Akaroa Wharf Resource Consent: Coastal Processes and Hazards Assessment

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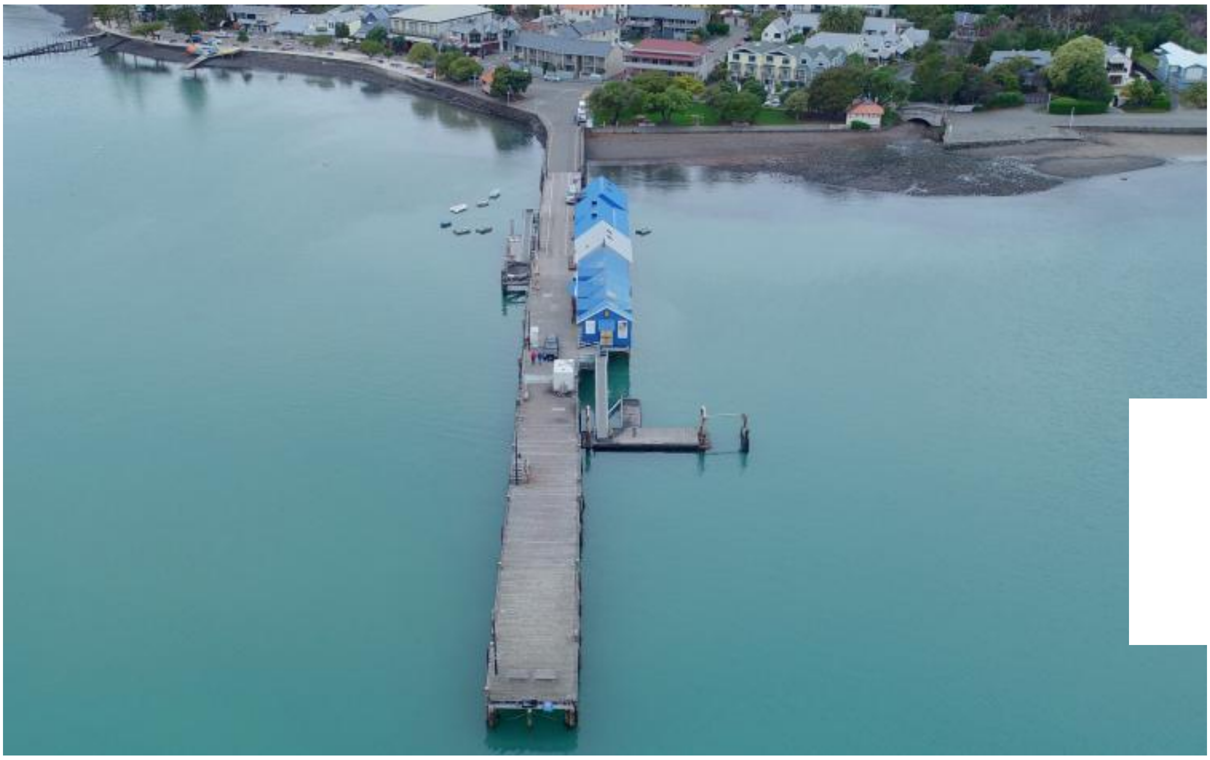

Akaroa Wharf Resource Consent: Coastal Processes and Hazards Assessment

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Executive summary

This coastal processes and hazards assessment has been prepared for Christchurch City Council (the Council) to accompany the Resource Consent Application for the proposed Akaroa Wharf replacement.

Coastal Processes

The key coastal processes relevant for the Akaroa wharf design are the wave climate and extreme sea level distribution, and the impact on these of future relative sea level rise (RSLR) and climate change.

The wave climate at the Akaroa wharf is dominated by low energy waves, with 30 years of wave hindcast indicating only 1% of significant wave heights (Hs) were greater than 0.5 m, the 100-year ARI (1% AEP) Hs heights being in the order of 0.7 - 0.8 m, and maximum individual wave heights up to 1.3 m to 1.4 m. These hindcast wave heights are lower than those reported in previous assessment, which were calculated from either recorded maximum daily or max gust wind speeds. A conservative estimate of nearshore wave set-up at the wharf from these extreme waves is 0.05 m due to the low energy wave environment and flat nearshore slopes.

Extreme static sea levels are the combination of astronomical tides, storm surge, variations in the mean level of the sea due to climate cycles, and wave set-up. Future RSLR due to a combination of global warming and VLM will increase the elevation of future extreme sea levels, so need to be accounted for in the wharf design. Total RSLR from present day (e.g. 2025) to 2100, including VLM of 2mm/yr, is estimated to 0.63 m under the “middle of the road” SSP2-4.5 scenario, 0.88 m under the ‘upper end” SSP5-8.5 scenario, and 1.23 m under the most extreme SSP5-8.5+ scenario.

The resulting projected 1-year, 10-year, and 100-year ARI extreme sea levels in 2100 are 2.60 m, 2.74 m, and 2.87 m (LVD37) under the SSP2-4.5 scenario, and 2.86 m, 3.00 m and 3.12 m under the SSP5-8.5 m scenario. However, there are large degrees of uncertainty with these projections, and that uncertainty increases with longer future time periods, with the likely range (i.e., p17-p83 values) for the 2100 SSP5-8.5 value being ± 0.35 m (e.g., 2100 100-year ARI sea level under SSP5-8.5+ scenario is projected to be 3.47 m LVD37). This is 0.41 m higher than the corresponding value given in Jacobs (2021) as the basis of the design for the height of the deck of the wharf, with the difference being due to 0.22 m higher estimate of extreme storm tide in the updated Tonkin &Taylor (2022) joint probability sea level, and 0.19 m higher RSLR from the inclusion of VLM.

Coastal Hazards

The land around the wharf is already exposed to current coastal inundation hazards from wave overtopping due to the low elevation of existing seawalls (approx. 11.5 m CDD) in relation to extreme water levels and storm wave run-up elevations. This exposure will get more frequent in the future due to sea level rise regardless of the wharf replacement. Inundation from static water levels at shore overtopping the sea walls is projected to only begin to occur in a rare event (e.g., 1–100-year ARI) under the high end (SSP5-8.5) SLR scenario by 2050, and in a 10-year ARI storm events by 2080 under both SLR scenarios. By 2100, without any change to the seawall levels, static water inundation is indicated to occur in a more frequent 1-year ARI storm events under both SLR scenarios, and by 2130, high spring tides are projected to overtop the current sea wall elevations under the high end SSP5-8.5 SLR scenario, and be very close to overtopping (e.g., within 0.1 m) under the lower SSP2-4.5 scenario. The wharf replacement will not alter these overtopping frequencies.

Coastal erosion is not predicted to be an issue adjacent to the wharf unless the current seawalls fail, with erosion distances under these circumstances being in the order of 4-5 m. Tonkin & Taylor (2021) considered that due to the status of these seawalls, any such failure would be rapidly repaired, such that the erosion exposure will not increase with time.

The wharf is also exposed to tsunami hazards, with Akaroa township having a history of tsunami inundation in at least three former tsunamis generated off the coast of Chile and Peru. Modelling of an extreme 2500-year tsunami event indicated that tsunami wave height at Akaroa township could be in the order of 4 m above water level and all low-lying land coastal was projected to be inundated, with depths greater than 2.5 m in places.

Assessment of Effects

The key components of the coastal processes to take account of in the wharf design are future extreme sea levels with SLR, wave height and energies, and sediment transport around piles. The coastal hazard policies of the NZCS (2010) and the Canterbury RCEP state that the consideration of coastal processes and hazards should include the effects of climate change particularly SLR, over a 100-year time frame. However, these policies do not specify what frequency of extreme sea levels should be considered in combination with RSLR.

The proposed wharf deck, has an elevation of 3.06 m LVD37 or 12.10 m CDD, which is in the order of 0.5 m above the current wharf level, and is designed to account for future RSLR, while still being practical in relation to adjacent land levels (including Beach Road), for which any decision on how and what climate change adaptation may occur are still some years away.

The design elevation is above the projected MHWS level by 2130 (taken as a 100-year period) with a 0.25 m freeboard even under the “high end” SSP5-8.5 RSLR scenario. Under this RSLR scenario, the likely future extreme sea levels in a rare 100-year event are projected to only start to interact with the wharf deck by 2100 (i.e. 75 years from present). This RSLR scenario and timeframe are consistent with the recommended allowances for RSLR for non-habitable assets with a functional need to be located at the coast (e.g. Category D activities) in the MfE (2024) coastal hazard guidance. Under this projection, interaction between extreme sea levels and the design wharf deck at this time (e.g. 2100) would be limited in depth to being in the order of 0.07 m and limited in duration to a short time period at the peak of high tide during the storm event. Under this RSLR scenario, by 2130 interaction between extreme sea levels and the wharf deck would increase in frequency to be likely to be occurring more frequently that once a year. Conversely, with the lower rate of RSLR scenario under the “middle of the road” SSP2-4.5 scenario, the projected most likely future extreme sea levels in a rare 100-year ARI event would only being to interact with the design deck elevation sometime between 2100-2130, and not until 2130 in an occasional 10-year ARI storm event.

It is considered that the proposed wharf structure will not have any adverse effect on any other coastal processes and will not exacerbate existing or projected coastal hazards.

Assuming that the stated demolition and construction methodology is followed, it is considered that any construction effects on coastal processes will be minor and temporary.

The construction of the temporary loading ramp adjacent to the Akaroa Boat ramp including the dredging 1000 m^3 of material to form of a berth pocket, will not have any distinguishable effects from the minor effects that already exist from the presence of the boat ramp.

Recommended mitigation measures

It is assumed that the structural design of the wharf will account for any temporary short duration interactions between the wharf deck and extreme sea levels in future rare storm events, so that they do not result in any structural damage to the wharf. These interactions are not projected to begin to occur until sometime after 50 years under the worst case RSLR scenarios (e.g. SSP5-8.5+), most likely closer to 80 years (e.g. SSP5-*.5 scenario), and possibility as long as 100 years under “middle of the road” sea level rise scenarios (e.g. SSP2-4.5).

But, if required by the consent authority to mitigate concerns around the health and safety of wharf users during extreme sea levels, a consent condition requiring the development of a wharf closure plan to apply during storm events could be applied. However, any such condition would be more relevant for any reconsenting of the wharf structure beyond that the initial 35-year term. Any such closure plan could also include tsunami events.

Consent conditions should also re-enforce the good practice demolition and construction methodology set out in the Holmes (2024) design report and the Enviser (2025) project description.

Important note about this report

The sole purpose of this report and the associated services performed by Jacobs is to present a coastal processes and hazards assessment to accompany the Resource Consent Application for the proposed Akaroa Wharf replacement and associated temporary loading ramp at the southern side of the Akaroa boat ramp, in accordance with the scope of services set out in the contract between Jacobs and Christchurch City Council ('the Client').

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

Jacobs derived the data in this report from information sourced from the Client and/or available in the public domain at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. Jacobs has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose described above and by reference to applicable standards, guidelines, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report, to the extent permitted by law.

This report should be read in full, and no excerpts are to be taken as representative of the findings. No responsibility is accepted by Jacobs for use of any part of this report in any other context.

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Acronyms and abbreviations

AEP	Annual Exceedance Probability
ARI	Average Recurrence intervals
CCC	Christchurch City Council
CD	Chart Datum
CDD	Christchurch Drainage Datum
CES	Canterbury Earthquake Sequence
C_{max}	Maximum individual crest height
The Council	Christchurch City Council
ECan	Environment Canterbury
The Harbour	Akaroa Harbour
H_{max}	Maximum individual wave height
H_s	Significant Wave Height
LINZ	Land Information New Zealand
LVD37	Lyttelton Vertical Datum 1937
MBES	Multibeam Echo Sounder
MfE	Ministry for the Environment
MSL	Mean Sea Level
MHWPS	Mean High Water Perigean Spring
MHWS	Mean High Water Spring
Mw	Moment magnitude
NZCPS	New Zealand Coastal Policy Statement
NZVD2016	New Zealand Vertical Datum 2016
RCEP	Regional Coastal Environmental Plan
RCP	Representative Concentration Pathways
RSLR	Relative Sea Level
SSPs	Shared Socio-Economic Pathways
SWAN	Simulating Waves Nearshore
Tm_{02}	Mean wave period
T_p	Peak Period
VLM	Vertical Land Movement

1. Introduction

1.1 Background

Jacobs have been commissioned by Christchurch City Council (the Council) to provide a coastal processes and hazards assessment to accompany the Resource Consent Application for the proposed Akaroa Wharf replacement and associated temporary loading ramp at the southern side of the Akaroa boat ramp. Jacobs have previously assessed various aspects of coastal processes and hazards in relation to the wharf replacement in 2019[1], 2021[2] and 2022[3]. Other relevant coastal processes reviews and coastal hazards reports relied on for this assessment include PDP (2024)[4] coastal processes assessment for Holmes Group for the wharf replacement design, and Tonkin and Taylor (2021)[5] coastal hazards assessment across the whole of Christchurch District Council's jurisdictional coastline.

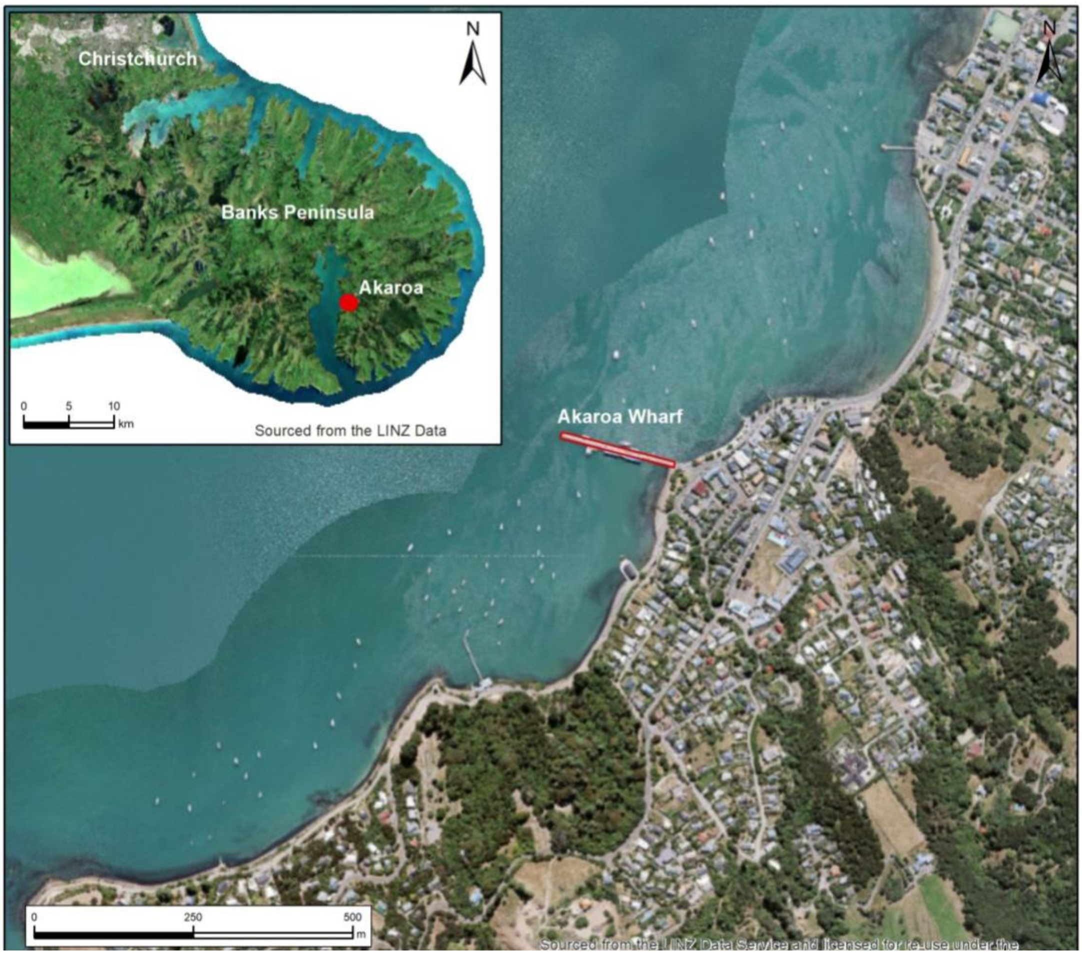

Figure 1.1: Overview map of Akaroa Wharf location. Inset map showing location of Akaroa Wharf in Akaroa Harbour in relation to Christchurch City and Banks Peninsula. Source: Jacobs (2019).

The existing wharf is approximately 185m long, consisting of 30 m long earth formation (e.g., reclamation also referenced as 'the abutment'), and a 155 m timber wharf. The wharf was originally constructed in 1887 of timber planks and piles, but has since had concrete pile encasements, and steel beams and bracings added

[1] Jacobs (2019) Akaroa Wharf Coastal Hazards Review. Report for CCC.

[2] Jacobs (2021) Akaroa Wharf Coastal Hazards Review: Report Addendum – Extreme Water Levels. Memorandum to CCC.

[3] Jacobs (2022) Akaroa wharf – confirming sea level rise for design. Memorandum for CCC, Planz, & Calibre.

[4] PDP (2024) Coastal Processes Report: Akaroa Main wharf. Report prepared for Holmes Group Limited.

[5] Tonkin & Taylor (2021). Coastal Hazards Assessment for Christchurch District – Technical Report. Report for CCC.

to aid in wharf repairs. Two buildings are connected to the wharf along its southern edge and are accessed from the wharf. Boats owned by the businesses located along the wharf and locals/individual users of the wharf moor on both sides of the wharf length. Services located on the wharf include fuel, power, telecommunications, wastewater, and water supply. The wharf is owned and maintained by the Council, with significant investment required to maintain it to a safe standard for public use. The buildings and associated primary supporting piles are privately owned, albeit reliant on the supporting piles at the wharf interface as owned by the Council.

The Council Statement of works for this coastal processes and hazards assessment states that the current Akaroa Wharf has reached the end of its design life, and it is no longer economically viable to maintain the existing structure. The Council is seeking to rebuild a new wharf in the general footprint of the existing wharf location to accommodate the current and future needs of both commercial and recreational wharf users.

1.2 Report Structure

The structure of this report is as follows:

- Section 2 – A brief description of the replacement wharf design and construction relevant to coastal processes and hazards.
- Section 3 - A summary of the Akaroa coastal process environment.
- Section 4 - An assessment of coastal hazard risks on the proposed activity.
- Section 5 – An assessment of potential effects of the proposed activity on coastal processes; and
- Section 6 – Conclusions and recommendations for mitigation measures and consent conditions

1.3 Vertical Datums

Data on sea and land levels in the Christchurch District is presented in four different vertical datums; Chart Datum (CD), Lyttelton Vertical Datum 1937 (LVD37), Christchurch Drainage Datum (CDD), and New Zealand Vertical Datum 2016 (NZVD2016), which makes comparisons between datasets confusing. Figure 1.2 presents the relationship between the datums. Data in this report is generally presented in LVD37 and CDD, being the two that the wharf design drawings are presented in.

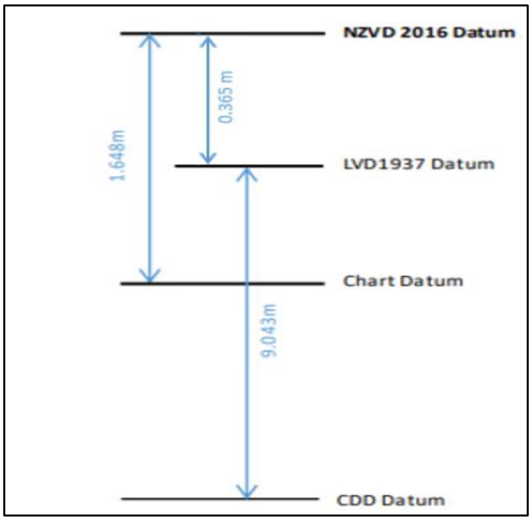

Figure 1.2: Relationship between commonly used vertical datums in the Christchurch District. Source: modified from Tonkin & Taylor (2021) by PDP (2024).

2. Summary of Replacement Wharf Design and Construction

2.1 Key Features of Design

The Akaroa Wharf is proposed to be rebuilt generally in the existing wharf's location with the footprint possibly shifted north by 1.5 – 2.5 m. The wharf length and width over the majority of its length are the same as the existing wharf, at 185 m long and 8 m wide. The existing wharf, including part of the 30 m long abutment and associated reclamation, is to be demolished.

The proposed design is for the wharf deck to be comprised of precast concrete prestressed panels with in-situ concrete topping, which will support the permanent loads of wharf furniture and the live load of pedestrians and vehicles (Holmes, 2024)[6]. New floating pontoons will be arranged on the northern and southern faces of the main wharf and will be accessed from the main wharf by gangways. The concept design shows the capping beams being supported by 24 rows of 710 mm diameter (size to be to be confirmed) reinforced concrete piles at 8 m centres.

A concept plan and long section of the proposed wharf is presented in Appendix A (sourced from Holmes, 2024 and Enviser, 2025[7]).

From the design report and concept plan, the relevant parts of the design for coastal processes are considered to be:

- Removal of the existing timber wharf structure, including piles. However, the new wharf is not going to extend under the existing buildings on the wharf (Black Cat & Blue Pearl), which are going to remain on their existing piles and are not going to be removed during the wharf rebuild. They will connect to the new wharf with a gangway.
- Removal of part the original 1887 abutment and associated reclamation back to the existing shoreline to accommodate the increase in deck height and lateral shift of the wharf.
- Existing seawalls on either side of the existing abutment will remain.
- A small area of new reclamation at the abutment enclosed by a concrete "L-wall" is proposed on the northern side of the boundary of the new wharf and the shoreline. This wall will be protected by an estimated 4-6 m rip rap zone at the base of the wall.
- The wharf height will be raised to 3.06 m LVD37 (12.10 m CDD) which is between 500-600 millimetres higher than the existing deck to provide some resilience to extreme sea levels and forecast sea-level rise.
- The wharf super structure is to sit on 24 rows of 710 mm diameter reinforced concrete piles at 8 m centres, which will be installed using sacrificial steel castings socketed into the basalt present below the seabed.
- The installation of 12-16 steel piles (710 mm diameter) required for the new floating pontoons (north and south). The installation of 18 timber piles to support the Black Cat and Blue Pearl buildings and associated access ways.

From this design it is assumed that the floor level of the existing wharf buildings that are to remain on their existing piles will be approximately 0.5 m lower that the deck of the new wharf.

To facilitate the wharf construction, a small loading ramp will be constructed on the southern side of the Akaroa boat ramp. This will require the following activities:

[6] Holmes (2024) Akaroa Wharf Renewal Design Features Report – concept Design. Report prepared for CCC.

[7] Enviser (2025) Akaroa Wharf redevelopment – Project Description (draft 2)

- A temporary reclamation on the beach on the south side of the boat ramp consisting of geotextiles, granular fill, and rip rap protection.
- Dredging the seaward approach to the ramp for approximately 90 m from the shoreline over a width of approximately 30 m by a mechanical excavator either based on a barge or from the shore at low tide.
- The dredging will involve the removal of approximately 1,500 m^3 of material, which will be placed to the southwest of the dredge area. This placement may be visible for a period of time until dispersed by natural wave and current processes.
- Two-four steel piles (610mm diameter) will be driven along the southern side of the existing boat ramp to form a training wall to facilitate the barge loading/unloading.

A concept diagram of the loading ramp is presented in Appendix B.

2.2 Construction Methodology

Construction includes both marine and landside activities, of which those relevant for coastal processes are set out below as presented in Enviser (2025).

2.2.1 Enabling Works

- Establish the loading ramp at the Akaroa Boat Ramp, including the undertaking of dredging of the berth pocket.
- Construct the "L-Wall" at the base of the existing wharf.

2.2.2 New wharf piling and old wharf demolition methodology

The piling methodology will rely on the existing wharf structure for support of the piling gates, so demolition will follow behind the landside piling front. The general sequence is as follows:

- The piling rig (crawler crane) will track out to the first bent on the wharf (i.e. proposed location of row of piles). A small section of existing deck will be cut out and removed in the location of one pile in the bent. (the other pile will likely sit outside the face of the existing wharf due to the alignment offset).
- The piling gate (two piles) will be placed on the existing wharf deck and secured in place. The piling rig will pitch and place the steel piles (all piles are expected to be pitched and driven in a single length). The piles will have a steel driving tip welded to the end to enable driving into the weathered basalt.
- Vibro piling methods (using a 100-ton crawler crane with an ICE 28RF vibro hammer) will be used to drive the piles as far as possible. A bore or percussion piling hammer will then be used to drive the piles until the desired embedment into the basalt is achieved. If the required embedment cannot be achieved with percussive piling, the pile may need to be removed, and a drill used to pre-drill a socket into the basalt before the pile is re-driven.
- Once the piles are installed, they will be filled with concrete and the capping beams will be put in place. Temporary platforms/grillage will be installed on the capping beam to allow the piling rig to advance to the next bent. Temporary piles may also be required to support this temporary works, but they will be the same diameter (or smaller) than the permanent piles.
- Eighteen timber piles will be driven between the wharf and the Black Cat and Blue Pearl buildings to provide support during construction.

- A second, marine-based piling crew will undertake a similar operation with a piling rig based on the barge. The marine-based rig will work from the outer end, install piles and then demolish the existing wharf. Once it has met up with the land-based rig, it will assist the land-based operation with the capping beams and placement of concrete in the piles. The marine plant will also be used to remove all the old timber piles that clash with the new, with the remainder cut at seabed level using HEB's hydraulic shears.

- Most wharf demolition materials will be shuttled by marine plant to the Laydown area by the loading ramp.

- Any remaining sections of wharf will be demolished, and the wharf deck can be constructed, which will comprise of:
    - Placement of precast deck elements on the capping beam
    - Installation of temporary formwork
    - Pouring the topping slab

- Finishing works (surface finishes, furniture installation, and electrical) will follow completion of the wharf deck.

The following additional information on the construction methodology and assumptions are also relevant for coastal processes:

- Material/soil etc from the removal of the abutment reclamation will be disposed of at an authorised land-based facility. If larger rocks are present, they may be re-used as riprap.

- Demolition material will be taken to an onshore laydown/staging area prior to disposal.

- Large construction materials (i.e., piles) are likely to be transported to the site via barge.

- That placement of materials on the seabed will be confined to within the design footprints for the wharf and floating pontoons.

2.2.3 Duration of works

A full programme is yet to be prepared, but initial estimates are an overall programme over 11-14 months comprising (noting some works are concurrent) of the following phases:

- Site setup: 1-2 months

- Demolition: 2-3 months

- Piling and deck: 5-6 months

- Deck furniture, services, and pontoons: 3-4 months

3. Akaroa Coastal Process Environment

3.1 Akaroa Harbour Geomorphology and Sediments

Akaroa Harbour is a coastal inlet located on the southern side of Banks Peninsula that formed in the eroded and partially flooded remnants of a basaltic shield volcano complex. Hydraulically, the Harbour has been classified as a "rock-walled tidal inlet" (Curtis 1985)[8], as the flow of water through the harbour is dominated by the tide rather than by fresh-water run-off from streams. The inlet is approximately 17 km long, and on average 2–3 km wide (Tonkin & Taylor, 2021). The LINZ bathymetry charts (LINZ, 1988)[9], show maximum water depths at the harbour entrance to be in the order of 25 m, sloping very gently to be in the order of -10 MSL in the central harbour between Akaroa Inlet and Wainui Bay.

The harbour seabed is gradually infilling with loess and volcanic sediment runoff from surrounding catchments, which along with patches of shell hash comprise the surface seabed sediment of the harbour. Hart et al (2009)[10] sampled the seabed sediment within the harbour and produced the sediment distribution map shown in Figure 3.2. Silts and clays were found to dominate most of the upper harbour, with clay dominating the central area between Akaroa Inlet and Robinsons Bay (possibly indicating a sediment sink zone). The lower harbour is dominated by fine sands and shell, with gravel being largely absent from most areas except for Wainui inlet and isolated patches around cliffs.

The sediments of French Bay in the vicinity of the Akaroa wharf are mapped as silty clays, grading into being more dominated by silt and then sand as it progresses further into Children's Bay, which has a sand beach. However, the sand at the main swimming beach in the township bordering Rue Lavaud has been renourished by sand sourced from outer Peninsula beaches. Closer to the wharf, the shoreline along Beach Rd on the north side of the wharf is comprised of vertical seawalls with a narrow fringe of rocks located along the toe that are exposed at low tide (Figure 3.1). The larger rocks against the sea wall appear to have been placed as toe protection, while smaller rocks lower on the profile appear to be natural. On the south side of the wharf the seawall fronting Britomart Reserve is fronted by a narrow all tide pebble and cobble beach, with an intertidal sand zone which gives way to a rocky low tide shore platform.

Figure 3.1: Placed and natural rocks along seawalls on the north side of the Akaroa main wharf.

[8] Curtis R. J. 1985. Sedimentation in a rock - walled inlet, Lyttelton Harbour, New Zealand. PhD thesis, Geography Department, University of Canterbury.

[9] LINZ (1988) Land Information New Zealand Bathymetric Chart NZ 6324.

[10].Hart, DE, Todd, DJ, Nation, TE, McWilliams, ZA. 2009. Upper Akaroa Harbour Seabed Bathymetry and Soft Sediments: A Baseline Mapping Study. Coastal Research Report 1/ECan Report 09/44

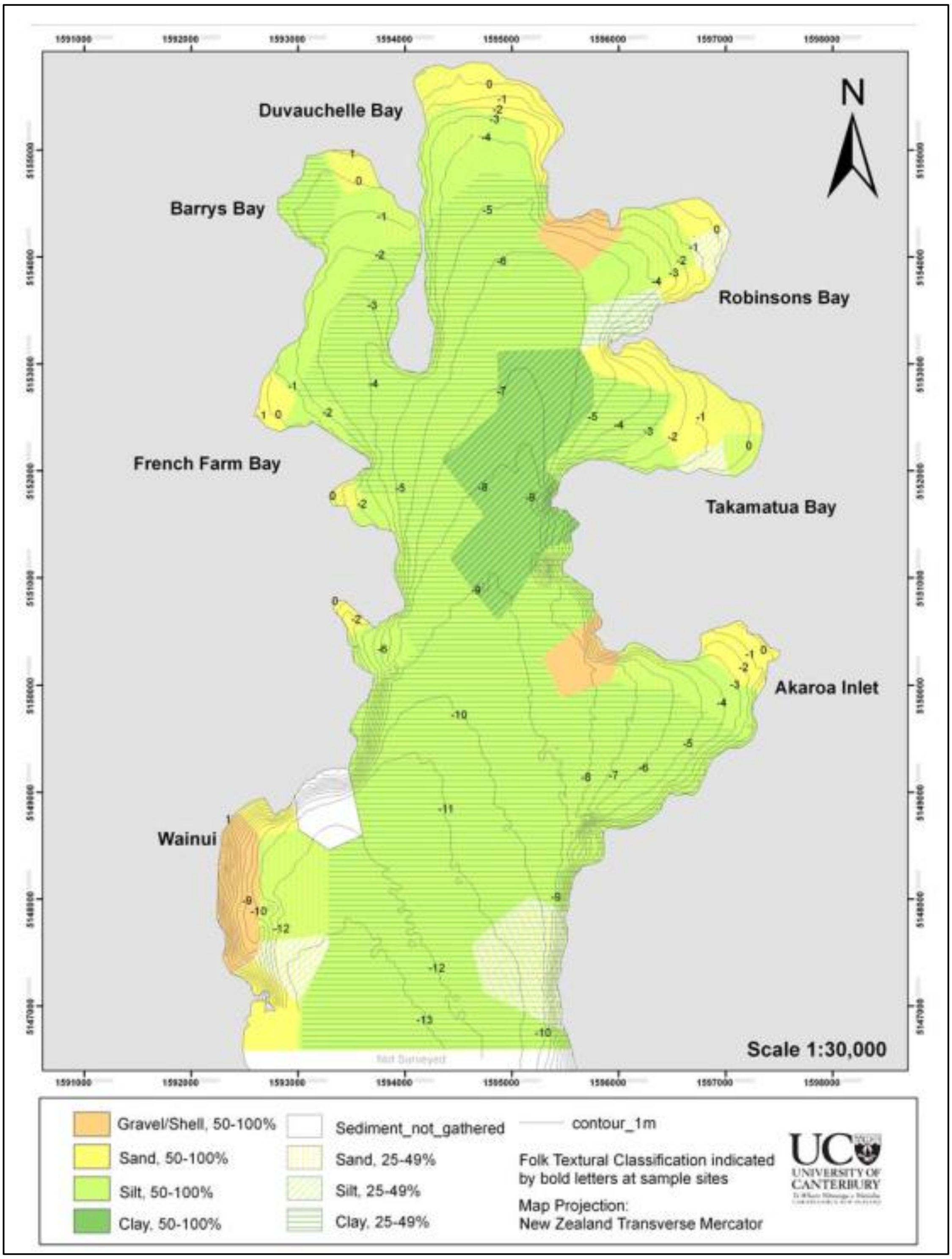

Figure 3.2: Upper Harbour sediment textures. Source: Hart et al (2009).

Bore sampling at the Akaroa Wharf presented in Holmes (2024a)[11] showed layers of clay, sandy silt, and silty sand down to a depth of 11 m below seabed at the seaward end of the wharf before striking semi and un-weathered basalt, with layers of completely weathered basalt beginning at depths of 8 m. Bore sampling at the landward end of the wharf showed gravel and sandy silt to a depth of 2 m below the seabed, followed by a 4 m thick layer of semi and un-weathered basalt, then a 9.5 m thick layer of silty clay and completely

[11] Holmes (2024a) Geotechnical interpretative report Akaroa wharf and Drummond's Wharf Foundation Design Advice. Report for CCC.

weathered basalt, before striking a layer of semi and highly weathered basalt at depths of 17.5 m below the surface.

In general, coastal erosion is not a significant issue in the Akaroa Harbour Basin due to the relatively low energy wave environment and because much of the harbour consists of hard, rocky shore platforms, reefs and cliffs that are resistant to erosion. Generally, any minor erosion is due to failure of coastal structures such as retaining walls or revetments (Christchurch City Council, 2007)[12].

3.2 Bathymetry

Updated bathymetry surveys of the area around the Akaroa Wharf were undertaken in 2020 by Southern Hydrographic[13]. Using Multibeam Echo Sounder (MBES). The resulting bathymetry is presented in Figure 3.3.

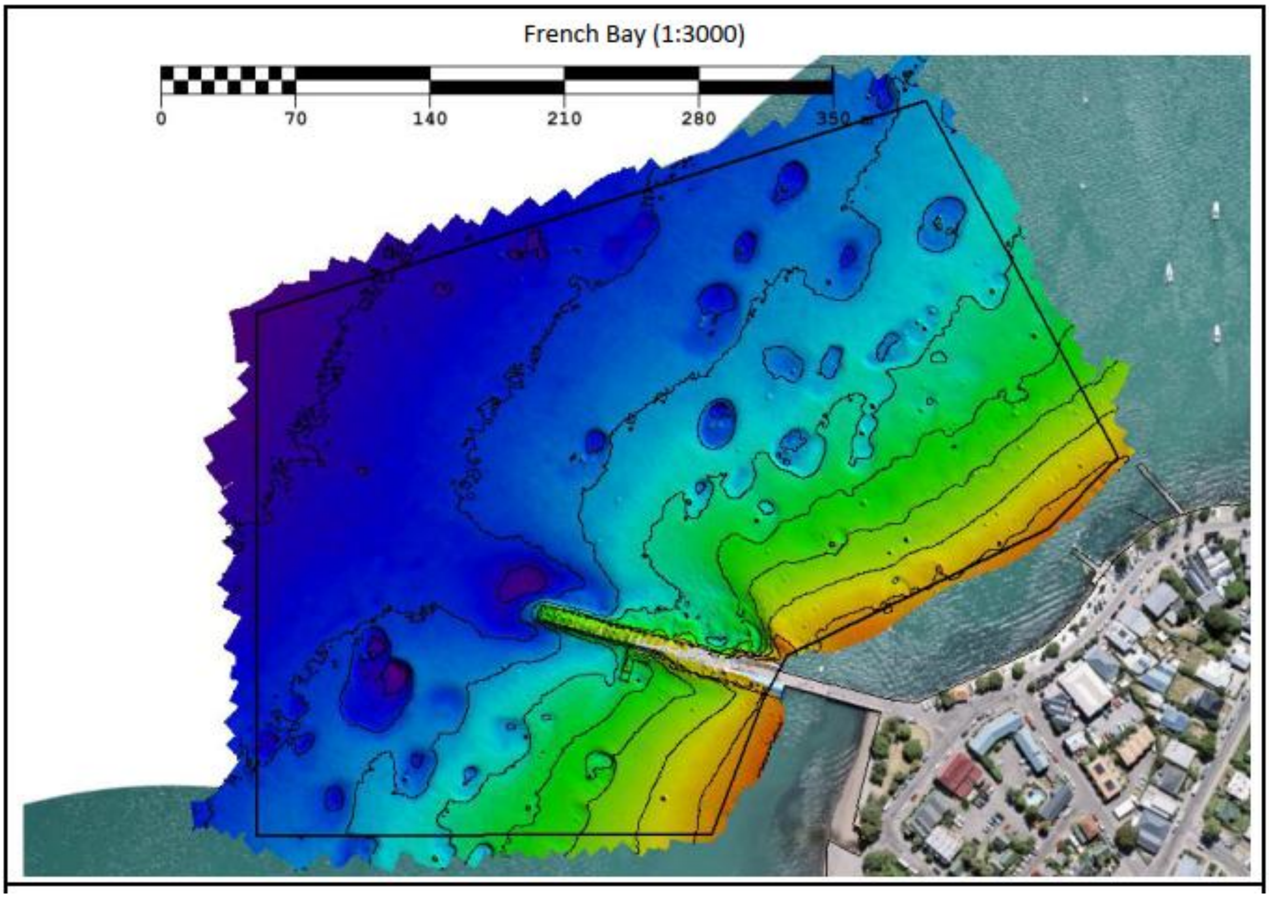

Figure 3.3: Bathymetry around Akaroa Wharf. Contours are on 0.5 m intervals from 0 m CD, with vertical datum given as CD. Source: Southern hydrographic (2020).

The mapping indicates that the seabed elevation at the end of the wharf is in the order 4.8 m CDD, -4.2 m LVD37, and -4.6 m NZVD2016[14]. This seabed elevation converts to a water depth in the order of 4.5 m at current MSL, and 5.7 m at current MHWS as defined in the LINZ secondary ports data set. This seabed elevation is in the order of 7.3 m below the proposed deck height of the new wharf.

[12] Christchurch City Council (2007). Akaroa Harbour Basin Settlements Study – Identifying the Issues.

[13] Southern Hydrographic (2020) Survey Summary Report. Prepared for CCC.

[14] From relationship between commonly used vertical datums presented in Tonkin & Taylor (2021).

The bathymetry shows that seabed elevations at the seaward end of the wharf are in the order of 0.5 m lower that the adjacent seabed both north and south of the Wharf. It is uncertain whether is a natural pattern or the result of some past dredging activities. However, no dredging has been carried out in recent years.

3.3 Wind and Waves

Akaroa Harbour is largely sheltered to ocean swell waves, with swell that does enter the harbour being significantly impacted by localised wave processes such as refraction. As reported in (DTec (2008)[15]; (sourced from Taylor (2003)[16]), mean southernly ocean Significant Wave Height (H_s) at the ECan deepwater wave buoy off Banks Peninsula of 1.8 m would produce a breaker height of 0.23 m at the headland between Duvauchelle and Robinsons Bay.

Therefore, waves within the harbour are largely generated by local winds blowing within the harbour, and hence are fetch limited from all directions.

3.3.1 Wind

Tonkin & Taylor (2021) reported that within Akaroa Harbour, the predominant wind direction is from the southwest and north northeast. This was confirmed for French Bay by a 42-year hindcast study (1979-2020) presented by MetOcean Solutions (2023)[17] for a site over water approximately 100 m from the end of the Akaroa wharf. The annual wind rose from this hindcast for 10-minute mean wind speeds at 10 m elevation is shown in Figure 3.4. This wind data is considered to be more relevant and representative than the data presented in other reports (Hicks & Marra (1998)[18], DTec (2008), Jacobs (2019), PDP (2024)). From the 41-year hindcast, mean wind speed for all directions was 3.98 m/s, and maximum wind speed was 17.74 m/s from the NNE, with over 17 m/s also being hindcast from the SSW.

For storm wave generation at the wharf, the most relevant wind directions are those with the longest fetch, being those blowing across the harbour from South-west thru to North-west directions. The results of the hindcast and fetch lengths for winds from these directions are presented in Table 3.1. Waves from winds blowing from more northerly and southerly directions are either refracted into the Akaroa Inlet, or have very short fetch lengths directly over water, so produce much smaller waves at Akaroa wharf.

[15] DTec Consulting Ltd. (2008). Akaroa Harbour Basin Settlements Study – Coastal Erosion and Inundation Project. Prepared for Christchurch City Council Strategy and Planning Group.

[16] Taylor A.J. (2003). Change and Processes of change on shore Platforms. Unpublished PhD thesis (Geography), University of Canterbury. 387p

[17] MetOcean Solutions (2023) Akaroa Harbour Wave Study – Metocean statistics for design of floating pontoons near Akaroa boat harbour. Report prepared for CCC

[18] Hicks, D. M. & Marra, J.J. (1988). Coastal processes and conditions in French Bay, Akaroa Harbour. Unpublished report prepared for Akaroa Marina Co. 46p.

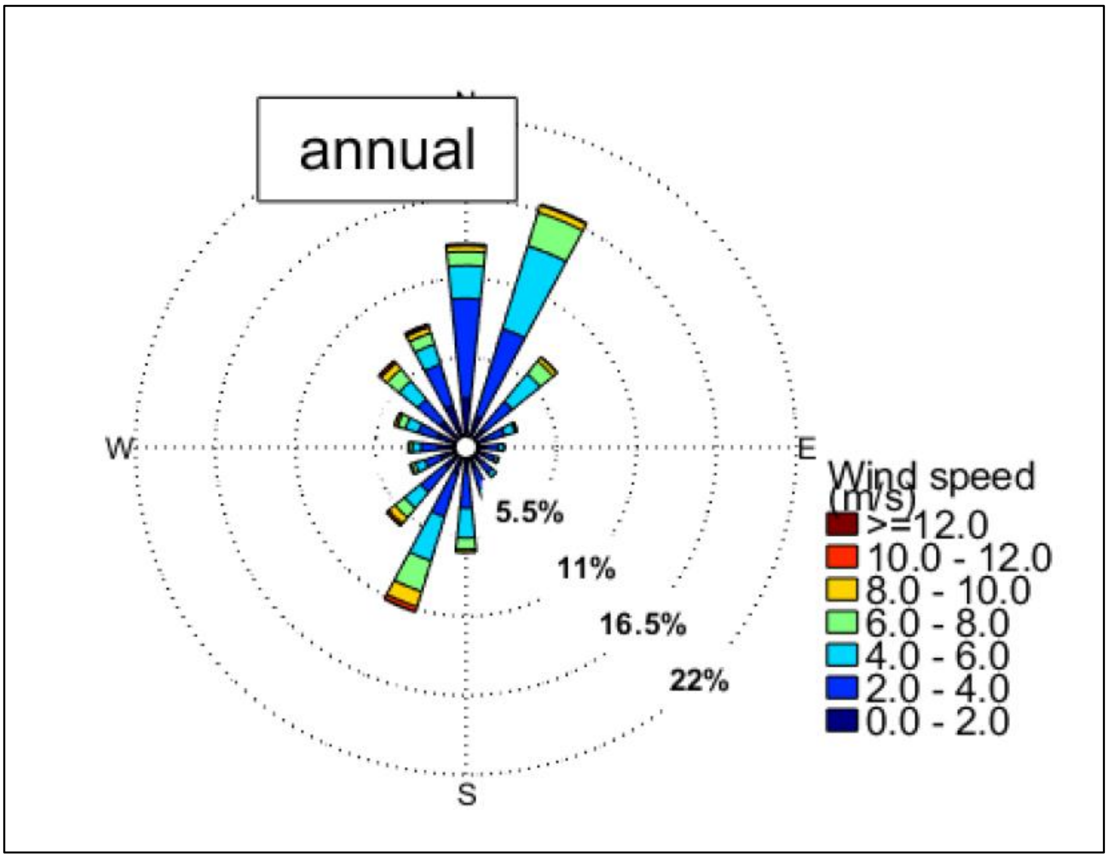

Figure 3.4: Annual wind rose (10-minute mean at 10 m AMSL) from 42 year hindcast (1979-2020) at site over water approximately 100 m from the end of Akaroa Wharf. Source: MetOcean Solutions (2023).

Table 3.1: Wind speeds (10 mins at 10 m elevation) from 42 year hindcast at Akaroa Wharf for main directions for storm wave generation. Wind data sourced MetOcean Solutions (2023).

Wind Direction	Fetch length to Akaroa Wharf (km)	Max 10 min wind speed (m/s)	% of winds in this speed band
North-west	5.4	16-18 m/s	>0.005%
West	3.6	14-16 m/s	>0.005%
South-west	5.2	16-18 m/s	>0.005%

3.3.2 Wave Climate

PDP (2024) notes that there is a large degree of variation in the methods and values of previous wave height estimates in and around Akaroa with wave heights ranging between 0.1 to 3 m, and states that "*many of the methods used are not highly accurate and based on visual observations, limited data, and generic equations*". However, PDP (2024) did not include the MetOcean (2023) SWAN (Simulating Waves Nearshore) model hindcast in their analysis. It is our consideration that this is the most appropriate database of the wave climate to use, as that dataset is: calculated by industry standard hindcast methods; is calculated for a site within 100 m of the end of wharf (water depth given as 5.7 m); and covers a 31-year time frame (1990-2020). The annual wave rose from the MetOcean report is re-produced in Figure 3.5, with data tables and statistics presented in the report for significant wave height (Hs) distribution, annual joint probability (%) of Hs and mean wave direction, annual joint probability (%) of Hs and Peak period (Tp), and return periods for storm wave Hs, Tp and mean wave period (Tm_{02}).

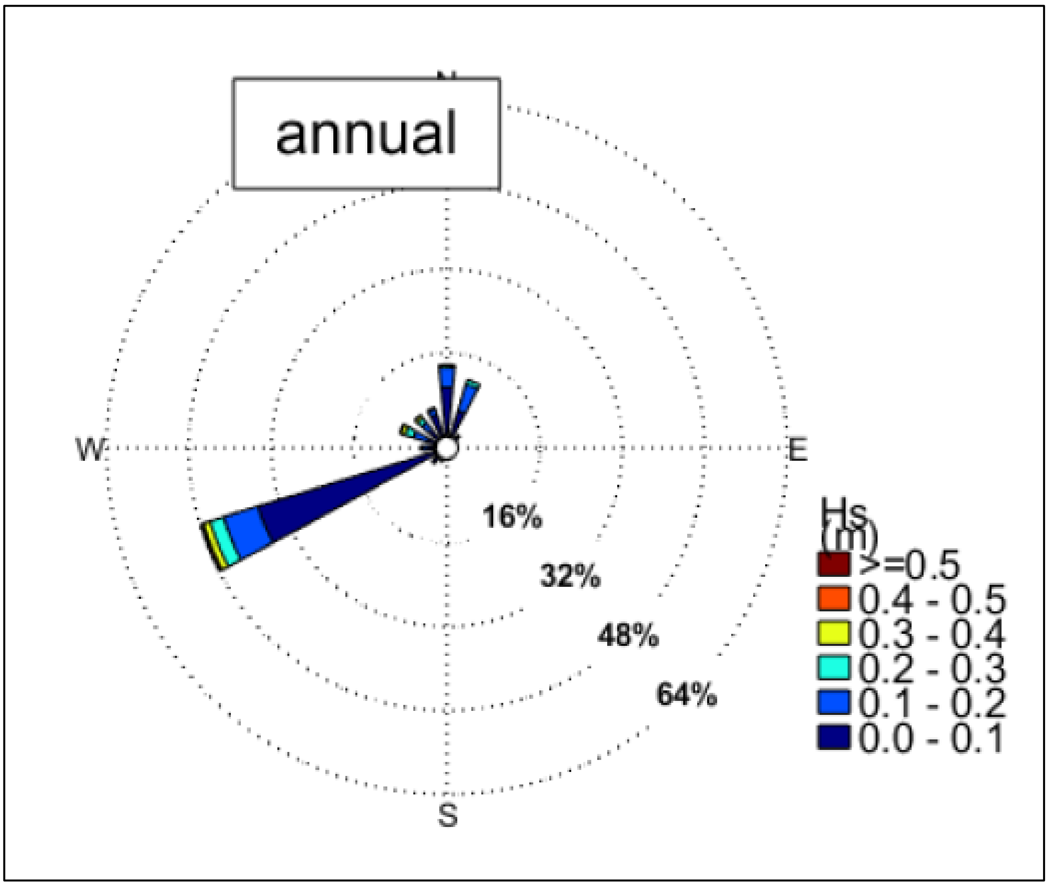

Figure 3.5: Annual wave rose from 31 year hindcast (1990-2020) at site over water approximately 100 m from the end of Akaroa Wharf. Source: MetOcean Solutions (2023).

As shown in Figure 3.5, the dominant wave approach direction is WSW (16 directional bins) with nearly 50% of all waves received at the Akaroa wharf being from this directional bin, being from both direct wind waves blowing across the harbour from this direction and refracted southerly waves from winds blowing up the harbour. Waves from the SW through to W directional window were also the largest Hs, having a maximum Hs of 0.73 m, but with <0.005% of waves having Hs > 0.7 m. The second most dominant wave direction is North - NNE, with close to 30% of waves being from this directional bin. However, these waves are generally smaller (due to short fetch length), with maximum Hs being 0.4-0.5 m. Waves approaching the wharf from a North-west directional bin have a lower frequency, occurring round 14% of the time, and have a maximum Hs of up to 0.6-0.7 m. A summary of the wave heights for each of the main approach directions (8 direction bins) at the wharf are shown in Table 3.2.

Table 3.2: Significant Wave Heights (Hs) for 31 year hindcast at Akaroa Wharf for main directions of wave approach (8 directional bins). Data sourced MetOcean Solutions (2023).

Wave approach direction	Fetch length to Akaroa Wharf	% of waves in directional bin	Maximum Hs band (m)	Most frequent Hs band for wave approach
North	1.2 km	25.7%	0.4 – 0.5 m	0.0 – 0.2 m
North-west	5.4 km	13.7%	0.6 – 0.7 m	0.0-0.2 m
West	3.6 km	39.6%	0.7 – 0.8 m	0.0-0.1 m
South-west	5.2 km	15.6%	0.7 – 0.8 m	0.0-0.2 m
Note: This data is from 8 directional rather than 16 directional bands in Figure 3.4.				

The hindcast statistics show that the wave climate at the wharf is dominated by low-energy waves. The distribution of Hs over the whole of the 31-year hindcast had a mean value of 0.1 m, with 65% of the waves being ≤ 0.2 m, only 1% being > 0.5 m. In conjunction with low wave heights, the distribution of peak wave periods (Tp) was dominated by very short period waves; 62% of Tp ≤ 2 seconds, only 23% > 12 seconds, and the largest Hs values associated with a Tp of 2-4 seconds.

It is noted that the most frequent wave height band is in the same order of magnitude as calculated by PDP (2024) by empirical methods recommended by CIRIA (2007)[19] for fully developed conditions from 4-years of maximum hourly mean wind speeds for a site in Akaroa township. However, the maximum Hs from the hindcast is much lower (e.g., 0.3 - 0.8 m lower) than the maximums calculated by Hicks and Marra (1988), DTec (2008) (as used by Jacobs (2019, 2021)), and PDP (2024) from maximum wind gusts. This is due to the fact that for waves to reach their maximum heights, sustained wind speeds from a constant direction are required, rather short-duration gusts. It is also noted that the 3s wind gust speeds calculated by the hindcast are much lower than given by Hicks and Marra (1988) and DTec (2008).

In terms of storm wave height return periods or Average Recurrence intervals (ARI's), Table 3.3 presents the heights for 1-year, 10-year, 50-year and 100-year Hs, Hmax (Max individual wave height), and Cmax (max individual wave crest)) from the MetOcean (2023) report. Given the industry standard method, close location over water, and 30-year length of the MetOcean hindcast, we consider these to be the most relevant wave heights for input into coastal hazard assessments. The results show that the 100-year ARI (equivalent to 1% AEP – Annual Exceedance Probability) Hs is in the order of 0.7 - 0.8 m for waves approaching the wharf from directions of SW around to NW, which is similar to maximum Hs calculated over the 30-years of the hindcast.

It is noted that these 100-year ARI heights are lower than those reported in Tonkin & Taylor (2021) for Akaroa Harbour in general (e.g., not site specific to Akaroa Wharf), and lower than those recommended in the Jacobs (2019) and PDP (2024) reports, calculated from either recorded maximum daily or max gust wind speeds at local land based sites, which may not have been sustained for sufficient time to generate maximum possible wave heights or taken into account depth limiting effect on wave heights close to the wharf.

Table 3.3: Storm wave return periods for wave height characteristics from for main directions of wave approach from 31 year hindcast at Akaroa Wharf. Data sourced MetOcean Solutions (2023).

Wave approach direction	Significant wave height (Hs) (m)				Max individual wave height (Hmax) (m);				Max individual wave crest height (Cmax) (m)			
	Return period (yrs)				Return period (yrs)				Return period (yrs)			
	1	10	50	100	1	10	50	100	1	10	50	100
North	0.31	0.41	0.48	0.50	0.65	0.80	0.85	0.89	0.41	0.51	0.55	0.56
North-west	0.54	0.67	0.76	0.79	1.08	1.25	1.35	1.38	0.69	0.81	0.88	0.90
West	0.40	0.56	0.66	0.70	0.82	1.10	1.25	1.31	0.51	0.71	0.83	0.88
South-west	0.52	0.65	0.73	0.77	1.06	1.27	1.38	1.42	0.68	0.82	0.90	0.93

As shown in Table 3.3, maximum individual wave heights could exceed the 100-year ARI Hs in a 1-year return period event and are projected to be up to 1.3 m to 1.4 m in a 100-year ARI event. Due to wave asymmetry, the maximum individual crest height above static water level is a little less, in the order of 0.9 m for a 100-year ARI event.

[19] CIRIA, CUR & CETMEF. (2007). The Rock Manual. The use of rock in hydraulic engineering (2nd Edition), C683, CIRIA, London (ISBN: 978-0-86017-683-1).

As per the recommendations of MfE (2024)[20], potential Climate Change impacts on the future extreme wave climate also needs to be considered, with the recommendation being that that sensitivity testing using a range of plausible increases of between 0 and 10% be undertaken. As per PDP (2024), we agree that a 10% increase should be restricted to the western and southern South Island, therefore consideration of a 5% increase is appropriate. This small percentage of increase would only add 0.04 – 0.05 m to the 100-year ARI Hs and Cmax wave heights.

3.4 Wave Set-up and Run-Up

Wave set-up and wave run-up are generally important considerations for coastal hazard assessments and design of structures located at the shoreline. Wave set-up is a sustained increase in mean water level at the shore due to transfer of momentum as waves break as they approach the shore. This increases the static sea at shore. Wave run-up is the up-rush of broken waves above the static water level across a beach or structure located at the shore. This is referred to as the dynamic water level.

3.4.1 Wave set-up

Wave set-up is generally very small in estuaries/harbours as it is dependent on wave height and period, and nearshore slope at the break point. Therefore, since the Akaroa wharf site is exposed to a low energy wave environment from refracted ocean swell and fetch limited local wind waves, wave set-up is limited in magnitude.

Jacobs (2019), relying on maximum daily SW wave characteristics information presented in DTec (2008), (Hs=1.08 m, Tm = 2.62 sec) and a nearshore slope of 1:100, calculated a theoretical maximum wave height of 0.01 (equation from Holman (1986)[21] to 0.03 m (from NIWA coastal calculator, Stephens et al 2015)[22]. Based on these results, Jacobs (2019) recommended a conservative estimate of wave setup around the wharf of 0.05 m. PDP (2024), applying their recommended maximum wave height (1.5 m) and period (3.53 s) and slope of 1:100 into the Holman (1986) wave set-up equation obtained a similarly low maximum set-up of 0.015.

Tonkin & Taylor (2021) applied a different empirical set-up formula (Guza and Thornton (1981)[23] that is not dependent on nearshore slope due to uncertainty on the bathymetry to obtain an order of magnitude higher maximum wave set-up height of 0.25 m for the entire Akaroa Harbour, although it was noted that set-up in some parts of sheltered embayments may be less than these estimates. Since wave set-up is highly dependent on nearshore slope, Jacobs (2021) undertook sensitivity testing of the Tonkin & Taylor estimate applying their maximum wave height estimates (e.g., 1.5 m) with 1:100 and 1:20 slopes in the Holman (1986), obtaining resulting wave set-ups of 0.06 m and 0.15 m respectively. From this Jacobs concluded that the Tonkin & Taylor estimate is most likely to be over conservative, being 0.2 m higher than the Jacobs (2019) recommended maximum set-up of 0.05 m. PDP (2024) agreed that the Jacobs estimate of 0.05 m wave set-up is appropriate for the Akaroa Wharf design but noted that this still significantly greater than their estimate of 0.015 m using slightly higher wave height estimates. Applying the smaller maximum wave heights from MetOcean (2023) results in a smaller wave set-up, less than 0.01 m, re-enforcing the conservativeness of the 0.05 m wave set-up estimate.

In relation to climate change impacts, we agree with PDP (2024) that the current estimates are already conservative and any increases with be insignificant.

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[20] Ministry for the Environment (2024). Coastal hazards and climate change guidance. Wellington: Ministry for the Environment.

[21] Holman R. A. (1986). Extreme Value Statistics for Wave Run-Up on a Natural Beach. Coastal Engineering, 9, 527-544

[22] Stephens, S, Allias, M, Robinson, B & Gorman, R, (2015). Storm-tides and wave runup in the Canterbury Region. Prepared for Environment Canterbury. Includes a Coastal Calculator tool for calculating storm tide, storm surge, wave set and run-up at multiple sites along the Canterbury open coast.

[23] Guza, R.T. & Thornton, E.B. (1981). Wave set-up on a natural beach. Journal of Geophysical Research: Oceans 86, 4133– 4137.

3.4.2 Wave Run-up

For the purpose of the Akaroa Wharf design, both Jacobs (2019) and PDP (2024) agree that a wave run-up assessment is unnecessary as there will not be run-up onto the wharf structure, and the structure will not impact run-up on the adjacent seawalls (e.g., it will remain as present). However, it is noted that run-up is likely to over top these walls in extreme events, which will occur more frequently with future sea level rise, and there is potential for wave-splash/spray onto the wharf during extreme water level/wave conditions.

DTec (2008), applying the maximum daily wave heights calculated from the wind record (Hs=1.08 m from SW direction), estimated the wave run-up elevation at the north end of Akaroa township (e.g., close to the wharf) as being 0.25 m for natural beaches, 1.2 m on sloping rock revetments, and 1.7 m on vertical seawalls. Tonkin & Taylor (2021) did not calculate wave run-up in Akaroa Harbour. Although not recalculated, these run-up elevations would be less for the smaller maximum Hs from the MetOcean (2023) hindcast, even with small increases in wave height with climate change.

3.5 Sea Levels

3.5.1 Astronomical Tides

The LINZ secondary ports dataset gives MSL at Akaroa as being 1.5 m CD (9.26 CDD, 0.22 m LVD37[24], -0.15 m NZVD2016), with the mean spring tide range of 2.3 m and mean neap tidal range of 1.2 m. This dataset gives the MHWS level as 2.7 m CD (10.46 CDD, 1.42 m LVD37[22], 1.05 m NZVD2016), being 1.2 m above MSL.

Jacobs (2021) noted that the values given in the LINZ database are only to one decimal place, so can vary by ± 0.05 m, and that MHWS definitions can vary, as the South Island east coast experiences monthly perigean and apogean tides (Stephens et al., 2015) with a single dominant spring tide per month (Perigean tide) rather than fortnightly spring and neap tides. The coastal calculator (Stephens et al., 2015) gives the MHWPS (Mean High Water Perigean Spring) of 1.08 m above MSL for Birdlings Flat (site in calculator closest to Akaroa Harbour) and 1.2 m above MSL at Lyttleton. Therefore, although the tidal range appears to be larger in Akaroa Harbour, a MHWS level closer to 1.15 m above MSL is considered to be more likely. However, as a conservative approach for the updated analysis, a MHWS level of 1.2 m above MSL is retained.

3.5.2 Storm Tides

Storm tide is the super elevation of the static sea level at shore due to the combination of astronomical tide, storm surge, and longer-term variations in the mean level of the sea. Storm surge is the height of the sea level above the astronomical tide resulting from the effects of low atmospheric pressure (barometric uplift) and high onshore winds (wind stress), which are usually associated with storm events, therefore commonly occur in combination with elevated wave heights. Variations in the mean level of the sea occur due to seasonal, interannual and inter-decadal weather and climate cycles, and can add up to ± 0.25 m to the mean level of the sea in New Zealand (Stephens & Bell, 2011)[25].

3.5.2.1 Storm Surge

DTec (2008), using air pressure reported at Christchurch airport and maximum wind speeds measured at the Akaroa MetService site, calculated maximum storm surge in Akaroa Harbour to be in the order of 0.6 m with a return period in the range 20 - 25 years. Jacobs (2019) noted that the return period for this magnitude of surge was likely to be longer due to the joint probability of astronomical tide and storm surge. Jacobs (2021) used the coastal calculator (Stephens et. al., 2015) to calculate a 1% AEP (e.g., 100-year return period) storm

[24] Note these MSL and MHWS levels are 0.05 higher than given in Jacobs (2022).

[25] Stephens S. Bell R. (2011) Toolbox 2.2.2: Causes of sea level variability. In Impacts of climate change on urban infrastructure & the built environment.

surge of 0.54 m at the nearby Birdlings Flat site where barometric pressure and wind stress were considered to be similar. From this, it was inferred that the calculated 0.6 m storm surge in Akaroa harbour was an appropriate estimate of the 1% AEP with-in the harbour. Similar pro-rata adjustments to more frequent Birdling Flat storm surge resulted in the inferred frequencies of present day extreme surge in Akaroa Harbour given in Table 3.4.

Table 3.4: Inferred present day storm surge elevations for Akaroa Harbour for current day conditions. Source Jacobs (2021)

Frequency (% AEP)	10% (10 yr ARI)	5% (20 yr ARI)	2% (50 yr ARI)	1% (100 yr ARI)	0.5% (200 yr ARI)
Storm Surge elevation (m)	0.50	0.53	0.57	0.60	0.63

Tonkin & Taylor (2021) do not report surge storm magnitudes or frequencies in Akaroa Harbour. PDP (2024) agreed with Jacobs (2021, 2022) that 0.6 m is an appropriate current day storm surge design level for the Akaroa Wharf, but with a suggestion to increase this by 5% in future projections to allow for potential increases due to climate change following guidance from MfE (2024). The resulting increase in surge elevation would be very small, only 0.03 m for a 100-year ARI event.

3.5.2.2 Extreme Sea levels

Jacobs (2021) estimated a 100-year ARI extreme static sea level at Akaroa from the combination of a 0.6 m storm surge with MHWS tide levels and 0.05 m wave set-up. The resulting present day 100-year ARI extreme sea level was estimated to be 2.02 m LVD37 (11.06 m CDD). This extreme sea level elevation was carried through into the Jacobs (2022) memorandum for wharf design.

However, due to the multiple ways that astronomical tide, storm surge and variations to the mean level of the sea can be combined to produce extreme sea levels, a more appropriate and accepted method for estimating extreme storm tide distributions is by a joint probability analysis of the astronomical tide and storm surge components. Future extreme storm tides also include relative sea level rise.

Tonkin & Taylor (2021) used a joint probability approach to estimate the present day 1-year, 10-year, and 100-year ARI extreme storm tides. For Akaroa Harbour this analysis gave a 100-year ARI extreme storm tide (i.e., excluding wave set-up) of 2.04 m NZVD2016 (T&T 2021; Table 7.2), corresponding to 2.41 m LVD37 or 11.45 m CCD, which is 0.44 m higher than the corresponding 100-year ARI extreme storm tide level from Jacobs (2021). However, as presented in Jacobs (2022) and confirmed by Tonkin & Taylor (2022)[26] these storm tide levels are too high due to applying the tidal level correction from Sumner rather than from Lyttleton. The resulting corrected 100-year ARI extreme storm tide from Tonkin & Taylor (2022) was reduced 0.22 m to 2.19 m LVD37 or 11.23 m CDD. This level is still 0.22 m higher than the corresponding 100-year ARI storm tide level given in Jacobs (2021) due to the use of the joint probability approach (e.g., it is likely that the maximum storm tide each year will be greater that the combination of MHWS and 0.6 m storm surge).

Tonkin & Taylor (2022) did not address the issue of the gross over-estimate of wave set up (i.e., 0.25 m) in the total static extreme water levels given in Tonkin & Taylor (2021; Appendix D, Table 11). As discussed above, it is considered that a more realistic set-up elevation of 0.05 m should be applied, which PDP (2024) consider to also be a conservative estimate. Applying this set-up to the Tonkin & Taylor (2022) extreme storm tides gives the present day extreme static sea levels which are presented in Table 3.5. The 100-year ARI extreme sea level is the same as presented in Jacobs (2022).

[26] Tonkin & Taylor (2022) Coastal Hazard Assessment for Christchurch District Addendum Report.

In relation to the proposed wharf deck, with present day sea levels, the 1-year ARI static sea level at the shore will be more than 1 m below the deck level, and the extreme 100-year ARI static sea level will be around 0.8 m below the deck level.

The impact of projected SLR on future extreme sea levels is examined in the next section.

Table 3.5: Present day extreme static sea levels at Akaroa from Tonkin & Taylor (2022) corrected for inclusion of 0.05 m wave set-up at shore.

Return Period	1-yr ARI	10-yr ARI	50-yr ARI	100-yr ARI	200-yr ARI
From T&T (2022) +0.05 m wave set-up (NZVD2016)	1.60	1.74	1.83	1.87	1.91
Extreme static sea level including wave set-up (LVD37)	1.97	2.11	2.20	2.24	2.28
Extreme static sea level including wave set-up (CDD)	11.01	11.15	11.24	11.28	11.32

3.5.3 Relative Sea Level Rise

Sea level rise (SLR) can be referred to as absolute/eustatic – which is SLR relative to Earth's centre, driven by climate change; or relative sea level rise (RSLR) which is the SLR relative the local land level, so also accounts for local vertical land movement (VLM). RSLR is what is recorded by long-term port tidal records referenced to the elevation of a land based benchmark and is what is important for coastal hazard assessments and wharf structure design purposes.

Hannah and Bell (2012)[27] derived sea level trends from New Zealand's available long-term port records (~1900–2008) and the average trend of RSLR was 1.7 ± 0.1 mm/year (from MFE, 2017)[28], with Christchurch having a higher-than-average rate of 2.12 ± 0.09 mm/year of RSLR (Tonkin & Taylor, 2021).

Predictions of future RSLR is complex, with multiple climate change scenarios predicted to accelerate contemporary rates of SLR by varying amounts, and VLM being spatially variable often displaying subsidence and uplift over short distances. For the Christchurch district, there is also potential temporal variations due to the impact of the Canterbury Earthquake Sequence (CES).

3.5.3.1 Sea level rise due to climate change

For coastal hazards and climate change assessments, the requirement of Policy 24 of the NZ Coastal Policy Statement (NZCS 2010) is to consider the latest available guidance on SLR, which is currently MfE (2024). For climate change driven SLR, this guidance adopts the five medium confidence Shared Socio-Economic Pathways (SSPs) scenarios presented in the IPCC 6th climate change assessment (IPCC, 2021)[29] that span a wide range of plausible societal and climatic futures, ranging from 1.5 degrees Celsius 'best-case' low emissions (SSP1-2.6) to over 4 degrees Celsius warming (SSP5-8.5) by 2100.

This most recent MfE guidance post-dates the Jacobs (2019, 2021) and the Tonkin & Taylor (2021) assessments, all of which applied the earlier MfE (2017) guidance based SLR under RCP (Representative concentration Pathways) pathways from the IPCC (2013) 5th climate change assessment. Jacobs (2022) and

[27] Hannah, J. & Bell, R.G. (2012). Regional sea level trends in New Zealand. Journal of Geophysical Research: Oceans 117(C1): C01004.

[28] Ministry for the Environment (2017). Coastal hazards and climate change: Guidance for local government. Wellington: Ministry for the Environment.

[29] International Panel for Climate Change (IPCC) (2021). Climate Change 2021: The Physical Science Basis. Contribution of Working Group I to the Sixth Assessment Report of the Intergovernmental Panel on Climate Change.

PDP (2024) assessments applying the more recent SSP SLR projections presented in MfE (2024). However as pointed out in Jacobs (2022), the updated magnitudes of absolute SLR were "very similar" to those used in Jacobs (2021), being 1.04 m by 2120 under RCP 8.5 or by 2100 under RCP 8.5+ scenario. Jacobs (2022) concluded that there was no justification to further update the SLR projections due to climate change in the wharf design.

The MfE (2024) guidance recommends using SLR estimates based on the median (50th percentile (p50)) value of projection for that scenario in the NZ SeaRise platform, and the upper-bound of the likely range, the 83rd percentile (p83) in NZ SeaRise from the high-end emissions scenario SSP5-8.5 to represent a more extreme sea level rise scenario referred to as SSP5-8.5+. The resulting absolute (or climate change), SLR projections for NZ for the "middle of the road" SSP2-4.5 scenario, and the "high end" SSP5-8.5 scenario, (including the p83 values for SSP5-8.5+ scenario) across multiple time frames are presented below in Table 3.6.

Table 3.6: Absolute (Climate change) SLR projections for NZ. Source: NZ SeaRise (2024) Platform.

Scenario	SSP2-4.5 Scenario			SSP5-8.5 Scenario		
Percentiles	p17	p50	p83	p17	p50	p83(2)
2050	0.16 m	0.22 m	0.29 m	0.20 m	0.26 m	0.32 m
2080	0.33 m	0.42 m	0.55 m	0.45 m	0.56 m	0.72 m
2100	0.43 m	0.56 m	0.74 m	0.66 m	0.82 m	1.06 m(3)
2130	0.59 m	0.80 m	1.08 m	0.94 m	1.25 m	1.67 m

Note:
(1) NZ SeaRise projections are from a 1995-2014 baseline with a mid-point (zero) at 2005.
(2) The p83 projections for SSP5-8.5 are the MfE (2024) SSP5-8.5+ projections.
(3) The highlighted value is for the same projection and timeframe as given in Jacobs (2021) for use in determining the design height of the wharf deck.

It is important to note that the NZSeaRise projections in Table 3.6 are from a 1995-2014 baseline sea level with a mid-point taken as 2005, whereas the projections presented in Jacobs (2021, 2022) are adjusted to be from a 2020 base data, being the present sea level at the time of the reporting which recognises that SLR had already occurred since 2005. As a result, the magnitude of absolute SLR to a future date presented in Table 3.6 are higher than presented in the Jacobs reporting to the same date. For example, the highlighted 1.06 m absolute SLR from 2005 to 2100 under the SSP5-8.5 (p83) scenario (e.g. SSP5-8.5+) in Table 3.6 is equivalent to 0.99 m SLR from a 2020 base date.

3.5.3.2 Vertical Land Movement (VLM)

The NZ SeaRise platform (https://searise.takiwa.co/), released in 2022 and updated in 2024, maps sea-level rise projections around the entire NZ coast at 2 km spatially resolution that incorporate the site-specific local VLM with the above five medium confidence climate change scenarios out to 2150. It is noted that the Jacobs (2019 & 2021) and Tonkin & Taylor (2021) assessments pre-date the release of the NZ SeaRise platform, therefore do not include consideration of VLM in the SLR projections. The Jacobs (2022) and PDP (2024) assessments do include the 2022 NZ SeaRise projections in their consideration of RSLR.

The VLM rates presented in the NZ SeaRise platform are derived from high resolution satellite observations (InSAR - an emerging scientific approach), but only includes a very short period of 8 years of data from 2003 to 2011. For Canterbury this data pre-dates the CES (2010-2011) and is therefore considered to be inter-seismic rates which are not inclusive of the changes in post-seismic rates which are known to have occurred following the CES (Otago University, 2022)[30]. MfE (2024) recognises that one of the biggest uncertainties in calculating RSLR projections is local VLM and acknowledges that "*Independently determining locally measured VLM rates may also be relevant for Christchurch, North Canterbury and Kaikōura, once monitoring establishes a consistent post-earthquake trend in VLM*".

To further understand post-CES VLM rates across the Christchurch district, CCC commissioned additional analysis by GNS (2023, 2024, 2025)[31] to understand what more recent post-seismic rates of VLM are at a district-wide scale, and to understand what the variability in these rates may be in the future. This analysis further highlighted the variability in VLM rates both spatially across the district, and also through time, in both pre- and post-CES.

Inter-seismic VLM Data reported by NZSeaRise platform

Figure 3.6 presents the inter-seismic VLM rates for the Christchurch District coast reported at 2 km spatial resolution in the NZ SeaRise platform. As shown in the Figure, these rates are highly variable across the district, with some reporting subsidence and some reporting uplift. However, most sites on the Banks Peninsula were reported as subsiding, generally in the range of 1-3 mm/yr, with some areas on the coast of southern Peninsula reporting subsidence up to -4 mm/yr.

For Akaroa Wharf, the closest NZ SeaRise site (site 4450) is located approximately 400 m south-west along Beach Rd, and the second closest site (site 4451) being located 1600 m NNE in Children's Bay. The inter-seismic VLM rates at these sites given by the NZ SeaRise (2024) are very similar, being -2.0 ± 2.4 mm/yr and -1.9 ± 2.3 mm/yr respectively. Applying these VLM rates at a consistent rate into the future would result in an increase to the absolute SLR projections (Table 3.5) of 0.1 m over 50 years and 0.2 m over 100 years. It is noted that the VLM subsidence rate given by NZ SeaRise (2022) for site 4450 that was reported in Jacobs (2022) and PDP (2024) was higher at -2.56 mm/yr for this site, which would result in a slightly higher RSLR of 0.042 m over a 75-year period to 2100 and 0.059 m to 2125.

[30] School of Surveying, Otago University (2022). Christchurch City Ground Height Monitoring. Vertical land motion in Eastern Christchurch. Report to Environment Canterbury.

[31] GNS (2023) Inter-seismic, co-seismic and post-seismic rates of vertical land movement in the Christchurch district and implications for future changes in sea level.

[31] GNS (2024). Addendum to: Inter-seismic, co-seismic and post-seismic rates of vertical land movement in the Christchurch district and implications for future changes in sea level.

[31] GNS (2025). Interim report of re-estimated LVM along ECan/CCC coastal strip.

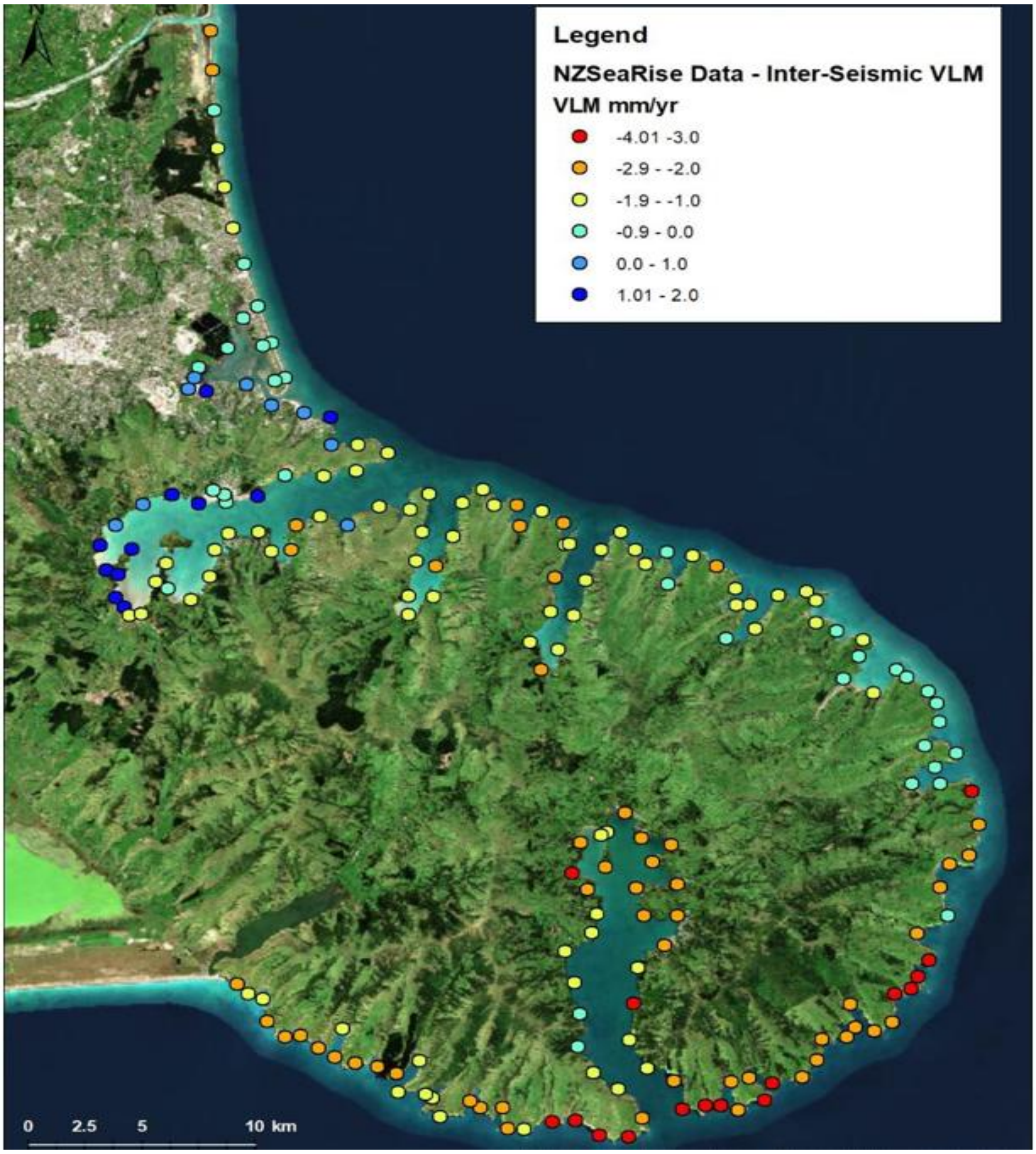


Figure 3.6: NZ SeaRise Inter-seismic VLM rates at 2km spacing along the Christchurch

GNS Post-seismic VLM Data

The 2010-2011 CES generally altered the rates of subsidence and uplift throughout the district both during the earthquake sequence (e.g., co-seismic) and after the sequence (post-seismic). The GNS (2002, 2023, 2025) analysis used InSAR and GNSS ground survey data from 2016 to 2022 to estimate the post-seismic rates of VLM across the Christchurch district. The results for Akaroa township are presented in Figure 3.7. These results are at a much higher resolution that the 2 km intervals of the NZSeaRise, with all sites displaying post-seismic subsidence.

Figure 3.6 shows a high level of variability of post-seismic VLM along the shoreline sites, ranging from -0.96 mm/yr to -2.40 mm/yr, and that the coastal sites generally report a higher rate of subsidence than the more inland sites. The site on the Akaroa main wharf has a reported post-seismic subsidence rate of -2.01 mm/yr, which is very similar to the NZ SeaRise inter-seismic rate, therefore there would be no difference in the projected future RSLR than from applying the NZ SeaRise inter-seismic rate. However, this site displays a higher rate than other coastal sites in close proximity, with the average of coastal sites within 250 m of the wharf being -1.24 mm/yr. Applying this lower VLM rate into future RSLR projections would result in a small lowering of 0.038 m over 50 years and 0.076 m over 100 years from the NZ SeaRise projection.

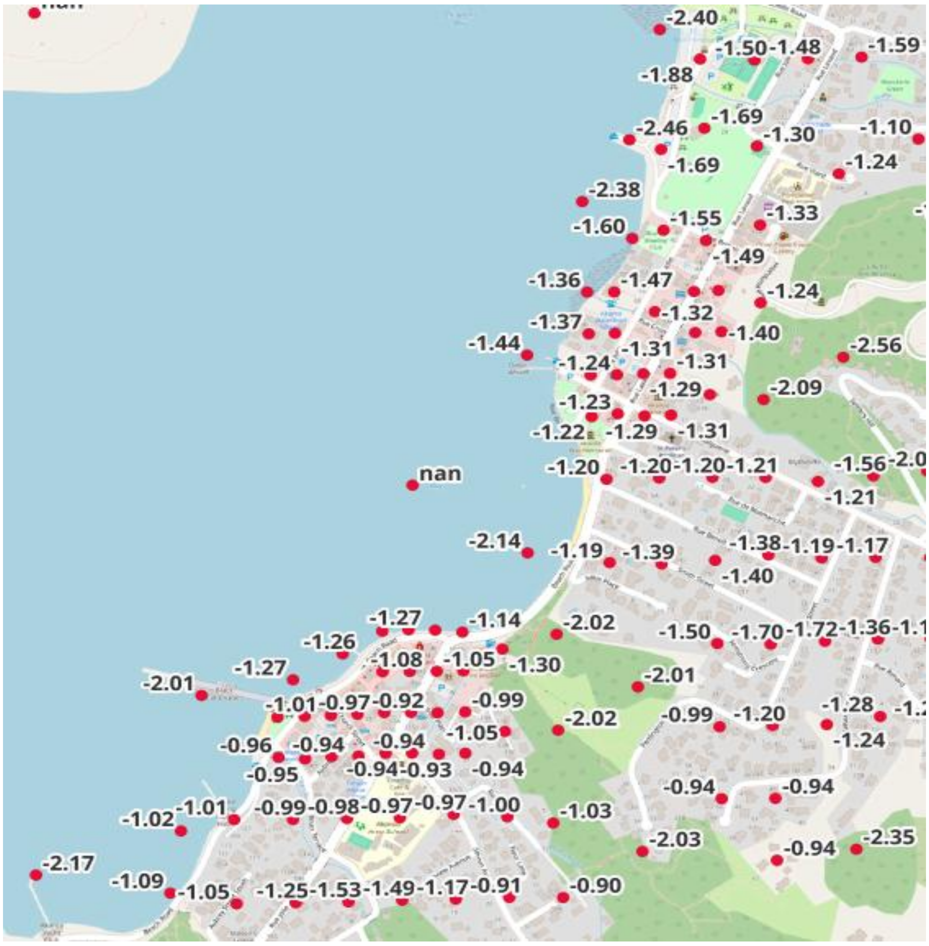


Figure 3.7: Post-seismic VLM rates (mm/yr) from GNS (2025) analysis

However, there is also some uncertainty about the length of time that these post-seismic rates may continue into the future. To address this issue, based on GNS advice, CCC have assumed that for asset management and flood management purposes, the post-seismic rates could continue for the next 30 years (e.g., up to 2050) (Simons-Smith pers com), following which inter-seismic rates will return. Under this assumption, the impact of lower post-seismic subsidence rate on RSLR would be a reduction of 0.023 m from the NZ SeaRise projections over both 50- and 100-year timeframes.

3.5.3.3 Projected RSLR

Given the general uncertainty about future VLM rates, and the reported very small differences in the inter-seismic and post-seismic rates around Akaroa Wharf, it is considered that using inter-seismic rates from the NZ SeaRise platform site 4450 are appropriate for the inclusion in projections of RSLR. The graphic of the resulting NZ SeaRise (2024) RSLR projections for site 4550 from a 1995-2014 (mid-point 2005) base date are presented in Figure 3.8, and then from a present day (2025) base date[32] in Table 3.7. Also presented in Table 3.7 is the increase from absolute SLR projections due to the inter-seismic VLM rate being projected to remain constant over time. As can be seem from the table, medium SLR projections (p50) are increased by 0.11 - 0.13 m by 2080 and 0.22 - 0.25 m by 2130 due to the inclusion of VLM.

[32] Involves reducing the NZ SeaRise RSLR values by the projected SLR from 2005 to 2025 (0.04-0.05m for p17, 0.13-0.135m for p50, 0.21-0.22m for p83)

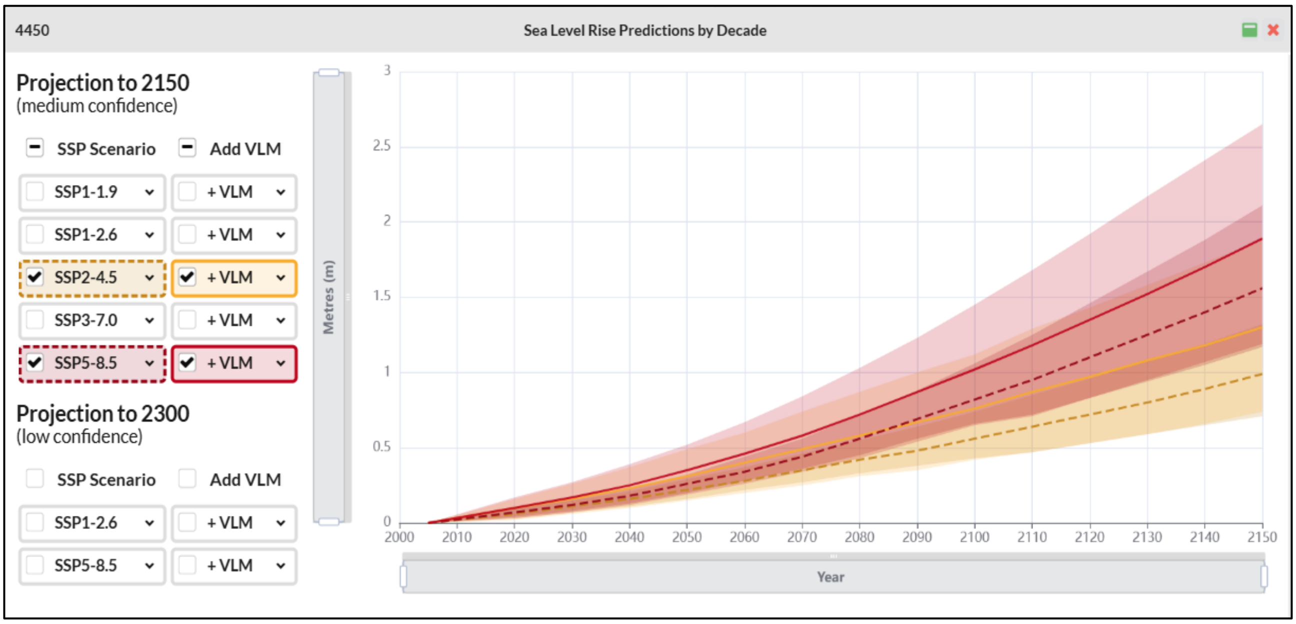


Figure 3.8: SLR projections from NZ SeaRise platform for Site 4450, located 400 m SW of Akaroa Wharf. Solid lines are RSLR projections including VLM at -2.0 mm/yr, and dashed lines are absolute SLR without VLM. Source NZ SeaRise platform (https://searise.takiwa.co/).

Table 3.7: RSLR projections for NZ SeaRise site 4550, located 400 SW of Akaroa Wharf with a linear VLM rate of -2 mm/yr from a 2025 base date. The figures in brackets are the magnitude of RSLR due to VLM. Source: Derived from NZ SeaRise (2024) and adjusted to 2025 base date.

Scenario	SSP2-4.5 Scenario			SSP5-8.5 Scenario		
Percentiles	p17	p50	p83	p17	p50	p83 (SSP5-8.5+)
2050	0.11 m (0)	0.12m (0.06)	0.17 m (0.11)	0.14 m (0)	0.21 m (0.04)	0.30 m (0.11)
2080	0.27 m (0)	0.45 m (0.13)	0.66 m (0.23 m)	0.38 m (0)	0.58 m (0.11)	0.81 m (0.23 m)
2100	0.38 m (0)	0.63 m (0.17)	0.91 m (0.29)	0.60 m (0)	0.88 m (0.15)	1.23 m[(4)] (0.30)
2130	0.55 m (0)	0.95 m (0.25)	1.37 (0.41)	0.90 m (0.01)	1.38 m (0.22)	1.95 (0.41)

Note:
(1) RSLR is from a 2025 base time.
(2) Figures in brackets are magnitude of rise due to inter-seismic VLM, taken as the difference between absolute SLR and RSLR given in NZ SeaRise
(3) The p83 projections for SSP5-8.5 are the MfE (2024) SSP5-8.5+ projections.
(4) The highlighted value is for the same projection and timeframe as given in Jacobs (2021) for use in determining the design height of the wharf deck.

The highlighted RSLR value in Table 3.7 (1.23 m by 2100 under SSP5-8.5+ scenario) is 0.19 m higher than the corresponding RSLR value given in Jacobs (2021) (i.e., 1.04 m by 2100) for inclusion in defining the design elevation of the wharf deck. This increase is entirely due to the inclusion of VLM in the estimate of RSLR. As discussed in Jacobs (2022) this will result in the extreme sea levels interacting with the proposed wharf deck elevation (3.06 m LVD37,12.10 m CDD) 10 - 15 years earlier than estimated in the Jacobs (2021) report. However, as also noted in the Jacobs (2022) report, there is a high level of uncertainty around both absolute SLR and VLM that occurs over these multi decadal to century time frames.

PDP (2024) debated whether the SLR component of the projected future extreme sea level for wharf design should be increased to 1.23 m to account for SSP5-8.5 projections (without VLM) to 2130, to align with MfE (2024) guidelines of 'Category C' developments (i.e., *"for existing coastal developments and asset planning"*). However, on the grounds of practicality of the wharf elevation in relation to the surrounding land elevation, the PDP report conclusion was to accept the 1.04 m SLR component and the 3.06 m LVD37 elevation for the wharf design.

I would also note that there are the following inconsistencies in the PDP argument:

1) The exclusive of VLM is inconsistent with the advice in MfE (2024)
2) The SLR magnitude used is from a base date from 2005, so should be reduced by 0.09 - 0.13 m to be from present day sea levels.
3) The Wharf could more correctly be considered to be a 'Category D' development under the MfE (2024) guidelines, being a *"Non-habitable, and functional need to be on the coast* (although it is not a "*short-lived asset*"), *which are either low consequences or readily adaptable*". Under this category, the recommended projected RSLR to applied in assessments is the SSP5-8.5 scenario to 2075. From Table 3.7, this would be in the order of 0.58 m (SSP5-8.5 (p50) by 2080).

3.5.4 Projected Future Extreme Sea Levels

Previous estimates of projected future sea level for design of the Wharf deck height were provided by Jacobs (2021, 2022) and PDP (2024). The Jacobs (2021) report combined a present day 100-year ARI storm tide level of 2.02 m LVD37 (from combining MHWS of 1.37 m with 0.6 m storm surge and 0.05 m wave set-up), with a 1.04 m RSLR (from a 2020 base date) to 2100 under a RCP8.5+ scenario to give a 100-year ARI extreme sea level for wharf design of 3.06 m LVD37 (12.10 m CDD). Jacobs (2022) recognised that due to joint probability analysis giving a higher 100-year ARI storm tide and the impact of VLM, would result in extreme sea levels being higher by 2100, therefore interaction of the design wharf desk with extreme sea levels would occur earlier than 2100.

Table 3.8 presents these updated projected future extreme sea levels from combining the present day extreme static sea levels given in Table 3.5, with the updated p50 RSLR projections for SSP2-4.5 and SSP5-8.5 for NZ SeaRise site 4550 given in Table 3.7 (e.g., adjusted to a 2025 base date).

Table 3.8: p50 Projected future extreme sea Levels with RSLR. Future values are the p50 values for each scenario from a 2025 base date.

	Frequency	MHWS		1 year ARI		10 year ARI		100 year ARI		
Time frame	Scenario	SSP2-4.5	SSP5-8.5	SSP2-4.5	SSP5-8.5	SSP2-4.5	SSP5-8.5	SSP2-4.5	SSP5-8.5	SSP5-8.5+ (1)
Present Day	NZVD2016 LVD37 CDD	1.05 1.42 10.46	1.05 1.42 10.46	1.60 1.97 11.01	1.60 1.97 11.01	1.74 2.11 11.15	1.74 2.11 11.15	1.87 2.24 11.28	1.87 2.24 11.28	1.87 2.24 11.28
2050	NZVD2016 LVD37 CDD	1.23 1.60 10.64	1.27 1.64 10.68	1.78 2.15 11.19	1.82 2.19 11.23	1.82 2.29 11.33	1.96 2.33 11.37	2.05 2.42 11.46	2.11 2.45 11.49	2.20 2.54 11.58

2080	NZVD2016 LVD37 CDD	1.50 1.87 10.91	1.64 2..01 11.15	2.05 2.42 11.46	2.19 2.56 11.60	2.19 2.56 11.60	2.33 2.70 11.74	2.32 2.69 11.73	2.45 2.82 11.86	2.68 3.05 12.09
2100	NZVD2016 LVD37 CDD	1.68 2.05 11.09	1.94 2.31 11.35	2.23 2.60 11.64	2.49 2.86 11.90	2.37 2.74 11.78	2.63 3.00 12.04	2.50 2.87 11.91	2.75 3.12 12.16	3.10 (2) 3.47 12.51
2130	NZVD2016 LVD37 CDD	2.00 2.37 11.41	2.44 2.81 11.85	2.55 2.92 11.96	2.99 3.36 12.40	2.69 3.06 12.10	3.13 3.50 12.54	2.82 3.19 12.23	3.25 3.62 12.66	3.82 4.19 13.23

Note:
(1) The SSP5-8.5+ scenario is the p83 values for SSP5-8.5 scenario.
(2) The highlighted values relate to the same sea level frequency and timeframe for RSLR as presented in Jacobs (2021) for the concept design of wharf deck.

The results presented in Table 3.8 highlights the change in frequency of current day extreme water levels with time due to SLR. For example, the current day 100-year ARI extreme sea level could be expected to occur on a monthly frequency by 2100 under the "high end" SSP5-8.5 RSLR scenario, and by 2130 under the "middle of the road" SSP2-4.5 RSLR scenario.

As can be seen from the highlighted values in the table, the updated extreme 100-year ARI (1% AEP) sea level by 2100 under the SSP5-8.5+ scenario by 2100 is 0.41 m higher than the corresponding value given in Jacobs (2021) (and agreed with by PDP (2024)) as the basis of the design for the height of the deck of the wharf. This difference is due to 0.22 m higher estimate of extreme storm tide in the updated Tonkin &Taylor (2022) joint probability sea level, and 0.19 m higher RSLR from the inclusion of VLM.

In terms of the design elevation of the wharf deck (3.06 m LVD37, 12.10 m CCD), the results in Table 3.8 suggest that interaction with extreme water levels would most likely occur (e.g., range of p17 - p83 levels) in a 100-year ARI event sometime between 2080 and 2100 under the SSP5-8.5 RSLR scenario, and sometime after 2100 under the SSP2-4.5 scenario. By 2130 under the SSP5-8.5 SLR scenario, this interaction is projected to occur in a 1-year ARI event, and in a 10-year ARI event under the SSP2-4.5 scenario. In relation to projected MHWS elevations in 2130, the design wharf deck elevation would have 0.25 m freeboard under the SSP5-8.5 RSLR scenario and 0.69 m freeboard under the SSP2-4.5 scenario.

3.6 Sediment Transport

Coastal sediment transport is dependent on the strength of ocean, tidal and wave currents, and breaking waves being sufficient to move the sediments found at the site of interest. No quantitative information on sediment transport has been sourced for the area around the Akaroa wharf. However, although the seabed surface sediments around the wharf are predominantly silts and clays, it is considered that there is a low energy sediment transport environment due to the lack of significant tidal and ocean currents and the low energy local wave environment. Maximum breaking waves are likely to break in less than 1 m water depths, so very close to the shore, and the depth of closure for beach sediment transport is likely to be limited to less than 5 m.

3.7 Summary of Key Coastal Processes for Akaroa Wharf Design

The key coastal processes relevant for the Akaroa wharf design are the wave climate, extreme sea level distribution, and the impact on these of future relative sea level rise and climate change.

A 31-year wave hindcast for a site within 100 m from the Akaroa Wharf (MetOcean solutions 2023) revealed that the wave climate is dominated by low energy waves. The dominant approach direction is from the WSW (50% of all waves), being from both direct wind waves blowing across the harbour and refracted southerly waves from winds blowing up the harbour, which is also the direction of the largest waves (Maximum Hs=0.73

m). The second most dominant wave direction is North – NNE (30% of waves), which are generally smaller due to short fetch length), with maximum Hs being 0.4 - 0.5 m. Wave approach from the NW is less frequent (14%), and have a maximum Hs of 0.6 - 0.7 m. Over the 31 year hindcast, 65% of the waves being ≤ 0.2 m, only 1% being > 0.5 m.

In terms of extreme waves frequencies, the hindcast statistics revealed that the 100-year ARI (1% AEP) Hs is in the order of 0.7 - 0.8 m for waves approaching the wharf from directions of SW around to NW. These wave heights are lower than those reported in Tonkin & Taylor (2021) for Akaroa Harbour in general, and lower than those recommended in the Jacobs (2019) and PDP (2024) reports, calculated from either recorded maximum daily or max gust wind speeds. However, maximum individual wave heights were estimated by the hindcast to be up to 1.3 m to 1.4 m in a 100-year ARI event.

A conservative estimate of wave set-up from these extreme waves is 0.05 m due to the low energy wave environment and flat nearshore slopes. This is up to five times less than the wave set-up estimated by Tonkin & Taylor (2021). Extreme wave run-up at the northern end of Akaroa township was calculated by DTec (2008) as being in the order of 0.25 m for natural beaches, 1.2 m on sloping rock revetments, and 1.7 m on vertical seawalls. These are most likely conservatively high estimates due to using a 1.5 m maximum wave height.

Storm tides and extreme sea levels (referred to as extreme static sea levels) are the combination of astronomical tides, storm surge, variations in the mean level of the sea due to climate cycles, and wave set-up. Estimates of current day extreme sea levels by Tonkin & Taylor (2021) were found to be flawed and were corrected based in the levels given in Tonkin & Taylor (2022) and wave set-up of 0.05 m to be 1.97 m, 2.11 m and 2.24 m LVD37 for 1-year, 10-year, and 100-year ARI events.

Future RSLR due to a combination of global warming and VLM will increase the elevation of future extreme sea levels, so need to be accounted for in the wharf design. However, there are large uncertainties in both these components of RSLR, which increase with time. Applying a 2mm/yr VLM rate from VLM sites most relevant to the Akaroa wharf to the medium value (p50) projected SLR due to global warming gives a total RSLR from present day (e.g., 2025) to 2100 of 0.63 m under the “middle of the road” SSP2-4.5 scenario, 0.88 m under the ‘upper end” SSP5-8.5 scenario, and 1.23 m under the most extreme SSP5-8.5+ scenario.

The resulting projected 1-year, 10-year, and 100-year ARI extreme sea levels in 2100 are 2.60 m, 2.74 m, and 2.87 m (LVD37) under the SSP2-4.5 scenario, and 2.86 m, 3.00 m and 3.13 m under the SSP5-8.5 m scenario. However, as pointed out above, there are large degrees of uncertainty with these projections, with likely range (i.e., p17-p83 values) for the 2100 SSP5-8.5 value being ± 0.35 m (e.g., 2100 RSLR under SSP5-8.5+ scenario is projected to be 3.47 m LVD37).

4. Assessment of current and projected future coastal hazards risks

Consideration of the current and projected future coastal hazards with climate change under the current wharf and seawall arrangements are required as a baseline to assess whether the proposed new wharf design will exacerbate these risks as required under Chapter 9 of the Regional Coastal Environment Plan (RCEP) (Coastal Hazards chapter). The following discussion presents this baseline position, and the assessment of any change in hazard risk is covered in section 5.3. Note that risk is the combination of exposure to the hazard (frequency, spatial extent, timing when will occur) with the vulnerability of the asset or land use to the hazard.

4.1 Coastal inundation hazards

Coastal inundation occurs when extreme sea levels overtop the elevation of the nature beach or coastal defence structures such that the land behind is flooded. This overtopping occurs in two forms;

1. By static water levels such that overtopping occurs continuously over various time periods around high tides (referred to as 'green water inundation'), and
2. By splash and spray from wave run-up that occurs only intermittently (referred to as 'blue water inundation') mainly around high tides, but likely to occur for longer each tide than green water inundation, although will involve less volume.

The shoreline in the vicinity of the Akaroa wharf is totally comprised of vertical seawalls, which promote high run-up elevations from waves breaking against the walls, so dynamic inundation is a large component of the coastal inundation hazard.

The CCC coastal hazards online portal presents maps of projected coastal inundation extent and depth bands with 0.2 m intervals of RSRL based on bathtub mapping of the extreme static sea level data from 1-year, 10-year and 100-year ARI events presented in Tonkin & Taylor (2021). However, as indicated in section 3.5 above, these extreme water levels are considered to be up to 0.4 m too high for 100-year ARI events due to these levels applying the tidal level correction from Sumner rather than from Lyttleton and the use of an over conservative estimate of wave set-up for the low wave energy environment at Akaroa. As a result, it is considered that the extreme static water levels presented in Table 3.8 provide a more appropriate estimation of the likelihood of coastal inundation hazards around the Akaroa Wharf.

Ground levels at the landward end of the existing Akaroa wharf are in the order of 11.5 m CDD (2.46 m LVD37), with the adjacent seawalls and land behind having a similar elevation. Therefore, green water coastal inundation is projected to occur whenever the static water levels are projected to exceed this elevation, and blue water inundation (splash and spray) when run-up against the seawalls exceeds this elevation.

Applying the static water levels from Table 3.8, Table 3.9 presents indicative "green" overtopping depths and the run-up elevation required for" blue water" inundation for the different combinations of frequency of extreme water level and RSLR scenarios. The results indicate that splash and spray from storm wave run-up against the vertical sea walls likely occurs in present day high spring tide water levels, which is constant with anecdotal observations. It is also most likely that this will increase in frequency, length of time, and volume with increasing water levels associated with more severe storms and SLR under both climate change scenarios through to 2050.

In contrast, "green water" inundation from static water levels at shore overtopping the sea walls is projected to only begin to occur in a rare event (e.g., 1–100-year ARI) under the high end (SSP5-8.5) SLR scenario by 2050. With 2080 projected SLR, "green water inundation" is indicated to occur in a 10-year ARI storm events under both SLR scenarios, with sea wall overtopping depths at peak tide possibly being up to 0.35 m in rare 100-year ARI storm events. In these conditions, "blue water" run-up inundation would be occurring sooner in the tide cycle, therefore occurring for longer time periods, adding to the inundation depths.

With 2100 projected SLR, without any changes to the seawall levels, “green water inundation” is indicated to occur in a more frequent 1-year ARI storm events under both SLR scenarios, with inundation depths being projected to be in the order of 0.5 m in a rare 100-year ARI event under both SLR scenarios. By 2130, high spring tides are projected to overtop the current sea wall elevations under the high end SSP5-8.5 SLR scenario, and be very close to overtopping (e.g., within 0.1 m) under the lower SSP2-4.5 scenario.

Table 3.9: Indicative maximum overtopping depths of “Green water” inundation and run-up required for “Blue water” inundation adjacent to the Akaroa wharf with an assumed elevation of 11.5 m CDD.

	Frequency	MHWS		1 year ARI		10 year ARI		100 year ARI	
Time frame	Scenario	SSP2-4.5	SSP5-8.5	SSP2-4.5	SSP5-8.5	SSP2-4.5	SSP5-8.5	SSP2-4.5	SSP5-8.5
Current Day	Green water	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil
	Blue water run-up required	1.0	1.0	0.5	0.5	0.35	0.35	0.2	0.2
2050	Green water	Nil	Nil	Nil	Nil	Nil	Nil	Nil	<0.1
	Blue water run-up required	0.85	0.8	0.3	0.25	0.15	0.1	0.05	
2080	Green water	Nil	Nil	Nil	0.1	0.1	0.25	0.25	0.35
	Blue water run-up required	0.6	0.35	0.05					
2100	Green water	Nil	Nil	0.15	0.4	0.3	0.55	0.4	0.65
	Blue water run-up required	0.4	0.15						
2130	Green water	Nil	0.35	0.45	0.9	0.6	1.05	0.75	1.15
	Blue water run-up required	0.1							

Any further assessment of inundation hazards at Akaroa township, and possible migration or adaptation options would occur under the Christchurch coastal adaptation programme[33].

4.2 Coastal erosion hazards

Tonkin & Taylor (2021) in mapping coastal erosion hazards within the Christchurch District, classified the seawalls along Akaroa township as “Class 1 structures” where any failure would be promptly repaired, such that hazard area for all time frames is set equivalent to the area affected by a structure failure and does not increase with future projected SLR. This hazard area is a function of the structure height and stable angle of repose for the filled material. For the Akaroa township, this erosion hazard distance in any such seawall failure was estimated as being most likely in the order of 4 m and very unlikely to exceed 5 m. In the scenario of the seawalls not being repaired, it was considered most likely that “*the fill material would rapidly erode and the shoreline will eventually move back towards its ‘original’ natural position*", but this scenario was not modelled.

Any further assessment of seawall failure and resulting erosion effects would occur under the Christchurch coastal adaptation programme.

[33] Christchurch coastal adaptation programme is a CCC programme about planning now for the effects of future coastal hazards with climate change on the city’s coastal communities. The programme was initiated in 2020 and will run for a number of years, working progressively across different coastal units within the city. It is unclear when adaptation planning within the Akaroa Harbour unit will begin.

4.3 Tsunami hazards

As set out in DTec (2008), there are references to the effects of tsunami in Akaroa Harbour in the major historical tsunami events of August 1868, May 1877, and May 1960, all of which were far-field tsunamis generated off the coast of Chile or Peru. Although the peak of these tsunami did not coincide with high tides, these accounts indicate that inundation of low-lying coastal land can occur during large events, particularly at Akaroa township where there are significant areas of low-lying reclaimed land, and the bays in the head of the harbour.

Lane et al (2014)[34] presents the results of a numerical tsunami modelling study of inundation in Canterbury resulting from a Mw 9.485 earthquake originating in the subduction zone off Peru, which represents a 2500-year return period event for wave heights at the Christchurch coast. The modelling assumed that the arrival of the largest tsunami wave coincided with MHWS tide. At Akaroa township, the modelled tsunami wave height was in the order of 4 m above water level and all low-lying land was projected to be inundated including the main street, with depths greater that 2.5 m in places. Maximum tsunami velocity in the harbour was modelled as in the order of 3 m/s, but less than 1m/s close to shore at Akaroa township.

[34] Lane et al (2014) Updated inundation modelling in Canterbury from a South American tsunami. Report for Environment Canterbury

5. Assessment of potential effects on coastal processes and hazards

Assessment of the potential effects of the proposed wharf construction and operation needs to be assessed against the relevant coastal processes and coastal hazard provisions of the NZCPS (2010) and the Canterbury Regional Coastal Environment Plan (RCEP).

Relevant policies of the NZCPS (2010) include:

- Policy 25: Subdivision, use, and development in areas of coastal hazard risks; and
- Policy 27 Strategies for protecting significant existing development from coastal hazard risk.

However, many of the clauses of these policies are not relevant as the new wharf needs to be in a coastal location that is exposed to coastal hazards (that is, removing the structure away from hazard risk is not practicable given the wharf function – Policy 25(d)), therefore wharf design needs to accommodate coastal processes such that hazards are avoided or reduced as far as practical.

Similarly, many of the coastal hazard provisions of the Canterbury RCEP are not relevant due to the need for the wharf to have a coastal location, with the relevant policy being:

- Policy 9.1(c) The continued use and protection of essential infrastructure and services should be provided for, where no reasonable alternative exists, in areas subject to coastal hazards, provided adverse effects on the coastal environment are avoided, remedied, or mitigated.

Both the policies of the NZCS (2010) and the Canterbury RCEP state that the consideration of coastal processes and hazards should include the effects of climate change particularly SLR, over a 100-year time frame. However, these policies do not specify what frequency of extreme sea levels should be considered in combination with RSLR.

5.1 Effects of coastal processes on proposed wharf design

The key components of the coastal processes to take account of in the wharf design are future extreme sea levels with RSLR, wave height and energies, and sediment transport around piles.

5.1.1 Extreme sea levels

The proposed wharf deck, with an elevation of 3.06 m LVD37 or 12.10 m CDD, in the order of 0.5 m above the current wharf level, is designed to account for future RSLR, while still being practical in relation to adjacent land levels (including Beach Road), for which any decision on how and what climate change adaptation may occur are still some years away.

Note that the following discussion relates to the deck elevation of the new wharf, and not the existing wharf buildings which will remain on their own existing piles and will have floor level elevations in the order of 0.5 m lower than the new wharf deck. Therefore, these buildings will be exposed to extreme water levels earlier in time, and more frequently, than the deck of the new wharf.

The new wharf design elevation is above the projected MHWS level by 2130 (taken as a 100-year period) with a 0.25 m freeboard even under the "high end" SSP5-8.5 RSLR scenario, with most likely future extreme sea levels (p50 values presented in Table 3.8) in a rare 100-year event projected to only start to interact with the wharf deck by 2100 (i.e. 75 years from present). Under this projection, interaction between extreme sea levels and the design wharf deck at this time would be limited in depth to being in the order of 0.07 m and would be limited in duration to a short time period at the peak of high tide in such an event. Under this RSLR scenario, by 2130 interaction between extreme sea levels and the wharf deck would increase in frequency to be likely to be occurring more frequently than once a year.

Under a worst case RSLR projection (e.g., p83 sea level projections (SSP5-8.5+)), this interaction in a rare 100-year ARI event could begin in 2080 (i.e. just over 50 years from present) and be occurring at around an annual frequency by 2100.

Conversely, with the lower rate of RSLR scenario under the "middle of the road" SSP2-4.5 scenario, the projected most likely future extreme sea levels in a rare 100-year ARI event would only begin to interact with the design deck elevation sometime between 2100-2130, and not until 2130 in an occasional 10-year ARI event.

In relation to the MfE (2024) guidelines on the recommended magnitude of RSLR to apply for coastal planning and policy for various categories of development, the new wharf could fall somewhere between Categories C and D, but as a non-habitable asset with a functional need to be located at the coast being more fitting with Category D, for which RSLR projections under the SSP5-8.5 scenario to 2130 and 2075 are recommended respectively. Applying the projected RSLR levels to the extreme sea levels frequencies at these at time frames, gives the following results:

- Category C – *land use controls for existing coastal uses and assets, for which the recommended RSLR scenario for assessment is the medium confidence SSP5-8.5 scenario out to 2130*.– Under this scenario and timeframe interactions between the currently proposed wharf deck elevation and extreme sea levels would be occurring more frequently that once a year. To achieve compliance with the recommendation under a 100-year ARI extreme sea level (e.g. 1% AEP at this magnitude of RSLR) would require the wharf deck to be raised more than 0.5 m from current design (e.g. to 3.62 m LVD37, 12.66 m CDD). This would result in the new wharf being in the order of 1 m above the current ground level, which we would consider to be unjustified in light of the high level of uncertainty with RSLR projections, the low probability of occurrence, the short duration of any interaction at high tide, and we understand the impractically of integration with the existing land levels.
- Category D – *Non-habitable, short-lived assets with a functional need to be at the coast, which are either low consequences or readily adaptable, for which the recommended RSLR scenario for assessment is the medium confidence SSP5-8.5 scenario out to* 2075. – Under this scenario the currently proposed wharf deck elevation would only begin to interact with extreme sea levels within this time frame under the upper limit of most likely (i.e. p83, or SSP5-8.5+ scenario) 100-year ARI water levels (e.g. 1% AEP. Therefore, the current proposed wharf deck elevation would comply with the MfE recommendations.

In conclusion, the above results indicate that some degree of interaction of the proposed wharf deck elevation with extreme sea levels are likely to occur before 100 years with RSLR. However, even in the most extreme sea level events (e.g. 100-year ARI or 1% AEP events) and most extreme RSLR (SSP5-8.5+) this is not projected to occur until at least 2080, and most likely not till 2100 under high end (SSP5-8.5) RSLR scenarios or until 2130 under middle of the road (SSP2-4.5) RSLR scenarios. In considering potential effects on the wharf structure of any such interactions, the low frequency and short duration of occurrence needs to be taken into account. It is assumed that the structural design of the wharf will account for any such interactions.

In terms of health and safety concerns for wharf users during extreme sea levels, it is noted that these are not projected to become an issue to at least 2080. Although this is well beyond the maximum 35 year term of a Coastal Permit for the wharf, these concerns could be mitigated by a consent condition requiring the development of a wharf closure plan during storm events. However, such a condition is considered unnecessary under this consent, and would be more relevant for any reconsenting of the wharf structure beyond the initial 35 year term. The key parameter to be considered in such a plan would be the forecast extreme sea level at which the plan is implemented, which would need to take account of predicted tide height, and forecast storm surge and wave heights.

5.1.2 Other coastal processes

It is assumed that the structural design of wharf and floating pontoons is sufficient to accommodate any wave impacts on the structures. It is also assumed that the low magnitude sediment transport regime would not result in any effects on design of the wharf piles.

5.2 Effects of new wharf structure on coastal processes

There is the potential of the components of the wharf structure to alter coastal processes such as waves by diffraction, wave run-up from the replacement of the existing abutment reclamation with a seawall, and seabed scour around the wharf piles.

5.2.1 Wave processes

Wave diffraction is the altering of wave approach direction and heights from interaction of structures with the wave train. This can result in the focussing or reduction of wave energy around the structure, leading to increased or decreased sediment transport and shoreline response to wave action. Wave diffraction could occur off the wharf piles or the wharf deck, and wave refraction off the new concrete “L-Wall” seawall and rip rap structure.

It is considered that the effects on wave processes will be similar to the minimal effects experienced from the existing wharf and abutment. Wave diffraction will be minimal and very localised from the wharf piles, so will have a less than minor impact on passage of waves to the shore. Although the wharf deck may interact with extreme storm waves under future (50 years+) RSLR scenarios, this interaction will be limited to the surface part of the wave rather than the whole water column, so would trend to result in a confused sea state around the wharf from reflection of wave crests rather than wave diffraction. It is considered that this would have minimal effect on waves arriving at shore. Since the proposed new “L-Wall” has the same alignment to the existing seawalls along the wharf abutment, it is anticipated that the wave refraction from the structure will be similar to the existing seawalls, and potentially reduced due to presence of the 4-6 m of rip rap in of the wall breaking up into wave energies and prompting wave break prior to striking the wall. The removal of part of existing abutment and associated reclamation is also not anticipated to alter any wave breaking and run-up processes from the existing environment.

5.2.2 Beach Processes

In line with the above assessment that there will not be any increase in wave run-up or energy levels from the new wharf and associated works, it is anticipated that there will be no increase in the current magnitude of beach scour or erosion as a result of these works. It is also likely that the placement of the 4-6 m rip-rap protection along the existing and new seawalls around the wharf abutment will reduce the potential for beach scour in these areas. However, details of this placement are only shown for a cross-section at the location of the new “L-Wall, so it is unknown how this is structure relates to the existing seawalls.

Based on the concept design presented in Appendix A, the partial removal of the wharf abutment and associated reclamation will not result in any significant change to the shoreline configuration in the vicinity of the wharf, therefore will continue to act as a barrier to longshore transport of beach sediments across the wharf footprint. However, sediment transport volumes are low, the barrier effect will not be any different to current conditions, and the local beaches will continue to be protected by the current seawall arrangements.

5.2.3 Seabed scour around wharf piles

Scour from water movements around piles is a common occurrence for wharves constructed on soft sediment coasts. Although no scour assessment is included in the Geotechnical reports sighted or carried out as part of this assessment, it is considered that any scour will be the same as occurs with the existing wharf piles (e.g. no issue over more than 100 years) and is assumed that the combination of low wave activity and deep wharf piles to be founded in basalt rocks would remove an issue with potential future pile scour. However, if this was found to be an issue in the future, this could be addressed by standard scour protection means (e.g. rock placed around the piles).

5.3 Effects of new wharf on coastal hazards

There is a need to consider whether the design and placement of the new wharf would exacerbate existing coastal hazards or lead to adverse effects from natural hazards on any other property. This can occur by the new structure altering the coastal processes that drive the magnitude and frequency of the hazards.

As established in Section 4, existing and future coastal inundation hazards are present at the wharf site, and that erosion may occur if existing (and proposed) seawalls fail in the vicinity of the wharf. However, since the new wharf will not alter the wave climate and extreme sea levels level, or wave run-up and sediment transport processes, it is considered that the new structure will not exacerbate any of these existing or likely future hazards.

The new wharf is also exposed to tsunami hazard and would be overtopped in the modelled 2500 year return period event at all tidal levels to the same extent as the hinterland in the vicinity of the wharf.

5.4 Construction effects

Potential construction effects on coastal processes, including the demolition of the existing wharf, are largely restricted to disturbance of the seabed in demolition and piling.

The construction methodology summarised in section 2.2.2 indicates that all demolition material including from the abutment reclamation will be removed to an onshore location, and existing piles will removed or cut back to seabed level, and that re-piling will be done from either a barge or off the wharf.

There is no indication that any machinery will be placed on the seabed during construction, and that no dredging around the wharf footprint will be required in construction.

Assuming that the stated demolition and construction methodology is followed, it is considered that any construction effects on coastal processes will be minor and temporary.

5.5 Temporary Loading Ramp

It is considered that the construction and operation of the temporary loading ramp on the southern side of the Akaroa Ramp will only have a minor and temporary effect on the local coastal processes in the immediately vicinity of the ramp. These effects will largely be associated with dredging of the berth pocket at the ramp to allow the designated construction vessel to be loaded. The dredge footprint is small (2,700 m^2), volumes are low (1500 m^3) and dredge depths shallow (max 1.5 m). This scale of change to the nearshore topography will have little impact on the local wave climate arriving at the shoreline in the vicinity of the boat ramp.

It is anticipated that only one dredging campaign will be undertaken at the start up of the project, and that the dredge depths will be sufficient to vessel operation over the timeframe of the wharf construction (maximum 14 months). It is also noted that dredging of the access channel to the boat ramp is an existing activity, and placement of dredge spoil will be in an area already used for this purpose. Given that repeat dredging of the access channel to the boat ramp has been required in the past, it is anticipated that berth pocket at the loading ramp will progressively infill with sand over time, hence will be a temporary feature of the bathymetry.

Given the placement of the loading ramp on the beach and the formation of a small training wall is adjacent to the existing boat ramp that already influences longshore sediment transport and shoreline responses, it is considered that any effects of this additional structure will be indistinguishable from the minor effects that already exist.

5.6 Recommended mitigation measures

From the above assessment, it is considered that the only potential effects of the proposed new wharf design on coastal processes is the projected interaction of the wharf deck with extreme sea levels in rare storm events with SLR at some time after a minimum period of 50 years, most likely closer to 80 years, and possibility as long as 100 years under “middle of the road” sea level rise scenarios.

It is assumed that the structural design of the new wharf will account for any such temporary short duration interactions so that they do not result in any structural damage to the new wharf.

In terms of health and safety concerns for wharf users during extreme sea levels, it is noted that these are not projected to occur to beyond the maximum 35-year term of a Coastal Permit for the wharf. However, as indicated in section 5.1.1, these concerns could be mitigated by a consent condition requiring the development of a wharf closure plan during storm events; but are considered unnecessary under this consent. However, they may be relevant for any reconsenting of the wharf structure beyond the initial 35 year term. Were the consenting authority to require a condition to be affixed to any initial term of consent, the key parameter to be considered in such a plan would be the forecast extreme sea level, which would need to take account of predicted tide height, and forecast storm surge and wave heights. Any such wharf closure plan could also include tsunami events.

Consent conditions should also re-enforce the good practice demolition and construction methodology set out in the Holmes (2024) design report and Enviser (2025) project description.

6. Summary

6.1 Coastal Processes

The key coastal processes relevant for the Akaroa wharf design are the wave climate, extreme sea level distribution, and the impact on these of future relative sea level rise and climate change.

Wave hindcast analysis revealed that the wave climate is dominated by low energy waves, with 65% of the waves over the 31-year hindcast period being ≤ 0.2 m, only 1% being > 0.5 m. Rare 100-year ARI (1% AEP) Hs heights were found to be in the order of 0.7 - 0.8 m for waves approaching the wharf from directions of SW around to NW. These wave heights are lower than those reported in Tonkin & Taylor (2021) for Akaroa Harbour in general, and lower than those recommended in the Jacobs (2019) and PDP (2024) reports, calculated from either recorded maximum daily or max gust wind speeds. However, maximum individual wave heights were estimated from the hindcast to be up to 1.3 m to 1.4 m in a 100-year ARI event. A conservative estimate of nearshore wave set-up at the wharf from these extreme waves is 0.05 m due to the low energy wave environment and flat nearshore slopes. This is up to five times less than the wave set-up estimated by Tonkin & Taylor (2021). Extreme wave run-up against the vertical seawalls at Akaroa was calculated by DTec (2008) as being in the order of 1.7 m. These are most likely conservatively high estimates due to using a 1.5 m maximum wave height.

Extreme static sea levels are the combination of astronomical tides, storm surge, variations in the mean level of the sea due to climate cycles, and wave set-up. Estimates of current day extreme sea levels by Tonkin & Taylor (2021) were found to be flawed and were corrected based in the levels given in Tonkin & Taylor (2022) and wave set-up of 0.05 m to be 1.97 m, 2.11 m and 2.24 m LVD37 for 1-year, 10-year, and 100-year ARI events.

Future RSLR due to a combination of global warming and VLM will increase the elevation of future extreme sea levels, so need to be accounted for in the wharf design. Total RSLR from present day (e.g. 2025) to 2100, including VLM of 2mm/yr, is estimated to 0.63 m under the "middle of the road" SSP2-4.5 scenario, 0.88 m under the 'upper end" SSP5-8.5 scenario, and 1.23 m under the most extreme SSP5-8.5+ scenario. However, there are large uncertainties in both these components of RSLR, which increase with time.

The resulting projected 1-year, 10-year, and 100-year ARI extreme sea levels in 2100 are 2.60 m, 2.74 m, and 2.87 m (LVD37) under the SSP2-4.5 scenario, and 2.86 m, 3.00 m and 3.12 m under the SSP5-8.5 m scenario. However, as pointed out above, there are large degrees of uncertainty with these projections, with likely range (i.e., p17-p83 values) for the 2100 SSP5-8.5 value being ± 0.35 m (e.g., 2100 100-year ARI sea level SSP5-8.5+ scenario is projected to be 3.47 m LVD37). This is 0.41 m higher than the corresponding value given in Jacobs (2021) as the basis of the design for the height of the deck of the wharf, with the difference being due to 0.22 m higher estimate of extreme storm tide in the updated Tonkin &Taylor (2022) joint probability sea level, and 0.19 m higher RSLR from the inclusion of VLM.

6.2 Current and future coastal hazards

The land around the wharf is already exposed to current coastal inundation hazards from wave overtopping due to the low elevation of existing seawalls (approx. 11.5 m CDD) in relation to extreme water levels and storm wave run-up elevations. This exposure will get more frequent in the future due to sea level rise.

In contrast, inundation from static water levels at shore overtopping the sea walls is projected to only begin to occur in a rare event (e.g., 1–100-year ARI) under the high end (SSP5-8.5) SLR scenario by 2050, and in a 10-year ARI storm events by 2080 under both SLR scenarios. By 2100, without any change to the seawall levels, static water inundation is indicated to occur in a more frequent 1-year ARI storm events under both SLR scenarios, and by 2130, high spring tides are projected to overtop the current sea wall elevations under the high end SSP5-8.5 SLR scenario, and be very close to overtopping (e.g., within 0.1 m) under the lower SSP2-4.5 scenario.

Coastal erosion is not predicted to be an issue adjacent to the wharf unless the current seawalls fail, with erosion distances under these circumstances being in the order of 4-5 m. Tonkin & Taylor (2021) considered that due to the status of these seawalls, any such failure would be rapidly repaired, such that the erosion exposure will not increase with time. However, in the scenario of the seawalls not being repaired, it was considered most likely that "*the fill material would rapidly erode and the shoreline will eventually move back towards its 'original' natural position*", but this scenario was not modelled.

The wharf is also exposed to tsunami hazards, with Akaroa township having a history of tsunami inundation in at least three former tsunamis generated off the coast of Chile and Peru. Modelling of an extreme 2500-year tsunami event indicated that tsunami wave height at Akaroa township could be in the order of 4 m above water level and all low-lying land coastal was projected to be inundated including the main street, with depths greater that 2.5 m in places.

6.3 Assessment of effects

The key components of the coastal processes to take account of in the new wharf design are future extreme sea levels with SLR, wave height and energies, and sediment transport around piles. The coastal hazard policies of the NZCS (2010) and the Canterbury RCEP state that the consideration of coastal processes and hazards should include the effects of climate change particularly SLR, over a 100-year time frame. However, these policies do not specify what frequency of extreme sea levels should be considered in combination with RSLR.

The proposed wharf deck, with an elevation of 3.06 m LVD37 or 12.10 m CDD, in the order of 0.5 m above the current wharf level, is designed to account for future RSLR, while still being practical in relation to adjacent land levels (including Beach Road), for which any decision on how and what climate change adaptation may occur are still some years away.

The design elevation is above the projected MHWS level by 2130 (taken as a 100-year period) with a 0.25 m freeboard even under the "high end" SSP5-8.5 RSLR scenario, with most likely future extreme sea levels in a rare 100-year event projected to only start to interact with the wharf deck by 2100 (i.e. 75 years from present). This RSLR scenario and timeframe are consistent with the recommended allowances for RSLR for non-habitable assets with a functional need to be located at the coast (e.g. Category D activities) in the MfE (2024) coastal hazard guidance. Under this projection, interaction between extreme sea levels and the design wharf deck at this time (e.g. 2100) would be limited in depth in the order of 0.07 m and would be limited in duration to a short time period at the peak of high tide in such an event. Under this RSLR scenario, by 2130 interaction between extreme sea levels and the wharf deck would increase in frequency to be likely to be occurring more frequently that once a year. Conversely, with the lower rate of RSLR scenario under the "middle of the road" SSP2-4.5 scenario, the projected most likely future extreme sea levels in a rare 100-year ARI event would only being to interact with the design deck elevation sometime between 2100-2130, and not until 2130 in an occasional 10-year ARI event.

The above assessment relates only to the deck elevation of the new wharf, and not the existing wharf buildings which will remain on their own existing piles and will have floor level elevations in the order of 0.5 m lower than the new wharf deck. Therefore, these buildings will be exposed to extreme water levels earlier in time, and more frequently, than the deck of the new wharf.

It is considered that the proposed new wharf structure will not have any adverse effect on any other coastal processes and will not exacerbate existing or projected coastal hazards.

Assuming that the stated demolition and construction methodology is followed, it is considered that any construction effects on coastal processes will be minor and temporary.

It is also considered that the construction of the temporary loading ramp adjacent to the Akaroa Boat ramp including the dredging 1500 m^3 of material to form of a berth pocket, will not have any distinguishable effects from the minor effects that already exist from the presence of the boat ramp.

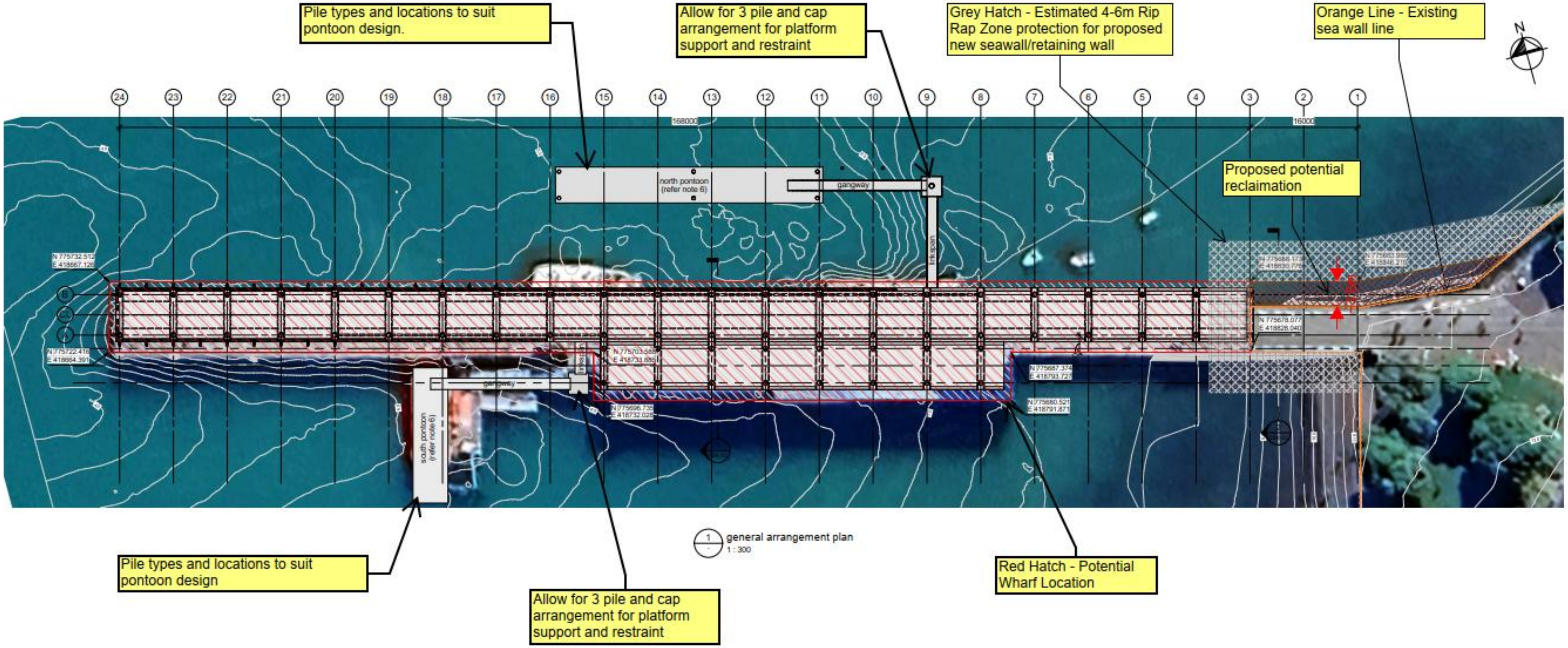
6.4 Recommended mitigation measures

It is assumed that the structural design of the new wharf will account for any temporary short duration interactions between the wharf deck and extreme sea levels in rare storm events, so that they do not result in any structural damage to the wharf. These interactions are not projected to begin to occur until sometime after 50 years under the worst case RSLR scenarios (e.g. SSP5-8.5+), most likely closer to 80 years (e.g. SSP5-8.5 scenario), and possibility as long as 100 years under "middle of the road" sea level rise scenarios (e.g. SSP2-4.5).

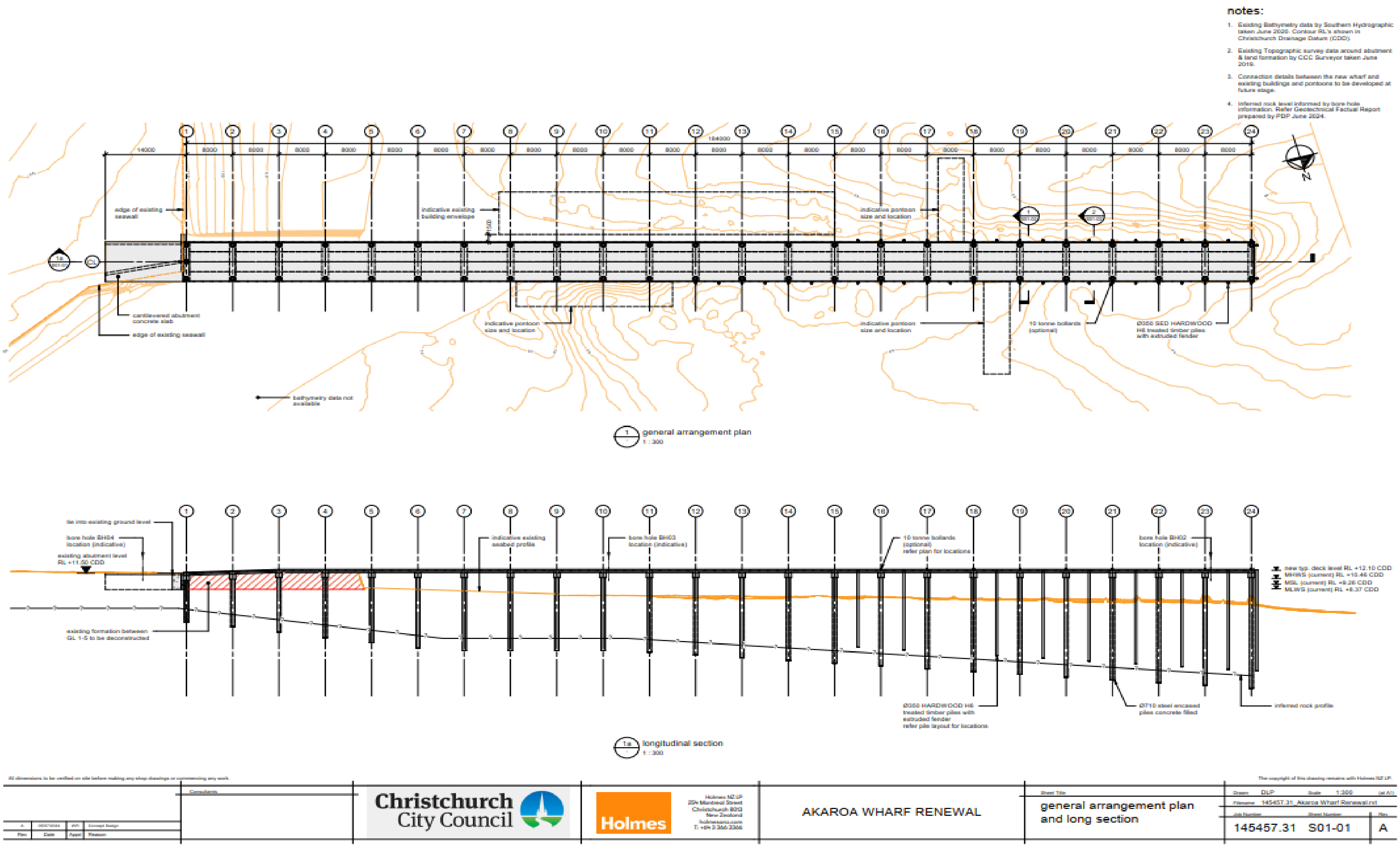
All of these time frames are beyond the maximum 35-year term of a Coastal Permit for the wharf. But, if required by the consent authority to mitigate concerns around the health and safety of wharf users during extreme sea levels, a consent condition requiring the development of a wharf closure plan to apply during storm events could be applied. However, any such condition would be more relevant for any reconsenting of the wharf structure beyond that the initial 35-year term. Any such closure plan could also include tsunami events.

Consent conditions should also re-enforce the good practice demolition and construction methodology set out in the Holmes (2024) design report and the Enviser (2025) project description.

Appendix A. Concept General Arrangements Plan and Long Section of New Wharf. (Sourced from Holmes (2024) Appendix F, and Ensiver (2025) Attachment A)



Pile types and locations to suit pontoon design.
Allow for 3 pile and cap arrangement for platform support and restraint
Grey Hatch - Estimated 4-6m Rip Rap Zone protection for proposed new seawall/retaining wall
Orange Line - Existing sea wall line
Proposed potential reclaimation
north pontoon (refer note 6)
gangway
south pontoon (refer note 6)
Pile types and locations to suit pontoon design
Allow for 3 pile and cap arrangement for platform support and restraint
Red Hatch - Potential Wharf Location
general arrangement plan
1 : 300



notes:
1. Existing Bathymetry data by Southern Hydrographic taken June 2020. Contour RL's shown in Christchurch Drainage Datum (CDD).
2. Existing Topographic survey data around abutment & land formation by CCC Surveyor taken June 2019.
3. Connection details between the new wharf and existing buildings and pontoons to be developed at future stage.
4. Inferred rock level informed by bore hole information. Refer Geotechnical Factual Report prepared by PDP June 2024.
edge of existing seawall
indicative existing building envelope
indicative pontoon size and location
cantilevered abutment concrete slab
10 tonne bollards (optional)
Ø350 SED HARDWOOD H6 treated timber piles with extruded fender
bathymetry data not available
general arrangement plan
1 : 300
tie into existing ground level
bore hole BH04 location (indicative)
existing abutment level RL +11.50 CDD
indicative existing seabed profile
bore hole BH03 location (indicative)
10 tonne bollards (optional) refer plan for locations
bore hole BH02 location (indicative)
new typ. deck level RL +12.10 CDD
MHWS (current) RL +10.46 CDD
MSL (current) RL +9.26 CDD
MLWS (current) RL +8.37 CDD
existing formation between GL 1-5 to be deconstructed
Ø350 HARDWOOD H6 treated timber piles with extruded fender refer pile layout for locations
Ø710 steel encased piles concrete filled
inferred rock profile
longitudinal section
1 : 300
Christchurch City Council
Holmes
AKAROA WHARF RENEWAL
general arrangement plan and long section
145457.31
S01-01
A

Appendix B. Concept Diagram of Temporary Berth Pocket at the Akaroa Boat Ramp. Source: Enviser (2025)