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.

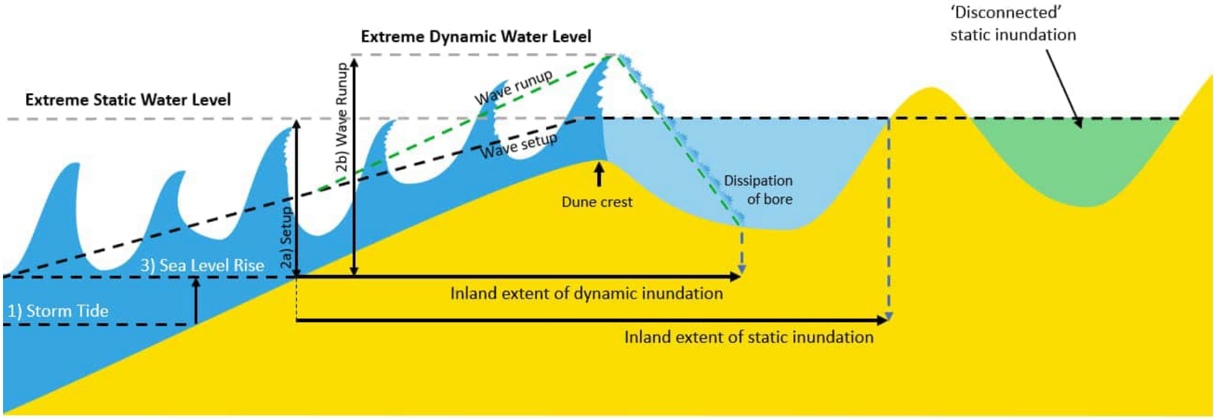

Figure 6.1: Schematisation of extreme water level components and combined extreme water levels.

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.

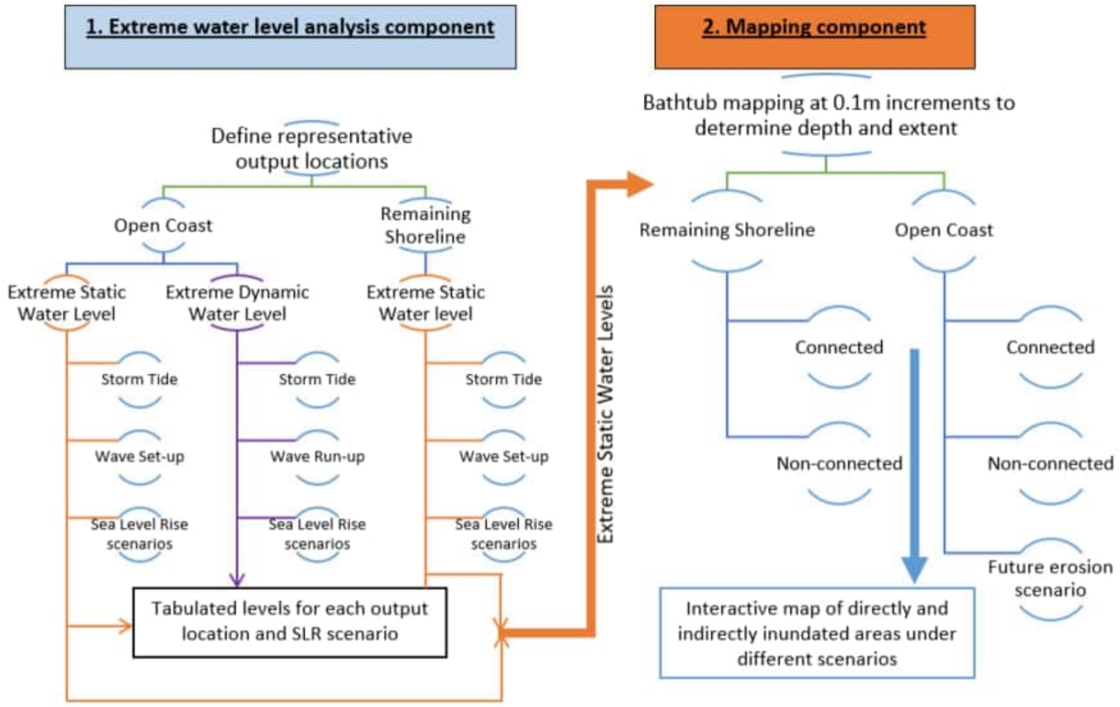

Figure 6.2: Proposed conceptual approach for inundation assessment and mapping.

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).

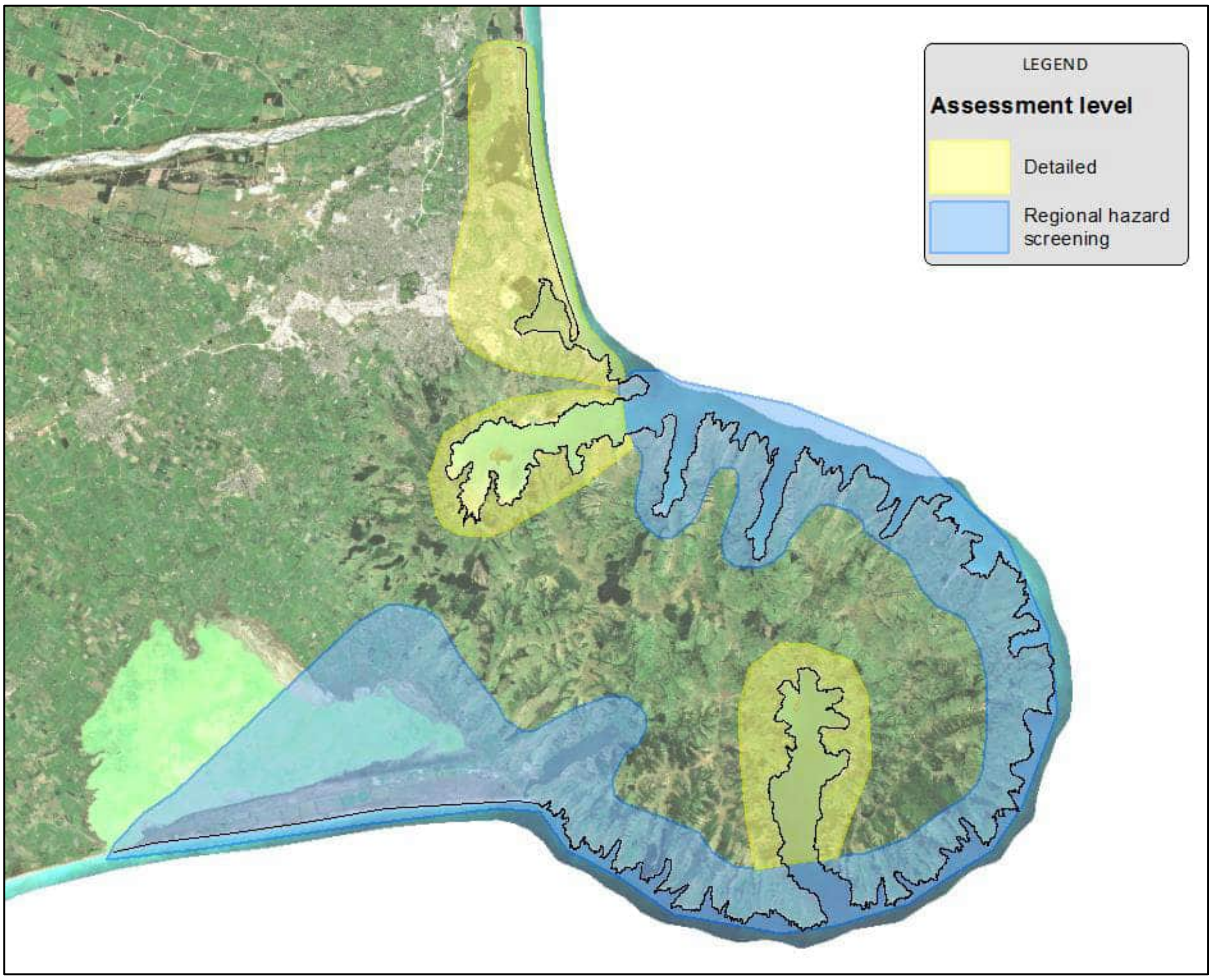

Figure 6-3 Christchurch district showing adopted extents and level of detail for the coastal inundation hazard assessment.

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.

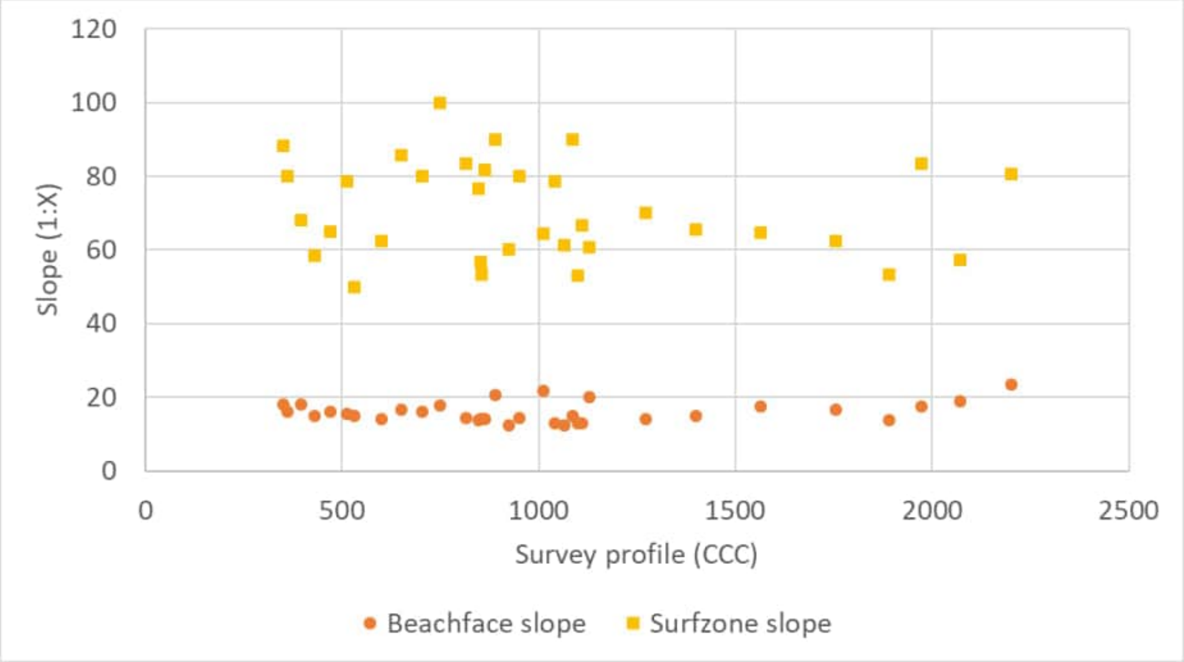

Figure 7.1: Alongshore beachface and surfzone slopes based on averaged surveyed profiles (note that CCC beach profiles run from south to north, e.g. CCC2000 is farthest north).

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).

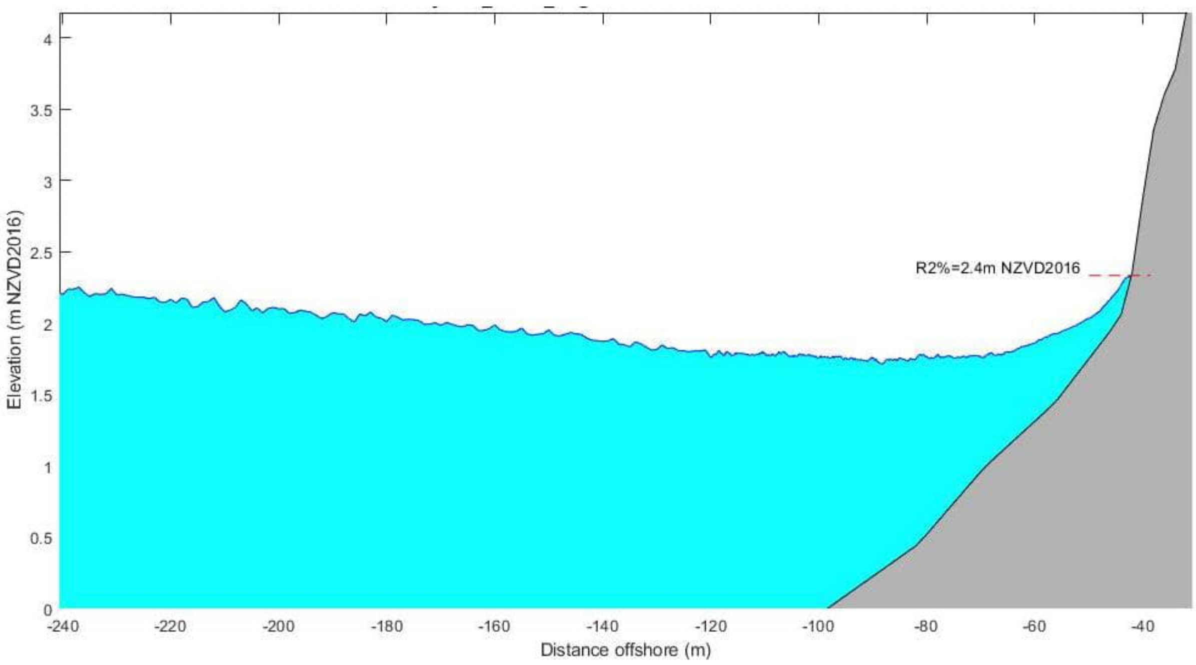

Figure 7.2: Example of XBeach NH model output.

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.

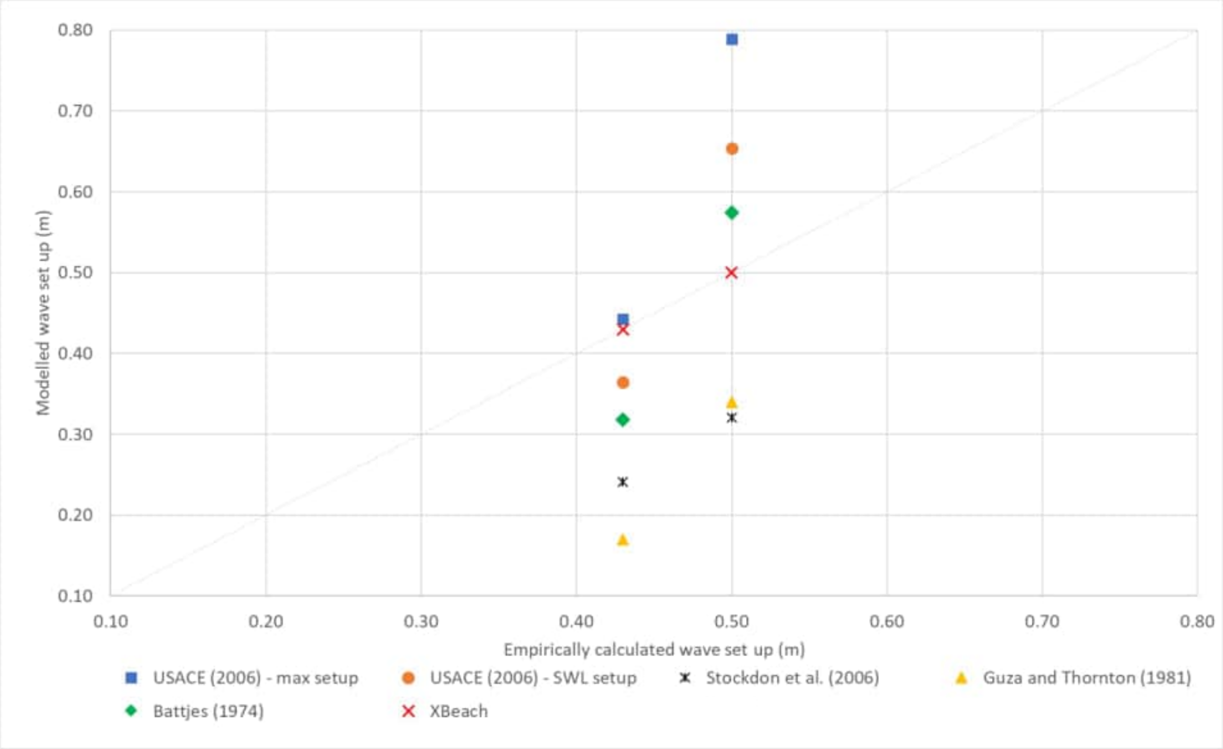

Figure 7.3: XBeach modelled versus empirically calculated wave set up.

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.

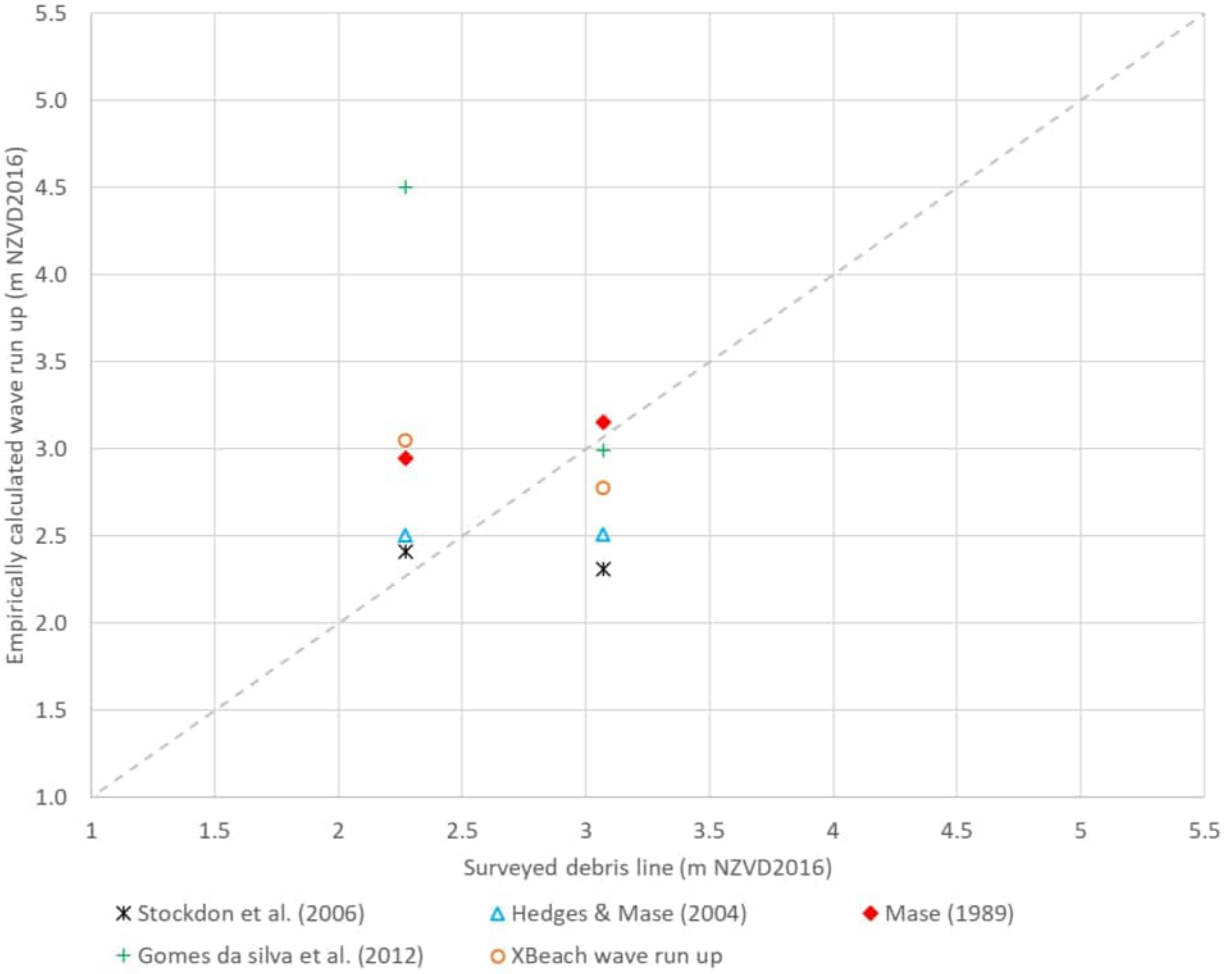

Figure 7.4: Comparison of surveyed debris line (assume wave run up extent) and empirically calculated wave run up.

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.

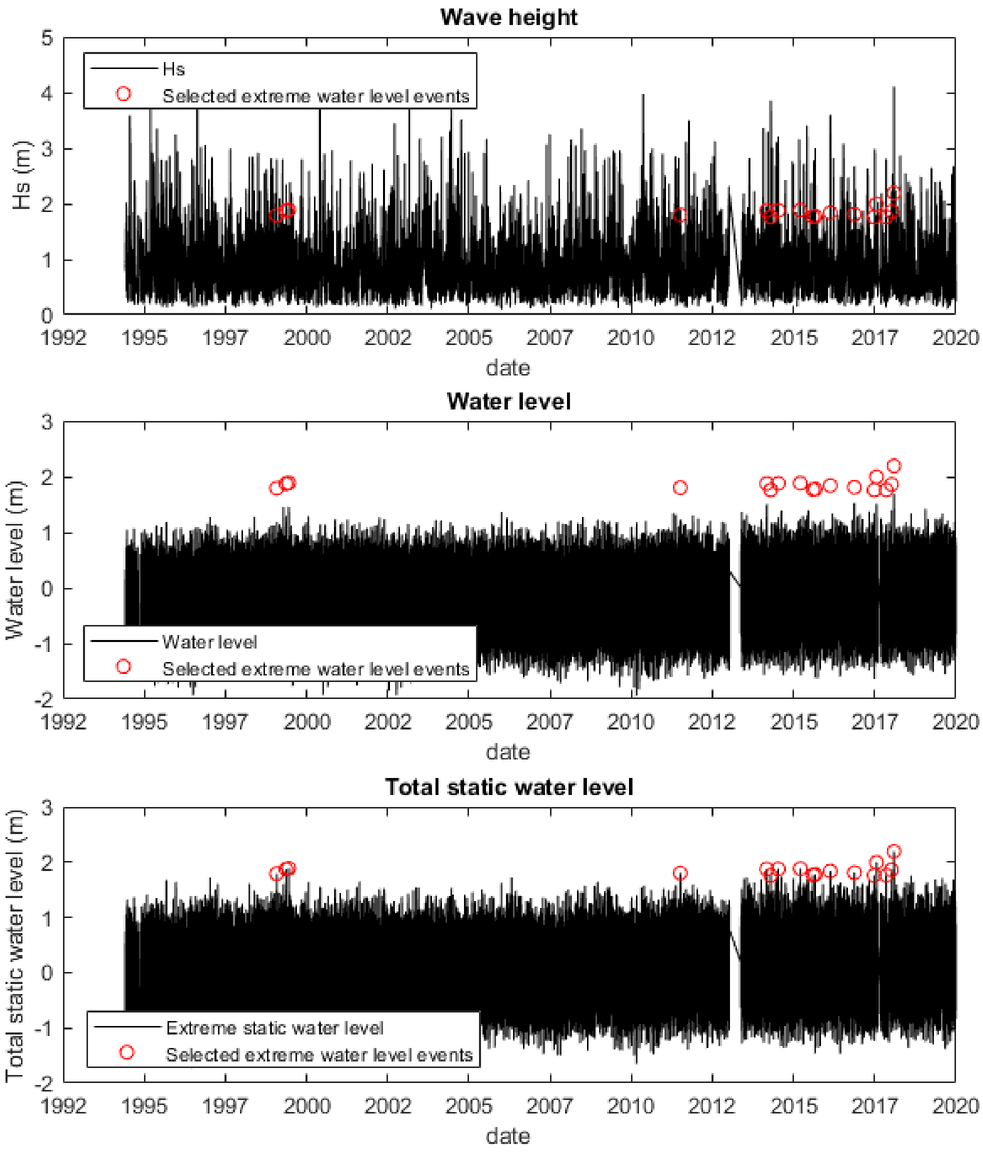

Figure 7.5 Example of extreme water level timeseries derived from a full wave height and water level timeseries.

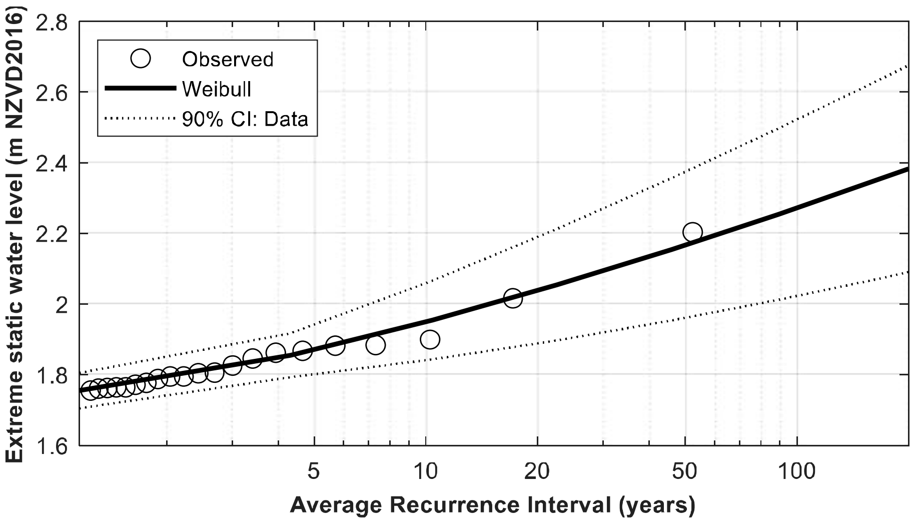

Figure 7.6 Example of extreme values analysis for static inundation level for the Christchurch open coast.

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.

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

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), $m = 0.1$).

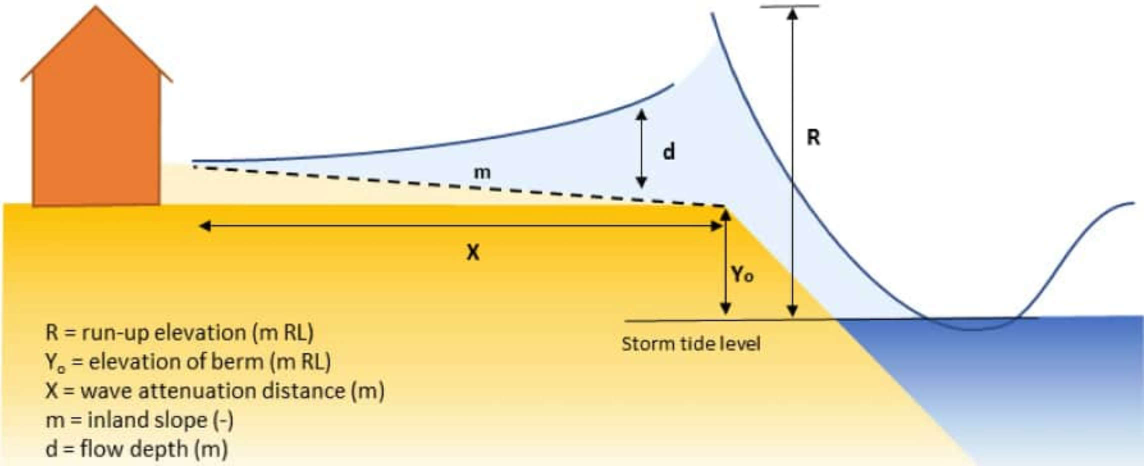

Figure 7.7: Run-up attenuation definition sketch (modified from Cox and Machemehl, 1986).

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.

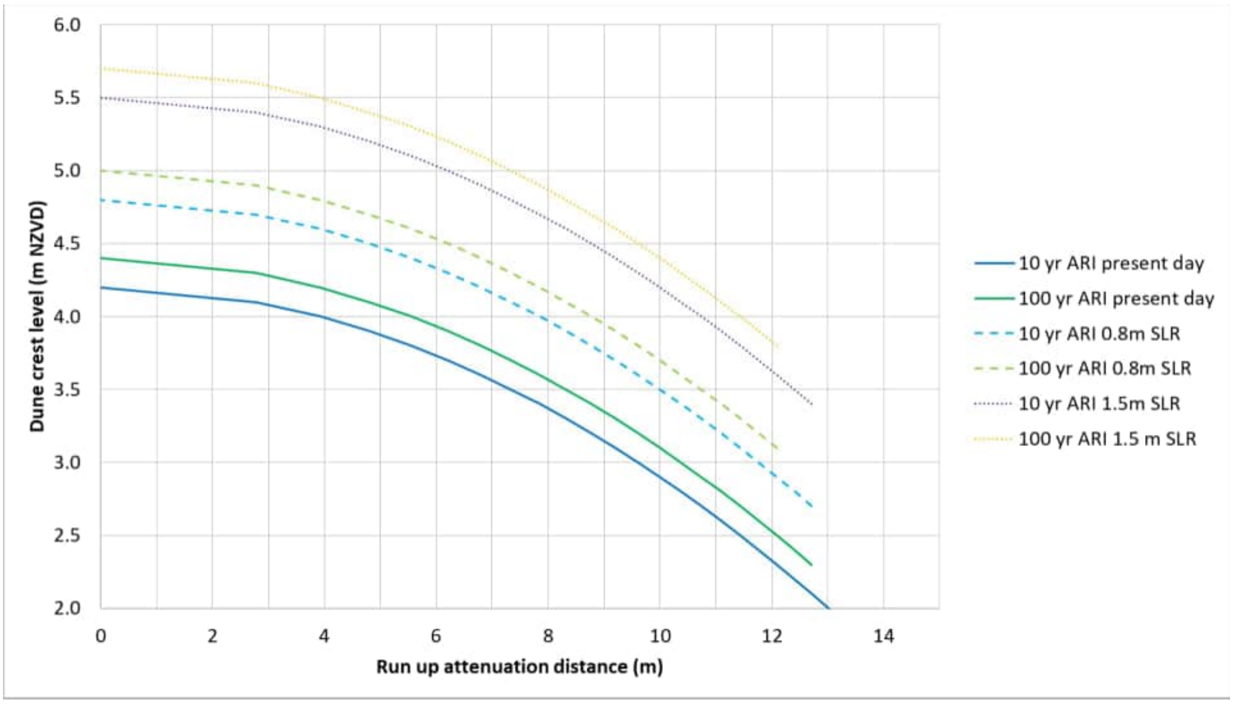

Figure 7.9: Run up attenuation distances from dune crest for a range of dune/seawall crest levels for a range of scenarios based on the modified Cox and Machemehl (1986) method.

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.

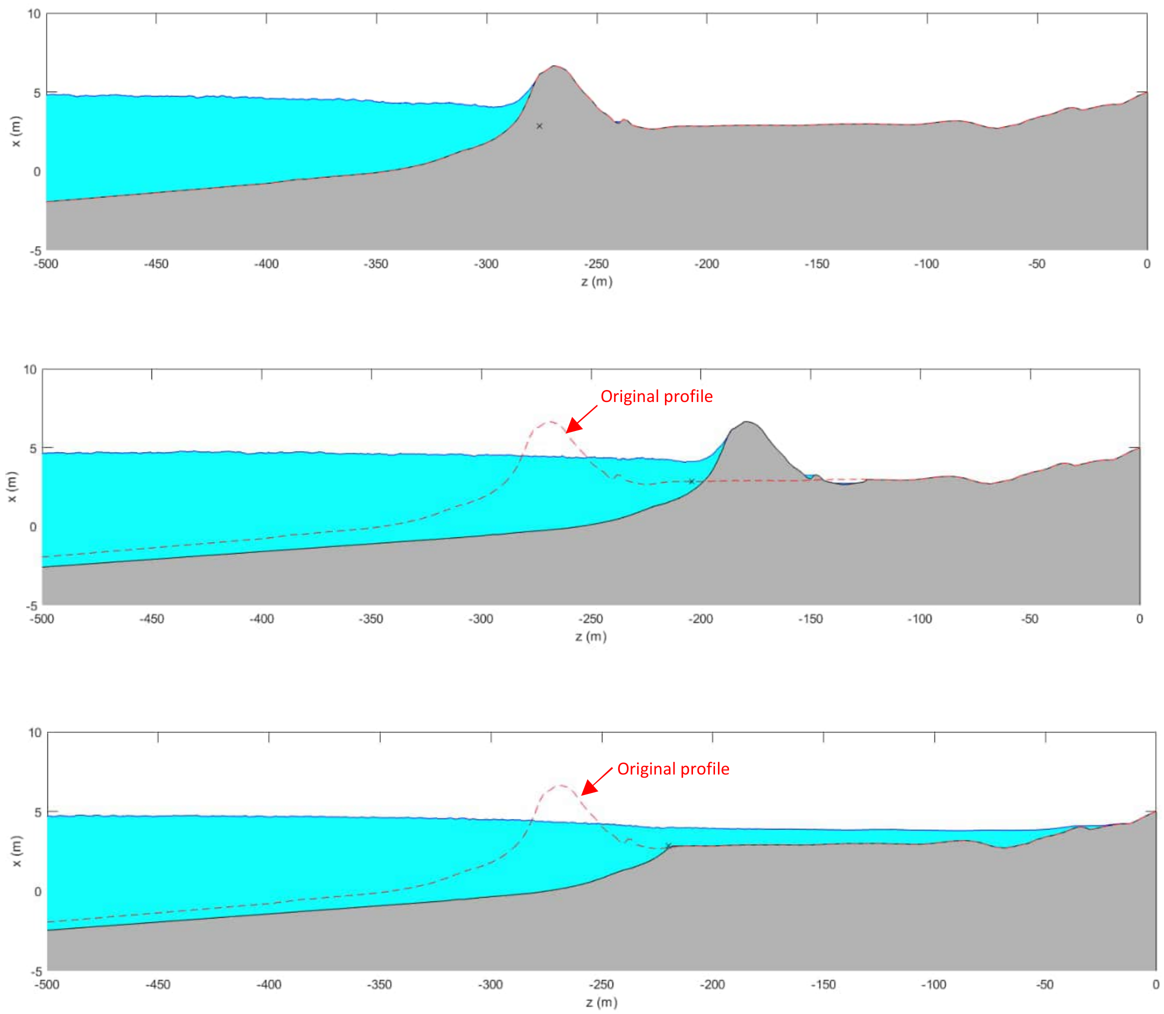

Figure 7.10: Xbeach results for original profile C1065 (1), retreated profile while maintaining dune shape (2) and retreated profile with dunes eroded (3).

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.

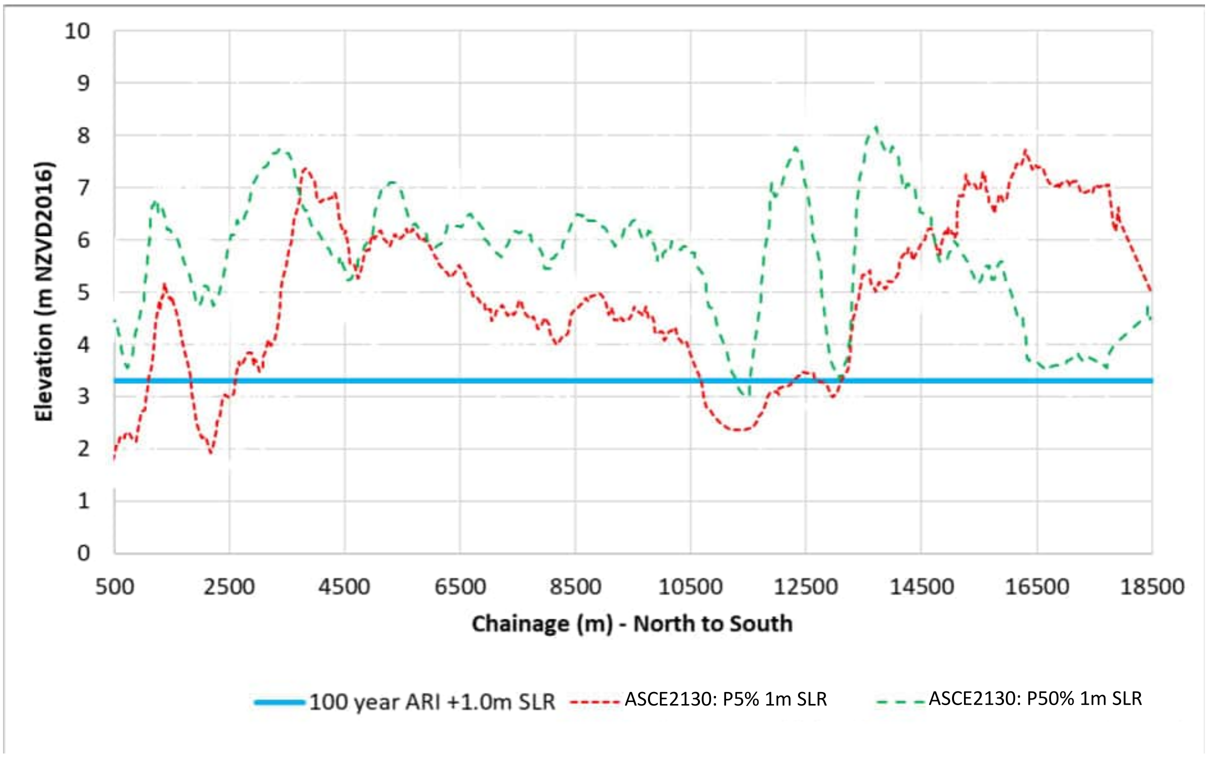

Figure 7.11: Alongshore backshore elevations at 2130 ASCE lines (both P50% and P5%) compared with 100 year ARI + 1 m SLR water level (chainage north to south)

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.

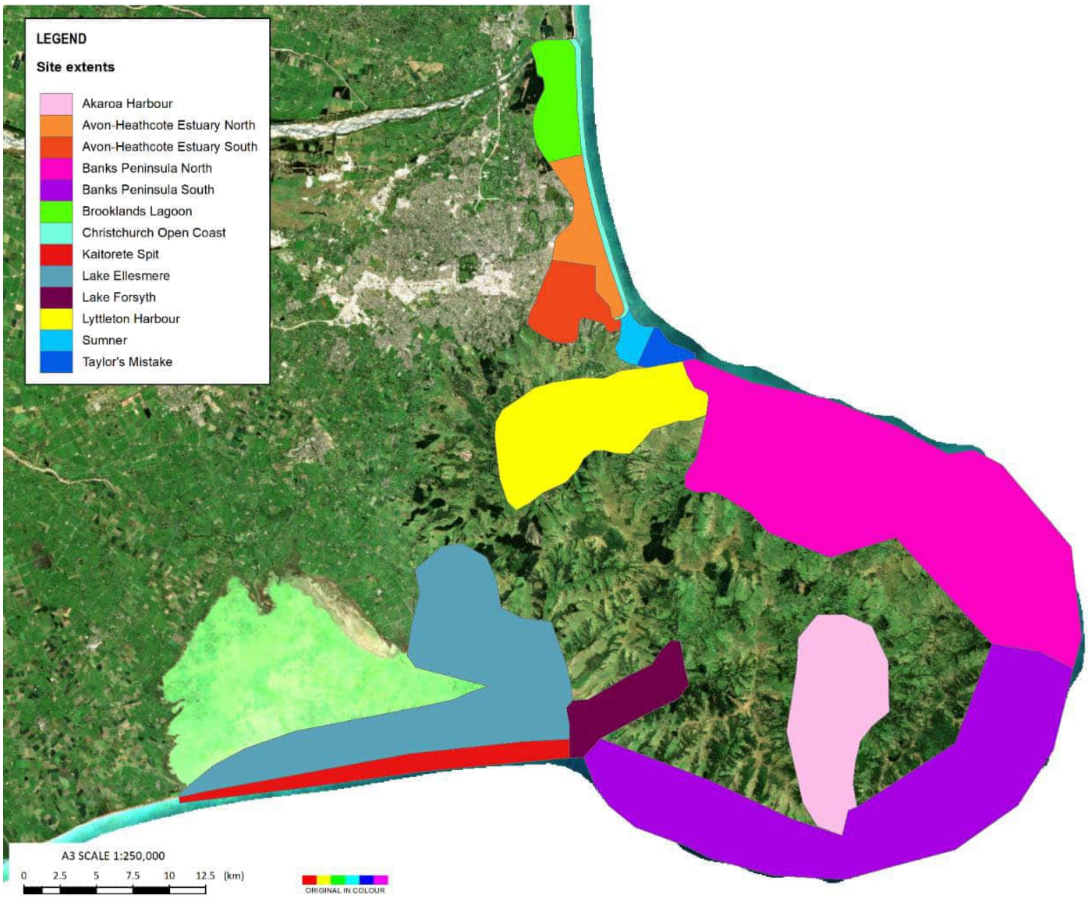

Figure 7.12: Site extents and location for Christchurch open coast, major harbours and estuaries and regional hazard screening sites.

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.

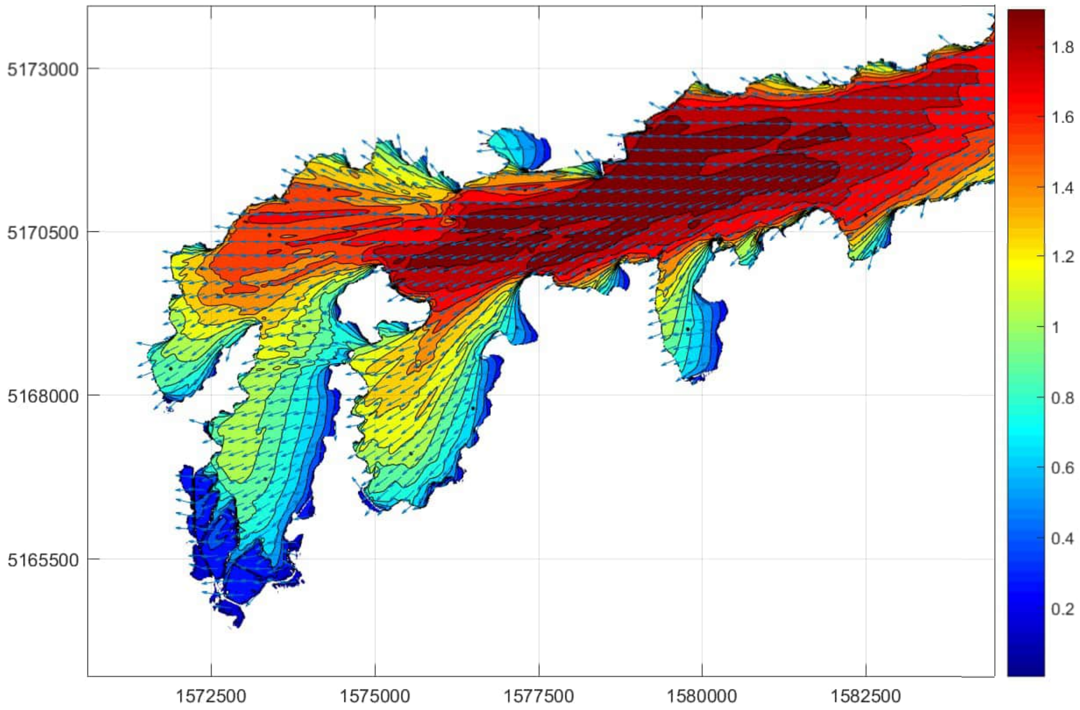

Figure 7.13: SWAN model results for Lyttelton Harbour using 100-year ARI easterly wind as input showing resulting significant wave height (H_s) in metres.

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

Site	ARI							
	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.

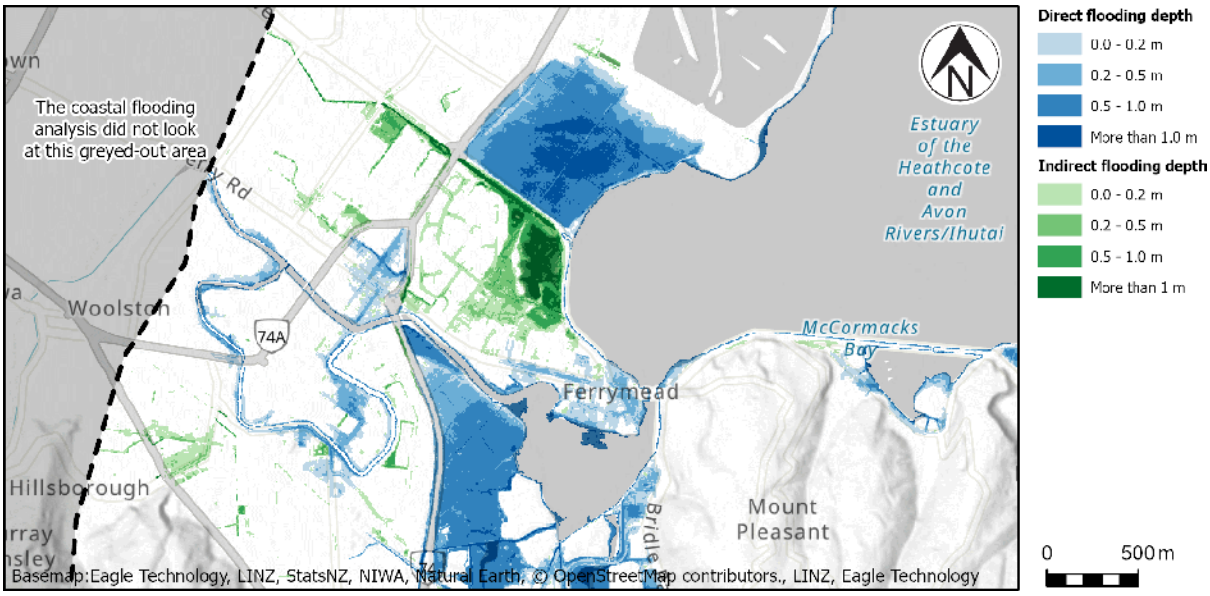

Figure 8.1: Example of static inundation depths for 100 year ARI water levels with for current-day sea level including areas connected to the shoreline (blue shading) and separate inundation areas that are not connected to the coast (green shading).

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.

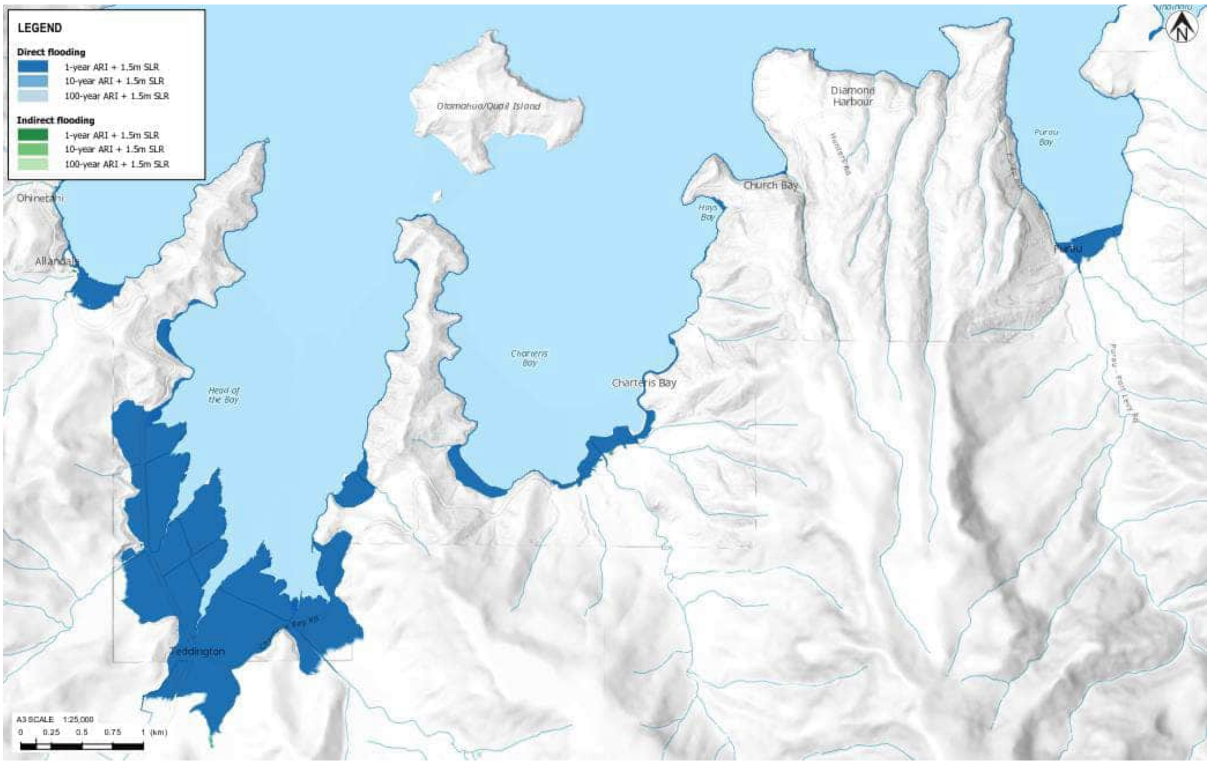

Figure 8.2: Example of static inundation extent map for Lyttelton Harbour (refer Appendix E for full map).

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.

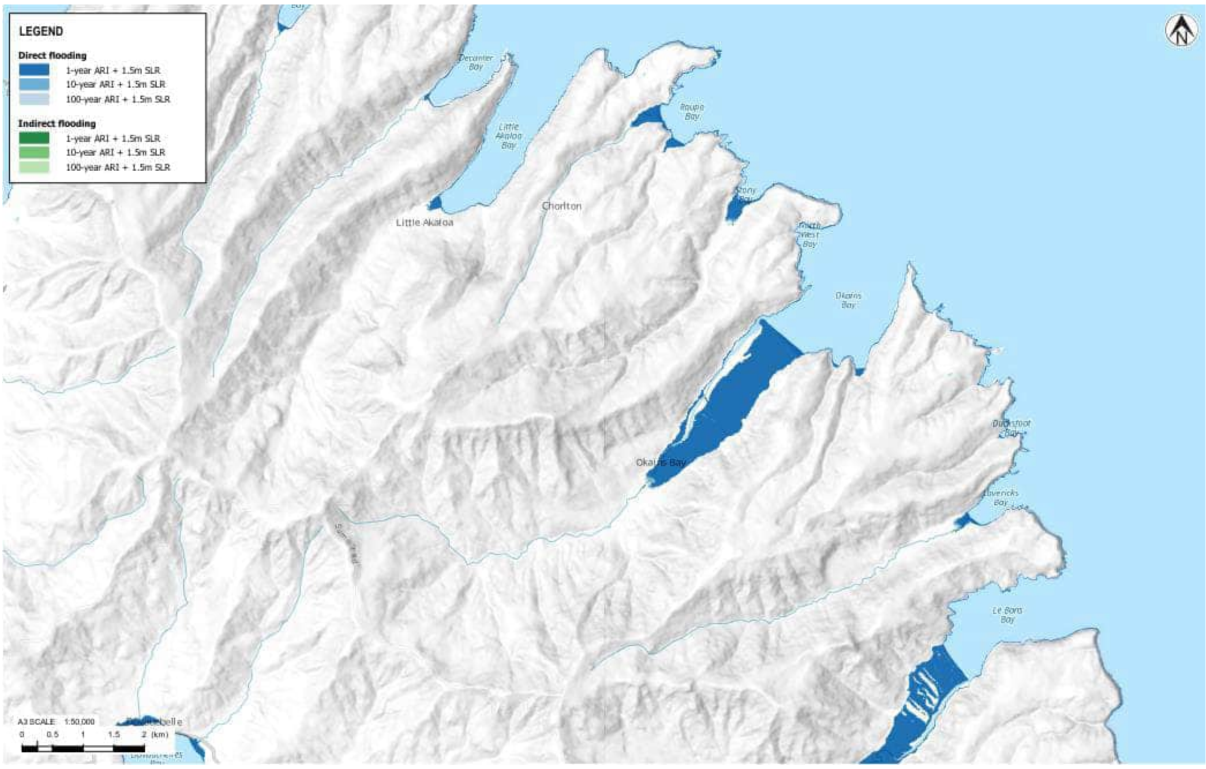

Figure 8.3: Example of static inundation extent map for Banks Peninsula (refer Appendix E for full map).

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.

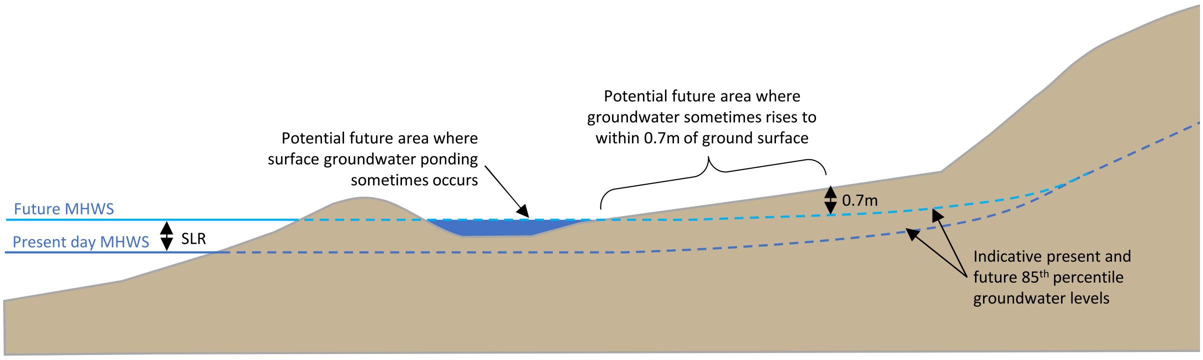

Figure 9.1: Conceptual model for indicative present-day and future groundwater levels for low-lying areas close to the coast around Banks Peninsula.

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).

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[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.

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11 References

Ashton, A.D., Walkden, M.J. and Dickson, M.E. (2011) Equilibrium responses of cliffed coasts to changes in the rate of SLR. Marine Geology. 284 (2011) 217-229.

Aqualinc (2020) LDRP45: Impacts of earthquakes and sea level rise on shallow groundwater levels. Report prepared for Christchurch City Council, August-2020.

Battjes, J.A., 1974. Computation of set-up, longshore currents, run-up and overtopping due to wind-generated waves. Delft University of Technology.

Beavan, R.J., Litchfield, N.J. (2012) Vertical land movement around the New Zealand coastline: implications for sea-level rise. GNS Science Report 2012/29. 41p.

Brown, L.J. and Weeber, J.H. (1992) Geology of the Christchurch Urban Area. Institute of Geological & Nuclear Sciences Ltd. 104p + Map.

Bruun P (1962) Sea-level rise as a cause of shore erosion. J. Waterways Harbors Div 88:117–130.

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

Cope, J. (2018). A coastal hazard screening assessment for Selwyn District. Environment Canterbury Regional Council. Report No. R18/68.

Cowell, P.J., Thom, B.G., Jones, R.A., Everts, C.H. and Simanovic, D. (2006) Management of Uncertainty in Predicting Climate-Change Impacts on Beaches. JCR 22(1)232-245.

Cox, J.C. and Machemehl, J. (1986). Overload bore propagation due to an overtopping wave. Journal of Waterway, Port, Coastal and Ocean Engineering, Vol. 112, No. 1.

CCC (2019). The stormwater and tide interface in Christchurch. 2019 Stormwater Conference and Expo.

da Silva, P.G, Medina, R., González, M., Garnier, R., 2019. Wave reflection and saturation on natural beaches: The role of the morphodynamic beach state in incident swash. Coast. Eng. 153, 103540. https://doi.org/10.1016/j.coastaleng.2019.103540

Dingwall, P. (1974). Bay-head Sand Beaches of Banks Peninsula, New Zealand. New Zealand Department of Scientific and Industrial Research. New Zealand Oceanographic Institute Memoir No. 15.

Findlay, R. and Kirk, R. (1988). Post-1847 changes in the Avon-Heathcote Estuary, Christchurch: A study of the effect of urban development around a tidal estuary, New Zealand Journal of Marine and Freshwater Research, 22:1, 101-127, DOI: 10.1080/00288330.1988.9516283

Forsyth, P.J. Barrell, D.J.A. and Jongens, R. (2008) Geology of the Christchurch Area. Institute of Geological & Nuclear Sciences Ltd. 67p + Map.

GHD (2018). Citywide Flood Modelling (LDRP044). Model Schematisation – Avon/Estuary, Heathcote and Sumner – Rev 6. June 2018.

GHD (2021). LDRP097 Multi-Hazard Baseline Modelling Joint Risks of Pluvial and Tidal Flooding. Report for Christchurch City Council. February 2021.

Goda, Y. (2003). Revisiting Wilson's formulas for simple wind-wave prediction. J. Waterway, Port, Coastal, and Ocean Engrg. 129(2), pp. 93-95.

Goring, D. (2008). Winds for the Avon/Heathcote Estuary. Mulgor Consulting Ltd.

Goring, D. (2016). Effects of development stages 2 and 3 on waves and tidal currents in Lyttelton Harbour. Prepared for Lyttelton Port of Christchurch.

Goring, D. (2018). Extreme sea levels at Christchurch sites: EV1 analysis. Mulgor Consulting Ltd.

Guza, R.T., Thornton, E.B., 1981. Wave set-up on a natural beach. J. Geophys. Res. 86, 4133–4137. https://doi.org/10.1029/JC086iC05p04133

Hallermeier, R.J. (1983) Sand transport limits in coastal structure designs. Proc. of Conf. on Design, Construction, Maintenance and Performance of Coastal Structures. ASCE, Arlington, pp 703–716.

Hart, D., Marsden, I., Todd, D., Vries, W. (2008). Mapping of the bathymetry, soft sediments and biota of the seabed of upper Lyttelton Harbour. Estuarine Research Report 36. Prepared for Environment Canterbury.

Hart, Deirdre E., UC Integrated Coastal Research Centre., DTEC Consulting Ltd., Canterbury (N.Z.), Environment Canterbury (2009). Upper Akaroa Harbour seabed bathymetry and soft sediments: a baseline mapping study.

Hedges, T & Mase, H. (2004). Modified Hunt's Equation Incorporating Wave Setup. Journal of Waterway Port Coastal and Ocean Engineering 130, 109-113.

Hicks, M., Gorman, R., Measures, R., Walsh., Bosserelle, C. (2018a). Coastal sand budget for Southern Pegasus Bay – Stage A. Prepared for Christchurch City Council. NIWA.

Hicks, M., Measures, R., Gorman, R. (2018b). Coastal sand budget for Southern Pegasus Bay – Stage B: Future sand budget. Prepared for Christchurch City Council. NIWA.

IPCC (2013). Working Group I contribution to the IPCC 5th Assessment Report "Climate Change 2013: The Physical Science Basis". Report by Intergovernmental Panel on Climate Change. June 2013.

IPCC (2021). Working Group I contribution to the IPCC 6th Assessment Report "Climate Change 2021: The Physical Science Basis". Report by Intergovernmental Panel on Climate Change. August 2021.

Komar, P.D., McDougal, W.G., Marra, J.J. and Ruggiero, P. (1999). The Rational Analysis of Setback Distances: Application to the Oregon Coast. Shore and Beach Vol. 67 (1) 41-49.

Kenderdine, SE; Hart, DE; Cox, RJ; de Lange; WP; Smith, MH. (2016) Peer review of the Christchurch Coastal Hazards Assessment Report. Review report produced for the Christchurch City Council, 18 August 2016, 74pp.

Kirk, R.M. (1987) Dynamics and management of sand beaches in southern Pegasus Bay, Christchurch. Unpublished report for Morris and Wilson Consulting Engineers Ltd.

Kriebel & Dean (1993). Convolution method for time-dependant beach-profile response. J. Waterway, Port, Coastal, Ocean Eng., 1993, 119(2): 204-226

MacPherson, J.M. (1978) Environmental geology of the Avon-Heathcote Estuary. Unpublished PhD. Thesis, University of Canterbury. 178p.

Mase, H (1989). Random Wave Run-up Height on Gentle Slopes, Journal of the Waterway, Port, Coastal and Ocean Engineering Division, American Society of Civil Engineers, pp 593-609.

Measures, R., Hicks, M., Shankar, U., Bind, J., Arnold, J., Zeldis, J. (2011). Mapping earthquake induced topographical change and liquefaction in the Avon-Heathcote Estuary. Report prepared for Environment Canterbury.

Measures, R., Cochrane, T., Caruso, B., Walsh, J., Horrell, J., Hicks, M., Wild, M. (2014). Analysis of Te Waihora lake level control options. Report prepared for Ngāi Tahu and Environment Canterbury. NIWA and University of Canterbury.

MfE (2017) Coastal Hazards and Climate Change. Guidance for Local Government in New Zealand. 3rd edition. Prepared for Ministry for the Environment. Ramsey, D., Gibberd, B., Dahm, J., Bell, R. (2012) Defining coastal hazard zones for setback lines: A guide to good practice. NIWA Ltd. 91 p.

Nicholls, R. J., Laron, M., Capobianco, M. and Birkemeier, W.A. (1998) Depth of Closure: Improving Understanding and Prediction. ICCE 1998

NIWA (2015). Storm-tides and wave runup in the Canterbury Region. Prepared for Environment Canterbury. NIWA.

NIWA (2019). Waitaki District Coastal Hazards. Prepared for the Otago Regional Council. January 2019.

Orchard, S., Hughey, K., Measures, R., Schiel, D. (2020). Coastal tectonics and habitat squeeze: response of a tidal lagoon to co-seismic sea-level change. Natural Hazards (2020). Springer Nature B.V. 2020.

Pearson, C., Denys, P., Denham, M. (2019). Christchurch City ground monitoring. Vertical ground motion at the Sumner tide gauge. Otago University School of Surveying. Prepared for Environment Canterbury.

Rogers, J., Raj, A., Dodd, T., Musa, M. (2020). Morphology change at the Avon-Heathcote Estuary mouth: A 2019 update. New Zealand Coastal Society Coastal News article. Issue 71, March 2020

Shand, R., Shand, T.D., McComb, P. and Johnson, D. (2011). Evaluation of empirical predictors of extreme run-up using field data. 20th Australasian Coasts and Ports Conference Sept 2011.

Shand, T., D., Wasko, C., D., Westra, S., P., Smith, G., Carley, J., T., Peirson, W., L. (2012). Joint Probability Assessment of NSW Extreme Waves and Water Levels, Water Research Laboratory, School of Civil and Environmental Engineering UNSW, Manly Vale, WRL Technical Report, WRL TR2011/29

Shand, T. D., Reinen-Hamill, R., Kench, P., Ivamy, M., Knook, P. and Howse, B. (2015). Methods for Probabilistic Coastal Erosion Hazard Assessments. Australasian Coasts & Ports Conference 2015, Auckland, New Zealand.

Shand, T. D., Goodwin, I. D., Mole, M. A., Carley, J. T., Coghlan, I. R., Harley, M. D. and Peirson, W. L. (2010), NSW Coastal Inundation Hazard Study: Coastal Storms and Extreme Waves, WRL Technical Report 2010/16.

Shulmeister, J. & Jennings, R. (2009). Morphology and morphodynamics of gravel beaches. Coastal Zones and Estuaries.

Single, M (2006). Timaru to Banks Peninsula coastal report status of gravel resources and management implications. Environment Canterbury

Soons, J., Shulmeister, J., Holt, S. (1997). The Holocene evolution of a well nourished gravelly barrier and lagoon complex, Kaitorete “Spit”, Canterbury, New Zealand. Marine Geology 138 (1997) 69-90.

Stockdon, H.F., Holman, R.A., Howd, P.A. and Sallenger, Jr., A.H. (2006). Empirical parameterization of set-up, swash, and run-up. Coastal Eng. 53(7), pp 573-588.

Thompson, G. (1994). Morphology of the Avon-Heathcote Estuary Mouth. Master Thesis. University of Canterbury.

T+T (2013). Effects of Sea Level Rise for Christchurch City. Prepared for Christchurch City Council. November 2013.

T+T (2016). LPC Channel Deepening Project. Review and summary of coastal process effects. Prepared for Lyttelton Port Company.

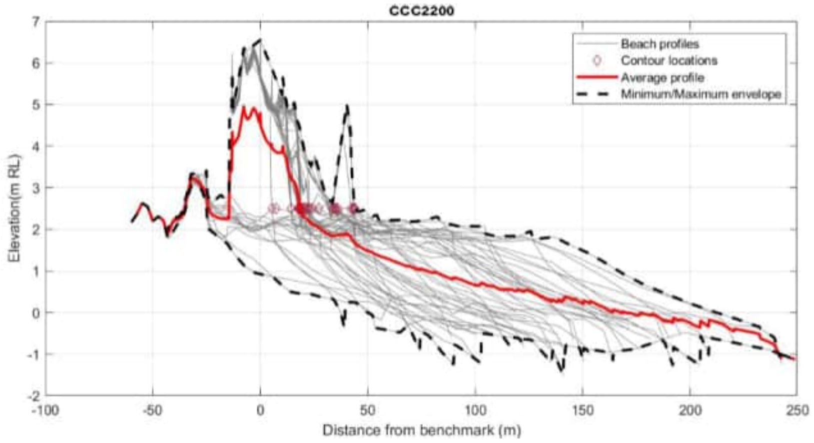
T+T (2017). Coastal Hazard Assessment for Christchurch and Banks Peninsula (2017). Prepared for Christchurch City Council. October 2017.

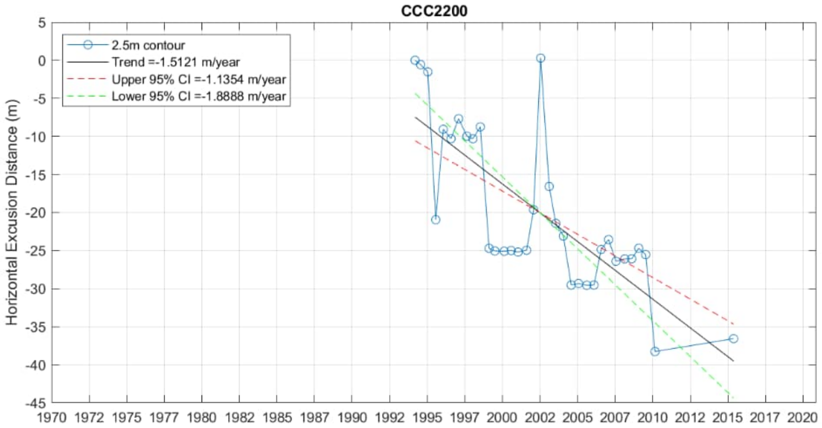
T+T (2021b). Coastal Flood Hazard Assessment for Northland Region 2019-2020. Prepared for Northland Regional Council. March 2021.

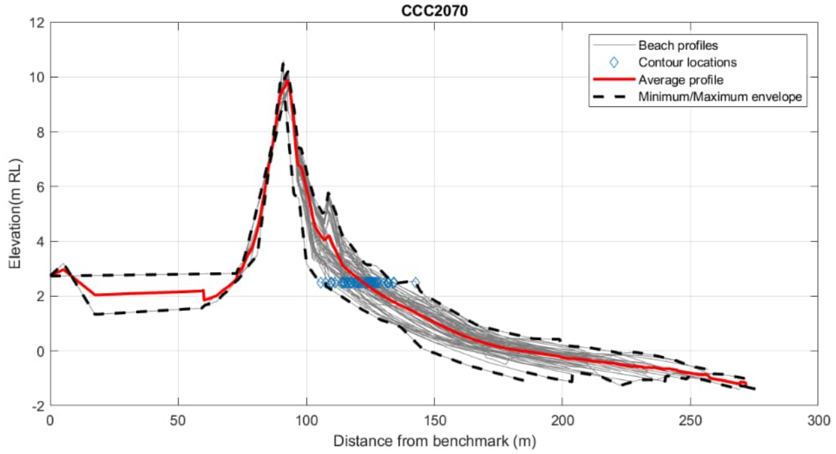
USACE (2006). Coastal Engineering Manual. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C

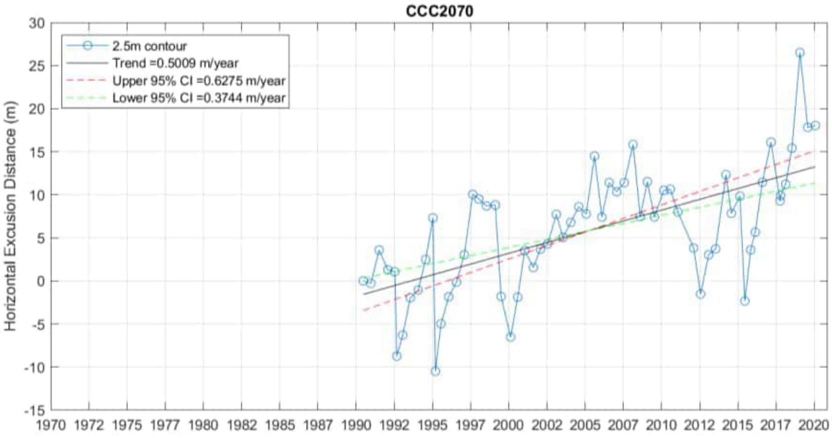
Van Ballegooy, S., Cox, S., Thurlow, C., Rutter, H., Reynolds, T., Harrington, G., Fraser, J., Smith, T. (2014). Median water table elevation in Christchurch and surrounding area after the 4 September 2010 Darfield Earthquake Version 2 (GNS science report 2014/18).

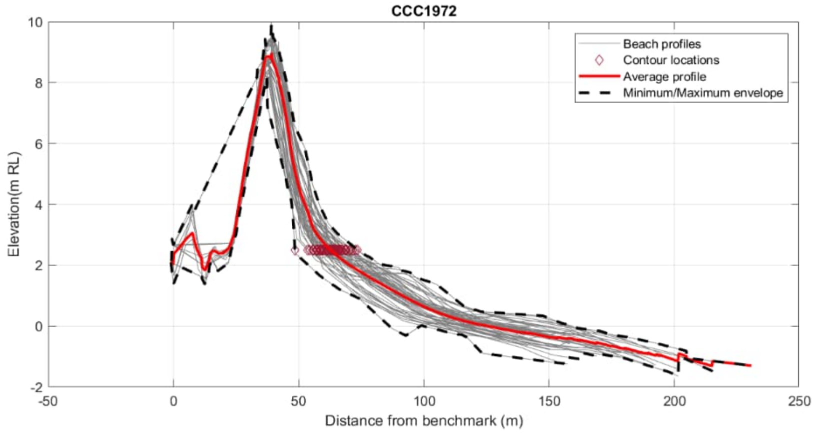
Appendix A: Beach profiles

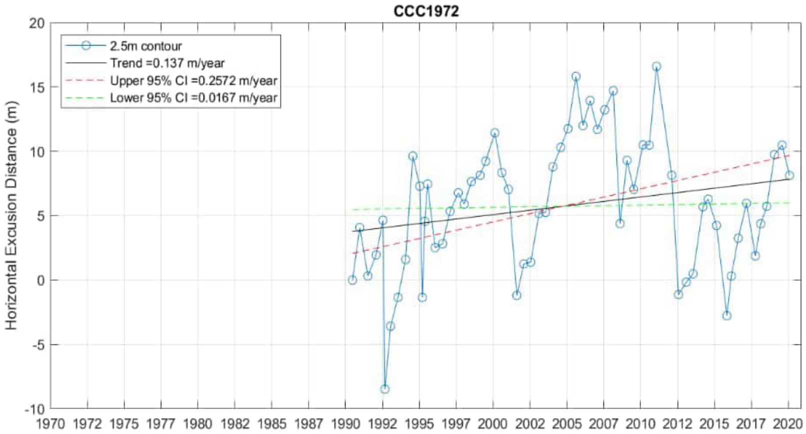

CCC2200
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
7
6
5
4
3
2
1
0
-1
-2
-100
-50
0
50
100
150
200
250
Distance from benchmark (m)

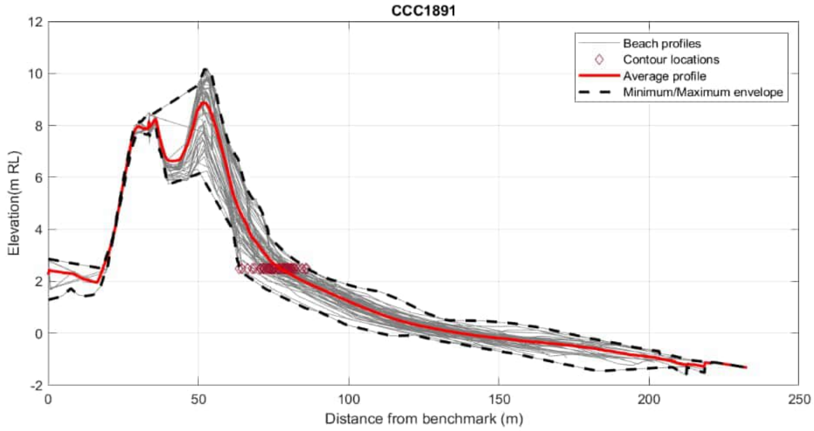

CCC2200
2.5m contour
Trend =-1.5121 m/year
Upper 95% CI =-1.1354 m/year
Lower 95% CI =-1.8888 m/year
Horizontal Excusion Distance (m)
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

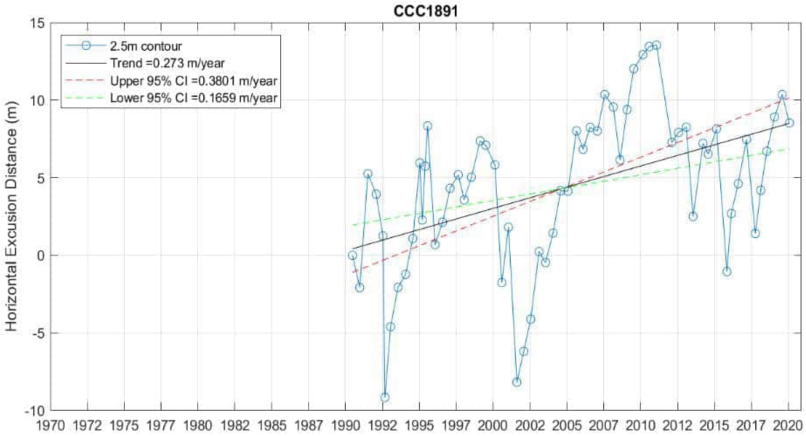

CCC2070
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
12
10
8
6
4
2
0
-2
0
50
100
150
200
250
300
Distance from benchmark (m)

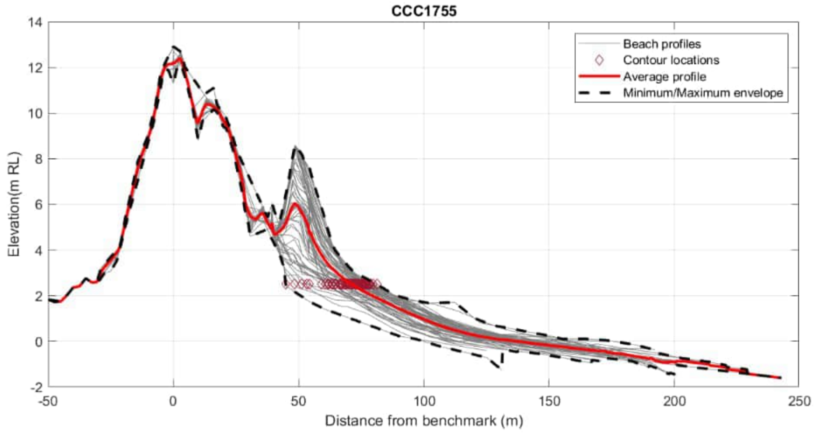

CCC2070
2.5m contour
Trend =0.5009 m/year
Upper 95% CI =0.6275 m/year
Lower 95% CI =0.3744 m/year
Horizontal Excusion Distance (m)
30
25
20
15
10
5
0
-5
-10
-15
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020


CCC1972
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
10
8
6
4
2
0
-2
-50
0
50
100
150
200
250


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)
20
15
10
5
0
-5
-10
1970 1972 1975 1977 1980 1982 1985 1987 1990 1992 1995 1997 2000 2002 2005 2007 2010 2012 2015 2017 2020


CCC1891
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
12
10
8
6
4
2
0
-2
0
50
100
150
200
250


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)
15
10
5
0
-5
-10
1970 1972 1975 1977 1980 1982 1985 1987 1990 1992 1995 1997 2000 2002 2005 2007 2010 2012 2015 2017 2020


CCC1755
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
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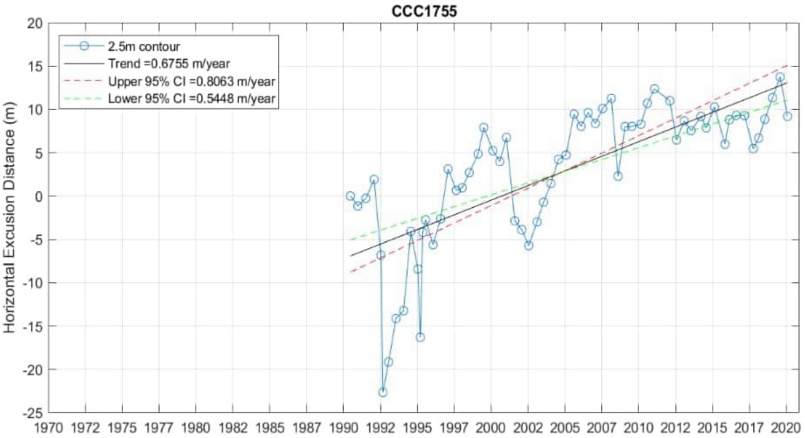
12
10
8
6
4
2
0
-2
-50
0
50
100
150
200
250


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)
20
15
10
5
0
-5
-10
-15
-20
-25
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
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1997
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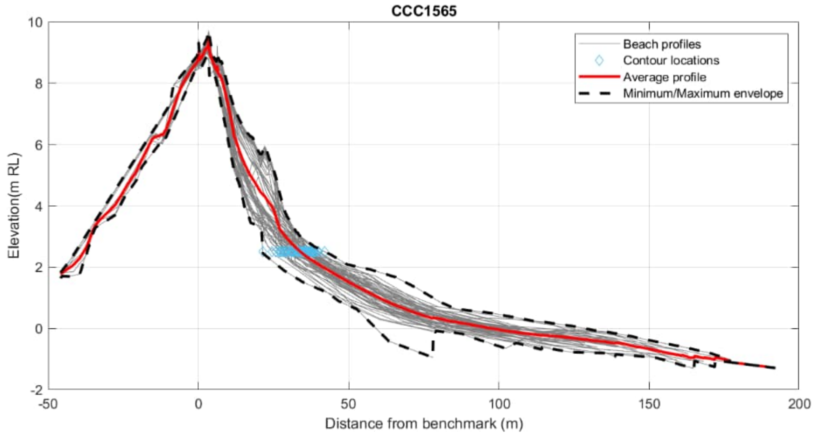
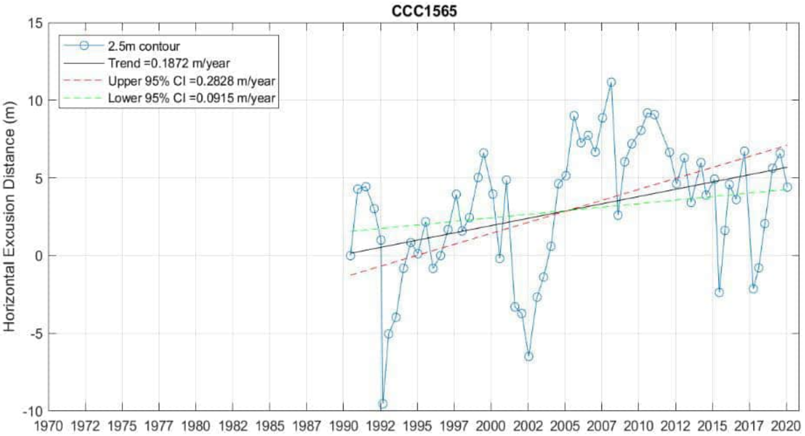
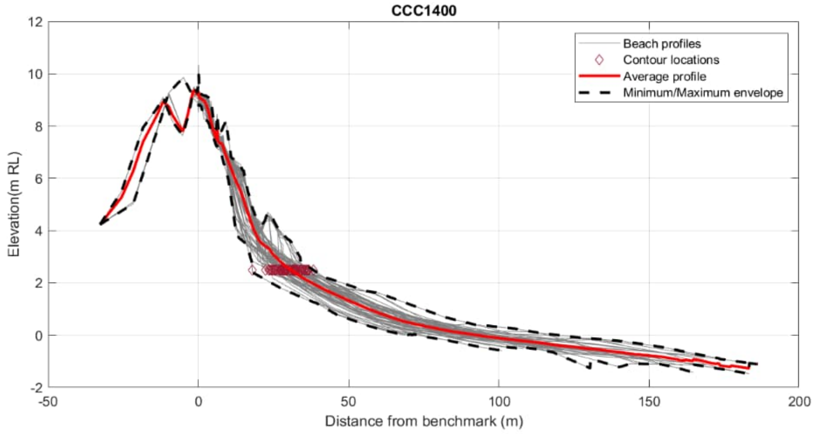
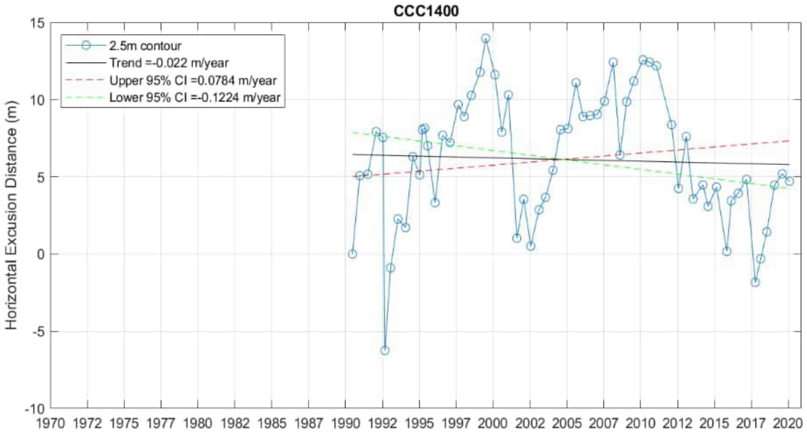
2002
2005
2007
2010
2012
2015
2017
2020


CCC1565
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
10
8
6
4
2
0
-2
-50
0
50
100
150
200


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)
15
10
5
0
-5
-10
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020


CCC1400
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
12
10
8
6
4
2
0
-2
-50
0
50
100
150
200
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)
15
10
5
0
-5
-10
1970
1972
1975
1977
1980
1982
1985
1987
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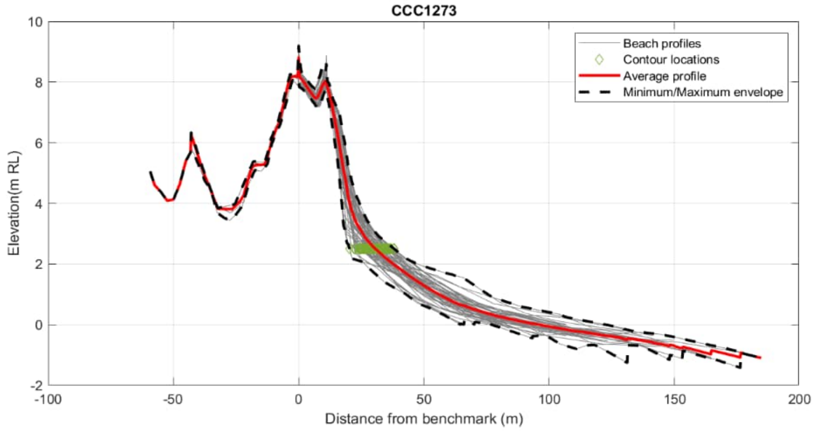
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020


CCC1273
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
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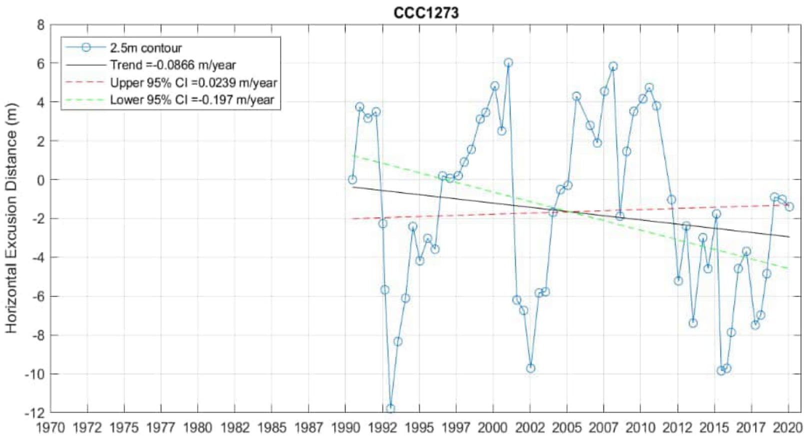
8
6
4
2
0
-2
-100
-50
0
50
100
150
200
Distance from benchmark (m)


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)
8
6
4
2
0
-2
-4
-6
-8
-10
-12
1970
1972
1975
1977
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1982
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1987
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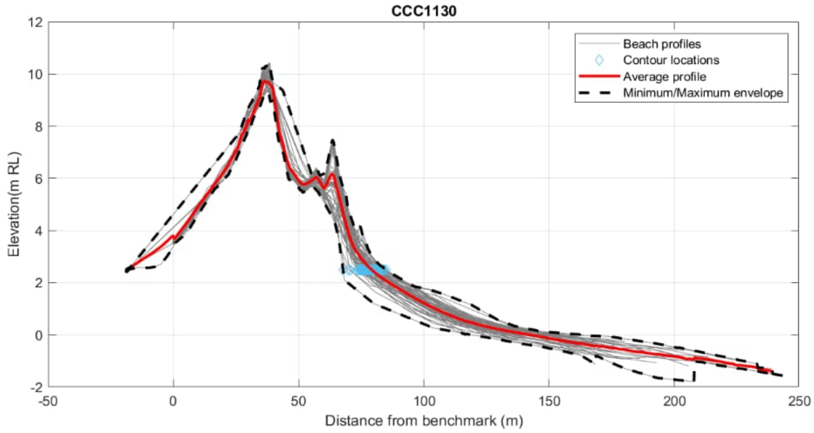
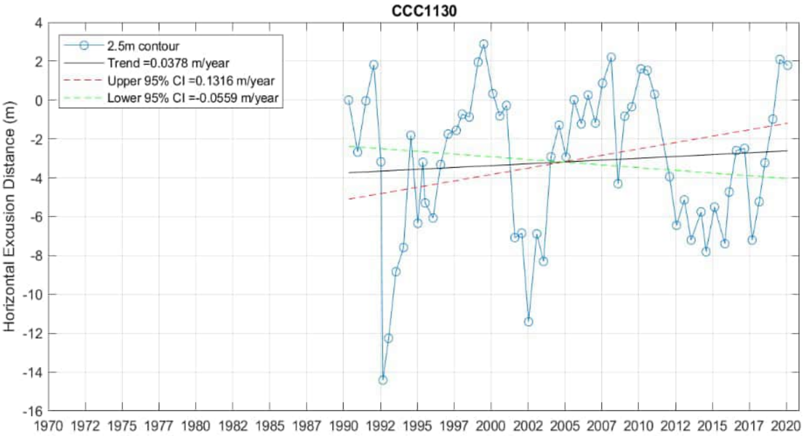
2002
2005
2007
2010
2012
2015
2017
2020


CCC1130
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
12
10
8
6
4
2
0
-2
-50
0
50
100
150
200
250


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)
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
1970
1972
1975
1977
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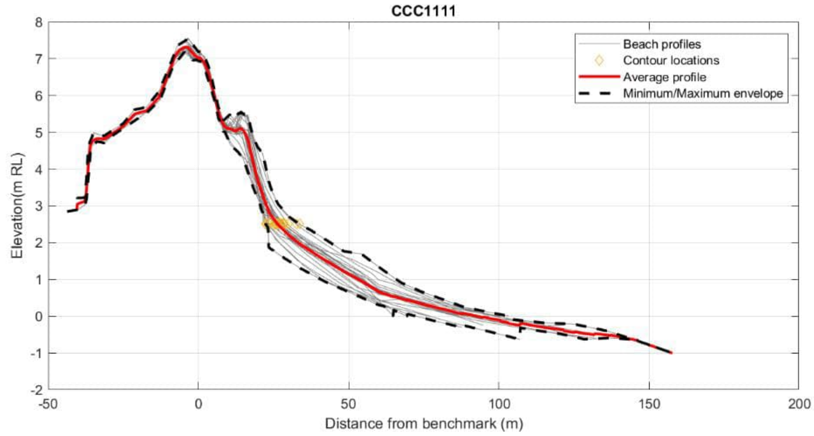
2012
2015
2017
2020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Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
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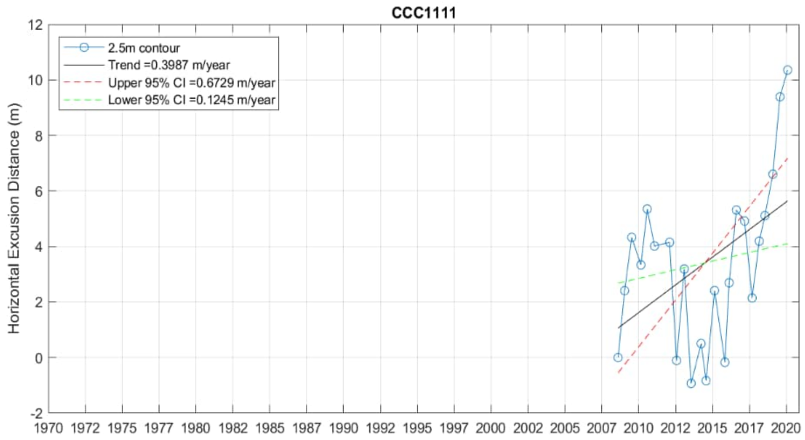
7
6
5
4
3
2
1
0
-1
-2
-50
0
50
100
150
200


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)
12
10
8
6
4
2
0
-2
1970
1972
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2010
2012
2015
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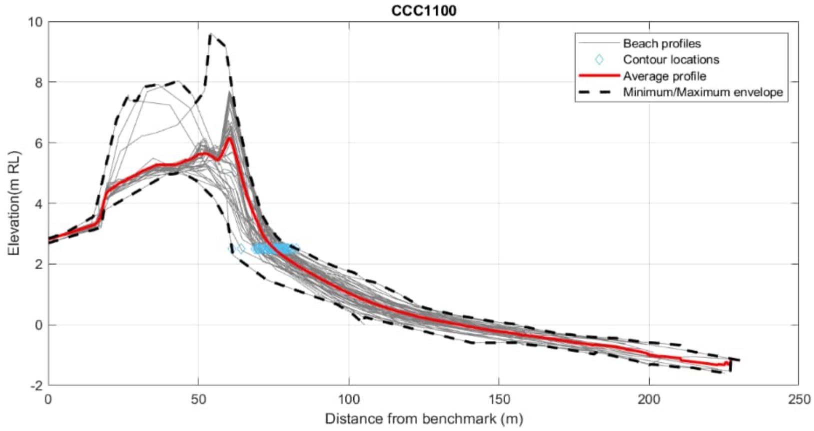
2020


CCC1100
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
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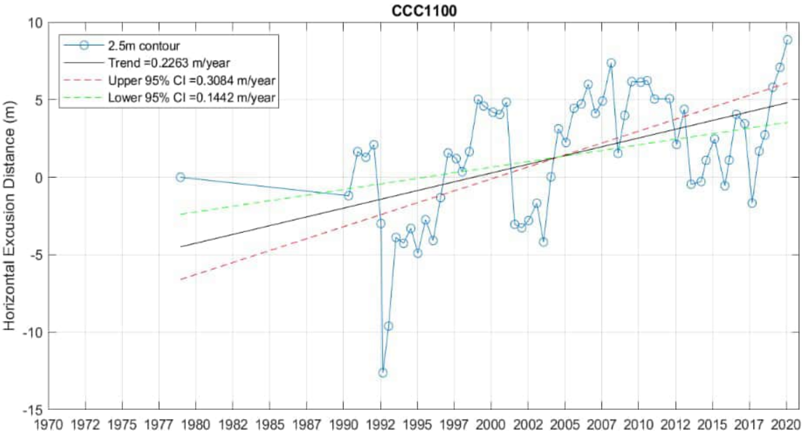
8
6
4
2
0
-2
0
50
100
150
200
250
Distance from benchmark (m)


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)
10
5
0
-5
-10
-15
1970
1972
1975
1977
1980
1982
1985
1987
1990
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1995
1997
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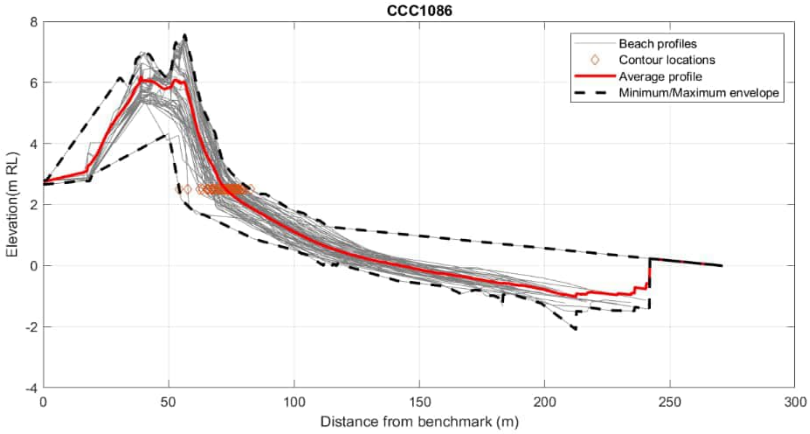
CCC1086
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
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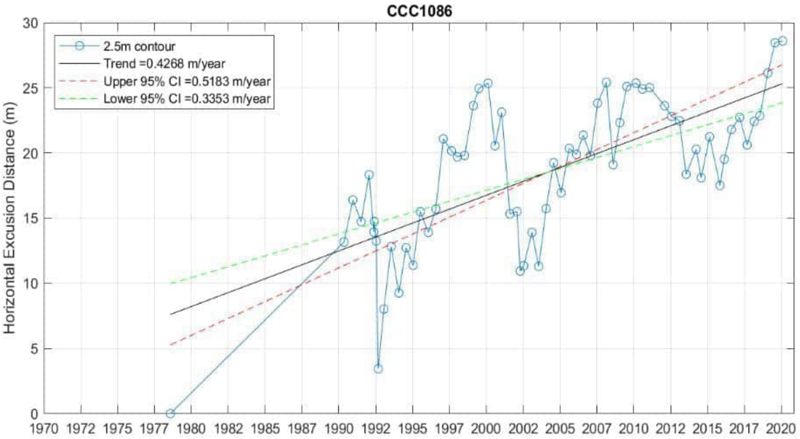
6
4
2
0
-2
-4
0
50
100
150
200
250
300
Distance from benchmark (m)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2.5m contour
Trend =0.4268 m/year
Upper 95% CI =0.5183 m/year
Lower 95% CI =0.3353 m/year
Horizontal Excusion Distance (m)
30
25
20
15
10
5
0
1970
1972
1975
1977
1980
1982
1985
1987
1990
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1995
1997
2000
2002
2005
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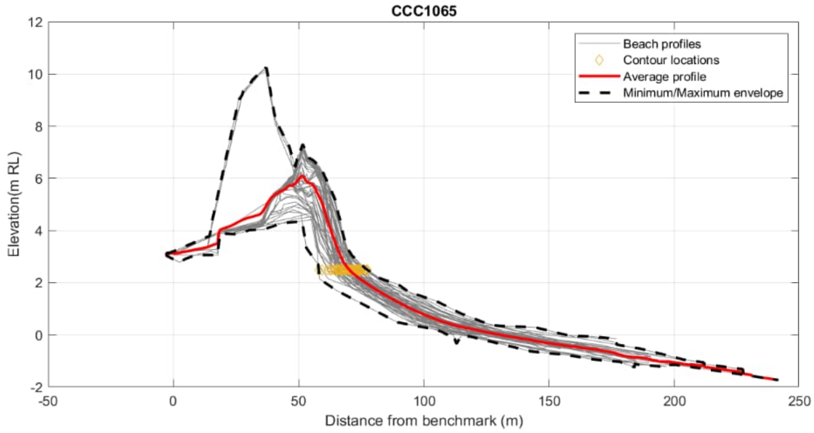
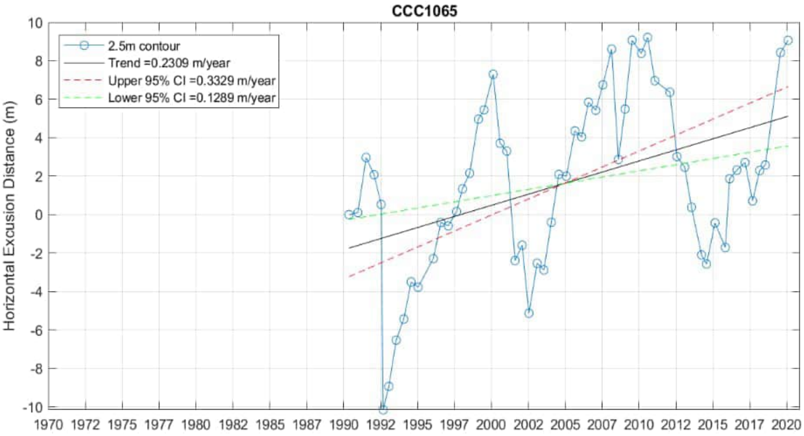

CCC1065
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
12
10
8
6
4
2
0
-2
-50
0
50
100
150
200
250


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)
10
8
6
4
2
0
-2
-4
-6
-8
-10
1970
1972
1975
1977
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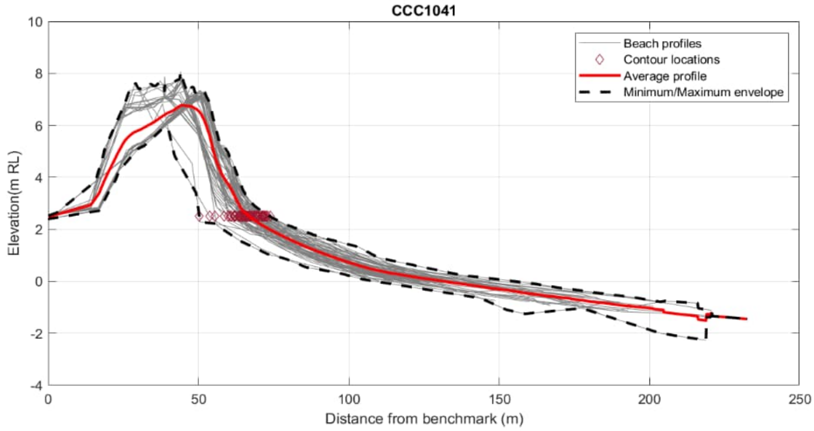
2017
2020


CCC1041
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
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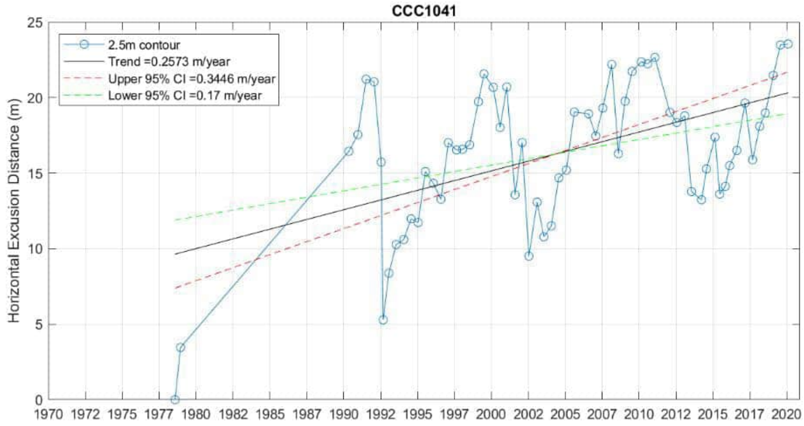
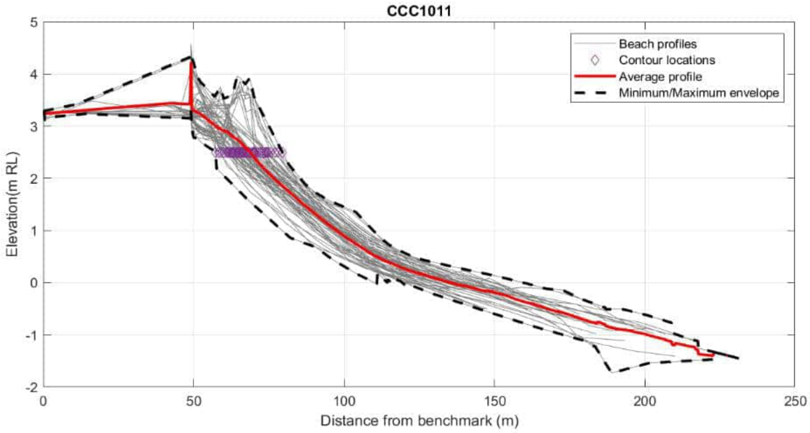
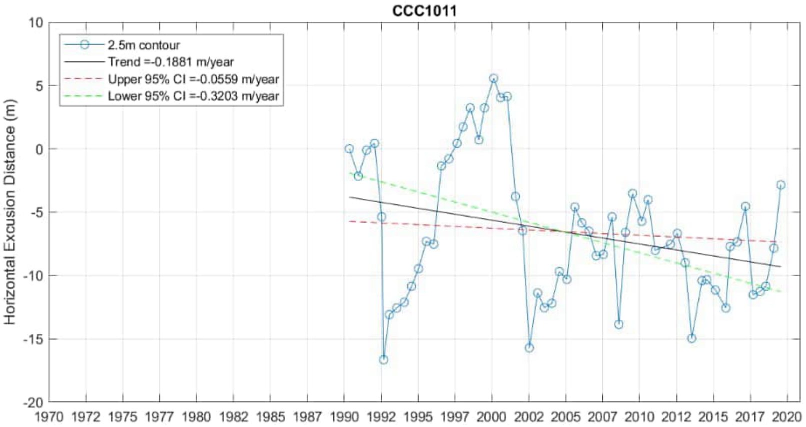
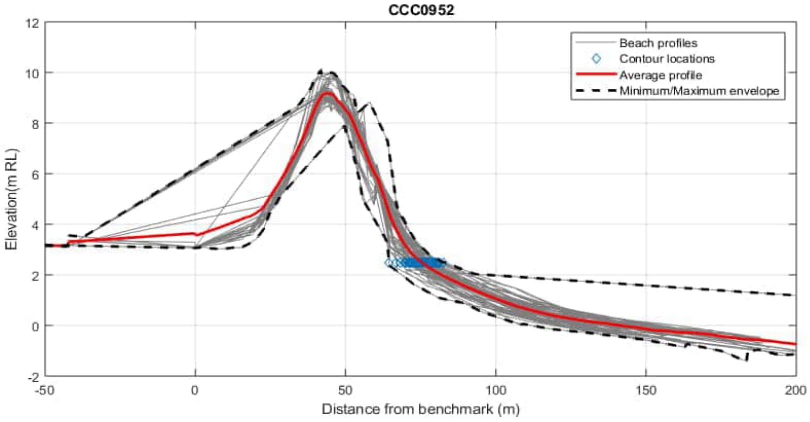
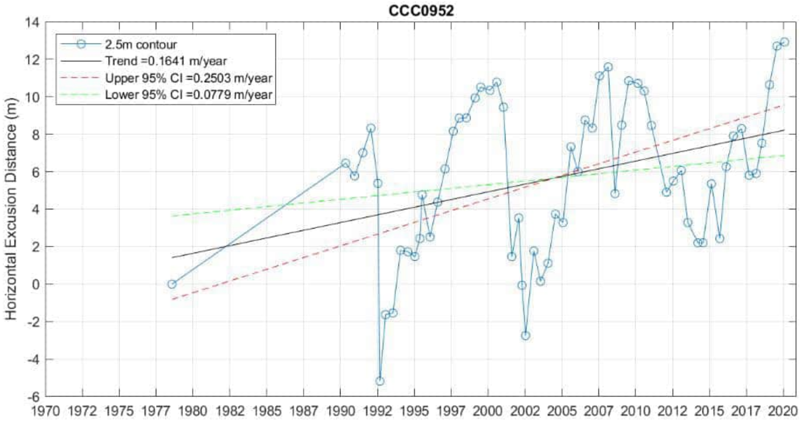
8
6
4
2
0
-2
-4
0
50
100
150
200
250

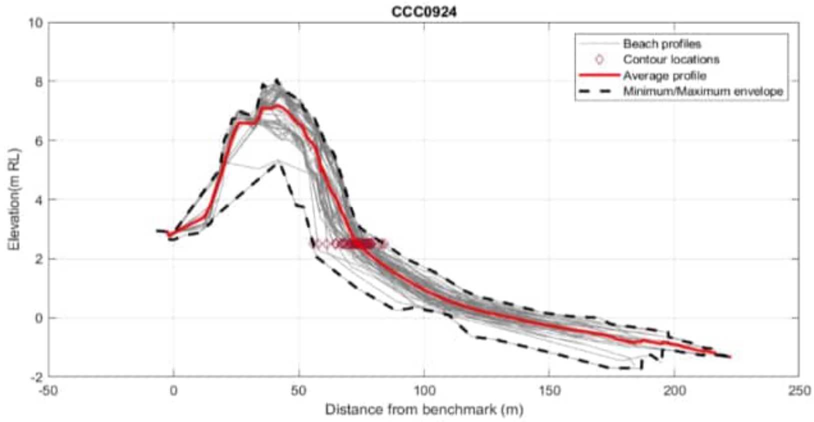

CCC1041
2.5m contour
Trend =0.2573 m/year
Upper 95% CI =0.3446 m/year
Lower 95% CI =0.17 m/year
Horizontal Excusion Distance (m)
25
20
15
10
5
0
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

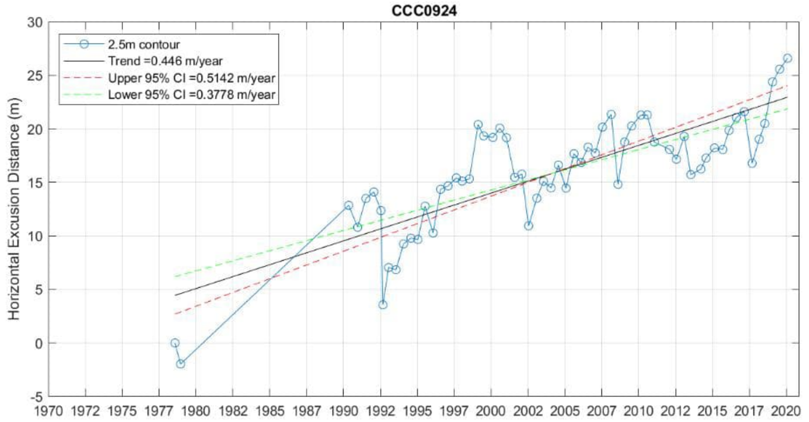

CCC1011
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
5
4
3
2
1
0
-1
-2
0
50
100
150
200
250

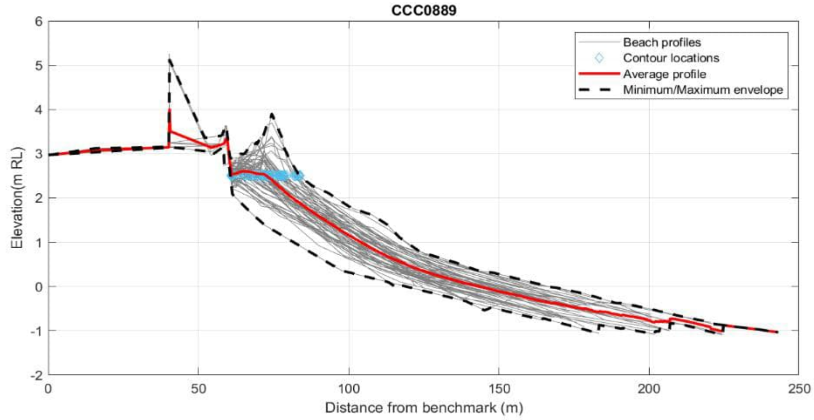

CCC1011
2.5m contour
Trend =-0.1881 m/year
Upper 95% CI =-0.0559 m/year
Lower 95% CI =-0.3203 m/year
Horizontal Excusion Distance (m)
10
5
0
-5
-10
-15
-20
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

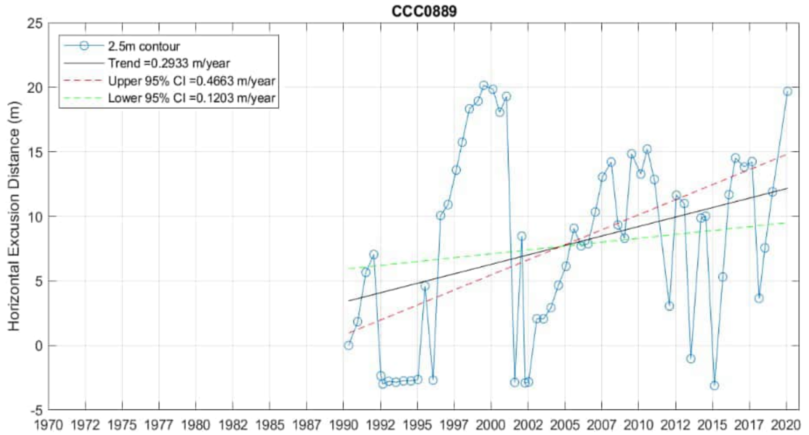

CCC0952
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
12
10
8
6
4
2
0
-2
-50
0
50
100
150
200

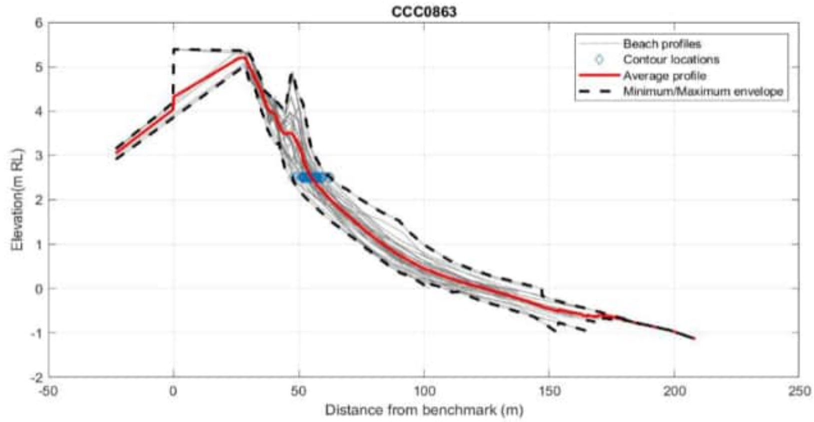

CCC0952
2.5m contour
Trend =0.1641 m/year
Upper 95% CI =0.2503 m/year
Lower 95% CI =0.0779 m/year
Horizontal Excusion Distance (m)
14
12
10
8
6
4
2
0
-2
-4
-6
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

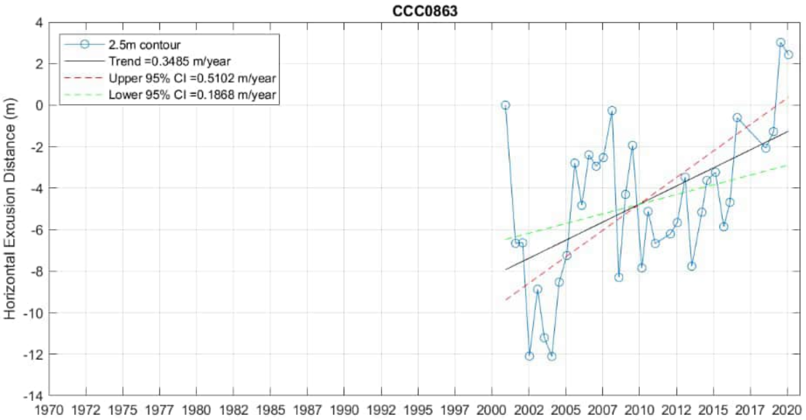

CCC0924
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
10
8
6
4
2
0
-2
-50
0
50
100
150
200
250

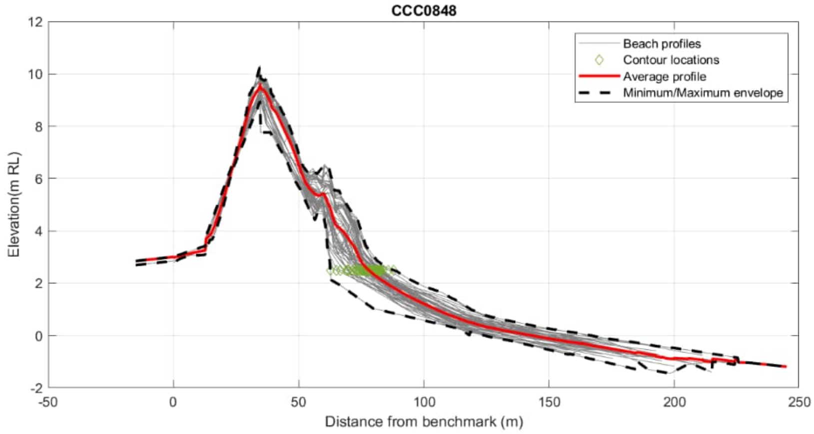

CCC0924
2.5m contour
Trend =0.446 m/year
Upper 95% CI =0.5142 m/year
Lower 95% CI =0.3778 m/year
Horizontal Excusion Distance (m)
30
25
20
15
10
5
0
-5
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

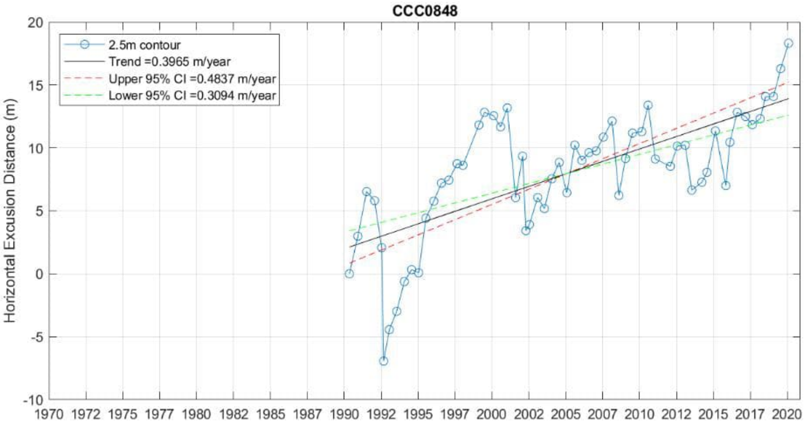

CCC0889
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
6
5
4
3
2
1
0
-1
-2
0
50
100
150
200
250

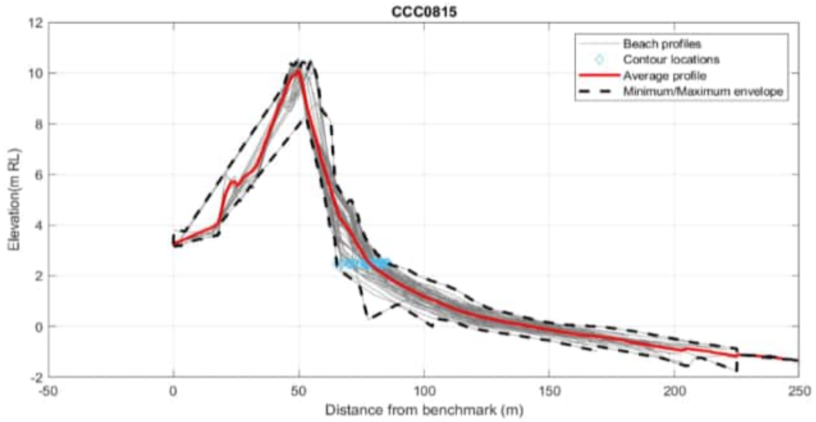

CCC0889
2.5m contour
Trend =0.2933 m/year
Upper 95% CI =0.4663 m/year
Lower 95% CI =0.1203 m/year
Horizontal Excusion Distance (m)
25
20
15
10
5
0
-5
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

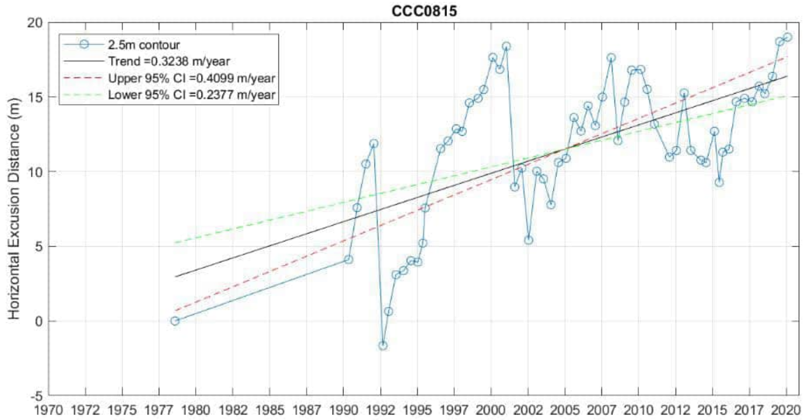

CCC0863
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
6
5
4
3
2
1
0
-1
-2
-50
0
50
100
150
200
250

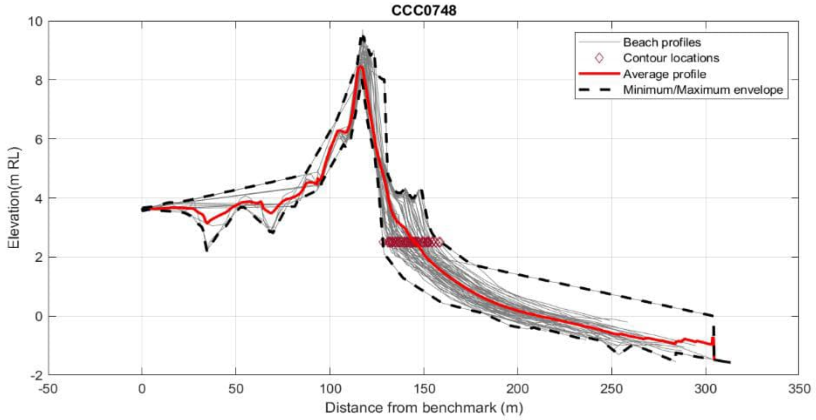

CCC0863
2.5m contour
Trend =0.3485 m/year
Upper 95% CI =0.5102 m/year
Lower 95% CI =0.1868 m/year
Horizontal Excusion Distance (m)
4
2
0
-2
-4
-6
-8
-10
-12
-14
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

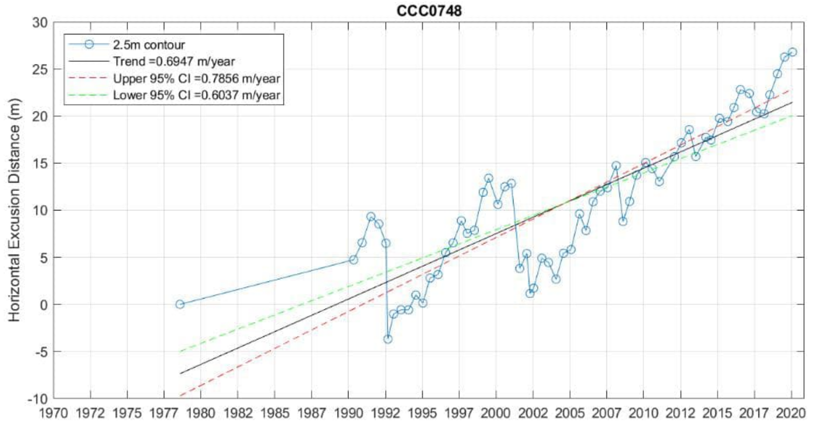

CCC0848
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
12
10
8
6
4
2
0
-2
-50
0
50
100
150
200
250

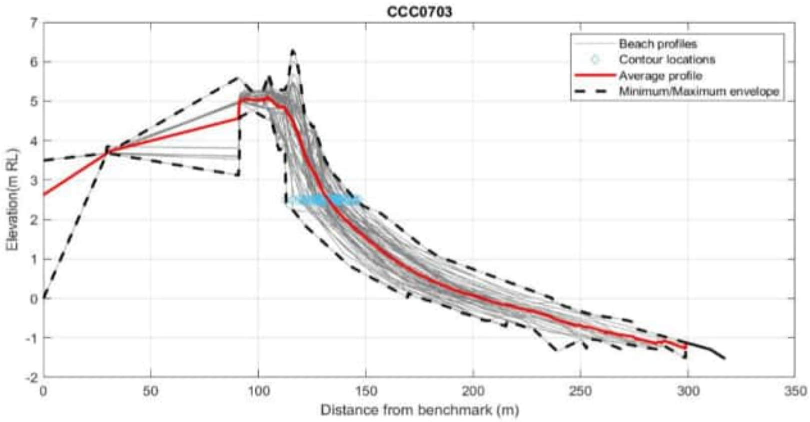

CCC0848
2.5m contour
Trend =0.3965 m/year
Upper 95% CI =0.4837 m/year
Lower 95% CI =0.3094 m/year
Horizontal Excusion Distance (m)
20
15
10
5
0
-5
-10
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

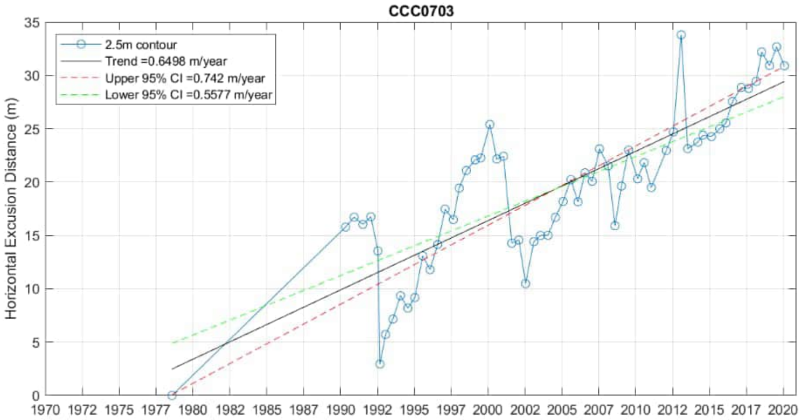

CCC0815
Elevation(m RL)
Distance from benchmark (m)
-50
0
50
100
150
200
250
12
10
8
6
4
2
0
-2
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope

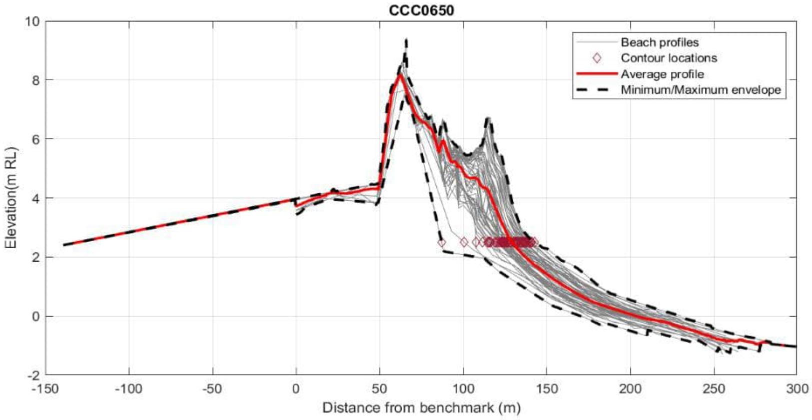

CCC0815
Horizontal Excusion Distance (m)
2.5m contour
Trend =0.3238 m/year
Upper 95% CI =0.4099 m/year
Lower 95% CI =0.2377 m/year
20
15
10
5
0
-5
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

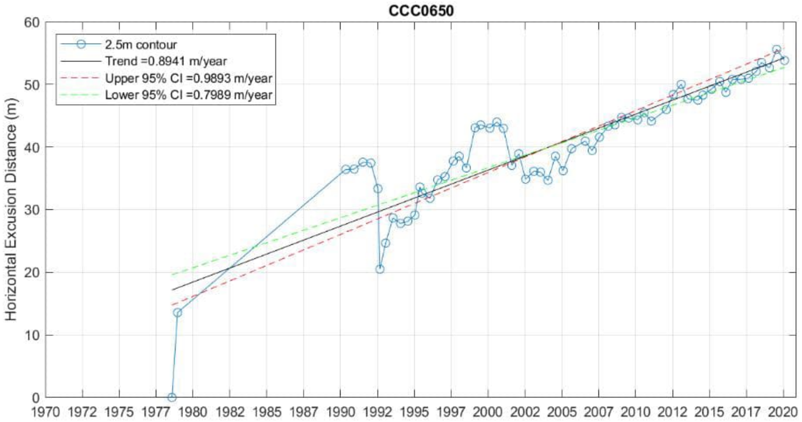

CCC0748
Elevation(m RL)
Distance from benchmark (m)
-50
0
50
100
150
200
250
300
350
10
8
6
4
2
0
-2
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope

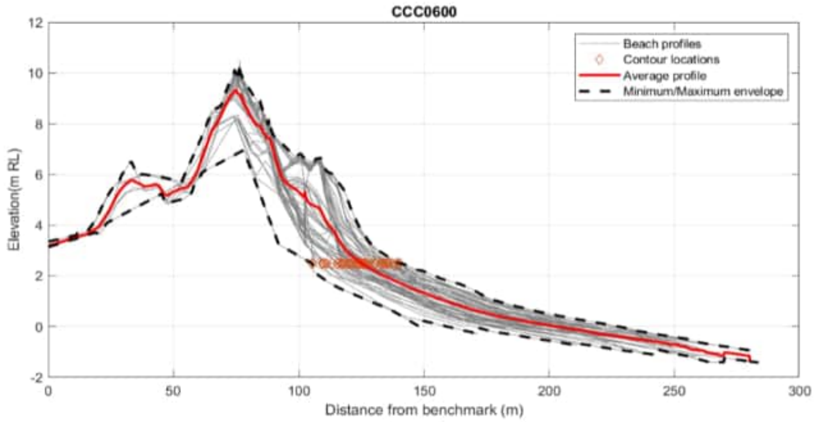

CCC0748
Horizontal Excusion Distance (m)
2.5m contour
Trend =0.6947 m/year
Upper 95% CI =0.7856 m/year
Lower 95% CI =0.6037 m/year
30
25
20
15
10
5
0
-5
-10
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

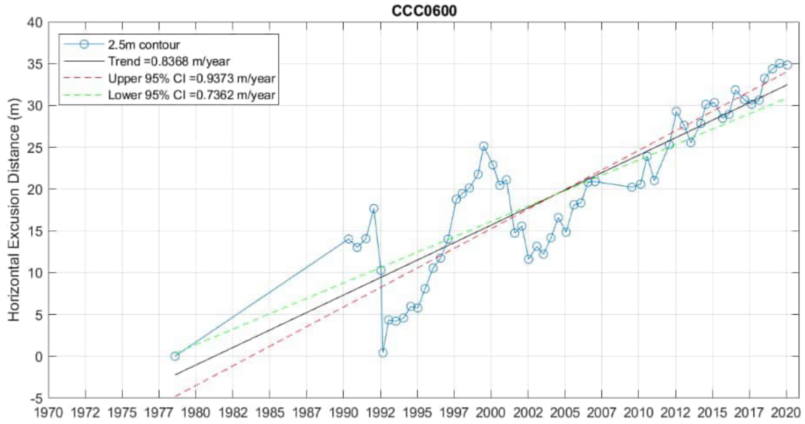

CCC0703
Elevation(m RL)
Distance from benchmark (m)
7
6
5
4
3
2
1
0
-1
-2
0
50
100
150
200
250
300
350
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope

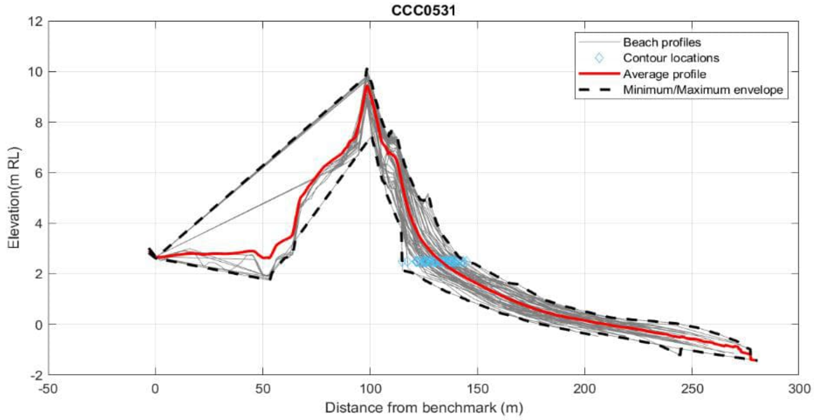

CCC0703
Horizontal Excusion Distance (m)
2.5m contour
Trend =0.6498 m/year
Upper 95% CI =0.742 m/year
Lower 95% CI =0.5577 m/year
35
30
25
20
15
10
5
0
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

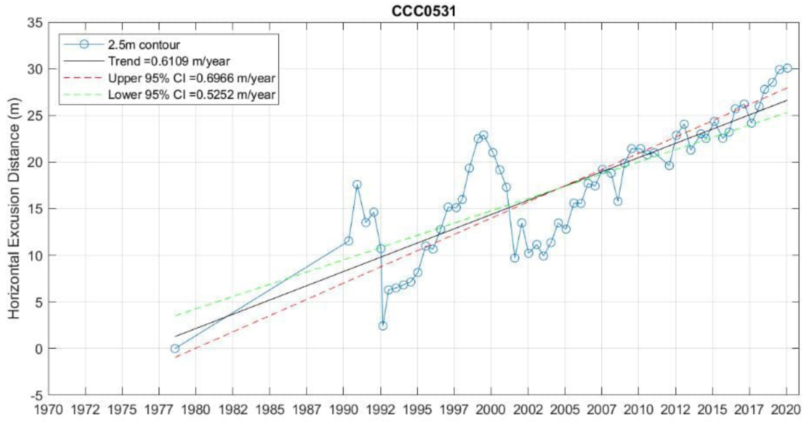

CCC0650
Elevation(m RL)
Distance from benchmark (m)
10
8
6
4
2
0
-2
-150
-100
-50
0
50
100
150
200
250
300
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope

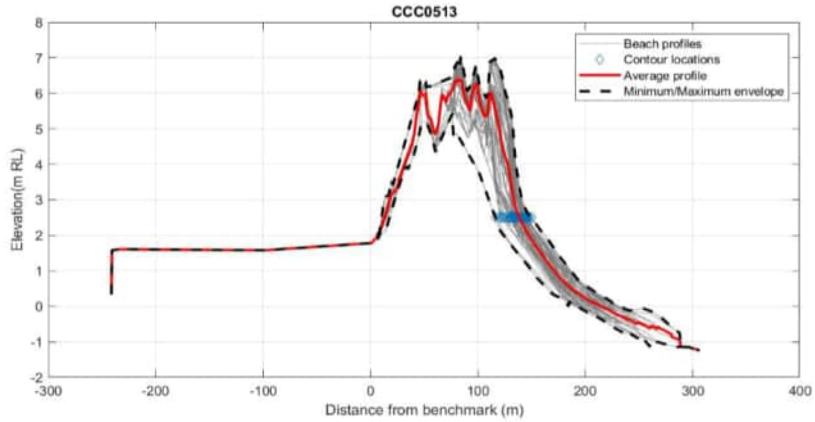

CCC0650
Horizontal Excusion Distance (m)
2.5m contour
Trend =0.8941 m/year
Upper 95% CI =0.9893 m/year
Lower 95% CI =0.7989 m/year
60
50
40
30
20
10
0
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

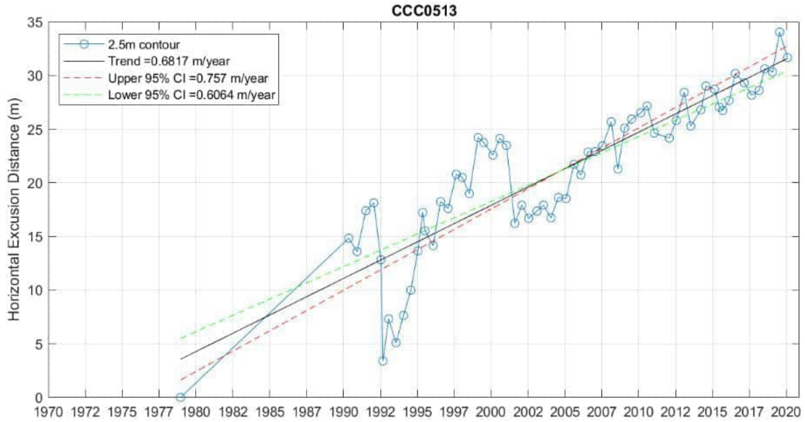

CCC0600
Elevation(m RL)
Distance from benchmark (m)
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
12
10
8
6
4
2
0
-2
0
50
100
150
200
250
300

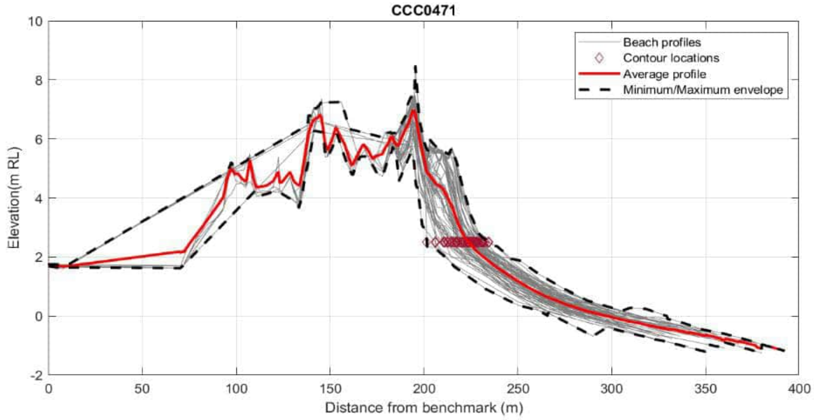

CCC0600
Horizontal Excusion Distance (m)
2.5m contour
Trend =0.8368 m/year
Upper 95% CI =0.9373 m/year
Lower 95% CI =0.7362 m/year
40
35
30
25
20
15
10
5
0
-5
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

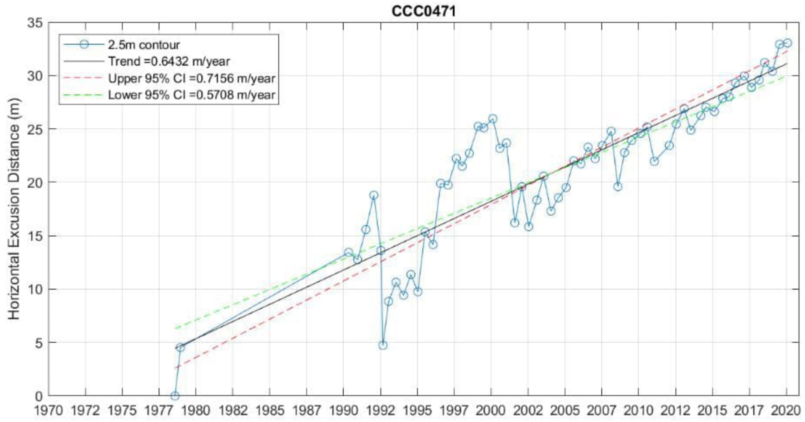

CCC0531
Elevation(m RL)
Distance from benchmark (m)
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
12
10
8
6
4
2
0
-2
-50
0
50
100
150
200
250
300

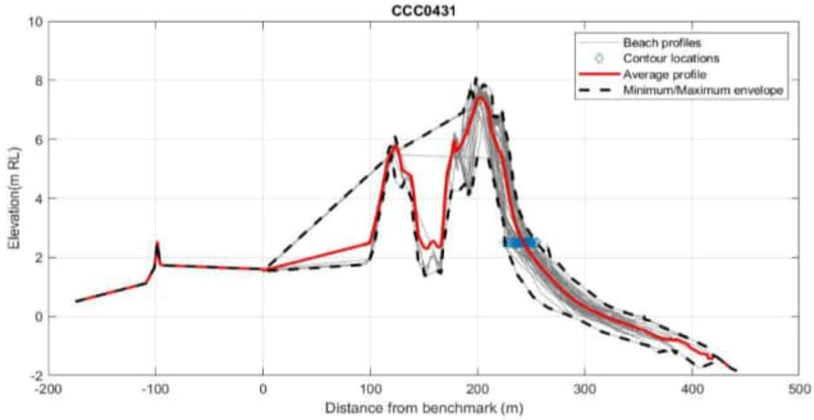

CCC0531
Horizontal Excusion Distance (m)
2.5m contour
Trend =0.6109 m/year
Upper 95% CI =0.6966 m/year
Lower 95% CI =0.5252 m/year
35
30
25
20
15
10
5
0
-5
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

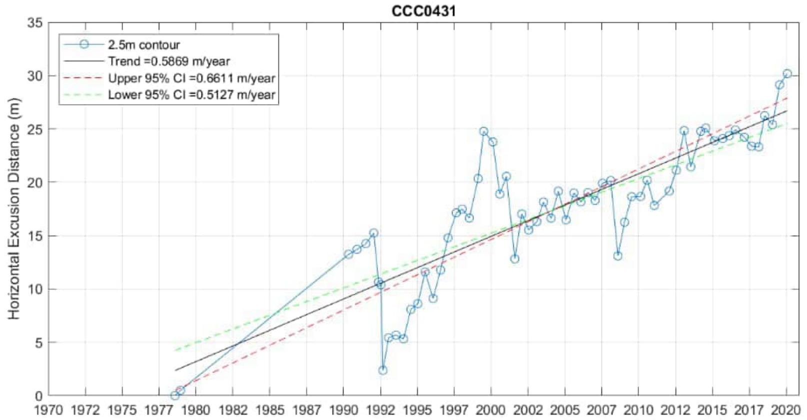

CCC0513
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
-2
-1
0
1
2
3
4
5
6
7
8
-300
-200
-100
0
100
200
300
400

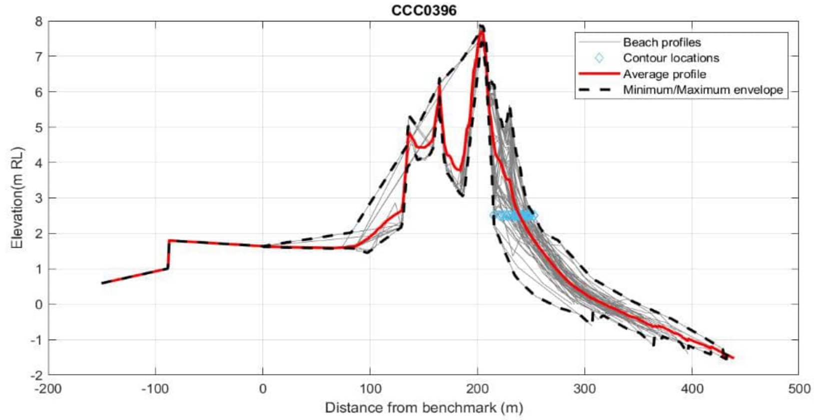

CCC0513
2.5m contour
Trend =0.6817 m/year
Upper 95% CI =0.757 m/year
Lower 95% CI =0.6064 m/year
Horizontal Excusion Distance (m)
0
5
10
15
20
25
30
35
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

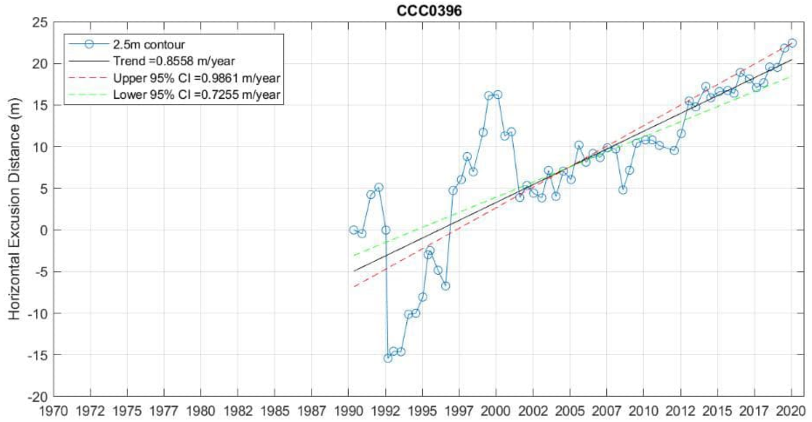

CCC0471
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
-2
0
2
4
6
8
10
0
50
100
150
200
250
300
350
400

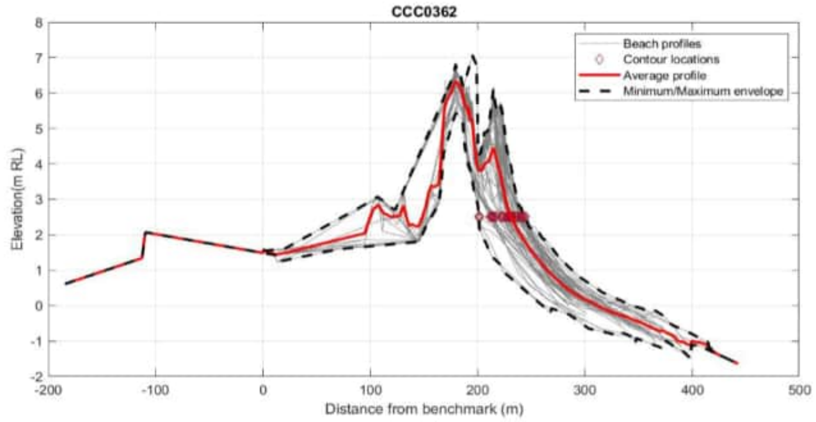

CCC0471
2.5m contour
Trend =0.6432 m/year
Upper 95% CI =0.7156 m/year
Lower 95% CI =0.5708 m/year
Horizontal Excusion Distance (m)
0
5
10
15
20
25
30
35
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

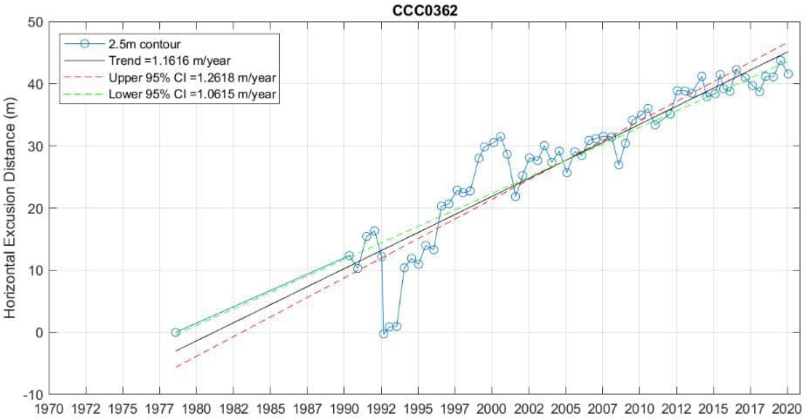

CCC0431
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
10
8
6
4
2
0
-2
-200
-100
0
100
200
300
400
500

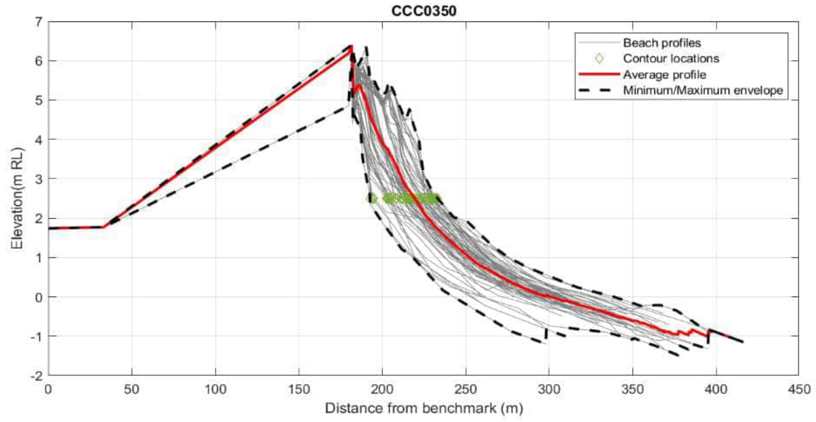

CCC0431
2.5m contour
Trend =0.5869 m/year
Upper 95% CI =0.6611 m/year
Lower 95% CI =0.5127 m/year
Horizontal Excusion Distance (m)
35
30
25
20
15
10
5
0
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

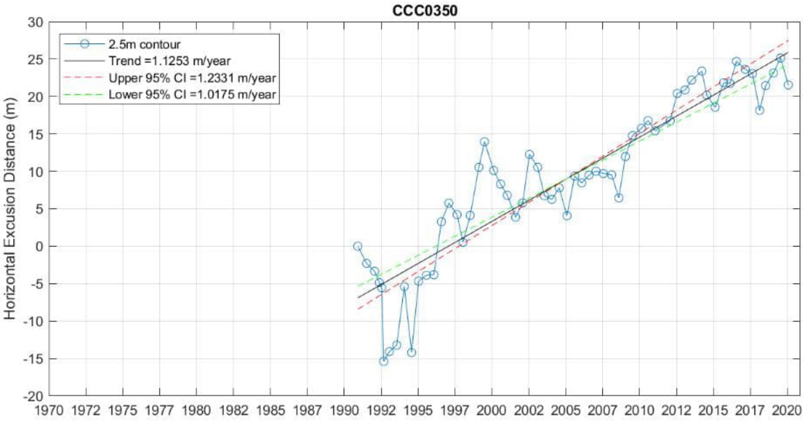

CCC0396
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
8
7
6
5
4
3
2
1
0
-1
-2
-200
-100
0
100
200
300
400
500

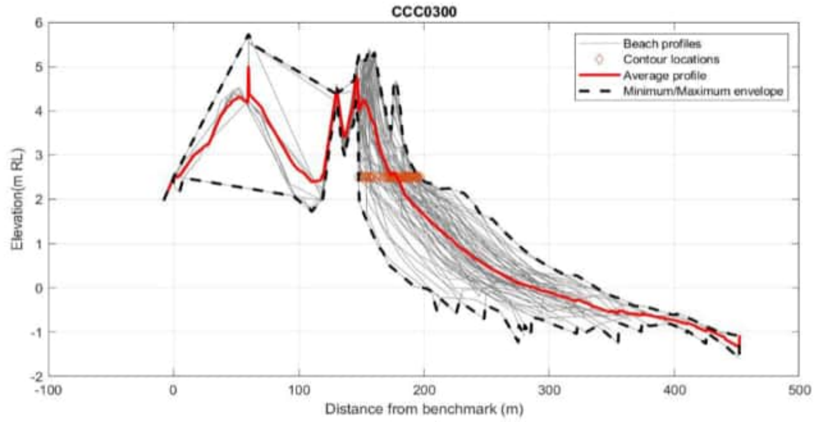

CCC0396
2.5m contour
Trend =0.8558 m/year
Upper 95% CI =0.9861 m/year
Lower 95% CI =0.7255 m/year
Horizontal Excusion Distance (m)
25
20
15
10
5
0
-5
-10
-15
-20
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

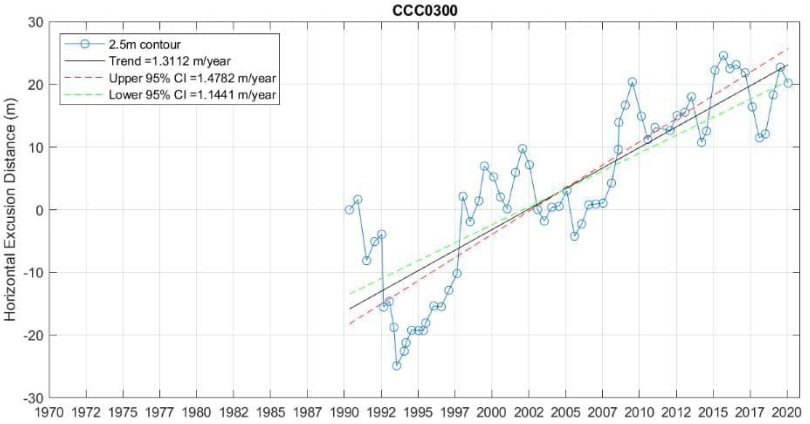

CCC0362
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
-2
-1
0
1
2
3
4
5
6
7
8
-200
-100
0
100
200
300
400
500

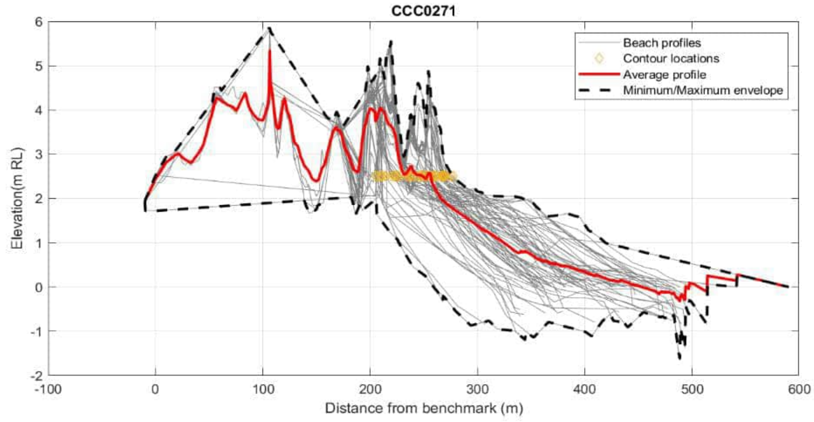

CCC0362
2.5m contour
Trend =1.1616 m/year
Upper 95% CI =1.2618 m/year
Lower 95% CI =1.0615 m/year
Horizontal Excusion Distance (m)
-10
0
10
20
30
40
50
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

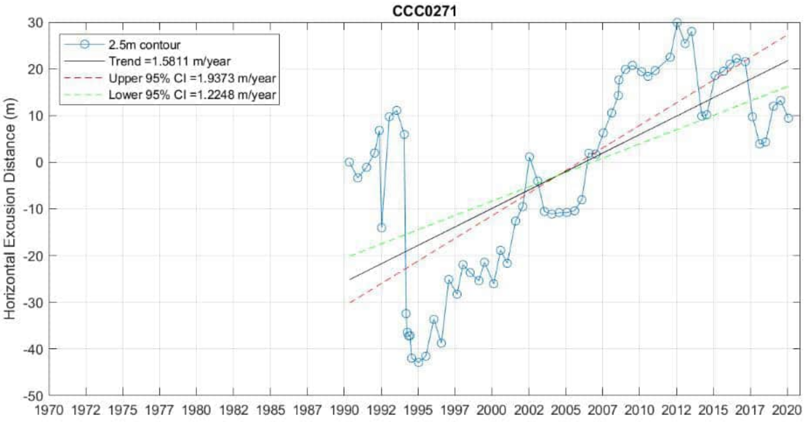

CCC0350
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
-2
-1
0
1
2
3
4
5
6
7
0
50
100
150
200
250
300
350
400
450


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)
-20
-15
-10
-5
0
5
10
15
20
25
30
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020


CCC0300
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
6
5
4
3
2
1
0
-1
-2
-100
0
100
200
300
400
500


CCC0300
2.5m contour
Trend =1.3112 m/year
Upper 95% CI =1.4782 m/year
Lower 95% CI =1.1441 m/year
Horizontal Excursion Distance (m)
30
20
10
0
-10
-20
-30
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020


CCC0271
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)
6
5
4
3
2
1
0
-1
-2
-100
0
100
200
300
400
500
600


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)
30
20
10
0
-10
-20
-30
-40
-50
1970
1972
1975
1977
1980
1982
1985
1987
1990
1992
1995
1997
2000
2002
2005
2007
2010
2012
2015
2017
2020

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.

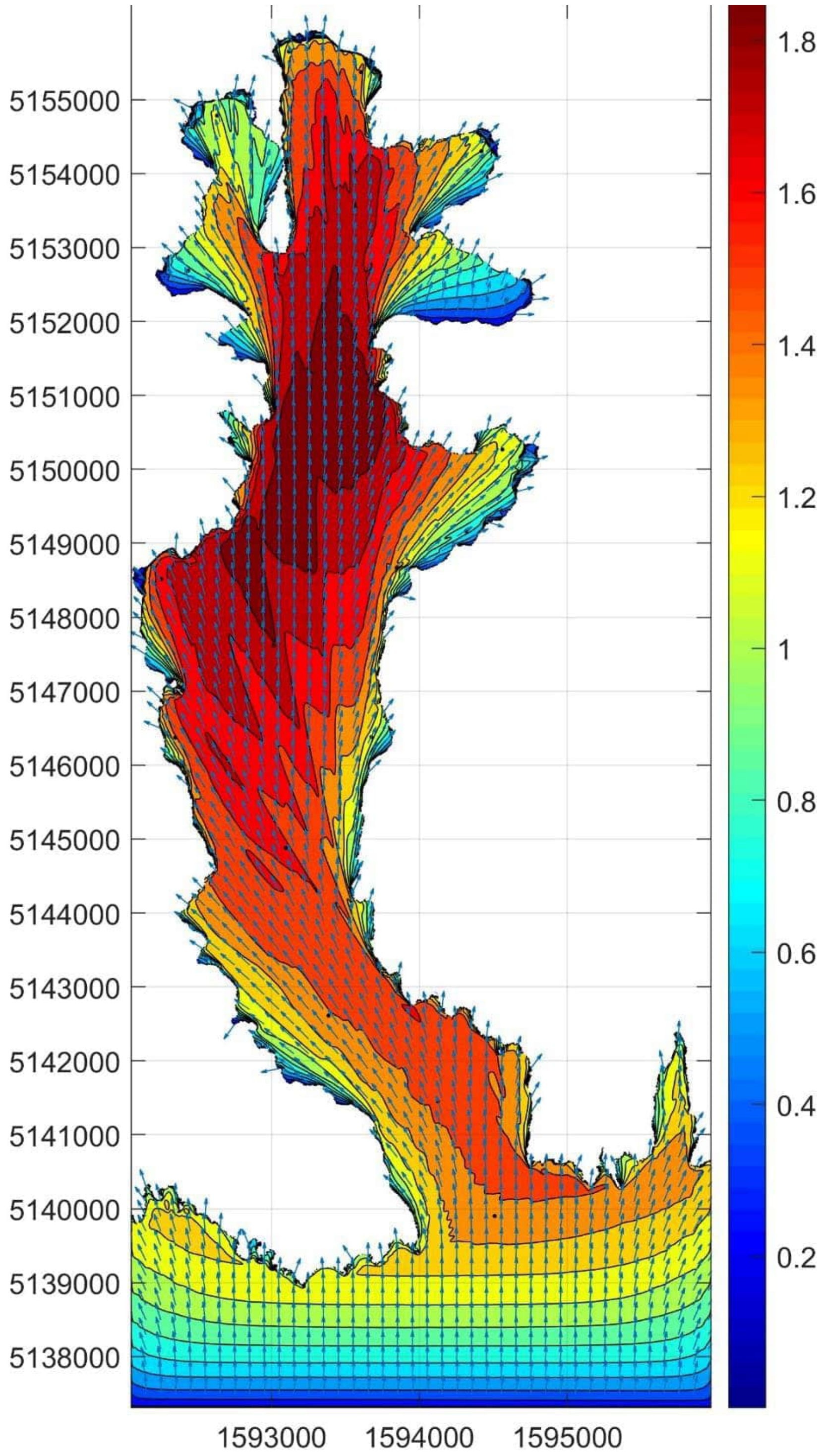


Figure Appendix B.1: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South - Wind generated waves.

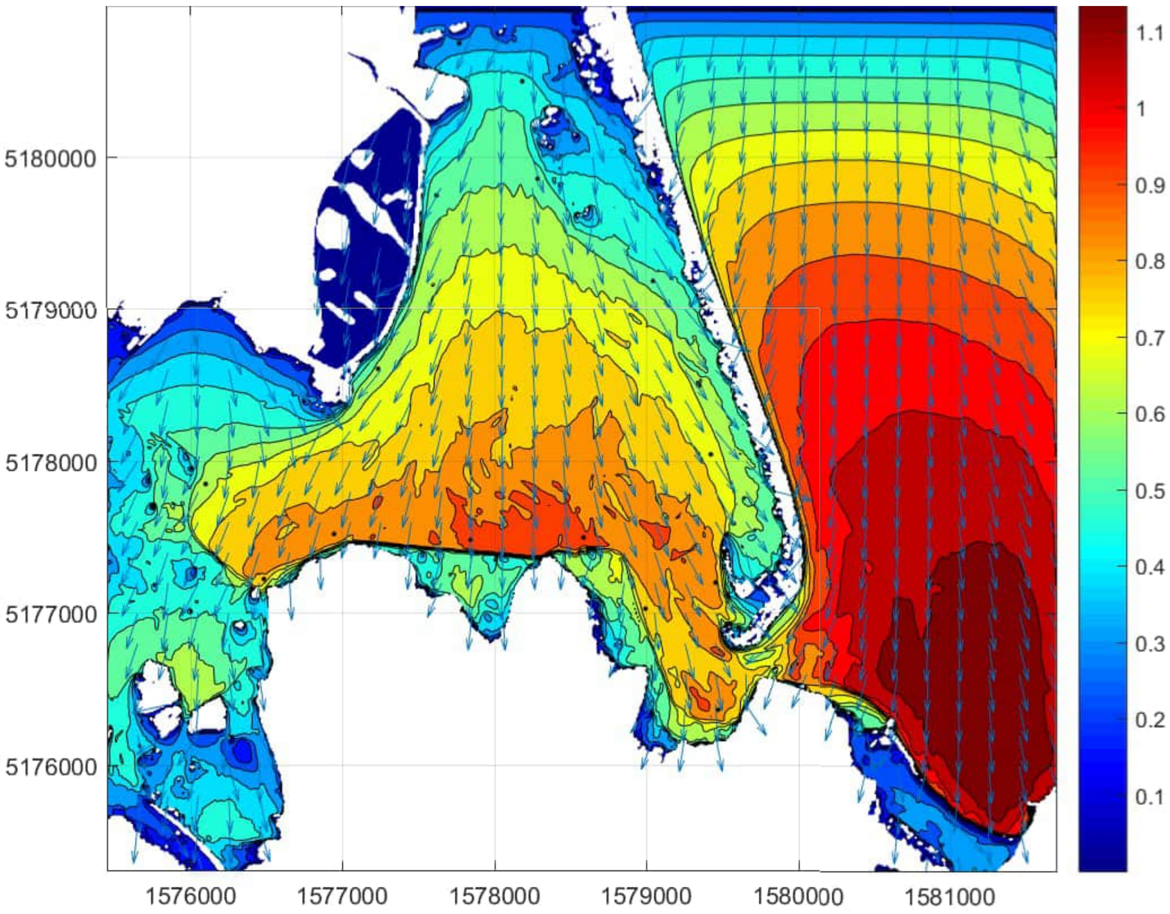


Figure Appendix B.2: SWAN model results for the Avon Heathcote domain – Significant wave height and direction during a 100-year ARI storm from the North – Wind generated waves.

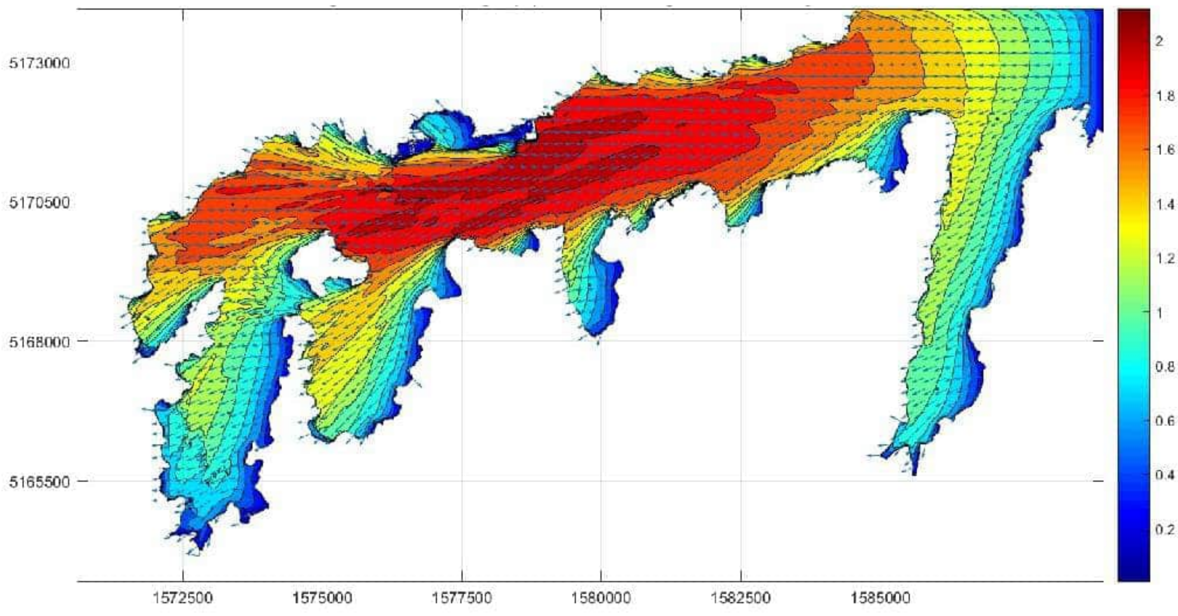


Figure Appendix B.3: SWAN model results for the Lyttleton domain – Significant wave height and direction during a 100-year ARI storm from the East – Wind generated waves.

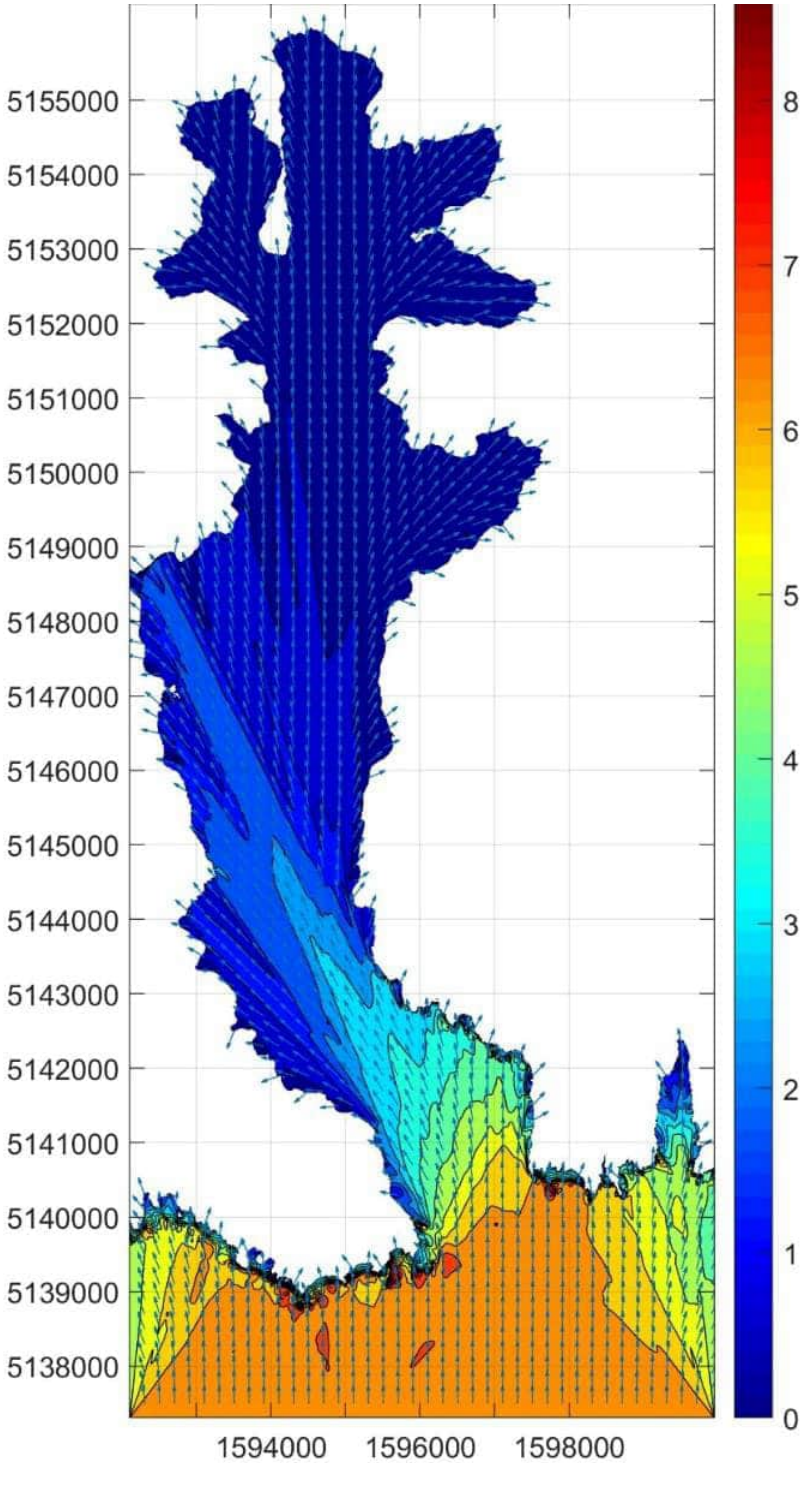


Figure Appendix B.4: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South – Swell.

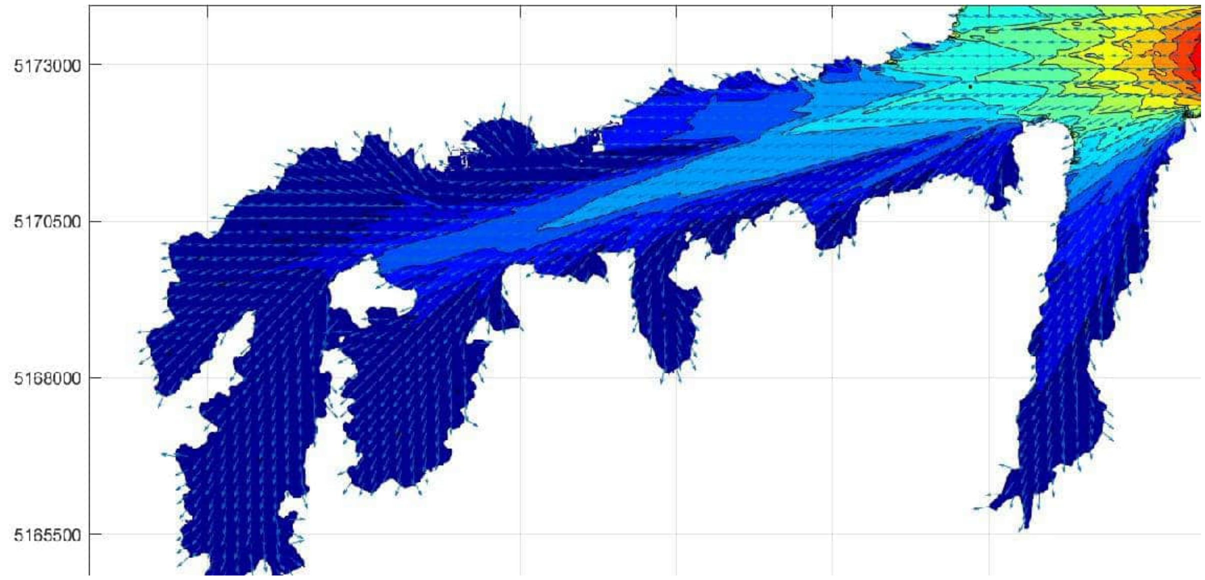


Figure Appendix B.5: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South- Swell.