The accretion rates tend to increase southwards and are largest within Cell 13, where the future ASCE are seaward of the current shoreline position. The high accretion rates at the southern end of the shoreline are likely a result of the net southward sediment transport and interactions with the ebb tidal delta. Over time the shoreline may slightly adjust orientation, with erosion towards the north and accretion towards the south, until an equilibrium is reached.

Due to the high accretion rates at the southern end of the shoreline (i.e. Cells 9 to 14), the future ASCE within these cells is most sensitive to changes in sediment supply from the Waimakariri River. A 28% increase in sediment supply from the Waimakariri River could reduce the 2130 1.5 m SLR ASCE by up to 27 m within Cell 13, and a 11% decrease in sediment supply could increase the future ASCE distance by 14 m. As long-term trends are smaller at the northern end of the beach (i.e. Cells 2 to 8), the future ASCE is less sensitive to changes in sediment supply. For example, within Cell 6, a 28% increase or an 11% decrease in sediment supply would result in the future 2130 1.5 m SLR ASCE shifting either seaward 5 m or landward 2 m.

The future ASCE are also influenced by the amount of SLR. Under low SLR scenarios the impact from long term accretion is likely to counteract any potential recession due to SLR, however higher SLR (e.g. more than about 0.4 – 0.6 m by 2130) is expected to overtake the impact of accretion and result in shoreline retreat. The tipping point at which SLR overtakes any impact of long-term accretion is dependent on the SLR scenario. For example, by 2080 under low SLR (i.e. 0.4 m), the

long-term accretion within Cell 13 dominates, whereas under a high SLR (i.e. 1.5 m) scenario there is a tipping point where erosion due to SLR dominates.

There is high uncertainty around the future erosion rates at the distal end of the Southshore Spit. As mentioned in Section 4.1.4, an increased tidal prism within the estuary is likely to enlarge in the tidal inlet and potentially increase erosion on the spit. However, quantification of this would require detailed investigation and modelling which is beyond the scope of this assessment. The previous assessment (T+T, 2017) adopted the most landward shoreline extent over the last 80 years, which provides more conservatism but still does not account for uncertainty of future processes.

Table 5.1: ASCE widths (m) for the P66% ('likely') and P5% ('very unlikely') ASCE along Christchurch Open Coast

Cell	Probability													28% increase in sediment supply	11% decrease in sediment supply
		Current	2050		2080			2130						2130	2130
		0 m SLR	0.2 m SLR	0.4 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	1 m SLR	1.2 m SLR	1.5 m SLR	1.5m SLR	1.5m SLR
1	P66%	-10	-14	-20	-15	-21	-27	-7	-13	-20	-26	-32	-41	-17	-54
	P5%	-24	-36	-43	-53	-61	-68	-71	-78	-85	-93	-101	-112	-89	-126
2	P66%	-10	-9	-15	-8	-14	-19	5	-1	-7	-13	-18	-27	-16	-32
	P5%	-23	-23	-30	-23	-31	-40	-11	-19	-28	-37	-46	-60	-51	-66
3	P66%	-9	-8	-14	-7	-13	-19	6	-1	-7	-13	-19	-29	-18	-34
	P5%	-22	-21	-28	-21	-27	-34	-9	-15	-22	-29	-36	-46	-37	-52
4	P66%	-9	-10	-17	-12	-18	-24	-4	-10	-16	-22	-28	-37	-30	-41
	P5%	-22	-23	-30	-26	-33	-41	-18	-25	-33	-41	-49	-61	-54	-64
5	P66%	-8	-12	-18	-16	-22	-28	-13	-19	-25	-31	-37	-46	-42	-48
	P5%	-15	-20	-27	-25	-33	-41	-23	-31	-39	-47	-55	-67	-64	-70
6	P66%	-8	-14	-22	-19	-28	-36	-14	-23	-31	-39	-48	-60	-55	-63
	P5%	-15	-22	-31	-30	-40	-50	-29	-39	-49	-59	-69	-85	-82	-87
7	P66%	-4	-10	-19	-16	-24	-32	-11	-19	-28	-36	-44	-57	-52	-59
	P5%	-11	-18	-28	-27	-36	-47	-25	-35	-45	-55	-66	-81	-79	-84
8	P66%	-7	-11	-17	-15	-20	-26	-10	-16	-22	-27	-33	-41	-37	-44
	P5%	-14	-20	-29	-27	-37	-47	-26	-35	-45	-54	-64	-80	-76	-81
9	P66%	-5	-5	-11	-4	-10	-17	9	2	-4	-11	-17	-27	-17	-32
	P5%	-12	-14	-26	-19	-31	-43	-10	-21	-33	-46	-58	-77	-68	-82
10	P66%	-7	-8	-14	-8	-13	-19	2	-3	-9	-14	-20	-28	-20	-33
	P5%	-14	-18	-29	-24	-36	-48	-15	-27	-39	-51	-63	-82	-74	-86
11	P66%	-6	-6	-14	-4	-12	-19	11	3	-5	-12	-19	-30	-18	-37
	P5%	-13	-15	-28	-20	-33	-47	-7	-20	-33	-47	-60	-81	-70	-87

Table 5.1 (continued): ASCE widths (m) for the P66% ('likely') and P5% ('very unlikely') ASCE along Christchurch Open Coast

Cell	Probability													28% increase in sediment supply	11% decrease in sediment supply
		Current	2050		2080			2130						2130	2130
		0 m SLR	0.2 m SLR	0.4 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	1 m SLR	1.2 m SLR	1.5 m SLR	1.5m SLR	1.5m SLR
12	P66%	-6	-2	-9	4	-3	-11	26	18	11	3	-4	-15	3	-24
	P5%	-13	-11	-24	-11	-24	-38	+11	-3	-16	-30	-44	-65	-47	-75
13	P66%	-5	6	-1	18	11	4	51	43	36	29	22	11	38	-3
	P5%	-12	-3	-14	+5	-7	-19	+37	+25	+13	0	-12	-31	-4	-45
14	P66%	-8	-7	-15	-6	-14	-22	11	3	-5	-13	-21	-33	-19	-41
	P5%	-13	-19	-30	-28	-39	-50	-25	-36	-46	-57	-69	-86	-78	-91

*Negative values are erosion and positive values are accretion.

5.2 Sumner

The ASCE distances for the P66% and P5% along Sumner are presented in Table 5.2. Cells 27 and 29 are classified as Class 1 structures (see Section 3.1.5). The ASCE within these two cells represents the immediate hazard is the structures were to fail and is a function of the structure height and stable angle of repose for the filled material. The future ASCE has been set equivalent to the current ASCE, which would be the case if the structures were promptly repaired if damaged. However, if the protection structures fail and are not promptly repaired then it is likely the shoreline will rapidly erode.

The current ASCE within Cell 28 represents the potential short-term storm cut and dune instability and is relatively small due to the significant volume of sand on the beach providing protection to the dunes along Clifton Beach. The future P5% ASCE ranges from -8 m by 2050 under low SLR, to -73 m by 2130 under high SLR. While there has been long-term accretion within Cell 28, the impacts of SLR are likely to overtake any long-term accretion.

There is high uncertainty around the future accretion rates within Cell 28. As mentioned in Section 4.2.4, an increased tidal prism within the estuary is likely to result in an enlarged tidal inlet and potentially increased erosion along the Clifton Beach shoreline.

Table 5.2: ASCE widths (m) for the P66% ('likely') and P5% ('very unlikely') ASCE along Sumner

Cell	Probability of exceedance	Current	2050		2080			2130					
		0 m SLR	0.2 m SLR	0.4 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	1 m SLR	1.2 m SLR	1.5 m SLR
27	P66%	-6	-6	-6	-6	-6	-6	-6	-6	-6	-6	-6	-6
	P5%	-8	-8	-8	-8	-8	-8	-8	-8	-8	-8	-8	-8
28	P66%	-1	-2	-9	-2	-9	-16	10	3	-5	-12	-19	-30
	P5%	-3	-8	-21	-15	-27	-40	-7	-18	-30	-42	-54	-73
29	P66%	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4
	P5%	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7

5.3 Taylors Mistake

The ASCE distances for the P66% and P5% along Taylors Mistake are presented in Table 5.3. The current ASCE, which represents the potential short-term storm cut and dune instability, ranges from -7 to -13 m for the P66% to P5%.

By 2050 under low SLR, the future ASCE ranges from -13 to -22 m for the P66% to P5% and by 2130 under high SLR, the future ASCE ranges from -47 m to -96 m.

Over the shorter timeframes (i.e. by 2050), the erosion distance is dominated by the potential short-term storm response, which in 2050, contributes approximately 40 to 50% of the total erosion distance. Over longer timeframes (i.e. by 2130), the short-term erosion component contributes only 15 to 35% of the total erosion distance. Over time the impact of long-term trends and SLR increases and subsequently has a greater influence on the total erosion distance.

Table 5.3: ASCE widths (m) for the P66% ('likely') and P5% ('very unlikely') ASCE along Taylors Mistake

Cell	Probability of exceedance	Current	2050		2080			2130					
		0 m SLR	0.2 m SLR	0.4 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	1 m SLR	1.2 m SLR	1.5 m SLR
30	P66%	-7	-13	-19	-19	-24	-30	-19	-24	-30	-35	-40	-47
	P5%	-13	-22	-32	-35	-45	-55	-40	-50	-60	-70	-80	-96

5.4 Avon-Heathcote Estuary

The ASCE distances for the P66% and P5% around the Avon-Heathcote estuary are presented in Table 5.4. The current ASCE represents the potential short-term storm cut and shoreline instability, which is relatively consistent across the estuary, ranging from -5 to -6 m for most cells.

Erosion protection structures, in varying condition, exist around the estuary and in these locations the current ASCE represents the immediate hazard if the structures were to fail. For Cell 15 to 24, the future ASCE has been assessed to represent the erosion hazard in absence of the structures.

Over shorter timeframes (i.e. by 2050) the ASCE distance is dominated by the potential short-term storm erosion which contributes 30 to 70% of the total erosion distance. Over longer timeframes (i.e. by 2080 and 2130), the long-term trends and response to SLR dominate the total erosion distance. Long-term erosion is largest within Cells 17 to 22, resulting in larger future ASCE, ranging from -35 to -48 m by 2130 under a high SLR scenario.

Cells 25 and 26 are classified as Class 1 structures (see Section 3.1.5). The ASCE within these two cells represents the immediate hazard if the structures were to fail and is a function of the structure height and stable angle of repose for the filled material. The ASCE is largest in Cell 26 where the structures are higher. The future ASCE have been set equivalent to the current ASCE, which would be the case if the structure was promptly repaired if damaged. However, if the protection structure fails and is not promptly repaired then it is likely the fill material will rapidly erode, and the shoreline will eventually move back towards its 'original' natural position (this scenario has not been modelled in this study).

Table 5.4: ASCE widths (m) for the P66% and P5% ASCE around the Avon-Heathcote Estuary

Cell	Probability of exceedance	Current	2050		2080			2130					
		0 m SLR	0.2 m SLR	0.4 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	1 m SLR	1.2 m SLR	1.5 m SLR
15	P66%	-4	-1	-5	2	-2	-5	12	8	5	2	-1	-5
	P5%	-6	-6	-10	-8	-11	-14	-5	-8	-11	-15	-18	-22
16	P66%	-4	-5	-8	-6	-9	-12	-4	-7	-10	-12	-13	-13
	P5%	-5	-7	-11	-10	-13	-17	-9	-12	-15	-19	-21	-22
17	P66%	-4	-11	-14	-18	-21	-24	-26	-29	-31	-34	-37	-39
	P5%	-6	-13	-17	-21	-25	-28	-29	-33	-36	-40	-43	-47
18	P66%	-4	-11	-14	-19	-21	-24	-26	-29	-31	-34	-37	-40
	P5%	-6	-13	-17	-21	-25	-28	-30	-33	-36	-40	-43	-48
19	P66%	-3	-10	-12	-16	-18	-19	-22	-24	-26	-27	-27	-27
	P5%	-5	-12	-15	-19	-22	-25	-27	-30	-33	-35	-36	-37
20	P66%	-4	-10	-12	-16	-18	-20	-22	-24	-26	-28	-30	-30
	P5%	-5	-12	-15	-20	-23	-26	-28	-30	-33	-36	-38	-41
21	P66%	-4	-10	-12	-15	-17	-19	-21	-23	-25	-27	-29	-31
	P5%	-6	-13	-14	-19	-21	-23	-27	-29	-31	-33	-35	-38
22	P66%	-4	-10	-12	-15	-17	-19	-21	-23	-25	-27	-29	-31
	P5%	-6	-12	-14	-19	-21	-23	-27	-29	-31	-33	-35	-38
23	P66%	-4	-9	-11	-13	-15	-17	-17	-19	-21	-23	-25	-27
	P5%	-6	-11	-13	-17	-19	-21	-24	-26	-28	-30	-32	-35
24	P66%	-4	-9	-13	-15	-18	-21	-19	-22	-26	-28	-30	-30
	P5%	-5	-12	-15	-19	-23	-26	-26	-29	-33	-36	-38	-39
25	P66%	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4
	P5%	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5
26	P66%	-8	-8	-8	-8	-8	-8	-8	-8	-8	-8	-8	-8
	P5%	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10

5.5 Harbours (detailed sites)

5.5.1 Lyttelton Harbour

The ASCE distances for the P66% and P5% around the detailed sites within Lyttelton Harbour are presented in Table 5.5. The current P5% ASCE is largest within Cass Bay (Cell 33) and Rapaki Bay (Cell 36), where the current ASCE accounts for potential instability of the high banks, ranging from -15 to -25 m. The current P5% ASCE along the harbour beaches (Charteris Bay, Hays Bay and Purau) is smaller, ranging from -6 to -7 m.

Erosion protection structures, in varying condition, exist around the Lyttelton Harbour sites and in these locations the current ASCE represents the immediate hazard if the structures were to fail. However, for the future ASCE, the structures have not been accounted for and the ASCE represents the erosion hazard in absence of the structures.

Long-term trends and SLR response are estimated to be relatively similar across the harbour sites. For the harbour beaches, the 2050 P5% ASCE under low SLR ranges from -10 to -12 m and by 2130 under high SLR is up to -32 m. For the harbour banks, the 2050 P5% ASCE under low SLR ranges from -6 to -27 m and by 2130 under high SLR, is up to -36 m.

For majority of the cells the short-term erosion (i.e. storm cut on beaches or bank instability) contributes to over 50% of the total erosion distance in 2050. Over time the impact of long-term trends and SLR response increases and subsequently has a greater influence on the total erosion distance.

Long-term erosion rates are assumed to be relatively similar across the harbour and therefore with no/low SLR there is little variation in the future ASCE. As with the current ASCE, the variation across the harbour banks is a function of the bank height, with higher banks having a larger ASCE. The harbour beaches, in particular Purau Bay, are more sensitive to SLR compared with harbour banks. For example, by 2130 the P5% ASCE for Purau varies by up to 10 m depending on the amount of SLR, whereas the 2130 P5% along the harbour banks only varies by up to 6 m under different SLR scenarios. For Charteris Bay and Hays Bay which are very low-lying, the maximum erosion extent, as a result of SLR, is assumed to be controlled by the height of the beach crest, so once the sea level exceeds the crest height and inundation occurs, there is no additional increase in erosion. This can be expected to occur beyond 1 m SLR.

The current and future P5% around Lyttelton Port (Cell 31) is -15 m, which represents the immediate hazard if the structures around the Port were to fail. As the shoreline around the Port is predominately reclamation fill it is it is likely structure failure without repair, would eventually result in the shoreline retreating to its 'original' natural position, however modelling this scenario was not within the scope of this study.

Table 5.5: ASCE widths (m) for the P66% ('likely') and P5% ('very unlikely') ASCE around the detailed sites in Lyttelton Harbour

Site	Cell	Probability of exceedance	Current	2050		2080			2130					
			0 m SLR	0.2 m SLR	0.4 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	1 m SLR	1.2 m SLR	1.5 m SLR
Lyttelton Port	31	P66%	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12
		P5%	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15
Corsair Bay	32	P66%	-6	-7	-8	-9	-9	-9	-10	-10	-11	-11	-11	-12
		P5%	-8	-10	-11	-13	-14	-14	-15	-16	-17	-19	-19	-21
Cass Bay (east)	33	P66%	-19	-21	-21	-22	-23	-23	-23	-24	-24	-25	-25	-26
		P5%	-25	-27	-27	-29	-30	-30	-31	-32	-33	-34	-35	-36
Cass Bay (west)	34	P66%	-7	-8	-9	-10	-10	-10	-10	-11	-12	-12	-12	-13
		P5%	-9	-11	-12	-14	-15	-15	-16	-17	-18	-19	-20	-22
Rapaki Bay (east)	35	P66%	-3	-5	-5	-6	-6	-6	-7	-7	-8	-8	-8	-9
		P5%	-4	-6	-7	-9	-10	-11	-12	-13	-14	-15	-16	-17
Rapaki Bay (west)	36	P66%	-12	-13	-13	-14	-15	-15	-15	-16	-16	-17	-17	-18
		P5%	-15	-17	-17	-19	-20	-21	-21	-22	-24	-25	-25	-27
Charteris Bay (west)	42	P66%	-4	-10	-12	-15	-18	-20	-19	-22	-24	-25	-25	-25
		P5%	-6	-12	-15	-19	-22	-24	-26	-29	-31	-32	-32	-32
Charteris Bay (east)	43	P66%	-5	-6	-7	-7	-8	-8	-8	-9	-9	-10	-10	-11
		P5%	-7	-9	-10	-11	-12	-13	-14	-15	-16	-17	-18	-19
Hays Bay	44	P66%	-4	-8	-10	-11	-13	-13	-12	-14	-14	-14	-14	-14
		P5%	-6	-10	-12	-14	-16	-17	-16	-18	-19	-19	-19	-19
Purau	45	P66%	-4	-8	-10	-11	-14	-16	-12	-15	-17	-19	-20	-20
		P5%	-6	-10	-13	-14	-17	-20	-17	-19	-22	-24	-26	-27

5.5.2 Akaroa Harbour

The ASCE distances for the P66% and P5% around the detailed sites within Akaroa Harbour are presented in Table 5.6. The current ASCE is largest along the northern end of Akaroa (Cell 73) and along the southern side of Takamatua Bay (Cell 75), where the ASCE accounts for potential instability of the high banks, ranging from -14 to -20 m for the P5%. The current ASCE tends to be smaller across the harbour beaches, ranging from -6 to -8 m for the P5%.

Erosion protection structures, in varying condition, exist around the Akaroa Harbour. In these locations the current ASCE represents the immediate hazard if the structures were to fail. However, for the future ASCE, the structures have not been accounted for and the ASCE represents the erosion hazard extent in absence of the structures.

For majority of the cells the short-term erosion (i.e. storm cut on beaches or bank instability) contributes to over 50% of the total erosion distance in 2050. Over time the impact of long-term trends and SLR response increases and subsequently has a greater influence on the total erosion distance.

Long-term erosion rates are assumed to be relatively similar across the harbour and therefore with no/low SLR there is little variation in the future ASCE. The harbour beaches are slightly more sensitive to SLR compared with harbour banks. For example, by 2130 the P5% ASCE for Wainui varies by up to 12 m depending on the amount of SLR, whereas the 2130 P5% along the harbour banks only varies by up to 6 m under different SLR scenarios. For low-lying beaches, such as Takamatua and Duvauchelle, the maximum erosion extent, as a result of SLR, is assumed to be controlled by the height of the beach crest, so once the sea level exceeds the crest height and inundation occurs, there is no additional increase in erosion. This can be expected to occur beyond 1 m SLR.

Cells 69, 70, 71 and 72 are along Akaroa township and are classified as Class 1 structures (see Section 3.1.5). The ASCE within these cells represents the immediate hazard if the structure were to fail and is a function of the structure height and stable angle of repose for the filled material. The future ASCE has been set equivalent to the current hazard area, which would be the case if the structure was promptly repaired if damaged. However, if the protection structure fails and is not promptly repaired then it is likely the fill material will rapidly erode, and the shoreline will eventually move back towards its 'original' natural position (this scenario has not been modelled in this study).

Table 5.6: ASCE widths (m) for the P66% ('likely') and P5% ('very unlikely') ASCE around the detailed sites in Akaroa Harbour

Site	Cell	Probability of exceedance	Current	2050		2080			2130					
			0 m SLR	0.2 m SLR	0.4 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	0.4 m SLR	0.6 m SLR	0.8 m SLR	1 m SLR	1.2 m SLR	1.5 m SLR
Akaroa township	69	P66%	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7
		P5%	-11	-11	-11	-11	-11	-11	-11	-11	-11	-11	-11	-11
Akaroa township	70	P66%	-6	-6	-6	-6	-6	-6	-6	-6	-6	-6	-6	-6
		P5%	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7
Akaroa township	71	P66%	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4
		P5%	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5
Akaroa township	72	P66%	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4	-4
		P5%	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5
Akaroa north	73	P66%	-15	-16	-17	-18	-18	-19	-19	-20	-20	-21	-21	-22
		P5%	-19	-21	-22	-23	-24	-25	-25	-26	-27	-28	-29	-30
Childrens Bay	74	P66%	-4	-5	-6	-7	-7	-7	-8	-8	-9	-9	-10	-10
		P5%	-6	-8	-8	-10	-11	-12	-13	-14	-15	-16	-17	-19
Takamatua Bay (bank)	75	P66%	-9	-11	-11	-12	-13	-13	-14	-14	-15	-15	-16	-16
		P5%	-14	-15	-16	-18	-19	-19	-20	-21	-22	-23	-24	-25
Takamatua Bay (beach)	76	P66%	-4	-8	-11	-12	-14	-14	-13	-15	-16	-16	-16	-16
		P5%	-6	-10	-13	-14	-17	-19	-17	-20	-21	-22	-22	-22
Duvauchelle Bay (beach)	79	P66%	-4	-8	-11	-12	-15	-17	-13	-16	-19	-19	-19	-19
		P5%	-6	-10	-13	-14	-17	-20	-17	-20	-23	-24	-24	-24
Duvauchelle Bay (bank)	80	P66%	-5	-7	-7	-8	-8	-9	-9	-10	-10	-11	-11	-12
		P5%	-6	-9	-10	-11	-12	-13	-14	-15	-16	-17	-18	-20
Duvauchelle Bay (beach)	81	P66%	-4	-8	-11	-12	-14	-15	-13	-16	-17	-17	-17	-17
		P5%	-6	-10	-13	-14	-17	-20	-17	-20	-22	-25	-26	-27
Wainui	91	P66%	-6	-10	-12	-13	-15	-17	-14	-16	-18	-21	-23	-26
		P5%	-8	-11	-14	-15	-17	-20	-18	-20	-22	-25	-27	-30

5.6 Banks Peninsula (regional hazard screening sites)

The ASCE distances for the regional beach and bank sites around Banks Peninsula are presented in Table 5.7. The current ASCE is largest on the exposed Banks Peninsula beaches and is smallest within the low, sheltered harbour banks. For example, at Hickory Bay and Le Bons Bay, the current ASCE ranges from -22 m to -25 m, whereas within Port Levy and Pigeon Bay current ASCE ranges from -3 m to -9 m.

Erosion protection structures, in varying condition, exist around the regional hazard screening sites. In these locations the current ASCE represents the immediate hazard if the structures were to fail. However, for the future ASCE, the structures have not been accounted for and the ASCE represents the erosion hazard in absence of the structures.

Over shorter timeframes (i.e. by 2050), the short-term erosion component dominates the total ASCE distance. For example, for majority of the cells around Banks Peninsula, the short-term erosion accounts for 40 to 70% of the total ASCE distance in 2050. Over longer timeframes the contribution of short-term erosion reduces as the impact of long-term trends and SLR increases. For example, by 2130 with 1.5m SLR, the short-term erosion accounts for less than 20% of the total ASCE distance.

Long-term erosion on outer Banks Peninsula beaches is typically negligible with some long-term accretion apparent in some areas such as Okains Bay. Subsequently, with low SLR, there is minimal difference between the 2080 and 2130 ASCE on these beaches. However, under high SLR scenarios the ASCE increases significantly for these beaches. Due to relatively low dune systems and the wave exposure, it is expected that under increasing sea levels, these beaches will shift a significant distance landward. There is however limited data on the closure depths (offshore profiles and wave climate) and therefore assumptions have been made in estimating the beach response on these shorelines. Subsequently the results on these beaches are likely to be conservative.

In contrast, the harbour beaches and banks tend to have slight long term erosion however they are less sensitive to SLR compared with the outer peninsula beaches, with the harbour banks being the least sensitive. For example, by 2130 the difference in ASCE distance for low and high SLR scenarios ranges from 1 m to 7 m on the harbour banks and is up 16 m difference on the harbour beaches. As the sea level rises the water depth within the harbour will increase, allowing greater wave heights to reach the shoreline and subsequently increase the erosion. However, as the harbour is a lower energy environment, erosion is likely to occur more episodically and slowly compared with the energetic open coast.

The ASCE around the cliffs is not derived from calculated distances but is instead mapped based on the area of steep coastal slopes (equal to or steeper than 1(H):1(V)), plus the 20 m buffer (see Section 4.6.5). The ASCE for the cliffs is spatially variable depending on the slopes and tends to be largest in areas where there is a high and steep coastal edge.

Table 5.7: ASCE widths (m) for regional beach and bank sites around Banks Peninsula

Site	Cell	Current	2080	2130	
		0 m SLR	+0.4 m SLR	+0.4 m SLR	+1.5 m SLR
Sandy Beach Rd	37	-6	-14	-17	-26
Allandale	38	-8	-16	-19	-25
Teddington	39	-7	-14	-17	-21
Moepuku	40	-9	-16	-18	-29
Charteris Bay	41	-9	-17	-20	-31
Port Levy	46	-3	-11	-13	-18
Port Levy	47	-3	-11	-13	-18
Port Levy	48	-7	-15	-16	-17
Port Levy	49	-8	-15	-17	-23
Holmes Bay	50	-8	-16	-19	-25
Pigeon Bay	51	-9	-18	-21	-32
Pigeon Bay	52	-9	-18	-21	-32
Menzies Bay	53	-15	-24	-27	-38
Decanter Bay	54	-14	-22	-25	-36
Little Akaloa	55	-16	-25	-28	-39
Little Akaloa	56	-18	-26	-30	-41
Raupo Bay	57	-14	-34	-34	-89
Raupo Bay	58	-14	-34	-34	-89
Stony Beach	59	-16	-36	-36	-91
Okains Bay	60	-22	+18	+68	+13
Lavericks	61	-23	-43	-43	-98
Le Bons Bay	62	-23	-25	-10	-65
Le Bons Bay	63	-22	-36	-31	-86
Hickory	64	-25	-45	-45	-100
Goughs Bay	65	-23	-43	-43	-98
Otanerito Bay	66	-15	-35	-35	-90
The Kaik	67	-6	-14	-17	-21
Akaroa south	68	-9	-17	-20	-24
Robinsons Bay	77	-12	-20	-23	-27
Robinsons Bay	78	-9	-17	-20	-31
Barrys Bay	82	-14	-22	-24	-29
Barrys Bay	83	-5	-12	-15	-19
Barrys Bay	84	-8	-15	-18	-23
Barrys Bay	85	-9	-17	-19	-33
Barrys Bay	86	-12	-20	-23	-27
French farm bay	87	-6	-14	-17	-21
French Farm Bay	88	-9	-17	-19	-33

Table 5.7 (continued): ASCE widths (m) for regional beach and bank sites around Banks Peninsula

Tikao Bay	89	-6	-14	-17	-21
Tikao Bay	90	-11	-18	-21	-26
Wainui south	92	-12	-20	-23	-27
Peraki Bay	93	-15	-35	-35	-90
Te Oka Bay	94	-17	-37	-37	-92
Tumbledown Bay	95	-17	-37	-37	-92

5.7 Kaitorete Spit

The ASCE distances for Kaitorete Spit are presented in Table 5.8. The current ASCE accounts for potential short-term storm cut and berm instability which is slightly larger in the centre of the spit.

The long-term trends gradually change from erosion at the southern end to accretion at the northern end and hence the variation in future ASCE. Accretion rates within Cell 96, near Birdlings Flat, are high and potentially will counteract any impacts from future SLR. As a result of the differences in long-term trends, shoreline orientation will change until equilibrium is reached with longshore transport.

Over short timeframes (i.e. 2080), the short-term storm response tends to dominate the future ASCE, particularly at the northern end where LT erosion is minimal. For example, within Cell 97 the short-term storm response contributes almost 80% of the total ASCE distance in 2080 with 0.4 m SLR.

Table 5.8: ASCE widths (m) for cells along Kaitorete Spit

Cell	Current	2080	2130	
	0 m SLR	+0.4 m SLR	+0.4 m SLR	+1.5 m SLR
96	-24	+12	+58	+42
97	-24	-32	-23	-39
98	-30	-66	-80	-96
99	-30	-78	-102	-118
100	-21	-68	-93	-108

6 Coastal inundation methodology

6.1 Conceptual approach

Coastal inundation is flooding of land from the sea. A range of different variables can contribute to coastal inundation including the astronomical tide, storm surge associated with low pressure weather systems, mean sea level fluctuations, wave effects and sea level rise. Coastal inundation is typically split up in static or dynamic inundation. Static inundation is combination of astronomic tide, means sea level fluctuations and storm surge (called storm tide) and wave set-up. Dynamic inundation is a combination of storm tide and wave run-up.

Extreme static and dynamic inundation levels have been considered separately due to the different inundation mechanisms. Static inundation could potentially inundate large areas due to the consistently elevated water level, whereas dynamic inundation due to wave run up is temporary and restricted to the coastal edge, typically in the order of 10-30 m (see schematisation in Figure 6.1).

The extreme static water levels and extreme dynamic water levels are based on the following combinations:

$$Extreme\ static\ water\ level = ST + SU + SLR \qquad (6.1)$$

$$Extreme\ dynamic\ water\ level\ = ST + RU + SLR \qquad (6.2)$$

Where:

ST = **Storm tide** (#1 in Figure 6.1) level defined by the combination of astronomical tide, storm surge and mean sea level fluctuations.

SU = **Wave set-up** (#2a in Figure 6.1) caused by wave breaking and onshore directed momentum flux across the surf zone.

RU = **Wave run-up** (#2b in Figure 6.1) being the maximum potential vertical level reached by individual waves above the storm tide level (note this component implicitly includes wave set-up).

SLR = **Sea level rise** (#3 in Figure 6.1) at specified increments (refer to Table 6.1).

The component values for each of the areas have been analysed as set out in Section 7. The resulting extreme static and dynamic water levels have been assessed (refer to Section 8) and rounded up to the nearest 0.1 m to allow for inaccuracies in data that was used.

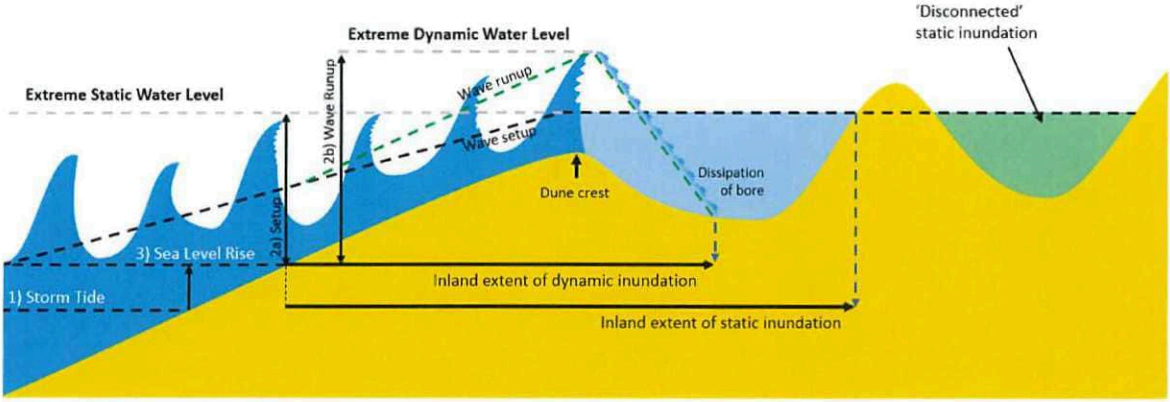

Figure 6.1: Schematisation of extreme water level components and combined extreme water levels.

Assessing and mapping coastal inundation takes a two part approach:

1 Assessing extreme water levels for representative locations along the open coast and within estuaries, lagoons and harbours resulting in a look up table of extreme levels for various scenario combinations.

2 Mapping static inundation (i.e., not dynamic inundation) extents and depths at 0.1 m increments around the entire coast (where covered by the 2018-2019 DEM). This has been referred to as "bathtub" inundation.

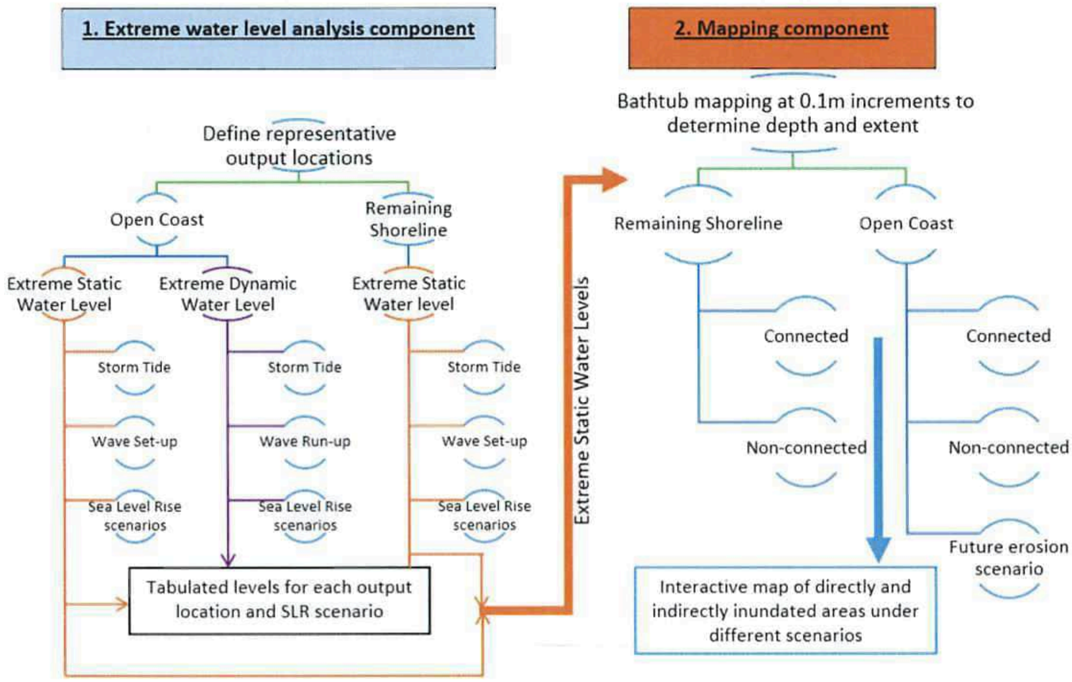

Figure 6.2: Proposed conceptual approach for inundation assessment and mapping.

6.2 Assessment level

Coastal inundation hazard levels have been assessed either to a detailed level (i.e. probabilistic) or regional hazard screening level (i.e. deterministic). In the sections that follow, detail on these approaches and the areas within which these approaches were applied are provided.

To undertake a detailed probabilistic inundation assessment, timeseries of water levels and wave heights are required, which are used to derive extreme values of total water level for different return periods. Alternatively, available reports or data including extreme values for different return periods could be used.

For the Christchurch open coast, water level data is available at the Sumner tide gauge and wave data is available from the MetOcean wave hindcast (1979 to 2019). Water level data is also available within the Avon-Heathcote Estuary, Brooklands Lagoon and Lyttelton Harbour. However, wave timeseries are not available in these locations. Wave timeseries are available at several locations along Banks Peninsula, however, these are situated offshore and have not been transformed to particular coastal locations.

NIWA (2015) also includes information on joint occurrence of storm tide and wave height and provides methods for calculating wave set up and run up for output locations along the open coast (excluding the Banks Peninsula). However, these levels are based on a hindcast from 1970 to 2000 and does not consider the effects of the 2018 storm events. As subsequent analysis of tide gauges by Goring (2018) and GHD (2021) show that 100-year ARI storm tide levels for Sumner are 0.2 m higher, the NIWA (2015) data has not been used for this study. Note that NIWA (2015) did not derive extreme levels from the Sumner tide gauge due to wave events affecting the quality of the water record (now resolved), however, the 100-year ARI level for Sumner is included in the report based on the Coastal Calculator.

Based on these data limitations we have assessed the appropriate level of detail for inundation assessment for the various parts of the shoreline, as summarised in Figure 6-3, and discussed further below.

6.2.1 Detailed inundation hazard assessment

Detailed assessments have been undertaken for the Christchurch open coast, Avon-Heathcote Estuary, Lyttelton and Akaroa Harbours. Due to different data availability, slightly different approaches have been used for each area.

Full probabilistic approach

A full probabilistic assessment is undertaken where both water level and wave timeseries are available. These timeseries are used to undertake extreme value analyses to derive return period water levels. A full probabilistic assessment has been undertaken for the following area:

- Christchurch open coast.

Quasi-probabilistic approach

A quasi-probabilistic assessment is undertaken where water level timeseries are available (or return period water levels based on water level timeseries, such as GHD, 2021), but wave timeseries are not available. This level of assessment is used for major harbours and estuaries that may be subject to super-elevation of water levels due to wave effects. For these harbours/estuaries numerical wave models (i.e. SWAN) have been set up, which use extreme wind speeds to model wind-generated waves to assess wave effects. Therefore, this level of assessment is a combination of probabilistically derived water levels with wave effects derived from the SWAN model added deterministically. A quasi-probabilistic approach has been undertaken for the following areas:

- Brooklands Lagoon.
- Avon-Heathcote Estuary.
- Lyttelton Harbour.
- Akaroa Harbour.

6.2.2 Regional inundation hazard screening assessment

A regional hazard screening assessment is undertaken where water level timeseries may be available, but nearshore wave timeseries is not available. This level assessment is used for the remaining shoreline for which no site-specific wave models (e.g. SWAN) have been set up, and use empirical formulas to assess the wave effects component. A regional hazard screening assessment has been undertaken for the following areas:

- Outer Banks Peninsula.
- Kaitorete Spit.
- Wairewa (Lake Forsyth).
- Te Waihora (Lake Ellesmere).

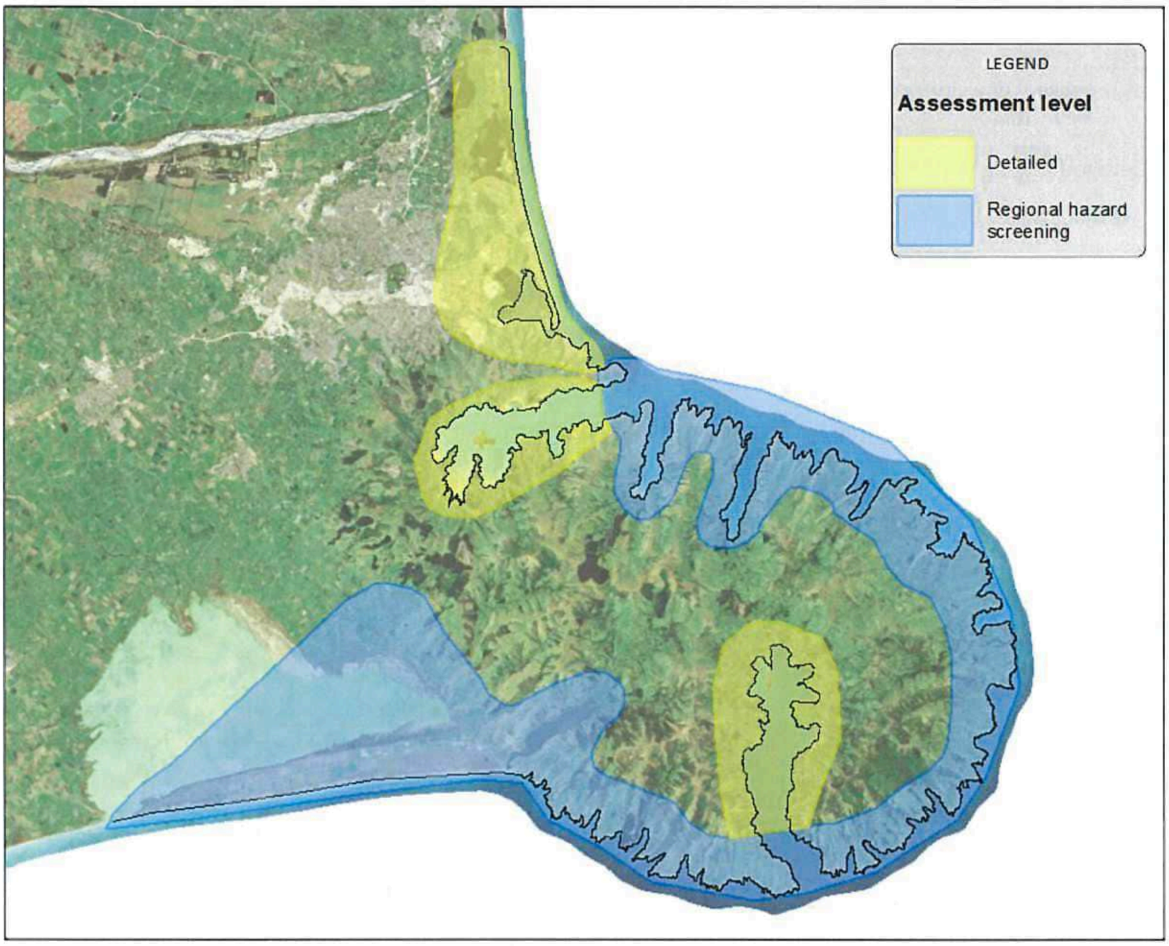

Figure 6-3 Christchurch district showing adopted extents and level of detail for the coastal inundation hazard assessment.

6.3 Scenarios

The previous Christchurch coastal hazard assessment (T+T, 2017) utilised a 1% AEP storm tide combined with 1% AEP wave height on the open coast. For the harbour sites, extreme wind speeds were used as input to derive wave heights and extreme water levels. Four sea level rise scenarios at two timeframes, 2065 and 2120 were utilised. The derived values were combined using a building block approach either directly or within a hydrodynamic model. Across the wider Canterbury region, recent studies have generally used a 1% or 2% AEP event, accounting for the joint probability of storm tide and wave effects via the NIWA coastal calculator. A single RCP 8.5+ scenario has been used in Selwyn District (ECan, 2018) and sea level rise increments between 0.2 and 0.7 m have been used in Waitaki District (NIWA, 2019) and Timaru District (2020). Elsewhere in New Zealand a range of approaches have been adopted, however, detailed assessments generally included multiple return events, and either multiple timeframes (generally 2030, 2050, 2080 and 2130) and RCP scenarios, or the use of incremental sea level rise scenarios.

MfE (2017) guidance recommends either direct usage of RCP scenarios or increments of sea level rise to inform adaptation planning. For this assessment, sea level rise increments have been adopted that can be aligned with timeframes and approximate RCP scenarios. Adopted assessment scenarios have been summarised in Table 6.1.

Table 6.1: Proposed assessment scenarios for inundation look up tables

Assessment	Relative sea level increment[1] (m)	Return period event[2]	Effect of erosion[3]
Detailed assessment[4]	0 +0.2 +0.4 +0.6 +0.8 +1.0 +1.2+1.5 +2.0	1 year ARI 10 year ARI 100 year ARI	-
	+1.5	100 year	Future P5% and P50% erosion for same scenario[2]
Regional screening assessment	0 +0.4 +1.5	1 year 10 year 100 year	-

1 Relative sea level combines the effect of both rising sea level and vertical land movement. Increments are specified relative to current-day sea level.

2 Return period events describe the Average Recurrence Interval (ARI) of an extreme water level (e.g. a 10 year ARI water level is a water level that is equalled or exceeded on average once every 10 years). Smaller ARI values represent lower water levels that occur more often, and larger ARI values represent higher water levels that occur less often.

3 Christchurch open coast only.

4 Both full probabilistic and quasi-probabilistic.

Future erosion may affect inundation hazard extents on the open coast, particularly where the eroded shoreline allows wave run up to propagate further inland (e.g., as a result of an eroded/lowered dune). This has been assessed initially for a single timeframe (2130) and high-end sea level rise scenario (1.5 m).

6.4 Mapping to determine inundation extent and depth

The areas potentially susceptible to static inundation have been mapped using a connected bathtub model. This approach maps inundation extents by imposing resulting static inundation levels on a boundary (i.e. coastline) of a DEM, and filling in the DEM where the topographic levels are below the static inundation level. This model differentiates areas below a specified inundation level that are connected to the coastal water body from those that are disconnected (Figure 6.1). The resulting inundation layers show both the extents and depths within the inundated extents.

In mapping the inundation extents using the bathtub approach it should be noted that these emerge from combining a DEM with a set of predicted extreme water levels. Inaccuracies in the DEM are likely to transmit to the resulting maps – and the reader is directed to the DEM limitations discussed in Section 2.1. Inaccuracies in the DEM are typically a result of post processing point cloud data, for instance removing roof or tree points and interpolating the levels from adjacent points. However, this would likely only result in localised inaccuracies.

For large inundated extents, the connected bathtub approach may result in conservative extents due to friction and unlimited peak flood duration. Flow through small openings such as stream mouths may similarly result in conservative inundation extents compared to reality. This could be resolved using a hydrodynamic model, however, for this assessment it was found that this results in similar inundation extents.

The previous T+T (2017) assessment utilised a hydrodynamic model to assess the extent of storm tide propagation within the Avon-Heathcote Estuary and Brooklands lagoon. Sensitivity analysis was undertaken between the hydrodynamic modelling results and the bathtub modelling results to confirm the suitability of the bathtub approach. Overall, the comparison concludes the bathtub approach is suitable for the intended purpose of this hazard assessment in adaptation planning work and other similar work acknowledging the level of detail and limitations of this assessment. Details on the model comparisons and justification for the bathtub approach is included in Appendix C.

For the Avon, Heathcote and Styx catchments we recommend that the bathtub model outputs (e.g. maps) are cut off upstream of the boundary defined in Figure 6.4. The boundary is based on hydraulic controls that have been identified within each of the major river systems. Within the mapped areas (downstream of the hydraulic control line shown in Figure 6.4), extreme inundation level is dominated by the sea level scenario applied. Upstream of these hydraulic control locations, extreme inundation level is increasingly influenced by river/stream flow, with lesser reliance on the sea level applied. On the Avon River the hydraulic control is approximately around Wainoni Road, on the Heathcote River it is near Radley Street and on the Styx River it is near Teapes Road. In these locations the flood plains narrow and subsequently there is a significant reduction in the peak inundation levels (via throttled flow) that may occur under the action of extreme sea level only (i.e. if river flow is not taken into account). Upstream of the hydraulic controls the bathtub model generally overestimates the extent of inundation because it applies a water level derived at the coast which is too high for the area further inland. The justification for this boundary is described in more detail in Appendix C. Extreme inundation of areas upstream of these control locations is best derived through joint probability modelling assessment, taking into account both sea level and river flow state. The Land Drainage Recovery Programme at Council focusses on planning in these areas and has existing models which are used.

The extent and depth of inundation was mapped for all areas with the most recently available DEM (2018-2019, except for Te Waihora/Lake Ellesmere which is 2008) at 0.1 m increments. Areas connected to the coastline that would be subject to direct inundation are shown separately from areas which are not connected but could be susceptible to inundation by piped connections and/or raised groundwater. Furthermore, disconnected areas may experience inundation due to rainfall that is unable to drain towards the sea. For these areas, the peak inundation level is limited by the peak sea level.

Wave run up on the open coast has not been mapped as run up is highly dependent on the site-specific beachface slope, relative dune/seawall crest level and whether run up exceeds the dune/seawall crest level. All these parameters change when different return period storms and sea level rise increments are considered and the variability in run-up elevations would therefore result in a large number of potential hazard lines. For the Christchurch open coast run-up attenuation distances are assessed for where the run-up levels exceed the coastal edge crest (e.g. at seawalls).

Areas subject to inundation under particular scenarios can be visualised using the online viewer, with sliders for event, timeframe and sea level rise scenario or for specific water level. This approach has the advantage that many of the combinations of event probability, timeframe and sea level rise scenario result in similar extreme water levels. Therefore, rather than having a multitude of similar and overlapping inundation maps, the incremental mapping would allow users to slowly increase water level and visualise inundated areas including depths. It also allows the user to independently evaluate the contribution to extreme level that is made by the different input parameters. This is likely to be more useful for public engagement and adaptation planning. Another advantage is that if any of the levels change due to reanalysis or updated data or guidance, only the lookup tables values need to be updated while mapping remains the same. The online viewer can be accessed at https://ccc.govt.nz/environment/coast/coastalhazards/2021-coastal-hazards-assessment

Figure 6.4: Recommended bathtub boundary shown in red (increased uncertainty in hydraulics on the western side of the red line).

6.4.1 Inundation protection structures

Existing stopbanks are already represented in the DEM (derived from LiDAR ground elevation survey information) which is used for the inundation analysis. Surveyed stopbank levels provided by CCC have been compared against the DEM derived from the 2018-2019 LiDAR survey to ensure all existing stopbanks are accurately captured (Figure 6.5). This showed that the DEM and survey levels are typically within 0.1 m, with the DEM typically being higher than surveyed levels. As the differences are within the derived water level accuracy, the DEM has been adopted directly without the need to "burn in" specified stopbank crest levels. Current and planned stopbanks will be identified on the maps.

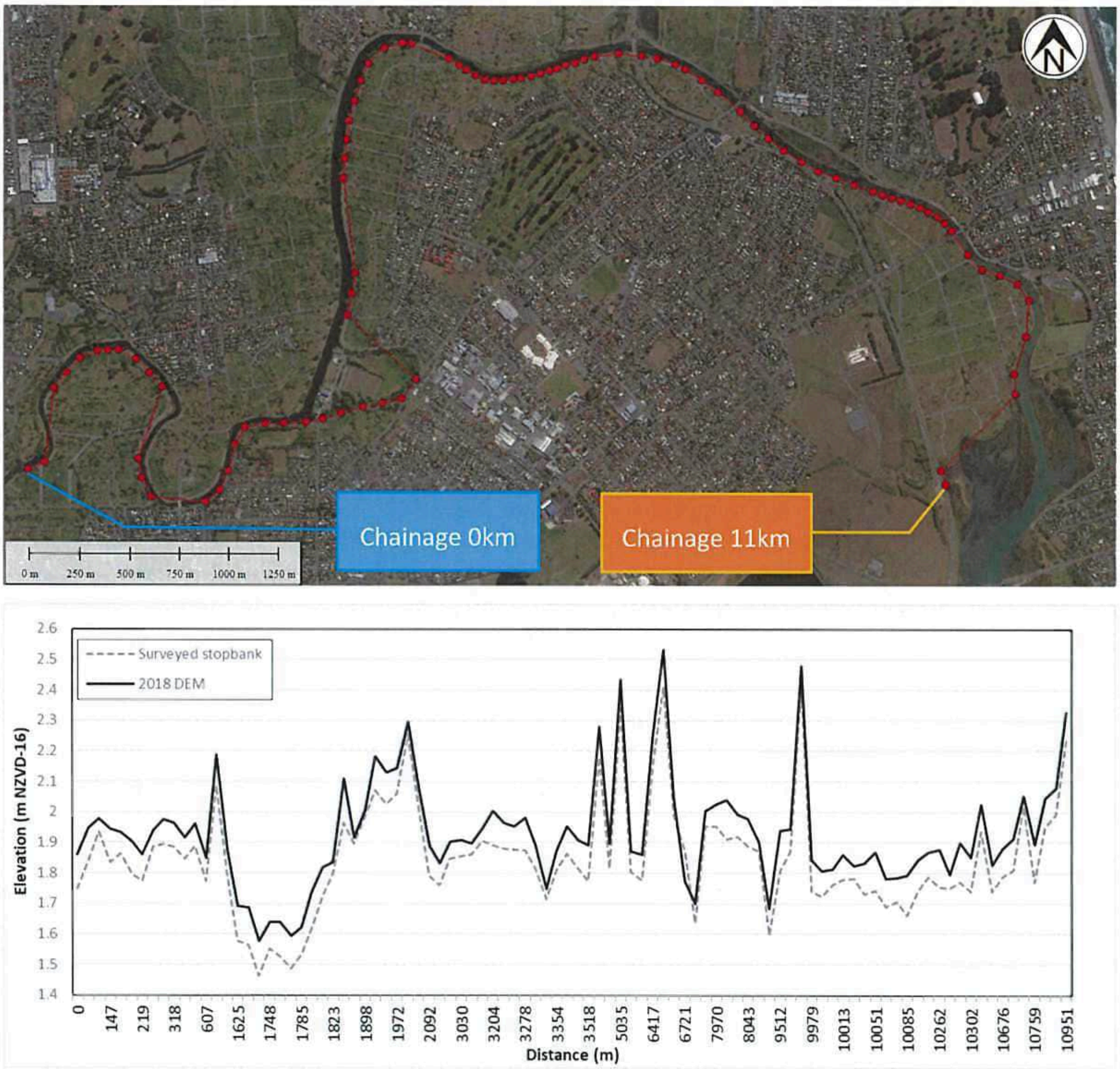

Figure 6.5: Right stopbank map (top panel) and difference between stopbank elevations in 2018-2019 DEM and surveyed levels along the right bank of the Avon River (lower panel).

7 Coastal inundation analysis

This section sets out the analysis of the extreme water levels including input data and output locations for the Christchurch open coast, major harbours and estuaries and regional hazard screening sites. The resulting extreme water levels for the selected scenarios (refer to Section 6.3) have been derived using the conceptual models set out in Section 6.1, and are set out in the next chapter (Section 8).

7.1 Christchurch open coast

The Christchurch open coast extends from the mouth of the Waimakariri River south, includes the mouth of the Avon-Heathcote Estuary and Sumner, and terminates at the eastern end of the beach at Taylors Mistake. Within this area, inundation levels have been assessed probabilistically (refer to Section 6.2.1).

The Christchurch open coast is susceptible to storm surges and to both open ocean swell and locally wind-generated waves from the easterly quadrant. Open ocean waves typically arrive from the north-east from storms at lower latitudes or from the south wrapping around Banks Peninsula. Storm surges could occur at the same time as large wave events; however these events are only partially dependent (e.g. large swell waves and storm are independent, but large local wind-waves and storm surge may be dependent). Wave effects such as wave set-up and wave run-up could locally further elevate the water level along the open coast.

7.1.1 Input data

For the Christchurch open coast the water level timeseries from the Sumner gauge and wave timeseries along the Christchurch open coast have been used to assess the extreme water levels. The water level timeseries includes hourly data from 1994 to 2020. The wave timeseries includes 3-hourly data from 1979 to 2020 extracted at the -10 m depth contour at locations set out in Section 2.5. Based on a review of the wave timeseries, the differences between the four output locations were found to be small (i.e. 0.1 m for 100-year ARI wave height, refer to Table 2.8) and therefore a single wave timeseries has been used to assess the open coast inundation levels.

In addition to wave and water level timeseries, beach profile slopes have been used to assess the wave effects component. The surfzone (relevant for wave set-up) and beachface (relevant for wave run-up) slopes have been reviewed by assessment of the average profiles of each survey profile dataset. The beach profiles were averaged by taking the average elevation across the profile taking into account all surveyed profiles but separately for each profile CCC location. The resulting slopes for each profile dataset are shown in Figure 7.1. The beachface slope is based on the beach slope around the extreme still water level (i.e. typically between 1 m and 4 m NZVD2016). The surfzone slope is based on the slope below the 1 m NZVD2016 contour offshore to where the surveyed profile extends (typically -1 m NZVD2016). As offshore elevation data is limited to a single -10 m depth contour from LINZ (i.e. no shallower depth contours), the beach profile dataset has been used.

Figure 7.1 shows that the surfzone slope is typically between 1(V):60(H) and 1:80, with slightly more variation in the profiles at the southern end of the shoreline (i.e. CCC362-CCC1065). However, a consistent alongshore upper bound slope of around 1:60 can be seen in Figure 7.1. The beachface slope is typically between 1:15 and 1:20 and consistent along the shoreline. Based on this both a single surfzone slope (1:65) and a single beachface slope (1:15) have been adopted for the open coast shoreline between Waimakariri and Southshore. For Sumner a surfzone slope of 1:65 and beachface slope of 1:15 were adopted based on available beach profile data. For Taylor's Mistake a surfzone slope of 1:50 and beachface slope of 1:15 were adopted based on available beach profile data.

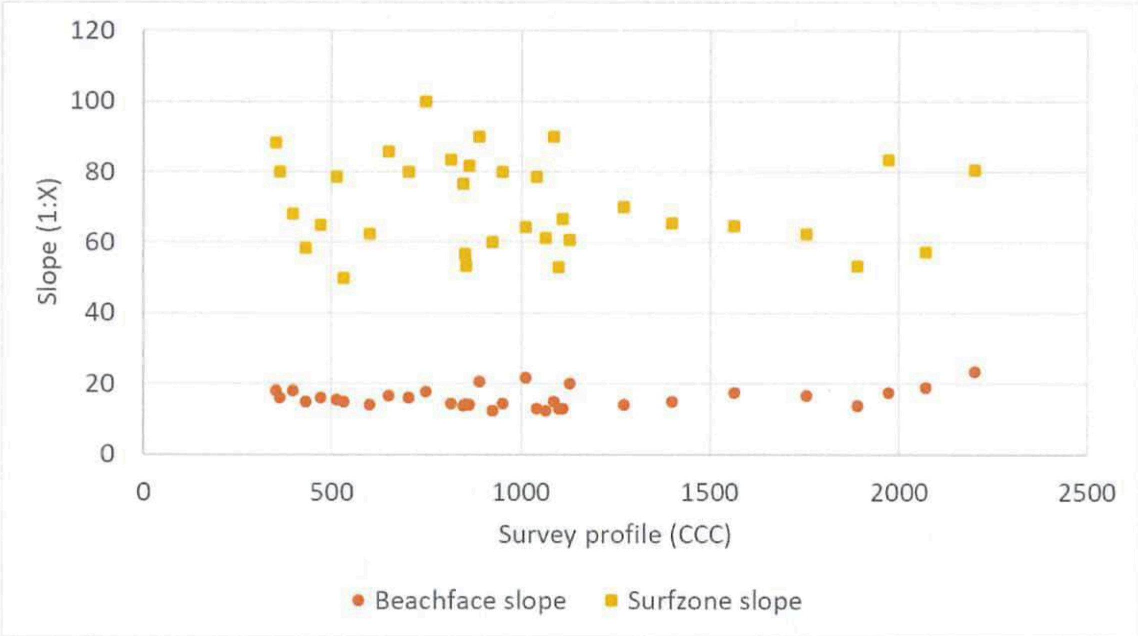

Figure 7.1: Alongshore beachface and surfzone slopes based on averaged surveyed profiles (note that CCC beach profiles run from south to north, e.g. CCC2000 is farthest north).

7.1.2 Analysis of extreme water levels

The extreme static water level is the result of the wave set-up superimposed on the still water level or storm tide occurring at that time. Traditional *building block* approaches apply wave set-up resulting from an extreme event onto a corresponding (or lesser) extreme storm tide level. While there appears a partial dependence between wave height and storm surge, there will be less dependence between wave height and storm tide where the independent astronomical tide is a primary contributor. This is particularly true for short duration events (or sheltered coastlines exposed to only a portion of the event) where the storm peak may not coincide with a high tide. This is in line with GHD (2021) who discuss independence between surge and tide. Therefore, the combined storm tide and wave setup have been calculated for a full time series with extreme value analysis undertaken on the resultant values (refer to Section 7.1.2.3). The joint occurrence of processes is therefore implicitly included in analysis.

The extreme dynamic water level is the result of wave run up (implicitly including wave set up) superimposed on the still water level or storm tide occurring at that time. The same analysis as for the extreme static water level has been undertaken to derive extreme dynamic water levels with the combined storm tide and wave run up calculated for a full time series with extreme value analysis undertaken on the resultant values.

Empirical equations have been used to calculate the wave set-up and wave run-up for a full timeseries. However, as there is a range of equations available, a numerical model was used to select the most suitable equation (refer to Section 7.1.2.1).

7.1.2.1 Validation of XBeach model

For the selection of appropriate wave setup formula, the numerical model XBeach NH (Deltares, 2015) has been utilised. XBeach NH (non-hydrostatic) is a numerical model that is able to transform offshore waves to the nearshore and simulate wave-induced set up and wave run up (see example of model in Figure 7.2). Two historic storms have been simulated by XBeach to extract wave run up and setup levels to compare with field data and values calculated by the empirical formulas. These storm events were selected based on NIWA (2015), which include recorded storm event dates and surveyed debris lines following storms at several locations along the open coast beach. Only two storm events were found to be suitable based on available data and recorded levels (i.e. no further representative data points were available).

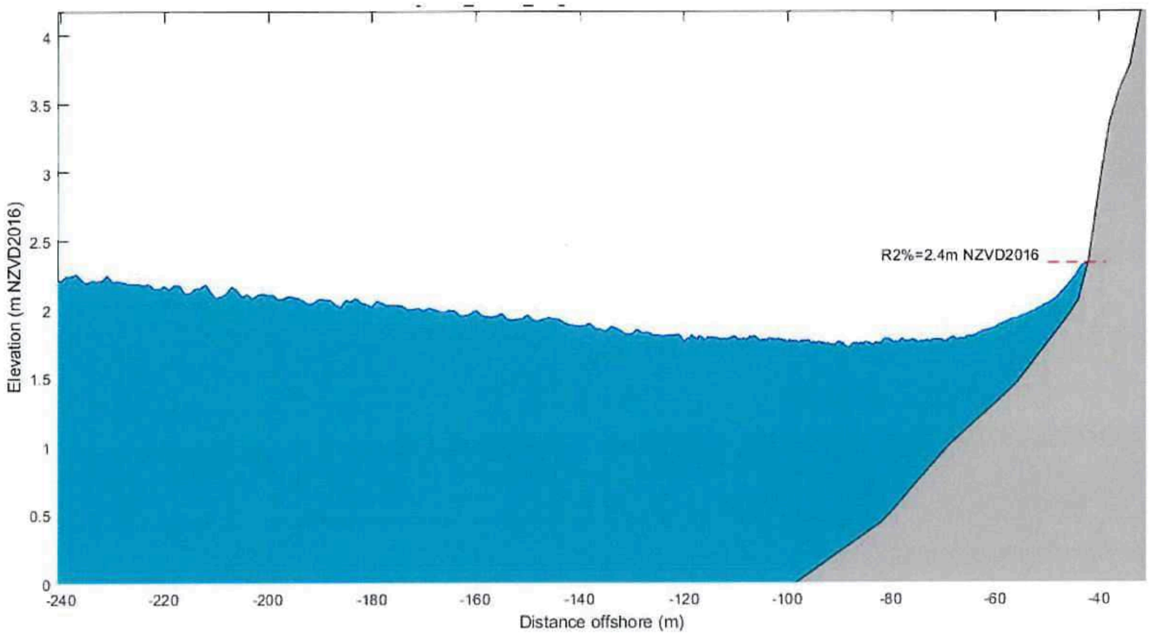

Figure 7.2: Example of XBeach NH model output.

XBeach has been run using the nearshore wave data from MetOcean and Sumner tide gauge water levels as input conditions, with surveyed beach profile information as the cross-shore profile. Table 7.1 shows the storm event dates, profile location and surveyed debris line levels.

Table 7.1: Storm event dates, location, surveyed levels and modelled wave run-up levels

Date	Location	Profile	Surveyed run-up debris line (m NZVD2016)[1]	Modelled wave run up[2] by XBeach (m NZVD2016)
3-4 March 2014	Waimairi	CCS1130	2.27	2.67
20-21 July 2001	New Brighton South	CCS362	3.07	2.4

[1]Source: NIWA (2015).

[2]$R_{2\%}$ (wave run up exceeded by 2% of wave run up events).

For these storms the surveyed debris lines, assumed to approximate the wave run up extents (refer to Shand et al., 2011), were 2.27 m NZVD2016 and 3.07 m NZVD2016 for respectively the 2014 and 2001 storms. The XBeach model simulated $R_{2\%}$ (wave run up exceeded by 2%) levels of 2.67 m NZVD2016 and 2.4 m NZVD2016 respectively. Therefore, the wave run up is overestimated by 0.4 m for the 2014 storm and underestimated by roughly 0.7 m for the 2001 storm. This may suggest that the modelled wave set up level may be slightly overestimated for the 2014 storm and slightly underestimated for the 2001 storm. However, for the purpose of selecting appropriate empirical formulas, the XBeach model results were used taking into account the over- and underestimations.

7.1.2.2 Calibration of empirical models

Wave set-up

A standard empirical formula has been used to calculate wave setup, with the empirical formulas by USACE (2006), Stockdon et al. (2006), Guza and Thornton (1981) and Battjes (1974) considered for this project.

The resulting wave setup heights modelled by XBeach (i.e. 0.43 m for 2014 storm and 0.5 m for 2001 storm) compared to the empirically calculated wave set up heights for the two storms are shown in Figure 7.3. This shows that both the Stockdon et al. (2006) and Guza and Thornton (1981) formulas underpredict wave set up compared to the XBeach modelled set up for both storms. The calculated maximum wave set up using the USACE (2006) formula is similar as the modelled wave setup for the 2014 storm (i.e. 0.42 m), but overestimates wave set up for the 2001 storm (i.e. 0.79 m vs 0.5 m). Both the Battjes (1974) formula and USACE (2006) - SWL (still water line) set up formula show a slight underestimation for the 2014 storm (i.e. -0.1 m and -0.05 m) and slight overestimation for the 2001 storm (i.e. + 0.08 m and + 0.15 m). Based on this comparison, both Battjes (1974) and USACE (2006) – SWL set up are the most similar to the modelled wave set up by XBeach (i.e. in terms of smallest sum of residuals). Taking into account that XBeach slightly overpredicts the 2014 storm run-up and underpredicts the 2001 storm run-up, the wave set-up calculated by both empirical models are expected to be similar to the actual wave set-up. As the Battjes (1974) formula is solely a function of the wave height and the USACE (2006) – SWL set up formula is a function of wave height, period and surfzone/beach slope, the latter formula is expected to predict wave set up better for a range of slope gradients and has been adopted for this study.

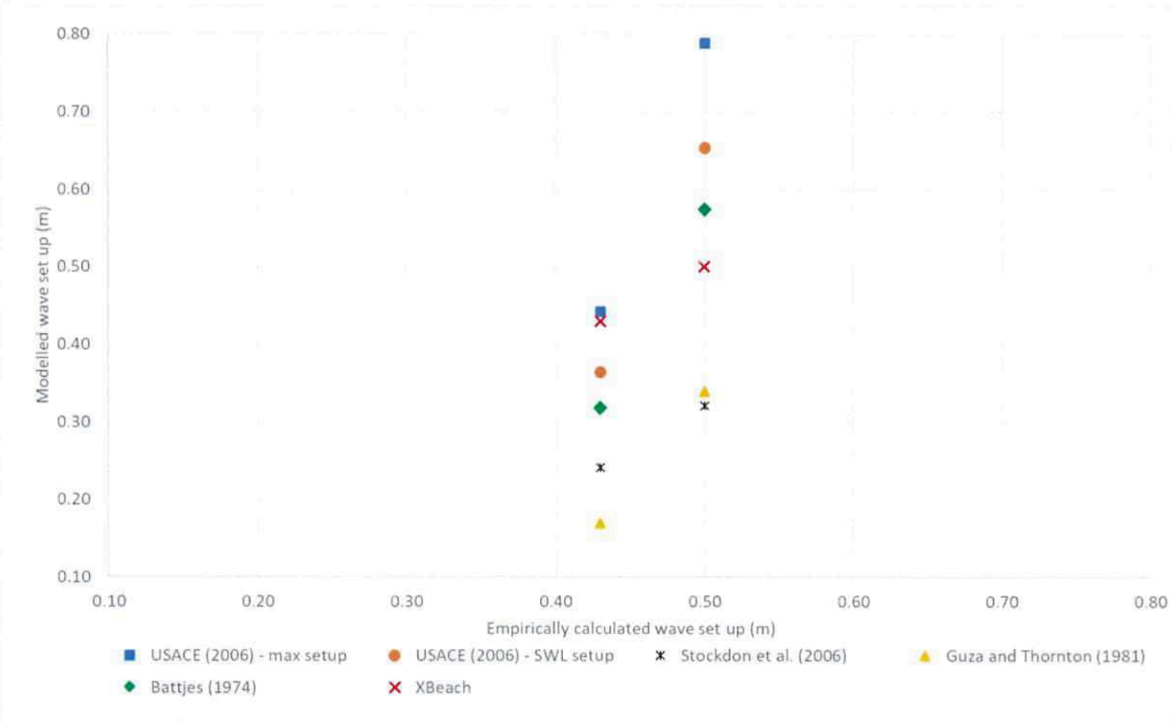

Figure 7.3: XBeach modelled versus empirically calculated wave set up.

Wave run-up

A range of empirical wave run up formulas have been considered to predict wave run up levels, including Mase (1989), Stockdon et al. (2006), Hedges and Mase (2004) and Gomes da Silva et al. (2012). In line with the review of the wave set up empirical formulas, the 2001 and 2014 storm events (refer to Table 7.1) have been used to compare wave run up levels. Figure 7.4 shows the comparison of surveyed debris lines, assumed to approximate wave run up extents, and empirically calculated wave run up levels for the 2001 and 2014 storm events.

Figure 7.4 shows that Gomes da Silva et al. (2012) significantly overpredicts the 2014 event wave (i.e. 4.5 m NZVD2016 versus 2.3 m NZVD2016, but reasonably predicts the 2001 event wave run up level (i.e. 3.1 m NZVD2016). Both Hedges and Mase (2004) and Stockdon et al. (2006) slightly overpredict run up for the 2014 event and underpredict wave run up for the 2001 event. Mase (1989) overpredicts the 2014 event run up (i.e. 2.9 m NZVD2016 versus 2.3 m NZVD2016), but accurately predicts the 2001 event run up.

Based on this comparison (i.e. sum of residuals), the Mase (1989) has been adopted for this study as the predicted run up for the most extreme event (i.e. 2001 event) was closest to the surveyed debris line. Gomes da Silva et al. (2012) also predicted a wave run up level close to the measured debris line, however, they significantly overpredict the 2014 event wave run up level. Both Hedges and Mase (2004) and Stockdon et al. (2006) predicted a wave run up level more than 0.5 m below the surveyed debris line for the 2001 storm.

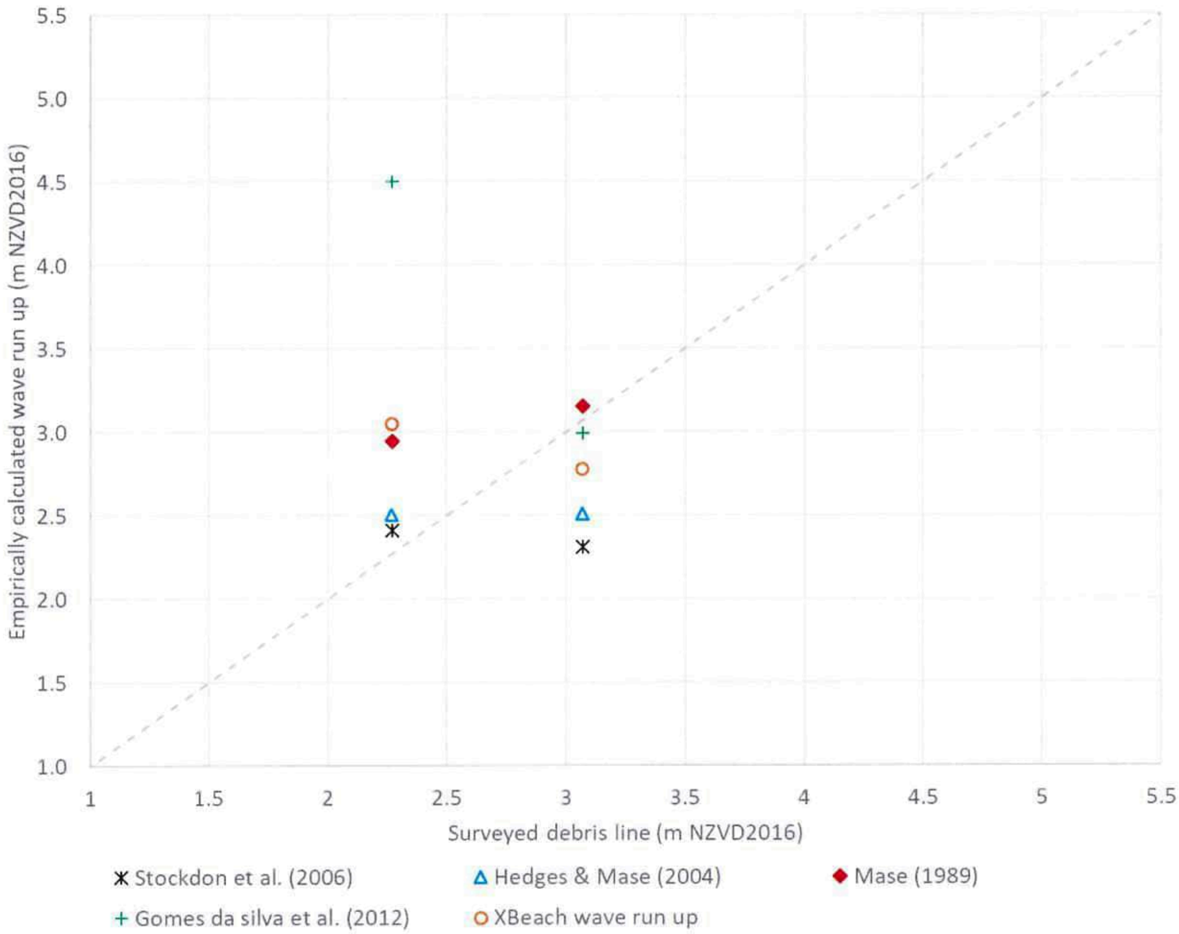

Figure 7.4: Comparison of surveyed debris line (assume wave run up extent) and empirically calculated wave run up.

7.1.2.3 Combined storm tide and wave effects

The following approach has been adopted to quantify the combined water level resulting from these components:

1 Develop hourly timeseries of nearshore wave heights based on the 1979-2019 wave hindcast data at the -10 m depth contour at each location along the shoreline provided by MetOcean.

2 Develop an equivalent hourly timeseries of water levels based on the 1994-2020 Sumner tide gauge record, which is expected to be representative of storm tide for the open coast. This water level includes the effect of the astronomical tide, storm surge and any medium-term sea level fluctuations.

3 Calculate wave effects (i.e. either set-up or run-up) for each timestep (1 hour) for the overlapping wave and water level timeseries (i.e. 1994-2019) and add to water level producing an extreme water level timeseries (i.e. either static or dynamic). As wave effects are dependent on wave height and beachface or surfzone slope, extreme water level timeseries have been created separately for the open coast from Waimakariri to Southshore, Sumner and Taylor's Mistake.

4 Undertake an extreme value analysis (EVA) to derive the 'structural' or combined extreme values based on the created timeseries. Analysis has been undertaken using a peaks-over-threshold method and a Weibull distribution which was found to represent wave-dominated extremes most accurately (Shand et al., 2010). The thresholds were selected to suit each individual area such that only extreme storms are included, with the EVA giving a reasonable fit through the data without the confidence intervals becoming too wide.

This approach provides a robust measure of the joint occurrence without requiring bivariate extreme value analysis which can introduce considerable additional uncertainty (Shand et al., 2012) with the dependence often biased by smaller events. Figure 7.5 shows an example of wave height (top panel) and water level (middle panel) timeseries, and the combined extreme water level timeseries (lower panel) for the Christchurch open coast. Figure 7.6 shows an example of an extreme value analysis on extreme static inundation levels for the Christchurch open coast.

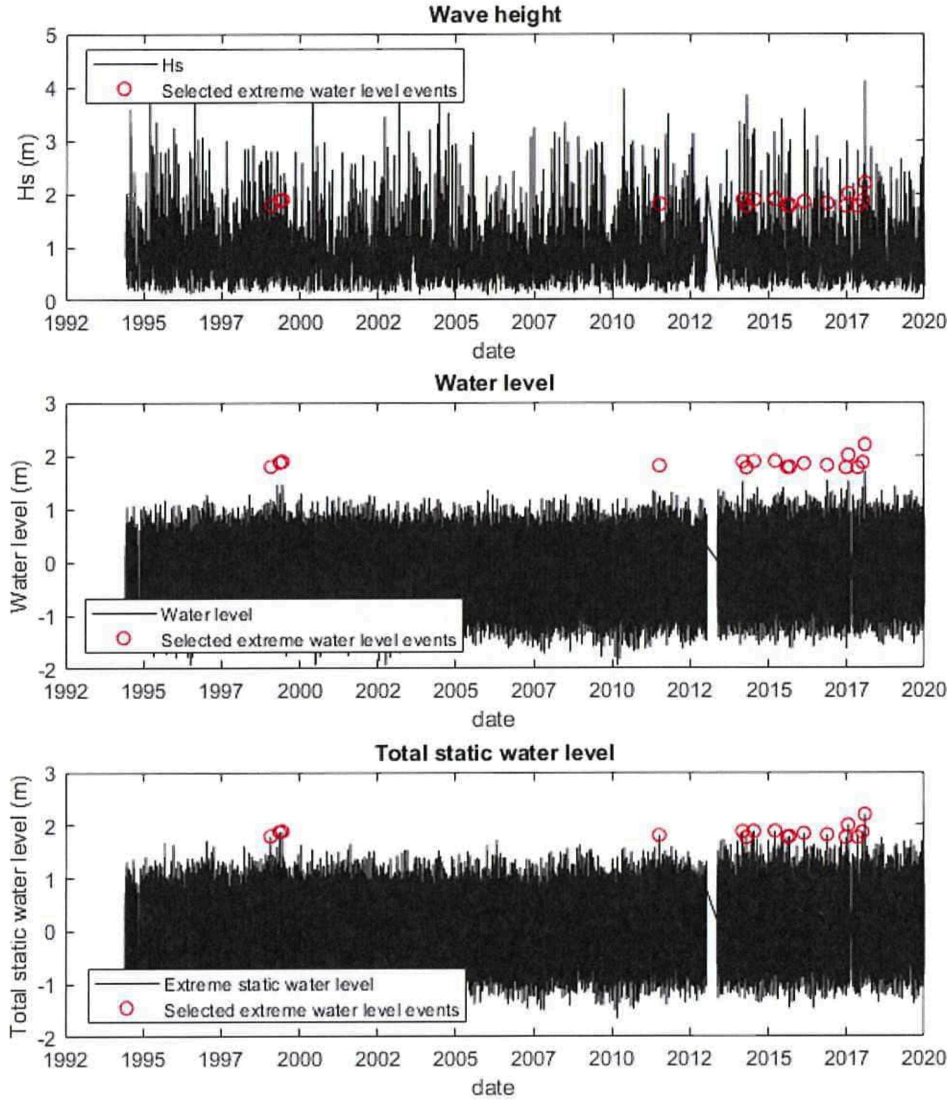

Figure 7.5 Example of extreme water level timeseries derived from a full wave height and water level timeseries.

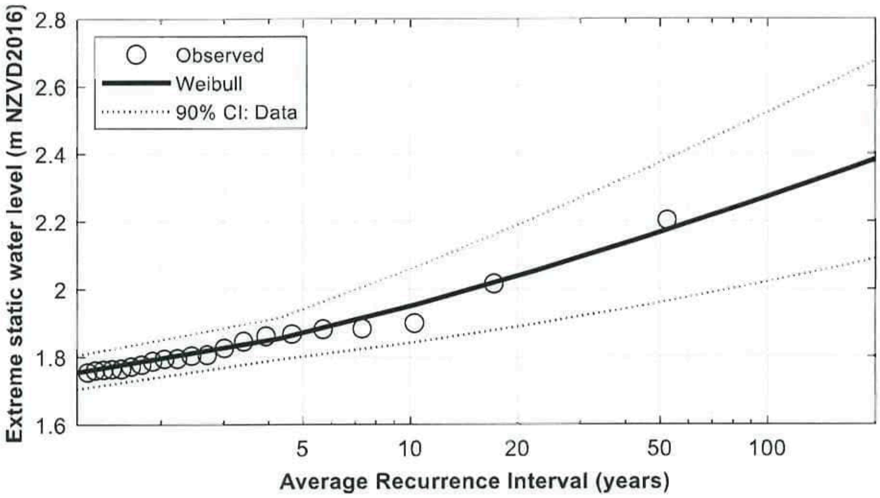

Figure 7.6 Example of extreme values analysis for static inundation level for the Christchurch open coast.

7.1.3 Attenuation of run-up

The Christchurch open coast shoreline is typically comprised of natural dunes. These dunes are typically high enough to limit wave run-up exceeding the dune crest, and therefore wave run-up extents have not been mapped (refer to Section 6.4). However, along roughly 450 m of shoreline at New Brighton and 170 m of shoreline at North New Brighton the dunes have been modified with seawalls built along these sections. Figure 7.8 shows the seawall at New Brighton. At these locations the run-up levels may differ from natural shoreline run up levels as a result of wave interaction with the structures, with waves overtopping the structures if not built high enough. Where the run up level exceeds the coastal edge (i.e. dune or seawall), it will overtop, but will be attenuated away from the coastal edge. This effect has been assessed based on the empirical formula by Cox and Machemehl (1986).

The formula to calculate the inland attenuation distance to zero water depth is shown in Equation 7.1. This formula has been modified from Cox and Machemehl (1986) who provide an equation to calculate the attenuation depth for a specified inland distance. A schematisation of run up attenuation is shown in Figure 7.7.

$$X = \frac{\sqrt{R-Y_0} \cdot A(1-2m) \cdot gT^2}{5\sqrt{gT^2}} \tag{7.1}$$

Where:

X	=	Wave run-up attenuation distance (m).
R	=	Wave run-up level including the storm tide (m RL).
Y_0	=	Dune crest elevation (m RL).
T	=	Wave period (s).
g	=	9.81 m/s^2.
A	=	Inland slope friction factor (default = 1, can be adjusted if calibration data available).
m	=	Positive upward inland slope valid for -0.5 < m < 0.25 (e.g. for 1(V):10(H), m = 0.1).

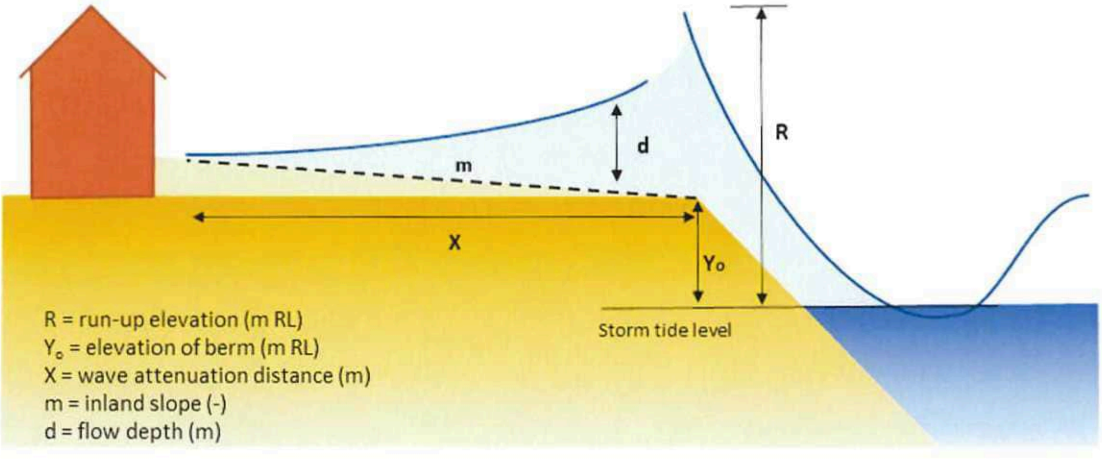

Figure 7.7: Run-up attenuation definition sketch (modified from Cox and Machemehl, 1986).

The attenuation of wave run up with distance inland is highly site-specific and is dependent on the run up elevation, crest level of the seawall or dune and backshore slope. Inland attenuation distances could therefore be calculated at high frequency intervals (e.g. 10 m) along the protected sections of the shoreline to account for the local changes in conditions/profile geometry. However, as shown in Figure 7.8 there are gaps in the seawall with waves running up through the gaps to behind the seawall as was the case during the July 2001 storm. Therefore, the calculated attenuation distances may not accurately represent the inland extent of wave run up.

Figure 7.8: New Brighton pier area during the July 2001 storms (source: Justin Cope, ECan).

Where the shoreline is alongshore uniform with no gaps in the dunes or the seawalls, the attenuation distance can be calculated with resulting distances as shown in Figure 7.9. The inland attenuation distance for 10 year ARI and 100 year ARI run up levels for the present-day (derived from Figure 7.7), and future timeframes allowing for 0.8 m and 1.5 m sea level rise have been graphed against the dune or seawall crest level. The lines shown in Figure 7.9 start at the respective static inundation level as static inundation would occur if this level exceeds the crest level. This shows that for a typical backshore level of 3 m NZVD2016 at North New Brighton and New Brighton that the inland attenuation distance could be in the order of 10 m for the present-day.

Figure 7.9 would therefore provide a useful indicator of run-up extents from the dune crest in addition to mapped static inundation extents.

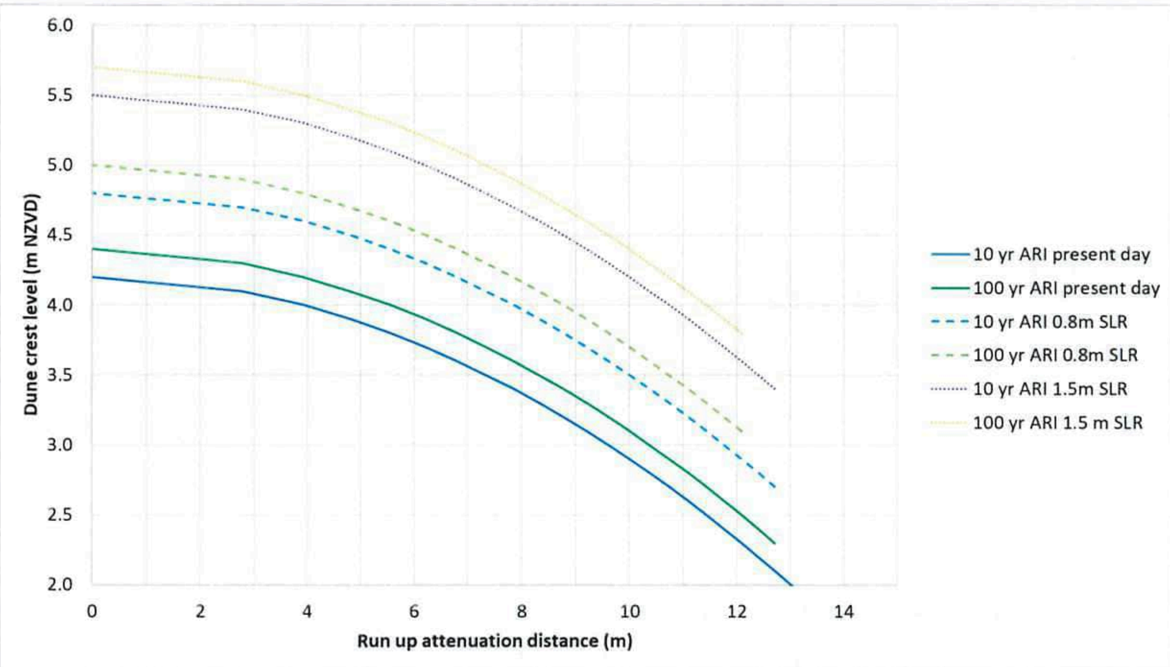

Figure 7.9: Run up attenuation distances from dune crest for a range of dune/seawall crest levels for a range of scenarios based on the modified Cox and Machemehl (1986) method.

7.1.4 Future erosion effects on static inundation

Future coastal change may affect the location and extents of static inundation and wave run up. In order to assess the effects of erosion on coastal inundation the numerical model XBeach NH (Deltares, 2015) has been used. A 100 year ARI joint-probability storm event including +1.5 m sea level rise has been run for the following profile geometries:

1 Original beach profile (C1065), assuming no beach response.
2 Profile maintaining original dune shape, retreated to predicted shoreline position at 2130.
3 Profile with dunes removed by erosion, retreated to predicted shoreline position at 2130.

The original beach profile at C1065 has been considered as the base scenario (1) and used to compare the results for the retreated shoreline scenarios (2 and 3) with. The original beach profile has been derived based on 2018-2019 LiDAR DEM supplemented by LINZ contour data offshore of the low tide contour. The retreated shorelines have been based on the original shoreline and have been shifted some 90 m landward, which is equal to the shoreline position at 2130, with a 5% likelihood of exceedance, considering +1.5 m sea level rise based on erosion hazard results. Scenario 2 assumes that the dunes roll back and maintain their current shape, scenario 3 assumes that when the shoreline retreats the dunes are eroded completely. Note that the classic Bruun rule suggest that the profile moves back and upward with sea level rise, which would mean that the dune crest would build up higher. As this would likely result in lower overtopping/inundation susceptibility compared to Scenario 2, it was assumed that the dune crest remains at its current level.

Figure 7.10 shows XBeach results for the three simulated scenarios. This shows that there is limited overtopping at the original profile (top panel), with similar limited overtopping occurring when the shoreline retreats -90 m landward while maintaining its original dune shape (middle panel). When the dunes are eroded completely, a 100 year ARI joint-probability storm event with 1.5 m sea level rise would result in static inundation (refer to Figure 7.10 - lower panel). This indicates that when dunes are able to maintain their shape (i.e. roll over landward), but not necessarily building up the crest level, the susceptibility to coastal inundation of the backshore remains similar when the dune retains its current position and geometry. However, when dunes are removed either due to erosion or anthropogenic interventions the backshore may become susceptible to static inundation as sea level rises.

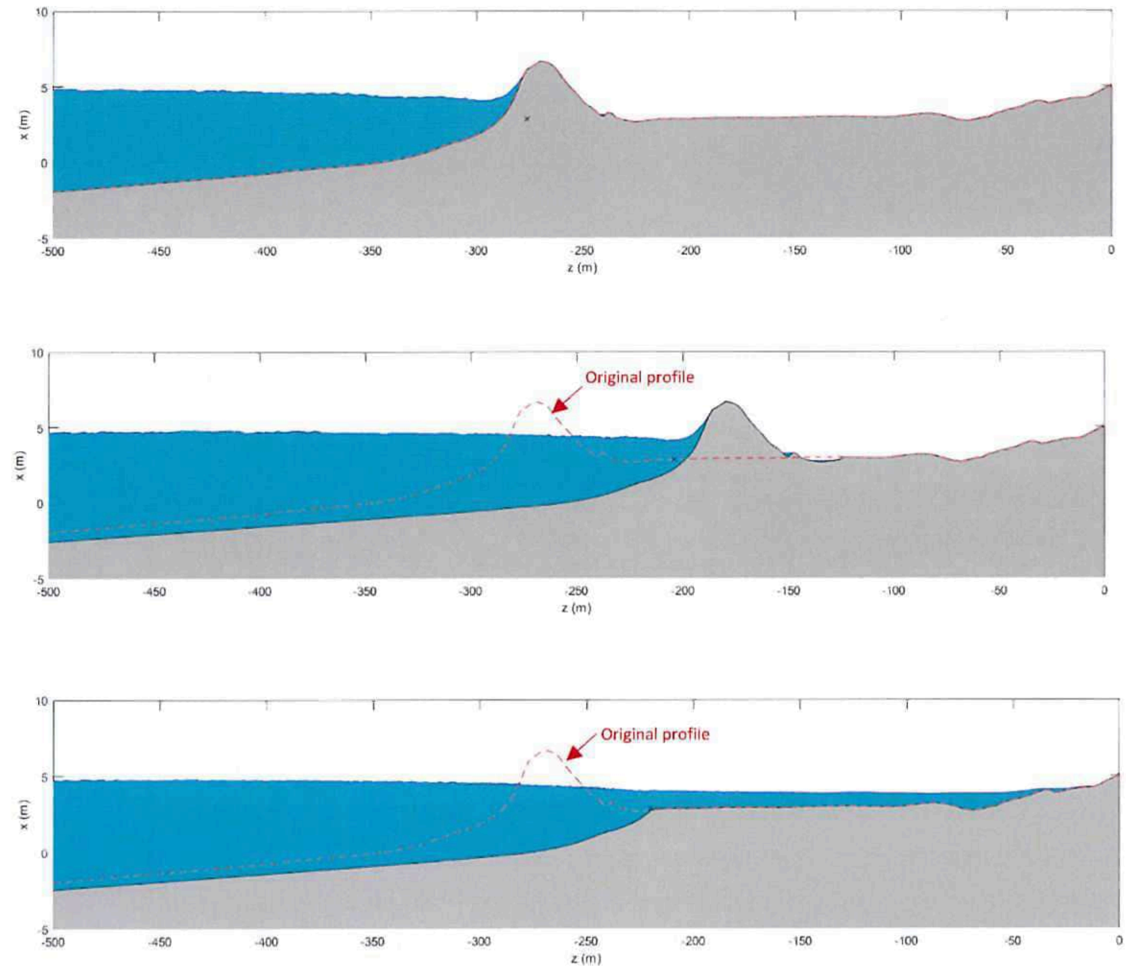

Figure 7.10: Xbeach results for original profile C1065 (1), retreated profile while maintaining dune shape (2) and retreated profile with dunes eroded (3).

It should be noted that the above assessed scenario 3 is an unlikely scenario (i.e. assuming the current dune management programme will be continued in the future), albeit it is reasonably similar to the shorelines at North New Brighton and New Brighton where seawalls have been built and dunes have been removed. Furthermore, where the future shoreline retreat is considerably less, the dune may only partly erode. A partly eroded dune may still provide protection against overtopping, however, the narrower the dune system the higher the likelihood of breaching during extreme storms becomes.

In order to assess whether eroded shorelines (assuming no dune roll over) have an effect on inundation, the P50% and P5% erosion lines at 2130 adopting 1 m of sea level rise have been mapped, with backshore elevations extracted. Figure 7.11 shows the extracted backshore elevations of the 2130 ASCE lines for both P50% and P5% adopting 1 m sea level rise compared with the 100 year ARI static inundation level plus 1 m sea level rise. Note that at chainage 15,000-18,000 the P50% ASCE line is situated seaward of the existing dunes as a result of long-term accretion, and therefore shows lower backshore levels.

Figure 7.11 shows that for the P50% ASCE line there is only a small section (~50 m wide near the North Beach surf club at CH11500) where the static inundation level exceeds the backshore level (by about 0.1 m), which would result in inundation in the vicinity of the surf club. For the P5% ASCE line, there is an approximately 1 km wide section (i.e. vicinity of seawalls at CH 11000 - 12000) where the static inundation level exceeds the backshore level by 0.5-0.7 m and would result in inundation of a large area behind the seawalls. In addition, at the northern end of the open coast shoreline adjacent to the Brooklands Lagoon, the 2130 ASCE P5% backshore levels are below the static inundation levels along two sections. This would likely result in inundation of the backshore along the Brooklands Lagoon. A map showing the inundation extents using the bathtub approach for the two scenarios is included in the interactive online map viewer (refer Section 1.3). This shows that future dune management may play a key role in mitigating future inundation hazard to the Christchurch open coast.

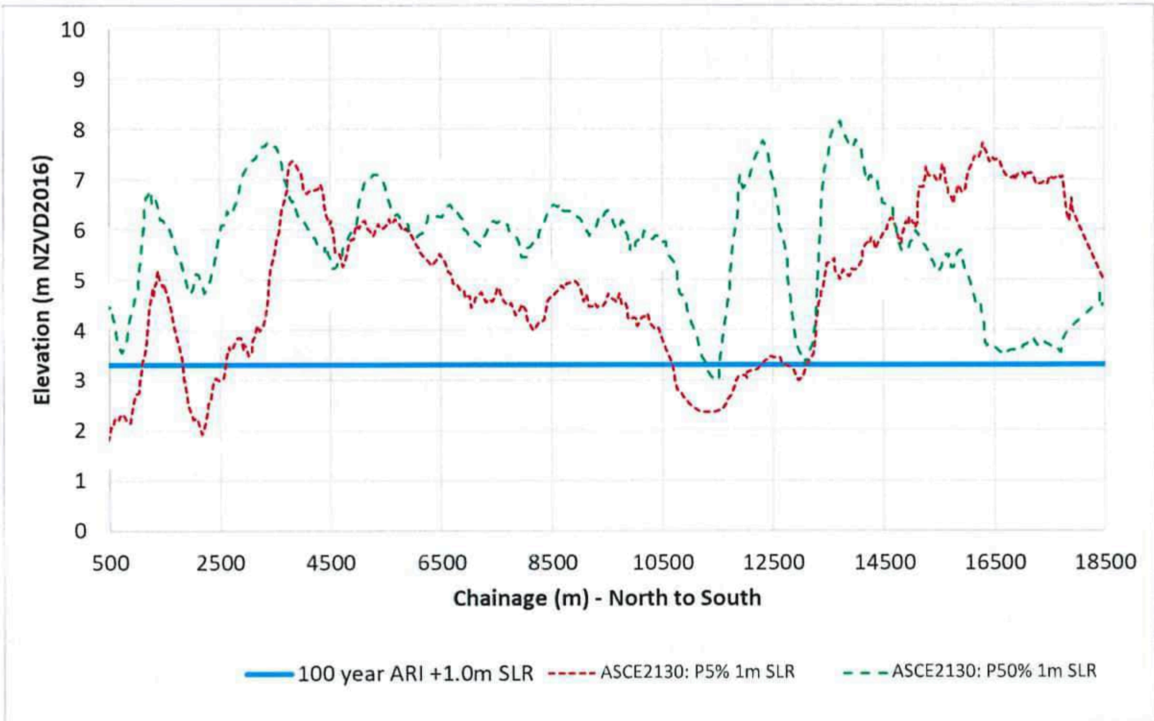

Figure 7.11: Alongshore backshore elevations at 2130 ASCE lines (both P50% and P5%) compared with 100 year ARI + 1 m SLR water level (chainage north to south)

7.1.5 Output locations

The static and dynamic inundation levels for the open coast depend on the water level timeseries, wave timeseries, and surfzone/beachface slope. As set out in Section 7.1.1 a single wave timeseries has been adopted for the open coast, including Sumner and Taylor's Mistake, and single surfzone/beachface slopes have been adopted separately for the open coast (from Waimakariri to Southshore), Sumner and Taylor's Mistake. Therefore, the following output locations have been adopted:

- Christchurch open coast from Waimakariri to Southshore.
- Sumner.
- Taylor's Mistake.

Figure 7.12 shows the extents of the Christchurch open coast sites/output locations.

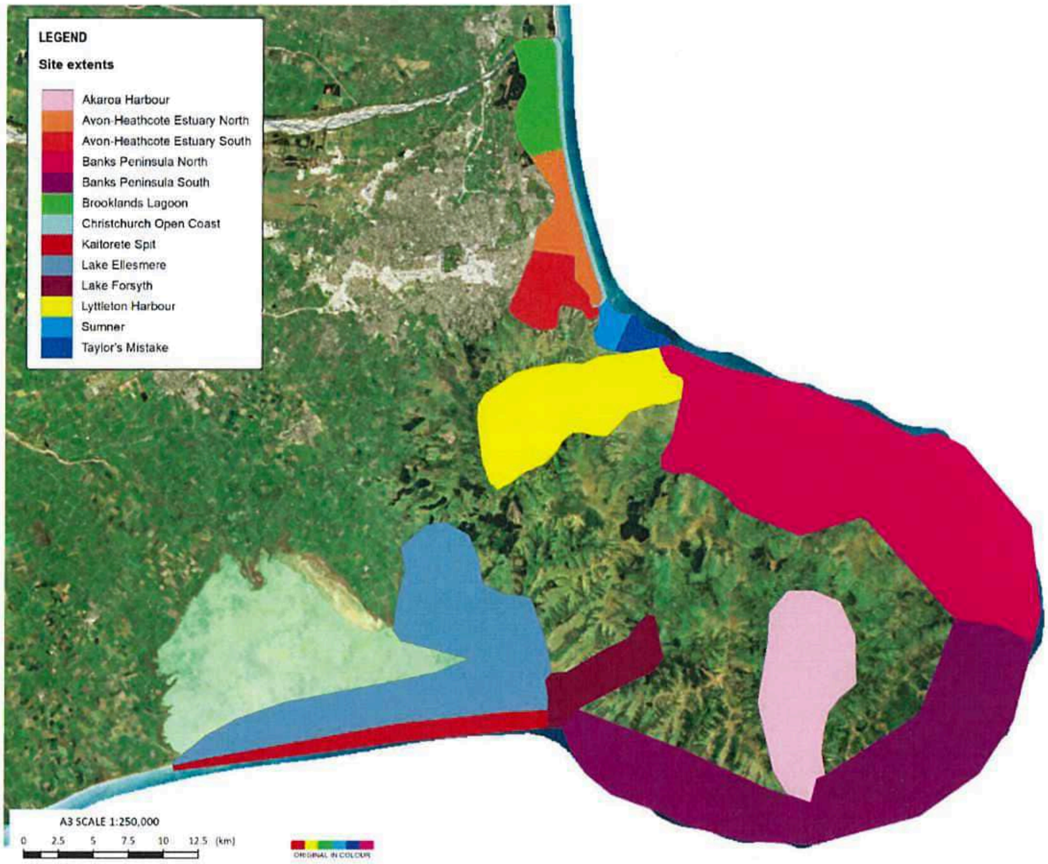

Figure 7.12: Site extents and location for Christchurch open coast, major harbours and estuaries and regional hazard screening sites.

7.2 Major harbours and estuaries

Major harbours and estuaries include the Brooklands Lagoon, Avon-Heathcote Estuary, Lyttelton Harbour and Akaroa Harbour, with inundation levels assessed quasi-probabilistically (refer to Section 6.2.1).

The major harbours and estuaries are typically exposed to open ocean swell that propagate through their entrances, with the largest swell in the vicinity of the entrance and reducing further into the harbours due to energy dissipation. The upper reaches of the harbours are more susceptible to local wind waves generated within the harbours. Storm surges could affect the entire shoreline within the harbours due to their large entrances and are more likely to coincide with large wind-generated waves when extreme storms move over the Christchurch region. Wave effects such as wave set-up and wave run-up could locally further elevate the water level along the shoreline within the harbours.

7.2.1 Input data

For Brooklands Lagoon, Avon-Heathcote Estuary and Lyttelton Harbour, extreme water levels are available as set out in GHD (2021). They analysed tide gauge records at Sumner, Bridge Street, Ferrymead and the Styx River, with resulting extreme water levels shown in Table 2.7. These recorded water levels are expected to implicitly include any river discharge and wind set-up effects.

No water level data is available from the GHD (2021) report for Akaroa Harbour. Therefore, water levels for the Akaroa Harbour have been based on water levels from GHD (2021) at the Lyttelton gauge, with an offset of the MHWS difference between the Lyttelton gauge (0.84 m NZVD2016) and Akaroa Harbour gauge (1.08 m NZVD2016) based on LINZ (2021). Table 7.2 shows the extreme water levels for the Akaroa Harbour. It should be noted that due to the difference in location, geometry and orientation of Lyttelton Harbour and Akaroa Harbour the exposure to storm surge may vary as well. NIWA (2015) suggest that the 100 year ARI storm tide levels at Birdlings Flat (southern side of Banks Peninsula) are approximately 0.1 m lower compared to Sumner (northern side of Banks Peninsula. However, as the 100 year ARI storm tide level at Lyttelton Port are in the order of 0.2 m lower than the 100 year ARI storm tide level at Sumner, it is reasonable to assume that storm surges within Lyttelton Harbour and Akaroa Harbour are similar.

Table 7.2: Extreme water levels (m NZVD2016) adjusted for Akaroa Harbour

	ARI							
Site	**1 yr**	**2 yr**	**5 yr**	**10 yr**	**20 yr**	**50 yr**	**100 yr**	**200 yr**
Akaroa Harbour	1.61	1.68	1.76	1.83	1.89	1.98	2.04	2.11

Wave timeseries data is not freely available, except for at the entrances derived from the MetOcean hindcast at the -10 m depth contours (refer to Table 2.8). However, in order to assess the wave effects within the harbours, numerical models have been set up to transform waves to the nearshore. SWAN models have been set up using the extreme wave heights as shown in Table 2.8, with separate runs undertaken including extreme wind speeds only as input based on ANZS1170.2 (2011) for a range of directions. An example of SWAN model results for the Lyttelton Harbour using the 100-year ARI easterly wind as input is shown in Figure 7.13. Appendix B includes more details on the SWAN models and example result maps for the three harbours.

The resulting typical significant wave heights extracted from the -2 m depth contour (inferred from SWAN model DEM) in the Lyttelton Harbour and Akaroa Harbour, and from the -1 m depth contour (inferred from the SWAN model DEM) in the Avon-Heathcote Estuary, are shown in Table 7.3. Note that these wave heights are typical ranges, with lower wave heights within smaller embayments, such as shown in Figure 7.13 for Lyttelton Harbour.

The largest waves within the harbours are typically locally generated by winds. Swell waves that propagate into the harbours are typically largest around the entrance and dissipate further up the harbours.

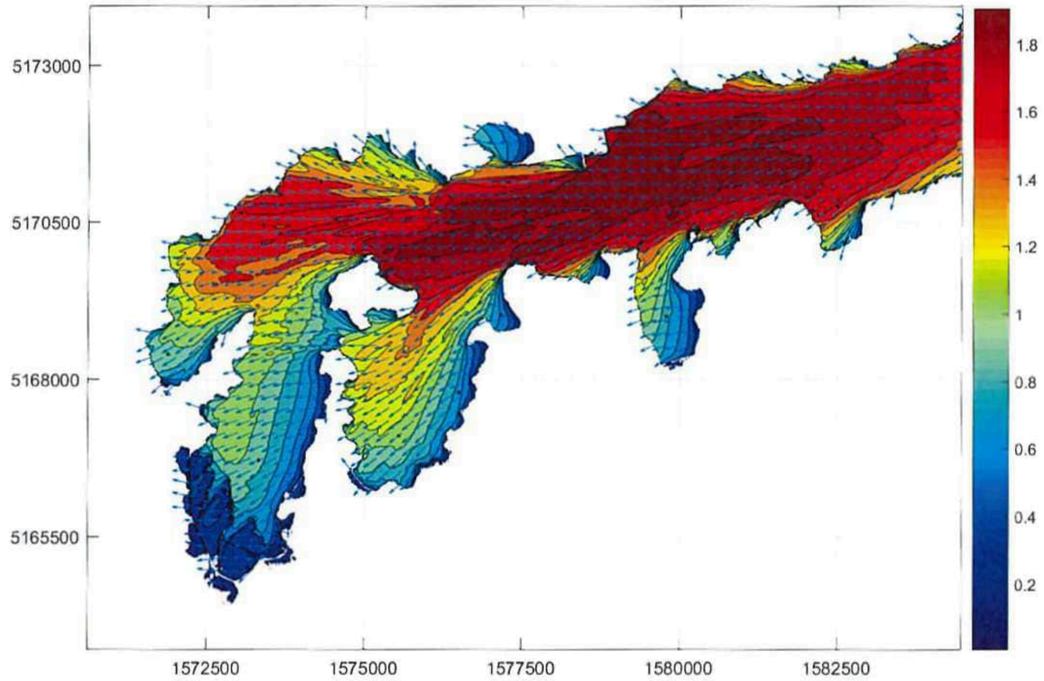

Figure 7.13: SWAN model results for Lyttelton Harbour using 100-year ARI easterly wind as input showing resulting significant wave height (H_s) in metres.

Table 7.3: Resulting typical significant wave heights (range in metres) from SWAN model results

Return period	Avon-Heathcote	Akaroa Harbour	Lyttelton Harbour
1 year ARI	0.3-0.5	0.5-0.9	0.5-1
10 year ARI	0.5-0.6	0.6-1.1	0.7-1.2
100 year ARI	0.6-0.8	1.0-1.5	1.0-1.5

7.2.2 Analysis of extreme static water levels

Extreme static water levels for the major harbours and estuaries have been assessed by summing the storm tide levels (refer to Table 2.7 and Table 7.2) and the wave set up component.

Due to the limited bathymetry data for the Lyttelton Harbour and Akaroa Harbour it is challenging to accurately derive beach or surfzone slopes, which are required for most empirical wave set up formulas. Therefore, the empirical formula by Guza and Thornton (1981) has been used, which is a function of the offshore wave height only:

$$\bar{\eta} = 0.17 \cdot H_s \tag{7.2}$$

Note that bathymetry information is available for the Avon-Heathcote Estuary, however, for consistency a single formula has been adopted for the major harbours and estuaries.

The resulting upper bound wave heights derived from the SWAN model results as set out in Table 7.3 have been used to calculate wave set-up. Table 7.4 shows the resulting wave set-up values for the 100 year ARI storms that have been adopted. It should be noted that some parts of sheltered embayments within the major harbours wave set-up may be less. However, for the purpose of this study (i.e. climate change adaptation planning or other similar assessments) these values have been applied for the entire harbours.

Note that for the Brooklands Lagoon the water depth is too shallow to run a SWAN model, with wave effects assumed to be smaller than 0.1 m. Therefore, no wave set-up has been added to the extreme water levels for Brooklands Lagoon.

Table 7.4: Resulting wave set-up values (m) for major harbours

Avon-Heathcote	Akaroa Harbour	Lyttelton Harbour
0.15	0.25	0.25

7.2.3 Output locations

Extreme static water levels across the major harbours and estuaries have been reviewed to determine the number of output locations. Based on the available information and analysis set out in the previous sections, output locations have been adopted for:

- Brooklands Lagoon.
- Avon-Heathcote Estuary:
  - North (i.e. near Avon).
  - South (i.e. near Heathcote).
- Lyttelton Harbour.
- Akaroa Harbour.

As tide gauges at Bridge St (Avon River) and Ferrymead St (Heathcote River) have been analysed separately with slightly different resulting extreme water levels, the Avon-Heathcote has been split up in two. The wave set-up component is similar for both side of the estuary depending on the wind direction. For both Lyttelton Harbour and Akaroa Harbour a single output point has been adopted as the majority of these harbours are affected by wave set-up induced by local wind waves, with swell wave typically smaller or similar. The entrances of Lyttelton Harbour and Akaroa Harbour have been excluded as they are more susceptible to swell. The entrances have been included in the Banks Peninsula output locations which are susceptible to swell waves (refer to Section 7.3). Figure 7.12 shows the extents of the major harbours and estuary sites/output locations.

7.3 Regional hazard screening sites

Regional hazard screening sites include the Outer Banks Peninsula, Kaitorete Spit, Te Waihora (Lake Ellesmere), Wairewa (Lake Forsyth), with inundation levels assessed deterministically (refer to Section 6.2.2).

The Banks Peninsula and Kaitorete Spit are both susceptible to storm surges and open ocean swell. The north side of the peninsula is susceptible to swell and storms from the north-east to east, with the south side of the peninsula including the Kaitorete Spit susceptible to swell and storms from the east to south-east. As the Banks Peninsula is typically comprised of sea cliffs, the majority of the shoreline may not be susceptible to coastal inundation. However, the low-lying embankments situated between the cliffs may be susceptible to coastal inundation.

Both Wairewa (Lake Forsyth) and Te Waihora (Lake Ellesmere) are lakes that are mostly closed off from the sea and are manually opened to drain water into the sea when consented trigger levels are reached. The lake levels are mainly affected by catchment inflows and are not affected by tides, surges and swell waves. The levels within the lakes can be further elevated by effects of locally wind generated waves.

7.3.1 Input data

Tide gauge record lake level timeseries are available in Wairewa (Lake Forsyth) and Te Waihora (Lake Ellesmere). However, no water level data is available for the Outer Banks Peninsula and Kaitorete Spit. Therefore, water levels for the Outer Banks Peninsula and Kaitorete Spit have been based on water levels from GHD (2021) analysis at the Sumner gauge, with an offset applied of the MHWS difference between the output locations. The MHWS differences between Sumner (0.89 m MSL) and Banks Peninsula North (0.89 m MSL), Banks Peninsula South (0.89 m MSL) and Kaitorete Spit (0.89 m MSL) have been based on NIWA (2015). This shows that there is no difference in MHWS between Sumner and the northern and southern side of the Banks Peninsula and Kaitorete Spit.

The extreme lake levels at Wairewa (Lake Forsyth) and Te Waihora (Lake Ellesmere) have been assessed by undertaking an extreme value analysis of the lake level gauge records. The lake level record implicitly includes catchment inflow effects, wind set-up effects and effects of periodically opening the mouth. The assessed extreme lake levels are shown in Table 7.5. Note that the lake levels in Wairewa (Lake Forsyth) are significantly higher and the lake levels in Te Waihora (Lake Ellesmere) are slightly lower than the open coast extreme levels, which is a result of being closed off from the sea, being opened when trigger levels are reached and not being affected by storm surges. These extreme levels are applicable while the current mouth opening management is in place, but may vary if the current management and trigger levels change in the future. Note that the lake levels are unlikely affected by sea level rise (for the range of scenarios considered in this assessment) as the lakes are typically closed from the sea, therefore, sea level rise will not be added to the extreme levels.

Table 7.5: Extreme water levels (m NZVD2016) for regional hazard screening sites

	ARI							
Site	**1 yr**	**2 yr**	**5 yr**	**10 yr**	**20 yr**	**50 yr**	**100 yr**	**200 yr**
Banks Peninsula – North/South & Kaitorete Spit[1]	1.37	1.44	1.52	1.59	1.65	1.74	1.8	1.87
Wairewa (Lake Forsyth)[2]	2.18	2.33	2.48	2.57	2.66	2.76	2.84	2.91
Te Waihora (Lake Ellesmere)[2]	1.04	1.1	1.21	1.29	1.38	1.5	1.6	1.69

[1] Source: GHD (2021) including offset based on MHWS difference from NIWA (2015).
[2] Source: Tide gauge extreme value analysis.

Wave data is available at the Lyttelton Harbour Entrance, Akaroa Harbour Entrance and at the Kaitorete Spit derived from the MetOcean hindcast at the -10 m depth contours. The wave data at the Lyttelton Harbour entrance has been assumed to be applicable to the northern side of the Banks Peninsula, with the wave data at the Akaroa Harbour entrance to be applicable to the southern side of the Banks Peninsula, both due to similar wave exposure. Extreme value analyses have been undertaken on the wave timeseries, with resulting extreme wave heights shown in Table 2.8.

7.3.2 Analysis of extreme water levels

The extreme static water levels for the open coast regional hazard screening sites (excluding the lakes) have been assessed by summing the storm tide levels and wave set up component. Wave set up has been assessed using the USACE (2006) empirical formula in line with the open coast approach for consistency. LINZ depth contours (i.e. 0 m, -2 m, -5 m and -10 m contours) have been used to assess the surfzone slopes for the Banks Peninsula (for sandy embayments) and Kaitorete Spit as this is the only available data source. A consistent surfzone slope of 1(V):65(H) was found for both the Banks Peninsula and Kaitorete Spit. The resulting wave set-up values are shown in Table 7.6, with the large set-up values being a result of the large offshore wave heights (refer to Table 2.8).

Table 7.6: Resulting wave set-up values (m) for regional hazard screening sites

Return period	Banks Peninsula – North	Banks Peninsula – South	Kaitorete Spit
1 year ARI	0.84	1.54	1.24
10 year ARI	0.96	1.84	1.35
100 year ARI	1.02	2.08	1.5

For Wairewa (Lake Forsyth) and Te Waihora (Lake Ellesmere) wave set up has been assessed using the Guza and Thornton (1981) formula, in line with the approach for the major harbours and estuaries. The wave heights have been derived using the fetch-limited based on Goda (2003) using extreme wind speeds from ANZS1170.2 (2011). Due to the shallow water depths within the lake the resulting wave heights are less than 1 m. The resulting wave set-up values for Te Waihora (Lake Ellesmere) and Wairewa (Lake Forsyth) is 0.1 m as a result of the shallow water depths.

7.3.3 Output locations

As water level and wave data along the Banks Peninsula and Kaitorete Spit is only available at discrete locations, the following output locations have been adopted:

- Banks Peninsula – North.
- Banks Peninsula – South.
- Wairewa (Lake Forsyth).
- Te Waihora (Lake Ellesmere).
- Kaitorete Spit.

Figure 7.12 shows the extents of the regional hazard screening sites/output locations.

8 Coastal inundation results

8.1 Christchurch open coast

The resulting present-day static inundation levels for the Christchurch open coast from Waimakariri to Southshore, including Sumner and Taylors Mistake are shown in Table 8.1. Future static inundation levels including selected, relative sea level rise increments and dynamic inundation levels are shown in Appendix D. Future static inundation extents for selected sea level rise scenarios are shown in Appendix E.

Table 8.1: Static inundation levels (m NZVD2016) for Christchurch open coast, including Sumner and Taylors Mistake

Return period	Christchurch open coast	Sumner	Taylors Mistake
1 year ARI	1.8	1.8	1.8
10 year ARI	2.0	2.0	2.0
100 year ARI	2.3	2.3	2.3
100 year ARI +0.4 m SLR	2.7	2.7	2.7
100 year ARI +1.5 m SLR	3.8	3.8	3.8

The resulting static inundation levels for the Christchurch open coast from Waimakariri to Southshore, Sumner and Taylor's Mistake are the same and range from 1.8 to 2.3 m NZVD2016 for 1 to 100 year return period. This is a result of using the same extreme storm tide levels with wave set-up for the different surfzone slopes having a minor effect (i.e. <0.1 m). These present-day levels will increase in the future with sea level rise as included in Table 8.1 for selected sea level rise increments. Appendix D shows present-day and future static inundation levels for a larger number of selected sea level rise increments.

Appendix E shows the static inundation extents for the 1, 10 and 100 year ARI static inundation levels allowing for 0.4 m and 1.5 m sea level rise. The 0.4 m and 1.5 m sea level rise scenarios have been considered for presentation of results as these bracket the upper and lower range at 2130 for the sea level rise scenarios recommended by MfE for adaptation planning. Figure 8.1 shows an example static inundation map for the open coast. Overview maps which show the variation in erosion distances across the district are provided in Appendix E. Inundation depth results for the full suite of sea level rise and sensitivity scenarios are available on the website viewer (refer Section 1.3).

The maps in Appendix E show that the Christchurch open coast from Waimakariri to Southshore is not subject to static inundation under both the 0.4 m and 1.5 m sea level rise scenarios where the dunes have not been modified. However, where the dunes are modified (i.e. Brighton Pier and Surf Livesaving Club) the bathtub modelling indicates that the backshore may be subject to static inundation. Under the 0.4 m sea level rise scenario the extents are relatively small, however, under the 1.5 m sea level rise scenario a larger backshore area may be susceptible to coastal inundation.

Both Sumner and Taylor's Mistake are susceptible to static inundation under the 0.4 m sea level rise scenario with the extent depending on the return period storm. Under the 1.5 m sea level rise scenario the majority of the townships are susceptible to static inundation.

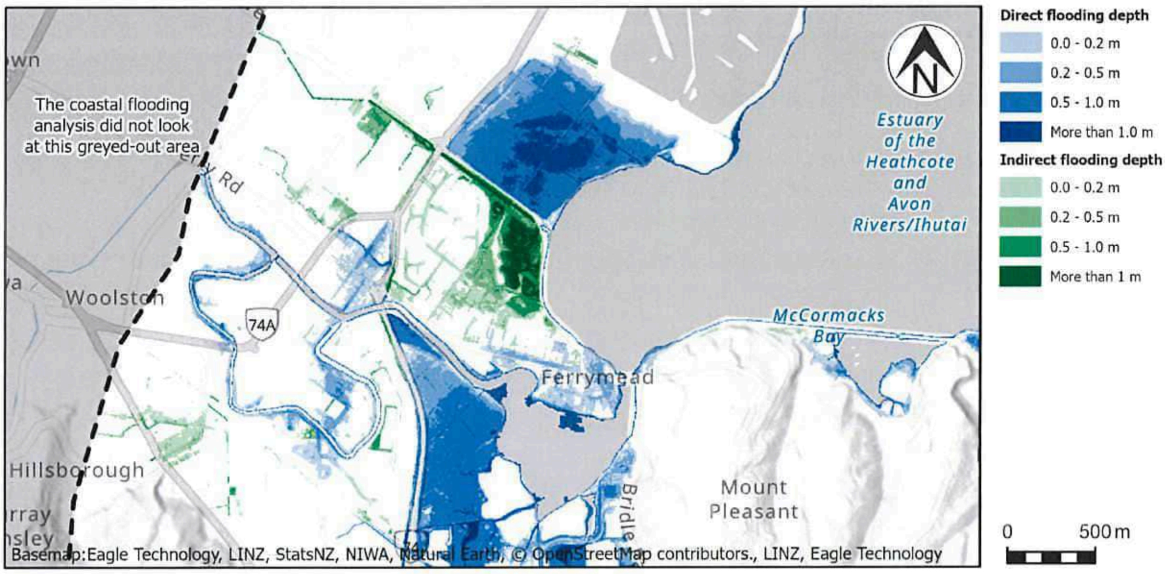

Figure 8.1: Example of static inundation depths for 100 year ARI water levels with for current-day sea level including areas connected to the shoreline (blue shading) and separate inundation areas that are not connected to the coast (green shading).

A map showing the inundation extents for future eroded shorelines is included in the online map viewer (refer Section 1.3). This shows that if the shoreline erodes to the 2130 ASCE P5% with 1 m of sea level rise then a 100 year ARI storm event may be able to break through the flattened dunes at North Beach and New Brighton, increasing the depth and extent of flooding from Waimairi Beach to South New Brighton. This indicates that future dune management may play a key role in mitigating future inundation hazard to the Christchurch open coast.

8.2 Major harbours and estuaries

The resulting present-day static inundation levels for the major harbours and estuaries are shown in Table 8.2. Future static inundation levels including selected, relative sea level rise increments are shown in Appendix D. Future static inundation extents for selected sea level rise scenarios are shown in Appendix E.

Table 8.2: Static inundation levels (m NZVD2016) for major harbours and estuaries

Return period	Brooklands Lagoon	Avon-Heathcote North	Avon-Heathcote South	Lyttelton Harbour	Akaroa Harbour
1 year ARI	1.4	1.5	1.5	1.6	1.9
10 year ARI	1.6	1.7	1.6	1.7	2.1
100 year ARI	1.8	2.0	1.8	1.8	2.3
100 year ARI +0.4 m SLR	2.2	2.4	2.2	2.2	2.7
100 year ARI +1.5 m SLR	3.3	3.5	3.3	3.3	3.8

Table 8.2 shows similar static inundation levels within the Brooklands Lagoon, Avon-Heathcote Estuary and Lyttelton Harbour ranging from 1.4 to 2.0 m NZVD2016 for present-day static inundation levels. This is a result of various factors influencing the water levels, such as exposure to waves, river discharge effects, wind set-up effects or exposure to storm surges. The static inundation levels within the Akaroa Harbour are 0.3-0.5 m higher compared to the other harbours, which is a result of

the MHWS being in the order of 0.3 m higher compared to Lyttelton or Sumner. The larger 100 year ARI water level at the northern side of the Avon-Heathcote Estuary compared to the southern side is a result of the higher water level analysed by GHD (2021) which is potentially affected by river discharges or wind set-up from a more dominant southerly wind. These present-day static inundation levels will increase with sea level rise, with future 100 year ARI static inundation for 0.4 m and 1.5 m sea level rise shown in Table 8.2.

Appendix E shows the static inundation extents for the major harbours and estuaries for the 0.4 m and 1.5 m sea level rise scenarios. Figure 8.2 shows an example static inundation map for Lyttelton Harbour. The static inundation extents under the 0.4 m sea level rise scenario within the Lyttelton Harbour are typically limited to the coastal edge, except for along the low-lying embayments at the southern side of the harbour, such as Teddington (see Figure 8.2). The extents of the areas susceptible to static inundation along the southern embayments increase under the 1.5 m sea level rise scenario, where the extents along the remaining, typically cliff shoreline, do not significantly increase. Note that parts of the Lyttelton Port may potentially be susceptible to static inundation under the 1.5 m sea level rise scenario.

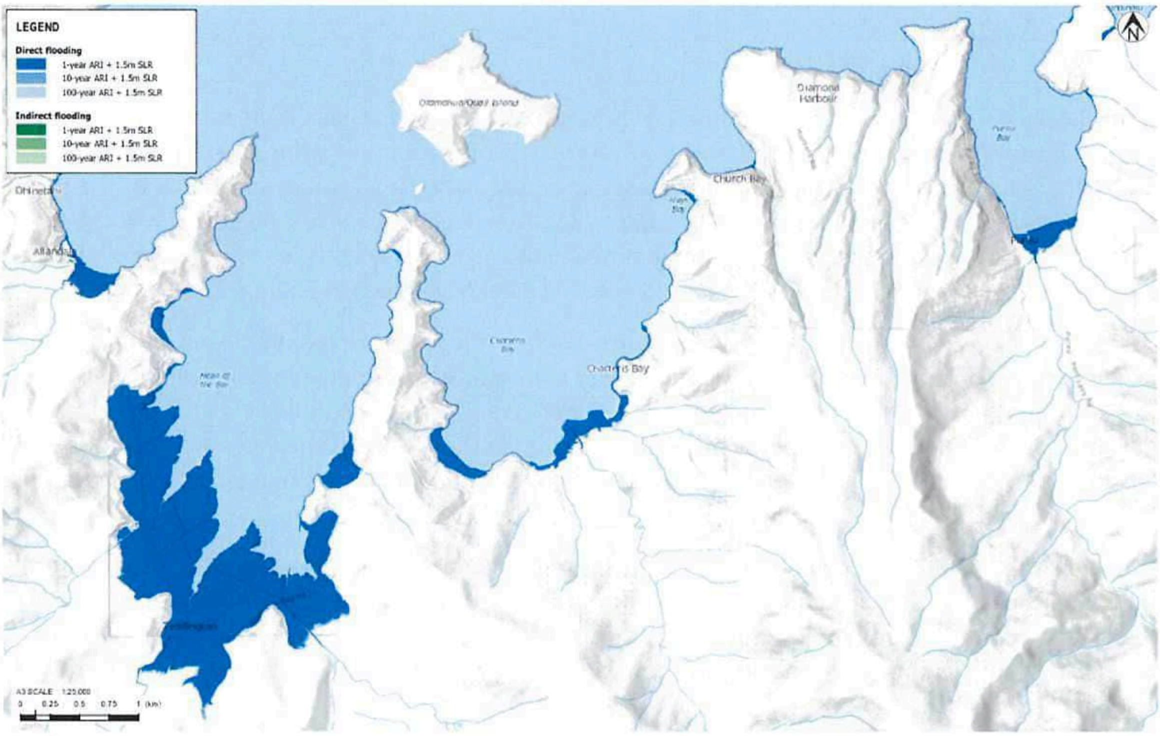

Figure 8.2: Example of static inundation extent map for Lyttelton Harbour (refer Appendix E for full map).

In the vicinity of Brooklands Lagoon, for the 0.4 m and 1.5 m sea level rise scenarios a large area is susceptible to static inundation. As the topography surrounding the Brooklands Lagoon is low-lying this is expected to occur. The static inundation within the Avon-Heathcote Estuary is typically within a few hundred metres of both the Avon and Heathcote rivers under the 0.4 m sea level rise scenario. This means that low-lying areas surrounding the estuary may already be susceptible under low sea level rise scenarios. For the 1.5 m sea level rise scenario large areas surrounding the Avon and Heathcote rivers are susceptible to static inundation.

Static inundation is typically limited to the low-lying embayments (e.g. Duvauchelle, Barrys Bay, Takamatua and Akaroa) within the Akaroa Harbour for both sea level rise scenarios. The remaining shoreline is typically comprised of cliffs, with inundation extents limited to the coastal edge.

8.3 Regional hazard screening sites

The resulting present-day static inundation levels for the regional hazard screening sites are shown in Table 8.3. Future static inundation levels including selected, relative sea level rise increments are shown in Appendix D. Future static inundation extents for selected sea level rise scenarios are shown in Appendix E.

Table 8.3: Static inundation levels (m NZVD2016) for regional hazard screening sites

Return period	Banks Peninsula North	Banks Peninsula South	Wairewa (Lake Forsyth)	Kaitorete Spit	Te Waihora (Lake Ellesmere)
1 year ARI	2.2	2.9	2.2	2.6	1.1
10 year ARI	2.5	3.4	2.6	2.9	1.4
100 year ARI	2.8	3.9	2.8	3.3	1.7
100 year ARI +0.4 m SLR	3.2	3.3	N/A	3.7	N/A
100 year ARI +1.5 m SLR	4.3	4.4	N/A	4.8	N/A

Table 8.3 shows that the static inundation levels vary considerably for each regional hazard screening site. The static water levels at the southern side of the Banks Peninsula are the largest as result of the highest wave set-up due to highest extreme wave heights (refer to Table 2.8). The static water levels at Kaitorete Spit and northern side of Banks Peninsula are lower due the lower extreme wave heights. These present-day static inundation levels will increase with sea level rise, with future 100 year ARI static inundation for 0.4 m and 1.5 m sea level rise shown in Table 8.3.

The extreme lake levels are not affected by storm surge as the lakes are mainly closed and resulting lake levels are controlled by catchment inflows and management of opening the lake mouth. Lake levels within Wairewa (Lake Forsyth) and Te Waihora (Lake Ellesmere) are unlikely affected by sea level rise (for the range of sea level scenarios considered in this study) as the lakes are typically closed from the sea. Therefore, sea level rise has not been added to the extreme levels (indicated with N/A in Table 8.3).

Appendix E shows the static inundation extents for the regional hazard screening sites for the 0.4 m and 1.5 m sea level rise scenarios. Figure 8.3 shows an example static inundation map for Banks Peninsula. The maps in Appendix E show that for the 0.4 m and 1.5 m sea level rise scenarios along the Banks Peninsula that low-lying embayments are susceptible to static inundation. The majority of the Banks Peninsula is comprised of sea cliffs with inundation extents limited to the coastal edge (i.e. cliff toe). The static inundation extents along the Kaitorete Spit are limited to the gravel barrier toe due to the elevated levels of gravel barrier crest.

As sea level rise has not been added to the extreme lake levels at both Te Waihora (Lake Ellesmere) and Wairewa (Lake Forsyth), the inundation extents shown in Appendix E represent the present-day scenario. The map in Appendix E shows that the majority of the lakeshore of Te Waihora (Lake Ellesmere) is susceptible to inundation for a 100 year ARI lake level, with only the upper reaches of the lakeshore of Wairewa (Lake Forsyth) susceptible to inundation for a 100 year ARI lake level.

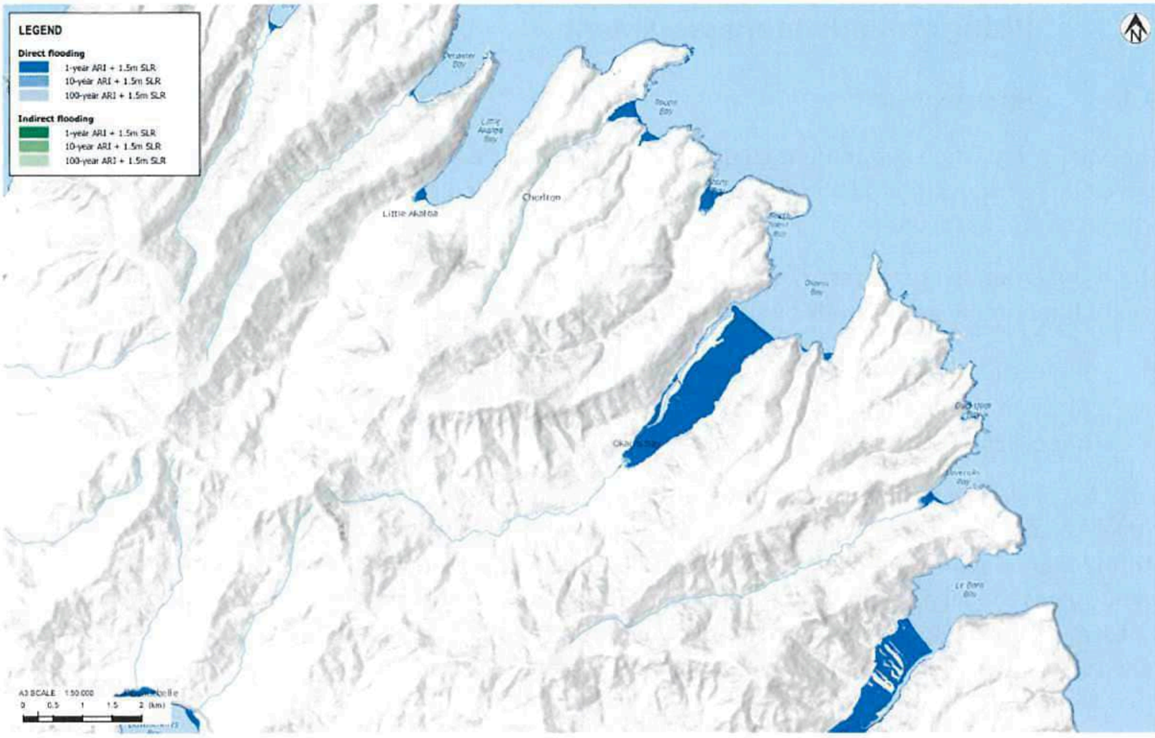

Figure 8.3: Example of static inundation extent map for Banks Peninsula (refer Appendix E for full map).

9 Rising groundwater assessment

9.1 Background

The Ministry for the Environment guidance (MfE, 2017) notes that climate change and sea level rise can result in rising groundwater levels in coastal lowlands, and this should be considered as part of a coastal hazard assessment.

The rising groundwater assessment undertaken as part of the current coastal hazard study relates to two of the primary groundwater issues which may be exacerbated by sea level rise:

- Inundation due to groundwater ponding (either temporary or permanent).
- A rise in the groundwater table level (which can impact buildings, infrastructure and how people can use the land).

MfE (2017) also identifies various other groundwater-related issues which may be exacerbated by climate change, such as salinisation, change in habitat, reduced hydraulic gradient, reduced stormwater infiltration and increased potential for earthquake-induced liquefaction. These issues and other secondary effects are beyond the scope of the current assessment. However, this assessment may help to identify locations where further efforts could be focussed in future if required to help inform adaption planning in particular areas.

It is emphasised that the groundwater models presented below and in Appendix Eare not intended to precisely predict groundwater levels on a local scale at a specific location or time. The models are instead intended to help inform adaptation planning by identifying at a region-wide scale general locations which are more likely to be affected by rising groundwater issues exacerbated by sea level rise. These models are not sufficiently detailed to identify individual property risks and more detailed assessment would be required to assess any property-level impacts.

9.2 Christchurch urban flat-land area

Aqualinc (2020) presents a model of current-day groundwater levels across the Christchurch urban flat-land area, and a high-level assessment of the potential magnitude and impacts of future changes in groundwater level due to climate change.

This assessment was undertaken as part of the Council's multi hazard study to inform floodplain management. It updates the previous regional shallow groundwater model for Christchurch (van Ballegooy et al. 2014), looks at trigger levels of when shallow groundwater becomes a problem for people and infrastructure, and provides information on the impacts of sea level rise and earthquake subsidence on groundwater levels. As noted in the report: *the purpose was not to accurately define the shallow groundwater hazard at a local scale, but rather to provide a high-level assessment at the city-wide scale.*

This existing information provides a detailed hazard assessment, and has already been accepted by CCC as sufficient to inform the current stages of adaptation planning. Therefore, no further assessment of groundwater levels in the Christchurch urban flat-land area has been undertaken as part of the current coastal hazard assessment. The groundwater model results from Aqualinc (2020) have simply been re-plotted onto the maps presented in Appendix E.

9.3 Banks Peninsula

As Banks Peninsula is outside the extent of the existing Aqualinc (2020) groundwater study, a regional rising groundwater hazard screening assessment was undertaken as part of the current coastal hazard assessment, to identify areas of low-lying land close to the coast around the peninsula.

In these low-lying coastal margins there is generally a relationship between groundwater level and sea level. Areas where the land level is only slightly above high tide level (or below it) are more likely to experience flooding or wet ground caused by high groundwater, and sea level rise could cause groundwater to become higher in these areas.

The screening assessment assumed that for land which is low-lying (below about RL 5 m NZVD 2016) and close to the coast (within about 5km) the 85th percentile groundwater level[3] is approximately equal to the high tide level. This approximation was developed based on data from 30 groundwater monitoring wells in low-lying areas close to the coast from Waimairi Beach to Southshore. The 85th percentile water level for all these monitoring wells was within ±0.3m of MHWS high tide level, with an average of 0.1m below MHWS.

A nominal MHWS high tide level of 0.8 m NZVD 2016 was adopted around all of Banks Peninsula, except for Akaroa Harbour where a nominal level of 1.1 m NZVD 2016 was assumed (refer Section 2.4.1). A rise in sea level was assumed to cause an equal rise in groundwater level in the coastal areas of interest (it is acknowledged that the sea level influence on groundwater level will dissipate with distance further inland from the coast). By comparing this groundwater level to the land level, a modelled depth to the 85th percentile groundwater level was derived. This is illustrated conceptually in Figure 9.1.

To provide an approximate sense-check of this simplified model, these screening assumptions were modelled across the Christchurch urban flat-land area and the results compared to the Aqualinc (2020) detailed hazard model. This comparison showed that for low-lying land close to the coast the screening model was generally identifying a broadly similar extent of rising groundwater hazard as the detailed model for current day and future sea level scenarios, when viewed at the broad regional scale which is relevant for initial hazard screening and adaptation planning.

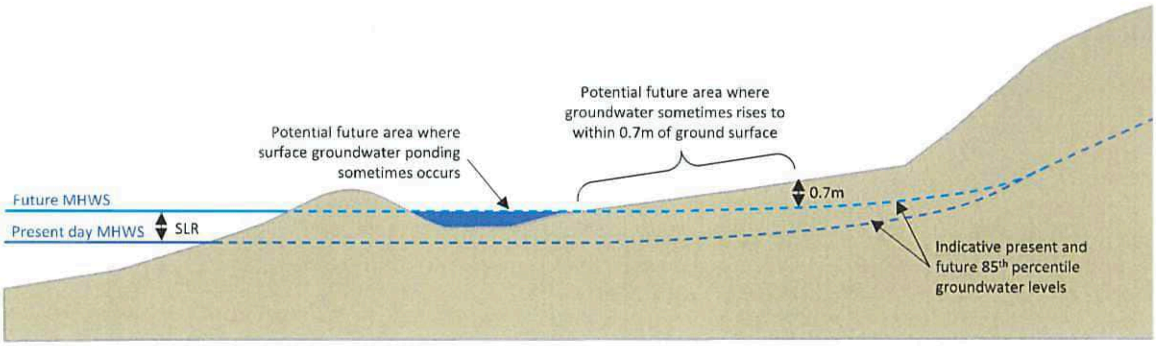

Figure 9.1: Conceptual model for indicative present-day and future groundwater levels for low-lying areas close to the coast around Banks Peninsula.

The results of the regional rising groundwater screening assessment are presented in Appendix E, with the mapped areas split into two categories to align with two of the key impact trigger levels identified in Aqualinc (2020):

- Projected groundwater levels sometimes rise up to or above the ground surface (e.g. surface ponding or increased land drainage demands).
- Projected groundwater levels sometimes rise to within 0.7 m of the ground surface (e.g. wet/soft ground underfoot or affecting buildings and infrastructure).

[3] The groundwater table is expected to sit below this level for 85% of the time (on average).

10 Applicability

This report has been prepared for the exclusive use of our client Christchurch City Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

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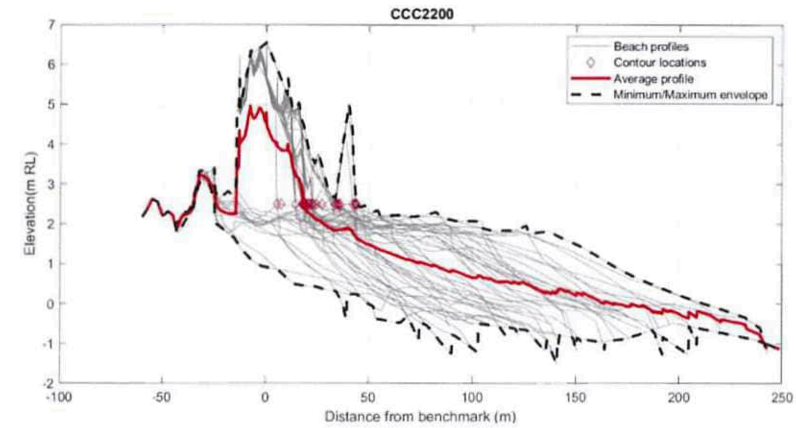
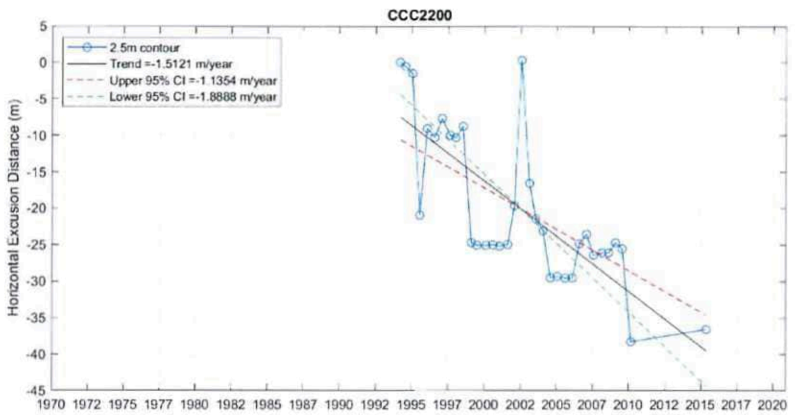
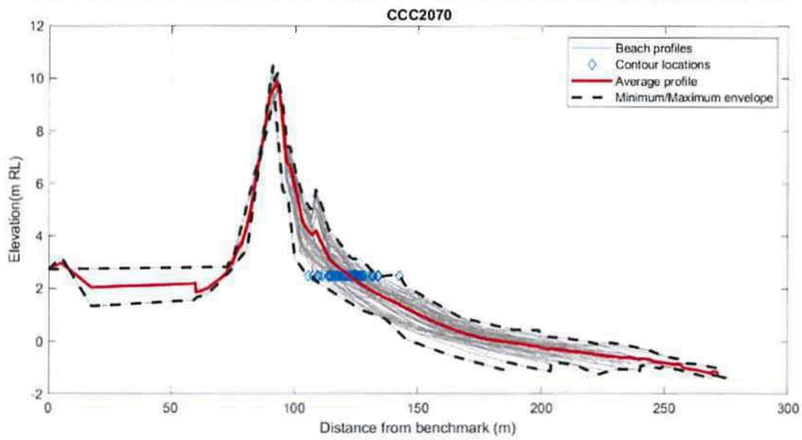
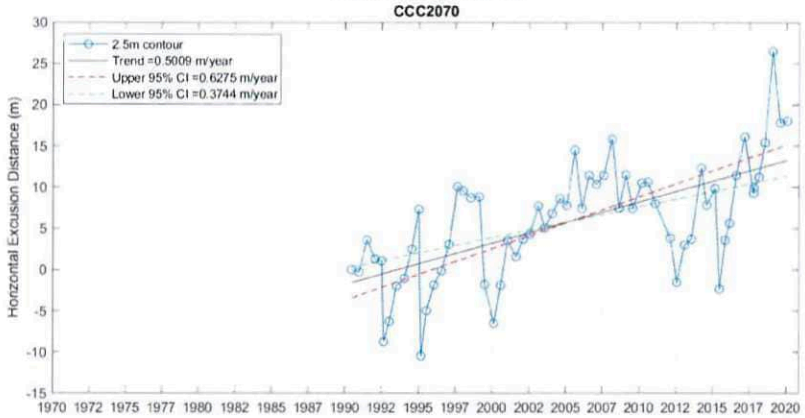
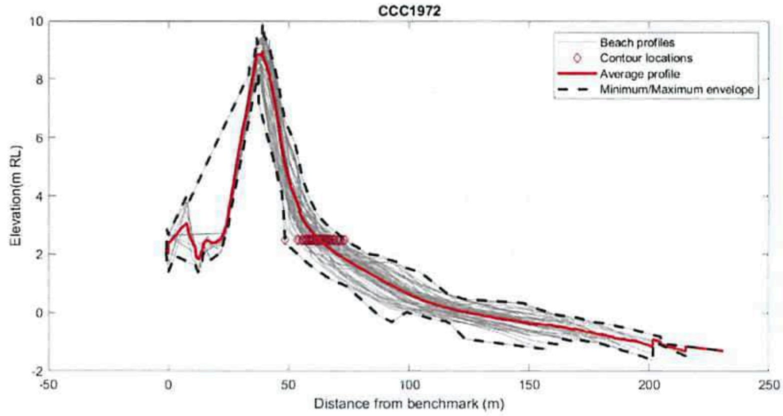
Appendix A: Beach profiles

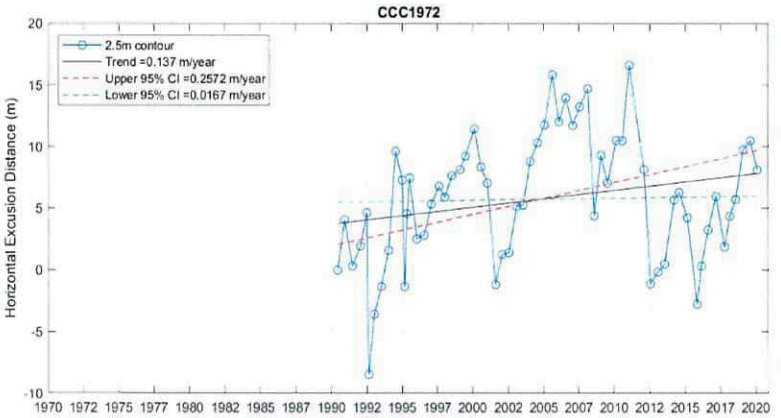

CCC2200
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

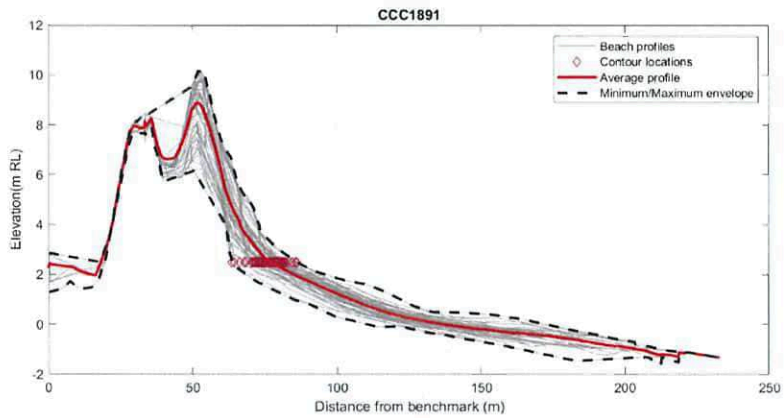

CCC2200
2.5m contour
Trend =-1.5121 m/year
Upper 95% CI =-1.1354 m/year
Lower 95% CI =-1.8888 m/year
Horizontal Excusion Distance (m)

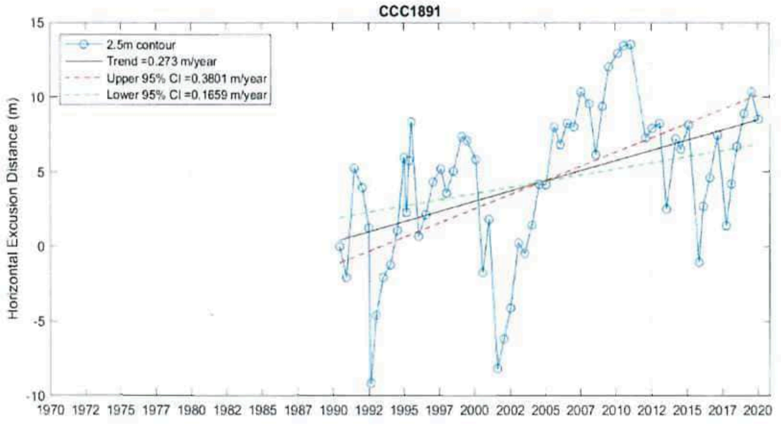

CCC2070
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

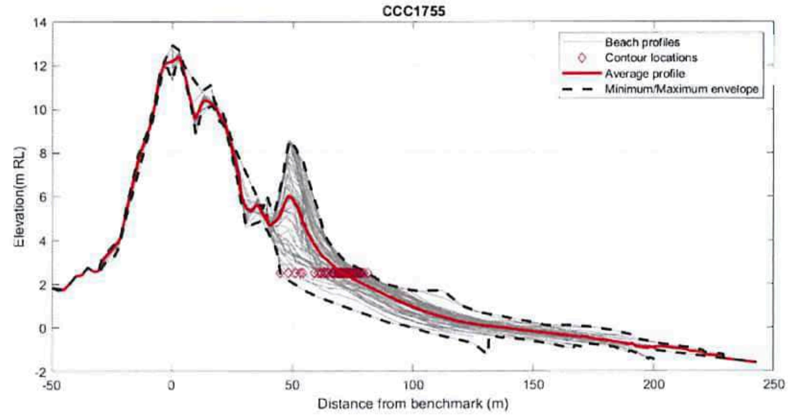

CCC2070
2.5m contour
Trend =0.5009 m/year
Upper 95% CI =0.6275 m/year
Lower 95% CI =0.3744 m/year
Horizontal Excusion Distance (m)

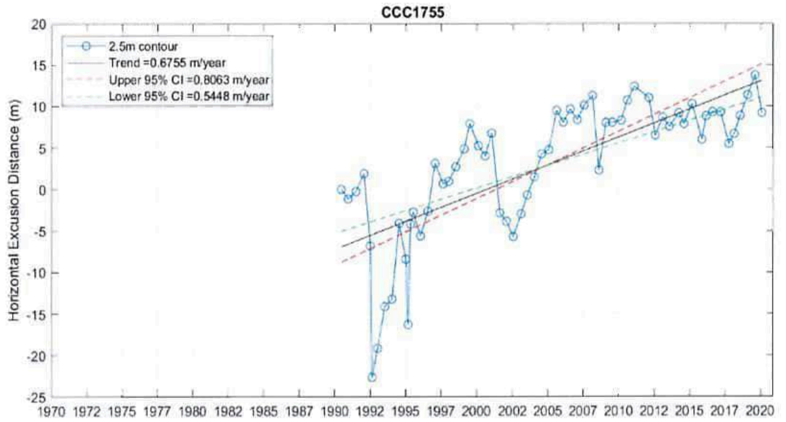

CCC1972
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

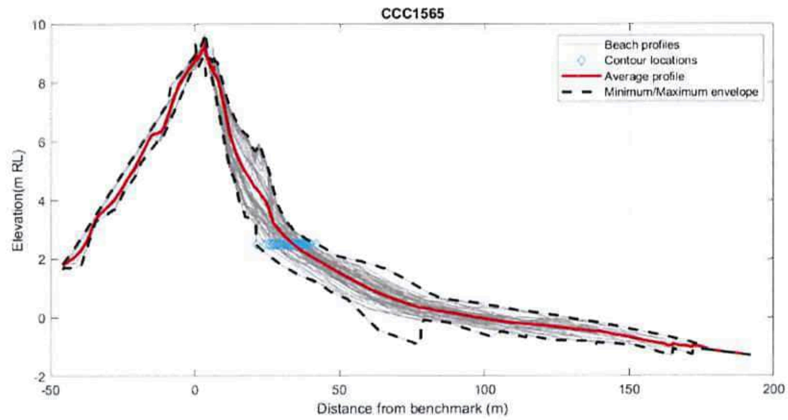

CCC1972
2.5m contour
Trend =0.137 m/year
Upper 95% CI =0.2572 m/year
Lower 95% CI =0.0167 m/year
Horizontal Excusion Distance (m)

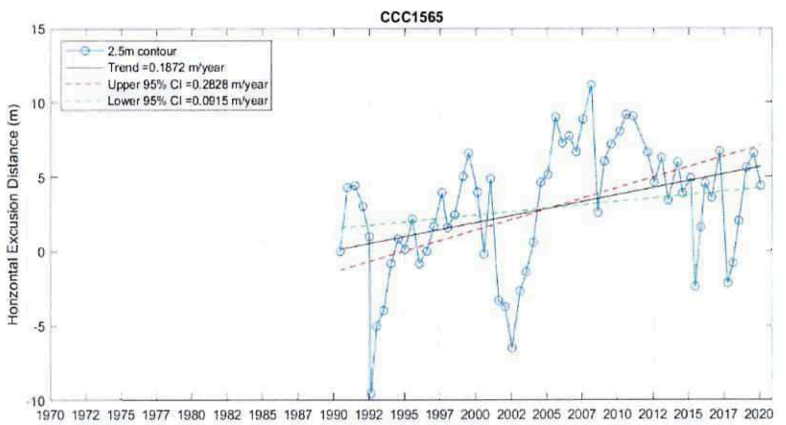

CCC1891
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

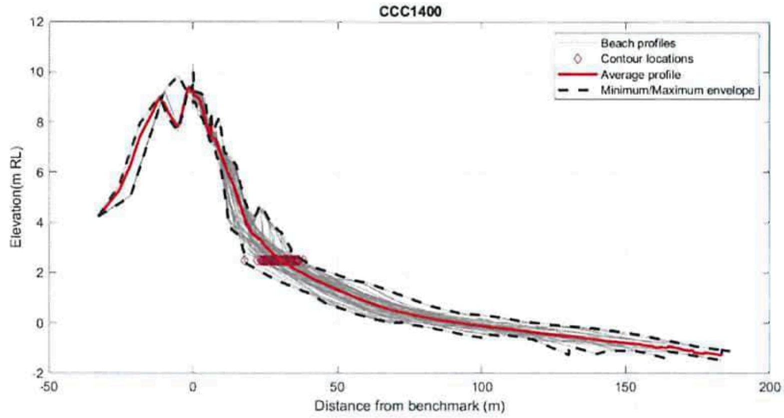

CCC1891
2.5m contour
Trend =0.273 m/year
Upper 95% CI =0.3801 m/year
Lower 95% CI =0.1659 m/year
Horizontal Excusion Distance (m)

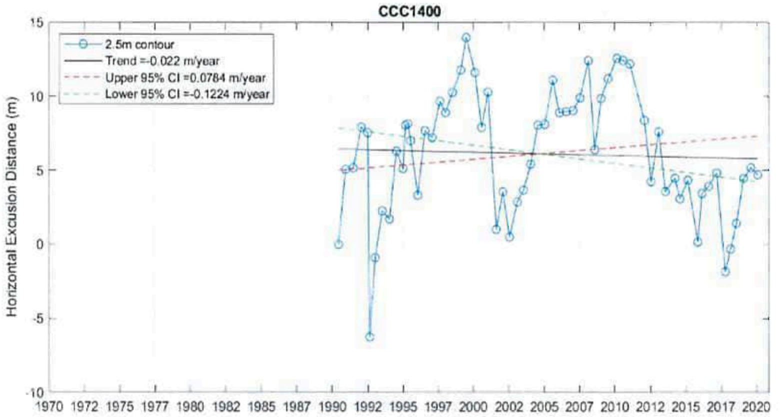

CCC1755
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

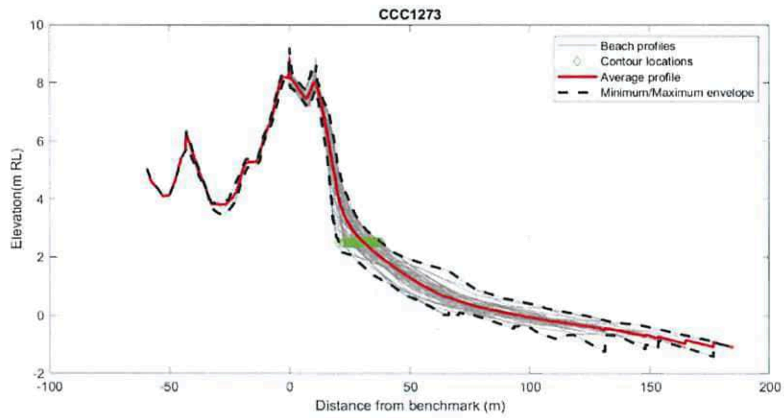

CCC1755
2.5m contour
Trend =0.6755 m/year
Upper 95% CI =0.8063 m/year
Lower 95% CI =0.5448 m/year
Horizontal Excusion Distance (m)

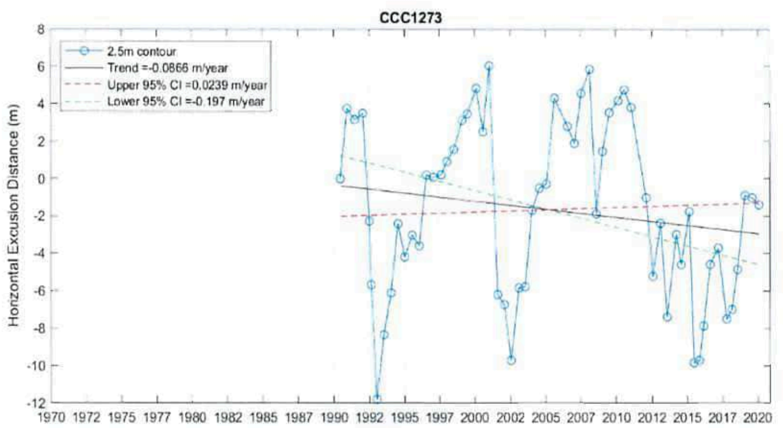

CCC1565
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

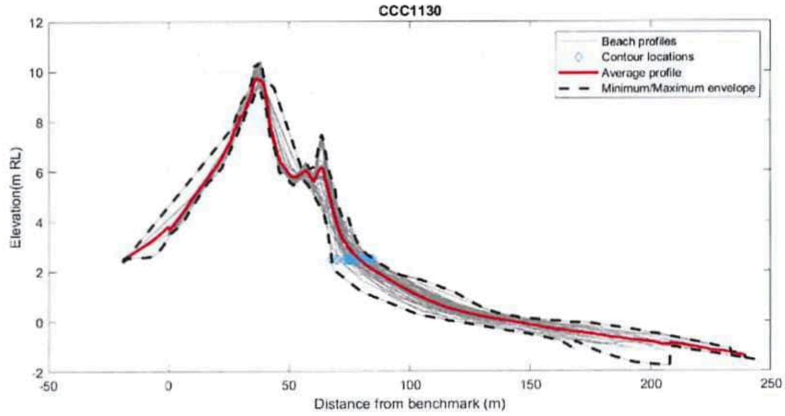

CCC1565
2.5m contour
Trend =0.1872 m/year
Upper 95% CI =0.2828 m/year
Lower 95% CI =0.0915 m/year
Horizontal Excusion Distance (m)

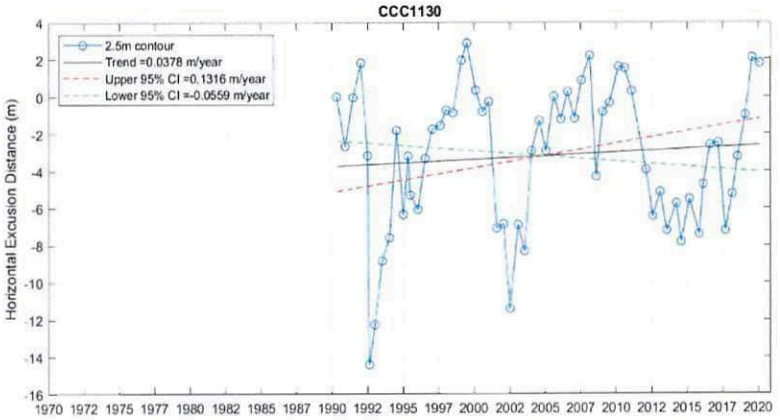

CCC1400
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

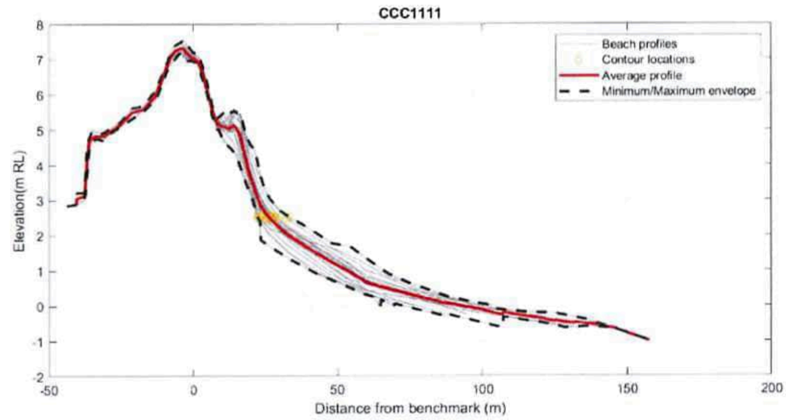

CCC1400
2.5m contour
Trend =-0.022 m/year
Upper 95% CI =0.0784 m/year
Lower 95% CI =-0.1224 m/year
Horizontal Excusion Distance (m)

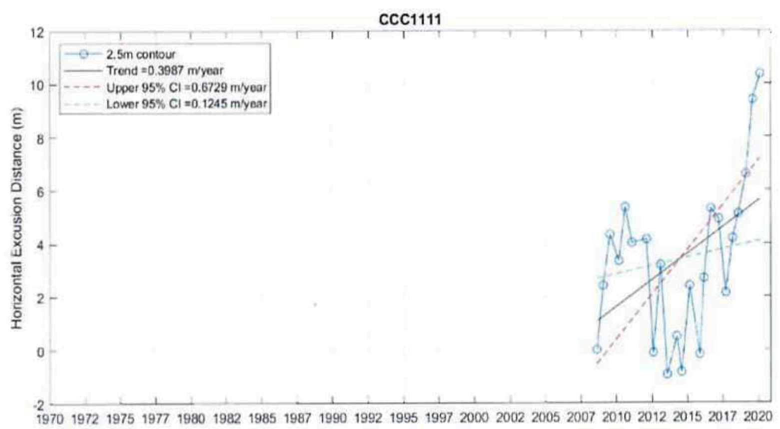

CCC1273
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

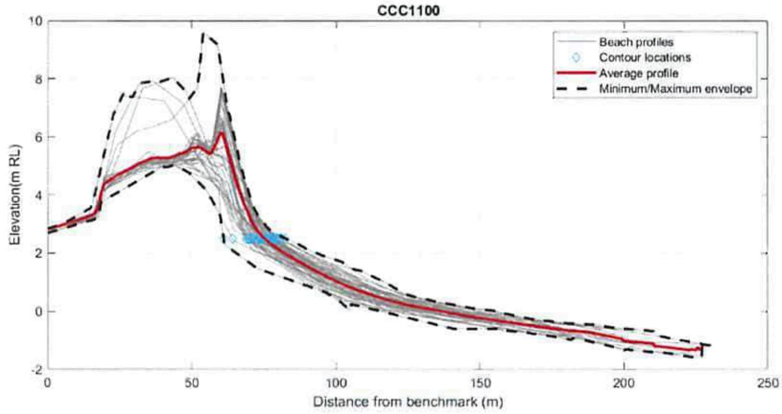

CCC1273
2.5m contour
Trend =-0.0866 m/year
Upper 95% CI =0.0239 m/year
Lower 95% CI =-0.197 m/year
Horizontal Excusion Distance (m)

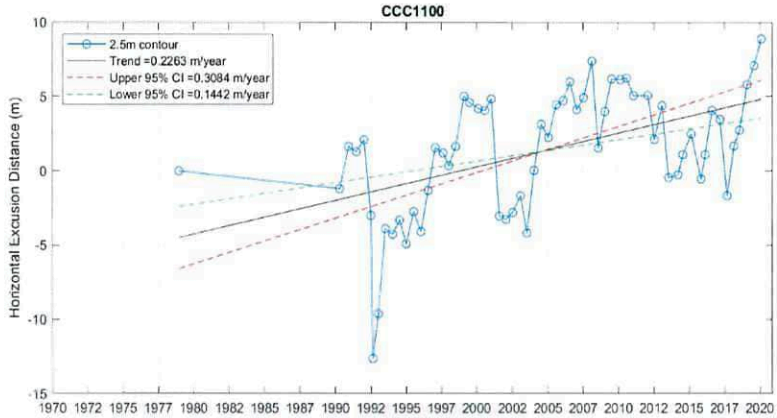

CCC1130
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

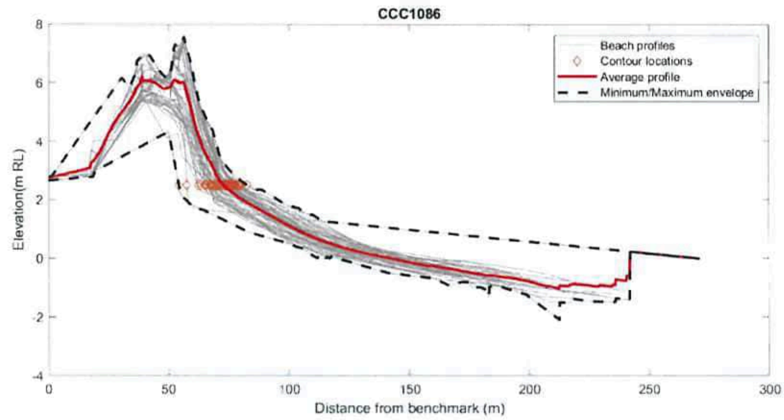

CCC1130
2.5m contour
Trend =0.0378 m/year
Upper 95% CI =0.1316 m/year
Lower 95% CI =-0.0559 m/year
Horizontal Excusion Distance (m)

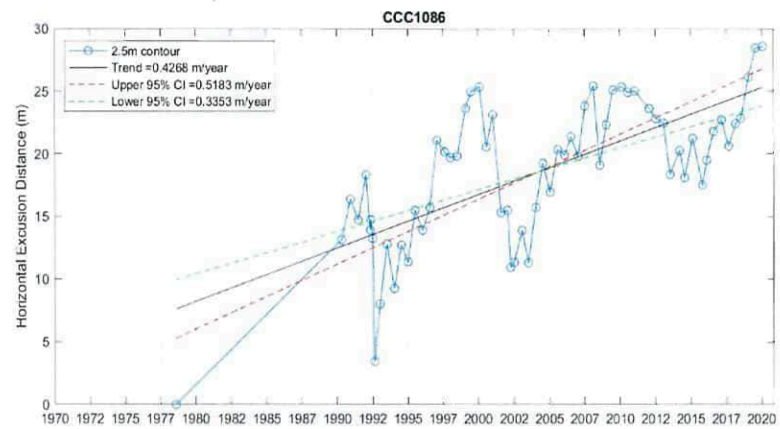

CCC1111
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

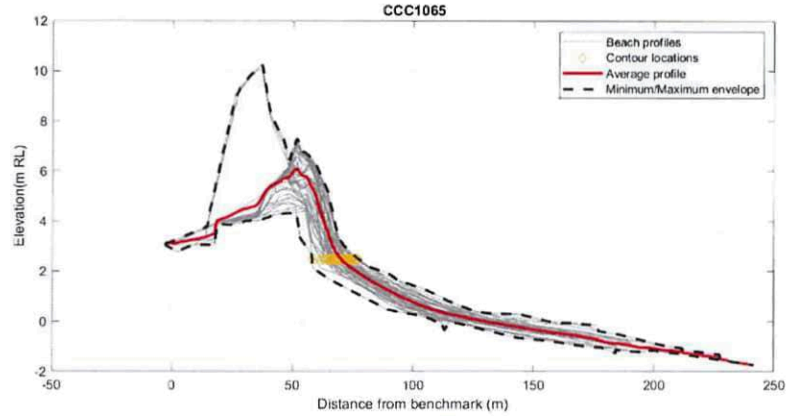

CCC1111
2.5m contour
Trend =0.3987 m/year
Upper 95% CI =0.6729 m/year
Lower 95% CI =0.1245 m/year
Horizontal Excusion Distance (m)

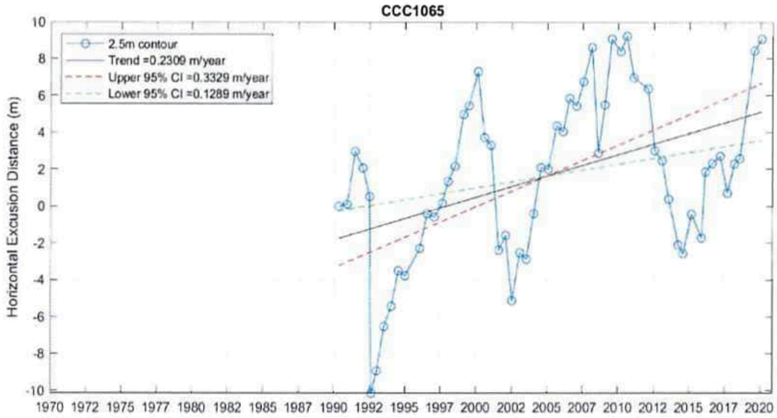

CCC1100
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

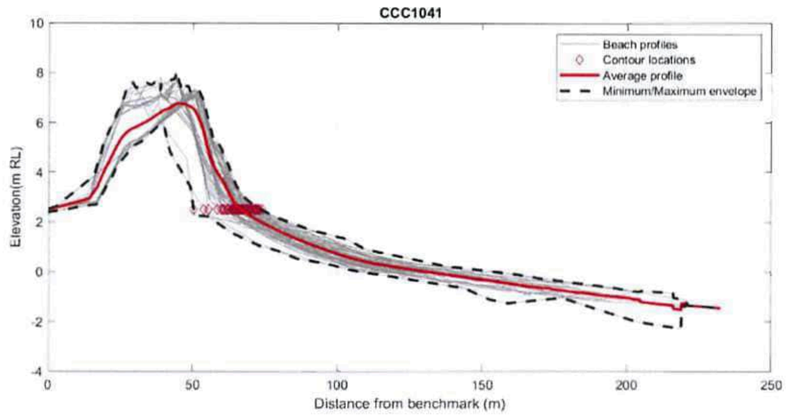

CCC1100
2.5m contour
Trend =0.2263 m/year
Upper 95% CI =0.3084 m/year
Lower 95% CI =0.1442 m/year
Horizontal Excusion Distance (m)

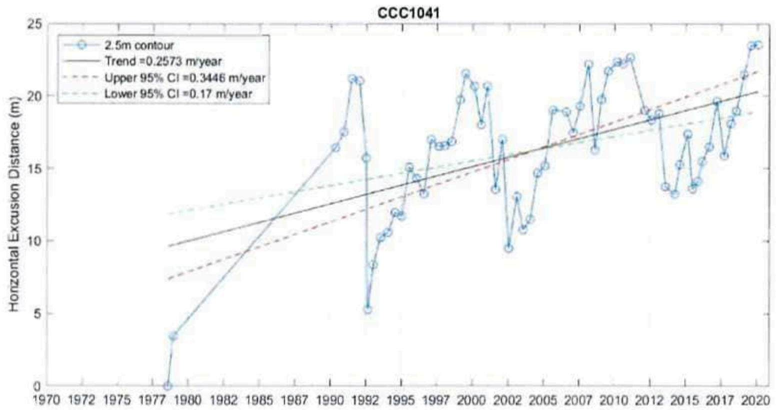

CCC1086
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

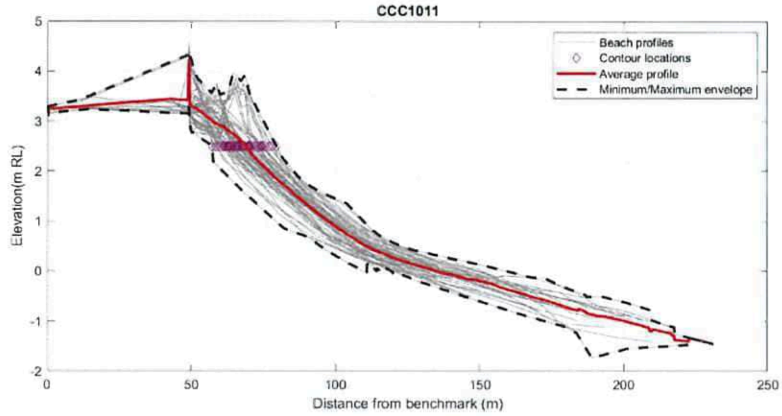

CCC1086
2.5m contour
Trend =0.4268 m/year
Upper 95% CI =0.5183 m/year
Lower 95% CI =0.3353 m/year
Horizontal Excusion Distance (m)

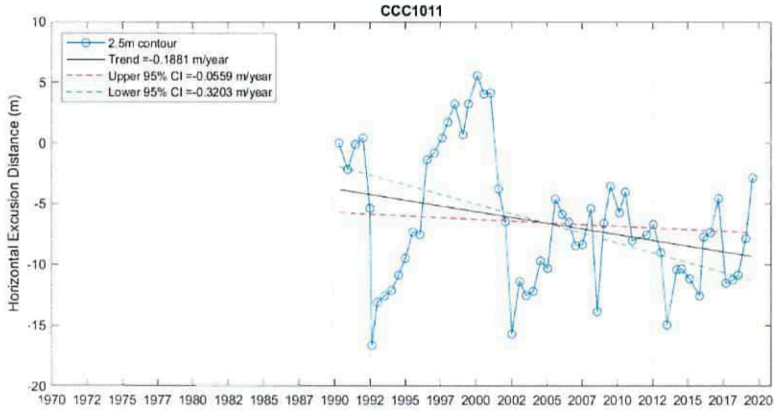

CCC1065
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

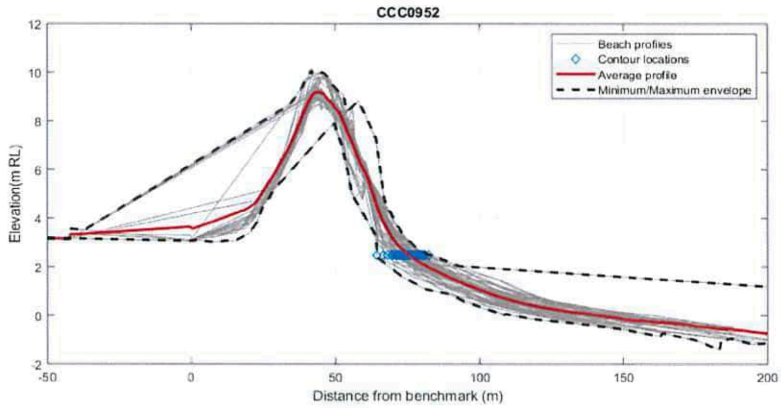

CCC1065
2.5m contour
Trend =0.2309 m/year
Upper 95% CI =0.3329 m/year
Lower 95% CI =0.1289 m/year
Horizontal Excusion Distance (m)

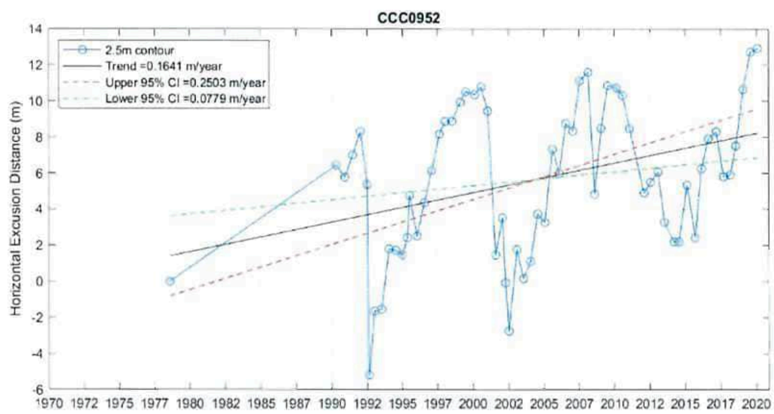

CCC1041
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

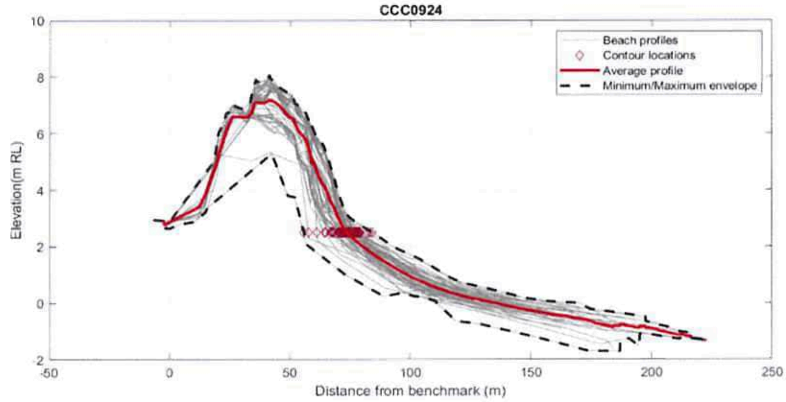

CCC1041
2.5m contour
Trend =0.2573 m/year
Upper 95% CI =0.3446 m/year
Lower 95% CI =0.17 m/year
Horizontal Excusion Distance (m)

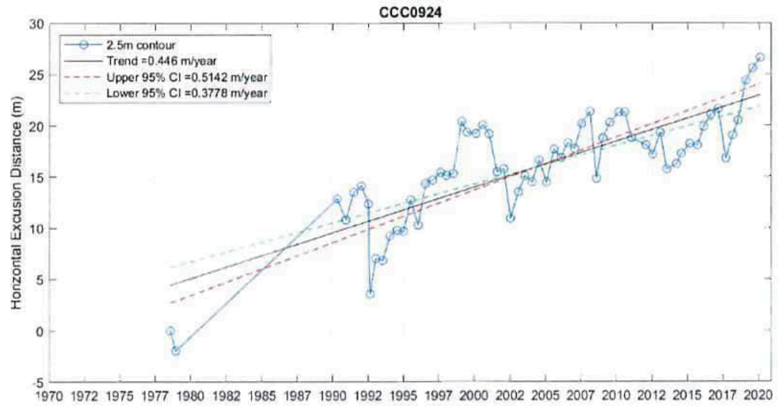

CCC1011
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

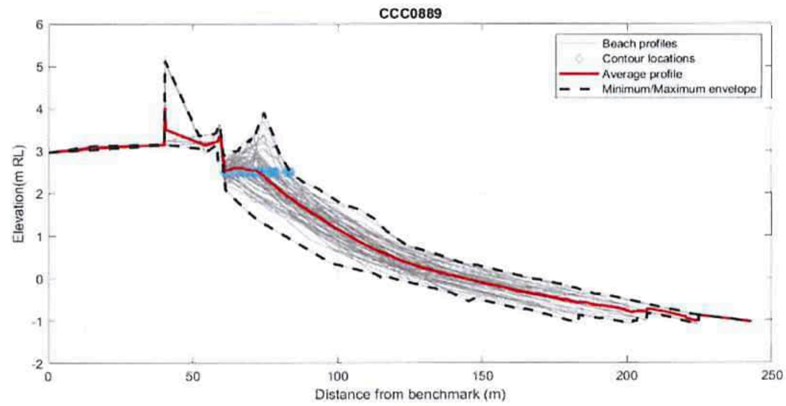

CCC1011
2.5m contour
Trend =-0.1881 m/year
Upper 95% CI =-0.0559 m/year
Lower 95% CI =-0.3203 m/year
Horizontal Excusion Distance (m)

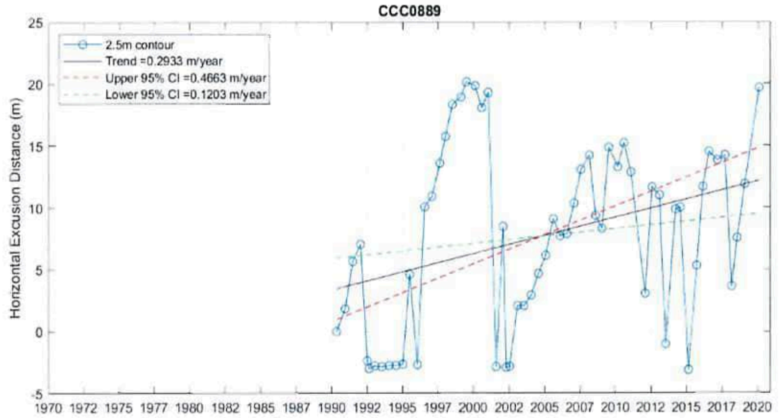

CCC0952
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

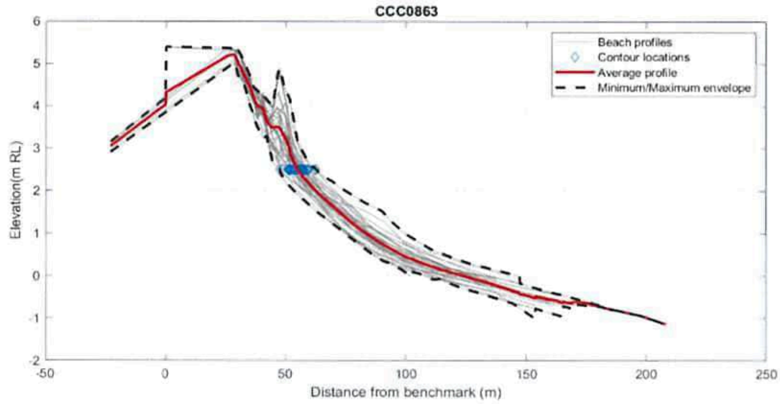

CCC0952
2.5m contour
Trend =0.1641 m/year
Upper 95% CI =0.2503 m/year
Lower 95% CI =0.0779 m/year
Horizontal Excusion Distance (m)

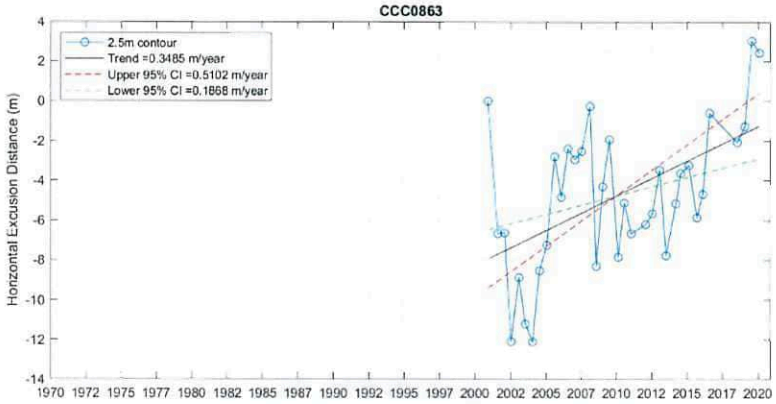

CCC0924
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

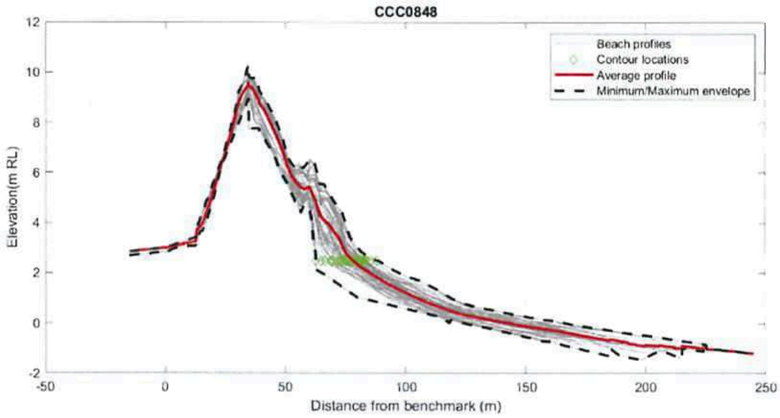

CCC0924
2.5m contour
Trend =0.446 m/year
Upper 95% CI =0.5142 m/year
Lower 95% CI =0.3778 m/year
Horizontal Excusion Distance (m)

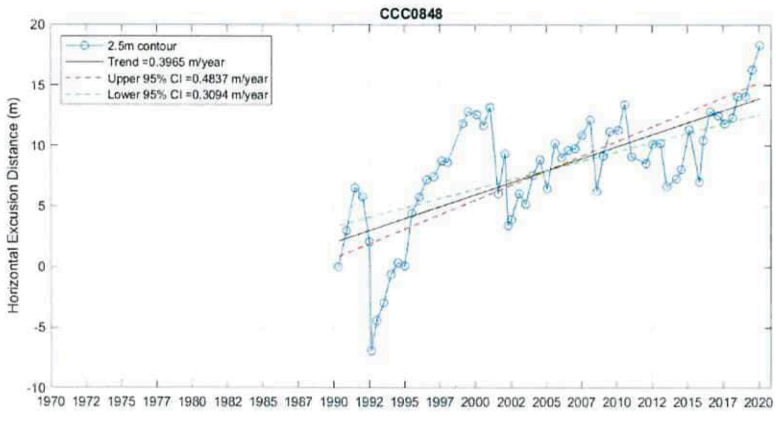

CCC0889
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

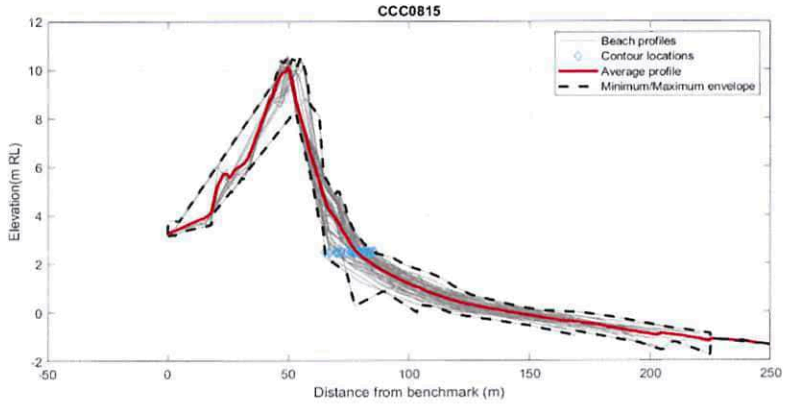

CCC0889
2.5m contour
Trend =0.2933 m/year
Upper 95% CI =0.4663 m/year
Lower 95% CI =0.1203 m/year
Horizontal Excusion Distance (m)

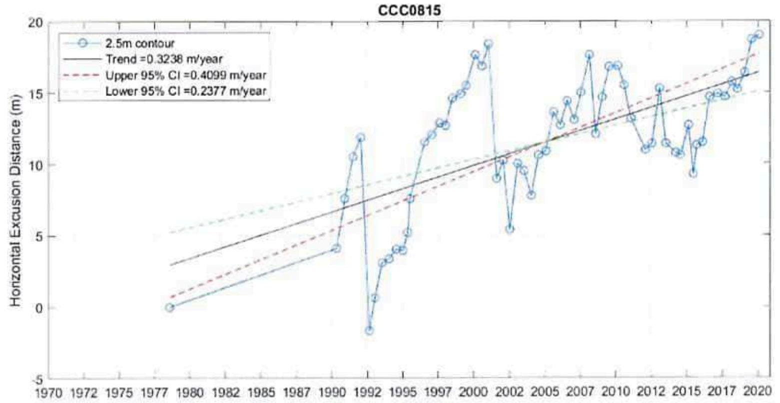

CCC0863
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

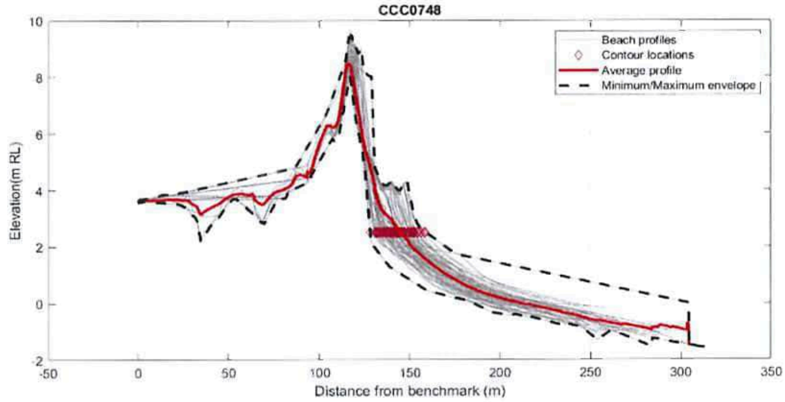

CCC0863
2.5m contour
Trend =0.3485 m/year
Upper 95% CI =0.5102 m/year
Lower 95% CI =0.1868 m/year
Horizontal Excusion Distance (m)

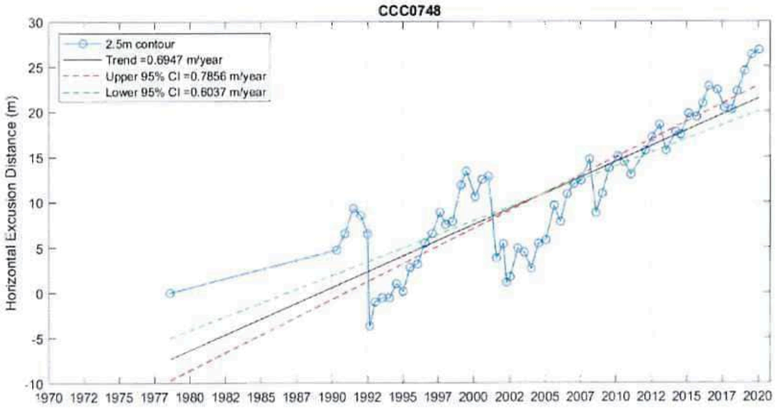

CCC0848
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

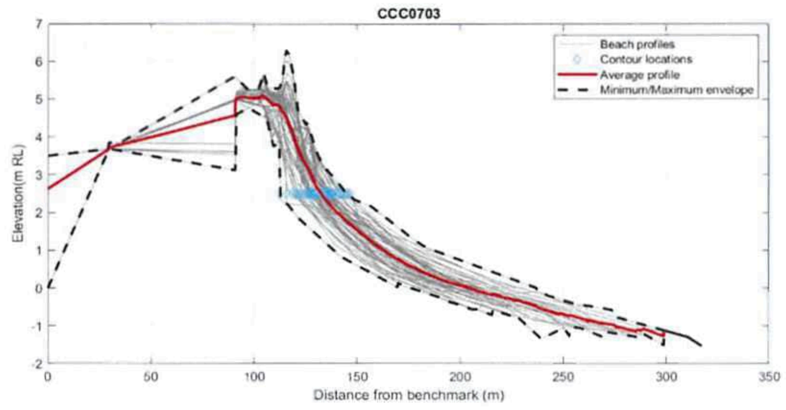

CCC0848
2.5m contour
Trend =0.3965 m/year
Upper 95% CI =0.4837 m/year
Lower 95% CI =0.3094 m/year
Horizontal Excusion Distance (m)

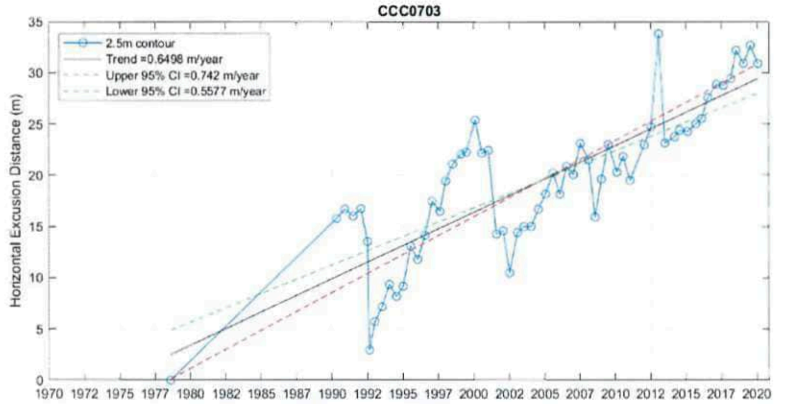

CCC0815
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

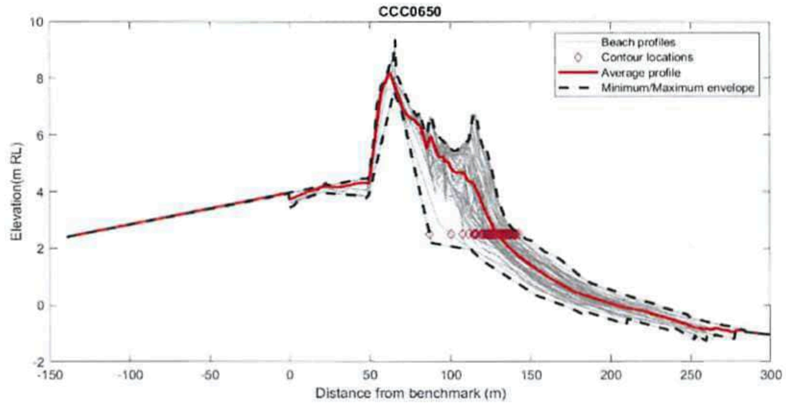

CCC0815
2.5m contour
Trend =0.3238 m/year
Upper 95% CI =0.4099 m/year
Lower 95% CI =0.2377 m/year
Horizontal Excusion Distance (m)

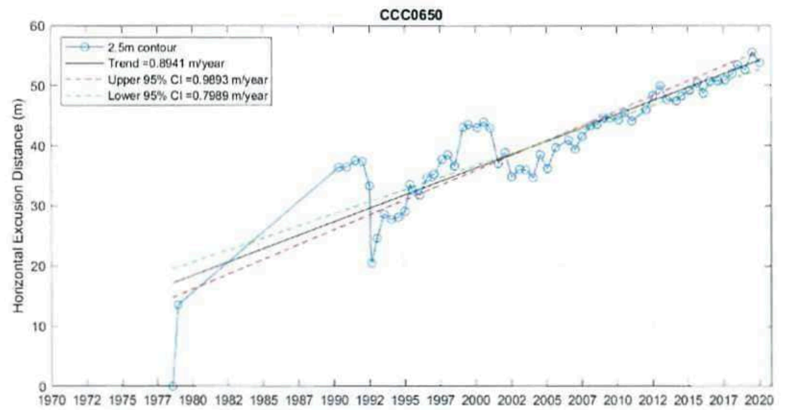

CCC0748
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

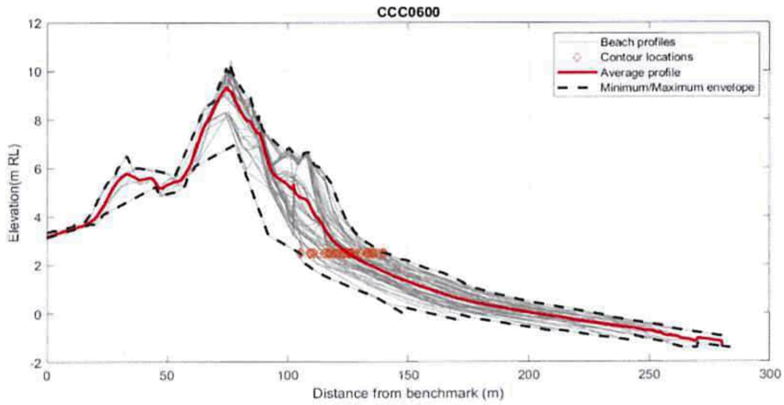

CCC0748
2.5m contour
Trend =0.6947 m/year
Upper 95% CI =0.7856 m/year
Lower 95% CI =0.6037 m/year
Horizontal Excusion Distance (m)

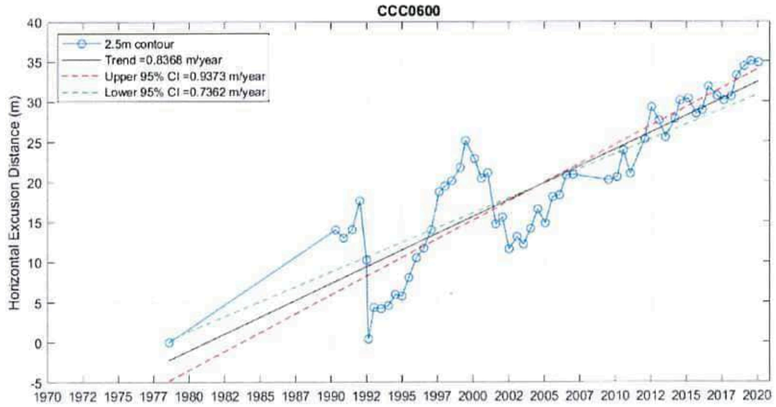

CCC0703
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

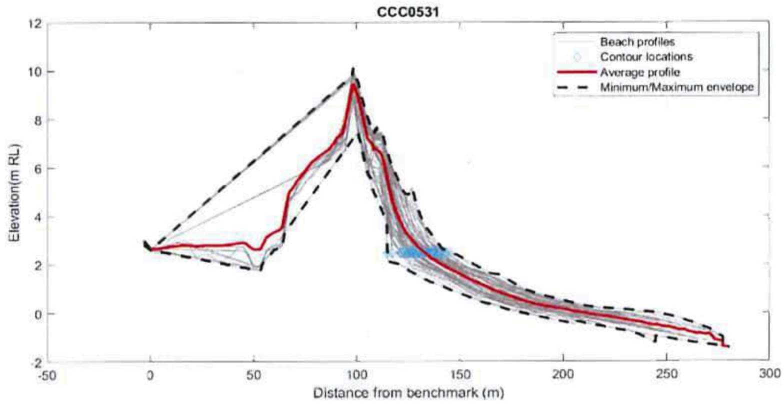

CCC0703
2.5m contour
Trend =0.6498 m/year
Upper 95% CI =0.742 m/year
Lower 95% CI =0.5577 m/year
Horizontal Excusion Distance (m)

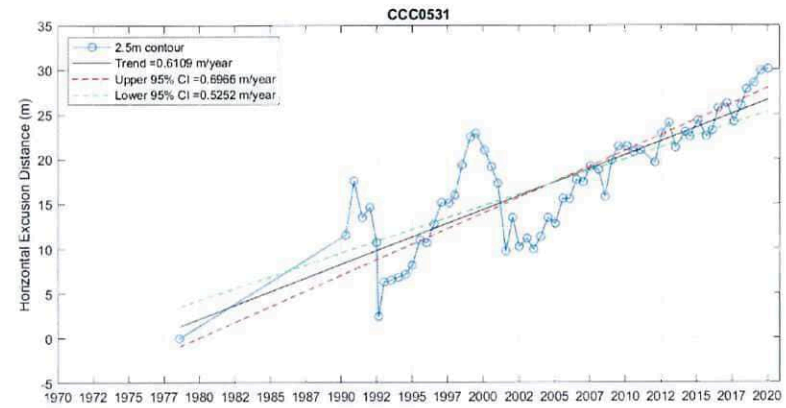

CCC0650
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

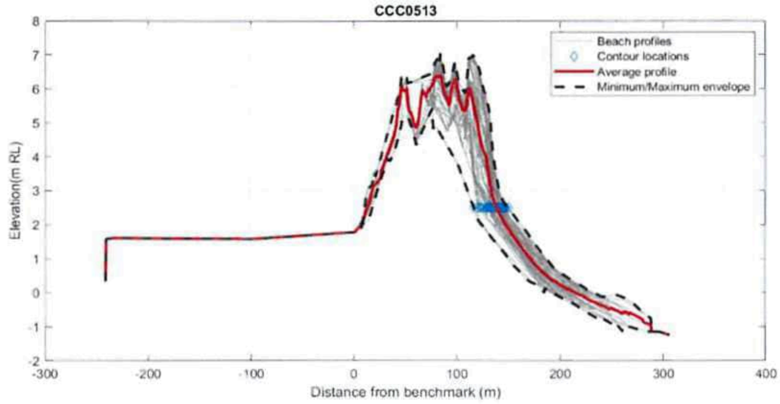

CCC0650
2.5m contour
Trend =0.8941 m/year
Upper 95% CI =0.9893 m/year
Lower 95% CI =0.7989 m/year
Horizontal Excusion Distance (m)

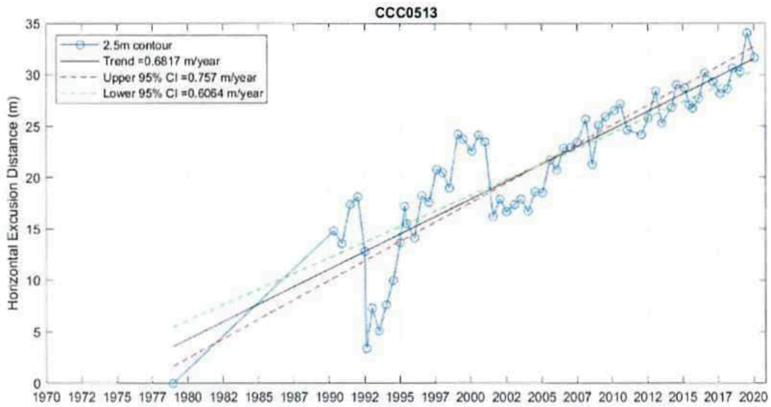

CCC0600
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

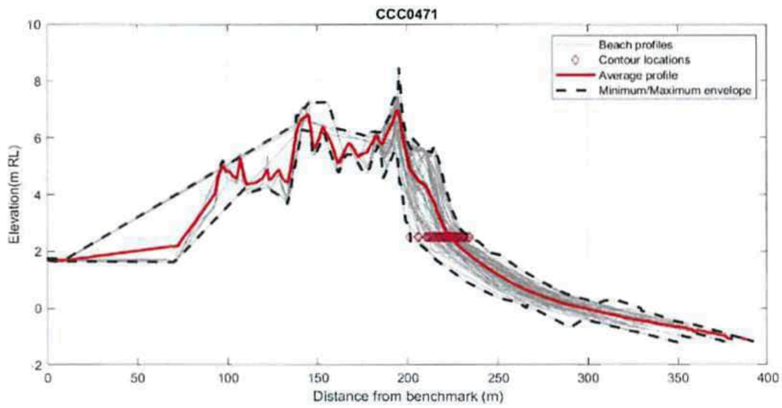

CCC0600
2.5m contour
Trend =0.8368 m/year
Upper 95% CI =0.9373 m/year
Lower 95% CI =0.7362 m/year
Horizontal Excusion Distance (m)

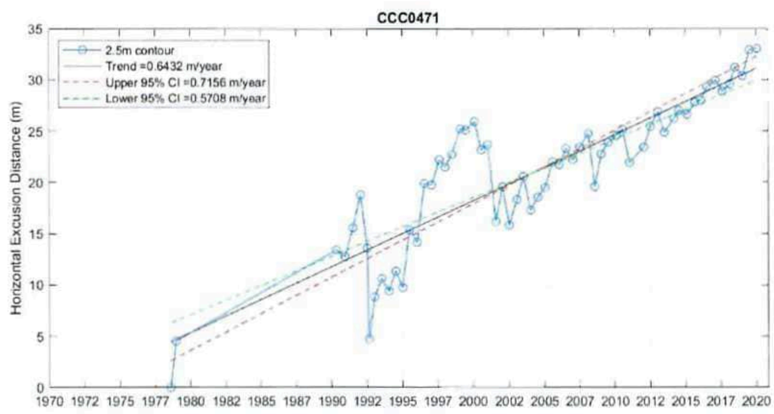

CCC0531
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

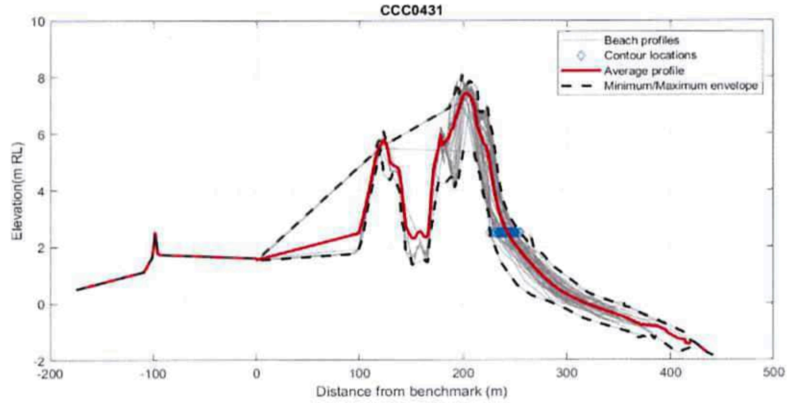

CCC0531
2.5m contour
Trend =0.6109 m/year
Upper 95% CI =0.6966 m/year
Lower 95% CI =0.5252 m/year
Horizontal Excusion Distance (m)

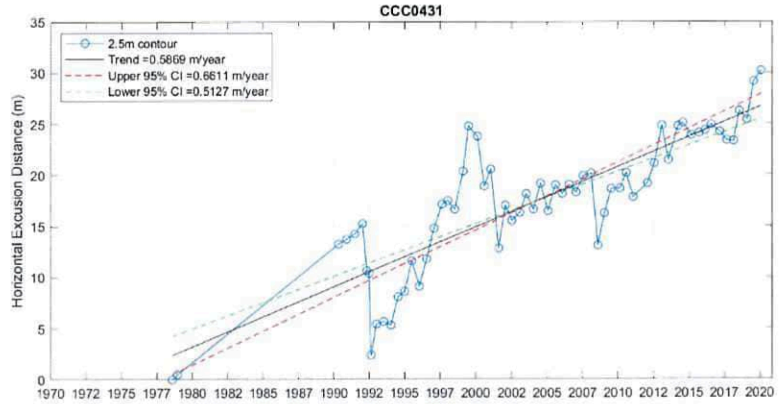

CCC0513
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

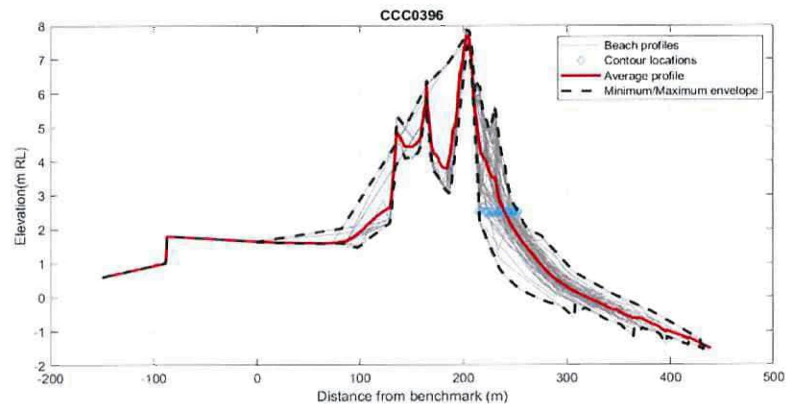

CCC0513
2.5m contour
Trend =0.6817 m/year
Upper 95% CI =0.757 m/year
Lower 95% CI =0.6064 m/year
Horizontal Excusion Distance (m)

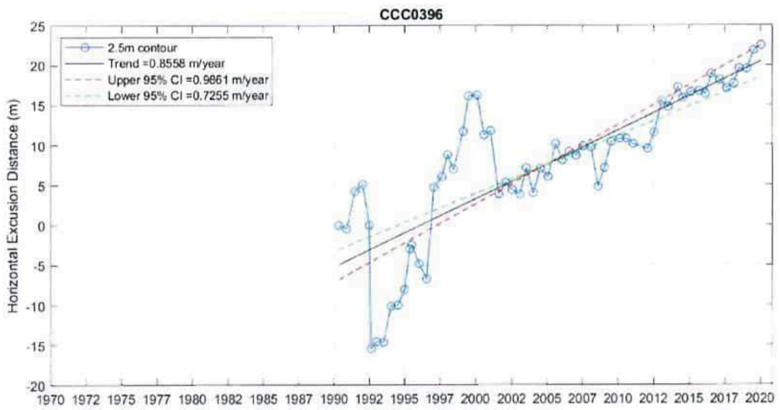

CCC0471
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

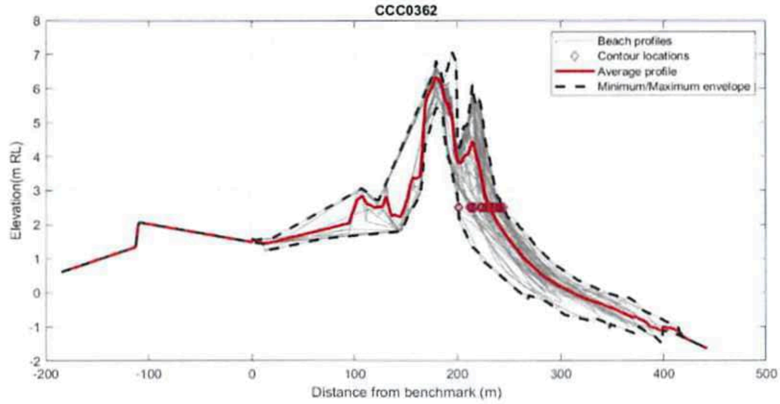

CCC0471
2.5m contour
Trend =0.6432 m/year
Upper 95% CI =0.7156 m/year
Lower 95% CI =0.5708 m/year
Horizontal Excusion Distance (m)

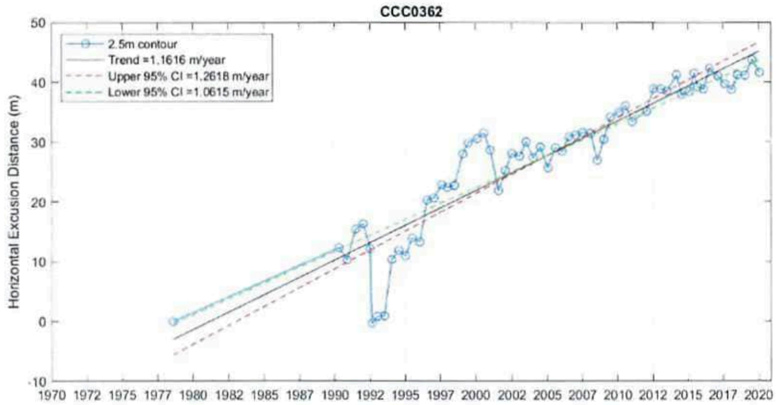

CCC0431
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

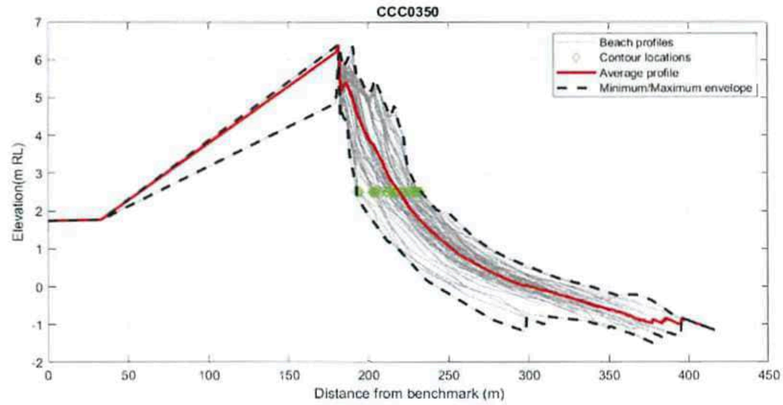

CCC0431
2.5m contour
Trend =0.5869 m/year
Upper 95% CI =0.6611 m/year
Lower 95% CI =0.5127 m/year
Horizontal Excusion Distance (m)

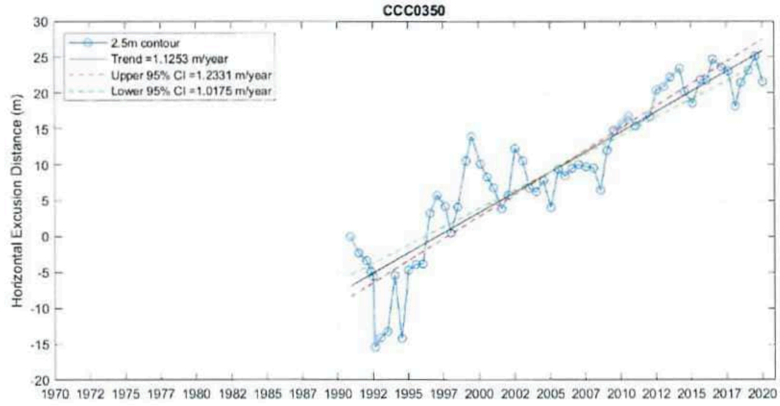

CCC0396
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)

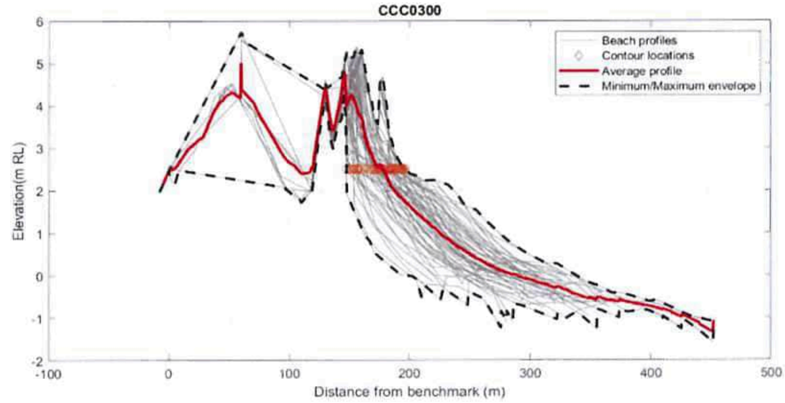

CCC0396
2.5m contour
Trend =0.8558 m/year
Upper 95% CI =0.9861 m/year
Lower 95% CI =0.7255 m/year
Horizontal Excusion Distance (m)

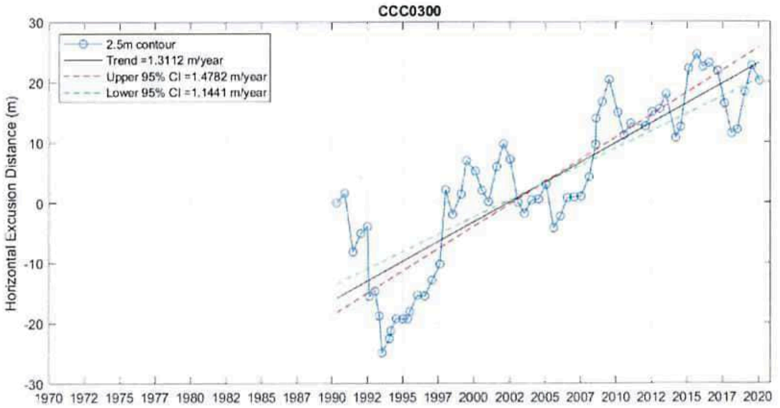

CCC0362
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


CCC0362
2.5m contour
Trend =1.1616 m/year
Upper 95% CI =1.2618 m/year
Lower 95% CI =1.0615 m/year
Horizontal Excusion Distance (m)


CCC0350
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


CCC0350
2.5m contour
Trend =1.1253 m/year
Upper 95% CI =1.2331 m/year
Lower 95% CI =1.0175 m/year
Horizontal Excusion Distance (m)


CCC0300
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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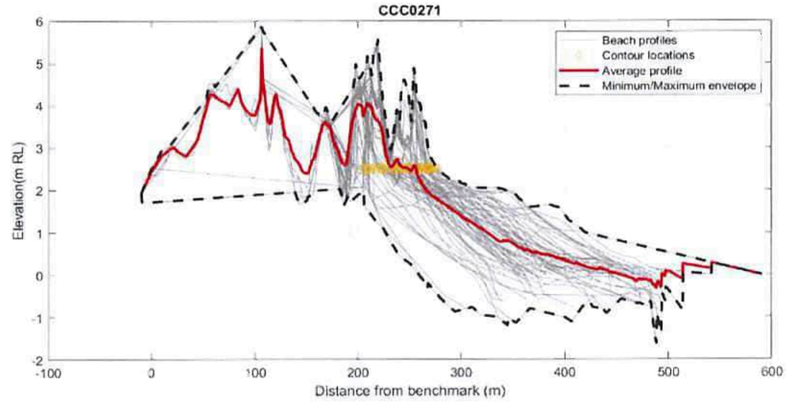
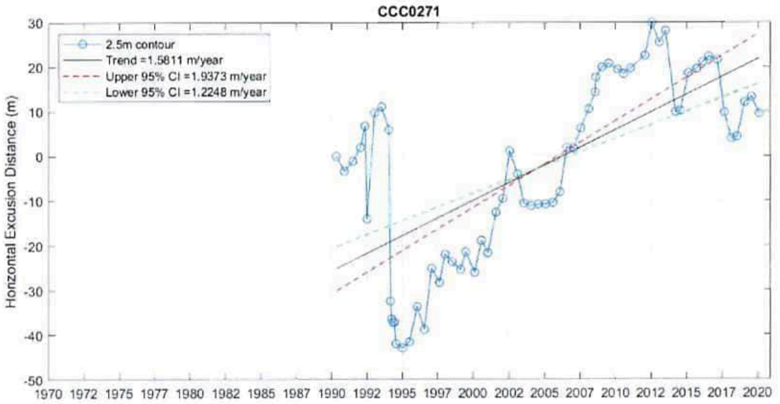
Upper 95% CI =1.4782 m/year
Lower 95% CI =1.1441 m/year
Horizontal Excusion Distance (m)


CCC0271
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


CCC0271
2.5m contour
Trend =1.5811 m/year
Upper 95% CI =1.9373 m/year
Lower 95% CI =1.2248 m/year
Horizontal Excusion Distance (m)

Appendix B: Wave transformation using numerical SWAN model

B1 Estuary and harbour sites

Numerical wave transformation modelling has been undertaken to transform offshore waves into the shoreline for Avon-Heathcote Estuary, Lyttleton Harbour and Akaroa Harbour.

B2 Model description

The numerical model SWAN (Simulating Waves Nearshore) has been used to undertake wave transformation modelling. SWAN is a third-generation wave model that computes random, short-crested wind-generated waves in coastal regions and inland waters by solving the spectral action balance equation without any restrictions on the wave spectrum evolution during growth or transformation. The SWAN model accommodates the process of wind generation, white capping, bottom friction, quadruplet wave-wave interactions, triad wave-wave interactions and depth induced breaking. SWAN is developed at Delft University of Technology in the Netherlands and is widely used by government authorities, research institutes and consultants worldwide. Further details of SWAN can be found in Booij et al. (1999).

B3 Model domains

Local model domains have been generated for Akaroa, Lyttleton and the Avon Heathcote Estuary (Appendix B Table 1).

Appendix B Table 1: Model Domains

Model Domain	Coordinates (lower left corner) [X,Y] NZTM2000	Domain size [X,Y]	Grid resolution
Akaroa	1592100, 5137300	7.9 x 19.0 km^2	10 m x 10 m
Avon Heathcote	1581700, 5175300	6.2 x 5.7 km^2	10 m x 10 m
Lyttleton	1570650, 5163700	17.8 x1 0.3 km^2	10 m x 10 m

B4 Wave transformation modelling

Wave transformation modelling has been undertaken to transform the offshore wave characteristics into nearshore wave conditions where they are used to calculate wave effects (i.e. set-up and run-up). Simulations have been undertaken for each model domain for a range of relevant wave periods and directions. This has resulted in wave height transformation coefficients being established between the offshore and nearshore positions for each relevant direction and period. Both wind generated waves and swell waves have been analysed.

Examples of SWAN model results for the 100-year ARI events showing the wave transformation are shown in the figures on the following pages:

- Figure Appendix B.1 to Figure Appendix B.3 show example results of the significant wave height of wind generated waves during a 100-year ARI windstorm event, with 1.5 m of sea level rise.
- Figure Appendix B.4 and Figure Appendix B.5 show example results of the significant wave height from offshore swell during a 100-year ARI windstorm event, with 1.5 m of sea level rise.

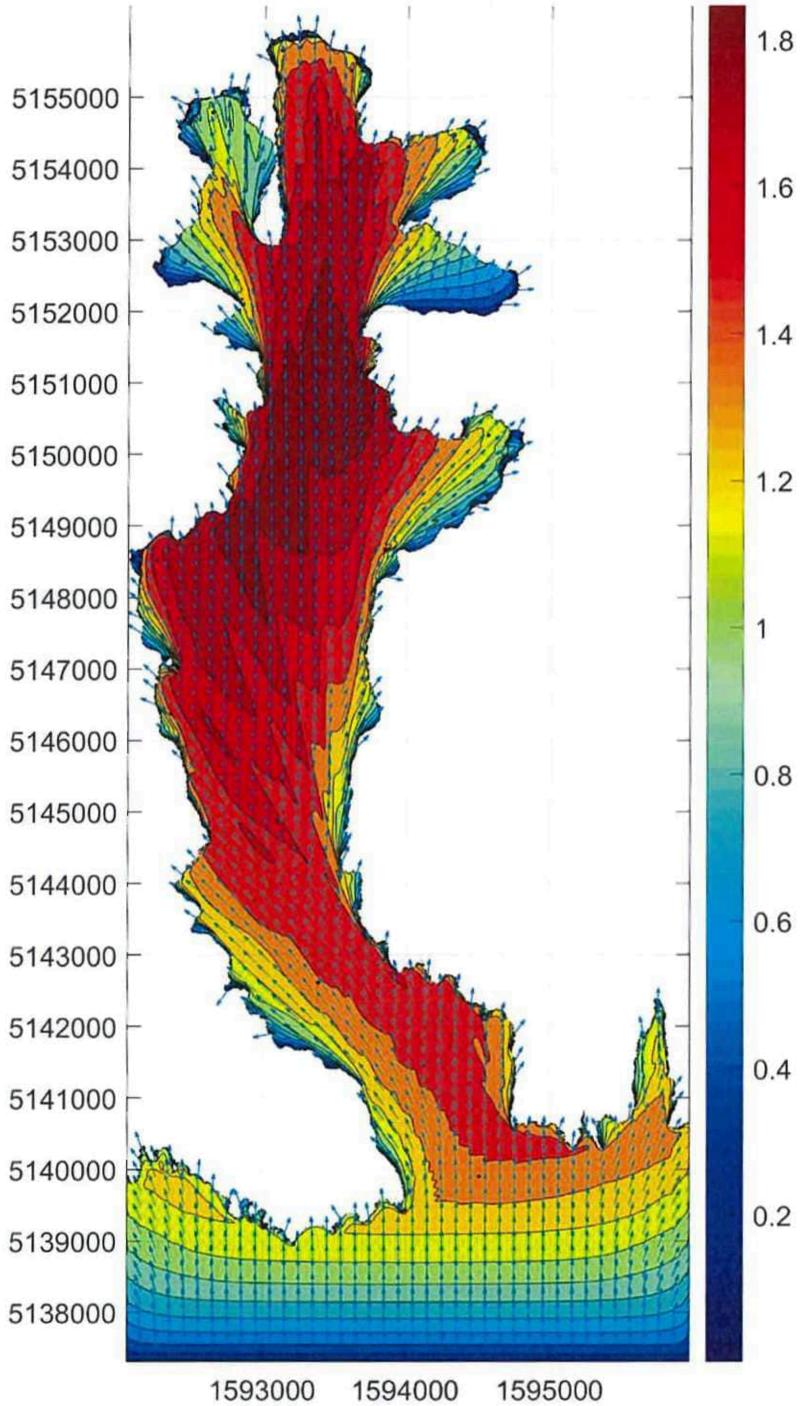


Figure Appendix B.1: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South - Wind generated waves.

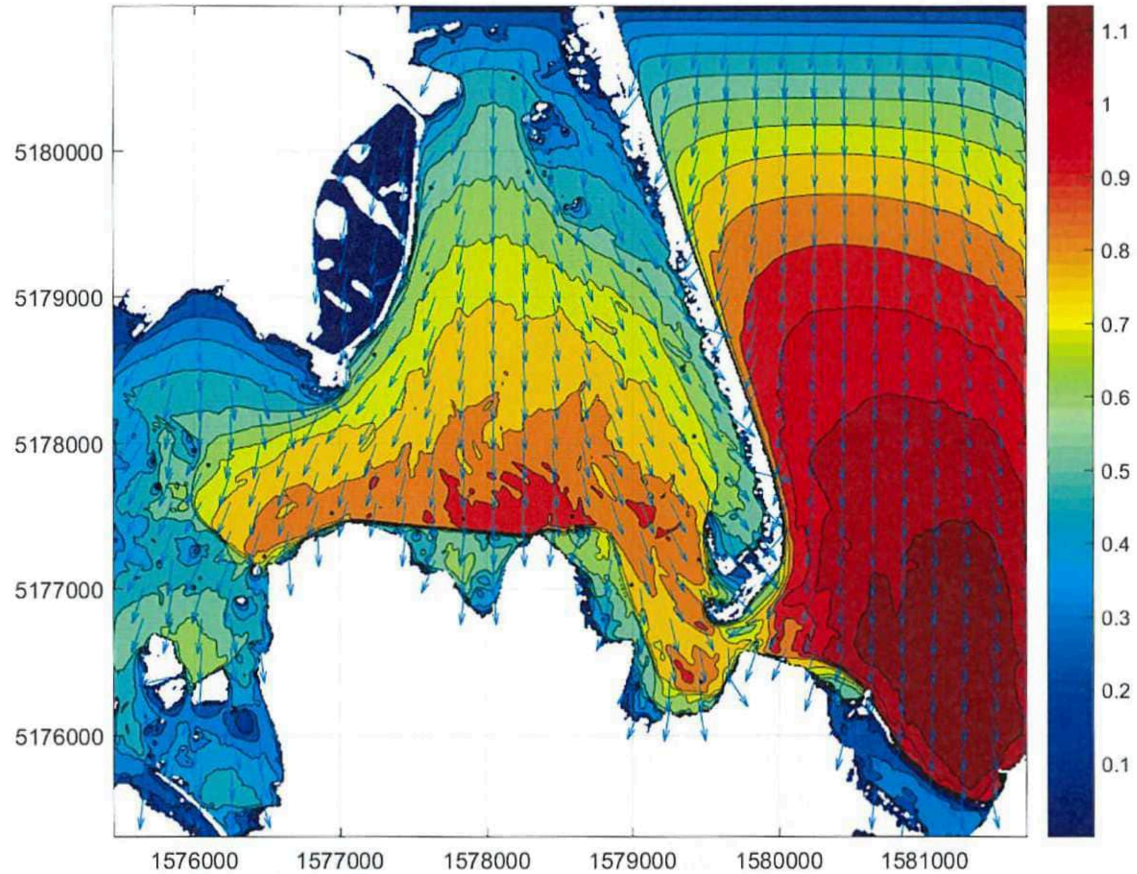


Figure Appendix B.2: SWAN model results for the Avon Heathcote domain – Significant wave height and direction during a 100-year ARI storm from the North – Wind generated waves.

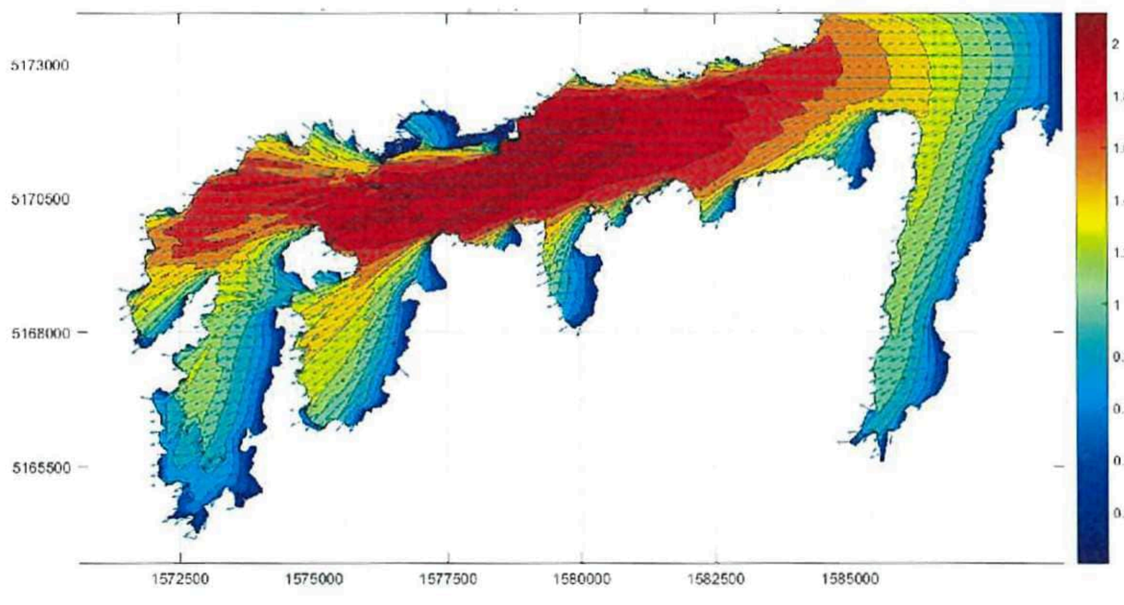


Figure Appendix B.3: SWAN model results for the Lyttleton domain – Significant wave height and direction during a 100-year ARI storm from the East – Wind generated waves.

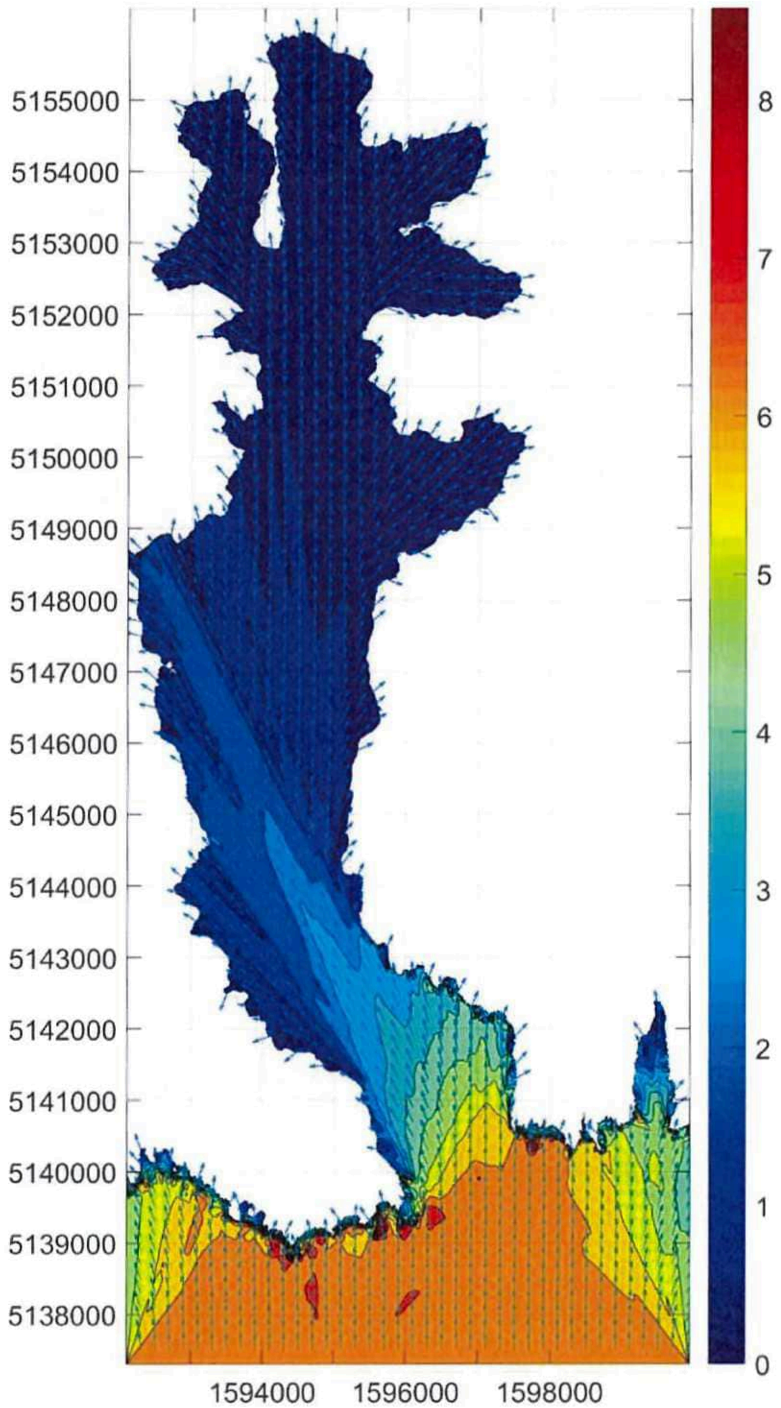


Figure Appendix B.4: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South – Swell.

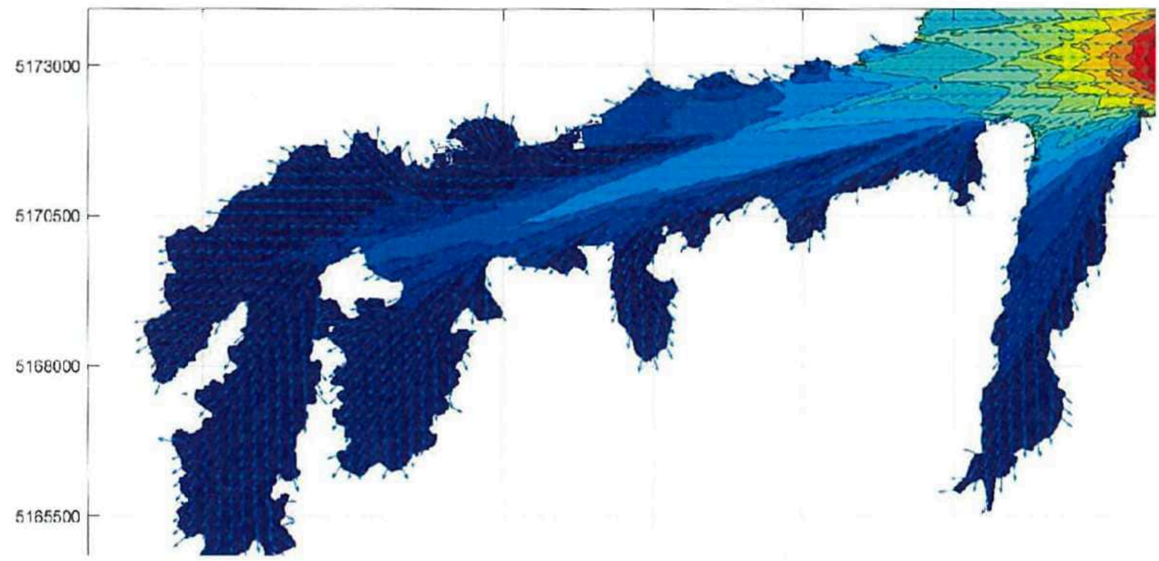


Figure Appendix B.5: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South- Swell.

Appendix C: Sensitivity assessment of bathtub approach

Tonkin+Taylor

Memo

To:	**Chch CHA Technical Reviewers**	**Job No:**	**1012976**
From:	**T+T technical team**	**Date:**	**20 October 2020**
Subject:	**Comparison of bathtub modelling with hydrodynamic modelling**		

1 Introduction

As part of the Christchurch City Council (CCC) Coastal Hazards Assessment (CHA) study, CCC require technical assessment to identify areas potentially susceptible to coastal inundation around Christchurch City. This memo explores the technical and practical advantages and disadvantages of two flood modelling options which are being considered for the CHA study: "bathtub modelling" and "hydrodynamic modelling".

The focus of the current CHA technical assessment is to produce "base" hazard information that can then feed into community engagement, risk evaluation and risk mitigation and adaptation planning undertaken by CCC in the future. Given the intent of the data usage for high-level public engagement and adaptation planning, and the large number of scenarios to be considered, T+T has proposed to adopt a connected-bathtub modelling approach to assess the approximate potential coastal inundation extents around Christchurch City. This approach is simpler than the alternative hydrodynamic modelling approach that has been previously applied, but the flexibility and responsiveness that it offers means that is considered more suitable for the specific intended purpose of this study (i.e. the initial engagement and risk evaluation stages of adaptation planning).

2 Modelling approach

The previous T+T (2017) assessment utilised a hydrodynamic model to assess the extent of storm tide propagation within the Avon-Heathcote Estuary and Brooklands lagoon. A hydrodynamic model has technical advantages compared to simpler approaches (such as bath tub modelling), because it takes full account of hydraulic performance that can limit the inundation extents based on tidal duration (i.e. there is a limit to how far floodwater can travel before the tide turns). However, the hydrodynamic modelling approach brings with it some practical disadvantages, for example:

- It is reliant on accurate definition of the boundary and forcing conditions such as the tidal boundary, fresh water inflows and wind. A hydrodynamic model requires values for all fresh water inflows whether static or time-varying, and there are numerous permutations that might be considered for joint probability between extreme sea level and stream/river flow.
- Hydraulic performance should be calibrated using data from the area of interest for similar scenarios. This brings in factors such as hydraulic roughness, vegetation growth and channel definition from the DEM that are not considered using a bathtub approach.
- Previous modelling noted difficulties accurately defining the seaward boundary including potential wave set up over Sumner Bar, and in defining the coincident rivers flow and winds. This is notably more complex if these boundaries are time-varying as would normally be applied in a hydrodynamic model. The bathtub approach avoids this.

- It is highly sensitive to the ground elevation model adopted, so results can be substantially impacted by small changes due to natural measurement variability between different ground level surveys (new LiDAR data has become available since the previous T+T hydrodynamic model was developed), and small changes in ground level (e.g. localised earthworks). This is particularly the case where hydraulic performance is dictated by bathymetry and channel dimensions
- The peak levels attained are subject to influence from surface water flow from streams and rivers, and these inflows demand careful consideration to ensure that a robust approach to joint probability between rainfall and sea condition is maintained.

These disadvantages make hydrodynamic analysis less useful than bathtub modelling for assessing scenarios where these future conditions and calibration parameters are unknown or highly uncertain. Furthermore, a more comprehensive city-wide flood model has been developed by CCC (since the 2017 T+T assessment) which can be used when more detailed modelling results are required in a specific location for other purposes (e.g. for setting floor levels or for detailed design of infrastructure). This means that a bathtub modelling approach is being considered as a methodology option for the current high-level coastal hazard assessment.

The bathtub approach would enable the updated coastal inundation assessment to be based on the latest available 2018 LiDAR ground level survey, and be readily updated for future ground surface models or to examine the effectiveness of any physical mitigation options being considered. This could also be achieved through re-development of a hydrodynamic model, but would require substantial time and cost to re-develop and calibrate (limiting the number of adaption scenarios that could practically be considered); and would still leave uncertainty regarding the absolute accuracy of model results because of uncertainty in the input parameters. The bathtub approach also utilises the specific extreme levels derived at gauges within the estuaries (so is directly linked to actual physical observations) rather than having to develop boundary conditions and achieve a match in the model.

The primary disadvantage of a bathtub modelling approach is that all areas across the city below the specified bathtub level are identified as inundated, which does not allow for changes in flood levels further away from the coast and rivers (although connected and unconnected areas can be separately defined). This means that it has a tendency to over-predict the absolute extent of coastal flooding for a specific scenario. It should also be noted that under flood event conditions, the bathtub model may under-predict extreme inundation.

However this tendency for over-prediction can be taken into account during adaptation planning, and the following key concepts clearly communicated in adaptation discussions:

- The bathtub analysis results are indicative rather than precise, so are best used to understand relative changes in risk from different adaptation options, rather than quantifying the absolute level of risk.
- An additional "buffer" should not be applied beyond the modelled areas, as the modelled extent already includes a degree of conservatism at the edges.
- The absolute accuracy of the results can vary across the study area and for different water levels and adaptation scenarios (e.g. the results may be more conservative in some situations, and less conservative in others).
- Uncertainty in future conditions (e.g. sea level and storm events) can have a more significant effect on inundation extent that the modelling approach, so adaptation planning should consider a range of possible future scenarios rather than focussing on a single model output.

In order to assess the suitability of the bathtub modelling method for use on this project, it has been compared with outputs from two different hydrodynamic models. The model comparisons and conclusions are presented within this memo.

3 Model results

The two hydrodynamic models which have been compared with bathtub modelling are the T+T (2017) TUFLOW model used for the previous coastal hazard assessment, and the CCC city-wide flood model which is used for a range of purposes (such as setting minimum floor levels). A description of the models and their assumptions is outlined below.

3.1 T+T (2017) TUFLOW model

T+T (2017) utilised a hydrodynamic model based on a surface water drainage model previously developed for the purpose of flood level estimation in response to rainfall events. Instead of allowing the model to respond to rainfall inputs, the revised model was run with zero rainfall (and hence zero inflow from rivers, drains, streams etc to the estuary) and was used to assess hydrodynamic response to storm tide applied at the seaward boundary. The "zero inflow" assumption was arrived at through agreement, in recognition of this being a simplification of the likely response. It was recognised that extreme sea level conditions were likely to occur concurrently with some rainfall, and whether or not this rainfall would be statistically significant was not able to be confirmed by analysis of past records with the period of record being of insufficient length. This brought into question the joint probability between rainfall and sea level. While a joint probability approach is suggested in the CCC WWDG, there is difficulty in attempting to simulate these events hydrodynamically. The reason for this is the timing between high tide and peak flow. Each of the waterways that contributes flow to the estuary is likely to have its own time of concentration, and it would not be possible to simulate a single tidal time series to make high tide occur concurrently with peak flow from all waterways. Sensitivity assessment indicated that peak water levels close to the coast are dominated by tidal conditions, and that further from the coast the peak levels would be dominated by surface flow. It was recognised that there is a margin within which the combination of flow and sea level gives rise to peak levels, but this was knowingly simplified in the 2017 modelling undertaken.

The model was constructed using the TUFLOW software package. The model includes the Avon, Heathcote and Styx catchments (Figure 3.1). Also shown in Figure 3.1 are the locations of key water level recording sites. For the model terrain, a bare earth digital elevation model (DEM) at 2 m resolution was created using a combination of LiDAR and estuary bathymetry files stitched together to make one DEM. The majority of the model used LiDAR data collected following the December 2011 Christchurch earthquake. Where LiDAR data were not available the model used LiDAR flown following the June 2011 Earthquake. The Avon-Heathcote Estuary bathymetry was based on surveys from March/April 2011 and January 2013 by NIWA.

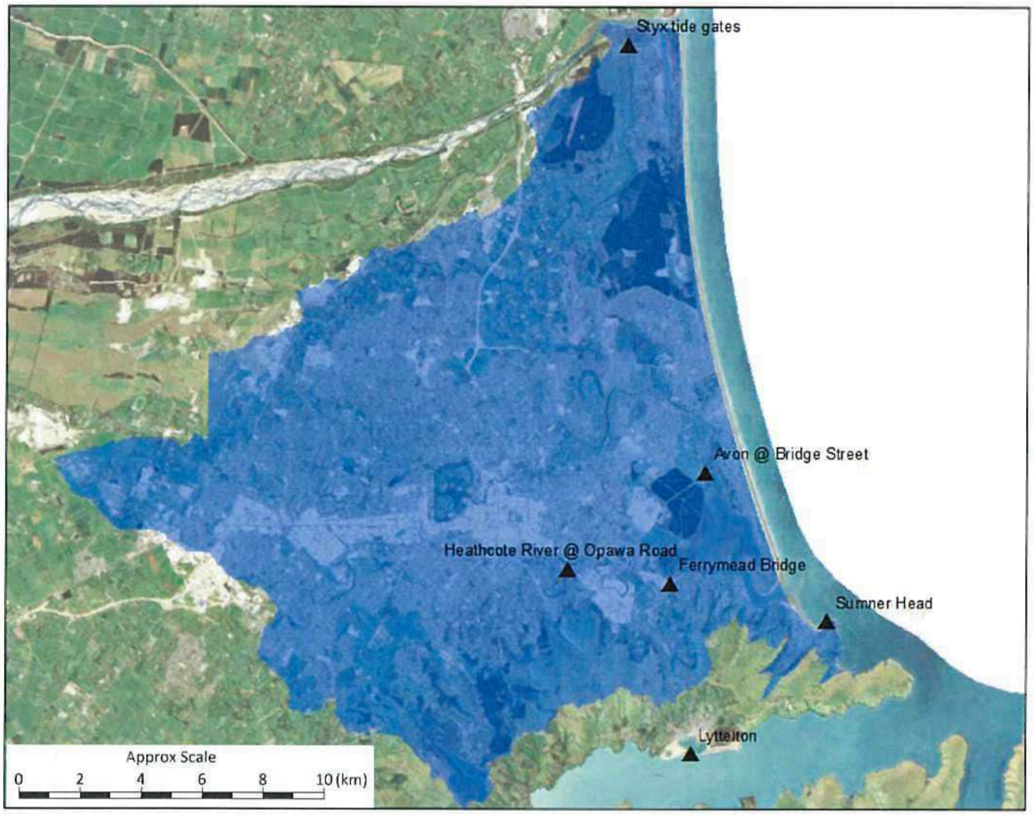


Figure 3.1: Extent of T+T (2017) TUFLOW model.

T+T (2017) applied a dynamic sea level as a downstream boundary condition, using a "building bock" approach which included the effects of astronomical tide, storm surge, an allowance for wave set up over Sumner Bar and sea level rise for future timeframes.

An important consideration with this modelling approach is that the results were mapped with a zero rainfall assumption. This means that there was no flow in the waterways that drain towards the estuary and Brooklands Lagoon at the time of the extreme sea level event. Given that storm surge is a contributor to extreme sea level, and that same storm could also cause rainfall at the same time, it is possible for a rainfall event of some magnitude to occur concurrently with the extreme sea level. In CCC guidance[1] the joint probability between rainfall and extreme sea level is specified, but due to differing response times of the many freshwater inflows, it is difficult to simulate these such that peak discharge and peak sea level occur concurrently. This is why this was not undertaken for the 2017 hydrodynamic modelling approach.

[1] CCC (2003), *Waterways, Wetlands and Drainage Guide – Ko Te Anga Whakaora mo Nga Arawai Repo*, Part B: Design, Christchurch City Council, February 2003.

3.2 CCC city-wide flood model

CCC have been undertaking the City-wide Flood Modelling Project (GHD, 2018). The main aims of the project are to increase the level of detail and produce an integrated city-wide model that includes the Avon, Heathcote, Parklands, Sumner, Styx and Halswell River catchments. Only the Avon catchment model has been provided for this model comparison (Figure 3.2).

In development of this CCC city-wide flood modelling, the models are set up to assess the contributions from both extreme sea level and statistically significant rainfall. Note that this differs from the T+T (2017) TUFLOW modelling approach, which assumed zero rainfall.

The current model is an 'existing' post-quake model calibrated with the March 2014 flood event. The current model is based on 2011 LiDAR, however where there were significant changes between the LiDAR data and the March 2014 model, modifications are understood to have been made. At the coastal boundary where there is no LiDAR the terrain has been artificially extended as a 2% slope down to below -3 mRL (LVD37). This is to ensure it extends below the low tide level of approximately -1 mRL (LVD37). The minimum mesh element is $12m^2$ (road width) and the maximum mesh element is $200m^2$ (flat land).

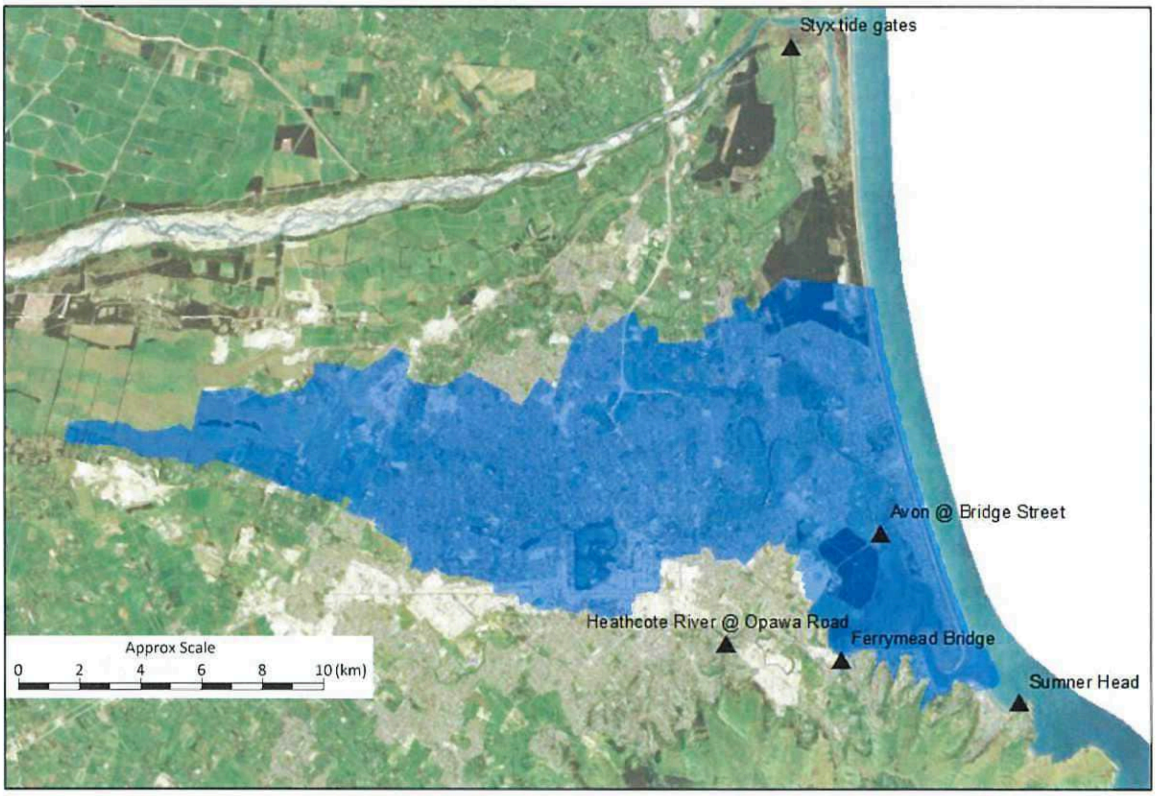


Figure 3.2: Extent of Avon catchment within the city-wide flood model.

The results from this model include both extreme sea level and input rainfall, such that there is not zero freshwater inflow to the coastal areas under design event conditions.

Using CCC guidance[2], the extreme flood levels in coastal areas can be influenced both by extreme rainfall and by extreme sea level. The guidance sets out the differing event likelihoods that should be combined to produce extreme flood level estimates. For example, to establish extreme flood level in response to a 1%AEP event, two separate events are specified, as follows:

- 1%AEP rainfall event, combined with 10%AEP sea level event
- 10%AEP rainfall event, combined with 1%AEP sea level event

The maximum water levels reached across the envelope of the two events above are combined, to yield the 1%AEP water levels. This approach is often termed "max-of-max", where the results are enveloped.

Rainfall is generally applied to the model as "direct rainfall" or "rain-on-grid" rainfall. This means that every cell in the model receives rainfall and can therefore be classified as "wet". For this reason it is necessary to adopt a depth threshold, below which flood depths are not considered relevant. In most cases any flood depths predicted, at maximum, to be less than 0.1 m are deleted with only cells where predicted flood depth exceeding 0.1 m being shown to be flood affected.

3.3 Bathtub model

The bathtub model identifies all areas that are below a defined water level connected to the coastline and characterises the depth at these locations. The bathtub also identifies non-connected areas which are below a defined water level. These areas are typically low-lying areas which may be susceptible to flooding through groundwater or through impeded surface water outlets.

The model is driven by a water level which is deemed representative of the inundated areas, in this case from water level gauges at Bridge St, Ferrymead Bridge and the Styx tide gates. Levels identified here will include components driving water levels such as tide, storm surge, river flows, rainfall, local wind effects and wave breaking over the Sumner and Waimakariri Bars. Extreme value analysis undertaken on these water levels implicitly includes these components without them having to be separated out and analysed separately in terms of their magnitude and joint likelihood of occurrence, as would be required for hydrodynamic modelling.

The downside is that only one level is identified for each catchment and so if this level varies significantly across an area in reality, the bathtub approach may under- or over-estimate flooded extents. However, the bathtub approach enables areas inundated under a range of water levels to be rapidly identified which is useful for engagement and adaptation planning where effects of incremental changes in event likelihood and sea level rise are of interest.

The bathtub model used for the following comparisons is based on the latest 2018 topographic LiDAR data which has been sourced as a 1m DEM (Figure 3.3).

[2] CCC (2003), *Waterways, Wetlands and Drainage Guide – Ko Te Anga Whakaora mo Nga Arawai Repo*, Part B: Design, Christchurch City Council, February 2003.

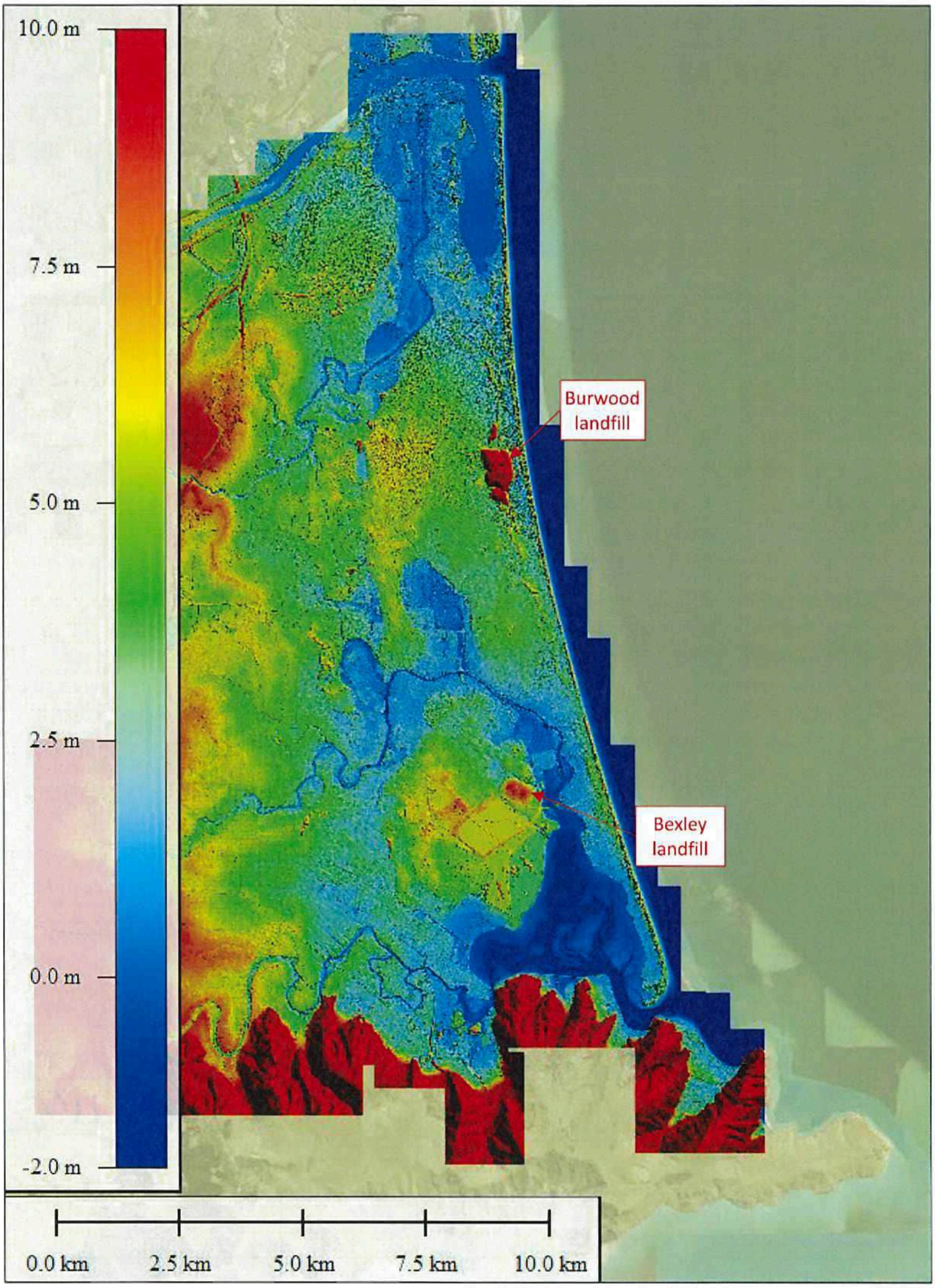


Figure 3.3: Example of 2018 DEM used for bathtub model.

4 Model comparisons

4.1 Comparison methodology

In order to assess the suitability of the bathtub approach for its intended purpose, comparisons were made against both hydrodynamic models. To cover the range of likely outcomes, sample results were taken for both low and high sea level rise scenarios from each of the hydrodynamic models.

Scenarios used for comparison are summarised in Table **4-1**. Peak water levels have been extracted from the hydrodynamic model outputs at three water level gauge locations (Bridge St, Ferrymead Bridge and Styx tide gates) (Table **4-1**). Based on the peak water levels from the two models for each scenario, an equivalent level was then used to drive the connected bathtub model. For the purposes of the comparison presented in this memo, which focusses on the Avon catchment results from the citywide flood model, the same level was applied in all catchments with a greater weighting given to the Bridge St level when setting this equivalent level. For the TUFLOW high sea level rise scenario the water level at Ferrymead Bridge is 0.2 m higher than the water level at Bridge St. For this scenario, a different bathtub level (3.2 m RL) was adopted for the Heathcote catchment. For the final bathtub analysis, levels will be selected separately for each of the three catchments.

Bathtub depths and extents connected to the coast were derived and filtered to show inundation extents for depths greater than 0.1 m. This was to make inundation extents comparable with both the TUFLOW and city-wide hydrodynamic model outputs (which use the same filtering, as discussed in Section 3.2).

For the CCC city-wide flood model, "max of max" water level raster files were provided by CCC for the Avon catchment. As the model results include rainfall it is not directly comparable with the bathtub model (i.e. flooding on every grid cell). The level from the city-wide model was converted to a depth by subtracting the model terrain. The estimated depths were then filtered to show depths greater than 0.1 m. The resultant file presented for model comparison is the maximum water level for areas where depths are greater than 0.1 m and are connected to the coastal margin or Avon River. Due to several differences in the model input and assumptions, the comparison between the bathtub model and city-wide flood model only provides an indicative comparison.

Table 4-1 Scenarios and water levels for comparison between bathtub and hydrodynamic models

Hydrodynamic model	Scenario	Peak water level within hydrodynamic model (m LVD37)			Adopted water level for bathtub model (m LVD37)
		Bridge St	Ferrymead Bridge	Styx tide gates	
T+T (2017) TUFLOW	Low sea level rise 2065 1% AEP RCP4.5	2.5	2.5	2.5	2.5
	High sea level rise 2115 1% AEP RCP8.5H+	3.0	3.2	3.1	3.0[1]
CCC (2020) city-wide flood model, Avon catchment	Present-day 0.2% AEP 0 m SLR	1.7	1.7	N/A	1.7
	High sea level rise 0.5% AEP 1.88 m SLR	4.0	3.9	N/A	4.0

[1]3.2 m bathtub scenario adopted for Heathcote catchment

4.2 DEM comparison

The three different models presented in this memo each use different ground elevation models, because they were developed at different times using the information then available. Slight differences in the DEMs used for each model are likely to contribute to some differences between the model outputs. The vertical accuracy of the 2018 DEM is +/-0.2m and the average difference between the 2018 DEM and the TUFLOW terrain is approximately 0.2m (Figure 4.1). The key differences are through Bottle Lake Forest (south of Brooklands Lagoon) and around the oxidation ponds near the Avon-Heathcote estuary, where the 2018 DEM is on average 0.2 m lower than the TUFLOW terrain.

There are also differences between the city-wide flood model terrain and the 2018 DEM (Figure 4.2). The typical difference is +/-0.2m with some of the key differences occurring due to filling associated with motorway, landfill and subdivision earthworks; along the stopbanks of the Avon River (possibly due to "burning-in" of stopbank crest levels); the Port Hills (possibly erroneous vertical difference caused by horizontal misalignment of the LiDAR survey over steep ground); and areas of changing vegetation in Travis Wetland and plantation forests between Burwood and Kainga.

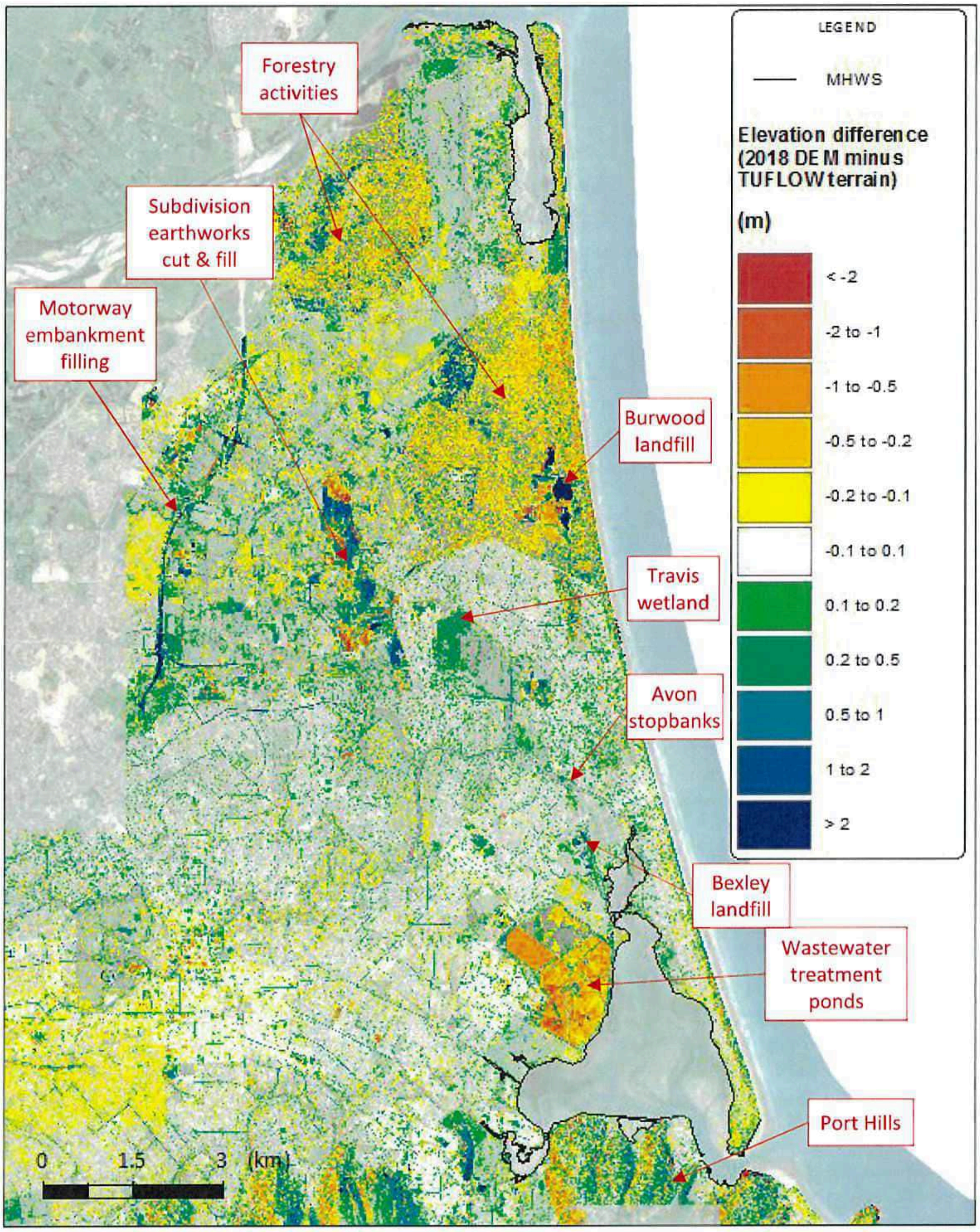


Figure 4.1: Elevation difference between the 2018 DEM used for the bathtub modelling and the terrain grid used within the TUFLOW model.

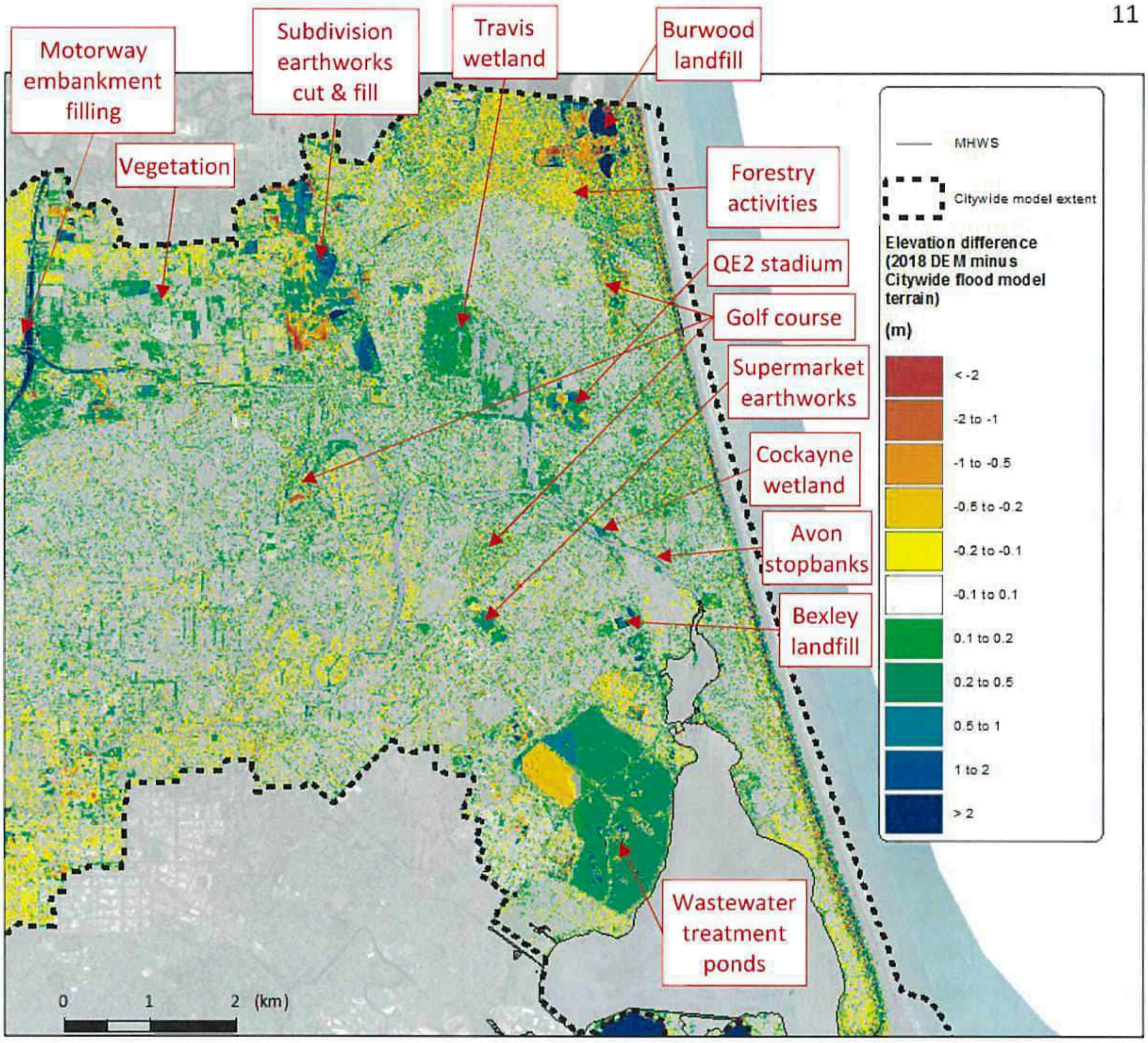


Figure 4.2: Elevation difference between the 2018 DEM used for the bathtub modelling and the terrain grid used within the CCC city-wide flood model.

5 Results

The results from the model comparisons are presented in Appendix A.

5.1 Comparison with T+T (2017) TUFLOW model

5.1.1 Low sea level rise scenario

In the coastal areas (i.e. downstream of Wainoni Road on the Avon River and downstream of Radley St on the Heathcote River) the bathtub model shows good agreement with the T+T (2017) TUFLOW model results. Further upstream of the Avon and Heathcote Rivers there are some differences between the bathtub and TUFLOW model results. These differences are as expected.

For the low SLR scenario (2065 RCP4.5) there is some difference upstream of Wainoni Road on the Avon River where the bathtub model overestimates the inundation extent through some of the low-lying areas around Avondale, Dallington and Linwood (Figure 5.1) compared to the TUFLOW model. Through Dallington the bathtub extent is approximately 350 m further than the TUFLOW inundation extent. These differences are largely due to the lower water elevations reached by the hydrodynamic model in the upstream limits. For example, the TUFLOW model indicates the water level reduces to approximately 2 m LVD37 through Linwood and Richmond, which is 0.5 m less than the water level at Bridge St and subsequently the level used for the bathtub (i.e. the bathtub overestimates inundation depth by up to 0.5 m in some areas upstream of Wainoni Rd).

Compared to the TUFLOW model, the bathtub also overestimates the inundation extent upstream of Radley St on the Heathcote River (Figure 5.2). The TUFLOW model indicates levels within the Heathcote River reduce to approximately 2 m LVD upstream of Rutherford St (Figure 5.2).

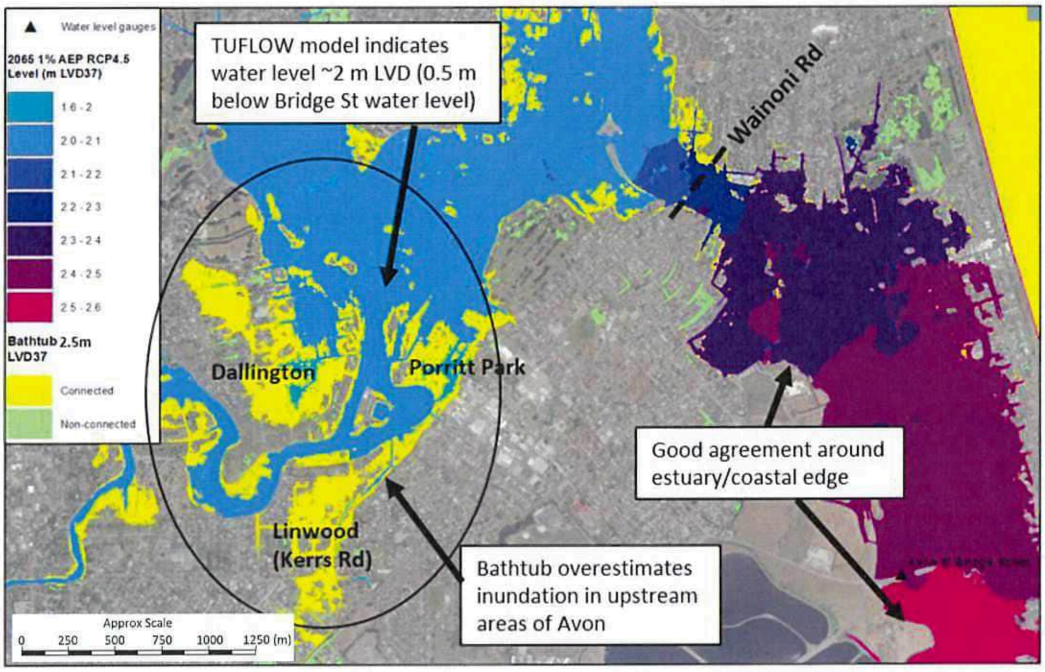


Figure 5.1: Comparison of bathtub and T+T TUFLOW results for a low SLR scenario (2065 1% AEP RCP4.5). Key areas of difference along Avon River (yellow and green shading are where bathtub flood extent is larger).

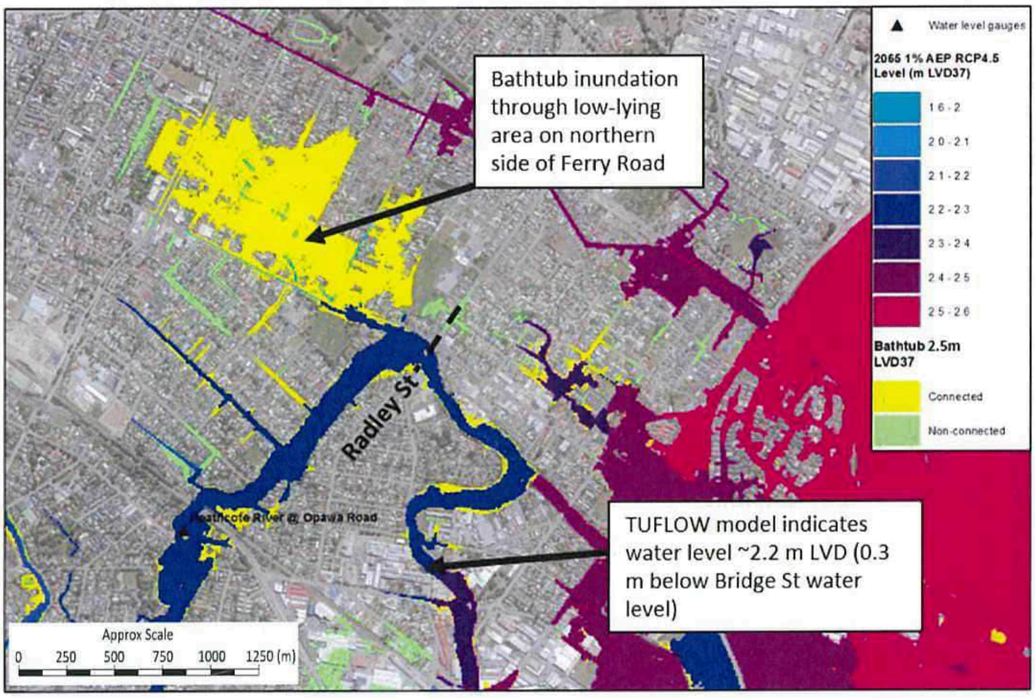


Figure 5.2: Comparison of bathtub and T+T TUFLOW model results for a low SLR scenario (2065 1% AEP RCP4.5). Key areas of difference along Heathcote River (yellow and green shading are where bathtub flood extent is larger).

5.1.2 High sea level rise scenario

For the high SLR scenario (2115 RCP8.5) the differences between hydrodynamic model and bathtub are substantially less. The bathtub slightly overestimates the inundation extent through Linwood compared to the TUFLOW model. The largest difference occurs along Gloucester Street where the bathtub inundation extent is up to 100 m further than the TUFLOW inundation extent (Figure 5.3). Again, this difference is due to reduction in the water elevation upstream from the coastal margin. The TUFLOW model shows water levels reducing to 2.7 m LVD37 which is 0.3 m below the Bridge St level.

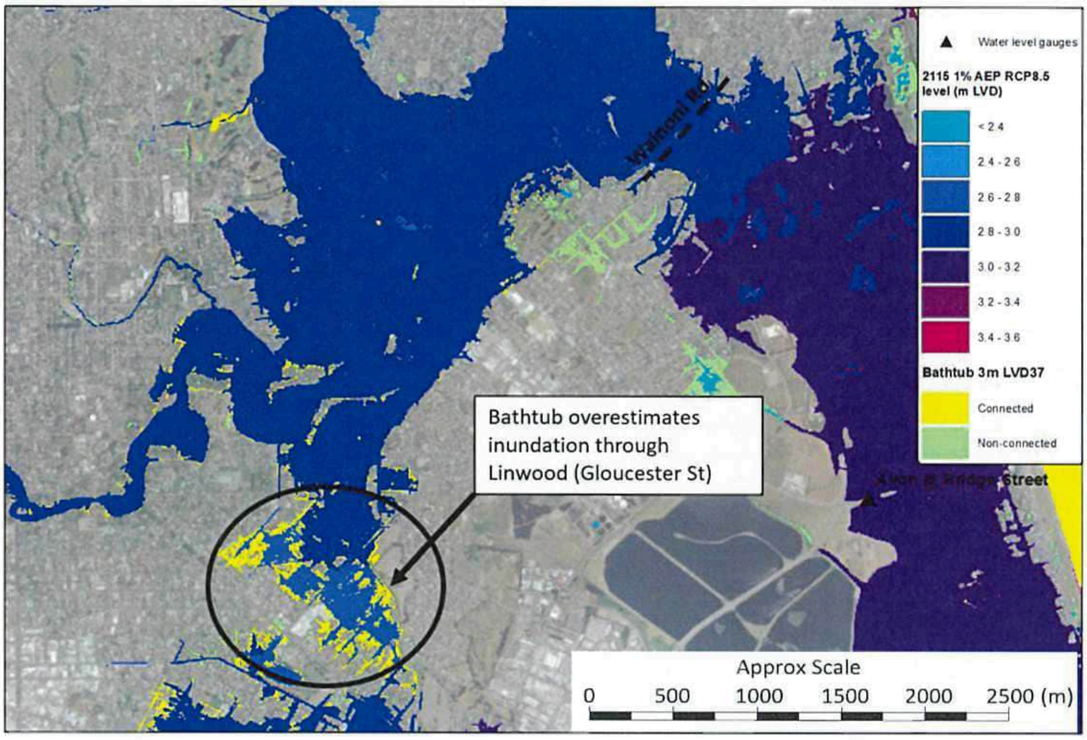


Figure 5.3: Comparison of bathtub and T+T TUFLOW model results for a high SLR scenario (2115 1% AEP RCP8.5). Key areas of difference along the Avon River (yellow and green shading are where bathtub flood extent is larger).

The bathtub also overestimates the inundation extent compared to the TUFLOW model (by up to 1 km) at the upstream limit through Bottle Lake Forest south of Brooklands Lagoon, and Chaneys Plantation west of Brooklands Lagoon (Figure 5.4). However, the areas inundated by the bathtub model are patchy indicating very low and uneven terrain (forested dunes). The TUFLOW model indicates the water level reduces rapidly across the uneven terrain. Over a horizontal distance of approximately 600 m the water level reduces from 3 m LVD37 to 2.7 m LVD37 which is 0.3 m less than the water level near the Styx tide gates and subsequently the bathtub level. The 2018 DEM used for the bathtub inundation is also approximately 0.2 m lower than the TUFLOW terrain through Bottle Lake Forest and therefore the bathtub inundation is expected to extend further landward.

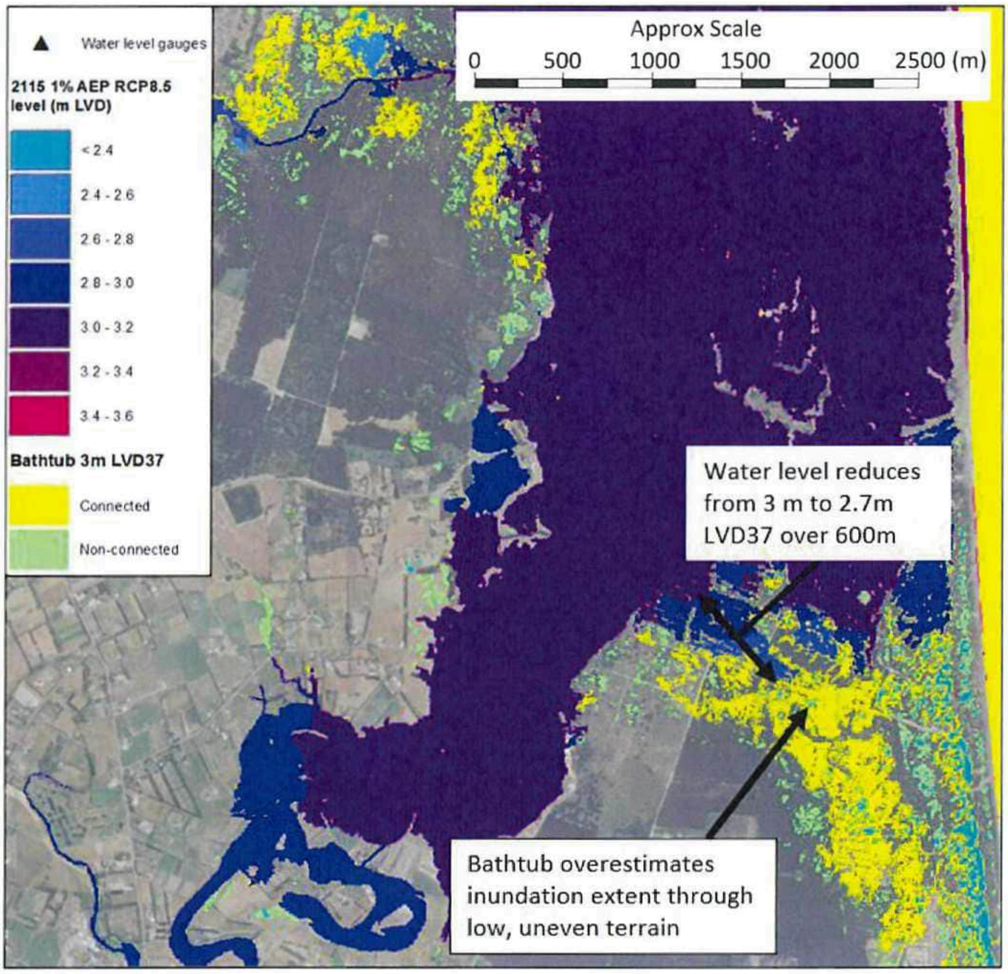


Figure 5.4: Comparison of bathtub and T+T TUFLOW model results for a high SLR scenario (2115 1% AEP RCP8.5). Key areas of difference near Brooklands Lagoon (yellow and green shading are where bathtub flood extent is larger).

On the Heathcote River, the higher SLR scenario generally shows good correlation with the TUFLOW model results (Figure 5.5). The TUFLOW results indicate the water levels reduce to approximately 2.9 m RL upstream of Radley Street which is 0.3 m lower than the water level at Ferrymead Bridge (3.2 m RL). Subsequently the bathtub overestimates the inundation extent by up to 130 m through parts of Phillipstown.

Overall, the bathtub shows better correlation with TUFLOW results for the higher SLR scenario. This is because under higher water levels the hydraulic controls in the catchments have less influence on dampening the upstream levels. For the lower SLR scenarios the bathtub overestimates the upstream inundation levels by approximately 0.3 to 0.5 m for the Heathcote and Avon catchments, respectively. Whereas for higher SLR scenarios the bathtub overestimates the upstream inundation levels by approximately 0.3 to 0.4 m for the Heathcote and Avon catchments, respectively. It should be noted that the upstream areas where these differences are shown, are excluded from the coast hazard maps which are the focus of this study.

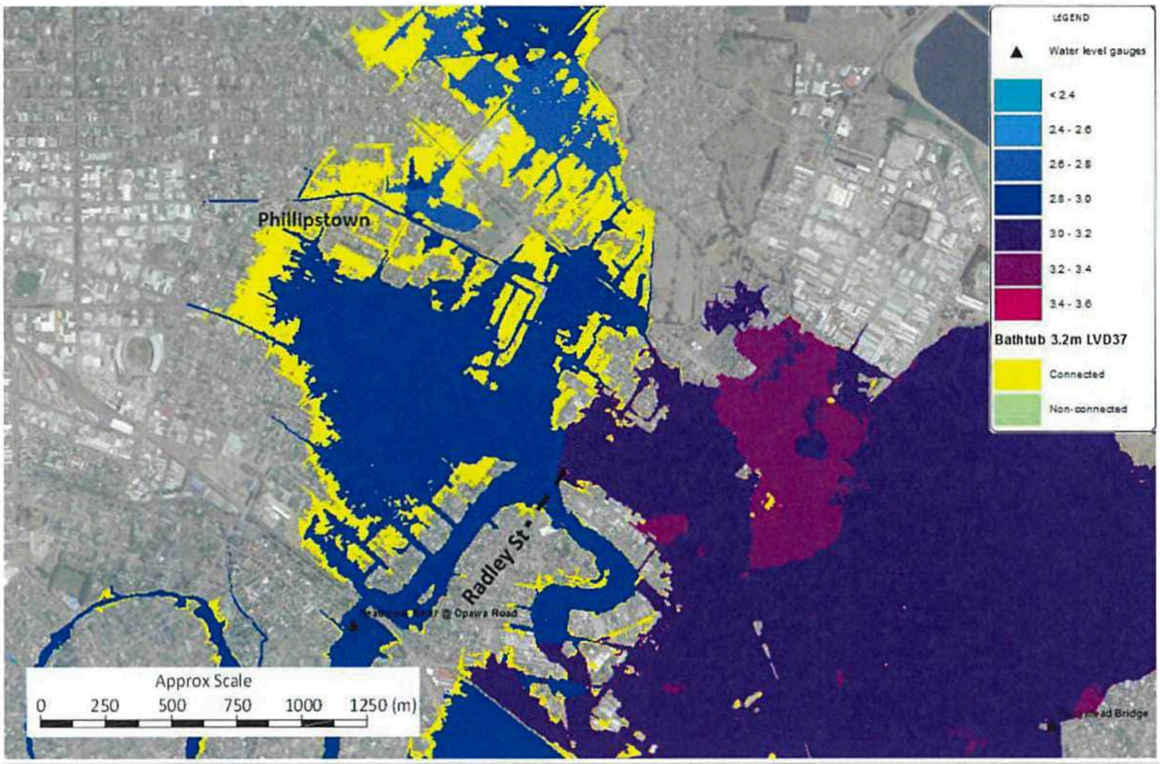


Figure 5.5: Comparison of bathtub and T+T TUFLOW model results for a high SLR scenario (2115 1% AEP RCP8.5). Key areas of difference along the Heathcote River (yellow and green shading are where bathtub flood extent is larger)

5.2 Comparison with CCC city-wide flood model

5.2.1 Present day scenario

In the coastal areas the bathtub model generally shows good correlation with the city-wide flood model for a present-day scenario (Figure 5.6).

The main difference is that the bathtub is non-connected in the areas where the city-wide flood model shows connected inundation (i.e. Travis Wetland, Horseshoe Lake Reserve, Avondale Park and Bexley). This difference is due to the bathtub model not including inundation via culverts or other below-ground infrastructure. While the bathtub does not identify it as being connected inundation, the extent of non-connected inundation is generally consistent with the extent of inundation from the city-wide model.

One other area of difference is through Bexley where the bathtub slightly overestimates the extent of non-connected inundation by up to 100 m and the inundation level by approximately 0.3 m.

The high water levels (>2 m LVD37) shown in pink in Figure 5.6 are likely to either be rainfall-driven ponding or surface water flow influenced. Both of these flood mechanisms are not considered to be "coastal inundation", and it is suggested that differences shown in these areas between the city-wide and bathtub models is not relevant for the coastal adaptation planning purposes of the current CHA study. These mechanisms responsible for these differences are also likely to exist in the Heathcote and Styx catchments, and the conclusion where these differences are deemed not relevant for coastal adaptation would also apply to these areas.

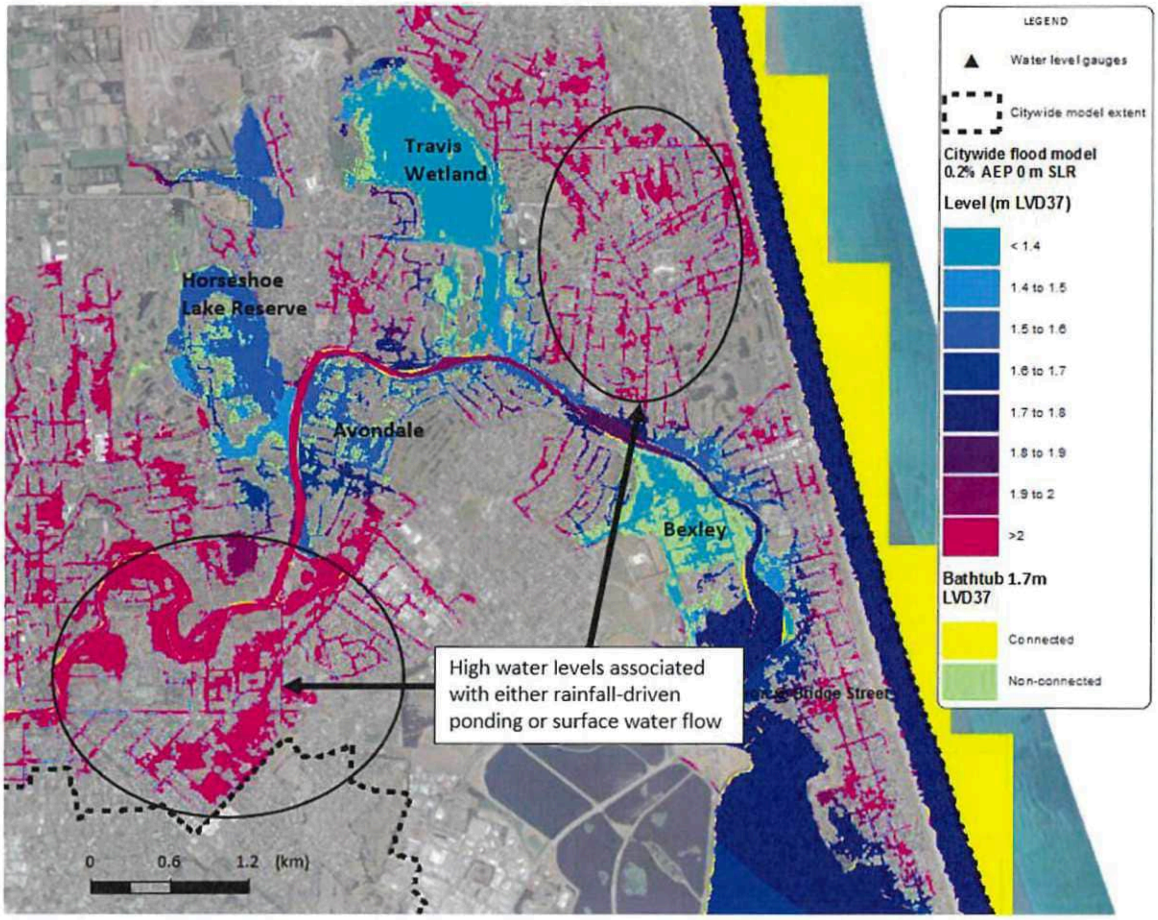


Figure 5.6: Comparison of bathtub and CCC city-wide flood model results for a present-day scenario (0.2% AEP 0 m SLR). Key areas of difference for Avon catchment (yellow and green shading are where bathtub flood extent is larger).

5.2.2 High sea level rise scenario

For a high SLR scenario the bathtub model shows good agreement with the city-wide flood model downstream from Wainoni Road. The city-wide flood model shows approximately a 0.5 m reduction in peak water level between Bridge St and just North of Wainoni Rd. The bathtub does not account for this reduction in water level and subsequently the 4 m LVD37 bathtub overestimates the extent of inundation upstream of Wainoni Rd, such as through North New Brighton, Burwood and Shirley (Figure 5.7). Inundation extents from the bathtub model are up to 500 m landward of the inundation extents from the city-wide flood model. Similar effects are anticipated in the Heathcote and Styx catchments, although the horizontal extent differences will be dependent on local ground slope in each case (ie not necessarily the same 500 m difference in extent, but level differences would be similar).

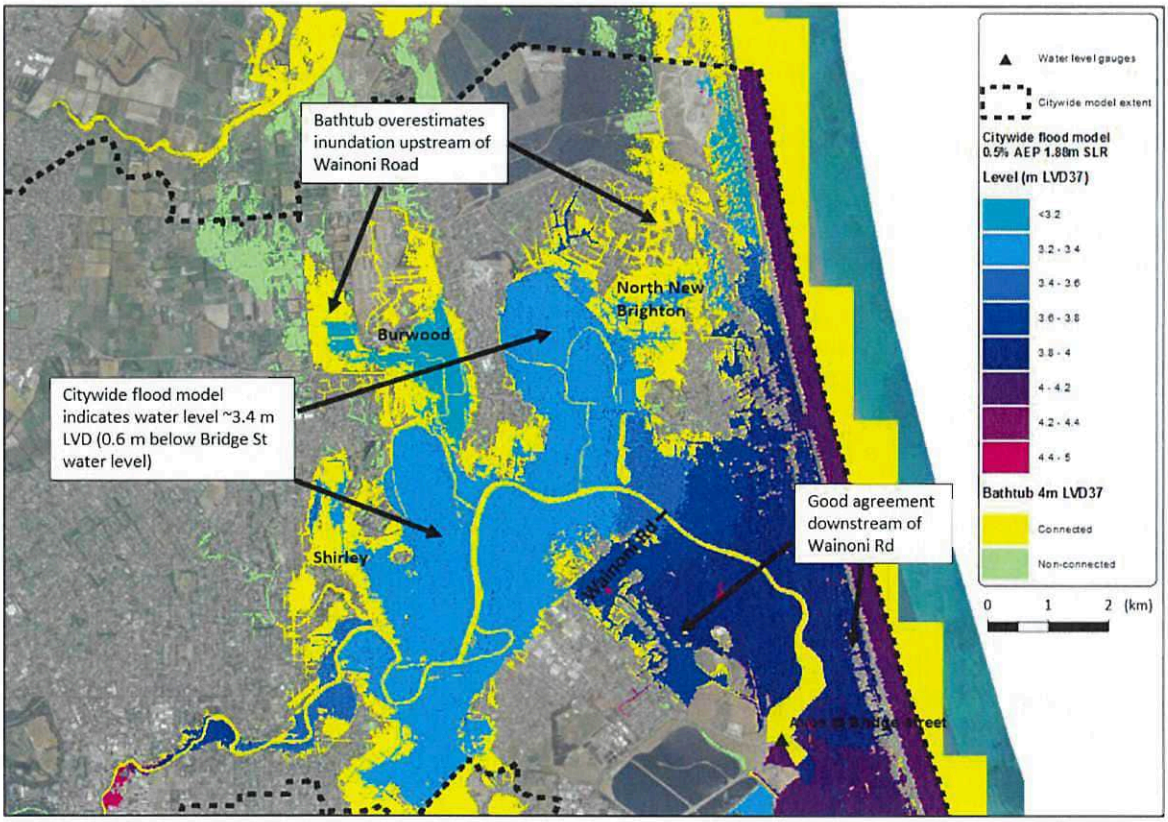


Figure 5.7: Comparison of bathtub and CCC city-wide flood model results for a high SLR scenario (2150 0.5% AEP 1.88 m SLR). Key areas of difference for Avon catchment (yellow and green shading are where bathtub flood extent is larger).

6 Conclusions

6.1 Recommended boundary for bathtub model output

In both the Avon and Heathcote catchments there are locations at which hydraulic control appears to notably affect the inland propagation of coastal inundation. Such hydraulic control would ordinarily tend to suggest that a hydrodynamic modelling approach would be preferred over a bathtub approach. This effect is common to both the city-wide and TUFLOW model results, at similar locations on both of these rivers.

On the Avon River the hydraulic control is approximately around Wainoni Road and on the Heathcote River it is near Radley Street. In these locations the flood plains narrow and subsequently there is a significant reduction in the water levels (via throttled flow). Upstream of the hydraulic controls the bathtub model generally overestimates the extent of inundation because it applies a water level derived at the coast which is too high for the area further inland.

The bathtub model tends to overestimate the landward extent slightly more on the Avon catchment compared with the Heathcote and Styx catchments. This is partly due to the hydraulic controls being less significant on the Heathcote and Styx catchments, but is also linked to local ground elevations and slopes. Similarly, the bathtub is most similar to the hydrodynamic results for the higher SLR scenario compared with the low SLR scenario. This is due to the hydraulic controls having less influence on the higher water levels.

In pursuit of a simple approach suitable for exploring a range of scenarios for adaptation planning, and on the simplification of there being just a single hydraulic control on both river systems, we have identified a boundary where we recommend the bathtub model outputs (e.g. maps) are cut off for the current CHA study. This boundary is shown in red in Figure 6.1 to Figure 6.3. At this boundary the difference between the water level in the bathtub model and hydrodynamic models varies between approximately 0.2m and 0.4m for the various scenarios and models.

Inland of these boundaries the CHA maps would be blanked out, with a note explaining that the interaction between rainfall and sea level rise was more complex in this inland area and so the city wide-flood model is the more appropriate source of information (e.g. via the CCC floor level viewer).

Even though the inland area wouldn't be shown on the maps in the final CHA report, the analysis results for this area would still be available for assessment if needed for some reason (e.g. to identify lower-lying parts of the CHAP adaptation engagement areas). In this case the results for the inland area would need to be used with careful technical guidance and an appreciation that there is increased uncertainty in the hydraulics so the extent and depth of inundation for a given scenario could be overstated. There may also be situations where it could be useful to create a separate bathtub model specifically for the inland area (e.g. using an inland level 0.5m lower than at Bridge Street). In inland areas, recognition of rainfall and surface flow contributions to extreme flood levels needs to be given, and in instances where high precision is required, a site specific peak flood level analysis may be required and would be recommended.

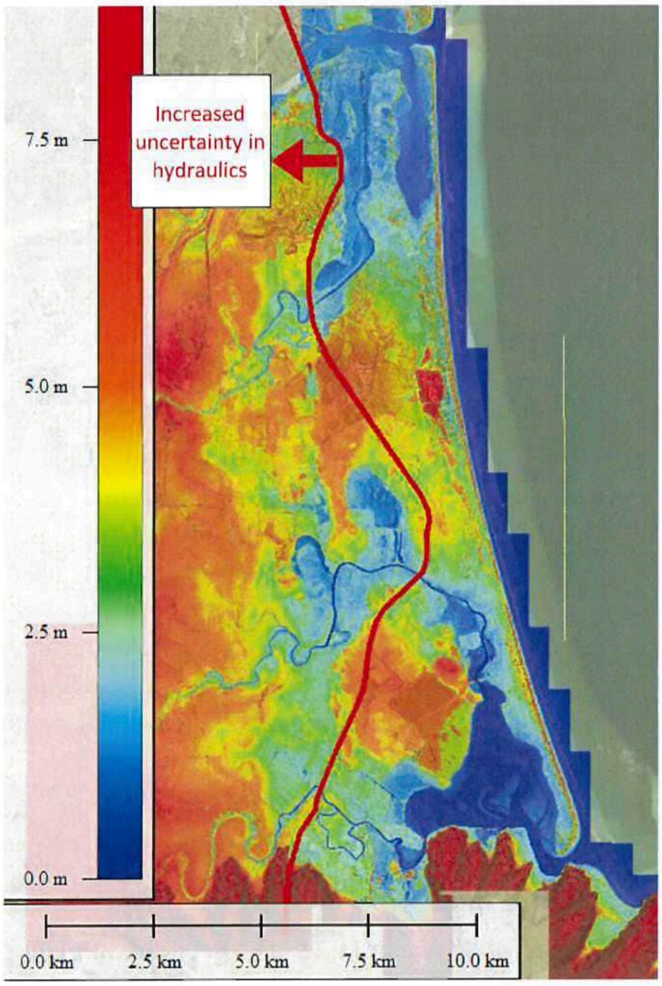


Figure 6.1: 2018 DEM used for bathtub model with the recommended bathtub boundary shown in red.

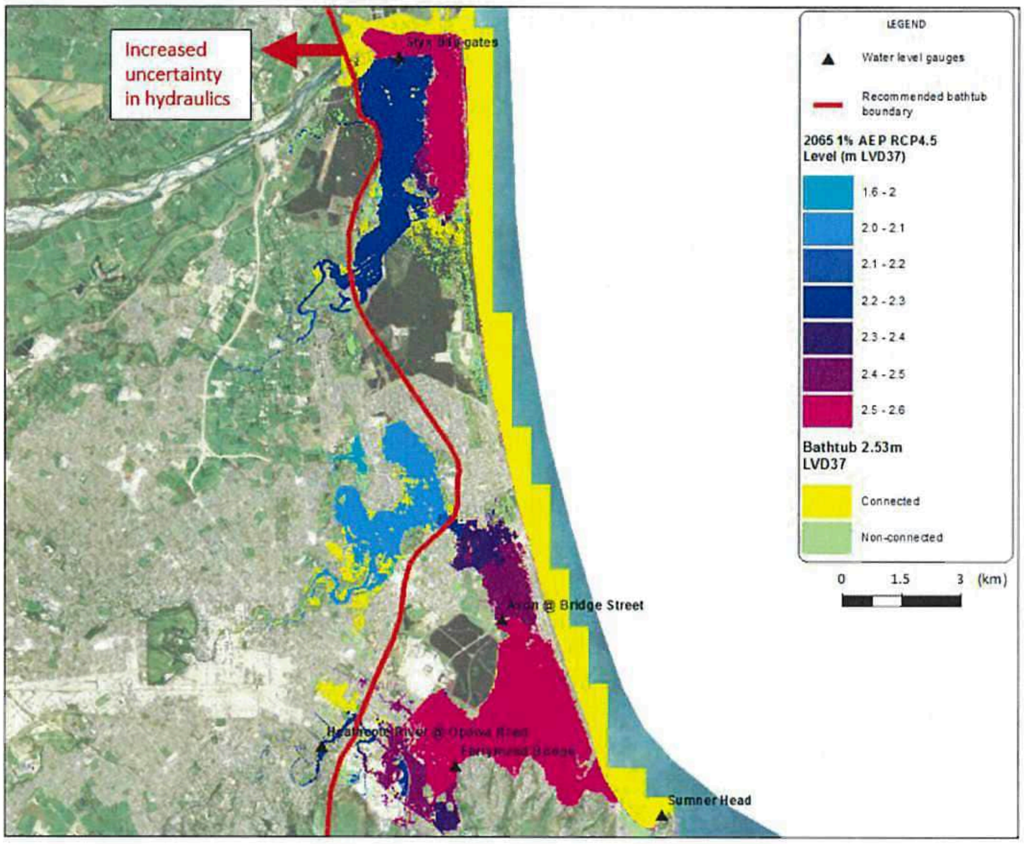


Figure 6.2: TUFLOW and bathtub model comparison with the recommended bathtub boundary shown in red

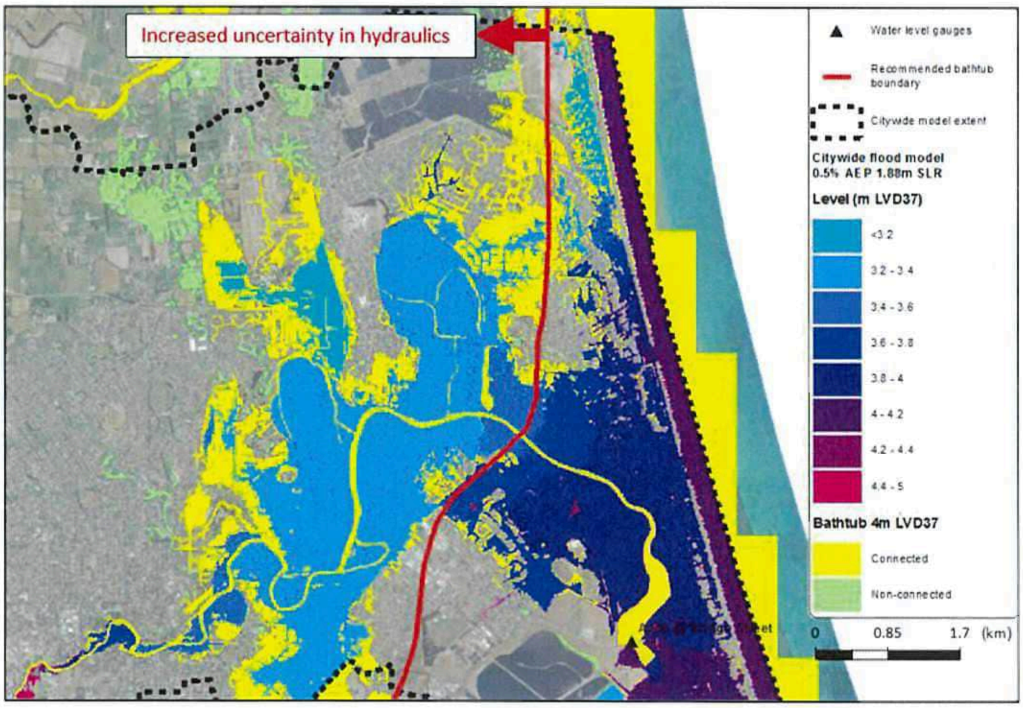


Figure 6.3: CCC city wide flood model and bathtub model comparison with the recommended bathtub boundary shown in red

6.2 Use of bathtub model for current adaptation planning purposes

Overall, there are some differences between the inundation extents derived using a bathtub approach with those derived using hydrodynamic modelling. These differences are negligible near the coastal edge (the Avon-Heathcote Estuary and Waimakariri River) and typically increase with distance inland. The primary reason for this difference is the reduction in water levels away from the coastline which occurs in the hydrodynamic model but is not allowed for in the bathtub modelling. It should be noted that the T+T (2017) TUFLOW modelling did not include river flows or rainfall, but the city-wide flood model does. These contributions may elevate the water levels away from the coast, particularly along the Avon and Heathcote Rivers, partially offsetting this difference. It is also noted that these differences are more pronounced at lower sea level rise scenarios and less pronounced at higher sea level rise scenarios.

Given the intended purpose of the current adaptation planning work and the large number of scenarios to be considered, the bathtub method appears to provide a suitable approach if the limitations are understood and accepted. The bathtub approach enables areas inundated under a range of water levels to be rapidly identified which is useful for engagement and adaptation planning where effects of incremental changes in event likelihood and sea level rise are of interest. For these purposes it is important to explore a wide range of uncertainties in the analysis inputs and outputs, and these uncertainties often have much larger impact on objectives and decision making than differences in modelled flood levels as a result of a more simplified analysis. This means that higher precision in the modelling would provide little, if any, meaningful benefit for engagement and adaptation purposes. More precise modelling might instead bring disadvantages for the adaptation project, if it limited the scope of analysis which could be practically undertaken, or the flexibility to respond quickly to requests for further information to explore particular scenarios of interest.

Furthermore, by basing levels on the most recent extreme values analysis of water level gauges within the estuary and lagoon at Bridge Street, Ferrymead and the Styx, the combined effects of tide, storm surge, river flows, rainfall, local wind effects and wave breaking over the Sumner and Waimakariri Bars are implicitly included in the derived extreme values and they do not need to be defined separately by joint probability analysis. This reduces the number of technical assumptions which might be subject to challenge (e.g. potential "weak links" in the analysis chain) or become superseded by future changes in agreed methodology or extreme water level frequencies, which could unnecessarily undermine public confidence in the results of the coastal hazard assessment.

A comprehensive assessment of joint probability and the various forcing factors is currently underway within the Land Drainage Recovery Programme and could be implemented within the Christchurch city-wide flood model once assessments are complete and there is widespread agreement on the technical assumptions. These more comprehensive models could be used in future stages of the adaptation planning work if more detailed site-specific analysis is required for a particular assessment (e.g. to help understand the effect of a proposed flood protection structure).

6.3 Summary of recommendations

We recommend that:

1. The current coastal hazard assessment utilises a connected bathtub approach based on extreme levels derived for Bridge Street, Ferrymead and the Styx with connected and non-connected areas defined.
2. Due to potential over-estimation of inundated areas upstream of the identified hydraulic control locations, the maps in the final CHA report only show the bathtub model results for the areas downstream of these locations.
3. The specific purpose and limitations of this modelling are clearly communicated, so it is understood that if more precise site-specific flood level information is required for other purposes (e.g. setting Building Consent floor levels, or detailed design of flood protection options as part of more detailed site-specific adaptation planning in future) it would be more appropriate to refer to detailed hydrodynamic models such the city-wide flood model.

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Appendix D: Coastal inundation levels

- Christchurch open coast inundation levels:
  - Appendix D Table 1 - Appendix D Table 6:
- Major harbours and estuaries inundation levels:
  - Appendix D Table 7 - Appendix D Table 11
- Regional hazard screening sites inundation levels:
  - Appendix D Table 12 - Appendix D Table 16

Appendix D Table 1: Static inundation levels (m NZVD2016) for Christchurch open coast

Return period	Present day	Relative sea level rise (m)							
		0.2	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.8	2.0	2.2	2.4	2.6	3.0	3.2	3.3	3.8
10 year ARI	2.0	2.2	2.4	2.6	2.8	3.2	3.4	3.5	4.0
100 year ARI	2.3	2.5	2.7	2.9	3.1	3.5	3.7	3.8	4.3

Appendix D Table 2: Dynamic inundation levels (m NZVD2016) for Christchurch open coast

Return period	Present day	Relative sea level rise (m)							
		0.2	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	3.8	4	4.2	4.4	4.6	5	5.2	5.3	5.8
10 year ARI	4.2	4.4	4.6	4.8	5	5.4	5.6	5.7	6.2
100 year ARI	4.4	4.6	4.8	5	5.2	5.6	5.8	5.9	6.4

Appendix D Table 3: Static inundation levels (m NZVD2016) for Sumner

Return period	Present day	Relative sea level rise (m)							
		0.2	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.8	2.0	2.2	2.4	2.6	3.0	3.2	3.3	3.8
10 year ARI	2.0	2.2	2.4	2.6	2.8	3.2	3.4	3.5	4.0
100 year ARI	2.3	2.5	2.7	2.9	3.1	3.5	3.7	3.8	4.3

Appendix D Table 4: Dynamic inundation levels (m NZVD2016) for Sumner

Return period	Present day	Relative sea level rise (m)							
		0.2	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	4.5	4.7	4.9	5.1	5.3	5.7	5.9	6.0	6.5
10 year ARI	4.9	5.1	5.3	5.5	5.7	6.1	6.3	6.4	6.9
100 year ARI	5.3	5.5	5.7	5.9	6.1	6.5	6.7	6.8	7.3

Appendix D Table 5: Static inundation levels (m NZVD2016) for Taylor's Mistake

Return period	Present day	Relative sea level rise (m)							
		0.2	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.8	2.0	2.2	2.4	2.6	3.0	3.2	3.3	3.8
10 year ARI	2.0	2.2	2.4	2.6	2.8	3.2	3.4	3.5	4.0
100 year ARI	2.3	2.5	2.7	2.9	3.1	3.5	3.7	3.8	4.3

Appendix D Table 6: **Dynamic inundation levels (m NZVD2016) for Taylor's Mistake**

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	4.5	4.7	4.9	5.1	5.3	5.7	5.9	6.0	6.5
10 year ARI	4.9	5.1	5.3	5.5	5.7	6.1	6.3	6.4	6.9
100 year ARI	5.3	5.5	5.7	5.9	6.1	6.5	6.7	6.8	7.3

Appendix D Table 7: **Static inundation levels (m NZVD2016) for Brooklands Lagoon**

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.4	1.6	1.8	2.0	2.2	2.6	2.8	2.9	3.4
10 year ARI	1.6	1.8	2.0	2.2	2.4	2.8	3.0	3.1	3.6
100 year ARI	1.8	2.0	2.2	2.4	2.6	3.0	3.2	3.3	3.8

Appendix D Table 8: **Static inundation levels (m NZVD2016) for Avon-Heathcote – North**

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.5	1.7	1.9	2.1	2.3	2.7	2.9	3.0	3.5
10 year ARI	1.7	1.9	2.1	2.3	2.5	2.9	3.1	3.2	3.7
100 year ARI	2.0	2.2	2.4	2.6	2.8	3.2	3.4	3.5	4.0

Appendix D Table 9: **Static inundation levels (m NZVD2016) for Avon-Heathcote – South**

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.5	1.7	1.9	2.1	2.3	2.7	2.9	3.0	3.5
10 year ARI	1.6	1.8	2.0	2.2	2.4	2.8	3.0	3.1	3.6
100 year ARI	1.8	2.0	2.2	2.4	2.6	3.0	3.2	3.3	3.8

Appendix D Table 10: **Static inundation levels (m NZVD2016) for Lyttelton Harbour**

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.6	1.8	2.0	2.2	2.4	2.8	3.0	3.1	3.6
10 year ARI	1.7	1.9	2.1	2.3	2.5	2.9	3.1	3.2	3.7
100 year ARI	1.8	2.0	2.2	2.4	2.6	3.0	3.2	3.3	3.8

Appendix D Table 11: Static inundation levels (m NZVD2016) for Akaroa Harbour

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.9	2.1	2.3	2.5	2.7	3.1	3.3	3.4	3.9
10 year ARI	2.1	2.3	2.5	2.7	2.9	3.3	3.5	3.6	4.1
100 year ARI	2.3	2.5	2.7	2.9	3.1	3.5	3.7	3.8	4.3

Appendix D Table 12: Static inundation levels (m NZVD2016) for Banks Peninsula – North

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	2.2	2.4	2.6	2.8	3.0	3.4	3.6	3.7	4.2
10 year ARI	2.5	2.7	2.9	3.1	3.3	3.7	3.9	4.0	4.5
100 year ARI	2.8	3.0	3.2	3.4	3.6	4.0	4.2	4.3	4.8

Appendix D Table 13: Static inundation levels (m NZVD2016) for Banks Peninsula – South

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	2.9	3.1	3.3	3.5	3.7	4.1	4.3	4.4	4.9
10 year ARI	3.4	3.6	3.8	4.0	4.2	4.6	4.8	4.9	5.4
100 year ARI	3.9	4.1	4.3	4.5	4.7	5.1	5.3	5.4	5.9

Appendix D Table 14: Static inundation levels (m NZVD2016) for Wairewa (Lake Forsyth)

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	2.2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10 year ARI	2.6	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
100 year ARI	2.8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Appendix D Table 15: Extreme lake levels (m NZVD2016) for Kaitorete Spit

		Relative sea level rise (m)							
Return period	**Present day**	**0.2**	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	2.6	2.8	3.0	3.2	3.4	3.8	4.0	4.1	4.6
10 year ARI	2.9	3.1	3.3	3.5	3.7	4.1	4.3	4.4	4.9
100 year ARI	3.3	3.5	3.7	3.9	4.1	4.5	4.7	4.8	5.3

Appendix D Table 16: Extreme lake levels (m NZVD2016) for Te Waihora (Lake Ellesmere)

Return period	Present day	Relative sea level rise (m)							
		0.2	0.4	0.6	0.8	1.2	1.4	1.5	2
1 year ARI	1.1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10 year ARI	1.4	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
100 year ARI	1.7	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Appendix E: Example maps

- **Coastal erosion maps**
- **Coastal inundation maps**
- **Rising groundwater maps**

To see the full suite of maps for the various scenarios analysed, use the online map viewer at https://ccc.govt.nz/environment/coast/coastalhazards/2021-coastal-hazards-assessment

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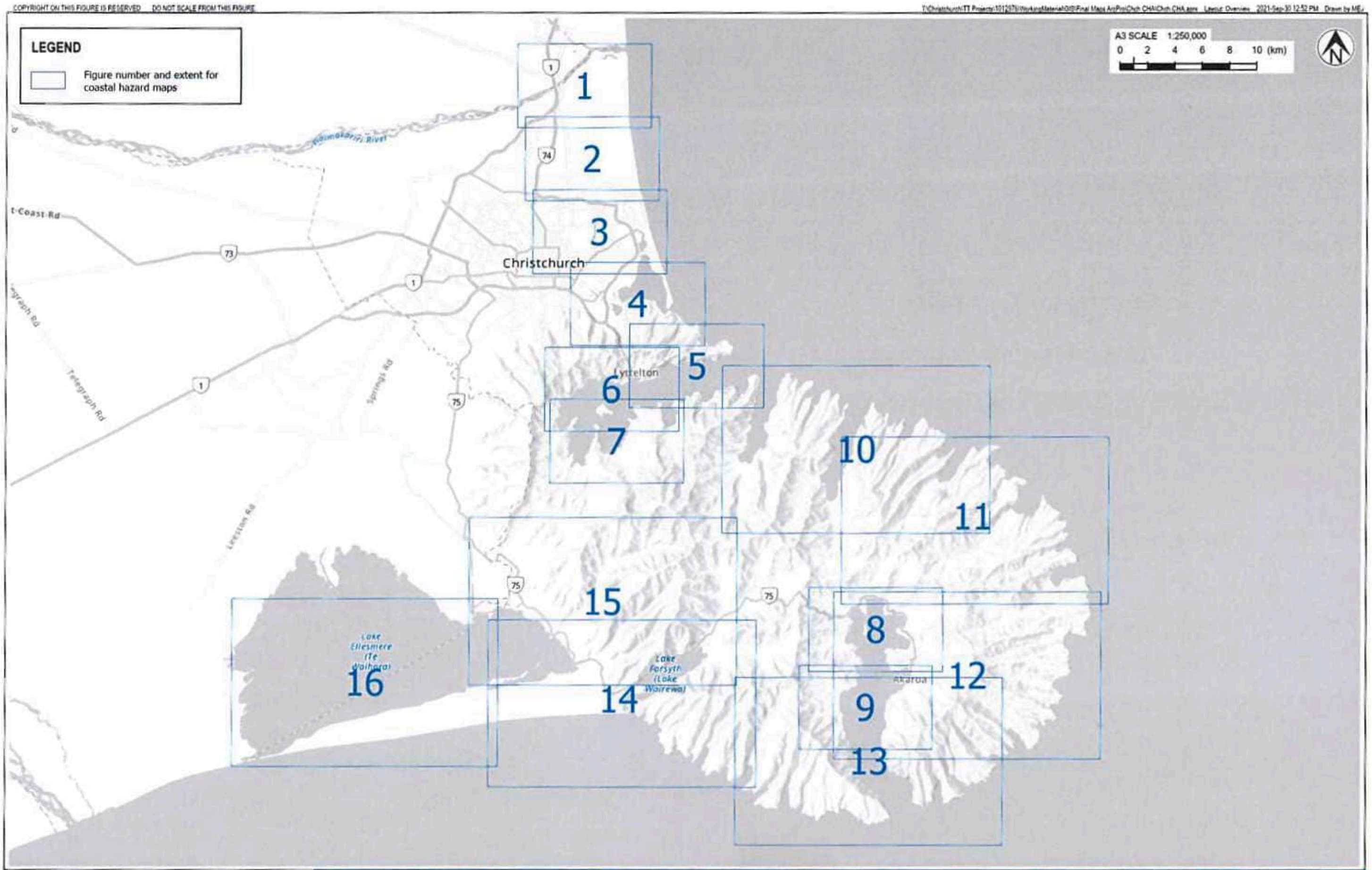

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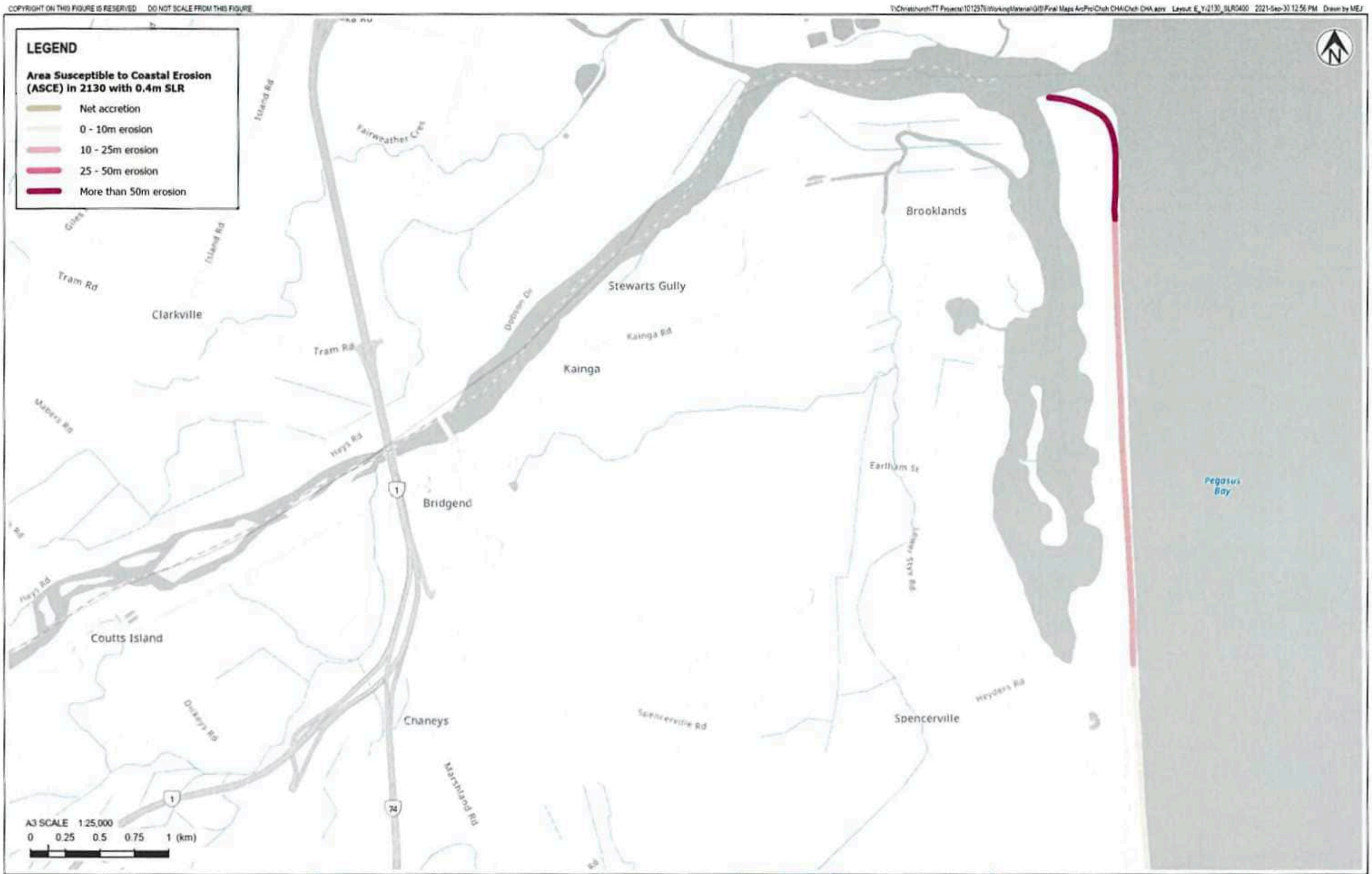

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NOTES:
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SCENARIO: YEAR 2130 WITH 0.4M SEA LEVEL RISE
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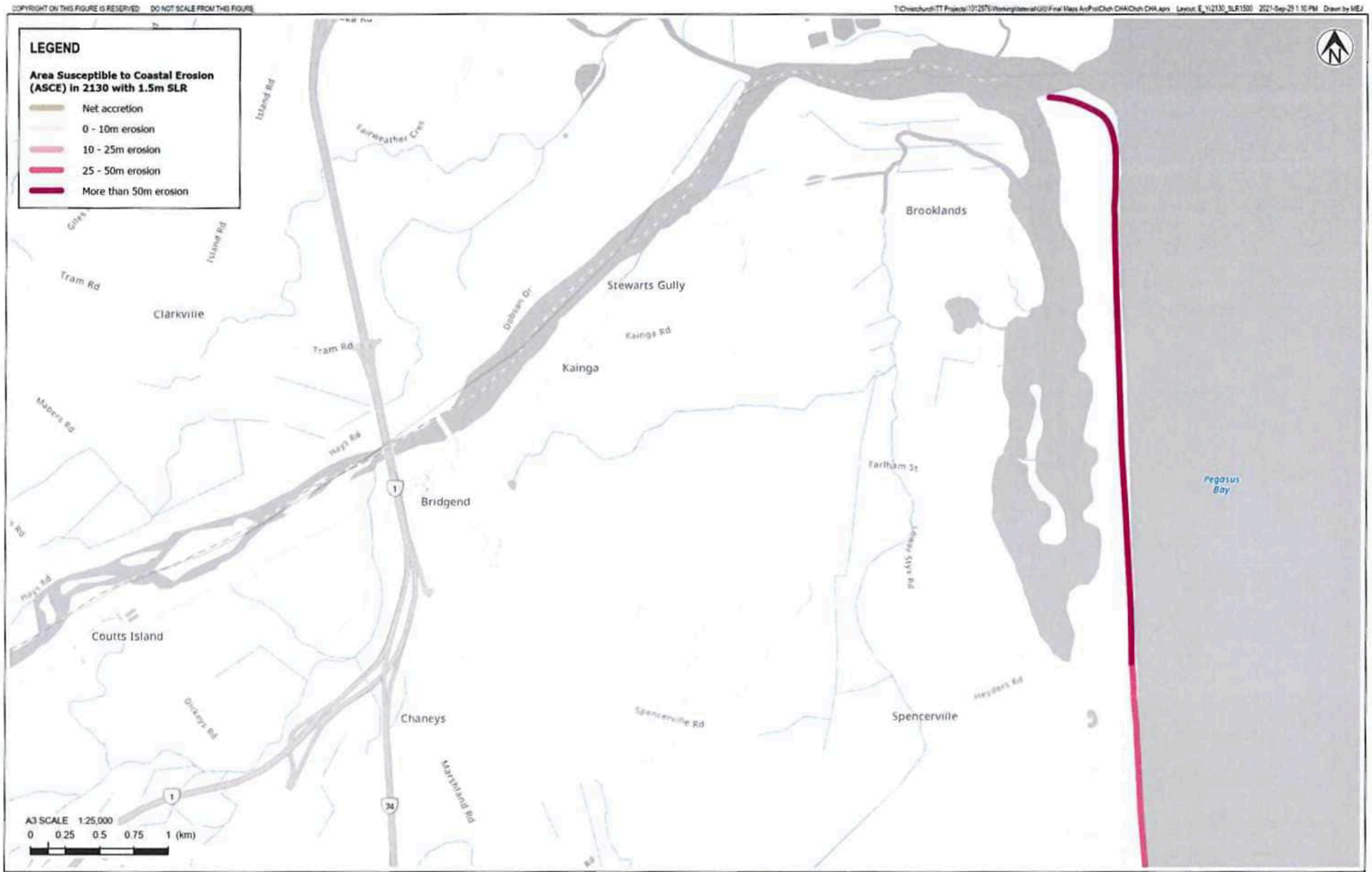

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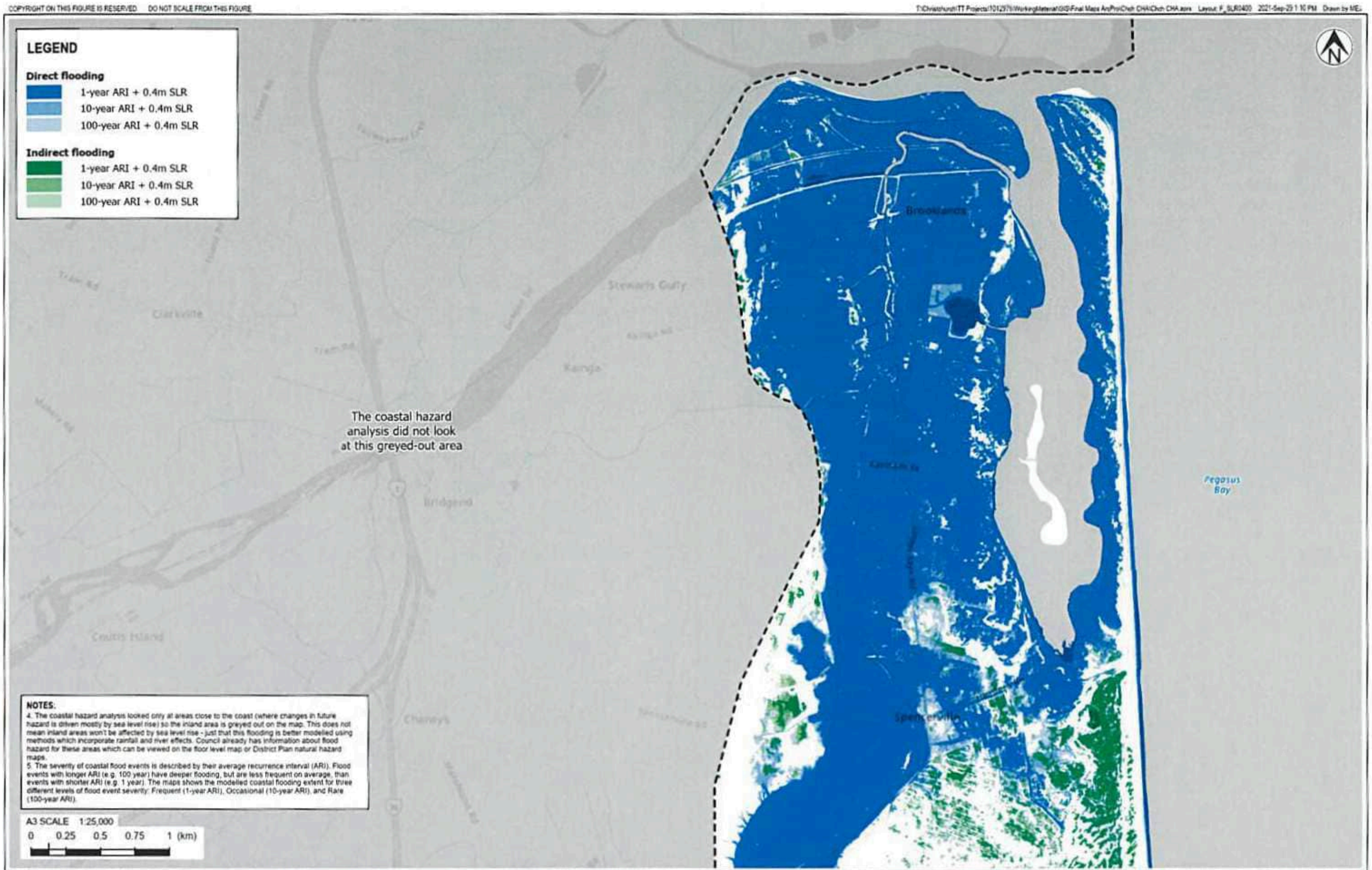

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TITLE: COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3): 1:25,000 FIG No. FIGURE 1C REV 1

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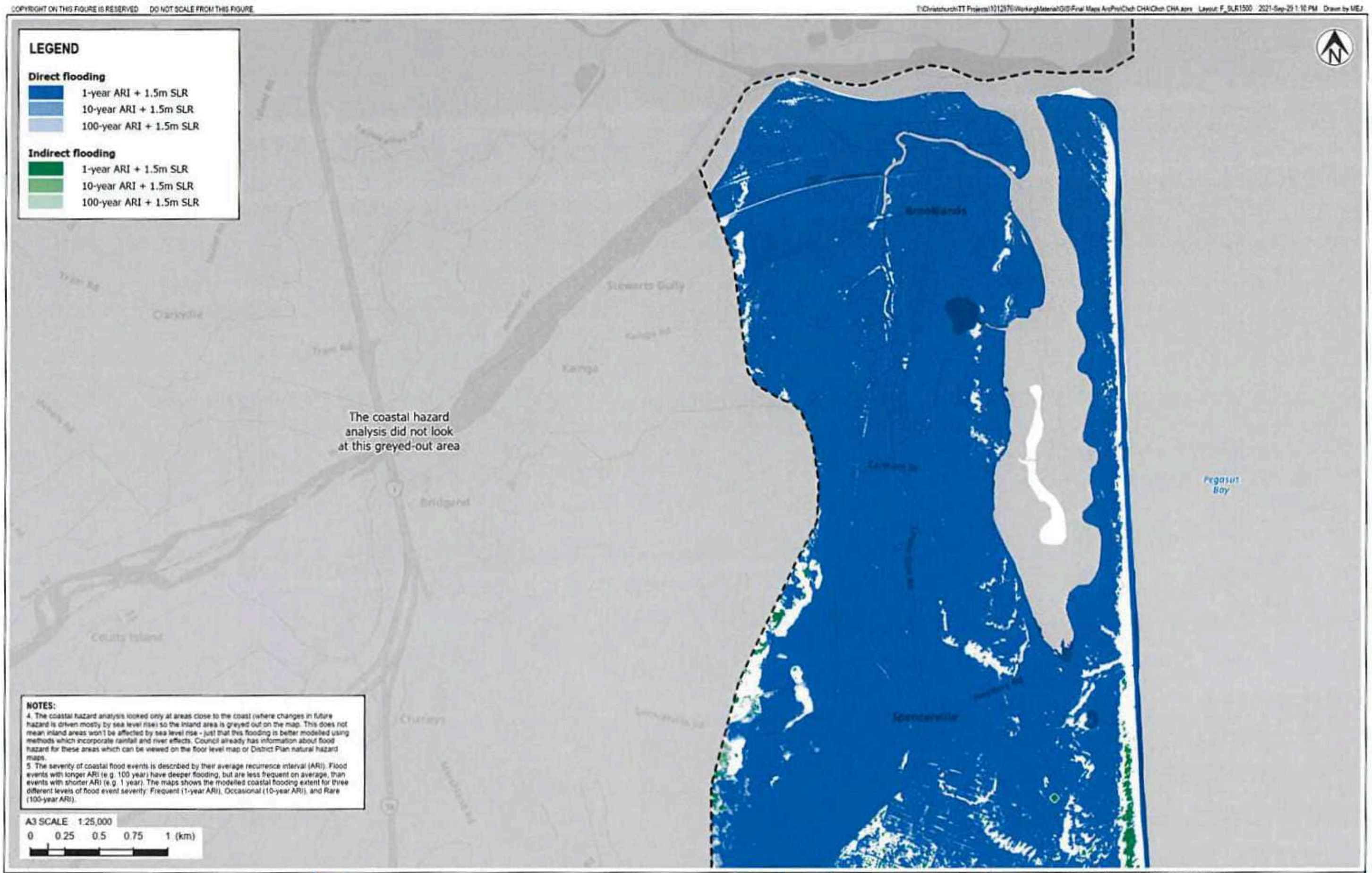

NOTES:

4. The coastal hazard analysis looked only at areas close to the coast (where changes in future hazard is driven mostly by sea level rise) so the inland area is greyed out on the map. This does not mean inland areas won't be affected by sea level rise - just that this flooding is better modelled using methods which incorporate rainfall and river effects. Council already has information about flood hazard for these areas which can be viewed on the floor level map or District Plan natural hazard maps.
5. The severity of coastal flood events is described by their average recurrence interval (ARI). Flood events with longer ARI (e.g. 100 year) have deeper flooding, but are less frequent on average, than events with shorter ARI (e.g. 1 year). The maps shows the modelled coastal flooding extent for three different levels of flood event severity: Frequent (1-year ARI), Occasional (10-year ARI), and Rare (100-year ARI).

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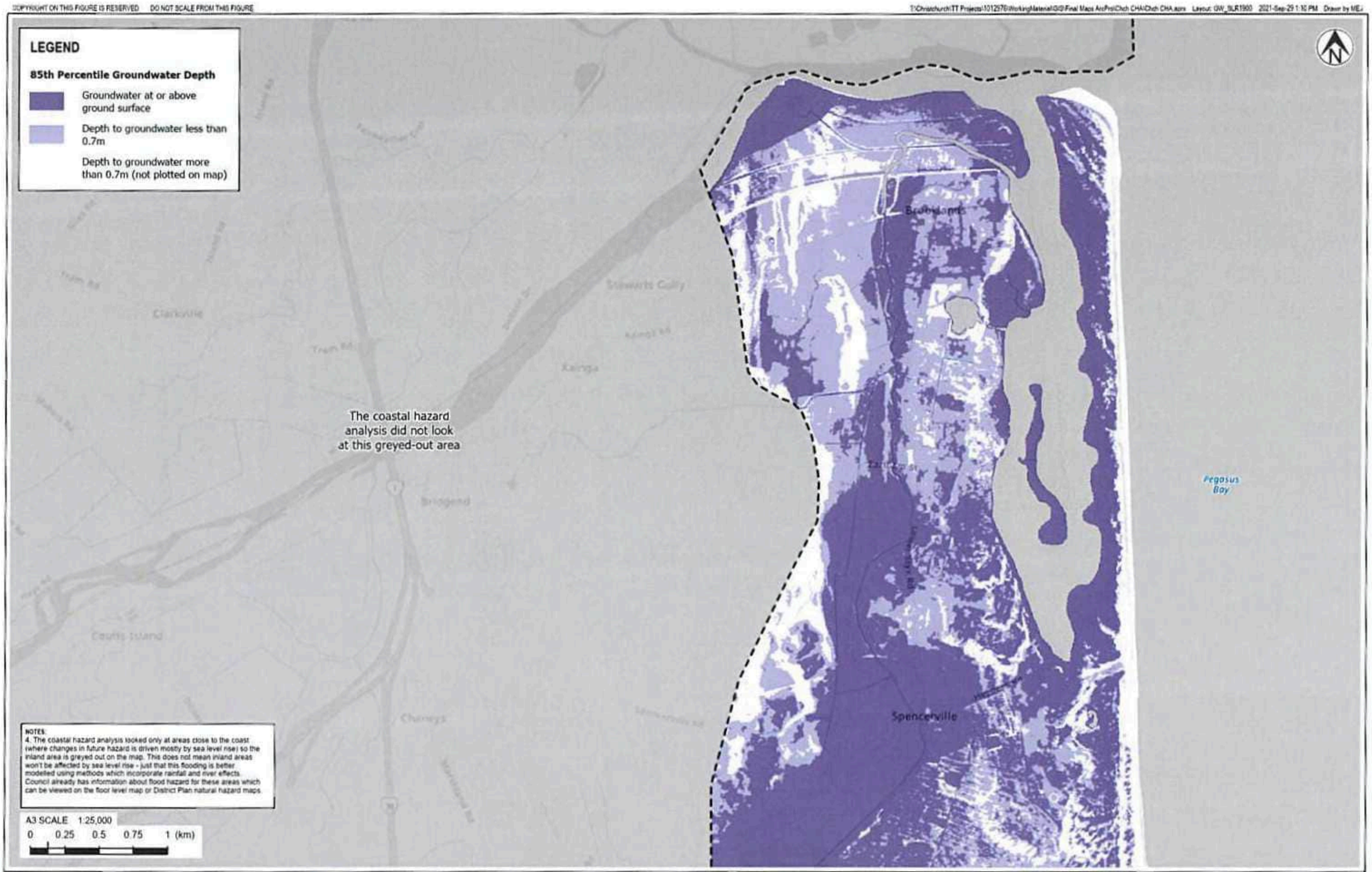

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SCENARIO: 1.9M SEA LEVEL RISE

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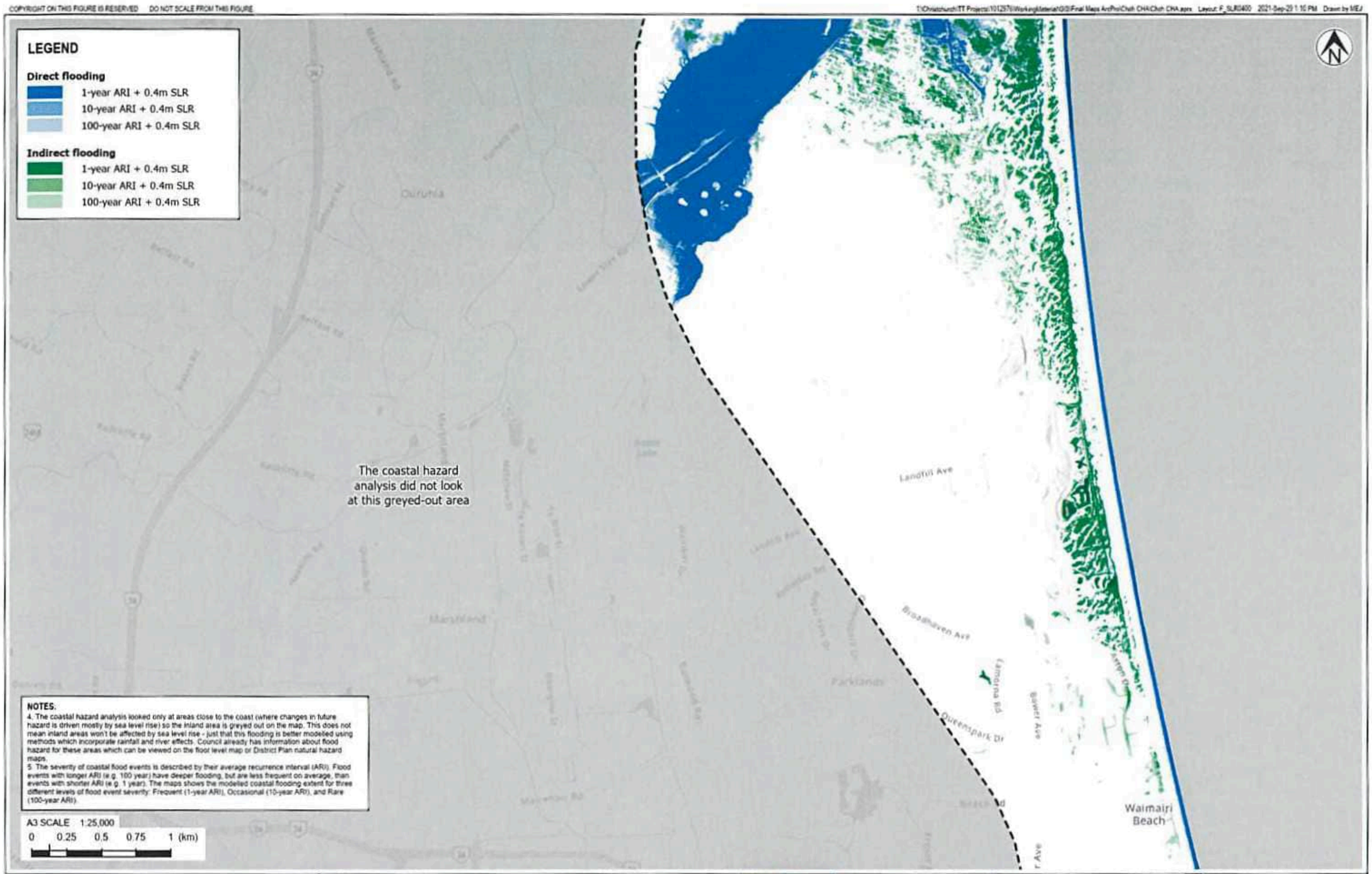

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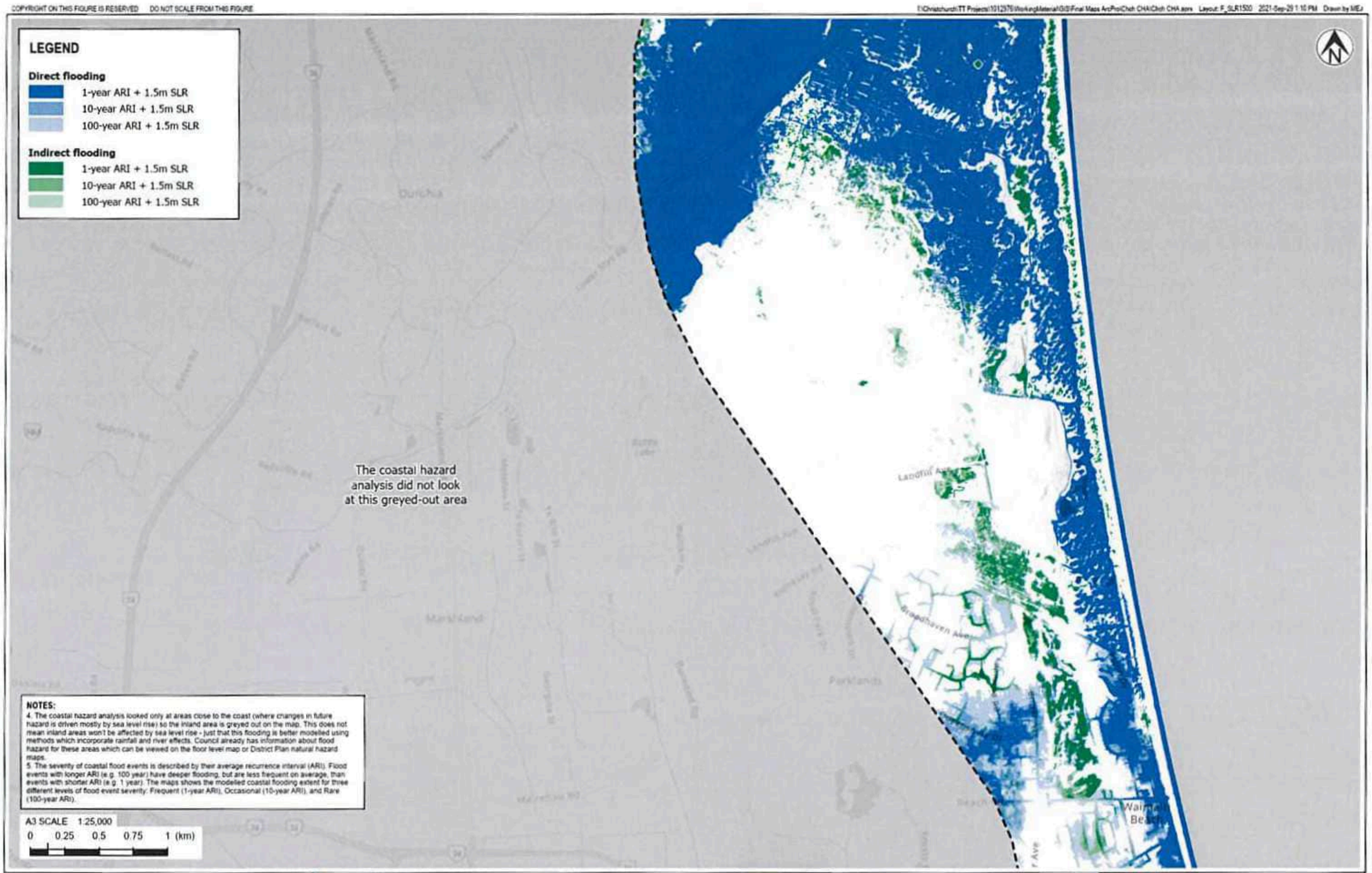

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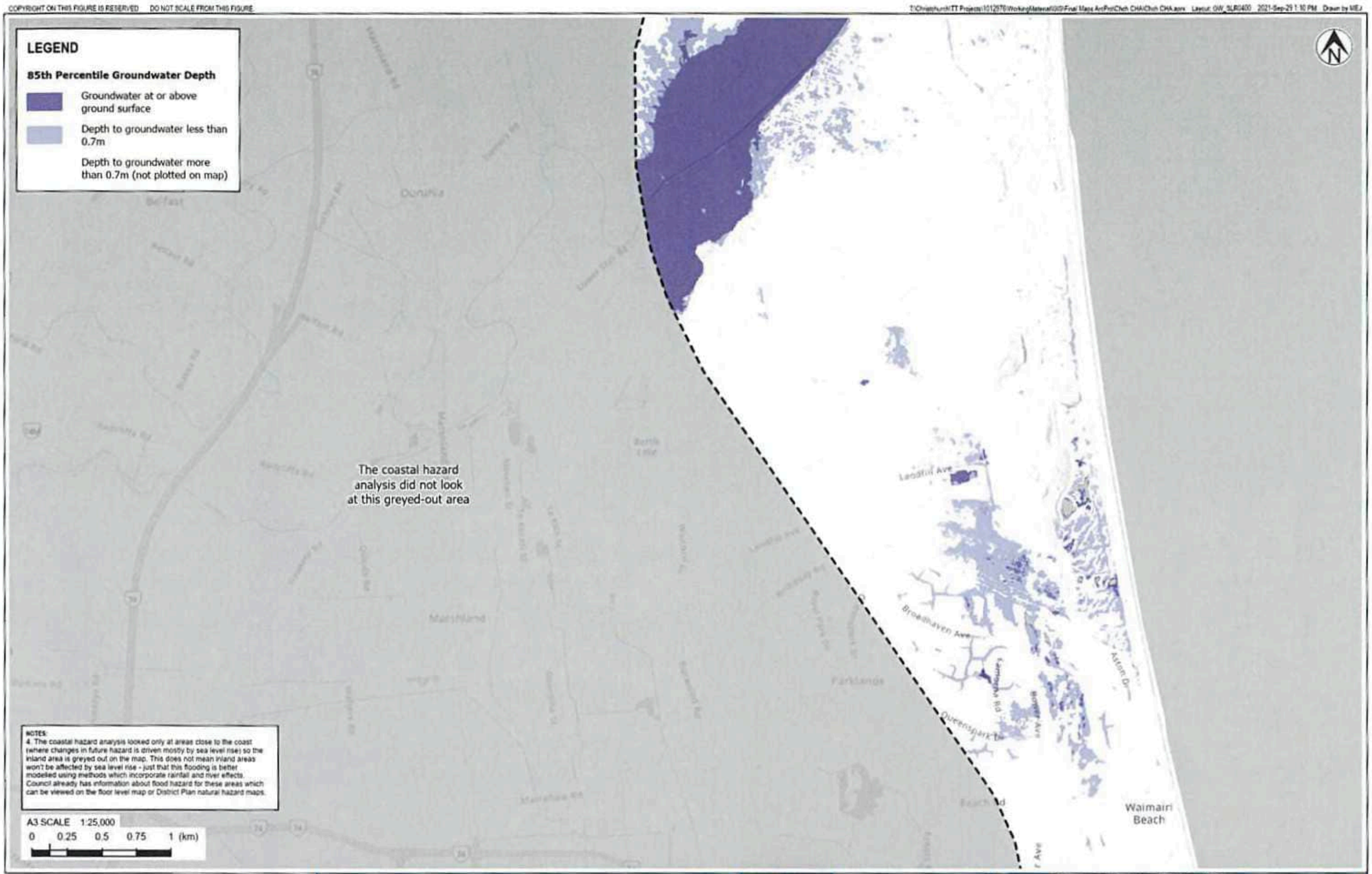

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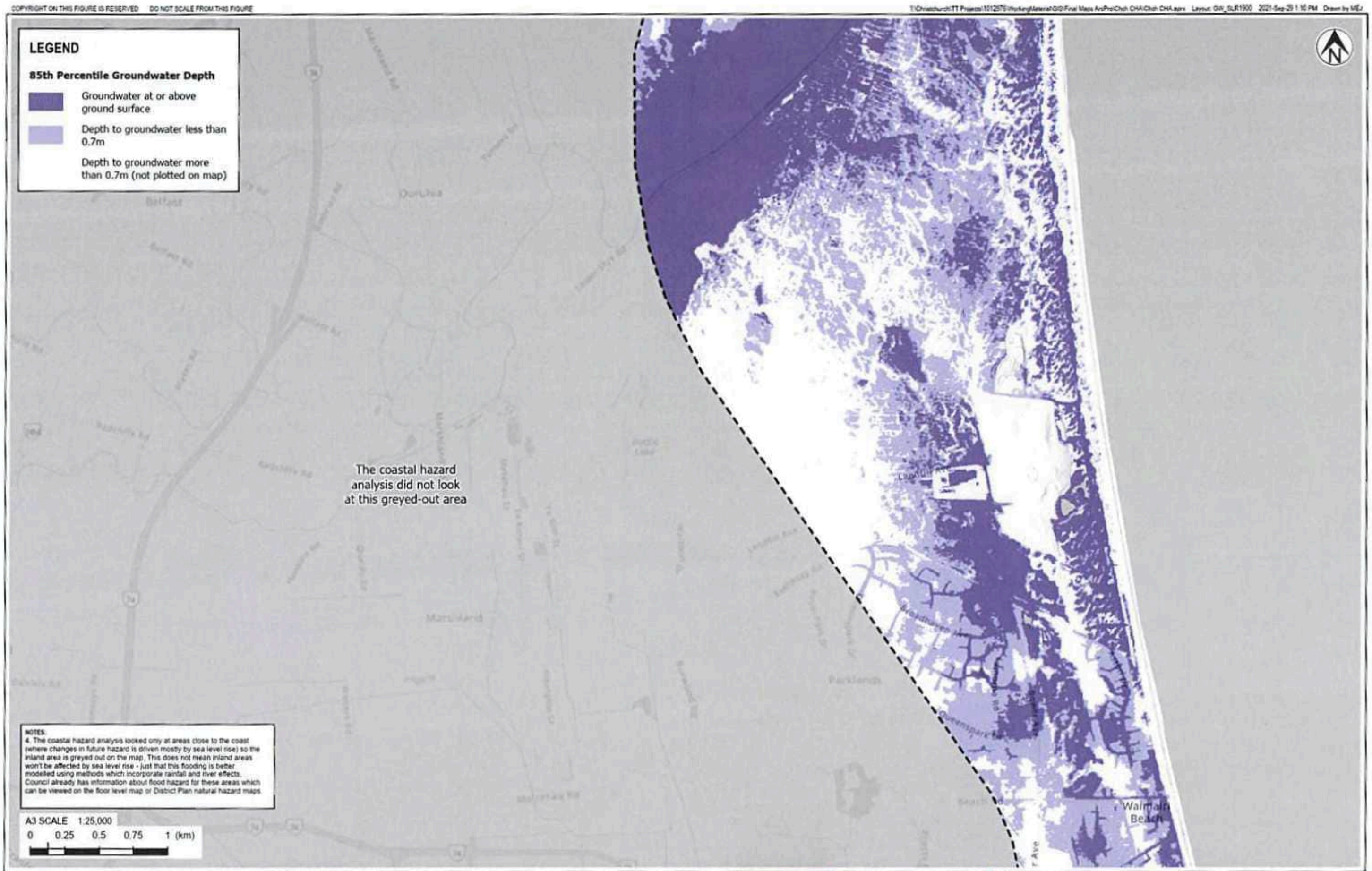

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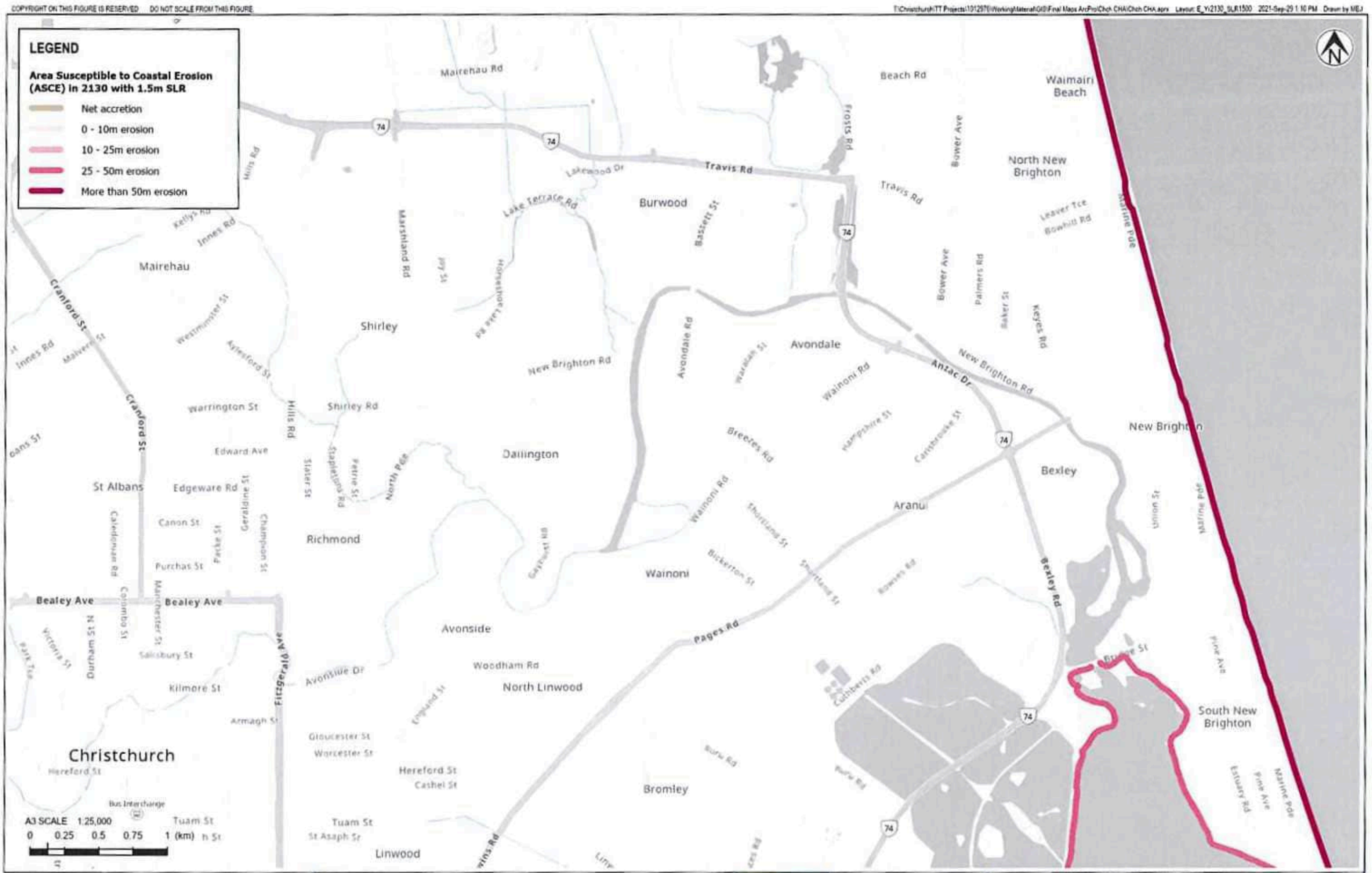

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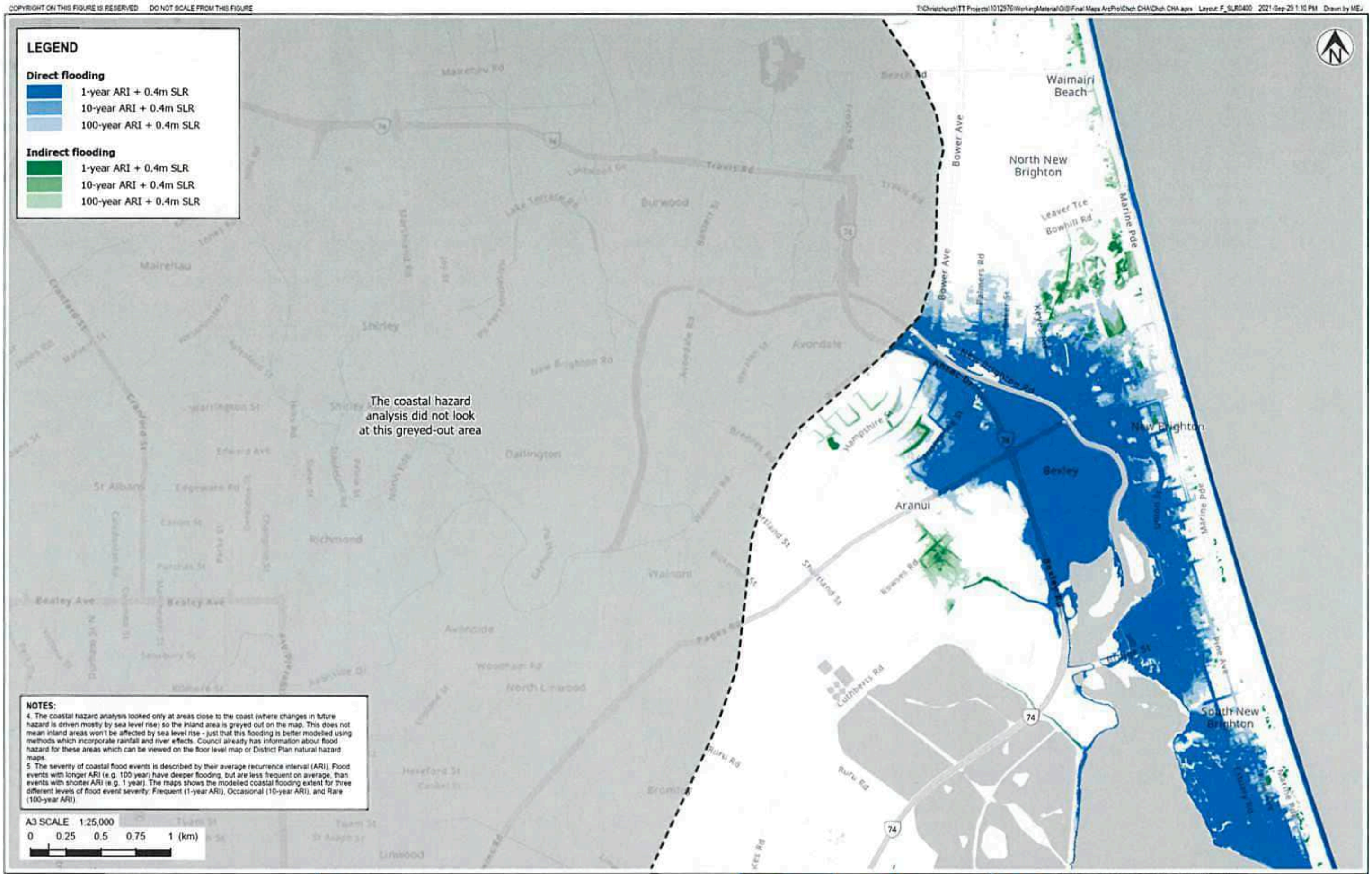

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SCENARIO: 0.4M SEA LEVEL RISE

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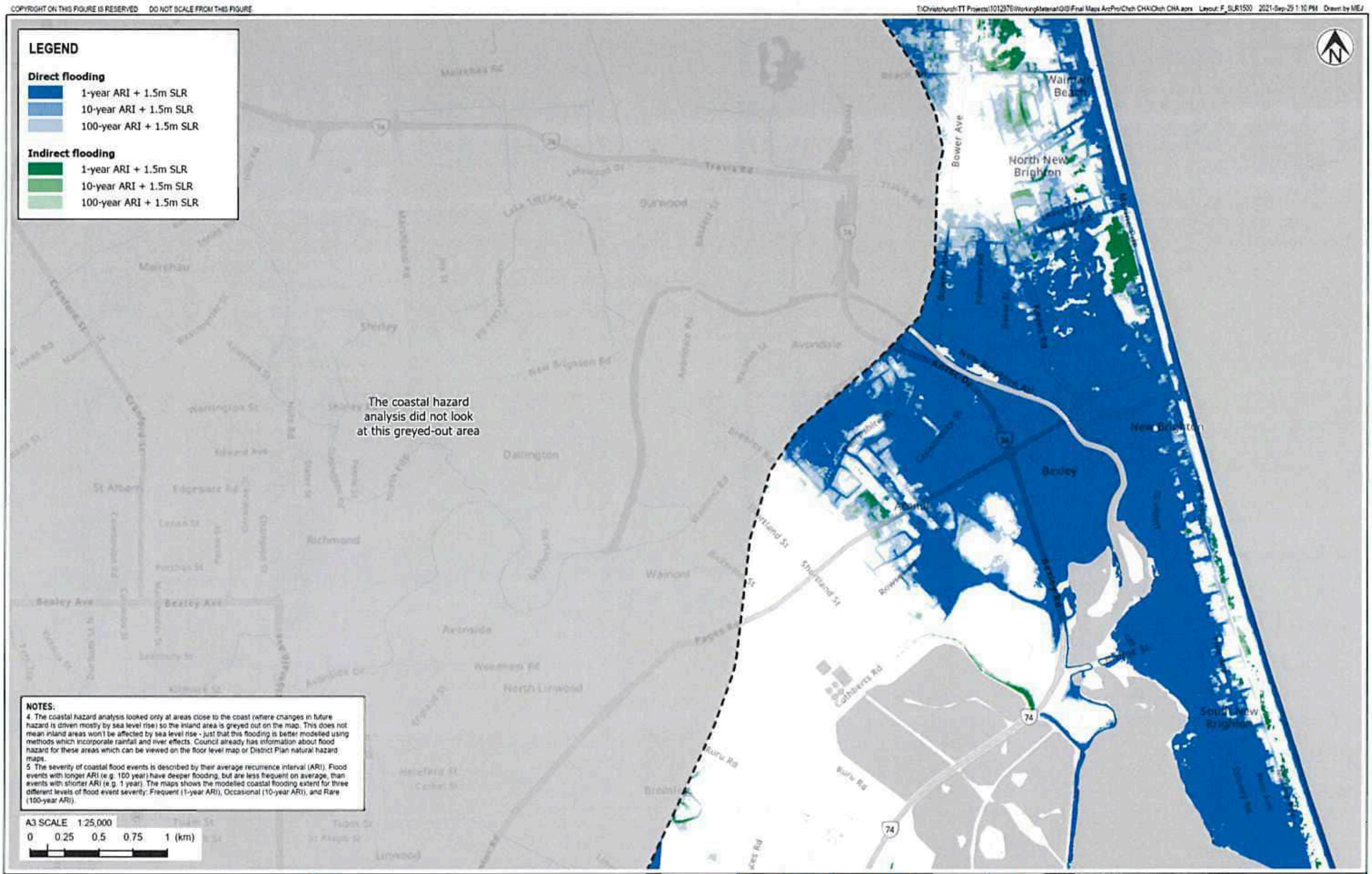

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APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL

PROJECT COASTAL HAZARD ASSESSMENT

TITLE COASTAL FLOODING ANALYSIS SCENARIO: 1.5M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 3D REV 1

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1. For Christchurch urban flatland area, map shows 85th percentile groundwater model from Aqualinc (2020) "LDRP45: Impacts of earthquakes and sea level rise on shallow groundwater levels." Report prepared for Christchurch City Council, August 2020.
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3. Basemap NZ Hillshade (Alpha): LINZ, Eagle Technology. NZ Topographic Map for use with relief - Grey: Eagle Technology, LINZ.

REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 3E REV 1

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1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 3F REV 1

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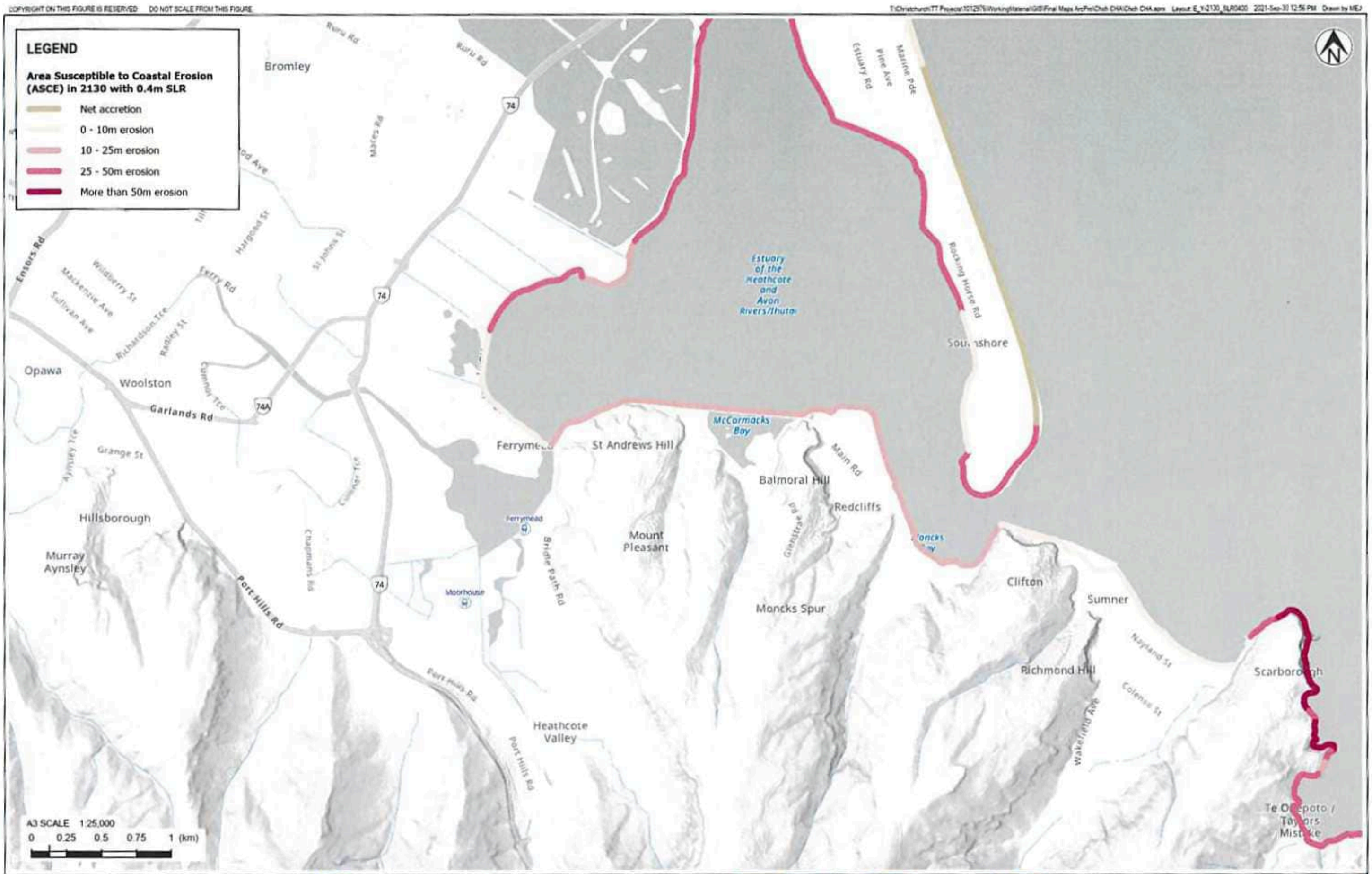

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CHECKED	RHAU	SEP.21
APPROVED: P. COCHRANE	DATE: 29/09/21	

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 4A REV 1

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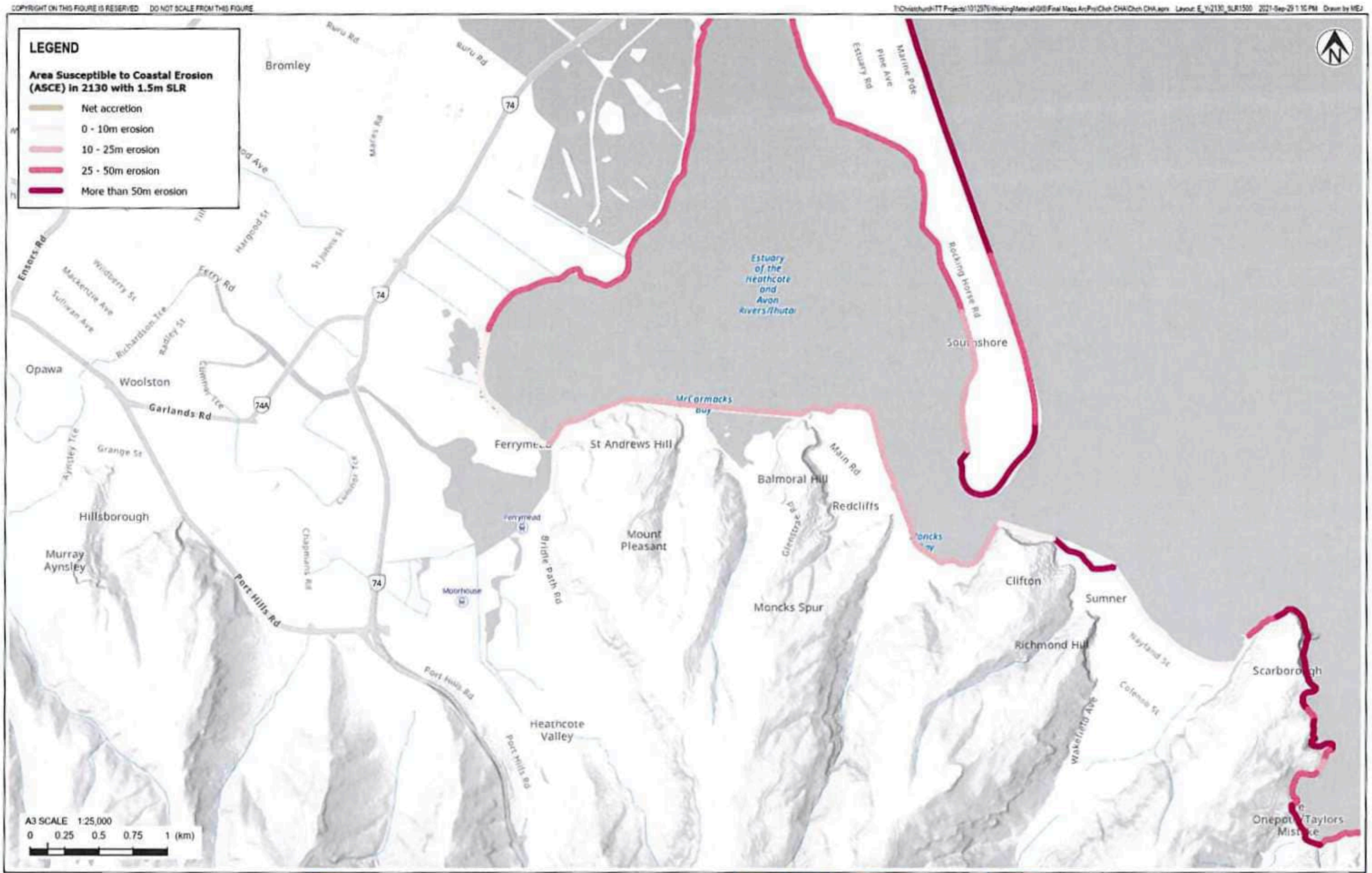

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CHECKED	RHAU	SEP.21
APPROVED: P. COCHRANE	DATE: 29/09/21	

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 1.5M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 4B REV 1

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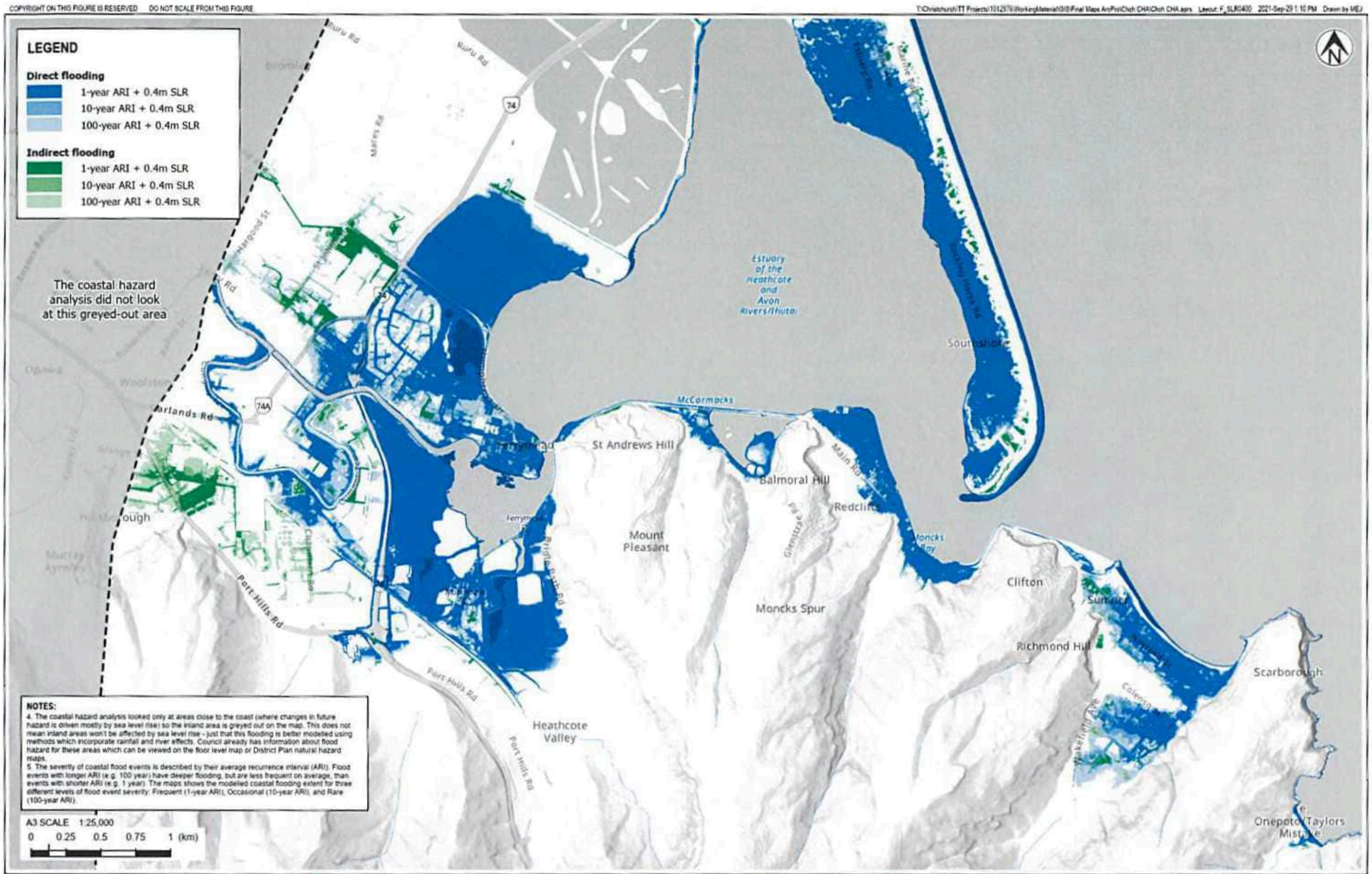

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NOTES:
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PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
APPROVED: P COCHRANE	DATE: 29/09/21	

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 4C REV 1

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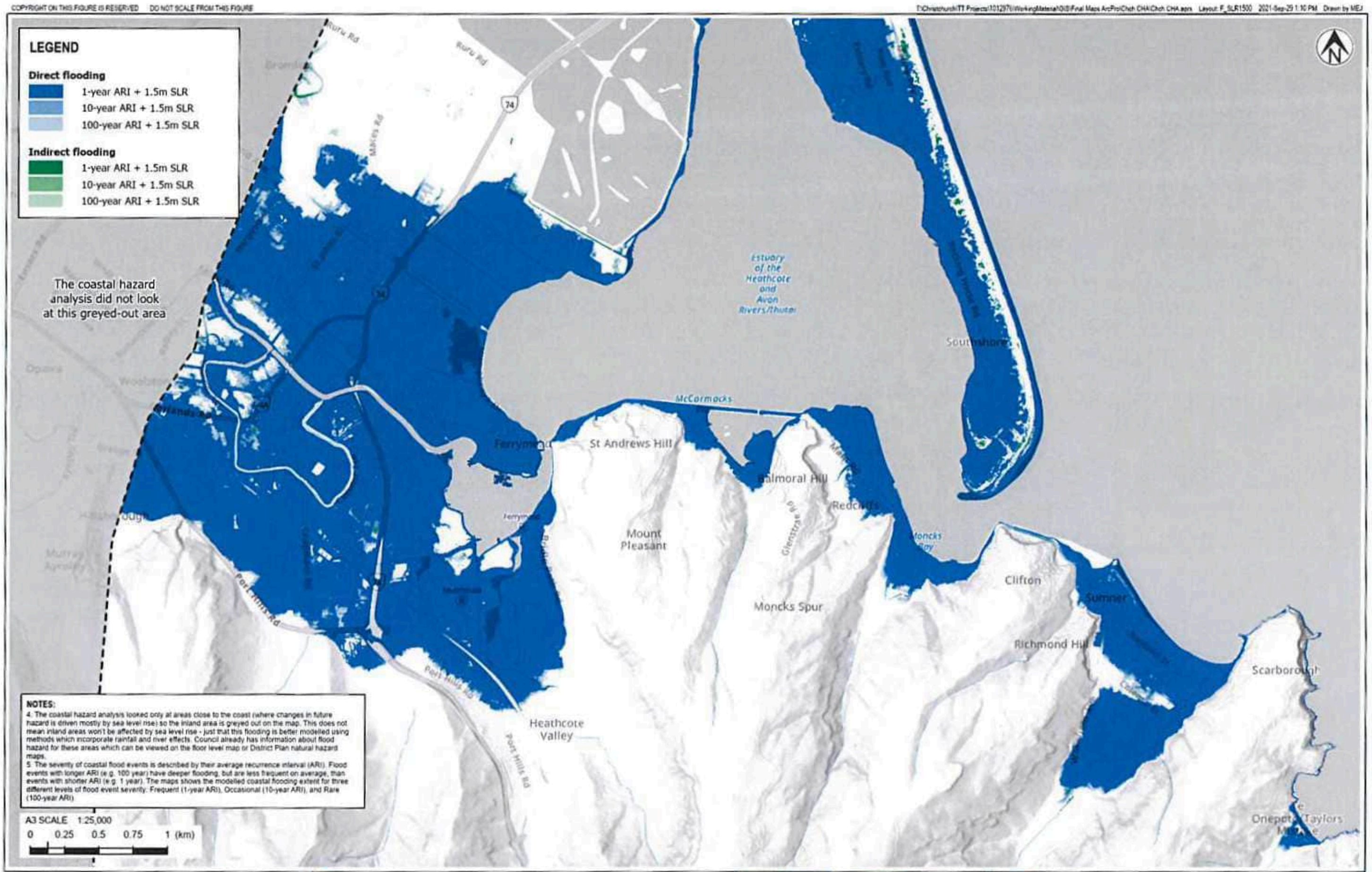

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DESIGNED	MEJ	SEP.21
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CHECKED	PPK	SEP.21
APPROVED	P COCHRANE	29/09/21

CLIENT: CHRISTCHURCH CITY COUNCIL
PROJECT: COASTAL HAZARD ASSESSMENT
TITLE: COASTAL FLOODING ANALYSIS SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3): 1:25,000   FIG No. FIGURE 4D   REV 1

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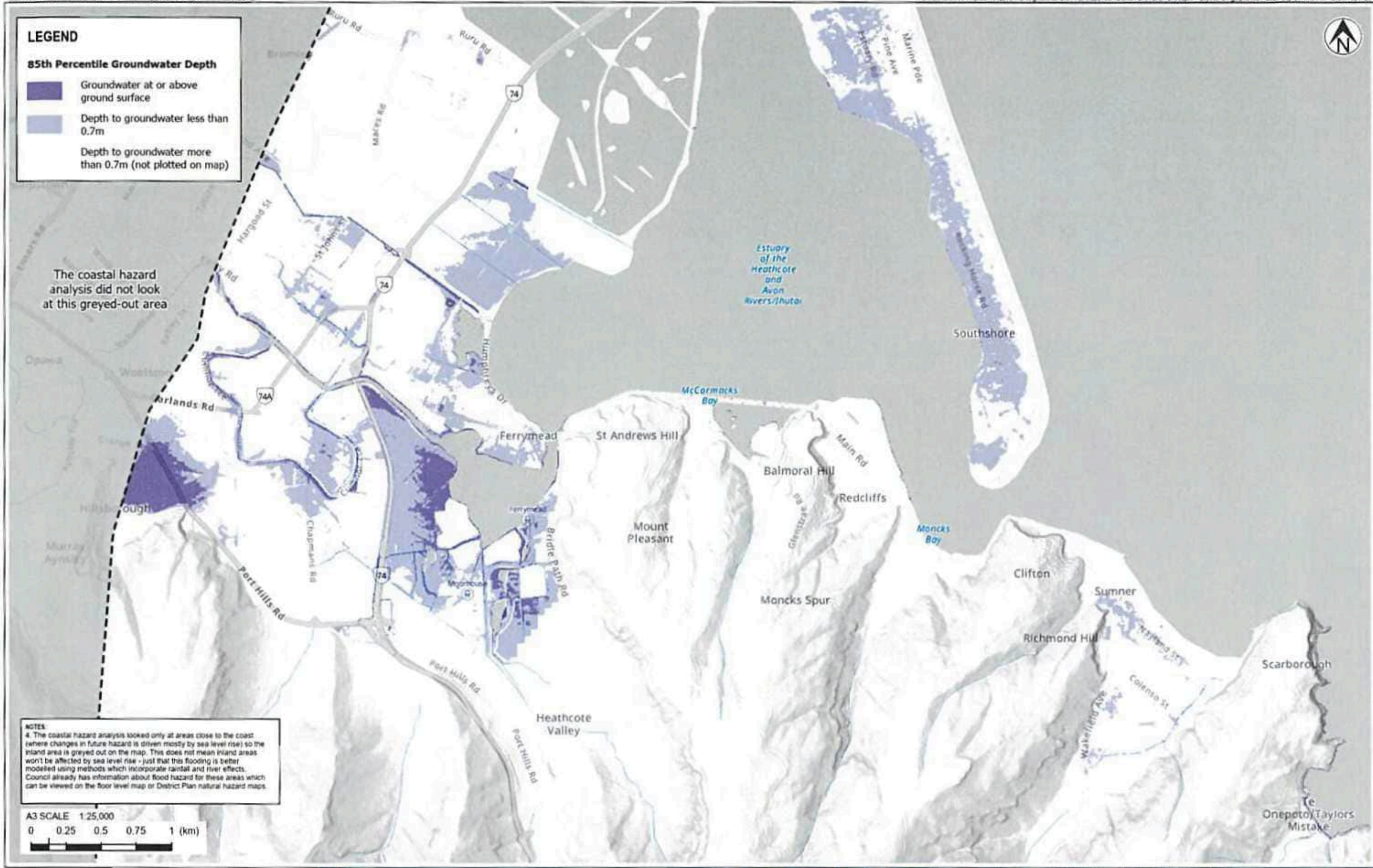

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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 4E REV 1

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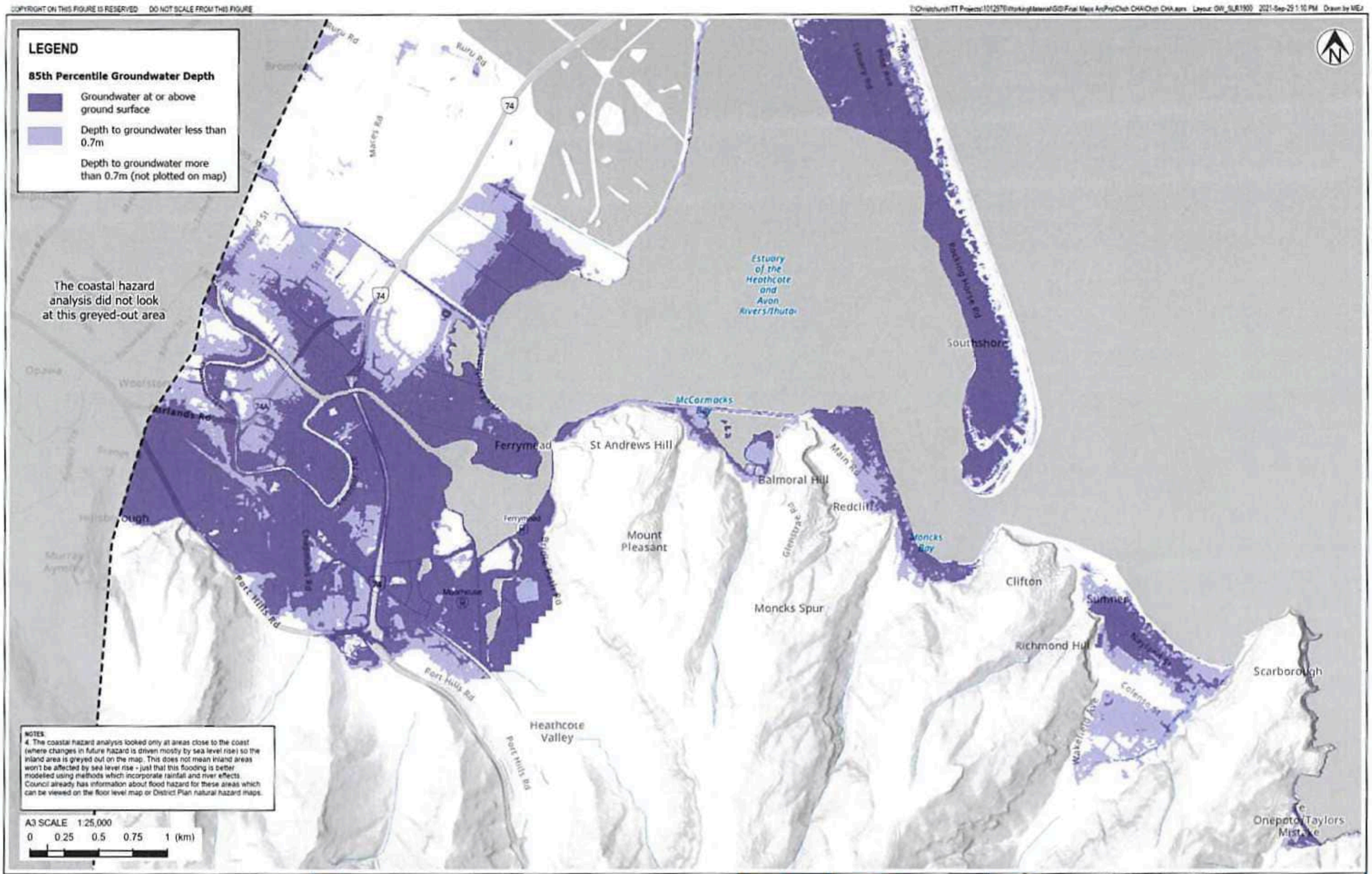

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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT: **CHRISTCHURCH CITY COUNCIL**

PROJECT: **COASTAL HAZARD ASSESSMENT**

TITLE: COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3): 1:25,000 FIG No. FIGURE 4F REV 1

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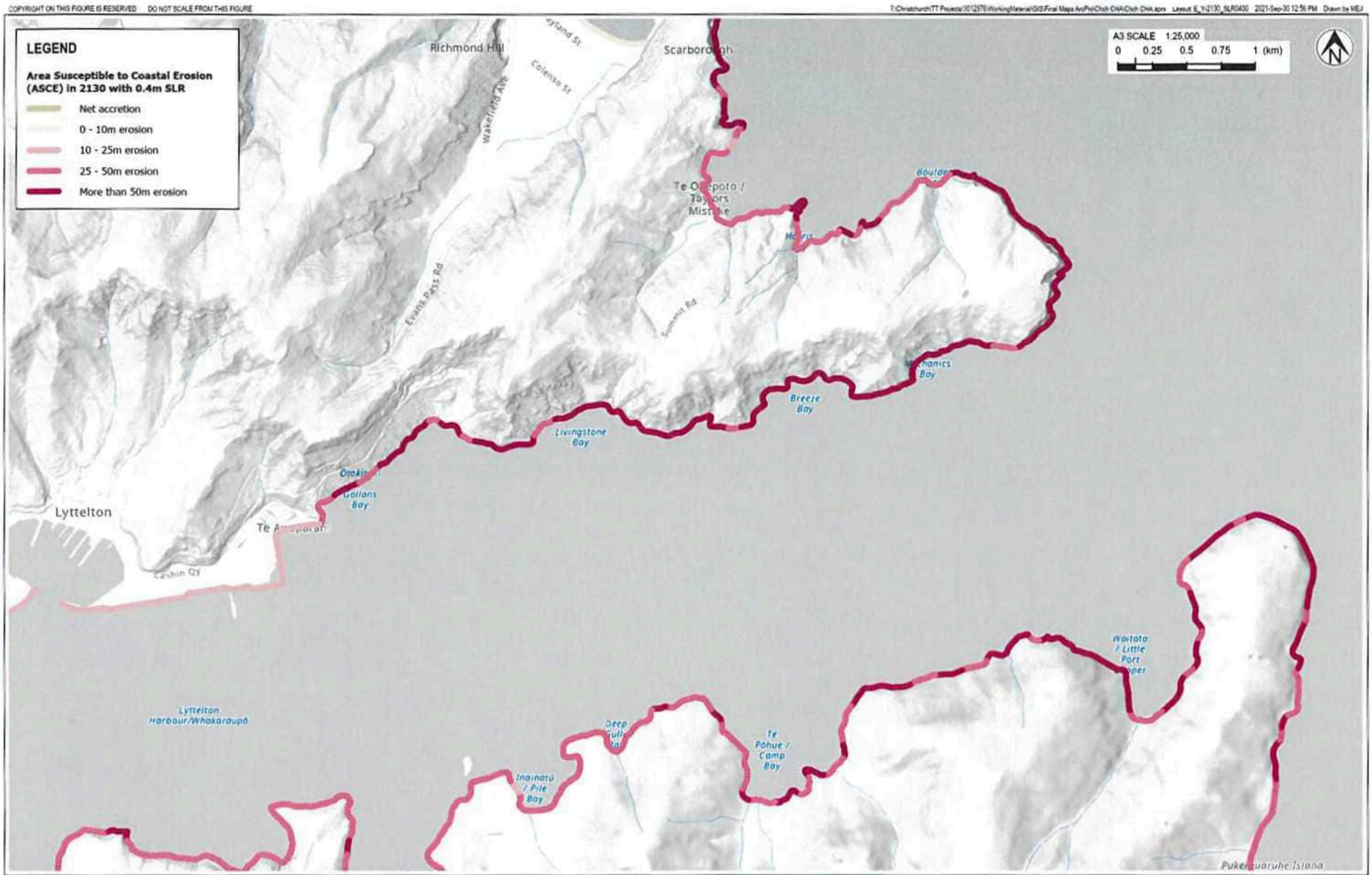

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1	Report issued	MEJ	RHAU	29/09/21		P. COCHRANE	29/09/21
REV	DESCRIPTION	GIS	CHK	DATE	LOCATION PLAN	APPROVED	DATE

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	RHAU	SEP.21

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CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 5A REV 1

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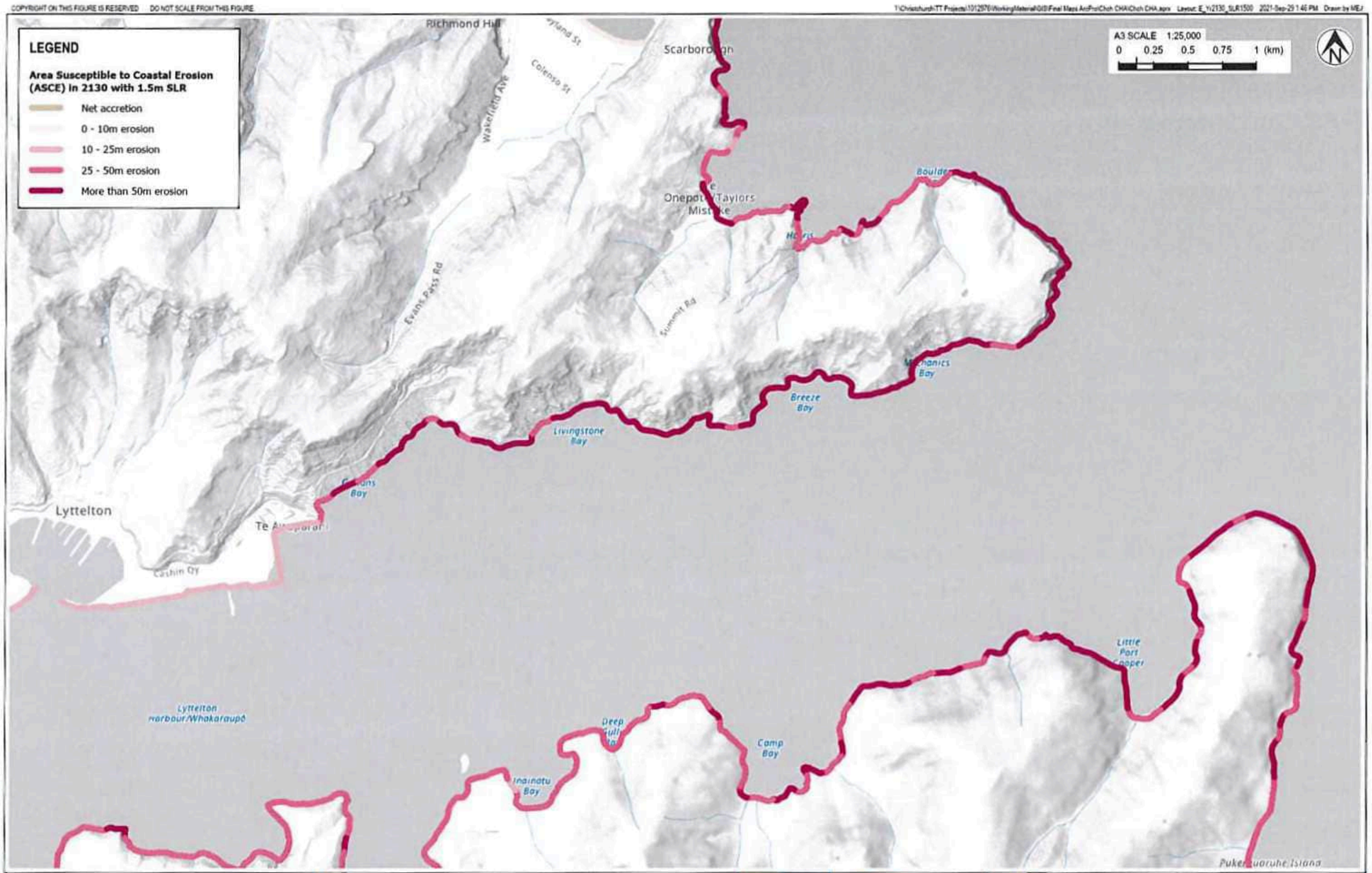

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PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	RHAU	SEP.21
APPROVED: P COCHRANE	DATE: 29/09/21	

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 1.5M SEA LEVEL RISE

SCALE (A3) 1:25,000   FIG No. FIGURE 5B   REV 1

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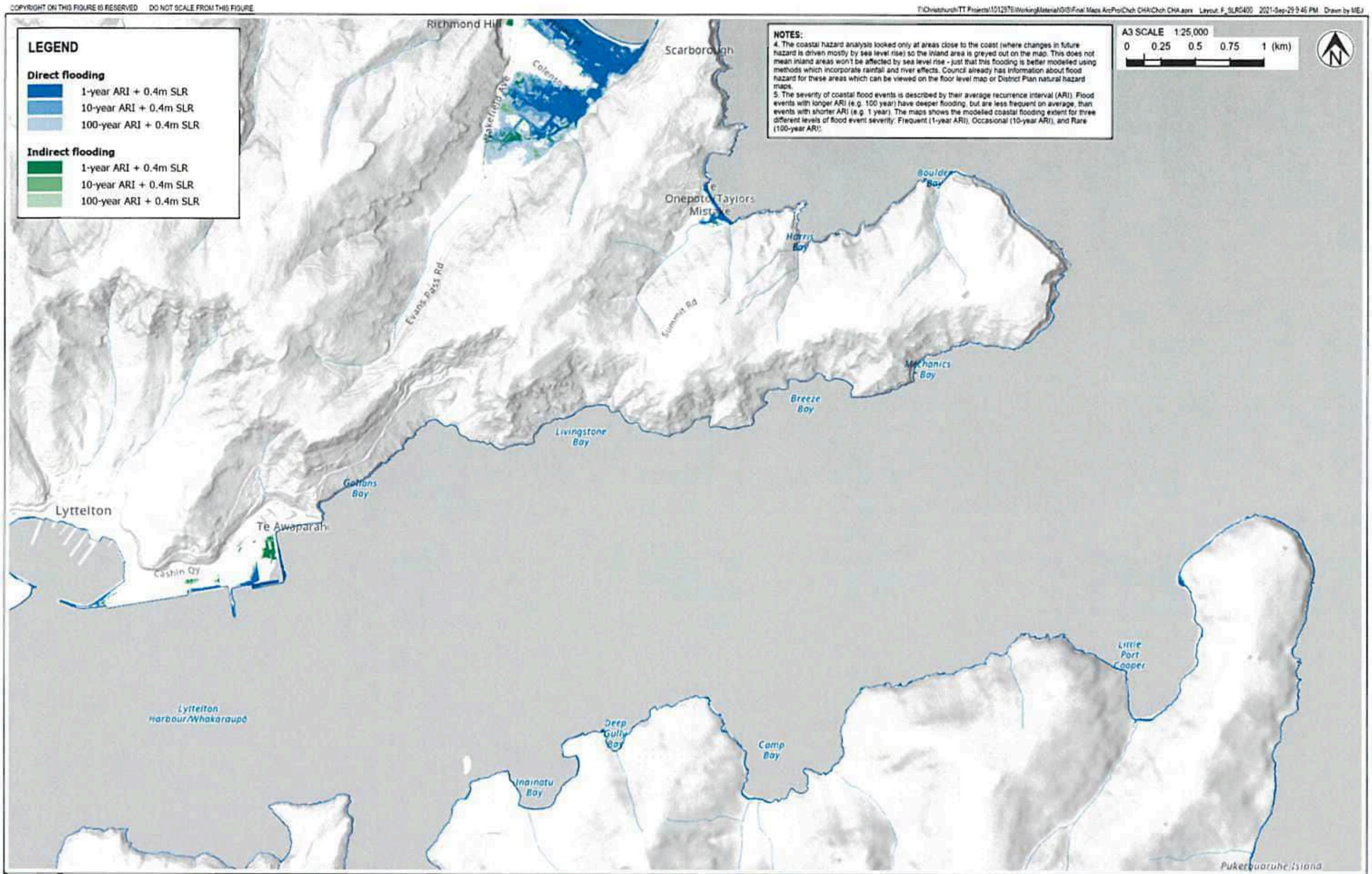

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REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

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PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 5C REV 1

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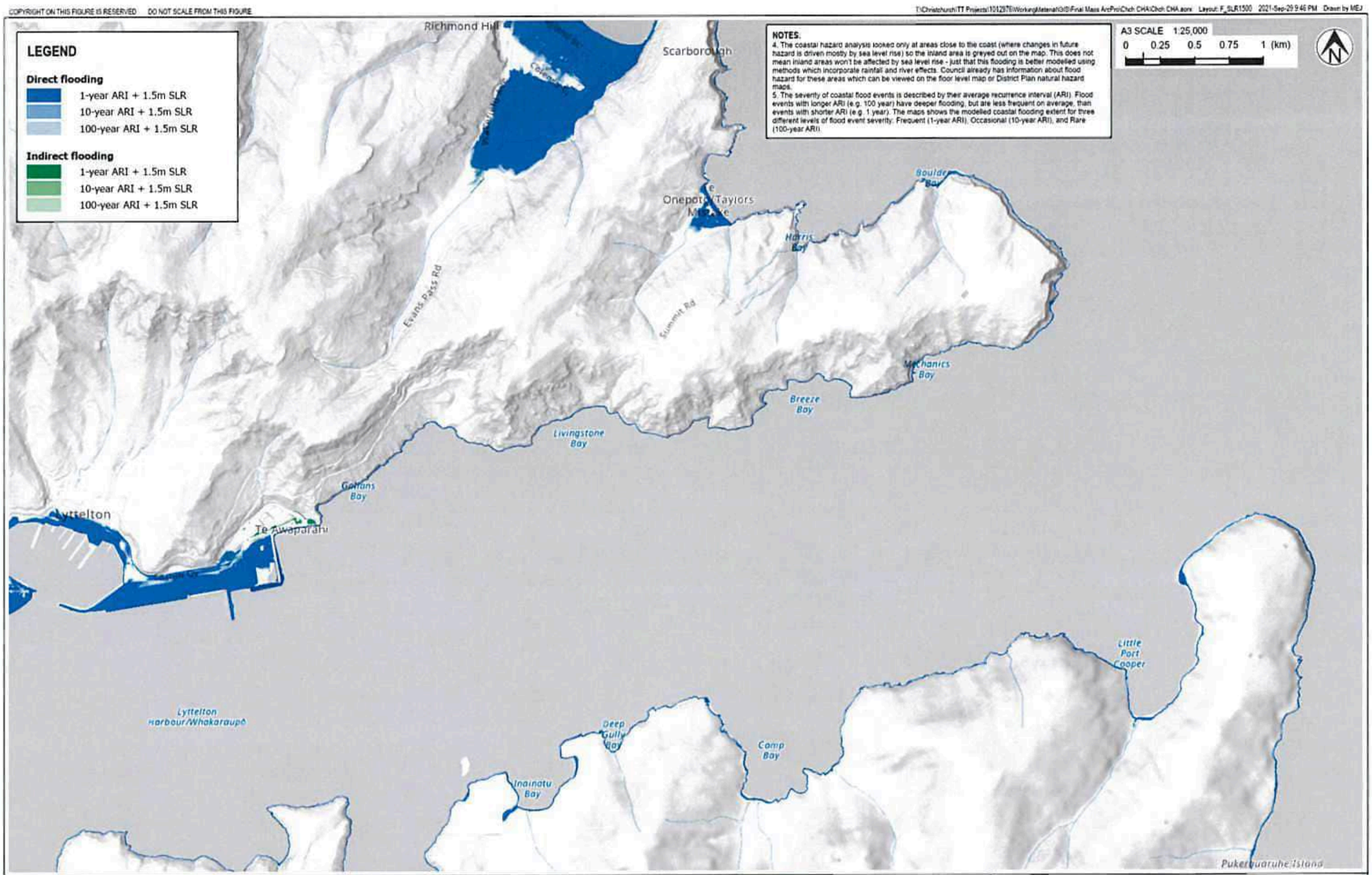

NOTES.

4. The coastal hazard analysis looked only at areas close to the coast (where changes in future hazard is driven mostly by sea level rise) so the inland area is greyed out on the map. This does not mean inland areas won't be affected by sea level rise - just that this flooding is better modelled using methods which incorporate rainfall and river effects. Council already has information about flood hazard for these areas which can be viewed on the floor level map or District Plan natural hazard maps.
5. The severity of coastal flood events is described by their average recurrence interval (ARI). Flood events with longer ARI (e.g. 100 year) have deeper flooding, but are less frequent on average, than events with shorter ARI (e.g. 1 year). The maps shows the modelled coastal flooding extent for three different levels of flood event severity: Frequent (1-year ARI), Occasional (10-year ARI), and Rare (100-year ARI).

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PROJECT No.	1012976	
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P. COCHRANE	29/09/21	
APPROVED	DATE	

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 5D REV 1

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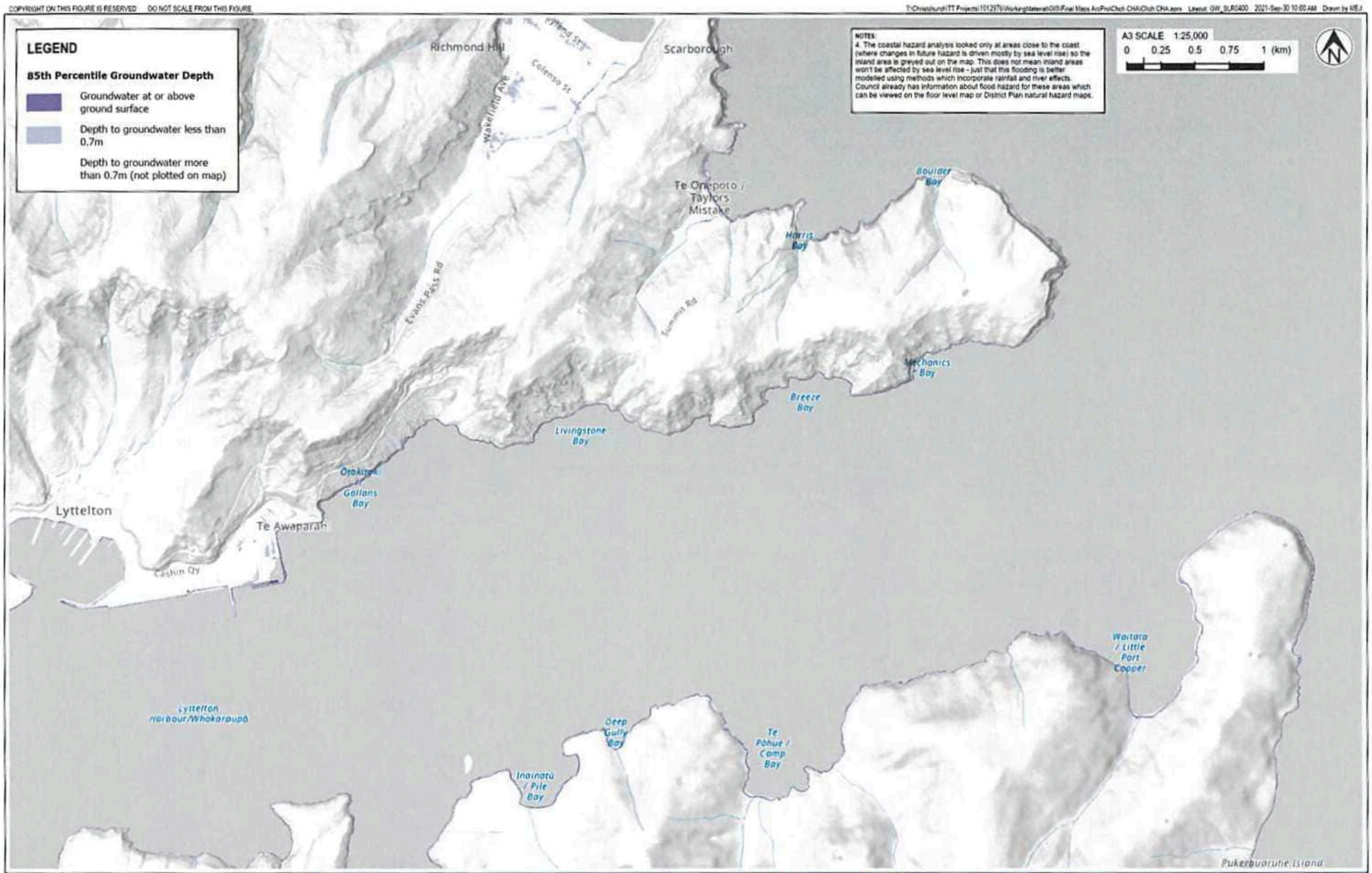

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REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 5E REV 1

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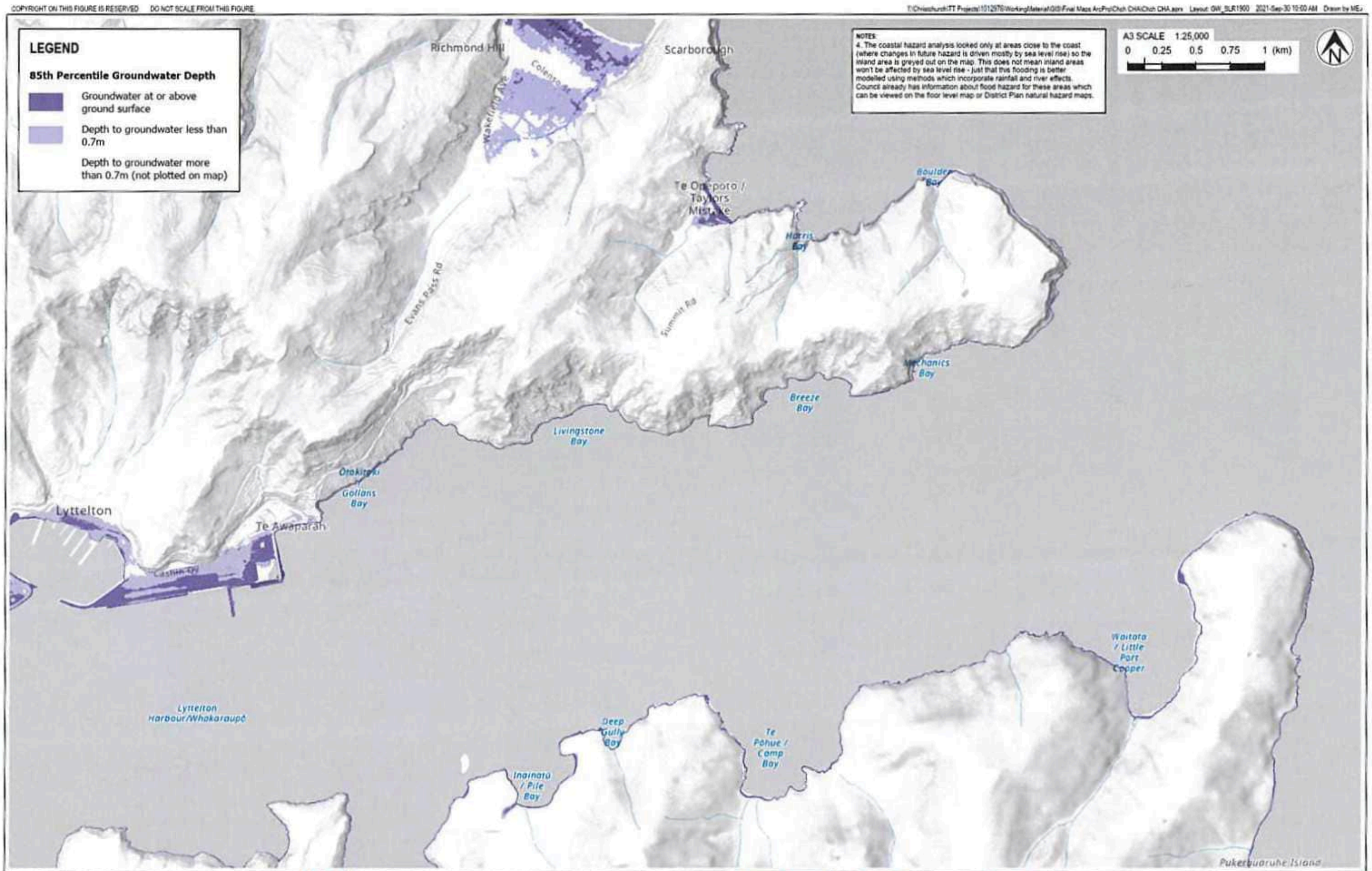

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APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL

PROJECT COASTAL HAZARD ASSESSMENT

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 5F REV 1

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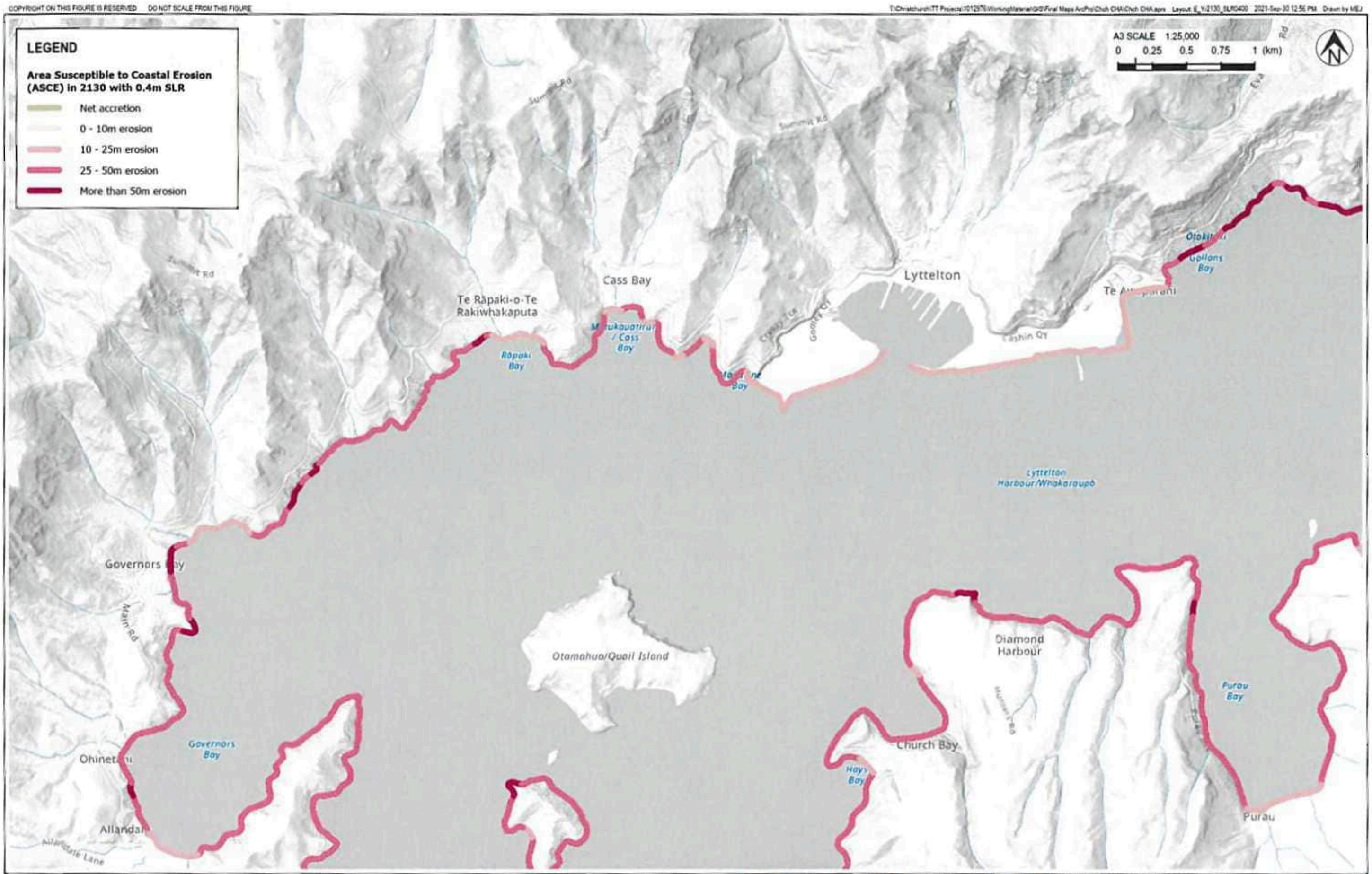

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APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 6A REV 1

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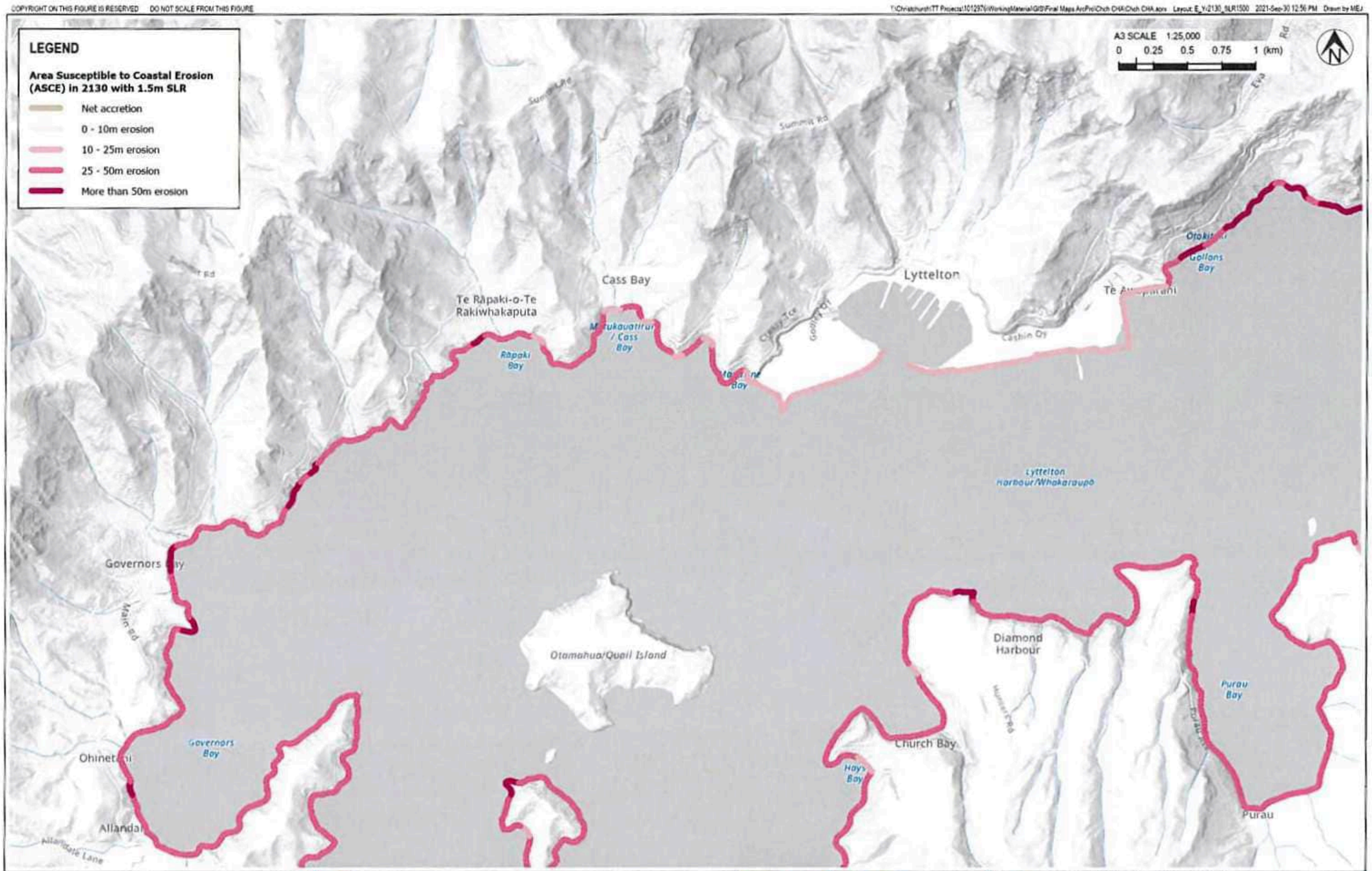

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PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21 (DATE)

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 1.5M SEA LEVEL RISE

SCALE (A3) 1:25,000   FIG No. FIGURE 6B   REV 1

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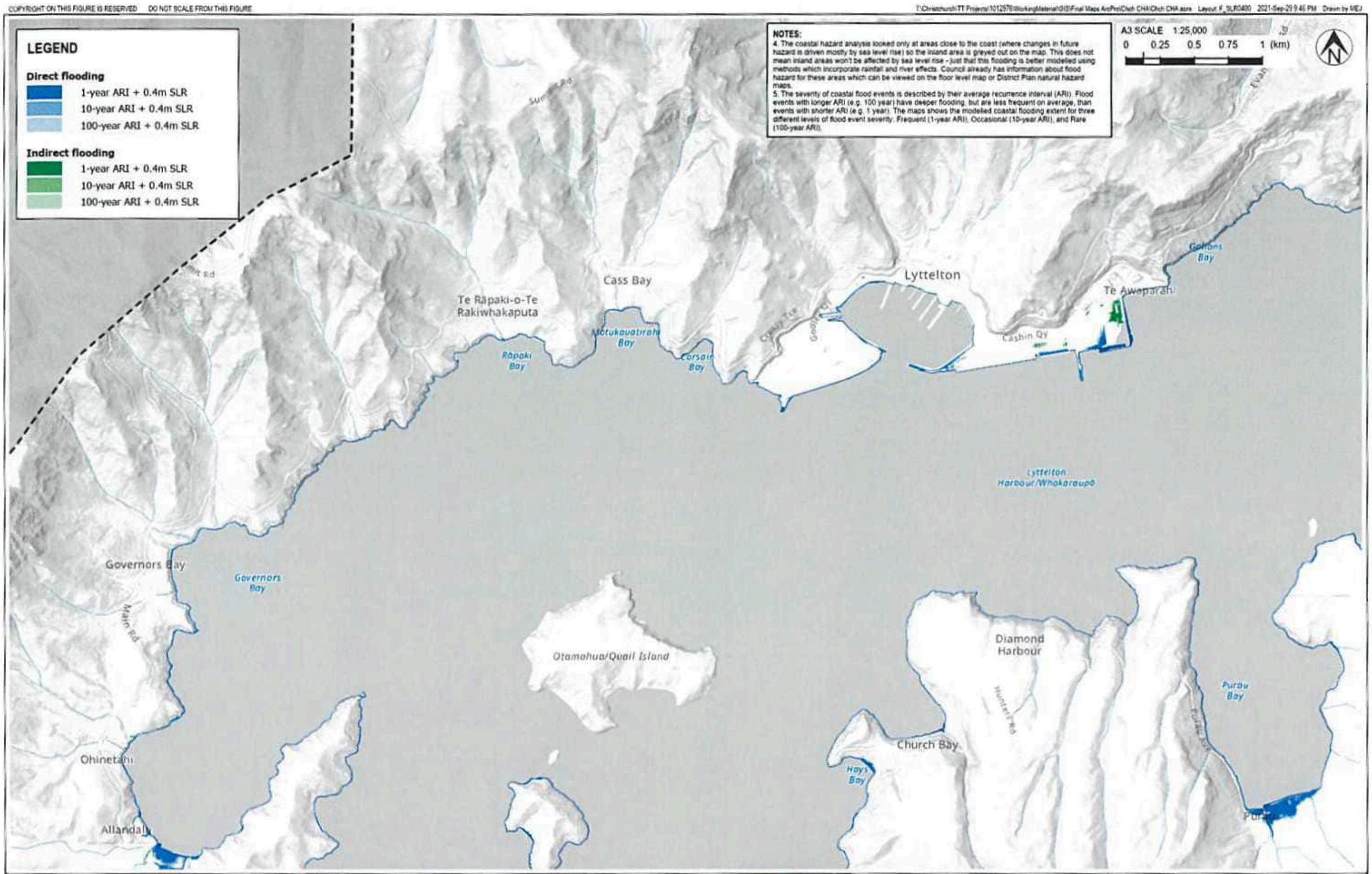

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APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 6C REV 1

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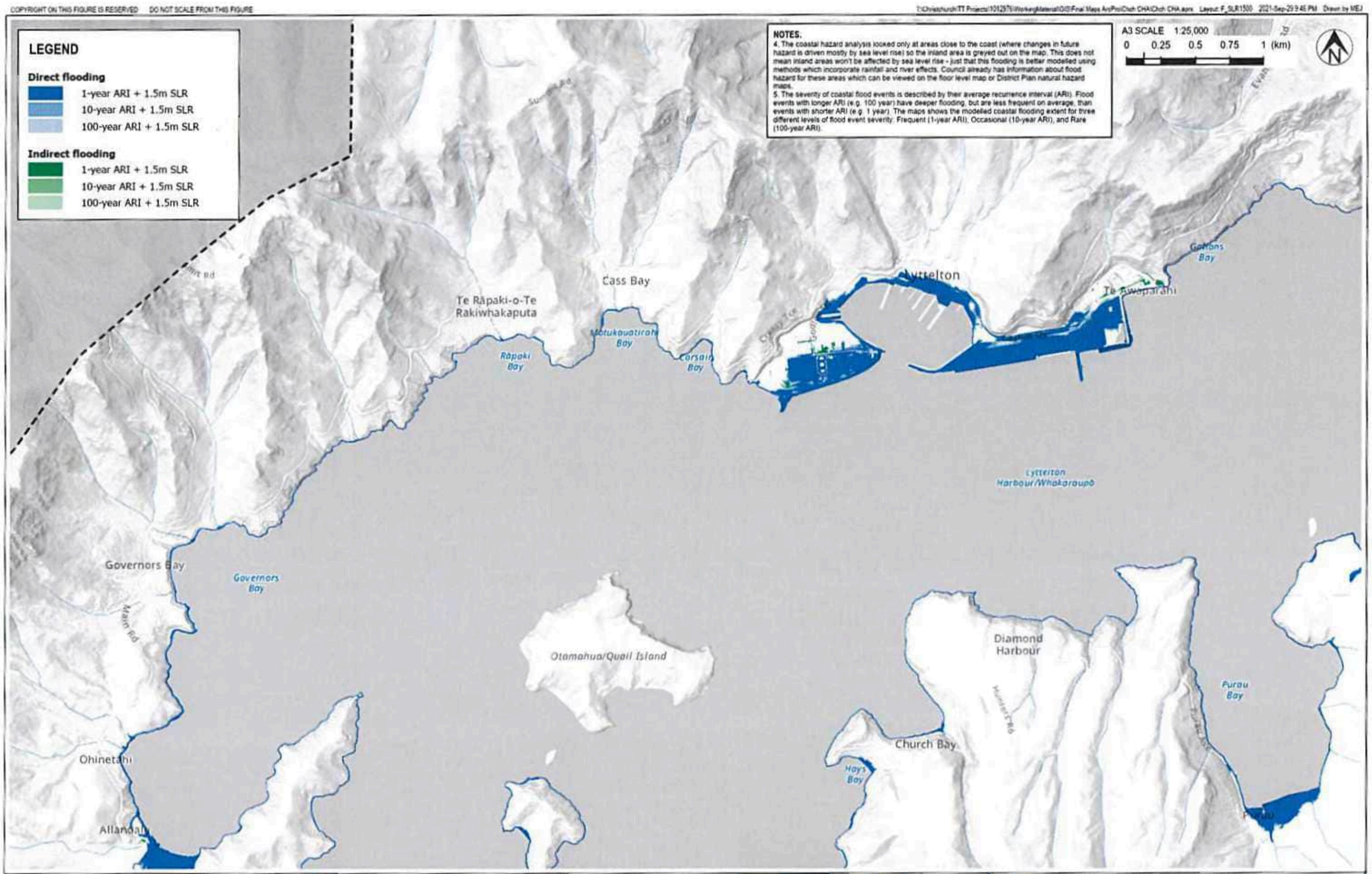

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NOTES:
1. 'Direct flooding' is where there is a direct path for water to flow overland from the coast.
2. 'Indirect flooding' is where the ground is below the modelled water level, but there is no direct overland flow path from the coast.
3. Basemap NZ Hillshade (Alpha): LINZ, Eagle Technology. NZ Topographic Map for use with relief - Grey: Eagle Technology. LINZ, StatsNZ, NIWA, Natural Earth, © OpenStreetMap contributors

REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
APPROVED	P COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:25,000   FIG No. FIGURE 6D   REV 1

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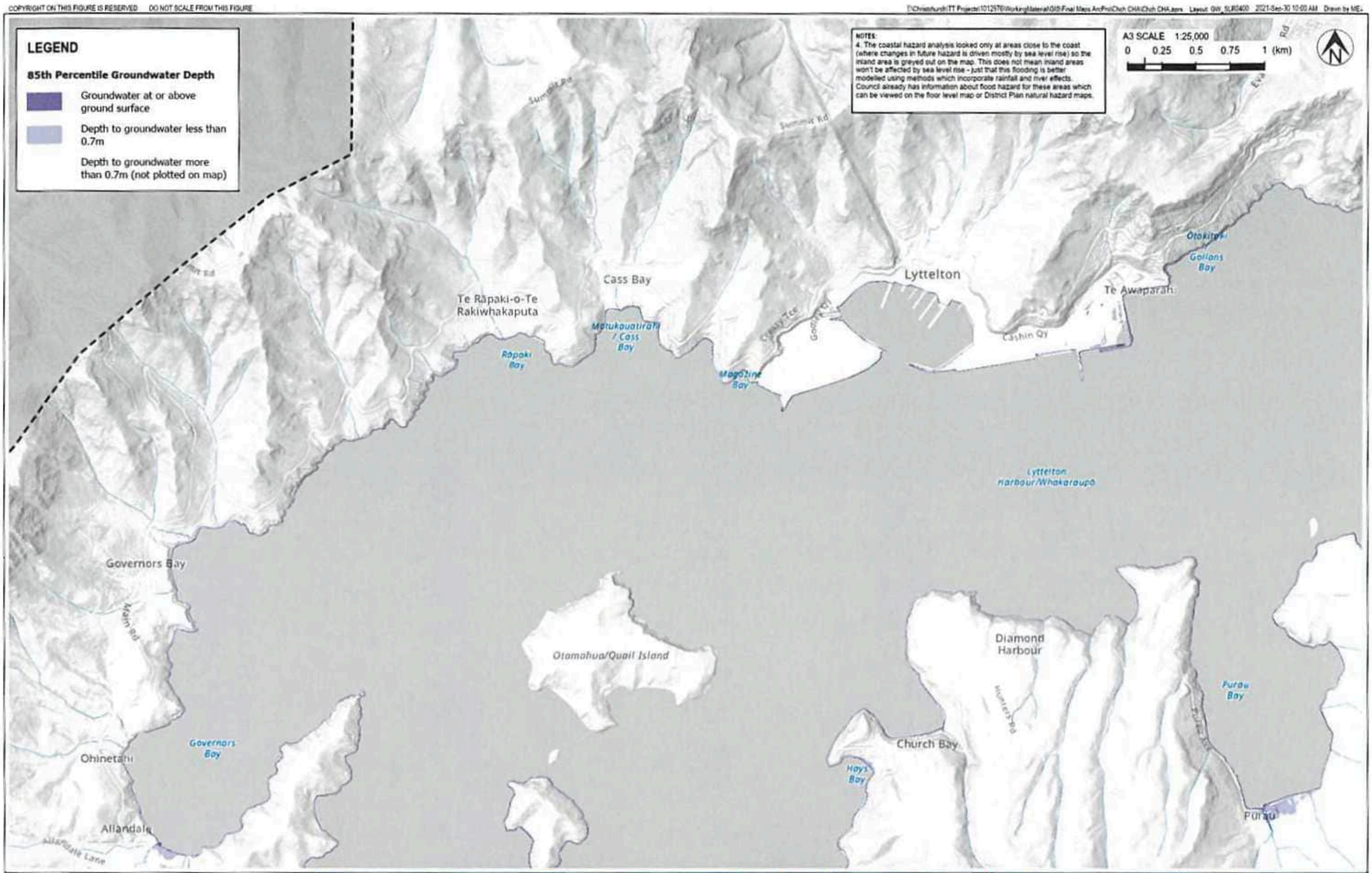
Layout: GW_SLR0400 2021-Sep-30 10:00 AM Drawn by MEJ



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NOTES:

1. For Christchurch urban flatland area, map shows 85th percentile groundwater model from Aqualinc (2020) "LDRP45: Impacts of earthquakes and sea level rise on shallow groundwater levels." Report prepared for Christchurch City Council, August-2020.
2. For Banks Peninsula, map assumes that 85th percentile groundwater level is approximately equal to MHWS high tide level.
3. Basemap NZ Hillshade (Alpha): LINZ, Eagle Technology. NZ Topographic Map for use with relief - Grey: Eagle Technology, LINZ.

REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21


LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
APPROVED: P. COCHRANE	DATE: 29/09/21	

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 6E REV 1

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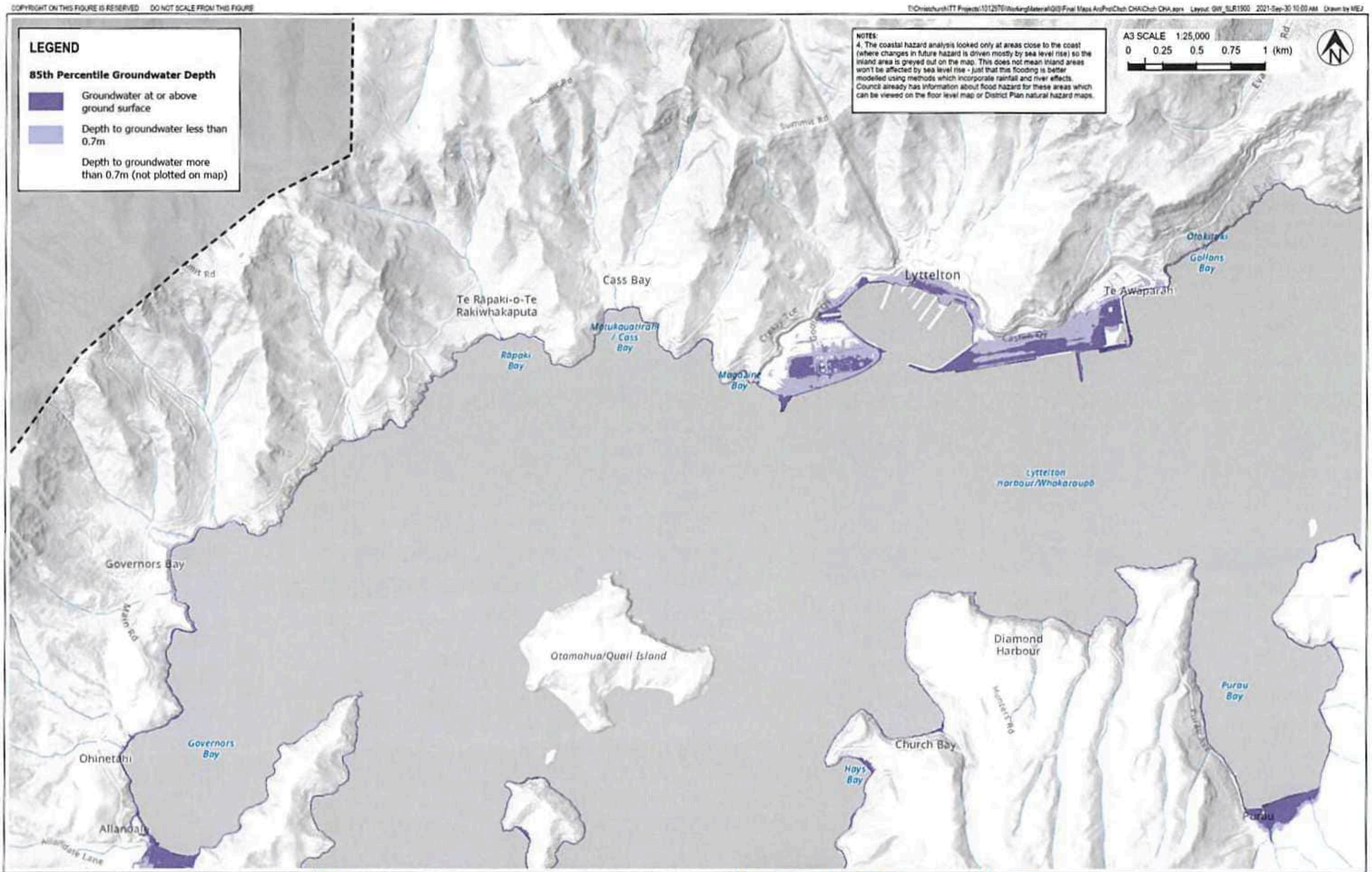

NOTES:			
1. For Christchurch urban flatland area, map shows 85th percentile groundwater model from Aqualinc (2020) "LDRP45: Impacts of earthquakes and sea level rise on shallow groundwater levels." Report prepared for Christchurch City Council, August 2020. 2. For Banks Peninsula, map assumes that 85th percentile groundwater level is approximately equal to MHWS high tide level. 3. Basemap NZ Hillshade (Alpha): LINZ, Eagle Technology. NZ Topographic Map for use with relief - Grey: Eagle Technology, LINZ.			
REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
APPROVED	P COCHRANE	29/09/21

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PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 6F REV 1

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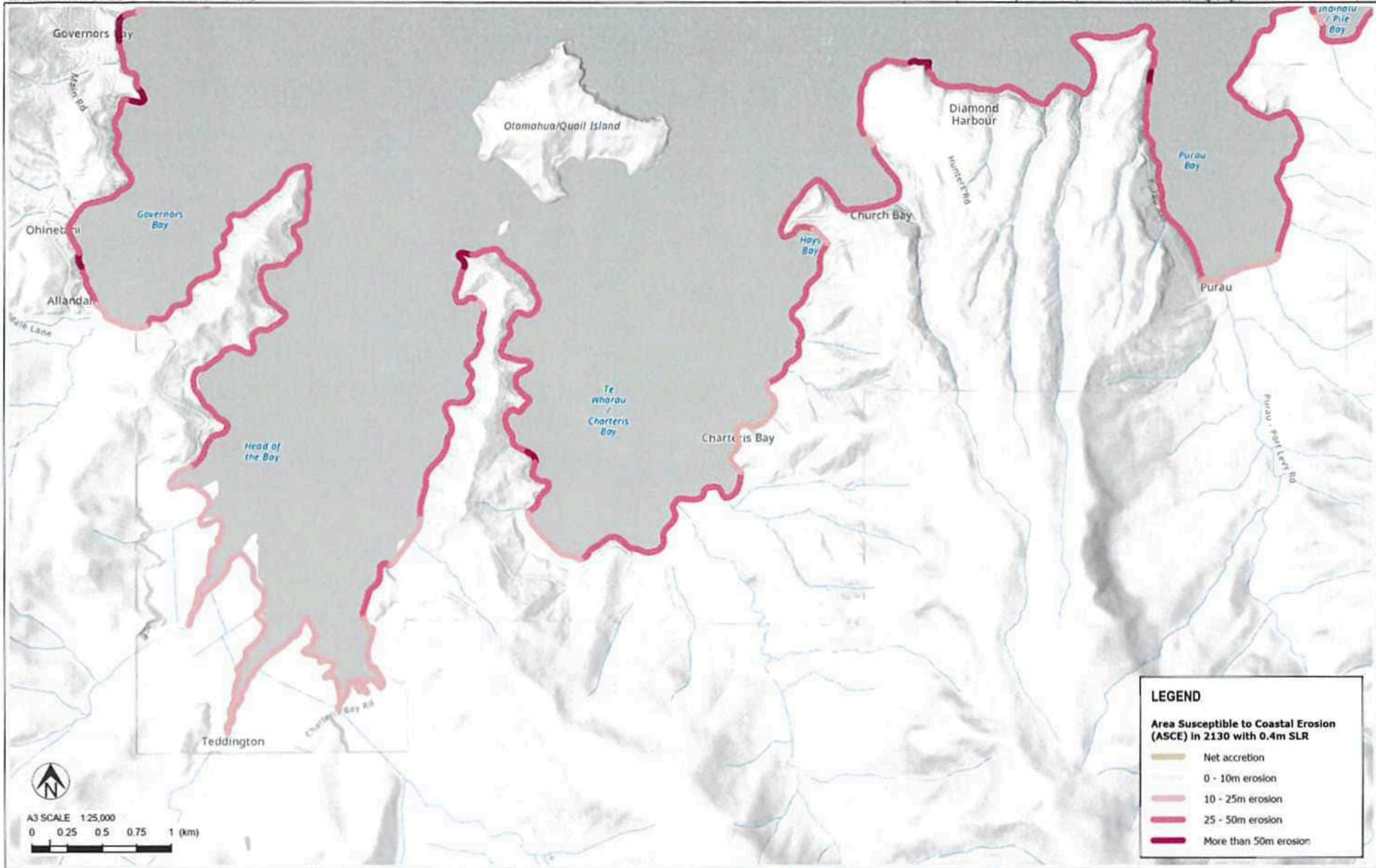

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NOTES:

1. For detailed analysis areas, this map shows the P5 erosion distance (5% probability that erosion will be greater for this scenario).
2. For regional screening analysis areas, this map shows the upper envelope erosion distance.
3. Basemap NZ Hillshade (Alpha): LINZ, Eagle Technology. NZ Topographic Map for use with relief - Grey: Eagle Technology, LINZ, StatsNZ, NIWA, Natural Earth, © OpenStreetMap contributors.

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1	Report issued	MEJ	RHAU	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
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CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL

PROJECT COASTAL HAZARD ASSESSMENT

TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 0.4M SEA LEVEL RISE

SCALE (A3) 1:25,000   FIG No. FIGURE 7A   REV 1

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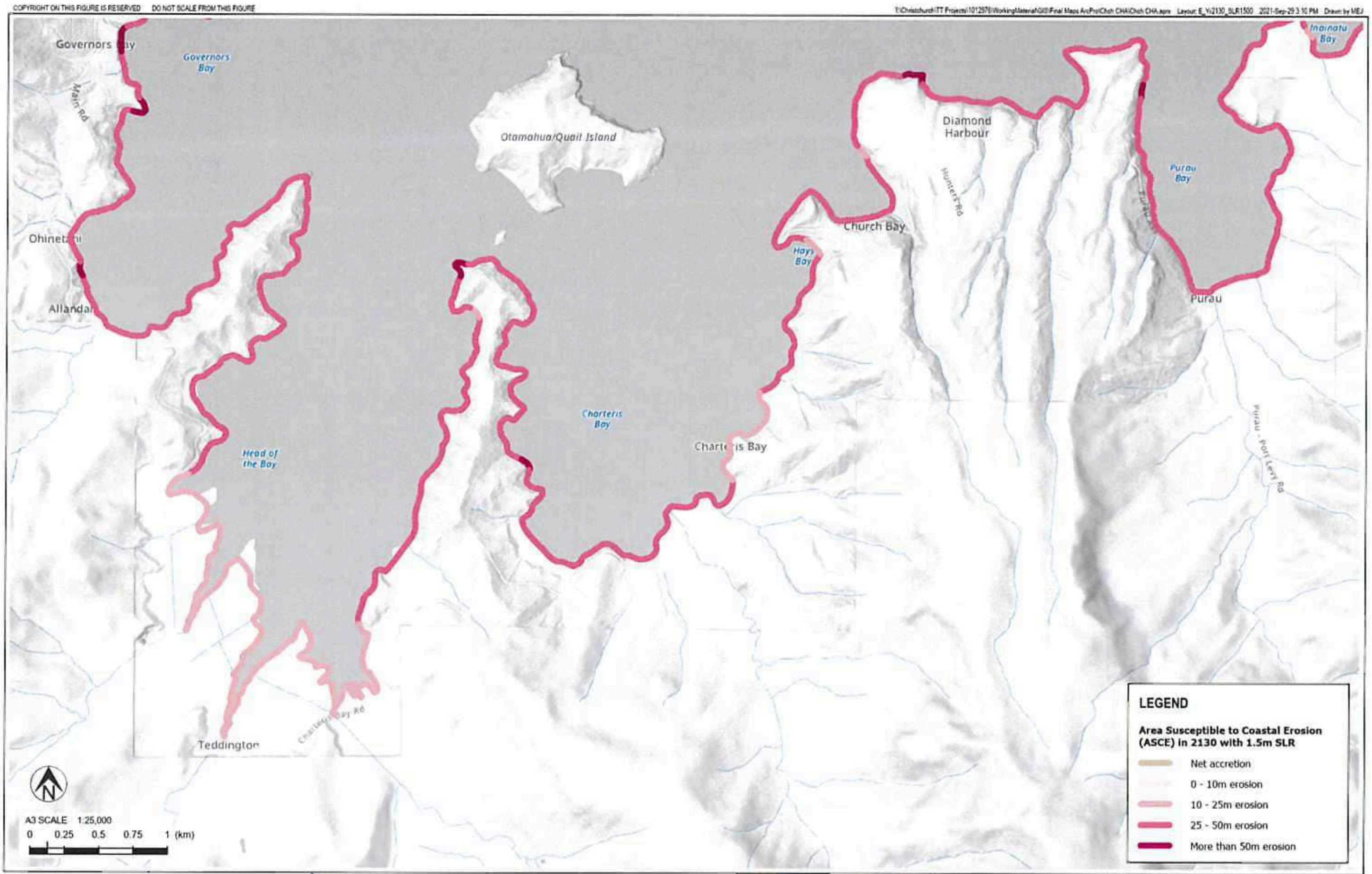

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REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	RHAU	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
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DRAWN	MEJ	SEP.21
CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 1.5M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 7B REV 1

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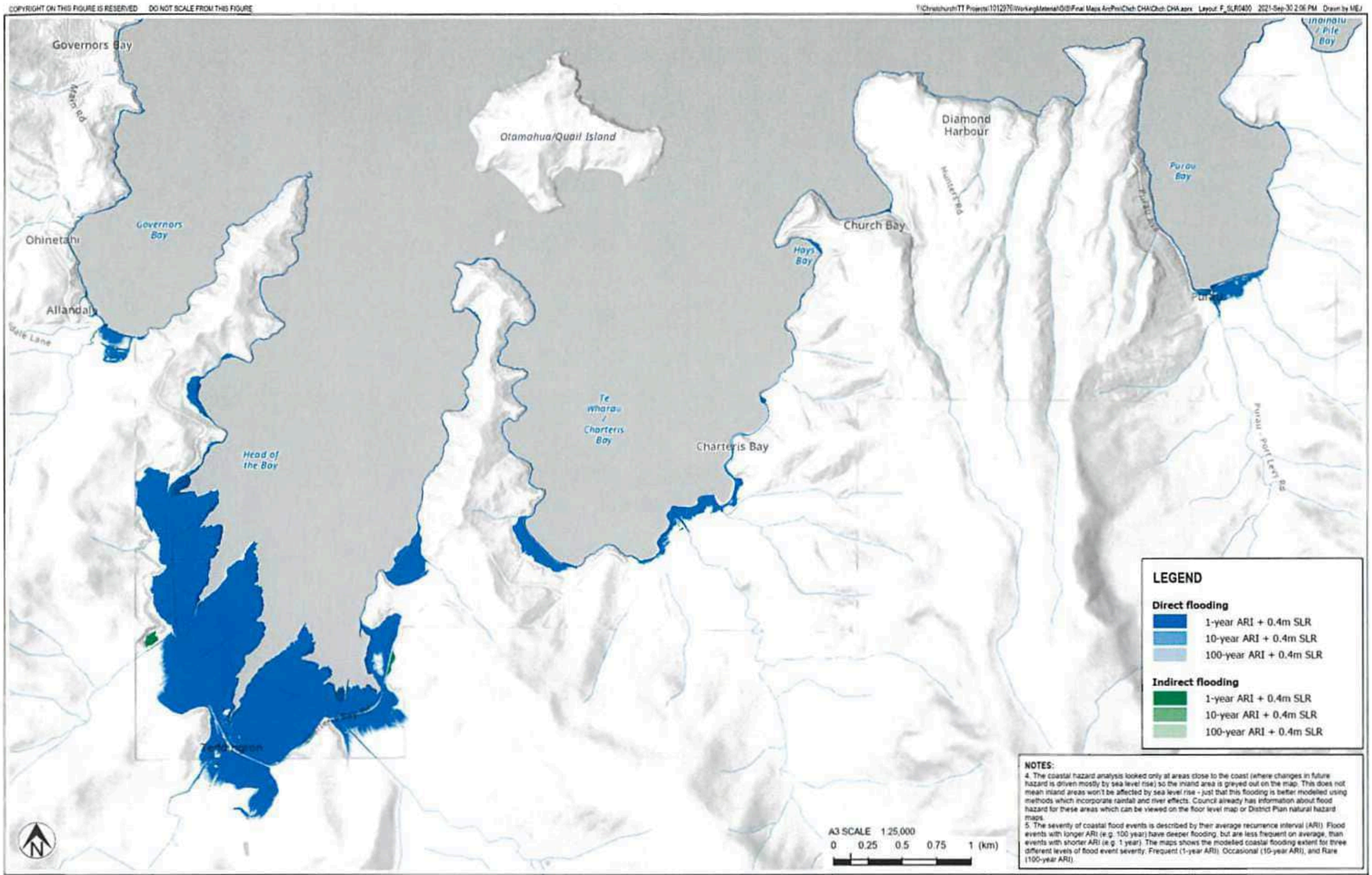

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REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP 21
DRAWN	MEJ	SEP 21
CHECKED	PPK	SEP 21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 7C REV 1

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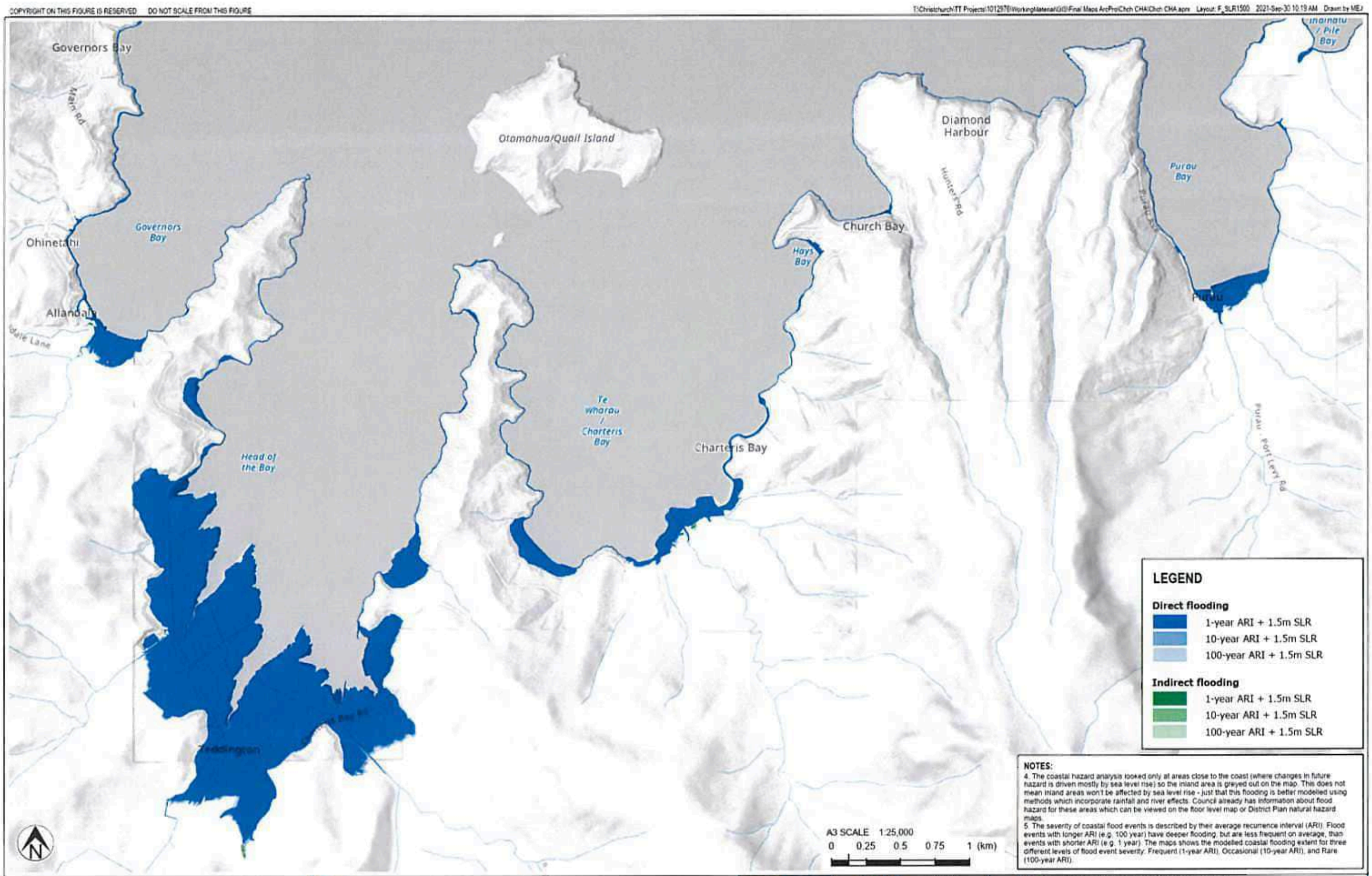

LEGEND

Direct flooding

- 1-year ARI + 1.5m SLR
- 10-year ARI + 1.5m SLR
- 100-year ARI + 1.5m SLR

Indirect flooding

- 1-year ARI + 1.5m SLR
- 10-year ARI + 1.5m SLR
- 100-year ARI + 1.5m SLR

NOTES:

4. The coastal hazard analysis looked only at areas close to the coast (where changes in future hazard is driven mostly by sea level rise) so the inland area is greyed out on the map. This does not mean inland areas won't be affected by sea level rise - just that this flooding is better modelled using methods which incorporate rainfall and river effects. Council already has information about flood hazard for these areas which can be viewed on the floor level map or District Plan natural hazard maps.
5. The severity of coastal flood events is described by their average recurrence interval (ARI). Flood events with longer ARI (e.g. 100 year) have deeper flooding, but are less frequent on average, than events with shorter ARI (e.g. 1 year). The maps shows the modelled coastal flooding extent for three different levels of flood event severity: Frequent (1-year ARI), Occasional (10-year ARI), and Rare (100-year ARI).

A3 SCALE 1:25,000

0 0.25 0.5 0.75 1 (km)

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REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
APPROVED	P COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL

PROJECT COASTAL HAZARD ASSESSMENT

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 7D REV 1

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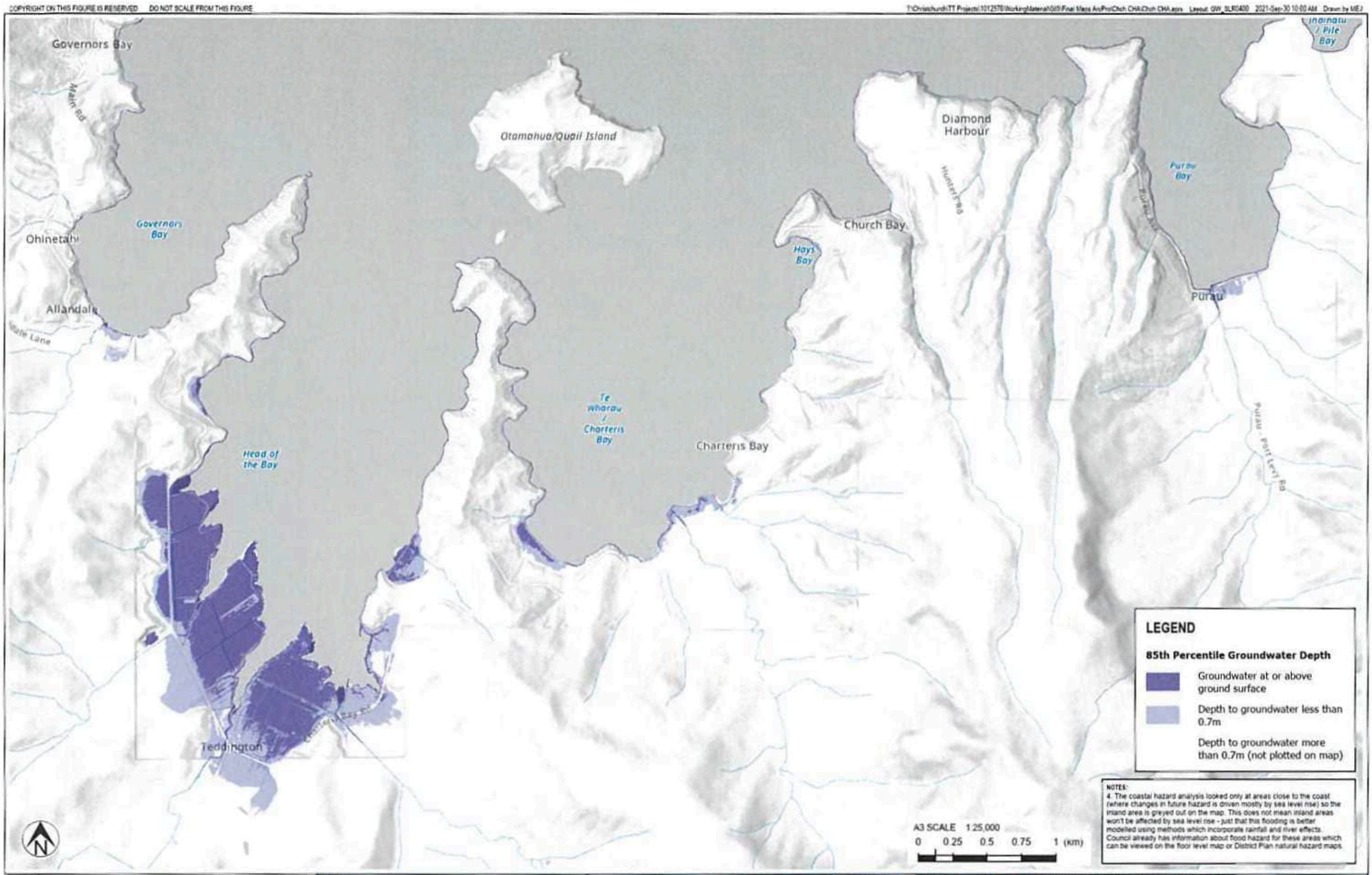

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REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

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DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
	P. COCHRANE	29/09/21
APPROVED		DATE

CLIENT CHRISTCHURCH CITY COUNCIL

PROJECT COASTAL HAZARD ASSESSMENT

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:25,000   FIG No. FIGURE 7E   REV 1

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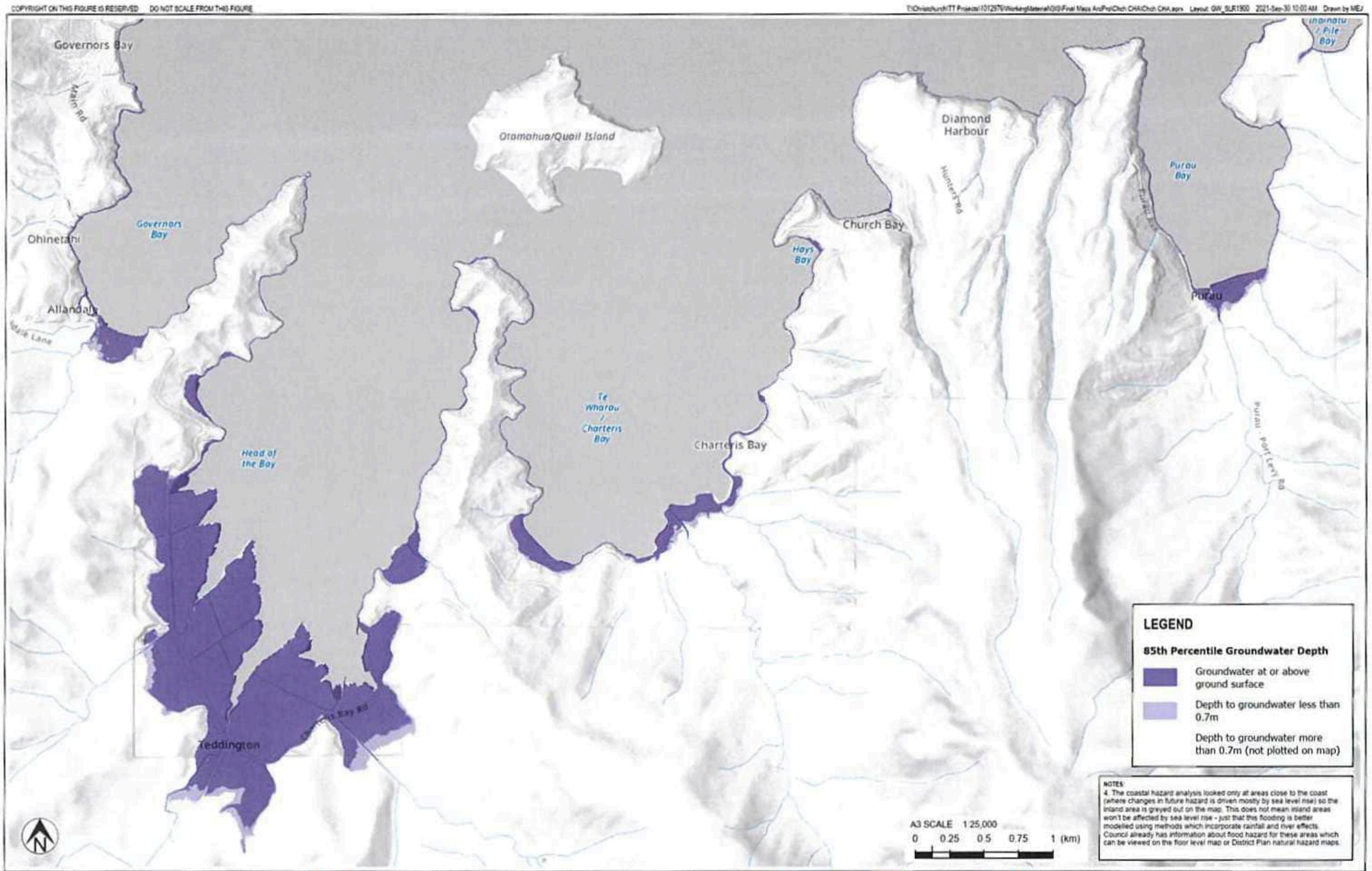

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REV	DESCRIPTION	GIS	CHK	DATE
1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 7F REV 1

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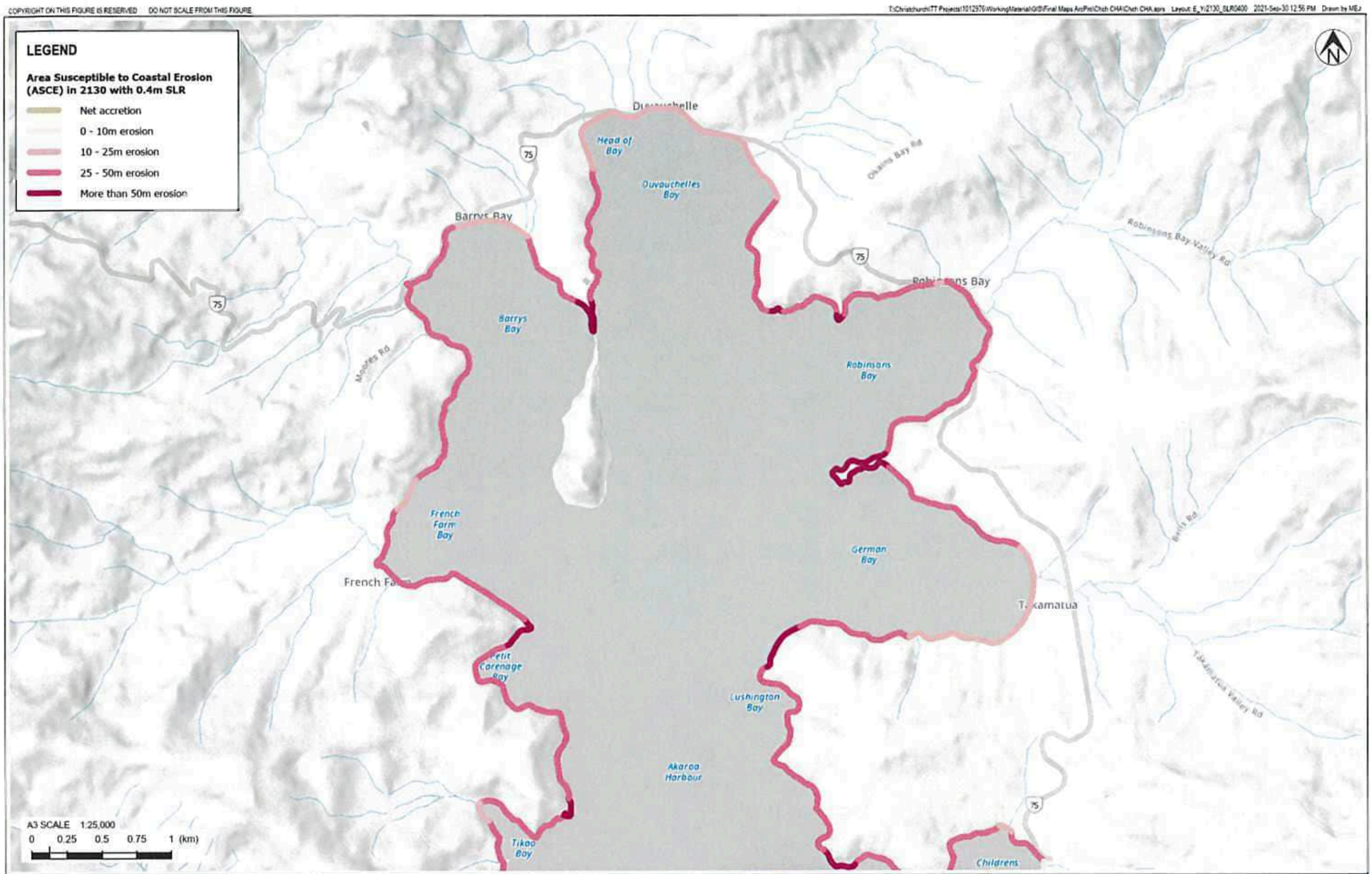

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REV	DESCRIPTION	D/S	CHK	DATE
1	Report issued	MEJ	RHAU	29/09/21

LOCATION PLAN

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 8A REV 1