REPORT

Tonkin+Taylor

Coastal Hazard Assessment for Christchurch District

Technical Report

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Christchurch City Council
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Cover photo: The Christchurch District coastline viewed from the International Space Station in 2014 (Credit: ESA/A.Gerst CC BY-SA 2.0. ID: 2014 945_5391).

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Glossary of terms

Term	Description
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
ASCE	Area Susceptible to Coastal Erosion
AWS	Automatic Weather Station
Beach face slope	Beach slope around the extreme still water level (i.e. typically between 1 m and 4 m NZVD2016).
Bruun Rule	A simple mathematical relationship that states: as sea-level rises, the shoreface profile moves up and back while maintaining its original shape
CCC	Christchurch City Council
CD	Chart Datum
Class 1 structures/significantly modified shorelines	Shorelines which have been significantly modified with erosion protection structures
CES	Canterbury Earthquake Sequence (2010-2011)
CI	Confidence interval
Coastal accretion	A long-term trend of shoreline advance and/or gain of beach sediment volume
Coastal erosion	Landward movement of the shoreline which may include both long-term retreat over several years or decades and short-term loss of sediment due to storms
Coastal hazard	Where coastal processes adversely impact on something of value resulting in a hazard
Coastal inundation	Flooding of land by the sea.
DEM	Digital Elevation Model
DS	Dune stability component
ECan	Environment Canterbury
EWS	Electronic Weather Station
H_c	Height of bank or cliff
H_s	Significant wave height
LiDAR	Light Detection and Ranging – a method of remotely deriving land elevation, generally from an aeroplane
LT	Long-term erosion component
LT_H	Historical long-term erosion component
LT_F	Future long-term erosion component
LVD-37	Lyttelton Vertical Datum 1937
m	Sea level rise response factor for cliffs
MfE	Ministry for Environment
MHWS	Mean high water springs – a measure of high tide based on a statistical exceedance of high tides in a month
MHWPS	Mean high water perigean springs. A perigean spring tide is the highest spring tide and occurs three or four times per year when the moon is closest to the earth.

Term	Description
MLWS	Mean low water spring – a measure of low tide based on a statistical exceedance of low tides in a month
MSL	Mean sea level. Sea level averaged over a long (multi-year) period
NZVD2016	New Zealand Vertical Datum 2016
RCP Scenario	Representative Concentration Pathways (RCPs) are four greenhouse gas concentration trajectories adopted by the IPCC for its fifth Assessment Report (AR5) in 2014
RL	Reduced Level
SLR	Sea level rise. Trend of annual mean sea level over timescales of at least three or more decades. Must be tied to one of the following two types: global – overall rise in absolute sea level in the world's oceans; or relative – net rise relative to the local landmass (that may be subsiding or being uplifted)
SL	SLR component
SS	Slope stability allowance
ST	Short-term erosion component
Surfzone slope	Slope below the 1 m NZVD2016 contour offshore to where waves start breaking or to where data is available.
T+T	Tonkin + Taylor (Tonkin & Taylor Ltd)
VLM	Vertical land movements

Executive summary

Christchurch City Council (CCC) has engaged Tonkin & Taylor Ltd. (T+T) to undertake a coastal hazard assessment (CHA) for the entire Christchurch district.

The intended purpose of this assessment is to help inform the CCC Coastal Hazards Adaptation Planning (CHAP) programme. The scope of the assessment has been developed in conjunction with Council's CHAP project team and technical reviewer, who have confirmed that the methodology described in this report is suitable for this intended purpose. For more information about adaptation planning and how the outputs of this coastal hazard assessment will be used, refer to the cover letter "Coastal Hazards Assessment Methodology: Purpose and context" which accompanies this report on the CCC website.

This report (the "Technical Report") provides an in-depth explanation of the environmental data that was used for the hazard assessment, the methodology that was applied and the analysis results. It collates all the technical details together in one place to provide a self-contained record of the work for technical review and future reference. As such, of necessity, this report contains a large amount of information which is highly technical in nature.

A companion report (the "Summary Report") is also available alongside this report on the CCC website and will be the more relevant and engaging report for most people. The Summary Report provides a more straightforward description of how the hazard was identified and analysed, and the key findings for each part of the Christchurch coastline.

1 Introduction

1.1 Background

There are two key factors which have driven the need for the coastal hazard assessment presented in this report:

- In October 2019, Christchurch City Council (CCC) resolved to address earthquake legacy issues along the Avon-Heathcote Estuary edge and to develop a coastal hazards adaptation planning programme of work for all Christchurch District coastal environments.
- Updated information on sediment supply, ground levels, storm events, groundwater and high tide statistics has recently become available. This information has implications for the identification of areas susceptible to coastal hazards.

Therefore, to support sound adaptation planning discussions with coastal communities and ultimately robust and defensible decisions by the Council, T+T have been commissioned to undertake this coastal hazard assessment for the entire Christchurch District.

Since an updated technical assessment is being undertaken, this has provided an opportunity to also incorporate the following:

- More recent topographic data and longer datasets of beach profiles, water level information and wave climates
- Suggestions from the 2016 Peer Review (Kenderdine et al. 2016) of the 2015 Coastal Hazards Assessment that were not able to be included in the previous 2017 assessment
- Additional scenarios and outputs designed for engagement and adaptation
- Wider geographic scope to cover the entire Christchurch District coastline (including the entire Banks Peninsula coastline)
- Ensure consistency of hazard identification with national-level guidance released since the previous assessment such as the 2017 Ministry for the Environment Coastal Hazards and Climate Change Guidance

1.2 Purpose of this coastal hazard assessment

The purpose of this assessment is to provide CCC with specialist technical coastal hazard (inundation, erosion and flood depth) and associated groundwater information, with the primary objective of presenting this information for public use in a format that is easily accessible, comprehensive and unambiguous. The focus of following technical report is to produce the “raw” hazard information. This information can then feed into engagement, risk evaluation and risk mitigation and adaptation planning undertaken by CCC in future.

The following assessment supersedes the previous coastal hazard assessments for the area undertaken by T+T between 2015 and 2018.

The primary intended purpose of the updated coastal hazard and groundwater information is to help inform coastal hazards adaptation planning for Christchurch District. The results of the assessment could also inform a range of other purposes, provided the uncertainties and limitations are understood and appropriately managed. These other uses might include review of the coastal hazards provisions in the Christchurch District Plan, infrastructure planning decisions, consenting applications and Civil Defence Emergency Management. In many cases, the results of this assessment may provide an initial hazard screening for these other purposes, with more detailed analysis then undertaken for specific locations and scenarios of interest.

It is important to note this assessment is not intended to map out a hazard overlay for inclusion in the District Plan, but provides information about hazards (and the uncertainty in our understanding of those hazards), which may be subject to further analysis and consultation to eventually determine if and where a hazard overlay should apply.

The assessment area covers the entire coastline of the Christchurch District extending from the Waimakariri River mouth in the north to the entrance of Te Waihora (Lake Ellesmere) in the south (refer to Figure 1.1). The assessment includes open coast and pocket beaches, estuaries and lagoons and cliffs and banks. The assessment area within the estuary and lagoons is limited to the area directly attached to the Coastal Marine Area (CMA) boundary.

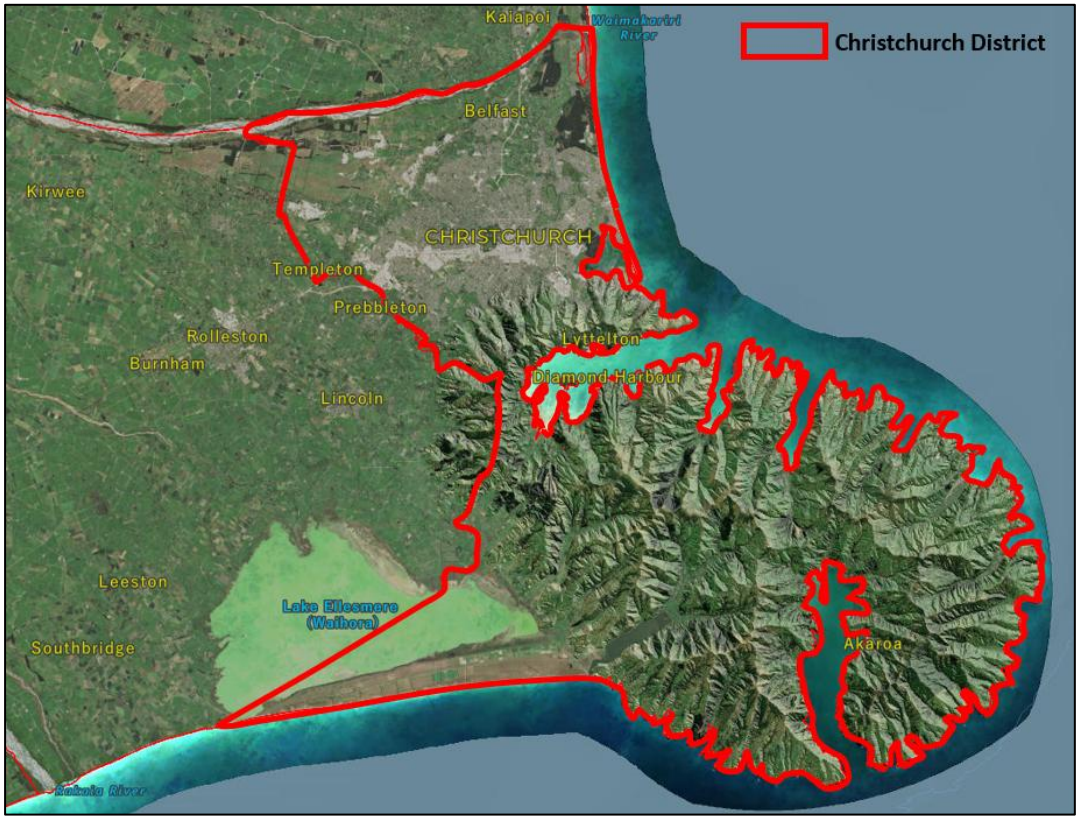

Figure 1.1: Christchurch district indicated by red polygon (source: Canterbury Maps).

1.3 Scope

The project has been undertaken in three stages:

Stage 1: Scoping and initial technical reporting

This initial scoping stage involved undertaking a review of the previous Christchurch coastal hazard assessment as well as assessments completed for the wider Canterbury region and New Zealand to inform a consistent approach. Available data sources were collated to facilitate technical discussion between T+T, CCC, ECan and the technical reviewer. Appropriate methodologies were developed to allow consistent identification of both coastal erosion and inundation hazards for the entire Christchurch District.

Stage 2: Technical assessment (this report)

This report includes a comprehensive assessment of coastal erosion and inundation hazard, and associated groundwater information for the Christchurch District, which is based on the

methodologies agreed upon in Stage 1. This report sets out available data that has been used, methodologies, analyses and results of both erosion and inundation hazards.

Stage 3: Communicating the hazard information

As part of Stage 3 the raw hazard information from the technical assessment has been translated into various more accessible forms to support community engagement and public awareness efforts. This includes a public-facing report and interactive website. The purpose of the website is to allow those with a particular interest to explore the results in more detail than is possible with the printed maps in this report. The interactive online map format makes it easy for users to explore the wide range of scenarios considered in the assessment (e.g., with slider controls to adjust sea level rise), and to zoom in to particular locations of interest. The online viewer can be accessed at https://ccc.govt.nz/environment/coast/coastalhazards/2021-coastal-hazards-assessment

1.4 Report layout

This report is structured as follows:

- Environmental data that has been used for this study is set out in Section 2.
- Coastal erosion methodology, analysis and results are set out in Sections 3 to 5.
- Coastal inundation approach, analysis and results are set out in Sections 6 to 8.
- Groundwater approach, analysis and results are set out in Section 9.

1.5 Reference levels

The vertical elevation or reference levels in this report are with respect to New Zealand Vertical Datum (NZVD2016) unless otherwise specified. As illustrated in Figure 1.2, NZVD2016 is 1.648 m above Chart Datum (CD) at Lyttelton based on LINZ (2021) and 0.35 – 0.4 m above Lyttelton Vertical Datum 1937 (LVD-37) depending on the exact location based on the spatial difference grid by LINZ (2021). Christchurch City Drainage Datum (CDD) is 9.043 m above LVD-37 based on NIWA (2011) and therefore 8.64-8.69 m above NZVD2016.

For example, a MHWS high tide water level at Lyttelton of 0.84m (NZVD2016) is equivalent to a level of 1.23m (LVD1937) or 10.28m (CDD.)

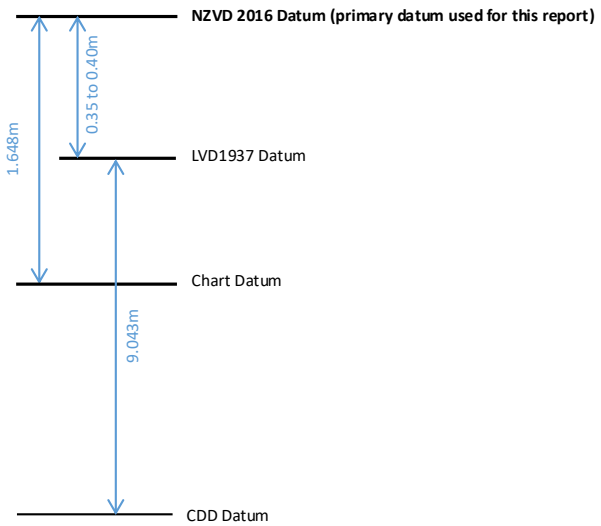

Figure 1.2: Relationship between vertical datums commonly used in Christchurch district

2 Environmental data

2.1 Topography and bathymetry

The following assessment uses the latest available LiDAR which for most of the region is a 1 m DEM flown in 2018 sourced from ECan (Figure 2.1). The most recently available LiDAR for Kaitorete Spit is a 0.45 m DEM LiDAR flown in 2008. Because two different surveys have been used there is a slight mismatch in levels at the join between surveys, which results in some minor artefacts in the inundation maps in the vicinity of Te Waihora (Lake Ellesmere).

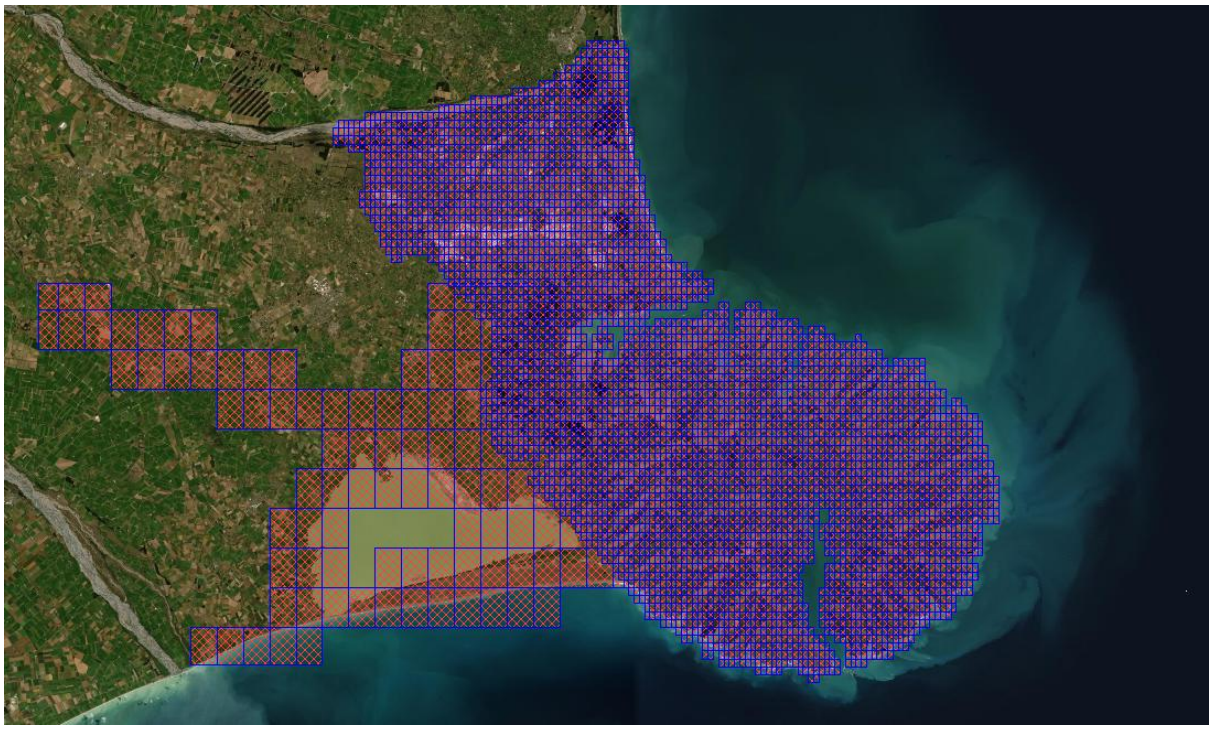

Figure 2.1: Extent of topography datasets. (Purple) 2018 1 m DEM including Christchurch City and Banks Peninsula. (Red) 2008 0.45 m DEM for available for Kaitorete Spit.

Relevant bathymetric surveys are summarised in Table 2.1. The recent 2018 LiDAR captures some of the intertidal flats in the Avon-Heathcote Estuary, however the tidal channels are excluded (Figure 2.2). Rogers et al. (2020) collected RTK surveys and single-beam echosounder data across the estuary mouth in April/May 2019.

There have been bathymetric studies completed for Upper Akaroa Harbour and Lyttelton Harbour (Hart et al., 2009 and Hart et al., 2008). The 2008/2009 bathymetries have been compared against 1952 bathymetric surveys. Hart et al. (2009) provides a 1 m contour map for entire Akaroa Harbour and 0.25 m contour maps for Wainui Bay, French Farm Bay, Barrys Bay, Duvauchelle Bay, Robinsons Bay, Takamatua Bay and Akaroa Inlet.

Table 2.1: Bathymetry sources

Location	Bathymetric surveys
CHCH open coast	Chart NZ 63 Kaikoura Peninsula to Banks Peninsula (1:200000)
Avon-Heathcote Estuary	April 2011 (Measures et al. 2011) January 2013 survey (NIWA) 2018 LiDAR (partial coverage on intertidal flats) Single-beam echosounder surveys (estuary mouth) collected by Rogers et al. (2020)
Banks Peninsula	Chart NZ 632 Banks Peninsula (1:75000)
Lyttelton Harbour	Chart NZ 6321 Lyttelton Harbour / Whakaraupō (1:25000)
Akaroa Harbour	Chart NZ 6324 Akaroa Harbour (1:30000) Chart NZ 6324 Akaroa Harbour: French Bay (1:15000) Hart et al. (2009)
Kaitorete Spit	Chart NZ 632 Banks Peninsula (extends partially)

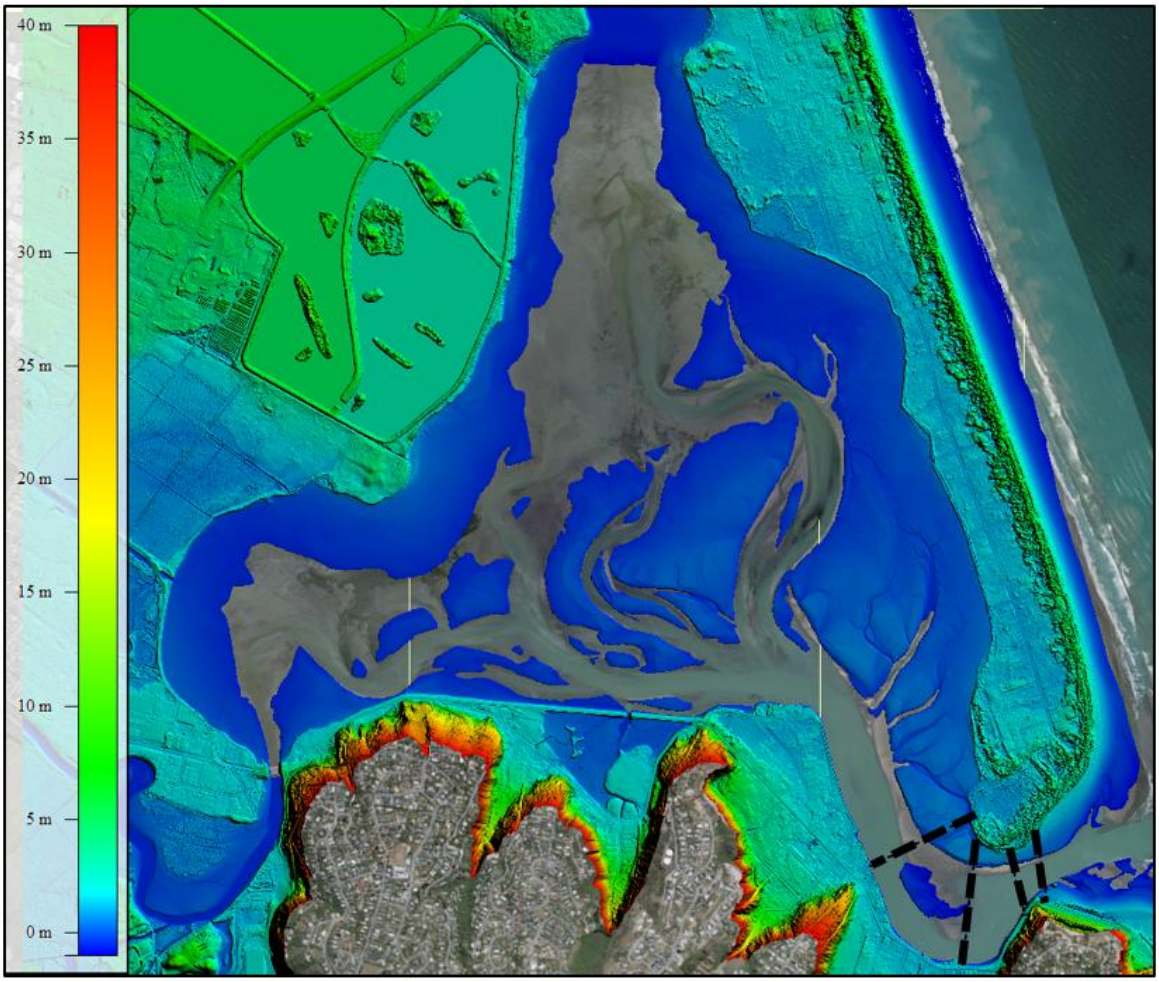

Figure 2.2: 2018 LiDAR extent across the Avon-Heathcote estuary. Dashed lines overlaid to show approximate location of survey transects reported by Rogers et al. (2020).

The DEM represents a bare earth terrain, with all buildings and above-ground features detected having been removed. Using this approach, it is sometimes possible that flooding is shown to occur through the area occupied by large buildings. This is because the model does not recognise these as buildings and works only off the (interpolated) DEM. Care should therefore be exercised in the

interpretation of results, particularly in areas where there is a high percentage of ground area covered by above-ground features (trees, buildings, etc). The same is also true of bridges that cross open waterways. In some cases the DEM excludes the bridge deck, and flooding is shown to exist over the bridge where, in reality, the DEM has ignored the bridge.

As can be seen in Figure 2.2 ground elevation from the DEM exists in the area covered by the wastewater treatment ponds at Bromley. It should be noted that elevation in these water bodies will not be invert levels as LiDAR does not penetrate water. As such any inundation shown over areas such as these, which are permanently covered in water, will need to be viewed in this context.

2.2 Aerial imagery

2.2.1 Latest available imagery

The most recently available aerial photography is 2019 imagery available for the entire Christchurch coastline sourced from ECan (Figure 2.3).

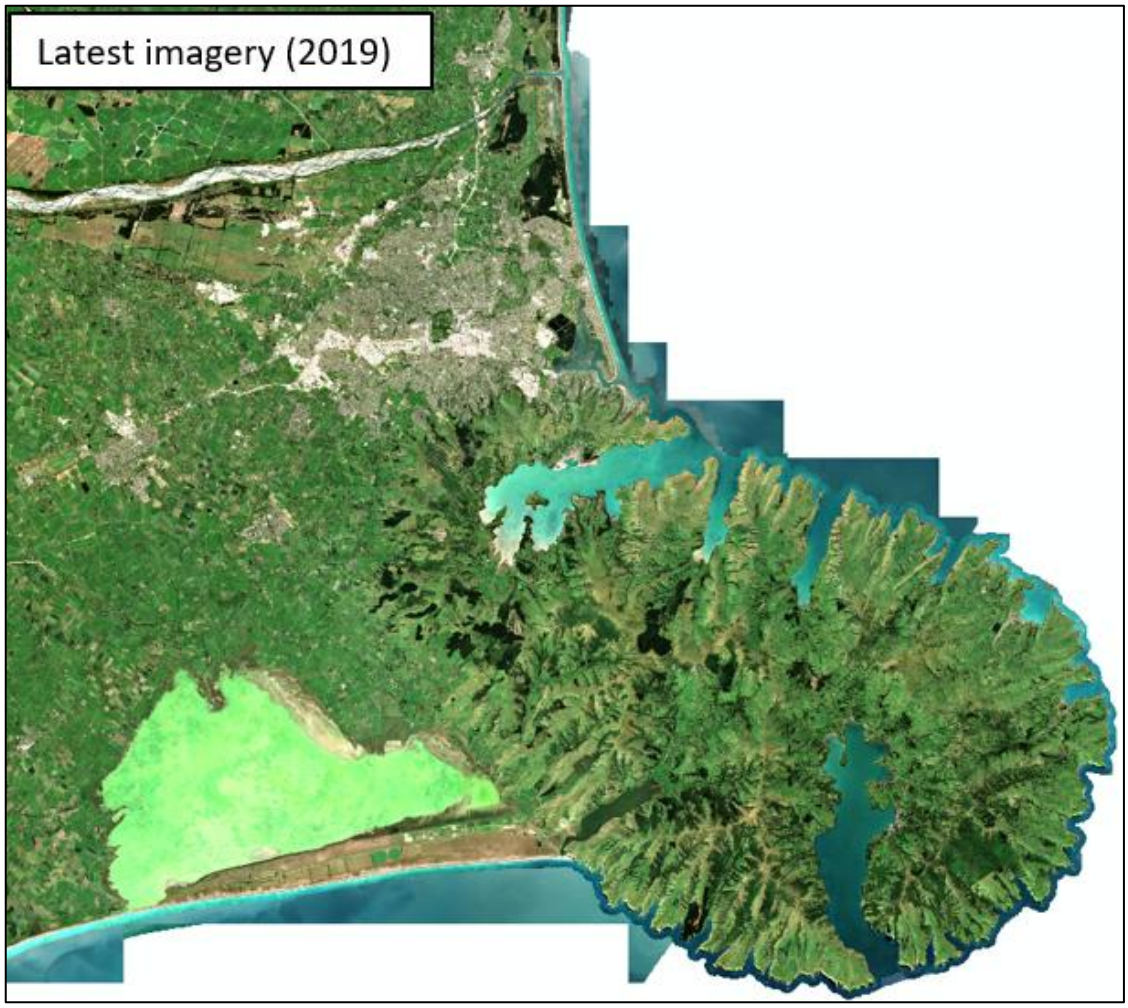

Figure 2.3: Latest available imagery sourced from ECan (2019).

2.2.2 Historic imagery

A summary of the historic aerial imagery available for the region is provided in Table 2.2. The earliest aerials for Christchurch City are from 1941. The earliest aerials available for the outer Banks Peninsula shoreline are from 1995.

Table 2.2: Summary of historic aerial imagery available

Location	Years available	Source
Christchurch open coast	1941, 1955, 1979, 1994, 2011, 2016, 2019	ECan GIS Server
Avon-Heathcote Estuary	1941, 1955, 1979, 1994, 2011, 2016, 2019	ECan GIS Server
Taylors Mistake	1941, 1945-1949, 1955, 1965-1969, 1970-1974, 1980-1984, 1995-1999, 2000-2004, 2016, 2019	ECan GIS Server
Lyttelton Harbour	1965-1969, 1970-1974, 1980-1984, 1995-1999, 2000-2004, 2010-2015, 2016, 2019	ECan GIS Server
Outer Banks Peninsula	1995-1999, 1980-1984, 2000-2004, 2016, 2019	ECan GIS Server
Akaroa Harbour	1980-1984, 1995-1999, 2000-2004, 2016, 2019	ECan GIS Server
Kaitorete Spit	1980-1984, 1990-1994, 1995-1999, 2010-2015, 2016, 2019	ECan GIS Server

2.2.3 Digitised shorelines

Digitised shorelines for the open coast, Avon-Heathcote Estuary and some of the Lyttelton and Akaroa Harbour beaches were previously provided by CCC for the T+T (2017) study. The 2017 study also included two shorelines for each of the harbour sites. Additional shoreline data has been digitised using the more recently available 2019 aerial imagery. A summary of digitised shorelines is provided in Table 2.3.

Table 2.3: Summary of digitised shorelines

Location		Years available
Christchurch open coast		1941, 1955, 1979, 1994, 2011,2016, 2019
Avon-Heathcote Estuary		1941, 1955, 1979, 1994, 2011, 2016, 2019
Lyttelton Harbour	Allandale	1973, 2016, 2019
	Teddington	1973, 2016, 2019
	Charteris Bay	1973, 2016, 2019
	Purau Bay	1973, 2016, 2019
Akaroa Harbour	Takamatua Bay	1980-1984, 2016, 2019
	Duvauchelle Bay	1980-1984, 2016, 2019
	Wainui	1980-1984, 2016, 2019

2.3 Beach profiles

ECan have collected beach profile data for a total of 57 locations along the Christchurch District coastline (Figure 2.4). The earliest of these surveys was completed in 1970. Majority of the Christchurch open coast profiles have been surveyed on a bi-annual basis since the 1990's, with additional surveys as necessary. Profiles along Kaitorete Spit have been surveyed on an annual basis. Beach profiles have also been collected, biannually since 2017, at four sites within Lyttelton Harbour by the Lyttelton Port Company as part of coastal monitoring for their dredging consents. A summary of the beach profile data available is provided in Table 2.4.

Table 2.4: Summary of beach profile data along the Christchurch coastline

Beach Profile Description		First survey date	Last survey date	Survey period (yr)	No. of Surveys
Code	**Name**				
C2200	Waimakariri River	11/03/1994	20/05/2015	21.2	34
C2070	Brooklands	22/06/1990	27/01/2020	29.6	63
C1972	Brooklands	22/06/1990	27/01/2020	29.6	62
C1891	Brooklands	22/06/1990	27/01/2020	29.6	62
C1755	Spencerville (Heyders Road)	22/06/1990	27/01/2020	29.6	62
C1565	Spencerville	22/06/1990	27/01/2020	29.6	61
C1400	Bottle Lake Forest	22/06/1990	27/01/2020	29.6	62
C1273	Bottle Lake Forest	22/06/1990	27/01/2020	29.6	60
C1130	Waimairi Beach (Larnach Street)	9/05/1990	31/01/2020	29.8	61
C1111	Waimairi Beach (Beach Road)	7/08/2008	31/01/2020	11.5	23
C1100	North New Brighton (Pandora Street)	9/05/1990	31/01/2020	41.2	61
C1086	North New Brighton (Pacific Road)	9/05/1990	31/01/2020	41.5	64
C1065	North New Brighton (Effingham St)	9/05/1990	30/01/2020	41.5	59
C1041	North New Brighton (Cygnet Street)	9/05/1990	30/01/2020	41.5	62
C1011	North New Brighton (Bowhill Road)	9/05/1990	26/07/2020	30.2	59
C0952	New Brighton (Rawhiti Street)	9/05/1990	30/01/2020	41.5	63
C0924	New Brighton (Lonsdale Street)	9/05/1990	30/01/2020	41.5	62
C0889	New Brighton (Hawke Street)	9/05/1990	30/01/2020	29.7	60
C0863	New Brighton 226 Marine Parade)	1/12/2000	30/01/2020	19.2	35
C0856	New Brighton 231 Marine Parade)	21/07/2004	30/01/2020	15.5	31
C0853	New Brighton 233 Marine Parade)	21/07/2004	30/01/2020	15.5	31
C0848	New Brighton (Hood Street)	9/05/1990	30/01/2020	29.7	60
C0815	New Brighton (Rodney Street)	9/05/1990	30/01/2020	41.5	61
C0781	New Brighton (Mountbatten Street)	9/05/1990	30/01/2020	41.5	61
C0748	South New Brighton (Jervois Street)	9/05/1990	28/01/2020	41.5	62
C0703	South New Brighton (Bridge Street)	1/08/1978	27/01/2020	41.5	62
C0650	South New Brighton (Beatty Street)	1/08/1978	28/01/2020	41.5	62
C0600	South New Brighton (Jellicoe Street)	1/08/1978	28/01/2020	41.5	57
C0531	South New Brighton (Halsey Street)	19/12/1978	28/01/2020	41.5	61
C0513	South New Brighton (Caspian Street)	18/12/1978	28/01/2020	41.5	63
C0471	South Shore (Heron Street)	31/07/1978	28/01/2020	41.5	62
C0431	Southshore (Penguin Street)	1/08/1978	27/01/2020	41.5	63
C0396	South Shore (Plover Street)	9/05/1990	27/01/2020	29.7	61
C0362	South Shore (Tern Street)	1/08/1978	27/01/2020	41.5	62
C0350	South Shore (Torea Street)	9/05/1990	27/01/2020	29.2	60
C0300	South Shore (South of Pukeko Place)	9/05/1990	27/01/2020	29.7	64
C0271	South Shore (End Rockinghorse Road)	9/05/1990	27/01/2020	29.7	65

Table 2.4 (continued): Summary of beach profile data along the Christchurch coastline

Beach Profile Description		First survey date	Last survey date	Survey period (yr)	No. of Surveys
Code	**Name**				
C0070	Sumner	9/05/1990	23/01/2020	29.7	59
C0112	Sumner	9/05/1990	23/01/2020	29.7	59
C0150	Sumner	9/05/1990	23/01/2020	29.7	59
C0180	Clifton	9/05/1990	24/01/2020	29.7	66
C0190	Clifton	9/05/1990	24/01/2020	29.7	60
C0221	Clifton	9/05/1990	24/01/2020	29.7	64
BPN8010	Taylors Mistake	21/07/1993	23/01/2020	26.5	53
BPN7998	Taylors Mistake	21/07/1993	23/01/2020	26.5	53
BPN7985	Taylors Mistake	21/07/1993	23/01/2020	26.5	53
BPN7975	Taylors Mistake	21/07/1993	23/01/2020	26.5	53
ECE3800	Birdlings Flat	04/03/1991	28/05/2020	29.3	30
ECE3755	Birdlings Flat	04/03/1991	26/06/2019	29.3	29
ECE3560	Kaitorete Spit	04/03/1991	28/05/2020	29.3	30
ECE2995	Kaitorete Spit	04/03/1991	28/05/2020	29.3	30
ECE2515	Kaitorete Spit	04/03/1991	28/05/2020	29.3	30
ECE1980	Kaitorete Spit	04/03/1991	28/05/2020	29.3	30
ECE1620	Kaitorete Spit	04/03/1991	28/05/2020	29.3	30
ECE1320	Kaitorete Spit	01/03/1991	27/05/2020	29.3	30
ECE1183	Kaitorete Spit	02/06/1970	27/05/2020	50	51
ECE1172	Kaitorete Spit	02/06/1970	27/05/2020	50	39
LPC	Corsair Bay	03/02/2017	27/01/2021	4	9
LPC	Rapaki	03/02/2017	27/01/2021	4	9
LPC	Purau Bay	03/02/2017	27/01/2021	4	9
LPC	Camp Bay	03/02/2017	27/01/2021	4	9

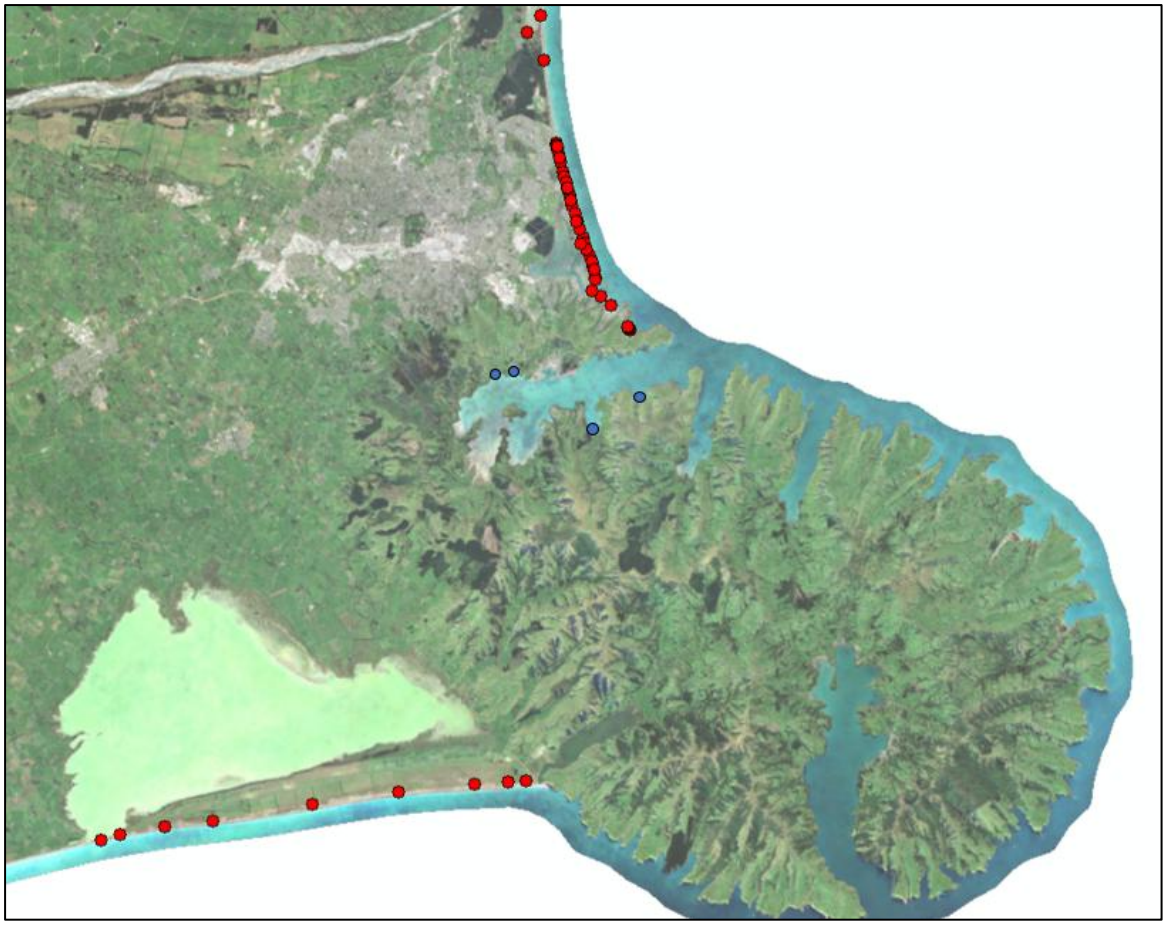

Figure 2.4: Location of ECan beach profile datasets (red dots) and LPC monitoring profiles (blue dots).

2.4 Water levels

2.4.1 Tide levels

Tidal levels for New Zealand ports are provided by Land Information New Zealand (LINZ) based on average predicted values over the 18.6 yr tidal cycle. Values for Lyttelton are presented in Table 2.5. The spring tidal range is approximately 2.2 m and the mean sea level is -0.22 m NZVD2016.

Table 2.5: Astronomical tide levels at Lyttelton Port (Source: LINZ, 2021)

Tide state	m NZVD2016
Highest Astronomical Tide (HAT)	1.07
Mean High Water Springs (MHWS)	0.84
Mean Sea Level (MSL)	-0.22
Mean Low Water Springs (MLWS)	-1.38
Lowest Astronomical Tide (LAT)	-1.49

MHWS levels for other locations within Christchurch included in LINZ (2021) are Sumner (0.76 m NZVD2016) and Akaroa (1.08 m NZVD2016), which are based on offsets derived by LINZ (2021) and converted to NZVD2016. This shows that the MHWS at Sumner is 0.08 m lower and at Akaroa (Tikao Bay) is 0.24 m higher compared with Lyttelton Port. The higher MHWS level in Akaroa is likely a result of tidal amplification, which increases towards the head of the harbour.

NIWA (2015) include MHWS and MHWPS levels for multiple locations within Christchurch the levels seem to be consistent along the Christchurch open coast (i.e. within 2 cm). The difference between MHWS and MHWPS is approximately 0.2 m. Refer to NIWA (2015) for a detailed description of different MHWS definitions in Canterbury.

2.4.2 Water level gauges

CCC and ECan have water level gauges across most of the rivers, harbours and lakes in Christchurch. The water level gauges relevant for this study are shown in Figure 2.5. A summary of the gauge information is provided in Table 2.6.

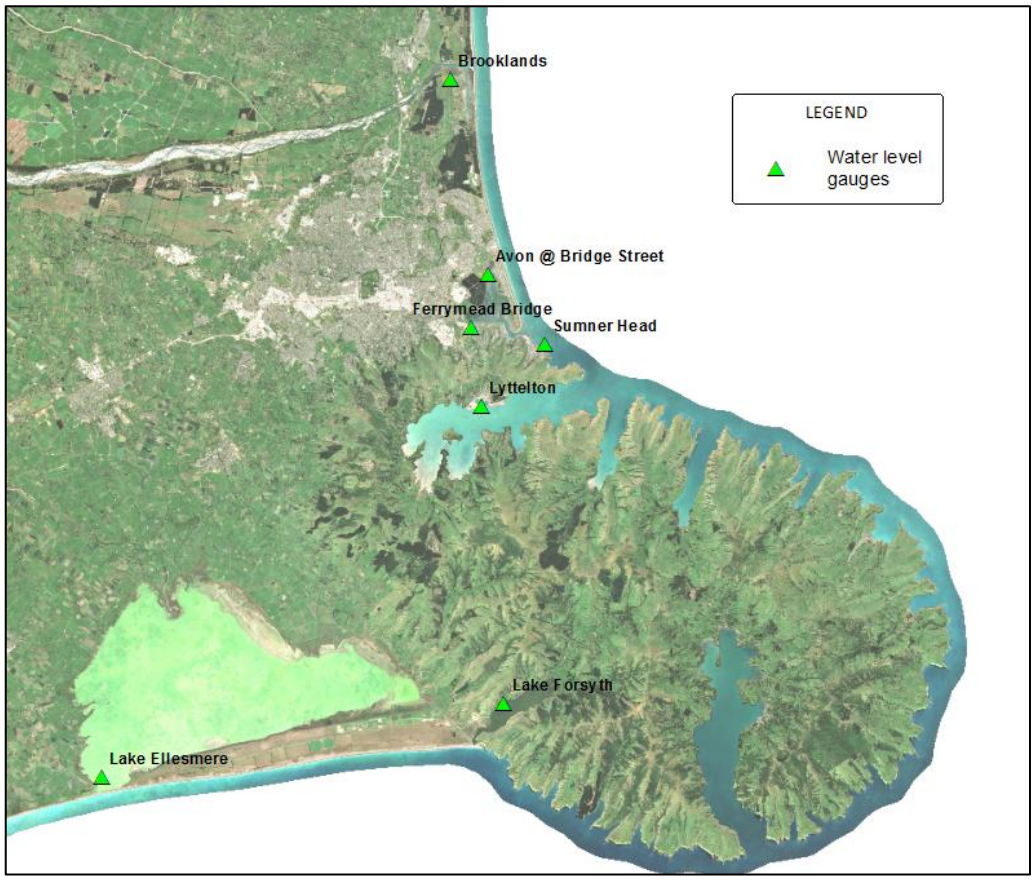

Figure 2.5: Location of relevant water level gauges.

Table 2.6: Summary of water level gauge data available

Location	Type	Start of data	Source
Brooklands (Styx tide gates)	Tidal	1990	CCC
Avon River at Bridge St	Tidal (with river influence)	1997	CCC
Ferrymead Bridge	Tidal (with river influence)	1974	CCC
Sumner Head	Tidal	1994	NIWA/ECan
Lyttelton Standard Port Gauge	Tidal	1998	LINZ/LPC
Wairewa (Lake Forsyth)	Lake level	1995	ECan
Te Waihora (Lake Ellesmere)	Lake level	1994	ECan

There have been several water level analyses completed using the Christchurch tide gauge data, such as NIWA (2015) and Goring (2018), with the latest analysis undertaken by GHD (2021). The

resulting extreme water levels for the analysed gauges are shown in Table 2.7. The Styx water level is taken below the tide gates and represents the level in the Brooklands Lagoon. Bridge St and Ferrymead are within the Avon-Heathcote estuary.

Table 2.7: Extreme water levels (m NZVD2016) based on tide gauge analysis (excl. wave effects)

	ARI							
Site	**1 year**	**2 year**	**5 year**	**10 year**	**20 year**	**50 year**	**100 year**	**200 year**
Sumner[1]	1.37	1.44	1.52	1.59	1.65	1.74	1.80	1.87
Bridge St	1.33	1.40	1.49	1.56	1.64	1.73	1.80	1.87
Ferrymead	1.31	1.36	1.44	1.50	1.56	1.63	1.69	1.75
Styx	1.44	1.50	1.58	1.64	1.69	1.77	1.83	1.89
Lyttelton	1.31	1.36	1.41	1.45	1.49	1.54	1.58	1.62

Source: GHD (2021), converted from CCC Datum to LVD-37 (-9.043 m) and then converted to NZVD2016 using difference grid from LINZ (2016).
[1]Levels include effects of infra-gravity (IG) waves.

2.4.3 Long-term sea levels

Historic rise in mean sea level around New Zealand has averaged 1.7 ± 0.1 mm/year with Christchurch exhibiting a higher rate of 2.12 ± 0.09 mm/year (MfE, 2017).

Climate change is predicted to accelerate this rate of sea level rise. The Ministry for the Environment (MfE, 2017) guideline recommends using four scenarios to cover a range of predicted future sea levels that reflect the inherent uncertainty.

1 NZ RCP2.6 M (Low to eventual net-zero emission scenario).

2 NZ RCP 4.5 M (Intermediate-low scenario).

3 NZ RCP 8.5 M (High-emissions scenario).

4 NZ RCP 8.5 H+ (Higher extreme RCP8.5H+ scenario, based on the RCP8.5 83rd percentile projection from Kopp et al. (2014)).

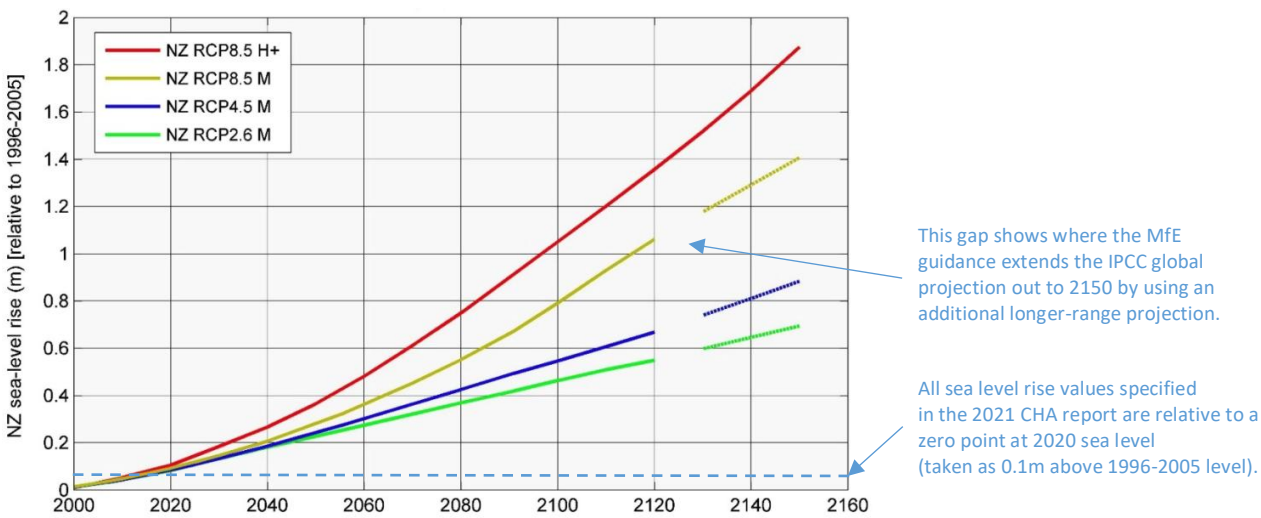

Figure 2.6: Four scenarios of New Zealand-wide regional sea-level rise projections as presented within the MfE 2017 guidance, with extensions to 2150 based on Kopp et al. (2014) (Adapted from MfE, 2017).

The MfE guidance scenarios shown in Figure 2.6 were based on IPCC (2013). As the 2021 CHA report was being finalised, IPCC (2021) was released. IPCC (2021) provides an updated suite of sea level rise projections, which differ from the IPCC (2013) projections in some details. However, the various increments of sea level rise adopted for the 2021 CHA (refer Sections 3.5 & 6.3) still provide good coverage across the range of updated sea level rise projections. This adaptability is one reason why a wide range of sea level rise increments were adopted for the 2021 CHA, rather than fixing the analysis to specific RCP projections.

2.5 Waves

MetOcean Solutions Ltd. have New Zealand-wide and nested wave hindcast models available from 1979 to 2020, with a Canterbury-wide hindcast (400 m domain) specifically for the Christchurch region. Wave timeseries including 3-hourly data from 1979 to 2020 extracted at the -10 m depth contour have been provided by MetOcean for several locations along the shoreline (refer to Figure 2.7). Extreme value analyses have been undertaken for the available timeseries to derive extreme wave heights. Table 2.8.shows the extreme significant wave heights at several locations along the open coast.

Table 2.8: Extreme open coast wave heights (H_s) (m) derived from MetOcean hindcast data

Site	Average Recurrence Interval (ARI)		
	1 year	**10 year**	**100 year**
Waimairi Beach	3	3.8	4.2
North New Brighton	3.2	3.9	4.3
South New Brighton	3.2	3.9	4.3
Sumner	3.3	3.9	4.2
Lyttelton Harbour Entrance	3.4	4.0	4.3
Akaroa Harbour Entrance	5.8	7.2	8.5
Kaitorete Spit	4.4	4.9	5.6

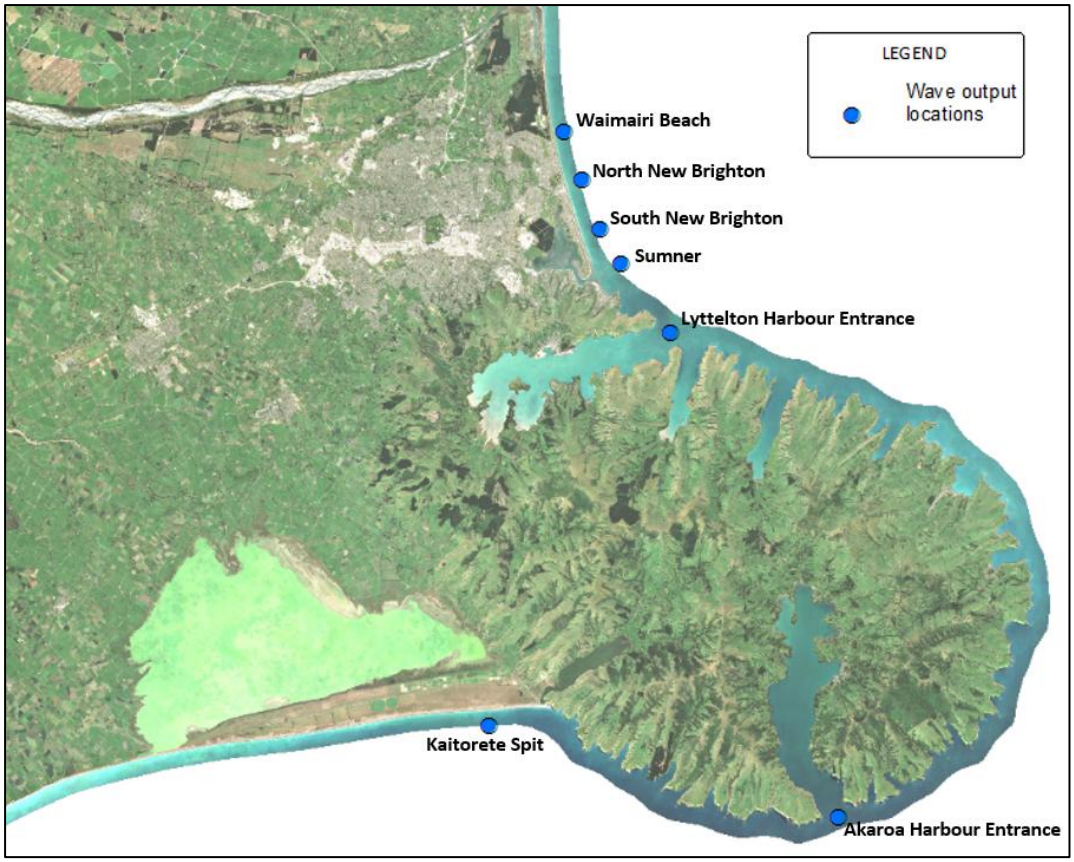

Figure 2.7: Wave output locations.

2.6 Winds

Wind data is available from the NIWA National Climate Database (CliFlo) and Metservice. The list of relevant weather stations is provided in Table 2.9 and shown in Figure 2.8. Other existing wind data records such as data from New Brighton Pier AWS (2009-2016) and wind data discussed by Goring (2008) including Oxidation Pond No. 3 (1993-1998) and Bottle Lake Forest (1997-2008), but have not been considered in this assessment due to their relative short lengths.

Table 2.9: Available weather stations around Christchurch region

Station name	Agent Number	Owner	Start of data	End of data
Christchurch Aero	4843	Metservice	31/12/1959	1/06/2020 (present)
Le Bons Bay Aws	4960	Metservice	18/01/1984	1/06/2020 (present)
Lyttelton Harbour	4903		27/07/1978	13/09/2013
Akaroa Ews	36593	NIWA	18/12/2008	1/06/2020 (present)
Akaroa Rue Lavaud	4951	Metservice	8/12/1977	11/05/2001
Bromley Ews	43967	NIWA	1/4/1967	31/7/1988

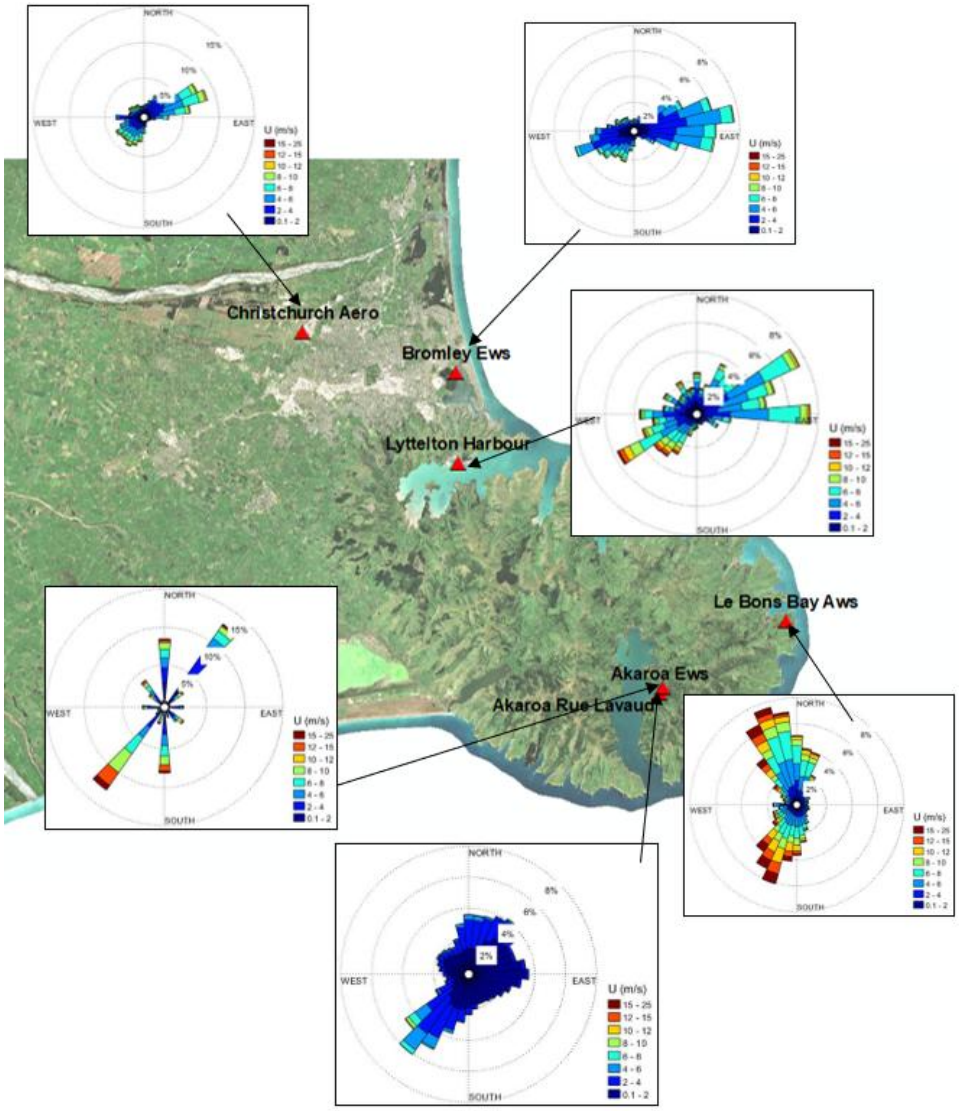

Figure 2.8: Locations of relevant weather stations around Christchurch region including wind roses.

At Christchurch Airport, winds are predominately from the northeast and the southwest quadrants (Figure 2.8). Winds at Bromley and Lyttelton Harbour are slightly more shore normal and predominantly from east-northeast and west-southwest. At Akaroa wind is predominantly from the southwest and north-northeast, and at Le Bons Bay the wind speed is significantly higher with the predominant directions from north-northwest and south-southwest. The higher wind speeds at Le Bons Bay are likely due to the exposed location and height of 236 m above mean sea level. At Akaroa Ews the wind speed is low likely due to its sheltered location.

Goring (2008) assessed whether the wind record at Christchurch Aero is representative for the Avon-Heathcote Estuary. He reviewed a relatively short dataset and compared this with wind data from the oxidation pond (i.e. at the north-western side of the estuary). He found that the southerly winds recorded at the airport are much smaller than those experienced over the estuary and may therefore not be representative for the estuary. The wind data from Bromley EWS is representative for the Avon-Heathcote Estuary based on Goring (2008).

2.7 Sediment supply

As part of Council's Multi-Hazards Study (LDRP 97, Jacobs (2017)), Hicks (2018a) assessed the current coastal sand budget for Southern Pegasus Bay, as summarised in Figure 2.9. The key sediment supply to the coast is sand from the Waimakariri River, which is estimated to be 745,000 m^3/yr. The assessment includes potential sand losses from the Waimakariri River such as loss associated with irrigation water abstraction, gravel extraction and entrapment in Brooklands Lagoon. Loss of sand through irrigation abstraction and gravel extraction is considered to have minor impact on the supply at the coast.

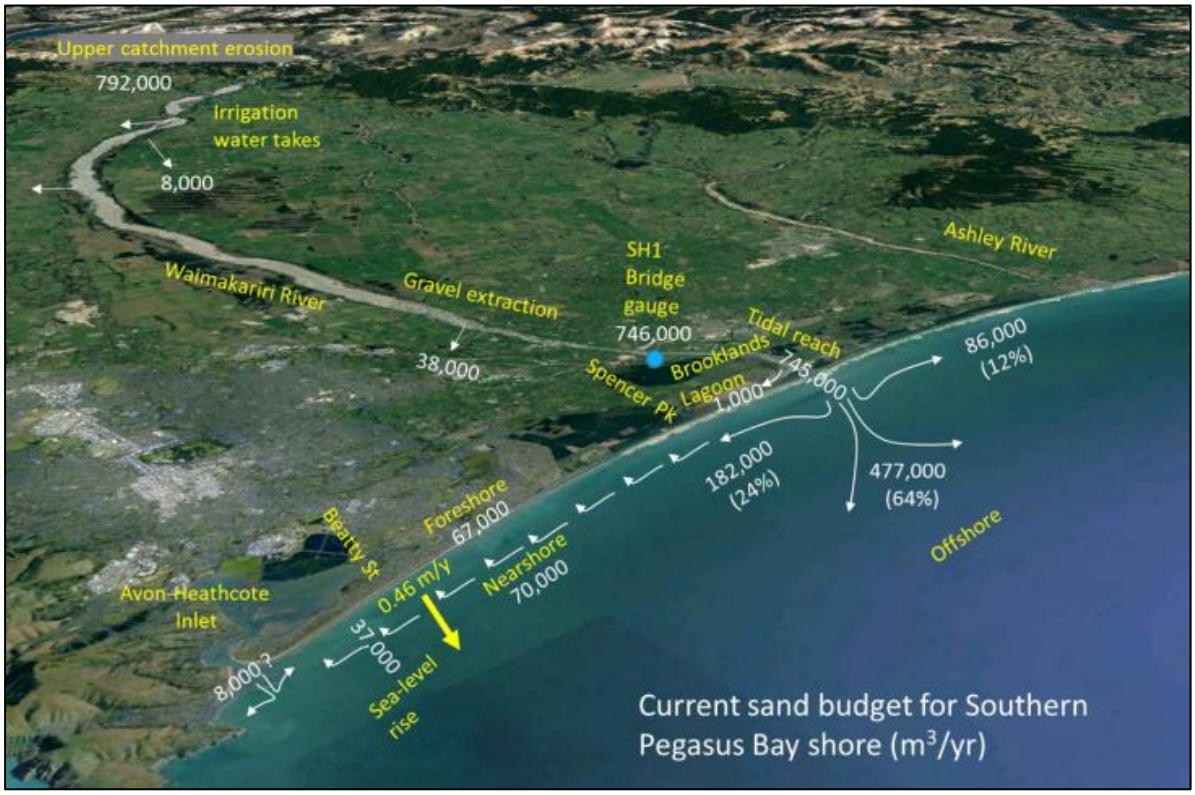

Figure 2.9: Current sand budget for Southern Pegasus Bay (sourced from Hicks (2018a).

Hicks (2018a) assessed longshore transport rates using a SWAN model based on the wave record from the Banks Peninsula buoy for the period September 2000 to December 2017. Results indicate the river sand enters a bi-directional longshore transport system, with approximately 24% transported southwards.

The study includes assessment of the sand volume changes along the Christchurch city beaches, sand exchange with the Avon-Heathcote Estuary and the beach 'demand' for sediment south of the Waimakariri River mouth.

Hicks (2018b) assesses the future sediment budget for Southern Pegasus Bay. The assessment includes a range of future scenarios (Table 2.10) including climate change effects on Waimakariri sand supply and effects of wave climate change and sea level rise.

Key findings from the study include:

- Changes in the Waimakariri sediment supply to the coast could vary from an 11% reduction in supply to a 28% increase.
- Climate-change-altered nearshore wave climate could alter the volume of sediment supply transported southwards:
  - The proportion of sediment transported southwards may reduce due to a reduction in wave energy from the NE quarter. For example, the proportion of sediment transported southwards may reduce by 10% under the RCP6.0M wave climate and by 25% under the RCP8.5M wave climate.
- SLR by itself with no change in offshore wave climate would increase the proportion of sand transported southwards.
- SLR and wave climate change would have compensating effects, however the wave climate change would prevail, resulting in reduced proportions transported south.

The study assesses the impacts of future sand budget on beach volumes and shoreline position and concludes that at least until 2120, the city shore sand budget should remain in surplus except under the RCP8.5M climate change scenario.

Table 2.10: Summary of future scenarios assessed by Hicks (2018b)

Scenario	Description	River load scenario	Wave scenario	SLR by 2120 (m)
A	Worst case independent combination	Total 11% reduction (8% by climate change, 3% by irrigation)	Wave climate aligning with RCP6.0	1.36
B	Worst case independent combination	Total 11% reduction (8% by climate change, 3% by irrigation)	Wave climate aligning with RCP8.5	1.36
C	RCP8.5+ SLR	Increased 28% by climate change, reduced 3% by irrigation	Wave climate aligning with RCP8.5	1.36
D	RCP8.5+ SLR, no wave change, inlet loss	Increased 28% by climate change, zero irrigation effect	Baseline	1.36
E	RCP8.5 Median* + ebb delta losses	Increased 28% by climate change, reduced 3% by irrigation	Wave climate aligning with RCP8.5	1
F	RCP8.5 Median*	Increased 28% by climate change, zero irrigation effect	Wave climate aligning with RCP8.5	1
G	RCP6.0	Increased by 9%	Wave climate aligning with RCP6.0	0.63
H	RCP2.6*	Total 11% reduction (8% by climate change, 3% irrigation)	Baseline	0.55
I	Landslide doubles river load	Load doubled	Wave climate aligning with RCP8.5	
J	Status quo	Baseline	Baseline	Baseline
K	Landslide doubles river load, no CC	Load doubled	Baseline	Baseline

Another study which includes sediment budget for the Canterbury region is Single (2006). Single (2006) investigated the gravel budget along the Canterbury Bight and determined that the Canterbury Bight is nearly in a state of gravel budget balance. The total river supply of gravel to the Canterbury Bight coast is about 176,700 m^3/yr (comprising Opihi/Temuka 19,400 m^3/yr; Orari 12,500 m^3/yr; Rangitata 28,000 m^3/yr; Hinds 16,000 m^3/yr; Ashburton 27,300 m^3/yr; Rakaia 73,500 m^3/yr).

2.8 Vertical land movement

2.8.1 Earthquake movement

The ground around Canterbury has experienced regional scale tectonic movements caused by the 2010 – 2011 Canterbury Earthquake Sequence (CES). These movements comprise both translation and elevation change which have deformed the ground surface. T+T (2013) analysed seven LiDAR datasets and ground-based survey points collected between 6 July 2003 and 17 February 2012 to quantify vertical ground displacement throughout Christchurch City, parts of the Port Hills and Sumner.

By separating regional-scale tectonic movement from total elevation change, the local effects of liquefaction induced elevation change (the ejection of sand, lateral spreading, topographic effects and the settlement of liquefied soils) could be isolated from tectonic ground movements.

Results show general subsidence across the city with subsidence from 0.1 m to more than 0.5 m (Figure 2.10) with the most pronounced subsidence occurring along the banks of the Avon River in the city's northeast. At the same time, the southeast of the city including the southern margins of the Avon-Heathcote Estuary and the Ferrymead area experienced uplift of up to 0.45 m, which can be attributed to tectonic related movement.

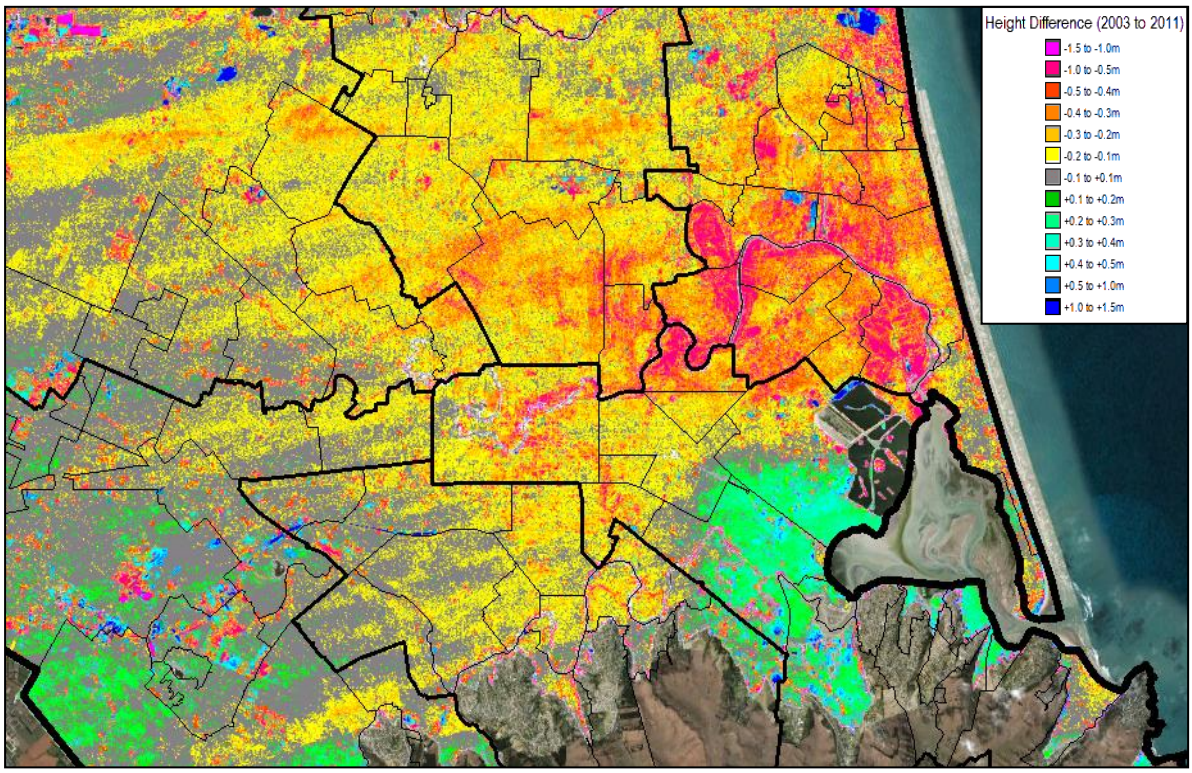

Figure 2.10: Summary of changes in ground elevation between 2003 and 2011 (source T+T, 2013).

Using pre and post-earthquake LiDAR surveys, Measures et al. (2011) calculated vertical change across the estuary and found that the northern part of the estuary subsided by 0.2 to 0.5 m while the southern part of the estuary rose by 0.3 to 0.5 m (Figure 2.11).

Beavan and Litchfield (2012) summarise 2010-2011 earthquake-induced changes along the Christchurch coastline as uplift of more than 0.05 m along the coastline from New Brighton to just south of Lyttelton Harbour. The maximum uplift of more than 0.4 m occurred within the Avon-Heathcote Estuary and coastal subsidence of more than 0.05 m from about New Brighton northwards towards Kaiapoi.

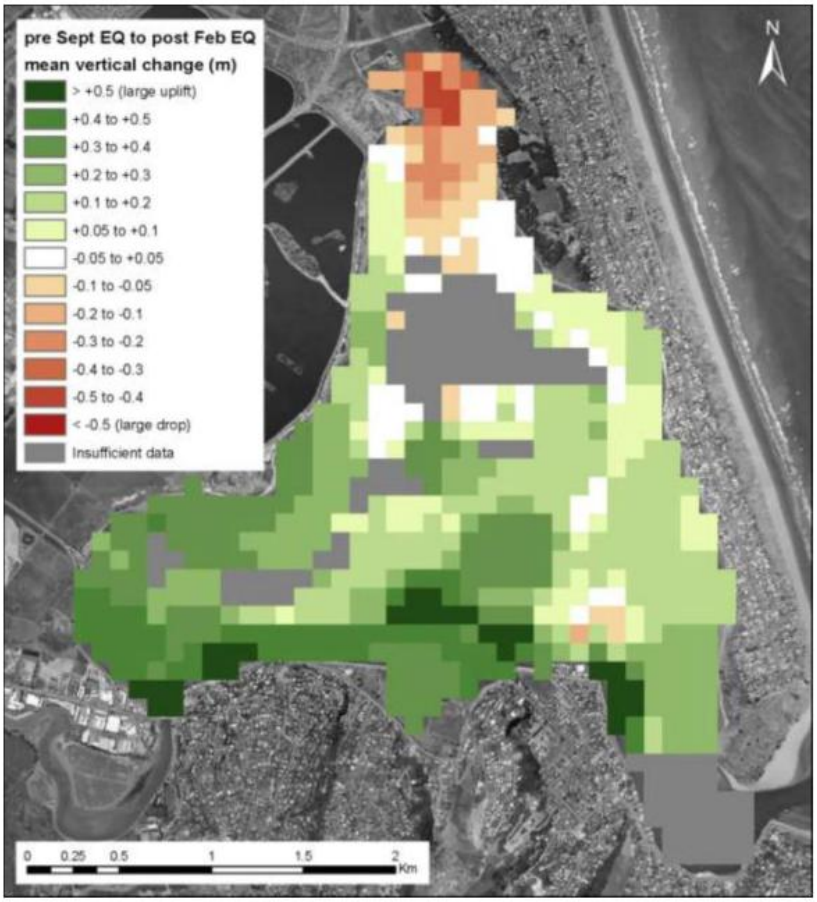

Figure 2.11: Vertical change pre-September 2010 earthquake to post February 2011 earthquake (figure sourced from Measures 2011).

2.8.2 Long-term vertical movement

Long term changes in land elevation may be caused by a number of processes including isostatic adjustment due to changes in mass loading on the Earth's-surface, long-term changes due to plate-tectonics, subsidence due to withdraw of fluids and subsidence due to the natural compaction of sediments (Beavan and Litchfield, 2012). Long-term tectonic movement in the Christchurch area includes tectonic uplift and strike-slip faulting producing the range and basin topography of northern and inland central Canterbury and subsidence of the braidplain (Forsyth *et al.,* 2008).

The School of Surveying at the University of Otago have recently assessed the vertical ground motion around Christchurch City based on the semi-continuous GNSS sites and continuous station (SMNT) at the Sumner tide gauge (Pearson et al., 2019). The assessment includes consideration of the co-seismic offsets and post seismic relaxation from the February 2016 Christchurch earthquake and the November 2016 Kaikoura earthquakes (Figure 2.12). Vertical velocities are corrected for the

coseismic offsets to estimate the long term rates shown in Figure 2.13. Rates of land subsidence over the 4-year data period range from 0.23 to 7.97 mm/yr near the Christchurch city coast. However, the length of data is short and Pearson et al. (2019) state that further monitoring is required to monitor the subsidence across eastern Christchurch.

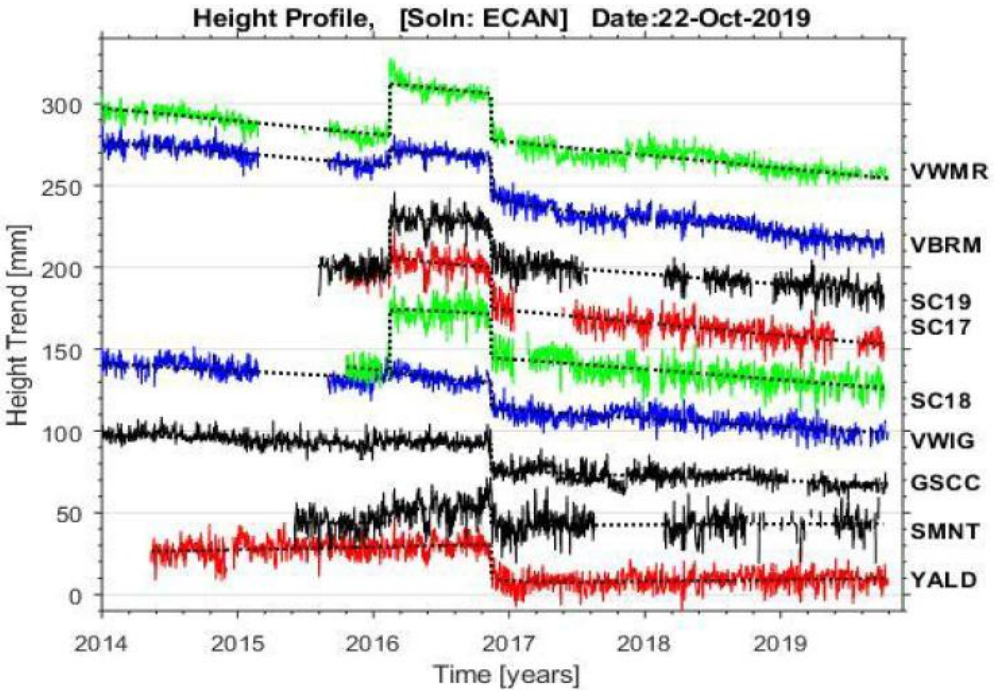

Figure 2.12: Height timeseries including the co-seismic offsets and post seismic relaxation for all sites (source Pearson et al. 2019).

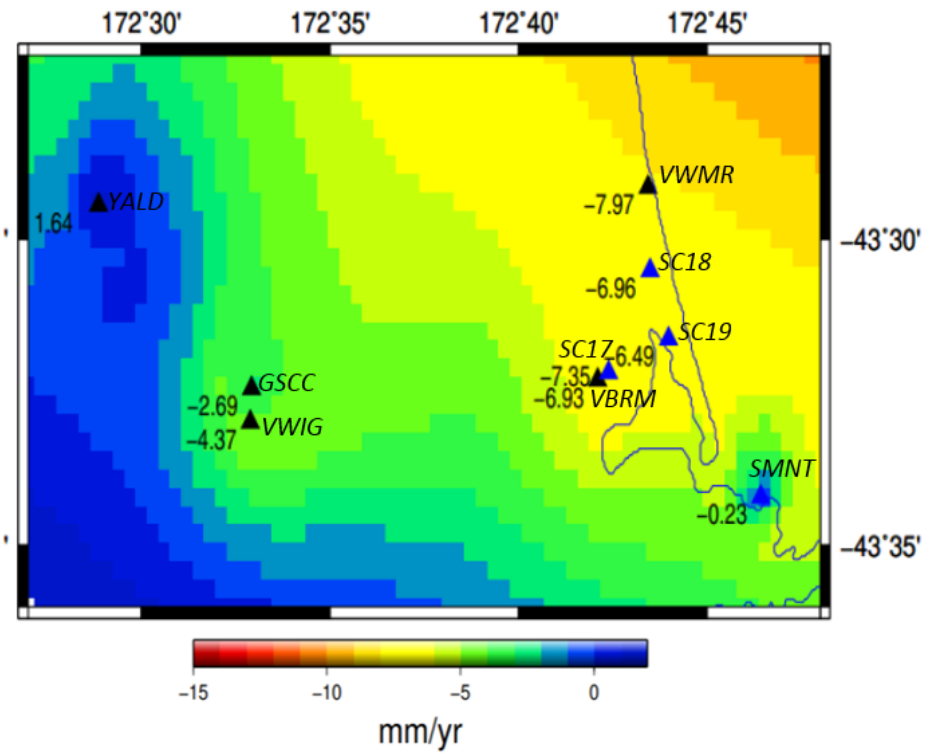

Figure 2.13: Contour map showing estimated long term vertical velocities (mm/yr), uplift (positive, blue) and subsidence (negative, green to orange) (sourced from Pearson et al., 2019). Estimated long term velocities are based on timeseries (2015 to 2019) corrected for coseismic offsets Coastal erosion methodology.

2.9 Anthropogenic influences

2.9.1 Dredging

March (2018) Lyttelton Port Company was granted resource consent to dredge the harbour shipping channel to increase its draught. The recent capital dredge included up to 18M m^3 of material deposited at an offshore disposal site located 6 km off Godley Head in 20m water depth (Figure 2.14). Ongoing maintenance dredging of some 0.9M m^3 per annum is deposited at existing disposal ground within the harbour. MetOcean Solutions Ltd (2016) undertook numerical modelling to investigate the morphological effects and sediment transport patterns associated with the disposal of capital dredged material at the offshore disposal site and the effects of the mound on the offshore wave height gradient.

T+T (2016) review and summarise the effects of the dredging on coastal process and note that the modelling (Figure 2.14) shows a small amount of wave focussing in the lee of the disposal site mound and defocussing on either side. Focussing of up to 4% may occur along South New Brighton with mean reductions of up to 2% between Sumner and Lyttelton Harbour. These changes are expected to diminish as the mound is eroded and seabed returns to its previous near horizontal form.

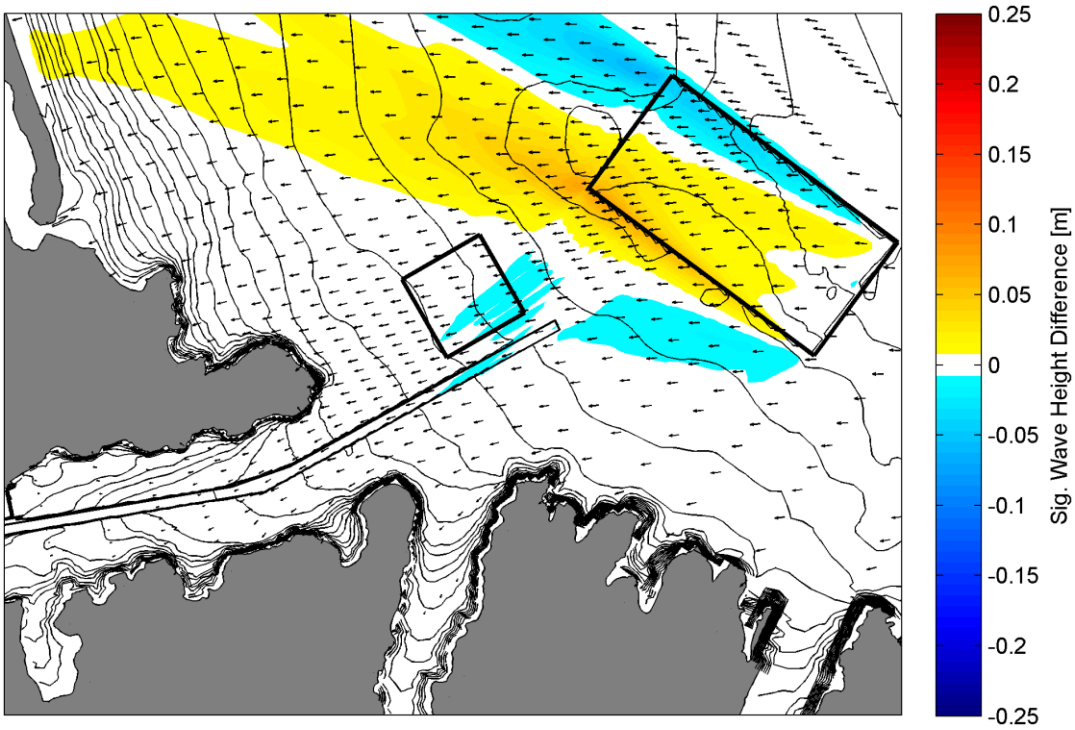

Figure 2.14: Difference in a weighted mean significant wave height (m) between the existing and with the elevated capital disposal ground (top) (image sourced from MetOcean Solutions Ltd (2016)

2.9.2 Gravel extraction

Single (2006) describes the gravel management within the Canterbury Bight. Historically sand has been mined from Kaitorete Spit (1952 to 1984). Peak annual extraction volumes of about 312,000m^3 occurred in the mid-1970s. Sediment is often excavated to open river channels to sea to prevent flooding. This occurs at Taumutu and Birdlings Flat. Due to the northwards sediment transport, excavated material is placed on the active beach north of the channels.

Gravel extraction operations from the lower Waimakariri River channel remove about 38,000 m^3/yr of sand. Most of the extraction occurs in the reach between 18 and 4 km upstream from the coast,

where the river's gradient starts to reduce, diminishing the river's capacity to transport gravel and prompting deposition. A gravel/sand transition occurs in the tidally influenced zone approximately 4 km from the coast, and from there downstream the Waimakariri River is a single-thread sand-bed river and carries no gravel bedload.

Hicks (2018a) indicates that gravel extraction from Waimakariri River has minimal impact on coastal sand budget. Even if extraction stopped, most of the extracted sand would not reach the coast as it would remain locked up in river-bed gravel deposits. (Hicks, 2018a).

2.9.3 Dune restoration

Dune enhancement measures have been applied along CHCH open coast since the 1870s when Marram grass was introduced with more enhanced measures such as dune reshaping and foredune planting of native sand-binding species applied since the 1990s. It is likely that the larger seaward growth of the dunes is a result of dune management. Figure 2.15 shows an example of dune management along North New Brighton.

Figure 2.15: Dune management along North New Brighton in February 1992

2.9.4 Artificial lake opening

Te Waihora (Lake Ellesmere) is intermittently closed and open to the sea. The lake is mechanically opened to the sea primarily to minimize flooding of adjacent agricultural land, flush poor quality water, and provide passage for migrating fish such as founder and eel (Measures et al., 2014). Since the mid 1850's there have been numerous schemes and proposals for how to open the lake.

Figure 2.16: Bulldozers removing the 'scab' to make the connection between the cut and sea (top). Excavator working to enlarge the freshly open cut (sourced from Measures et al., 2014)

Wairewa (Lake Forsyth) is also artificially controlled to reduce flooding. While it is closed due to the substantial accumulation of gravel, there is evidence that it was probably permanently open until about the middle of last century (Soons et al., 1997). The Lake has been artificially opened from time to time for more than 140 years. Lake openings have included beach and canal openings.

- Beach openings involve excavation of a 4 m wide channel dug through the gravel on the shortest route to the sea, typically angled towards the south-east. The removed gravel is generally deposited on the eastern side of the beach so that it is less likely to be deposited back into the channel by littoral drift processes.
- Canal openings occur through the canal which was constructed in 2009. The canal is approximately 20 to 30 m wide and 900 m long and runs along the toe of the cliff at the eastern end of the beach. A rock groyne (70 m long) has been constructed at the mouth of the canal using local boulders. This structure provides some protection from wave action and limits the gravel close-off of the canal. Opening of the canal typically involves an excavator removing gravel from the blocked canal outlet (Wairewa Rūnanga Incorporated, 2013).

3 Coastal erosion methodology

3.1 Conceptual models for coastal types

Areas Susceptible to Coastal Erosion (ASCE) varies depending on the coastal type and the key drivers of erosion and instability for those coastal types. In the Christchurch district, these coastal types include unconsolidated sandy beaches and gravel barriers, consolidated banks and harder cliffs. The conceptual models proposed for each of these coastal types is set out below.

3.1.1 Sandy beaches

The ASCE for sandy beaches accounts for short-term, storm induced erosion (either singular or a series) and the response of an over-steepened dune as it regresses back to a stable slope. Additionally, longer term change is accounted for as the position of the coastline may change as a result of imbalances in the sediment budget (both positive and negative) and changes in the relative sea level due (a combination of regional sea level and local land level).

Methods for assessing and combining these parameters are shown in Equation 2.1 (Current ASCE) and Equation 2.2 (Future ASCE) and in Figure 3.1 and have been widely used in New Zealand since Gibb (1998) and are used in most contemporary assessments. This model as was used by T+T (2017) which the peer review panel (Kenderdine et al., 2016) found generally acceptable once the methods of component derivation were refined.

$$Current\ ASCE_{Beach} = ST + DS \tag{3.1}$$

$$Future\ ASCE_{Beach} = (LT\ \times T) + SL + ST + DS \tag{3.2}$$

Where:

ST = Short-term changes in horizontal shoreline position related to storm erosion due to singular or a cluster of storm events or short-term fluctuations in sediment supply and demand, beach rotation and changes in wave climate (m).

DS = Dune stability allowance. This is the horizontal distance from the base of the eroded dune to the dune crest at a stable angle of repose (m).

LT = Long-term erosion rate of horizontal shoreline movement (m/year).

T = Timeframe (years).

SL = Horizontal shoreline retreat caused by increased in mean sea level (m).

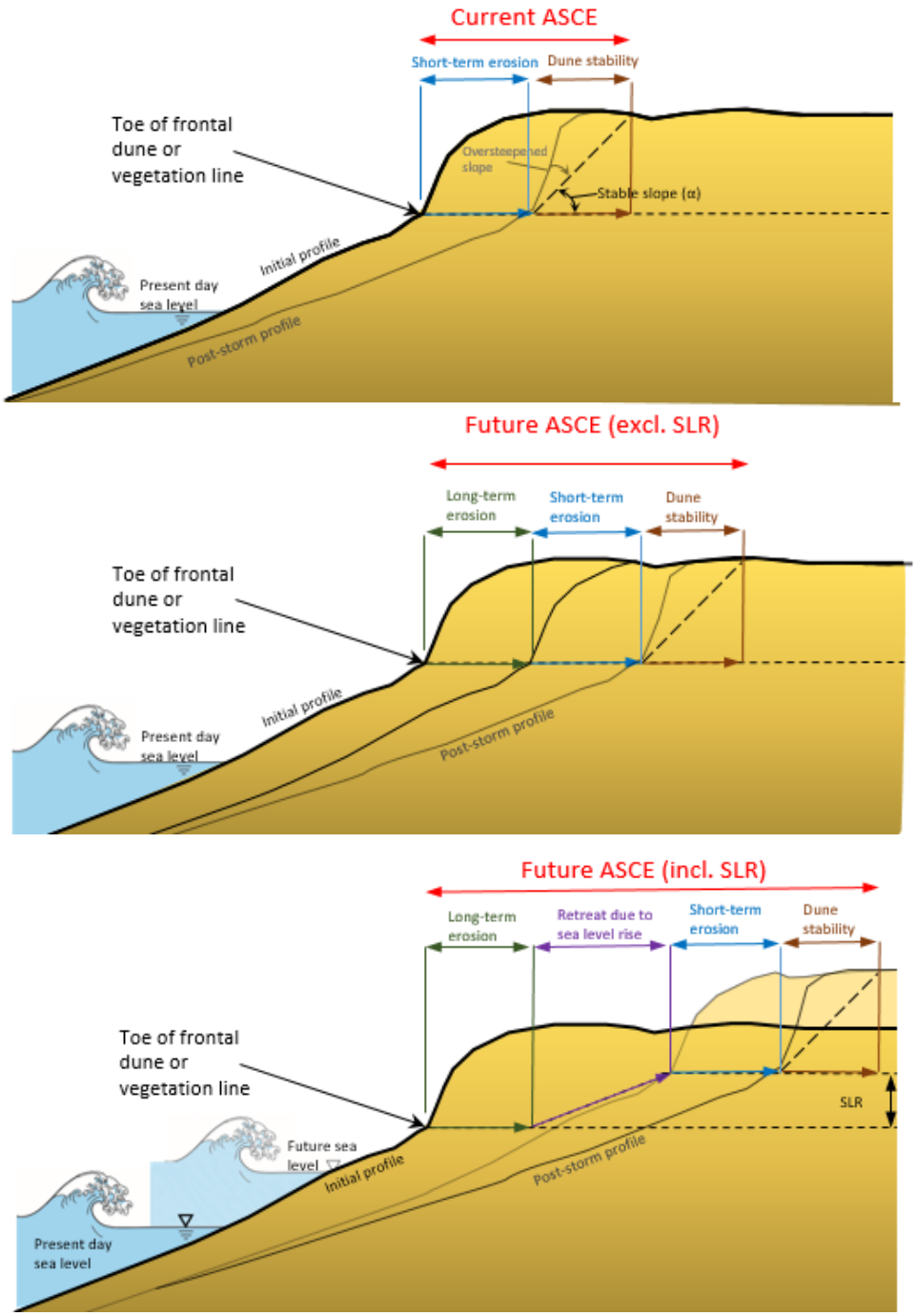

Figure 3.1: Definition sketch for Areas Susceptible to Coastal Erosion on open coast sand beach shoreline.

3.1.2 Mixed sand gravel beaches

Erosion processes along mixed sand gravel beaches can be complex and differ depending on the proportion of sand and gravel. Some gravel barriers can be dominated by erosion due to rollover, where the barrier is overwashed by storm waves and subsequently gravel is shifted landward. This process occurs at the southern end of Kaitorete Spit. However, along majority of Kaitorete Spit erosion is dominated by storm waves moving sediment offshore from the beach face. The ASCE for the mixed sand gravel beach along Kaitorete Spit has been established from the cumulative effect of four main parameters as shown in Figure 3.2 and Equation 2.3 and 2.4.

$$Current\ ASCE_{Gravel} = ST + SS \quad (3.3)$$

$$Future\ ASCE_{Gravel} = (LT\ \times T) + SL + ST + SS \quad (3.4)$$

Where:

ST	=	Short-term changes in horizontal shoreline position related to storm erosion.
SS	=	Slope stability.
LT	=	Long term rate of horizontal shoreline movement (m/yr).
T	=	Timeframe (years).
SL	=	Horizontal shoreline retreat caused by increased mean sea level (m).

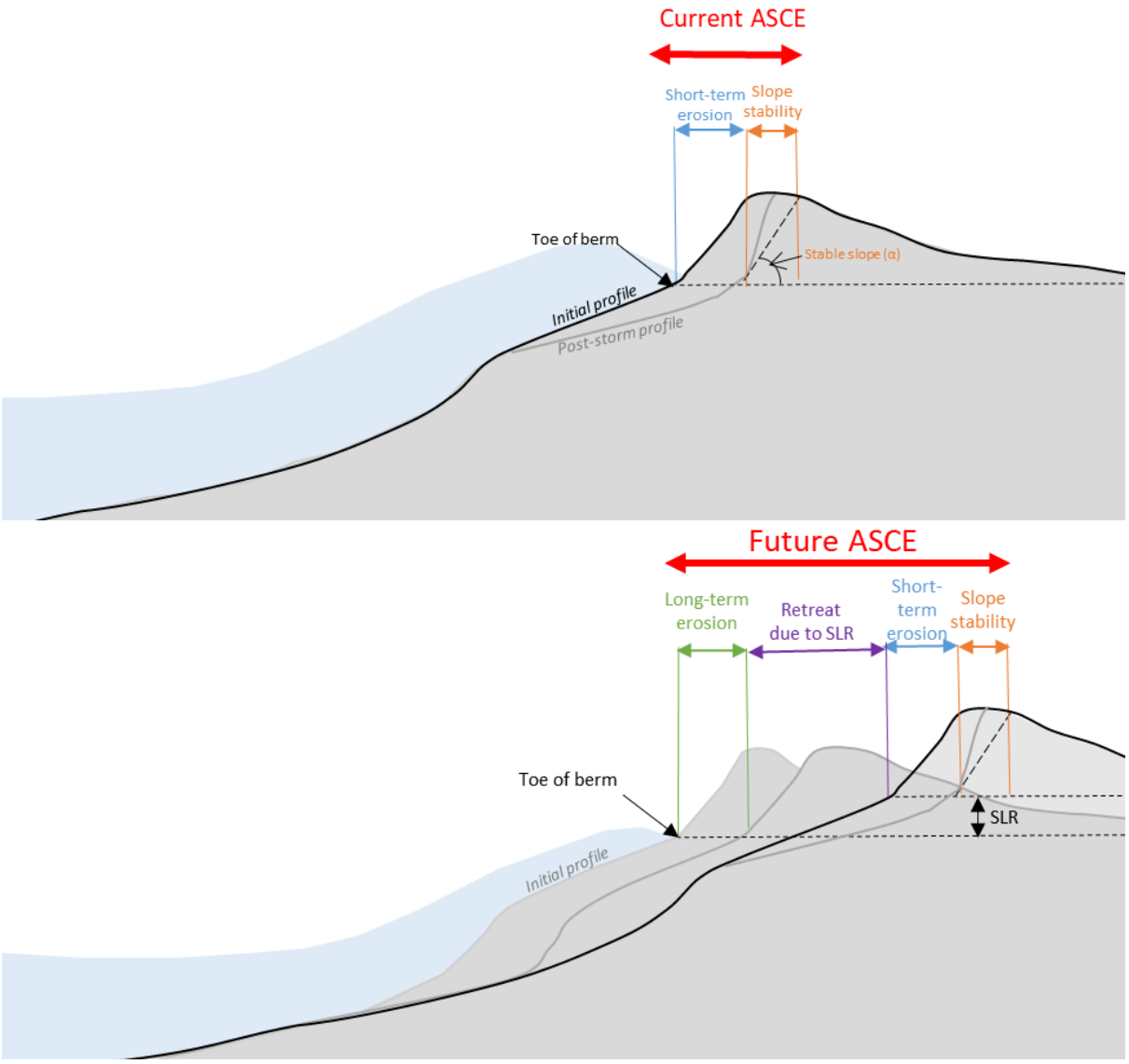

Figure 3.2: Definition sketch for Areas Susceptible to Coastal Erosion on mixed sand gravel beaches.

3.1.3 Banks

Banks, comprising weakly consolidated materials generally along estuary and sheltered harbour environments, are not able to rebuild following periods of erosion but rather are subject to a one-way process of retreat. Coastal erosion of harbour banks typically has two components:

- **Toe erosion**

  A gradual retreat of the bank toe caused by weathering, marine and bio-erosion processes. This retreat will be affected by global process such as SLR and potential increases in soil moisture.

- **Slope instability**

  Episodic instability events are predominately due to the decrease in material properties of the bank or yielding along a geological structure. Instability causes the slope to flatten to an angle under which it is 'stable'. Slope instabilities are influenced by processes that erode and destabilise the bank toe, including marine processes, weathering and biological erosion or change the stress within the slope.

The conceptual model for bank erosion is shown in Equations 2.5 and 2.6 and in Figure 3.3. This is the same model as was used by T+T (2017) which the peer review panel found generally acceptable after modifications.

$$Current\ ASCE_{Bank} = (H_C/tan\alpha) \quad (3.5)$$

$$Future\ ASCE_{Bank} = (LT\ x\ T)\ x\ SL + (HC/tan\alpha) \quad (3.6)$$

Where:

H_C = Height (m) of bank.

α = The characteristic stable slope angle (°).

LT = Long-term retreat (regression rate), (m/year).

SL = Factor for the potential increase in future long-term retreat due to SLR effects.

T = Timeframe over which erosion occurs (years).

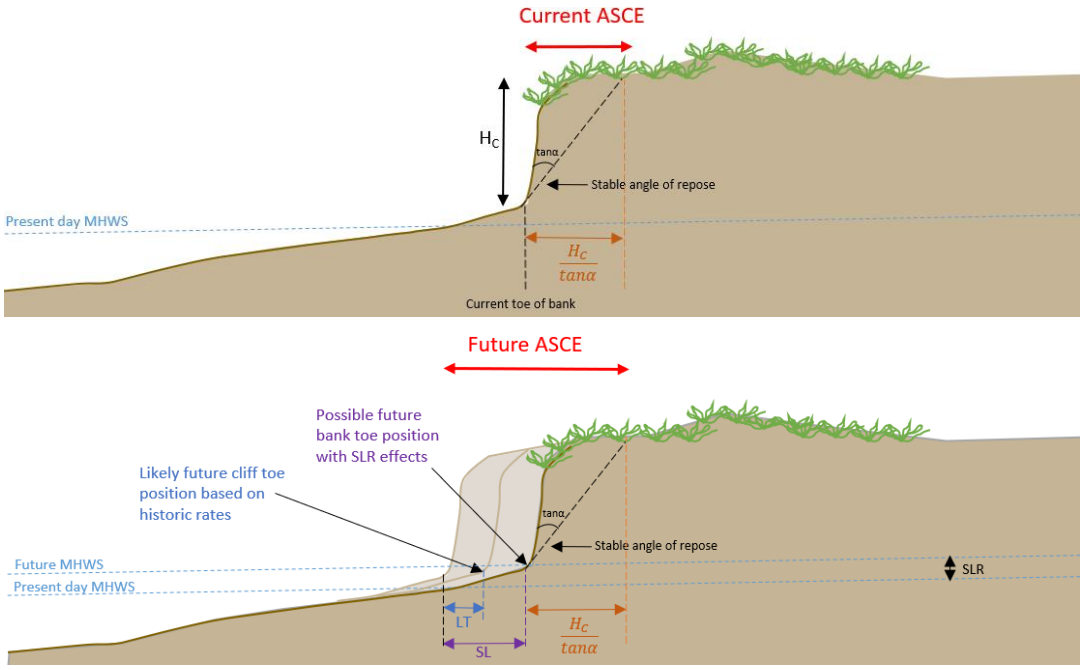

Figure 3.3: Definition sketch for Areas Susceptible to Coastal Erosion on bank coastlines.

3.1.4 Hard cliffs

Cliffed coasts around Banks Peninsula typically comprised of harder volcanic rock and like the bank shorelines are not able to rebuild following periods of erosion but rather are subject to a one-way process of retreat.

Due to the scale of assessment and minimal coastal erosion rates along the volcanic cliffs around Christchurch District, the cliffs have been assessed based on a simplified conceptual model (Figure 3.4). The model identifies the steep coastal edge which is potentially unstable due to coastal processes (assumed to be 1(H):1(V) and includes a setback which accounts for a range of factors including the physical scale of potential cliff failure mechanisms, long-term toe erosion and precision limitations involved with defining the unstable slope area. Where the coastal cliff edge is flatter than 1(H):1(V), a setback, based on the upper ASCE calculated for harbour beaches and banks, has been applied from the coastal edge. While this method is not a detailed cliff projection method, it is suitable for a regional coastal hazard screening assessment.

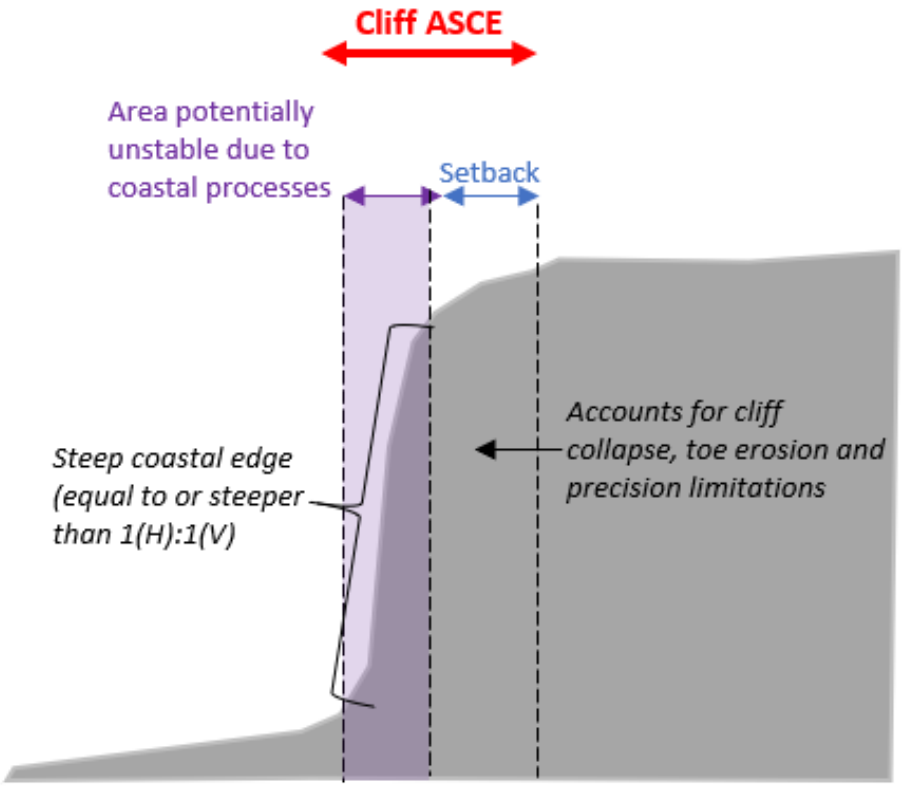

Figure 3.4: Conceptual model for ASCE along hard cliff shorelines.

3.1.5 Protected shorelines

Coastal erosion protection structures around Christchurch City district have been classified into three different categories:

Class 1 – Significantly modified shorelines

There are three locations where the shoreline has been significantly modified with land reclamation and hard protection structures. These locations include the southern shore of the Avon-Heathcote estuary, Sumner Beach, Lyttelton Port and Akaroa township (refer Figure 3.5).

These structures (or previous iterations) have generally been present since at least the 1940s. Because these shoreline modifications are so extensive and have been in place for so long, it has not

been feasible to use past observations of erosion rates to estimate what the long-term erosion rates would be in the absence of structures so a different approach is required.

In many instances, significant development has since occurred behind these structures, which has historically relied upon the protection provided. Failure of these structures would likely cause significant disruption to the wider community or city. Considering these wider implications, the New Zealand Coastal Policy Statement recognises that hard protection structures may be the only practical means to protect existing infrastructure of national and regional importance, increasing the likelihood that the structures in some of these areas may be maintained or immediately repaired if damaged.

For the coastal erosion assessment, the current (short-term) hazard area represents the immediate hazard if the structure were to fail, considering the structure height and characteristic stable angle of fill material (i.e. 2H:1V) (Figure 3.7).

The future (long-term) hazard area has been set equivalent to the current hazard area, which would be the case if the structure was promptly repaired if damaged (Figure 3.7). However, if the protection structure fails and is not promptly repaired then it is likely the fill material will rapidly erode, and the shoreline will eventually move back towards its 'original' natural position (this scenario has not been modelled in this study but could be assessed in future as part of adaptation planning if relevant).

Figure 3.5: Extent of significantly modified shorelines.

Figure 3.6: Example of significantly modified shoreline in Akaroa.

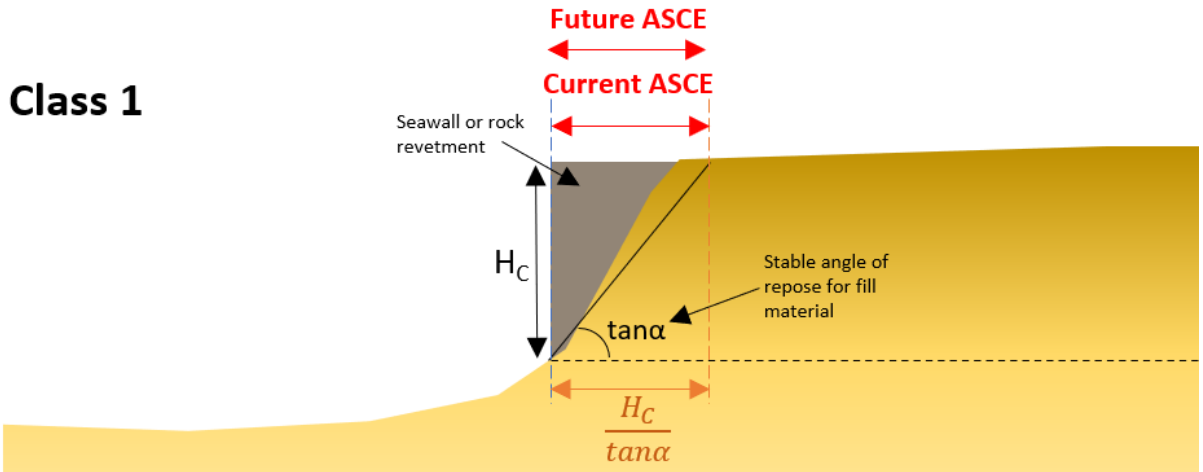

Figure 3.7: Schematic of ASCE for Class 1 structures.

Class 2 – Functional private and public structures

These include functional, consented private structures and functional public structures. The consent status has been chosen as a key factor for the private structures as it relates to the legal ability to undertake repair if damaged and reflects on the degree of engineering involved in their design and construction. If they fail or are damaged, these may be able to be repaired during the consent term. However, it is unknown if they will be re-consented at the end of consent term so the degree of protection in the long term is uncertain. For public structures, the consent status is a less relevant classification factor because many of these structures pre-date the Resource Management Act and so will not have resource consent for their construction.

Another relevant consideration is the condition of the structure, however this was more difficult to consistently determine and incorporate into the assessment. For example, there are consented private structures along the eastern margin of Avon-Heathcote estuary which have been damaged, resulting in some no longer being functional.

Figure 3.8: Example of functional public structures (left) Corsair Bay (right) French Farm Bay.

Class 3 - Informal, non-consented structures

These include all non-consented and/or informal structures. They may have limited effectiveness at reducing erosion and are less likely to be repaired if damaged. This means the long-term erosion (and effects of sea level rise) could be similar to the adjacent unprotected coast (i.e. as if the structure was not present).

Figure 3.9: Example of informal, non-consented structures along the Avon-Heathcote Estuary.

It is not possible to reliably distinguish between Class 2 and Class 3 structures using the currently-available information, and it would still be difficult even if more detailed information was collated. Long-term erosion effects may also be similar for both classes. Therefore Class 2 and Class 3 structures are treated in the same way for this study.

Known structures are shown on the hazard map for context. However, the impact of Class 2 and 3 structures on future erosion was not considered in the assessment and the mapped erosion hazard was based on the characteristics of the adjacent unprotected shoreline (i.e. as if the structure was not present). This allows the long-term importance of these structures to be considered as part of adaptation planning, acknowledging they may provide some degree of protection against erosion now and into the future but also showing what could be at risk if they were to fail.

3.2 Baseline derivation

The baseline is the shoreline to which ASCE values are referenced and mapped from. This is the dune toe or seaward edge of vegetation for beach shorelines, the cliff/bank toe for consolidated shorelines and the toe of the structure for Class 1 structures (significantly modified shorelines). The

baseline has been derived using a combination of the most recently available LiDAR (2018) and most recently available aerial imagery (2019).

3.3 Defining coastal behaviour cells

Each site has been divided into coastal cells based on the shoreline composition and behaviour which can influence the resultant hazard. Factors which may influence the behaviour of a cell include:

- Morphology and lithology.
- Exposure to waves.
- Profile geometry.
- Backshore elevation.
- Historical shoreline trends.

3.4 Assessment level

Coastal erosion hazard across the Christchurch City District has been assessed at either a regional hazard screening level or a detailed level. The adopted level of assessment varies to suit the context and available information. Report 1 includes further detail on the rationale of the assessment level for each area. The two approaches are outlined below.

3.4.1 Regional hazard screening (deterministic approach)

Regional hazard screening is intended to identify areas that are potentially exposed to coastal hazards and show where more detailed hazard (and eventually risk and vulnerability) assessments should be focussed. Regional assessments are typically undertaken at a coarse spatial resolution, are often based on limited data and therefore derive simpler or generic hazard component values. The hazard values are assessed by combining the individual parameters using a deterministic ('building-block') approach, with uncertainty incorporated into the derived values. A deterministic calculation assumes fixed values for the input parameters. Each individual parameter is usually selected conservatively (to give a less favourable outcome than average), which means that when these multiple unfavourable assumptions are added it represents an "upper end" scenario (Figure 3.10).

The spatial scale of this level of assessment is relatively coarse (i.e. 1 – 10km resolution), with mapping appropriate for the level of detail and spatial scale.

3.4.2 Detailed hazard assessment (probabilistic approach)

Detailed hazard assessments are intended to provide a more thorough understanding of the coastal processes, uncertainties, and the effects of different future sea level rise scenarios. Therefore, the individual processes, likelihood of occurrence, uncertainty and inter-relationship should be more thoroughly understood and combined in a robust manner (e.g. probabilistic approach) (Cowell et al. 2006, Shand et al. 2015, T+T, 2017).

A probabilistic calculation assumes a range of values for each input parameter with probability distribution functions in the form of either a normal distribution, a triangular distribution or extreme value distribution. A normal distribution is used where sufficient data is available and this data is (near) normally distributed. A triangular distribution is used where limited data is available. The triangular distribution contains the best estimate (mode), lower and upper bounds of the component based on either available data or heuristic reasoning based on experience. An extreme value distribution has been used where extreme values are not included in the available data and should be included.

Probability distributions constructed for each component are randomly sampled and the extracted values used to define a potential ASCE distance. This process is repeated 10,000 times using a Monte Carlo technique. An example of a probability distribution of the resultant ASCE width is shown in Figure 3.10.

The probabilistic approach provides both a "best-estimate" and an understanding of the potential range of outcomes as well as a more transparent way of capturing and presenting statistical viability and uncertainty.

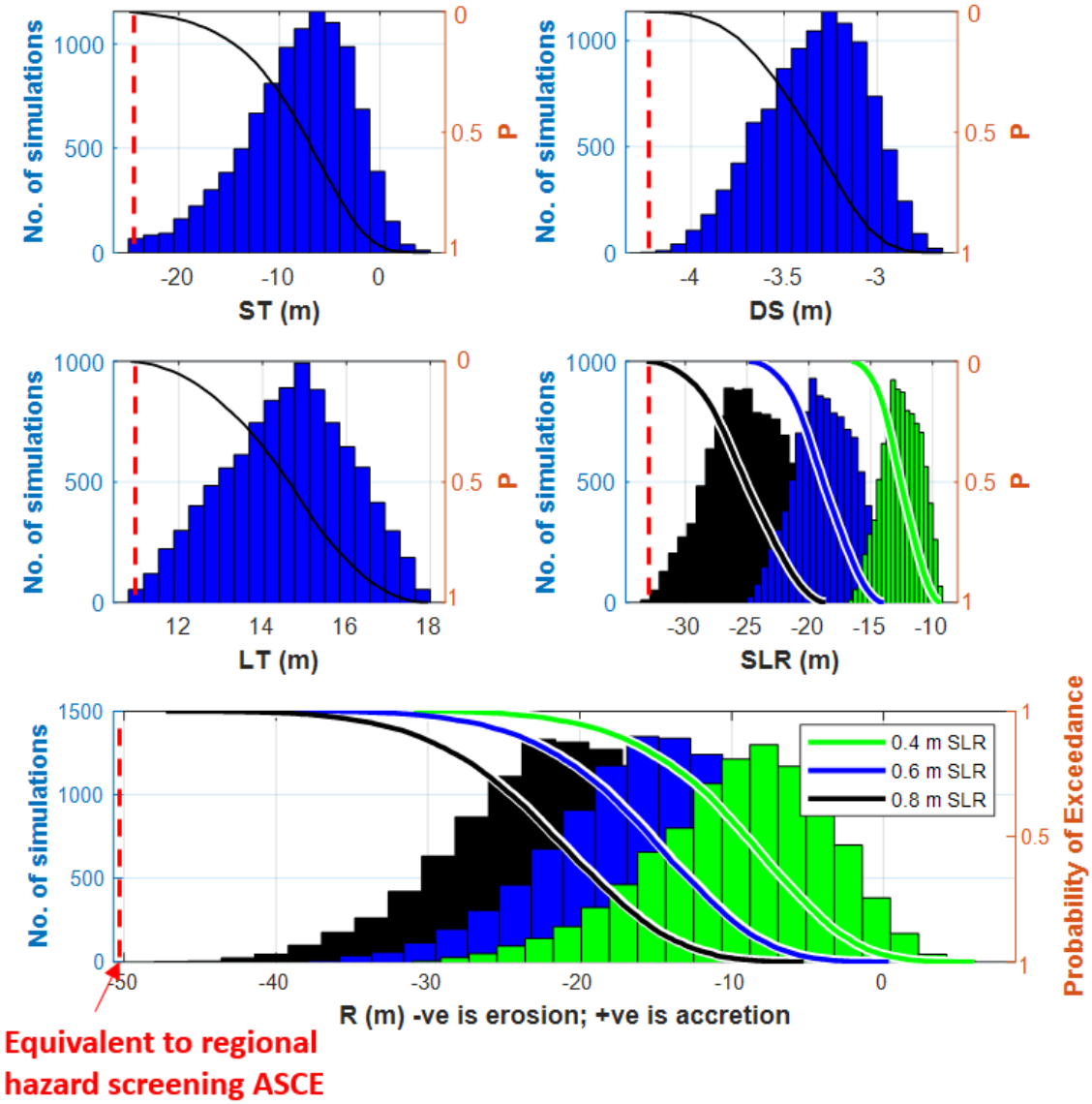

Figure 3.10: Example of component and ASCE histogram cumulative distribution functions of parameter samples and resultant ASCE distances for the probabilistic approach. Red dashed lines demonstrate upper bound component values added together to get the resultant regional hazard screening (deterministic) ASCE.

3.4.3 Spatial extent

3.4.3.1 Detailed erosion assessment

Detailed erosion assessment has been completed for the Christchurch open coast beaches and selected areas within Lyttelton and Akaroa Harbours. Due to different data availability slightly different levels of detailed probabilistic assessment have been completed for each area.

Full probabilistic approach

Where there is sufficient data including historic shorelines and beach profile datasets, full probabilistic analysis has been completed, including statistical analysis of shoreline position and profiles. The full probabilistic approach has been completed for the Christchurch open coast beaches:

- Waimakariri to Southshore.
- Sumner.
- Taylors Mistake.

Quasi-probabilistic approach

Where there are data limitations (i.e. no beach profiles or limited historic aerial imagery), a more detailed assessment was still feasible, however some generic assumptions have been made around parameter bounds, including short term and long term components. This approach has been adopted for the following areas:

- Avon-Heathcote estuary.
- Beach and bank shorelines along existing major settlements within Lyttelton and Akaroa harbours (refer Figure 3.11 and Figure 3.12).

3.4.3.2 Regional hazard screening erosion assessment

The regional hazard screening assessment includes a deterministic approach where the upper bound parameters are adopted for each cell. Regional hazard screening assessment has been completed for the following areas:

- All hard cliffs.
- Beaches and banks within Lyttelton and Akaroa harbours away from major settlements.
- All beaches and banks around Outer Banks Peninsula.
- Kaitorete Spit.

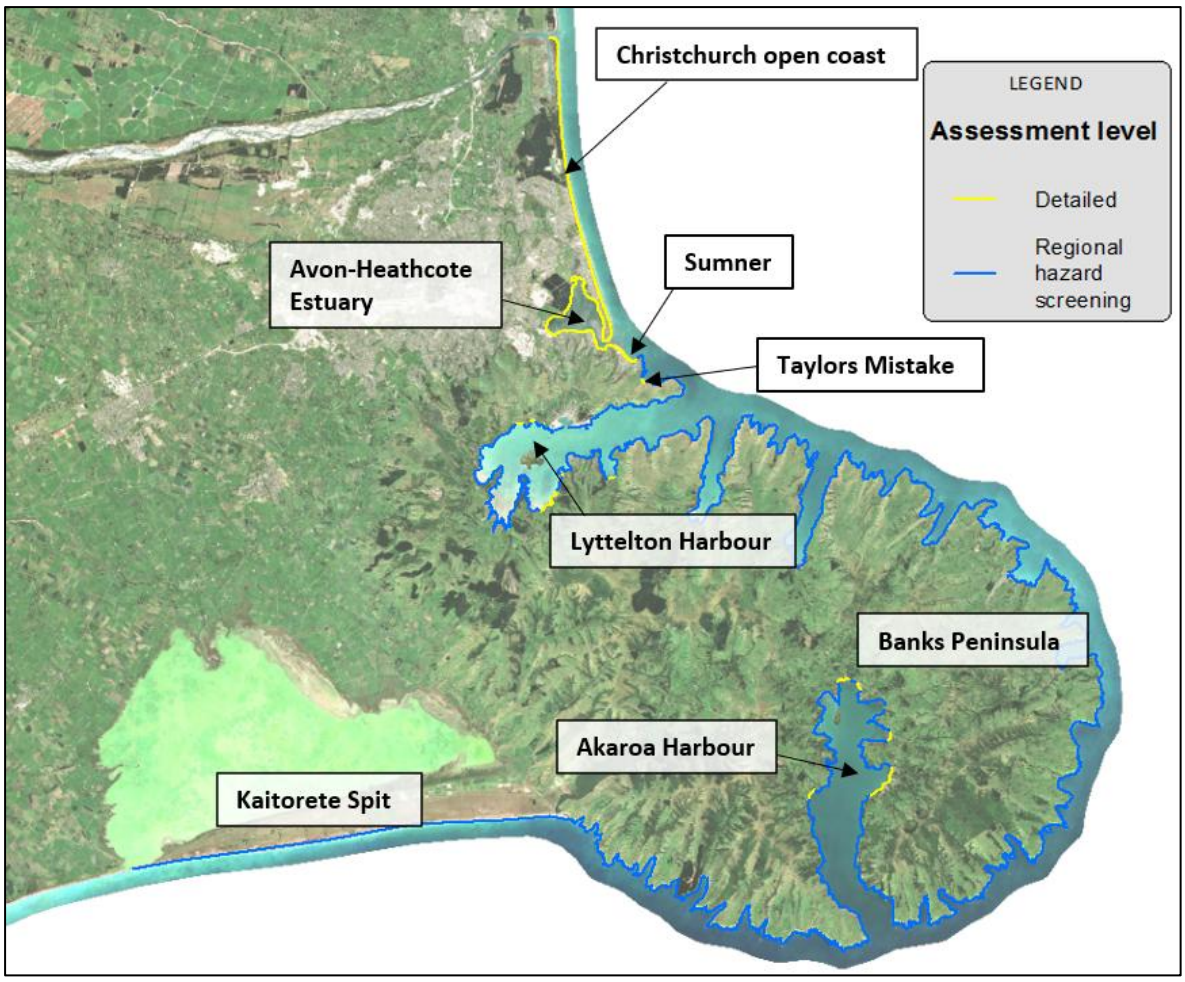

Figure 3.11: Christchurch district showing extents and level of detail for the coastal erosion assessment.

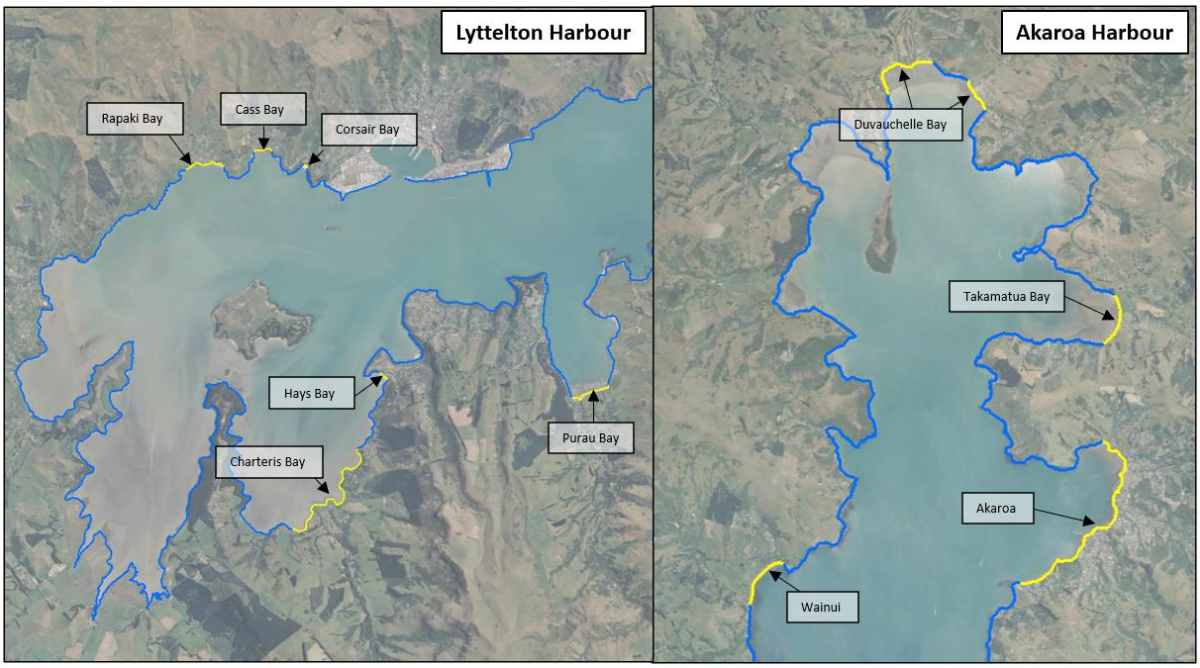

Figure 3.12: Location of detailed (yellow) and regional hazard screening (blue) coastal erosion assessments within Lyttelton and Akaroa Harbours.

3.5 Scenarios

The MfE (2017) guidance recommends either direct usage of RCP scenarios or increments of sea level rise to inform adaptation planning. For this assessment, increments of sea level rise have been adopted which can be aligned with timeframes, approximate RCP scenarios and allowance for vertical land movement. Erosion assessment scenarios are summarised in Table 3.1.

Table 3.1: Erosion assessment scenarios

Assessment	Timeframe	Relative sea level increment[2] (m)	Likelihood scenarios[3]	Sediment supply reaching beaches[4]
Detailed[1]	Current – 2030	0	The following range of likelihoods mapped as a gradient: Pmin P99% P95% P85% P66% P50% P33% P15% P5% P1% Pmax	N/A
	2050	+0.2		Scenario 1 No change to sediment supply
		+0.4		
	2080	+0.4		
		+0.6		
		+0.8		
	2130	+0.4		
		+0.6		
		+0.8		
		+1.0		
		+1.2		
		+1.5		
	2150	+2.0		
	2130	+1.5		Scenario 2 Reduced supply (11% reduction)
	2130	+1.5		Scenario 3 (Increased supply 28% increase)
Regional screening assessment	Current – 2030	0	Upper bound (assumed)	-
	2080	+0.4		
	2130	+0.4		
		+1.5		

[1] Both full probabilistic and quasi-probabilistic.

[2] Relative sea level combines the effect of both rising sea level and allowance for vertical land movement. Increments are specified relative to 2020 sea level.

[3] This provides an indication of the probability of a modelled erosion extent occurring for a particular storm event. For example, land mapped within the P95% extent is very likely to be eroded in the type of event being modelled, whereas erosion of land within the P5% extent is very unlikely (but not impossible) in that type of event.

[4] The sediment supply reaching the beaches depends on both the amount of sediment discharged by the Waimakariri River, and the amount of this sediment which is transported southwards along the coast. This was assessed for Christchurch open coast beaches only, as it is not relevant for other beaches.

3.5.1 Long-term vertical land movement

MfE (2017) recommends consideration of vertical land movement (VLM), such as uplift or subsidence caused by creeping tectonic plates, because changes in land level can accelerate or decelerate the local effects of a rise in absolute sea level. It is recommended that any significant long-term VLM (>10 years) should be factored into local predictions of future relative sea level[1].

Long-term records of VLM are limited for Christchurch region. The recent work completed by Pearson et al (2019) shows notable subsidence on the eastern side of Christchurch (-0.2 to -0.7 mm/yr) (see Section 2.8). However, as the ground level monitoring covers only a short period after the Canterbury earthquakes (2015 to 2019) and has limited spatial coverage across the city, this data does not provide a reliable basis for extrapolating VLM for decades into the future or defining a pattern of movement across the district.

Therefore, rather than "locking in" a specific VLM rate and spatial pattern in the assessment, land movement will be treated as another source of uncertainty in the prediction of future relative sea level at a particular location. This means that different combinations of absolute sea level rise and local land subsidence can be explored, to give a better understanding of the range of possible future conditions.

The incremental analysis approach is preferred over selection of a specific combination of timeframe, sea level and vertical land movement as it provides a more nuanced understanding of potential effects over a range of future conditions, which is more useful for adaptation planning purposes. An additional high-end scenario has been included in each series of erosion and inundation analyses to provide sufficient "headroom" for the most unfavourable combinations of absolute sea level rise and vertical land movement estimates to be considered.

3.6 Mapping methodology

3.6.1 Regional hazard screening maps

For the regional hazard screening sites, where there is a single ASCE distance for each assessment scenario, the ASCE have been mapped as a polygon, offset horizontally from the baseline. The width of the polygon represents the calculated ASCE distance. For the cliff shorelines, the ASCE has been mapped based on the method described in Section 4.6.5. An example of the mapping for the regional hazard screening ASCE is provided in Figure 3.13.

[1] Relative sea level is measured relative to a fixed surface point on land (e.g. a tide gauge), whereas absolute sea level is measured relative to the centre of the earth. Any changes in relative sea level at a particular location represent the combined effect of vertical land movement and changes in absolute sea level. It is this relative sea level rise which is most relevant for community adaptation planning.

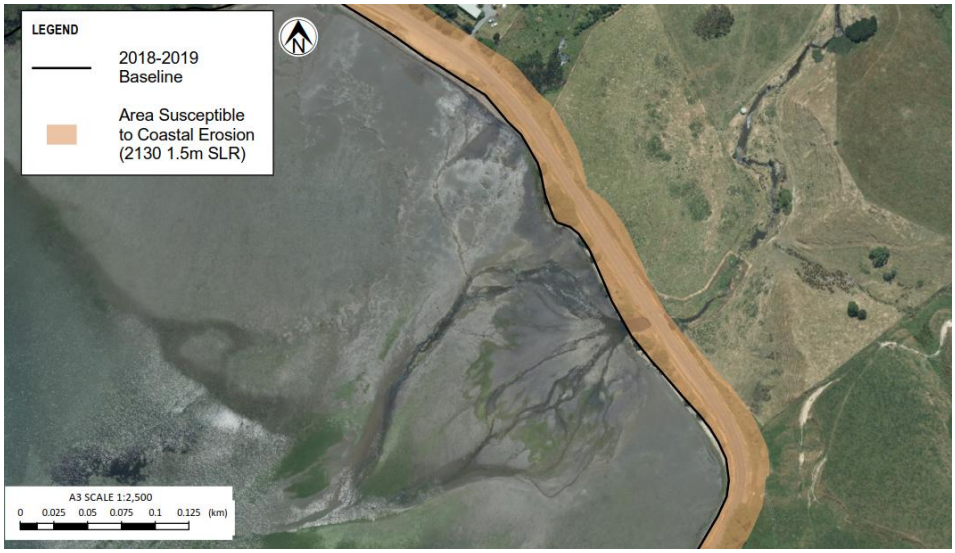

Figure 3.13: Example ASCE map for the regional hazard screening sites.

3.6.2 Detailed hazard assessment maps

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

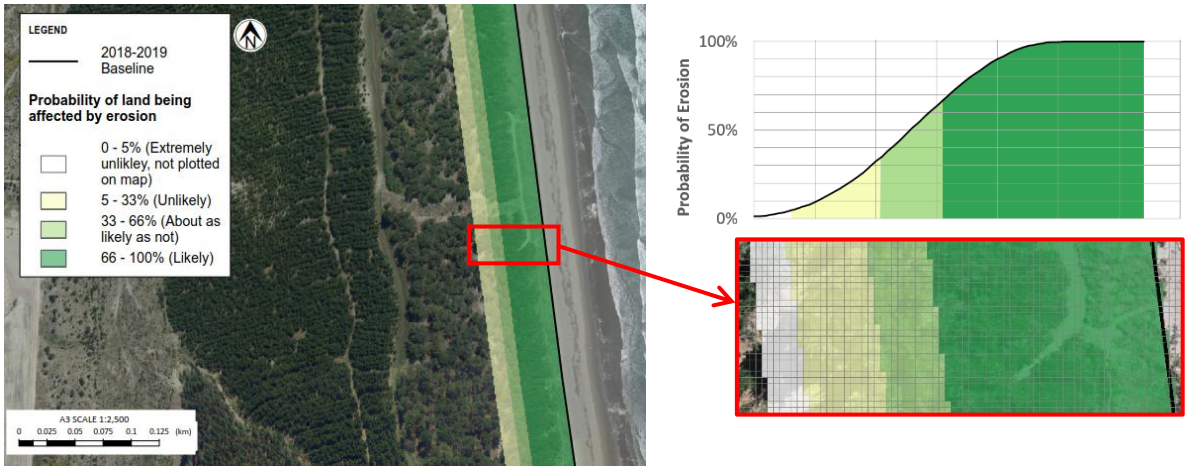

Figure 3.14: Example of raster ASCE mapped for a detailed site showing spatial extent of erosion and corresponding probabilities of occurrence.

4 Coastal erosion analysis

4.1 Christchurch open coast

The Christchurch open coast is located at the southern end of Pegasus Bay on the eastern edge of extensive gravel outwash plains derived from the Southern Alps. At the northern extent, is the mouth of the wide, braided, Waimakariri River and at the southern extent is the inlet to the Avon-Heathcote Estuary. The shoreline predominately faces east and is sheltered from southerly swell due to the presence of Banks Peninsula to the south.

At the end of the last glaciation period, sea level rose several meters until about 6000 years ago, when it reached approximately the present-day level. Since then, the sea level has been relatively static, however, the coastline has prograded seaward several kilometres as a result of fluvial and marine deposition (Brown and Weeber, 1992). Succession of beach deposits, sand dunes, estuaries and lagoons has since occurred, and the current eastern suburbs of Christchurch are located on extensive areas of sand dunes and old dune ridges.

Kirk (1987) suggests that this progradation material has been predominantly supplied by material from south of Banks Peninsula, which has moved northward around the peninsula and onto the Banner Bank before being reworked landward. Kirk considers that this reworking has now ceased, and that the coastline has reached equilibrium with sand supplied by the Waimakariri River replacing sand removed offshore and south of the open coast by current coastal processes.

The present-day shoreline generally has similar morphology with a dune backshore and a relatively flat, fine sand beach (Figure 4.1). There are several locations where the backshore has been modified including the Brighton Pier seawall and North New Brighton seawall.

Figure 4.1: Site photos along Christchurch open coast. (Top left) Erosion scarp on Southshore Spit, (top right) vegetated, accreting dunes near Waimairi, (bottom left) dune planting and fencing near Waimairi, (bottom right) seawall at New Brighton.

4.1.1 Cell splits

The Christchurch open coast has been divided into 14 coastal cells (1 to 14) based on the shoreline behaviour which can influence the resultant hazard (Figure 4.2). Cell 1 includes the northern tip of the Brooklands Spit which is influenced by the dynamics of the Waimakariri River mouth. Cell 14 includes the southern tip of the Southshore Spit which is influenced by dynamics of the Avon-Heathcote estuary mouth. Cells 7 and 9 have partially modified dunes due to the presence of the

North New Brighton and New Brighton seawalls. A key factor influencing the remaining cell splits is the variation in historic shoreline trends.

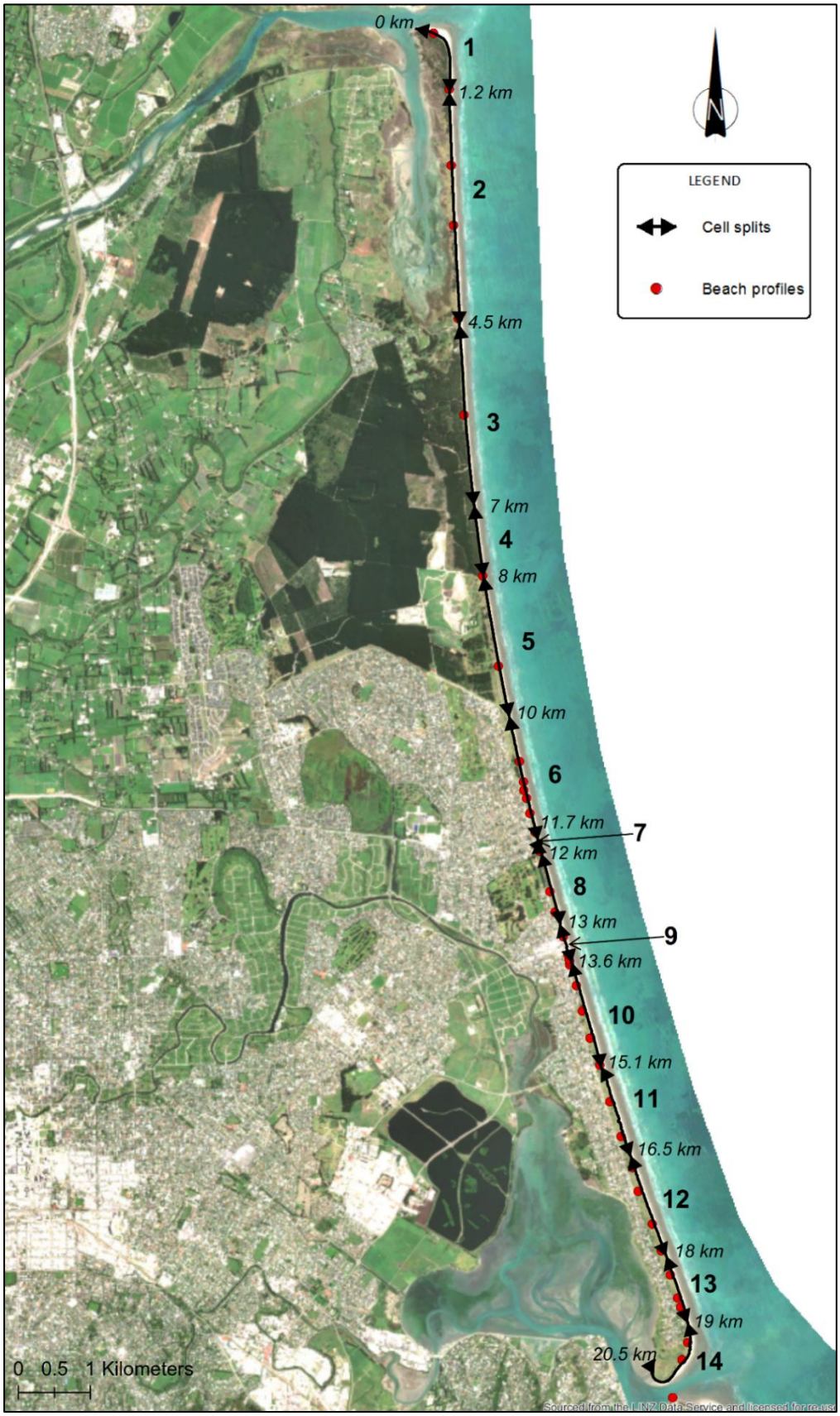

Figure 4.2: Overview of cell extents with cell numbers and chainages along the Christchurch open coast.

4.1.2 Short term component (ST)

Unconsolidated coastlines (beaches) undergo short-term cycles of storm-induced erosion (i.e. storm cut) due to single or clusters of storms followed by periods of re-building. The erosional component of these cycles (i.e. landward movements) needs to be accounted for in the coastal hazard assessment.

4.1.2.1 Approach

The short-term component along the Christchurch open coast has been assessed based on the beach profile datasets (see Appendix A). The profile data provides information on both the long-term movement of the dune toe (see Section 4.1.4) as well as the short-term storm fluctuations.

Based on visual inspection of the beach profiles the dune toe level (i.e. the baseline to which ASCE is offset from) was estimated to be around 2.5 m RL. The short-term component has been quantified using statistical analysis of the inter-survey storm cut distances. The inter-survey storm cut distance is the horizontal landward retreat distance measured between two consecutive surveys (Figure 4.3). We note that due to the relatively long period between surveys the distances may not represent the maximum excursion that may have occurred between the time periods. However, the data set provides the best source of information to analyse.

Figure 4.4 shows that while there has been net accretion at the dune toe, the dune toe position fluctuates over time with periods of erosion and accretion. The profile data shows several stormy periods where the dune toe has retreated a significant distance landward (i.e. up to 15 m retreat at profile CCC1041 during the 1992 storms) (Figure 4.5). Following the storm events, the dune tends to show gradual recovery and accretion (Figure 4.4).

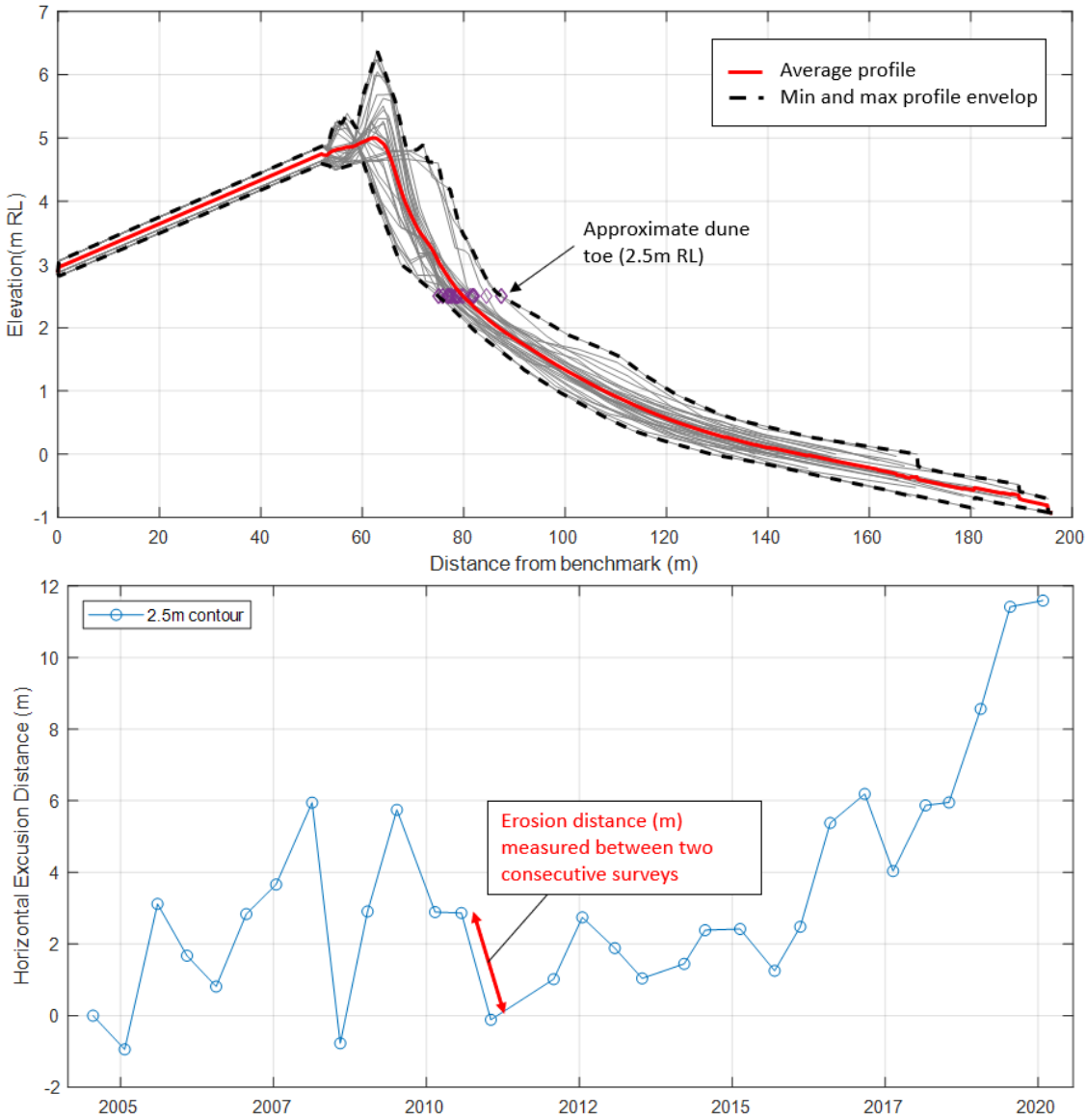

Figure 4.3: Example of beach profile data (CCC0856) used to assess the short term component. (Top) Beach profiles showing the average profile and envelop of change. The dune toe position (2.5 m RL) contour is marked with a purple diamond on each profile. (Bottom) horizontal excursion distance measured at the 2.5 m RL contour.

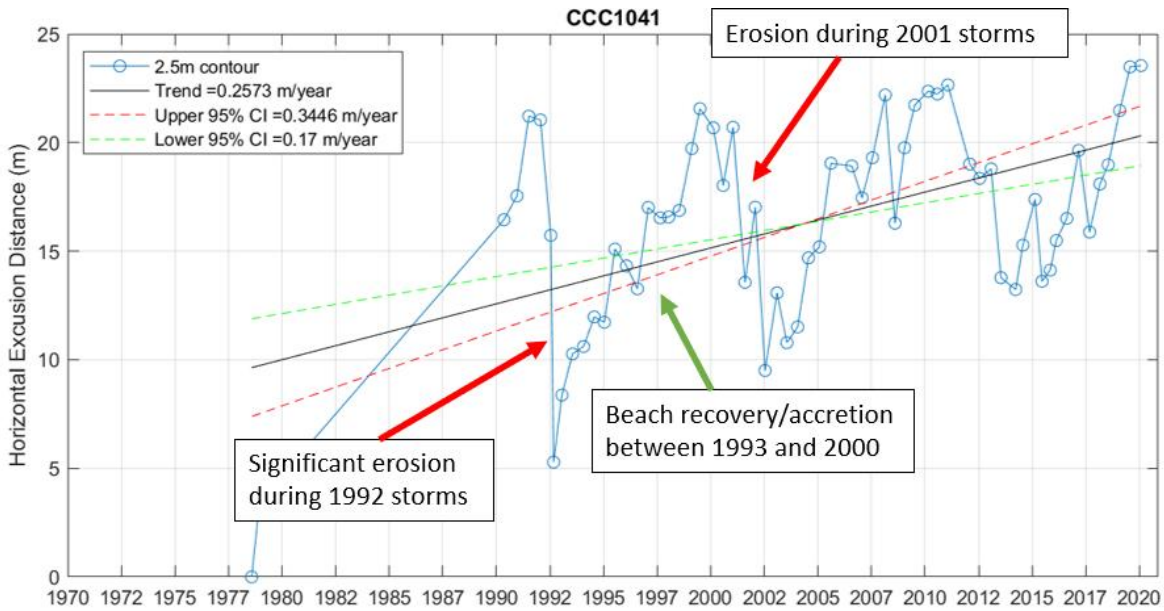

Figure 4.4: Horizontal excursion plot for profile CCC1041 showing significant retreat during stormy years followed by periods of gradual beach recovery and accretion.

Figure 4.5: Photograph taken in vicinity of CCC1041 in August 1992, showing the post-storm dune (source: Justin Cope, ECan).

The mean and maximum inter-survey storm cut distances for all profiles along the open coast have been derived and are shown in Table 4.1. A full set of excursion distances and profile plots for all profiles is presented in Appendix A. Figure 4.6 shows a temporal-spatial plot of the dune toe movements for each alongshore beach profile and for each survey date. The dune toe movements measured at profile CCC0889 are likely influenced by the backshore seawall and has therefore been omitted from the analysis. The matrix shows 4 major storm events/periods (1992, 2001, 2008 and 2014) where erosion was measured at almost each profile along the coast (Figure 4.6). There are two events (2015 and 2017) where only the northern end of the coast has shown erosion with minimal movement at the southern end. While the beach generally has similar exposure, the response to storms may differ slightly at the northern and southern ends.

Based on spatial variation in inter-survey distances, the coast can be broadly divided into four areas: the northern end (Cells 2 to 4), the southern end (Cells 5 to 13) and the distal ends of the Brooklands spit (Cell 1) and the Southshore spit (Cell 14). Average storm cut distances appear to be slightly larger at the northern end of the shoreline compared to the southern end. The distal ends of the two

spits (profiles C2200 and C0271) show large fluctuations, with up to -19.4 m inter-survey storm cut at the Brooklands Spit and -38.4 m inter-survey storm cut at the end of Southshore Spit (Table 4.1).

Table 4.1: Mean and maximum inter-survey storm cut distances for each beach profile

Cell	Profile	Chainage (km)	Mean inter-survey storm cut (m)	Maximum inter-survey storm cut (m)
1	C2200	0.25	-4.9	-19.4
2	C2070	1.15	-5.1	-17.8
	C1972	2.2	-4.9	-13.1
	C1891	3.06	-3.7	-10.4
	C1755	4.4	-4.0	-15.8
3	C1565	5.75	-3.4	-10.5
4	C1400	8.0	-3.2	-13.8
5	C1273	9.3	-3.3	-12.2
6	C1130	10.65	-2.9	-11.2
	C1111	10.95	-2.2	-4.3
	C1100	11.05	-2.4	-9.6
	C1086	11.18	-2.8	-9.8
	C1065	11.4	-2.1	-10.7
	C1041	11.67	-2.9	-10.4
7	C1011	11.95	-3.7	-11.3
8	C0952	12.53	-2.4	-10.6
	C0924	12.83	-2.2	-8.8
9	C0889	13.2	-5.6	-22.2
	C0863	13.48	-3.3	-8.0
	C0856	13.55	-2.1	-6.1
	C0853	13.58	-1.9	-6.7
10	C0848	13.63	-2.7	-9.0
	C0815	13.93	-2.7	-13.5
	C0748	14.68	-2.3	-10.2
	C0703	15.08	-3.1	-10.7
11	C0650	15.6	-2.0	-12.9
	C0600	16.14	-2.5	-9.9
12	C0531	16.6	-2.1	-8.3
	C0513	16.94	-1.9	-9.4
	C0471	17.44	-2.4	-8.9
	C0431	17.83	-2.5	-8.0
13	C0396	18.18	-2.3	-15.4
	C0362	18.52	-2.3	-12.4
	C0350	18.65	-2.5	-9.8
14	C0300	19.15	-4.1	-11.6
	C0271	19.45	-6.1	-38.4

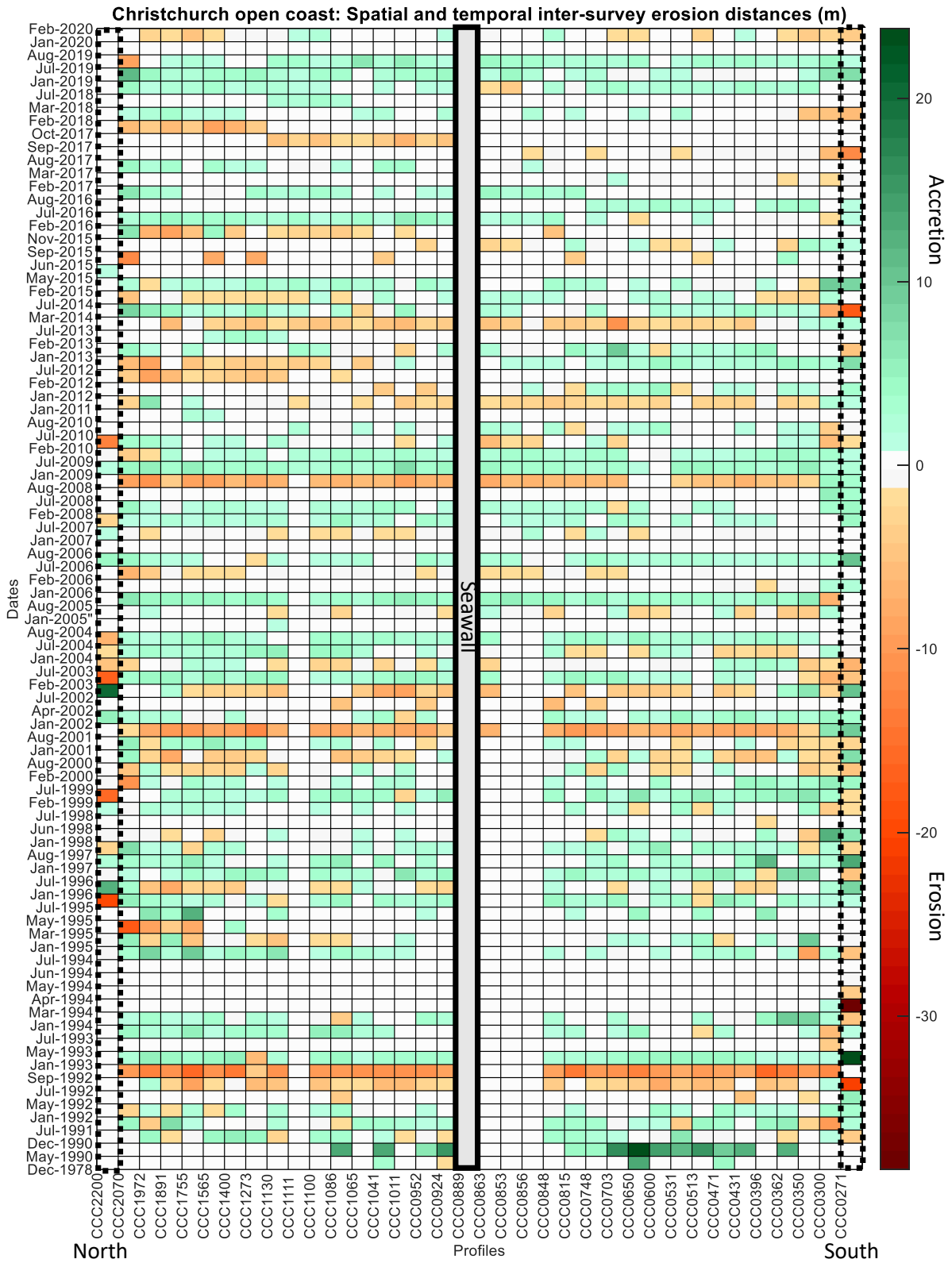

Figure 4.6: Matrix showing inter-survey changes in dune toe position for each profile through time. Dashed lines show the profiles influenced by spit dynamics at either end of the coast. Profile CCC0889 is influenced by the New Brighton seawall and therefore has been excluded from the analysis.

4.1.2.2 Adopted values and distribution

The inter-survey storm cut distances are based on a 30-year dataset of 6-monthly surveys, with maximum possible/extreme distance possibly not measured. In order to derive extreme values from a limited number of observations (i.e. 30 years of 6-monthly surveys), an extreme value analysis has been undertaken.

T+T (2017) reviewed a range of data selection methods including Peaks Over Threshold (POT) and Annual Maximum (AM) approaches. The POT method includes a threshold level (i.e. minimum storm cut distance) that can be used to increase the population size of shorter datasets and/or omit smaller events which may not belong to the same statistical population. The AM method selects the maximum inter-survey erosion distances for each year (i.e. if two surveys are carried out within a year, the largest inter-survey erosion distance is selected) within a time series and for a particular coastal cell. Note that as a result of inter-survey erosion distances the largest cumulative erosion across a series of storms (for example the 1992 storms) may not always be captured and therefore the resulting erosion may potentially be less than possible on an annual basis. The AM method was previously adopted in T+T (2017) and was agreed with the peer review panel and therefore has been adopted within this assessment.

At the distal end of the spits (Cells 1 and 14) there is limited profile data (i.e. one to two profiles). Due to the limited data points within these cells, the AM method is less appropriate as the resulting extreme value curve becomes skewed to the small, normal fluctuations that occur in beach position at the distal end of the spit and results in unrealistic storm cut values. Subsequently for Cells 1 and 14 the POT approach has been adopted for selecting storm cut distances. A threshold of -4 m was adopted for the spits, which is equivalent to approximately the average standard deviation of the open coast and represents the day-to-day fluctuations. Therefore, this approach would filter out the small day-to-day fluctuations and would result in a more realistic extreme value distribution of storm cut.

T+T (2017) tested a range of distributions and found a Generalized Extreme Value (GEV) Type 1 (Gumbel) distribution to have the best fit to the observed data. Therefore, for this assessment the Gumbel distribution has been adopted.

An example of the extreme value distribution for the inter-survey storm cut along the southern profiles is shown in Figure 4.7. Based on the distribution the 5 year ARI storm cut equates to -8 m and the 100 year ARI storm cut equates to -29 m (Figure 4.7). A summary of the extreme value distributions which have been adopted for the cells along the open coast is provided in Table 4.2. The 100 year ARI storm cut distance is also included for context.

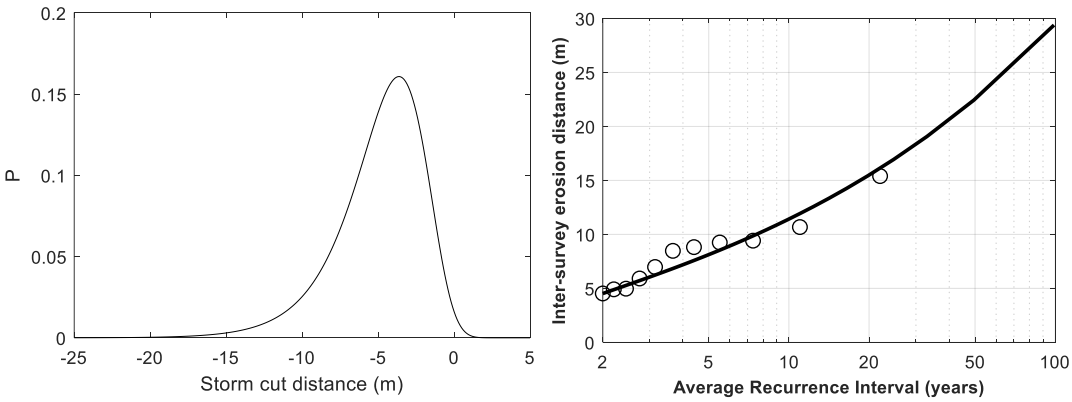

Figure 4.7: Example of extreme value distribution and curve for the profiles along the southern section of shoreline (Cells 9 to 13).

Table 4.2: Summary of extreme value distributions for inter-survey storm cut distances along the Christchurch open coast

Cells	Profiles	Mean alongshore inter-survey storm cut (μ) (m)	Shape parameter (σ)[1]	Resultant 100 year ARI storm cut (m)
1	Brooklands Spit (CCC200)	-11	7	-22
2 to 4	Northern profiles (CCC2070 to CCC1273)	-5.9	4.7	-22
5 to 13	Southern profiles (CCC1130 to CCC0300)	-3.6	2.3	-29
14	Southshore Spit (CC0271)	-5.5	1.9	-41

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

4.1.3 Dune stability (DS)

The dune stability factor delineates the area potentially susceptible to erosion landward of the erosion scarp. The parameter assumes that storm erosion results in an over-steepened scarp which must adjust to a stable angle of response for loose sand. The dune stability width is dependent on the height of the existing dune and the angle of repose for loose sand. The dune stability factor is outlined in Equation 4.1.

$$DS = \frac{H}{2(tan\alpha_{sand})} \tag{4.1}$$

Where H is the dune height from the eroded base to the crest and α_{sand} is the stable angle of repose for beach sand (ranging from 30 to 34 degrees). In reality, the formation of a talus slope at the toe will allow the scarp to stand at steeper slopes (unless subsequently removed), hence the dune height is divided by 2.

Dune heights were obtained from 2018-2019 LiDAR and checked against beach profile data. Dune crest elevations were extracted at 100 m intervals along the coast. The average dune toe elevation (2.5 m RL) was subtracted from the dune crest elevations, resulting in the dune height. Parameter bounds have been defined based on the variation in dune height within the coastal cell and potential range in stable angle of repose (Table 4.3 and Table 4.4).

Table 4.3: Dune stability component values

Cell	Dune stability component values		
	Lower (degrees)	Mode (degrees)	Upper (degrees)
1 to 14	30	32	34

Table 4.4: Dune height component values

Cell	Dune height component values		
	Lower (m)	Mode (m)	Upper (m)
1	1	2	3
2	4	5	7.5
3	3.5	4	5
4	3	4	5
5	4.5	5.5	6.5
6	4	5	7
7[1]	0.5	0.8	1
8	4	5	6
9[2]	1	1.5	2
10	4	5	6
11	3	3.5	4.5
12	2.5	3	5
13	1.8	2	3
14	2	3	4

[1] North New Brighton seawall.

[2] New Brighton seawall.

4.1.4 Long-term trends (LT)

The long-term rate of horizontal coastline movement includes both ongoing trends and long-term cyclical fluctuations. These may be due to changes in sea level, fluctuations in coastal sediment supply or associated with long-term climatic cycles such as IPO (Interdecadal Pacific Oscillation).

4.1.4.1 Methodology

Long-term trends have been evaluated by the analysis of historic shoreline positions. Beach profile data has also been assessed; however the profile datasets are shorter than historic shorelines (i.e. 1990 to 2020) and subsequently are less suitable for interring long-term rates.

Shoreline data has been derived from geo-referenced historical aerial photographs. Software developed by T+T has been used to measure the distance to each shoreline from an assumed baseline at 50 m increments alongshore. A weighted linear regression analysis has then been undertaken on each set of shoreline measurements to estimate long-term rates between 1941 and 2019. In weighted linear regression, more reliable data (lower error values) are given greater emphasis or weight towards determining a best-fit line. Weighting of the shoreline data was estimated based on the Root Mean Square (RMS) Error associated with georeferencing and digitising. The older shorelines are typically weighted lower than the more recent shorelines. By calculating trends along the entire shoreline, rather than at a low number of discrete points (i.e. beach profile surveys), alongshore variation in long-term trends can be determined more accurately and either be used to inform parameter bounds or to separate the site into coastal behaviour cells.

Regression rates have been calculated both including and excluding the 1941 and 1955 shorelines (Figure 4.8). The 1941 and 1955 shorelines were excluded due to the significant changes in the Waimakariri River mouth and development of the Brooklands spit during this period. The average long-term rates measured from the beach profiles have also been plotted for comparison with the historic shoreline trends (Figure 4.8).

The data shows that majority of the shoreline has experienced net accretion between 1941 and 2019. Since 1974, the shoreline has generally shown a trend of increased accretion from north to south, with largest fluctuations around the spits. Based on the 95% confidence intervals, the uncertainty in long-term trends is largest near the spits and smallest near New Brighton (Cells 6 to 9). It should be noted that while the shoreline has shown historic accretion, significant storm cuts have also been experienced during this time. For example, up to 15 m retreat near North New Brighton during 1992 and approximately 10 m retreat near South New Brighton during 2013 (see Section 4.1.2)

Brooklands Spit to Bottle Lake Forest (cells 1 to 5)

There are no digitised shorelines along the Brooklands Spit for 1941 and 1955 as the Spit was only partially formed and there was minimal vegetation established (Figure 4.9).

Prior to 1940, the Waimakariri River mouth opened to the sea approximately 3 km south from its current position. During a flood event in 1940 the river mouth shifted north to its present position and the old river channel infilled. By the 1970s the Brooklands Spit had formed, and marram grass and pine trees were planted to stabilise the shifting dune sands. Between 1973 and 1977 a series of storms washed away 15 to 18 m of dune and in 1978 storm waves breached the spit resulting in a 250 m wide gap approximately 3 km south of the river mouth (cell 2).

The rock bank on the northern side of the river mouth was constructed to help keep the mouth in its current position. The historic shorelines show that between 1979 and 2011 there was significant erosion at the tip of the spit. Boyle (2017) also report that in 2012 there was concern around rapid erosion at the tip of Brooklands Spit. Continued monitoring and analysis completed by Boyle (2017) found the erosion ceased in 2013 and the Spit has since started migrating northwards.

From 1940s to 1970s the significant river mouth changes and formation of the spit appeared to have impact on the adjacent shoreline. For example, through Bottle Lake Forest (cells 3 to 5) there was rapid shoreline accretion between 1941 and 1974 as the shoreline re-adjusted (Figure 4.10). Since the 1970s the river mouth and spit have been ‘relatively’ stable. Therefore, long-term rates post 1970s are likely to be a more accurate representation of future rates and subsequently have been adopted (Table 4.5).

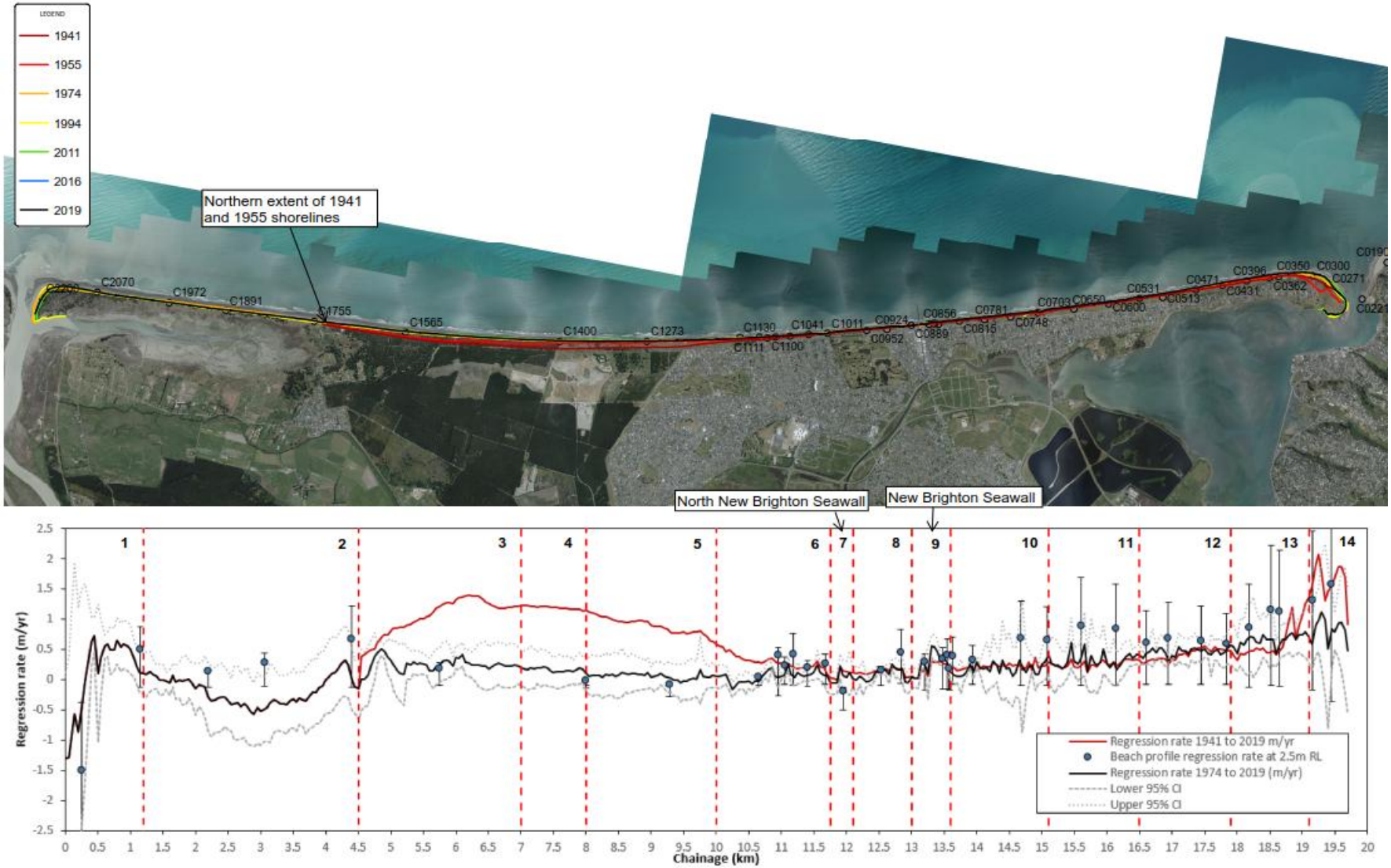

Figure 4.8: Regression analysis of historic shorelines. 95% confidence intervals included for regression rates between 1941 and 2019 (red) and 1974 and 2019 (black). Profile regression rates overlaid with error bars based on 95% confidence intervals. Refer to Table 4.1 for profile chainage locations.

Figure 4.9: Historic aerials showing growth of the Brooklands Spit.

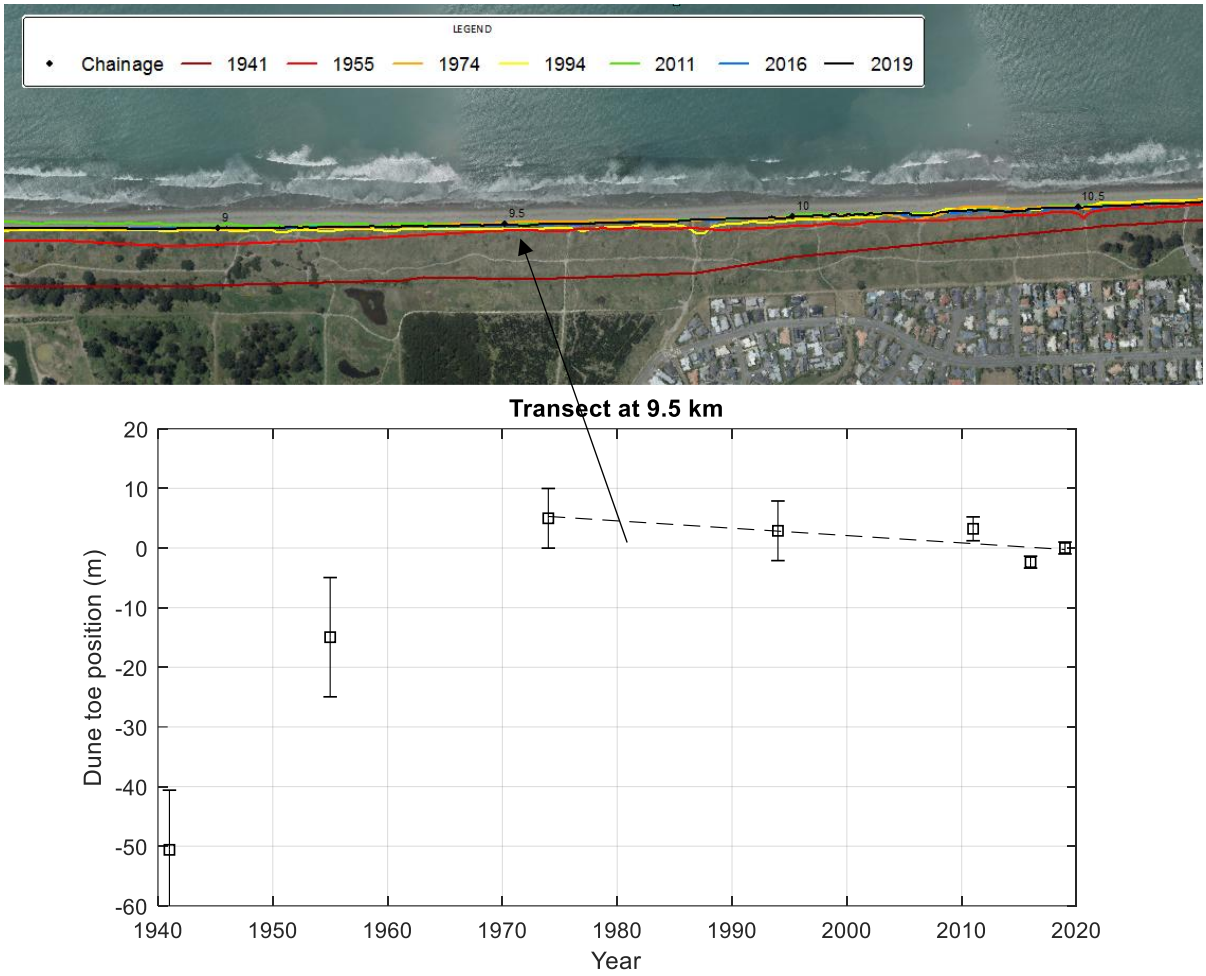

Figure 4.10: Historic shorelines and regression plot at chainage 9.5 km (Bottle Lake Forest, Cell 5). Distances are relative to 2019 shoreline position.

Waimairi to Southshore (city shoreline)

The city shoreline (i.e. Cells 6 to 13) shows average accretion rates ranging from 0.1 m/yr near Waimairi and up to 0.7 m/yr along Southshore. Recent accretion rates have been high near New Brighton and Southshore (Cells 10 to 13) and hence the regression rates from the beach profile dataset (1990 to 2020) are higher compared to the regression rates from the historic shorelines (Figure 4.8). Figure 4.11 shows how the shoreline near South New Brighton (Cell 10) experienced accretion between 1941 and 1974, followed by erosion until 1994 and then accretion until 2019. Profile data indicates that the erosion between 1974 and 1994 shorelines is likely the result of the 1992 storm event.

Hicks et al (2018a) noted that the phase of accretion since 2011, along Cells 10 to 13, may be associated with effects from the earthquakes. Following the earthquake there was a reduction in the tidal prism of the Avon-Heathcote estuary and subsequently a reduced volume on both the ebb and flood tidal deltas at the inlet entrance. This reduction in delta size has potentially resulted in a surplus of sand being supplied to the adjacent shoreline and hence the period of increased accretion following 2011.

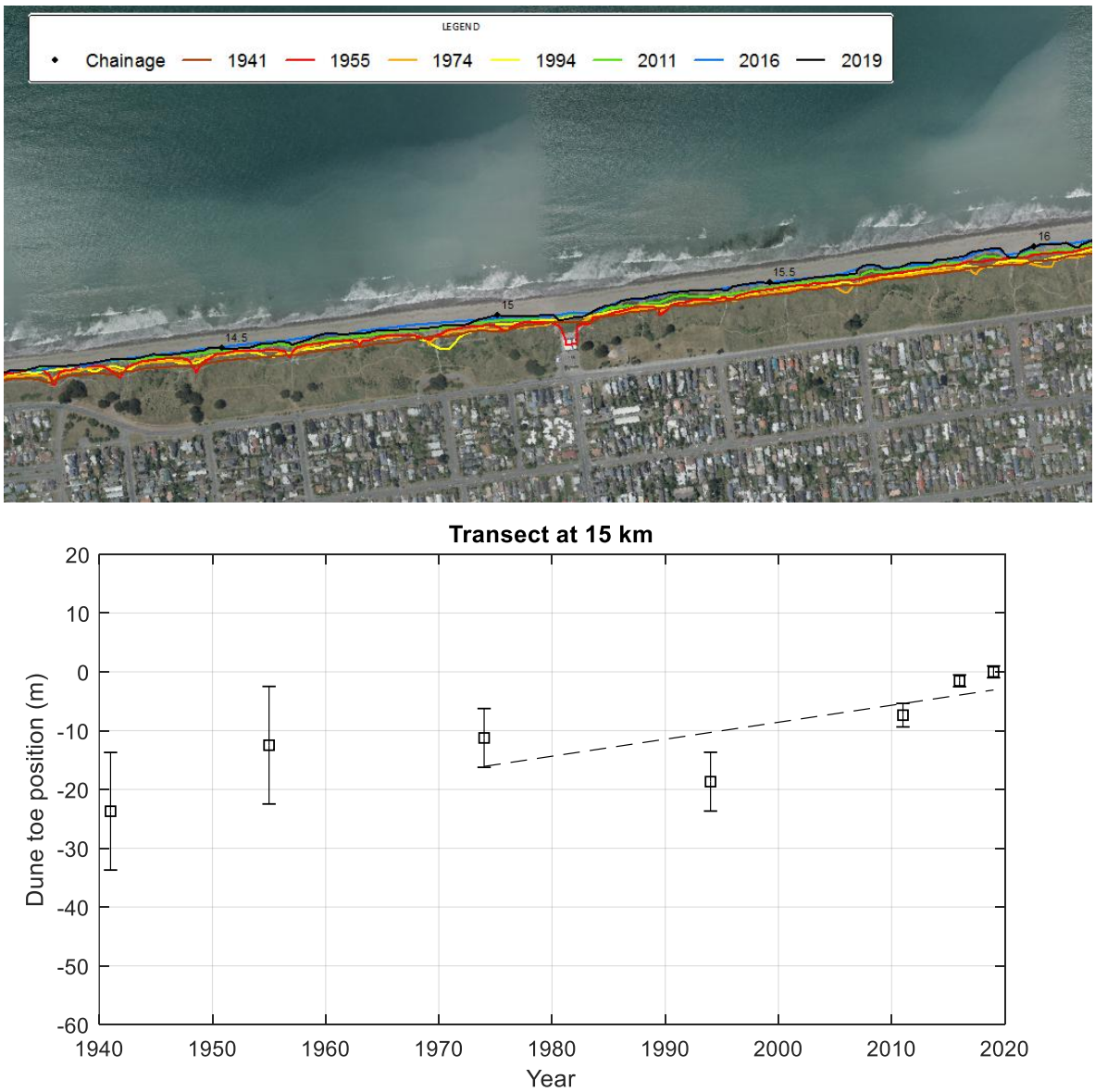

Figure 4.11: Historic shorelines and regression plot at chainage 15 km (South New Brighton, Cell 10). Distances are relative to 2019 shoreline position.

Distal end of Southshore Spit (Cell 14)

The distal end of the Southshore Spit is very dynamic and is largely influenced by changes in the adjacent inlet delta system.

Historic shorelines mapped from aerial photographs indicates the spit has shown net accretion since 1941 (Figure 4.12). However, previous studies indicate that there were three periods of erosion along the spit between 1918 and 1949. Findlay and Kirk (1988) reported an erosional phase occurred between 1918 and 1922 and then again between 1930 and 1937. The most significant erosion occurred between 1940 and 1949 where the spit eroded up to 500 m. This erosion is evident in the shoreline data shown in Figure 4.12. Following this significant erosion, a sandbag groyne was constructed and later upgraded in the 1950s. The period from 1950s to 1974 was generally dominated by accretion and then followed by a small period of erosion from the 1980s to 1994. Between 1994 and 2016 the shoreline continued to accrete and then between 2016 to 2019 there has been erosion (Figure 4.12).

Findlay and Kirk (1988) note that the main ebb channel from the Avon-Heathcote estuary historically flowed south-east past Shag Rock to an outlet near Cave Rock (Sumner). The channel shifted to its current position during 1938 and it is understood that this shift was an important factor contributing to the extensive erosion of the spit during the 1940s.

Due to the dynamic interactions with the inlet delta system, there is uncertainty in the future long-term rates around the spit. For example, an increased tidal prism within the Avon-Heathcote is likely to result in an enlarged ebb tidal delta, widening of the inlet and subsequently erosion of the spit.

The Canterbury Earthquake Sequence (CES) resulted in 0 to 0.4 m uplift across the estuary which reduced the tidal prism by ~12 to 18% (Measures et al 2011). This reduced tidal prism may have contributed to the flux of sediment which was observed on the adjacent spit following 2011.

Prior to CES, Rodgers et al (2020) state that there was a theorized 400-year period of gradual subsidence and tidal prism increase. Rodgers et al., (2020) concluded that while the long-term increase in tidal prism was interrupted with the earthquake uplift, it appears to have resumed.

Future changes in relative sea level are likely to affect the tidal prism of the estuary and subsequently the volume of sand stored in the ebb tidal delta and the adjacent spit. Hicks et al., (2018b) suggest that the tidal inlet is likely to enlarge in the future, resulting in an increased tidal prism and potentially increased erosion on the spit. Quantification of this is, however, beyond the scope of this assessment.

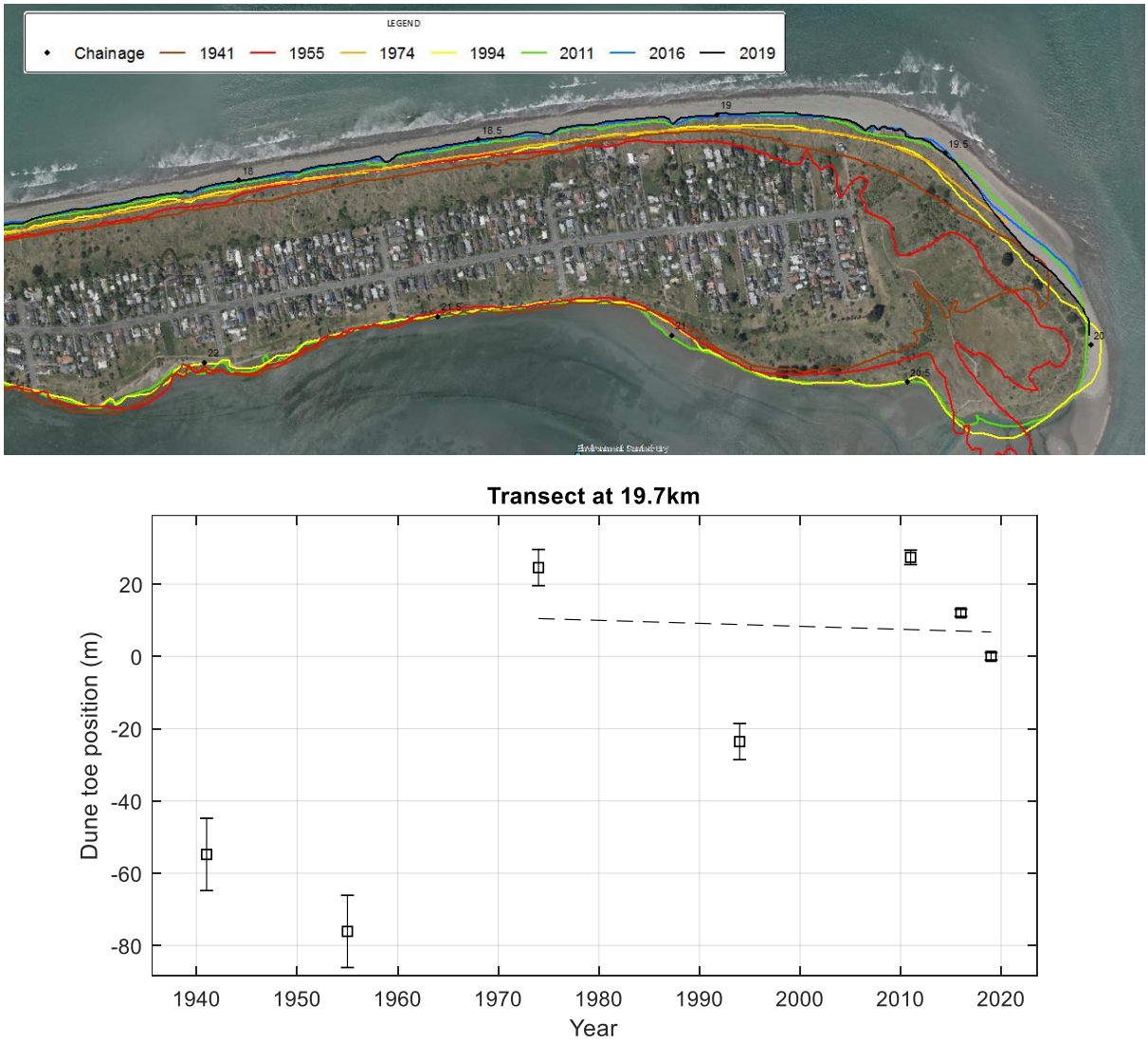

Figure 4.12: Historic shorelines and regression plot at chainage 19.7 km (distal end of Southshore Spit).

4.1.4.2 Adopted values and distribution

For this assessment a triangular distribution has been adopted for the LT component, with minimum, maximum and modal values defined within each cell. The parameter bounds have been rationalised based on the variation in the mean regression rate within each cell. For example, the upper bound is based on the maximum mean regression trend within each cell and the lower bound is based on the minimum mean regression trend within each cell. This is used in preference to a 95% confidence interval due to the often very wide range in confidence intervals due to the limited data points. LT trends within Cells 7 and 9 (seawalls) are based on the trends measured within the adjacent cells. Adopted LT parameter bounds for each cell along the Christchurch open coast are shown in Table 4.5.

Table 4.5: Adopted long-term component values for current sediment budget scenario

Cell	Long-term rate (m/yr)[1]		
	Upper	Mode	Lower
1	0.60	0	-0.60
2	0.30	0.25	0.18
3	0.30	0.25	0.18
4	0.18	0.16	0.14
5	0.12	0.08	0.00
6	0.20	0.10	-0.04
7	0.20	0.10	-0.04
8	0.20	0.10	-0.04
9	0.40	0.25	0.10
10	0.30	0.20	0.10
11	0.40	0.30	0.20
12	0.47	0.45	0.40
13	0.70	0.65	0.60
14	0.7	0.2	-0.10

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

4.1.4.3 Potential climate change effects on sediment supply

The key contributor to long-term accretion along the Christchurch open coast is the sediment supply from the Waimakariri River. Hicks et al (2018a) investigated the present day and future sediment budget for the Waimakariri River and concluded that the river contributes 182, 000 m^3/yr to the sediment budget along the Christchurch open coast shoreline, south of the river mouth.

Under future climate change conditions, the sediment supply to the Christchurch open coast may change. Based on the findings from Hicks (2018b) three future sediment supply scenarios have been assessed:

- Scenario 1: Current sediment budget. Assume current long-term rates continue.
- Scenario 2: 11% reduction in sediment supply to the coast due to climate change effects upstream.
- Scenario 3: 28% increase in sediment supply to the coast due to climate change effects upstream.

Hicks (2018b) conclude that the increase in sediment supply is a likely scenario. The long-term rates have been adjusted based on Equation 6 within Hicks (2018b)[2] (excluding the sea level rise component as this is accounted for separately within this study, see Section 0). A summary of the adjusted long-term rates accounting for the future sediment budget scenarios is shown in Table 4.6.

[2] $\Delta y = (\frac{Q_s T_e P_s}{BH} - \frac{SL}{H})$ where Q_s is the total river sand supply to the coast, T_e is the proportion of this river sand retained on the beach profile, and P_s is the proportion of the retained sand that is transported south from the river mouth to the city shore, H is the profile height above the closure point, B is the length of shoreline, S is the sea level rise and L is sum of the beach width above MSL.

Table 4.6: Adopted long-term rates (m/yr)[1] for sediment budget climate change scenarios

Cell	11% reduction in sediment supply			28% increase in sediment supply		
	Upper	Mode	Lower	Upper	Mode	Lower
1	0.48	-0.12	-0.72	0.82	0.22	-0.38
2	0.24	0.20	0.14	0.41	0.34	0.25
3	0.24	0.20	0.14	0.41	0.34	0.25
4	0.14	0.13	0.11	0.25	0.22	0.19
5	0.10	0.06	-0.02	0.16	0.11	0.02
6	0.16	0.08	-0.05	0.27	0.14	-0.03
7	0.16	0.08	-0.05	0.27	0.14	-0.03
8	0.16	0.08	-0.05	0.27	0.14	-0.03
9	0.32	0.20	0.08	0.55	0.34	0.14
10	0.24	0.16	0.08	0.41	0.27	0.14
11	0.32	0.24	0.16	0.55	0.41	0.27
12	0.38	0.36	0.32	0.64	0.62	0.55
13	0.56	0.52	0.48	0.96	0.89	0.82
14	0.56	0.16	-0.12	0.96	0.27	-0.06

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

4.1.5 Response to sea level rise (SLR)

Geometric response models propose that as sea level is raised, the equilibrium profile is moved upward and landward conserving mass and original shape. The most well-known of these geometric response models is that of Bruun (Bruun, 1962, 1988) which proposes that with increased sea level, material is eroded from the upper beach and deposited offshore to a maximum depth, termed closure depth. The increase in seabed level is equivalent to the rise in sea level and results in landward recession of the shoreline (Figure 4.13).

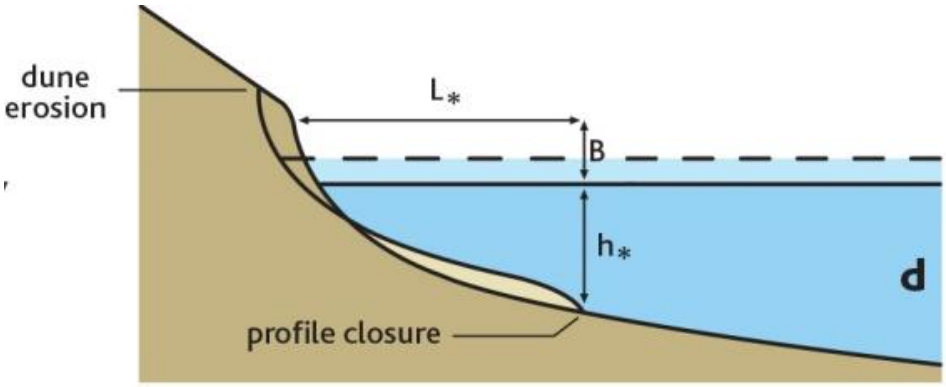

Figure 4.13: Schematic diagrams of the Bruun model for shoreline response (after Cowell and Kench, 2001).

The inner parts of the profile exposed to higher wave energy are likely to respond more rapidly to changes in sea level. For example, Komar (1999) proposes that the beach face slope is used to predict coastal erosion due to individual storms. Deeper definitions of closure including extreme wave height-based definitions (Hallermeier, 1983), sediment characteristics and profile adjustment records (Nicholls et al., 1998) are only affected during infrequent large-wave events and therefore may exhibit response-lag.

To define parameter distributions, the Bruun rule has been used to assess the landward retreat of three different active beach slope profiles (Figure 4.14):

1 Active beach face, average dune toe position to low water mark (lower bound).

2 Inner closure slope, average dune crest to inner Hallermeier closure depth (modal value).

3 Outer closure slope, average dune crest to outer Hallermeier closure depth (upper bound).

The Hallermeier closure definitions are defined as follows (Nicholls et al., 1998):

$$d_l = 2.28\, H_{s,t} - 68.5\, (H_{s,t}^2 / g T_s^2) \cong 2\, x\, H_{s,t} \quad (4.2)$$

$$d_i = 1.5 \times d_l \quad (4.3)$$

Where d_l is the closure depth below mean low water spring, H_s is non-breaking significant wave height exceeded for 12 hours in a defined time period, nominally one year, and T_s is the associated period. For this study the deep water (non-breaking) wave climate parameters of H_s and T_p were based on the MetOcean wave hindcast data (1979 to 2019) from the 10 m depth contour (Table 4.7). Adopted slopes are based on average beach profiles and LINZ bathymetric contour data within each cell. A summary of the representative profiles and closure depths is presented in Table 4.8.

Table 4.7: Inner and outer profile closure depth estimates derived from Hallermeier's definitions with wave parameters sourced from the MetOcean wave hindcast

Location	Profile[1]	Significant wave height[2], $H_{s,12hr}$ (m)	Wave period[3], $T_{p,12hr}$ (s	Inner closure depth, dI (m)	Outer closure depth, di (m)
Southshore	CCC0396	2.99	7.75	7.1	10.7
Southshore	CCC0431	2.99	7.75	6.8	10.2
Brighton	CCC748	2.99	7.75	6.7	10.0
Parklands	CC1086	2.6	8.57	6.7	10.0
Waimairi	CC1273	2.6	8.57	6.7	10.0
Spencerville	CC1565	2.89	9.33	6.2	9.2
Brooklands	CC1972	2.89	9.33	6.2	9.2

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

[2] Non-breaking significant wave height exceeded for 12 hours over a year.

[3] Wave period corresponding to the non-breaking significant wave height exceeded for 12 hours over a year.

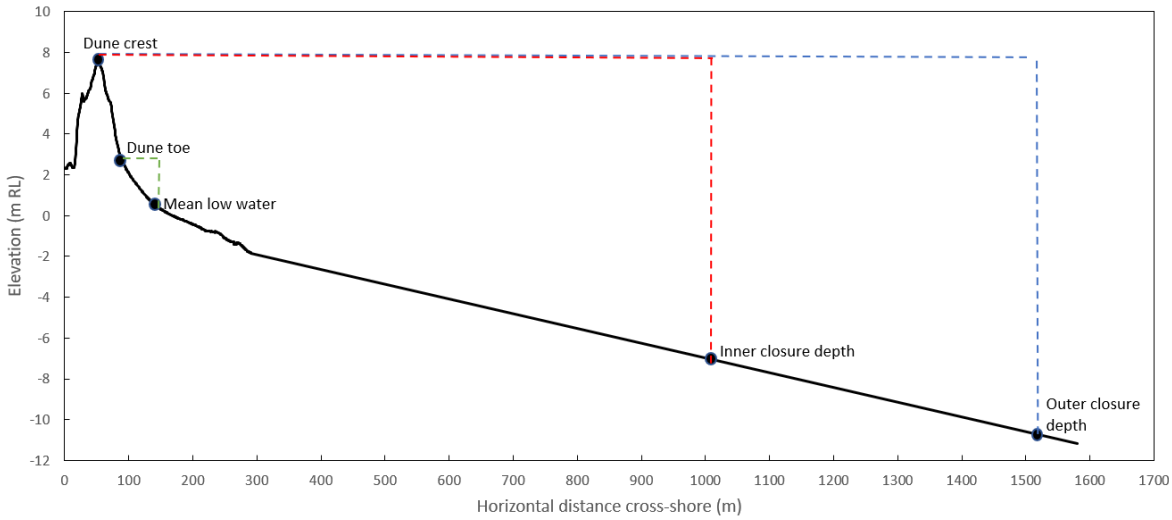

Figure 4.14: Extents of active profiles for the Christchurch Open Coast shoreline.

Table 4.8: Adopted slopes for each cell based on the profiles summarised in Table 2.7

Cells	Slope[1]		
	Lower	Mode	Upper
1 to 2	0.019	0.020	0.061
3	0.024	0.029	0.043
4 to 5	0.021	0.025	0.050
6 to 7	0.017	0.019	0.034
8	0.017	0.019	0.061
9	0.014	0.015	0.055
10	0.014	0.015	0.068
11 to 12	0.012	0.015	0.046
13 to 14	0.014	0.016	0.046

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

4.1.6 Summary of components

Adopted component values for the Christchurch open coast are summarised in Table 4.9. Overall, the erosion susceptibility is slightly higher at the northern end of the shoreline (i.e. Cells 1 to 4) where the accretion rates are lower and the short-term storm cut potential is higher.

The assessed component values for this assessment are generally similar to the T+T (2017) assessment. The previous T+T (2017) assessment only included the open coast shoreline south of Waimairi Beach and therefore is only comparable with the updated Cells 6 to 14. For the short term component, the revised extreme value distribution is a similar shape with a slightly larger mean storm cut value compared with the previous assessment. The revised assessment includes additional data and hence there is a slight difference in values.

The long-term component is generally similar to the T+T (2017), where the average long-term rates ranged from 0.14 to 0.44 m/year. The modal values adopted in the updated assessment range from 0.1 to 0.45 m/yr through Cells 6 to 12. The updated accretion rates are slightly larger than the T+T (2017) values within Cells 13 and 14, which is likely the result of increased accretion over the recent years.

For the SLR response, T+T (2017) based the closure depths on data from the ECan wave buoy, offshore from Banks Peninsula. The adopted significant wave height was 4.2 m with a period of 10.8 s. The updated assessment has based the closure depths on the MetOcean wave hindcast data from the 10 m depth contour and subsequently the significant wave heights have been reduced, resulting in shallower closure depths and reduced closure slopes for the minimum and mode parameter bounds.

4.1.7 Uncertainties

Key uncertainties in the erosion hazard assessment along the Christchurch open coast shoreline include:

- Tidal inlet response to SLR and the subsequent effects on the long term shoreline trends at the distal end of the Southshore Spit.
- Future sediment supply from the Waimakariri River and subsequently the long term accretion rates.

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

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

4.2 Sumner

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

4.2.1 Cell splits

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

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

4.2.2 Short term component (ST)

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

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

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

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

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

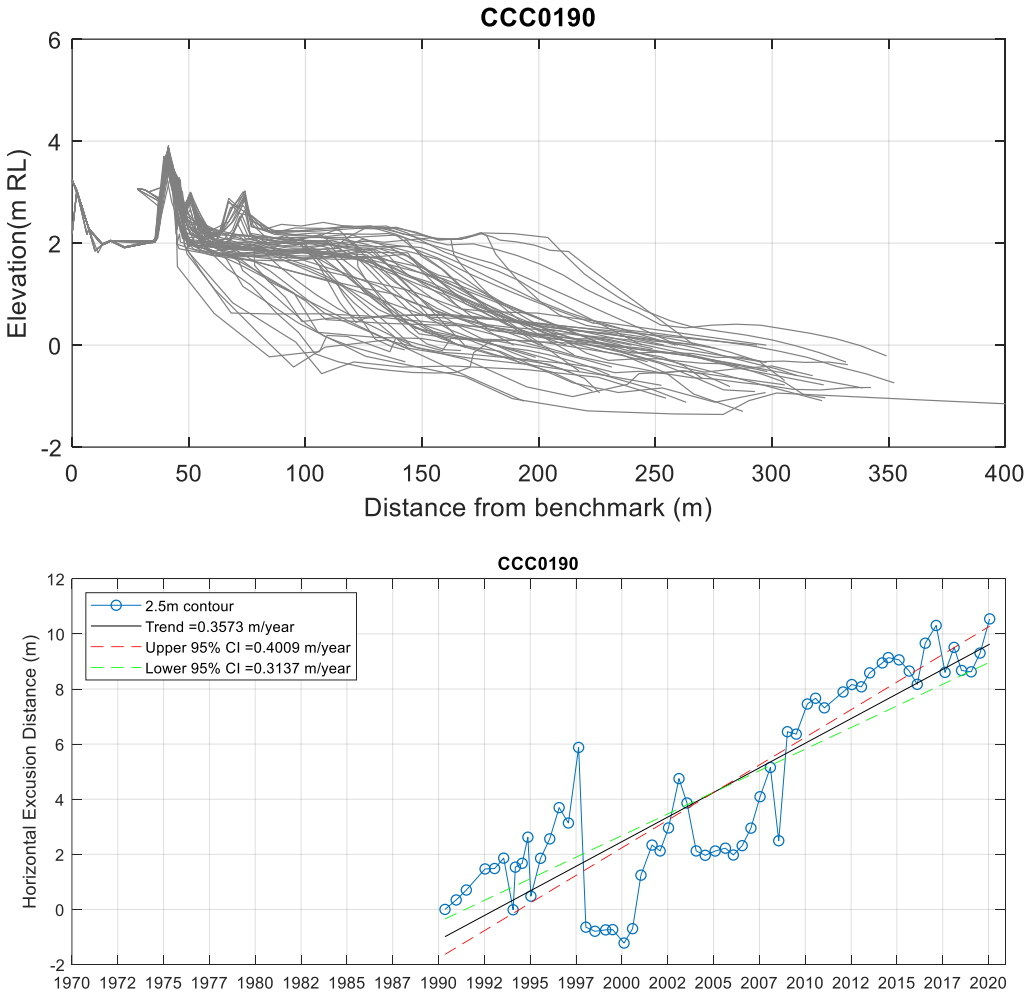

Figure 4.17: Beach profiles and regression plot for profile CCC0190 within Cell 28, Sumner.

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

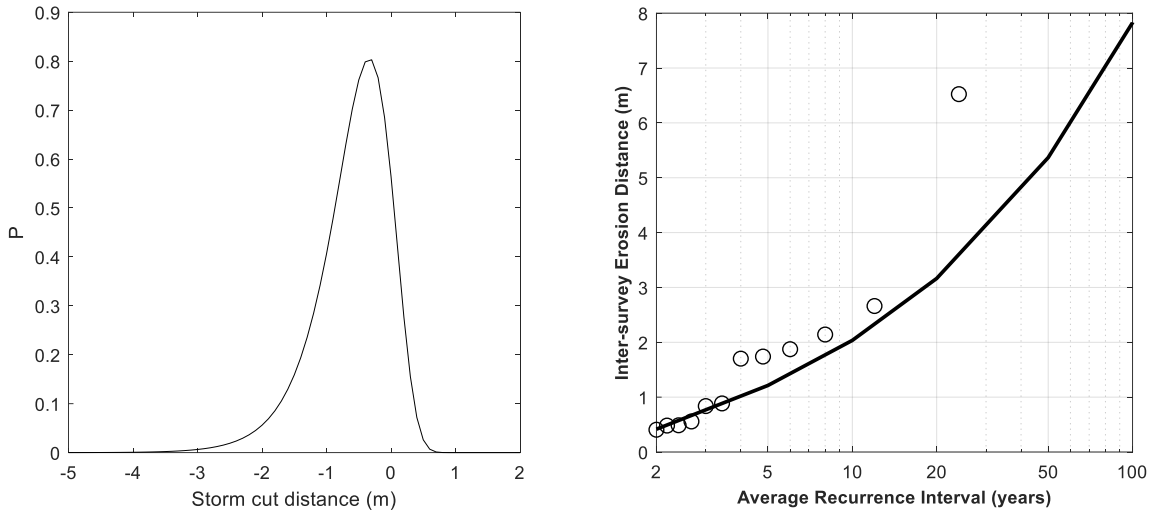

Figure 4.18: Example of extreme value distribution and curve for inter-survey storm cut distances at Clifton Beach (profile CC00190).

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

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

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

4.2.3 Dune stability (DS)

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

Table 4.11: Dune stability component values for Sumner Beach

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

Table 4.12: Dune and structure height component values

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

[1] Height of Class 1 structure.

4.2.4 Long-term component (LT)

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

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

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

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

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

Figure 4.19: Historical shorelines along Clifton Beach.

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

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

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

4.2.5 Response to sea level rise (SLR)

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

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

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

4.2.6 Summary of components

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

4.2.7 Uncertainties

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

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

Table 4.15: Adopted component values for the Sumner shoreline

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

4.3 Taylors Mistake

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

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

4.3.1 Cell splits

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

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

4.3.2 Short term component

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

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

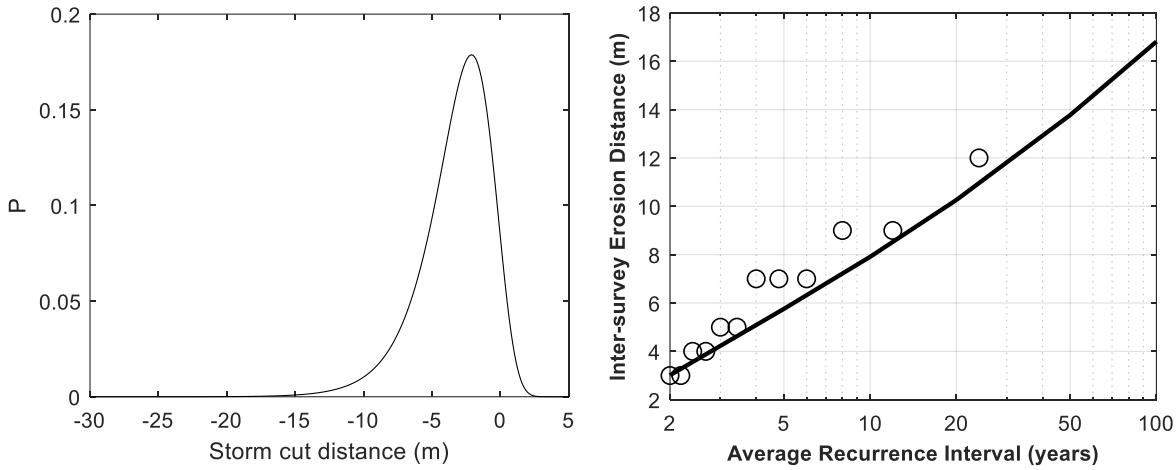

Figure 4.22: Extreme value distribution for inter-survey storm cut distances along Taylors Mistake.

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

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

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

4.3.3 Dune stability (DS)

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

Table 4.17: Dune stability component values for Taylors Mistake

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

Table 4.18: Dune height component values for Taylors Mistake

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

4.3.4 Long-term trends (LT)

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

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

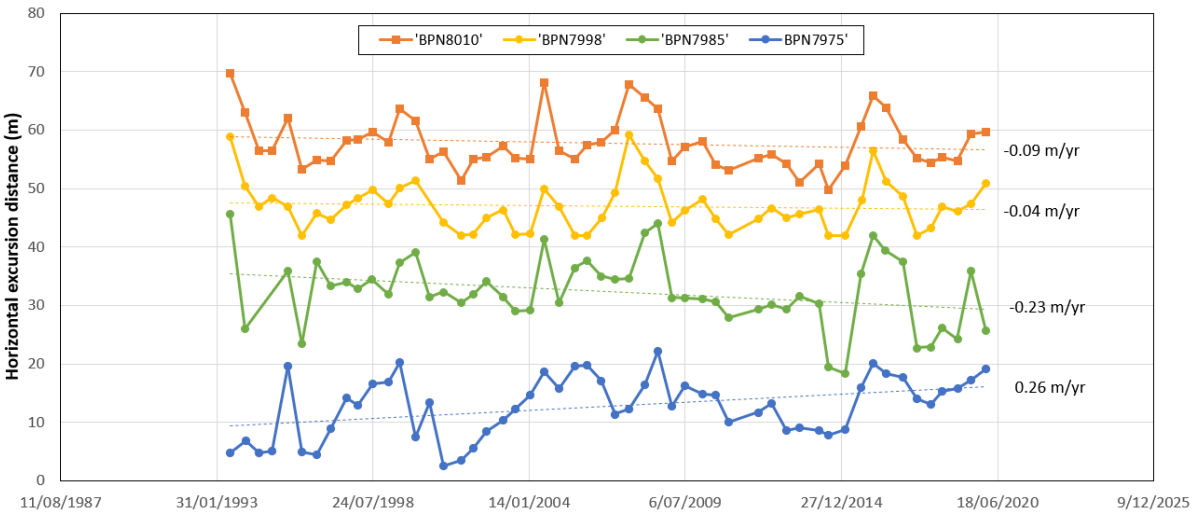

Figure 4.23: Regression analysis of beach profiles along Taylors Mistake.

Figure 4.24: Historic shorelines for Taylors Mistake.

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

Table 4.19: Adopted long-term rates along Taylors Mistake

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

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

4.3.5 Response to sea level rise (SLR)

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

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

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

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

4.3.6 Summary of components

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

4.3.7 Uncertainties

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

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

Table 4.21: Adopted component values for Taylors Mistake

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

4.4 Avon-Heathcote estuary

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

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

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

4.4.1 Cell splits

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

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

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

4.4.2 Short term component (ST)

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

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

Where:

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

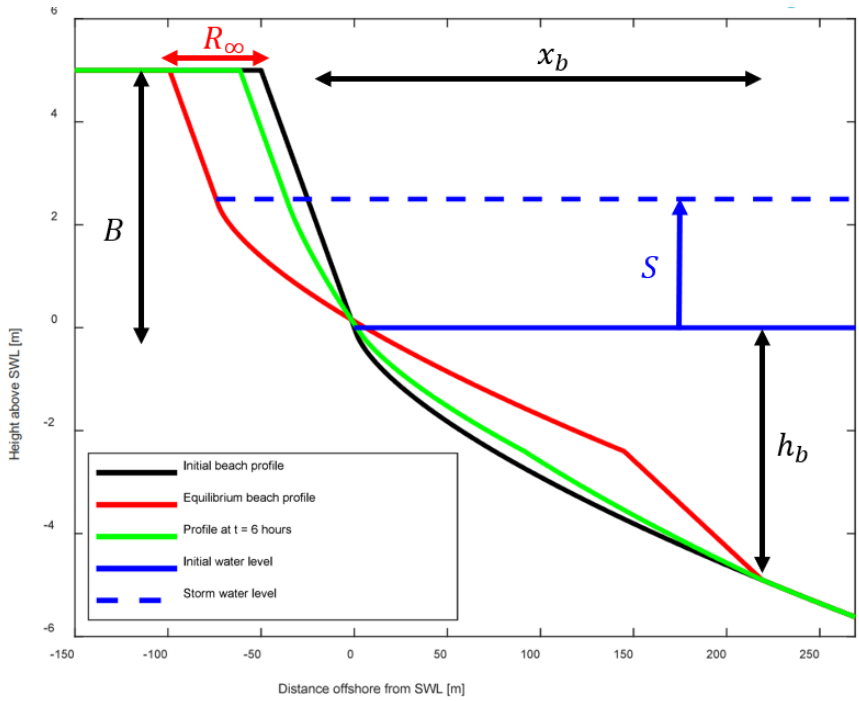

Figure 4.28: Schematic showing the beach storm response based on Kriebel and Dean (1993).

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

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

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

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

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

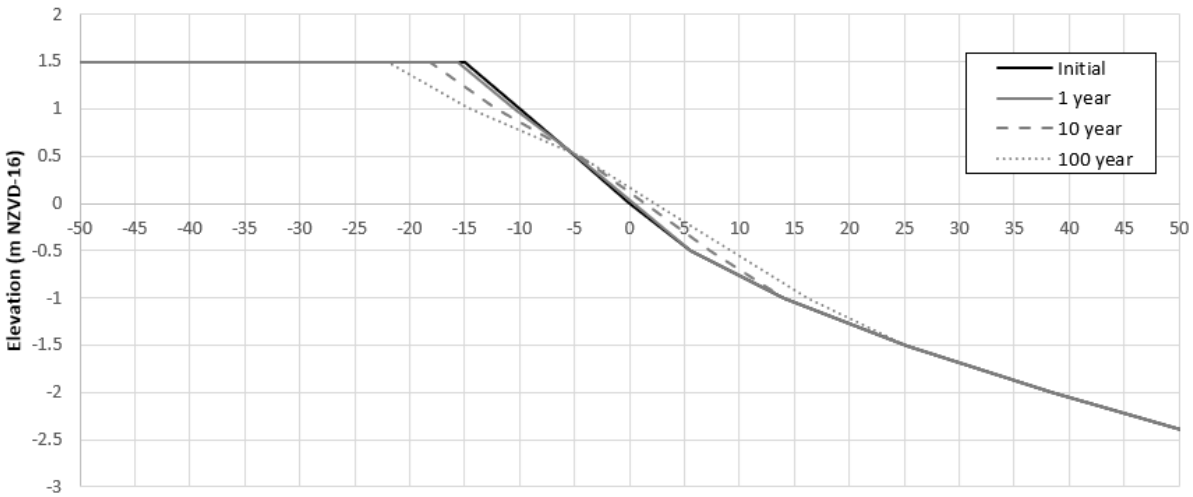

Figure 4.29: Example of beach response for the estuary shoreline under different storm conditions.

4.4.3 Long-term trends (LT)

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

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

4.4.3.1 Impact of the 2011 Canterbury Earthquakes

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

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

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

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

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

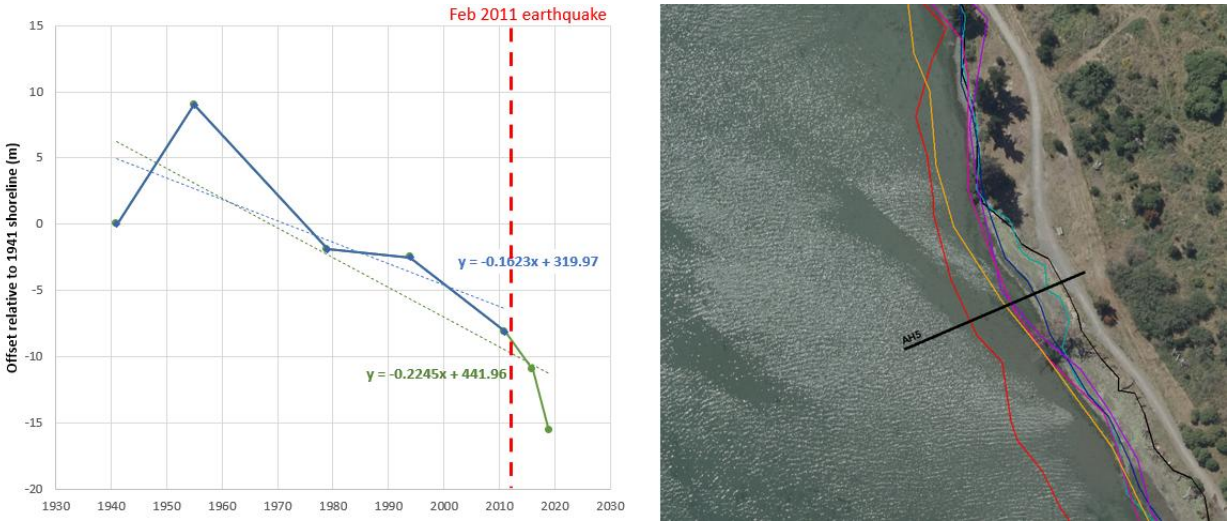

Figure 4.31: Example of long-term trends assessed using historic shoreline data within the Avon Heathcote Estuary. Profile is AH5 within Cell 19.

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

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

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

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

4.4.4 Dune/bank stability (DS)

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

4.4.5 Response to sea level rise (SLR)

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

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

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

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

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

Where:

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

tanα = Average slope of the upper beach.

In low energy environments there is likely to be insufficient energy to reform the beach crest to match the increase in sea level and subsequently once sea levels exceed the crest, inundation becomes the more significant controlling factor. Therefore, the maximum potential extent of SLR induced erosion for low-lying beach areas is assumed to be controlled by the crest height above the MHWS. Where the beach crest is higher than the projected sea level rise, the sea level value has been used. This means that when sea level exceeds the crest height and inundation occurs, there is

no additional increase in erosion of the present-day shoreline. This method approximately follows the method by Komar et al. (1999), with the MHWS adopted as the dune-toe level.

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

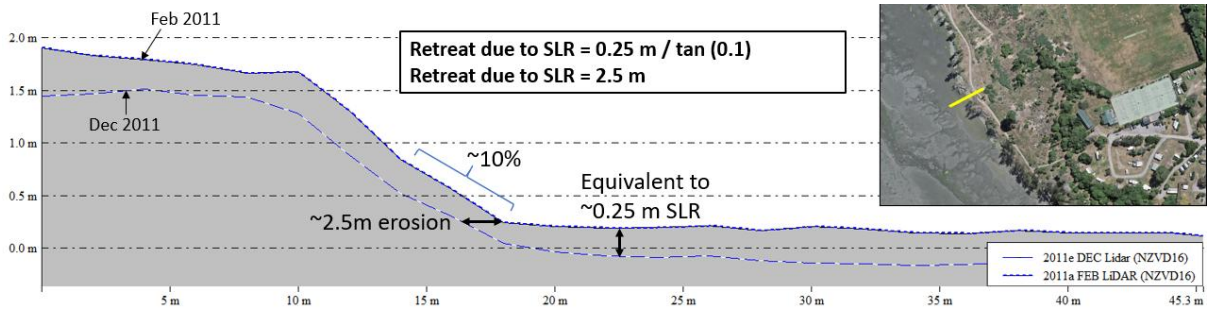

Figure 4.32: Example of shoreline response to SLR within the Avon-Heathcote Estuary.

4.4.6 Summary of components

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

4.4.7 Uncertainties

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

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

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

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

4.5 Banks Peninsula harbours (detailed sites)

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

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

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

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

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

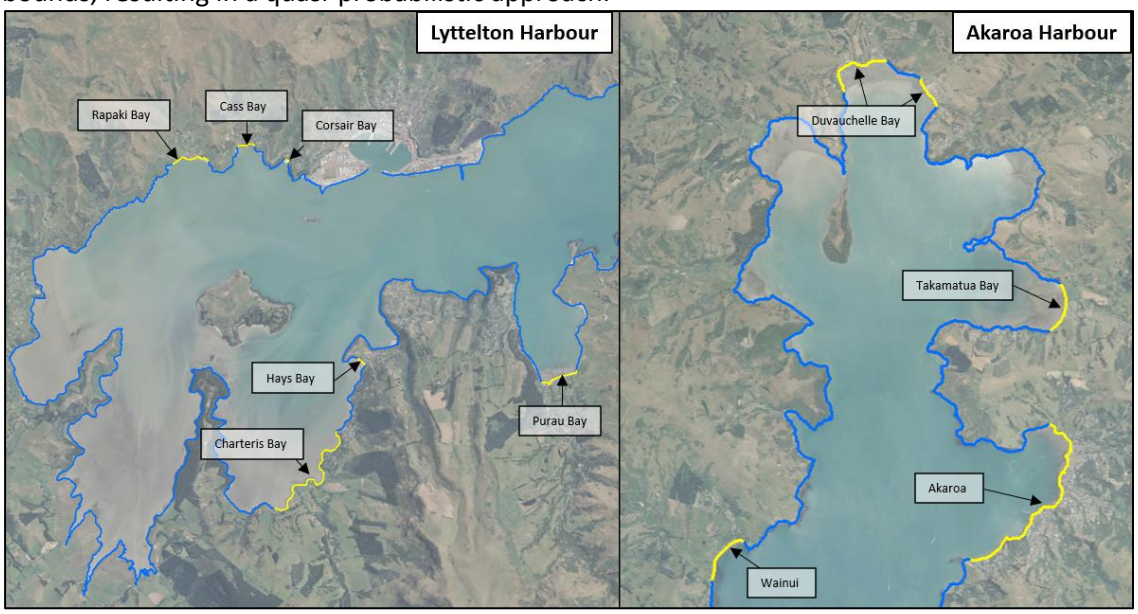

Figure 4.33: Overview map of detailed sites (yellow line) within the harbours.

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

4.5.1 Short term component (ST)

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

While there is some variation in the exposure of the harbour beaches, most of the sites are fronted with tidal flats which dissipate wave energy and therefore the depth-limited wave heights at each

site are similar. Subsequently, the short term parameter bounds for all harbour sites has been assumed the same. For the consolidated banks the short term component is not applicable as the banks behave differently to the unconsolidated beaches (see Section 3.1.3).

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

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

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

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

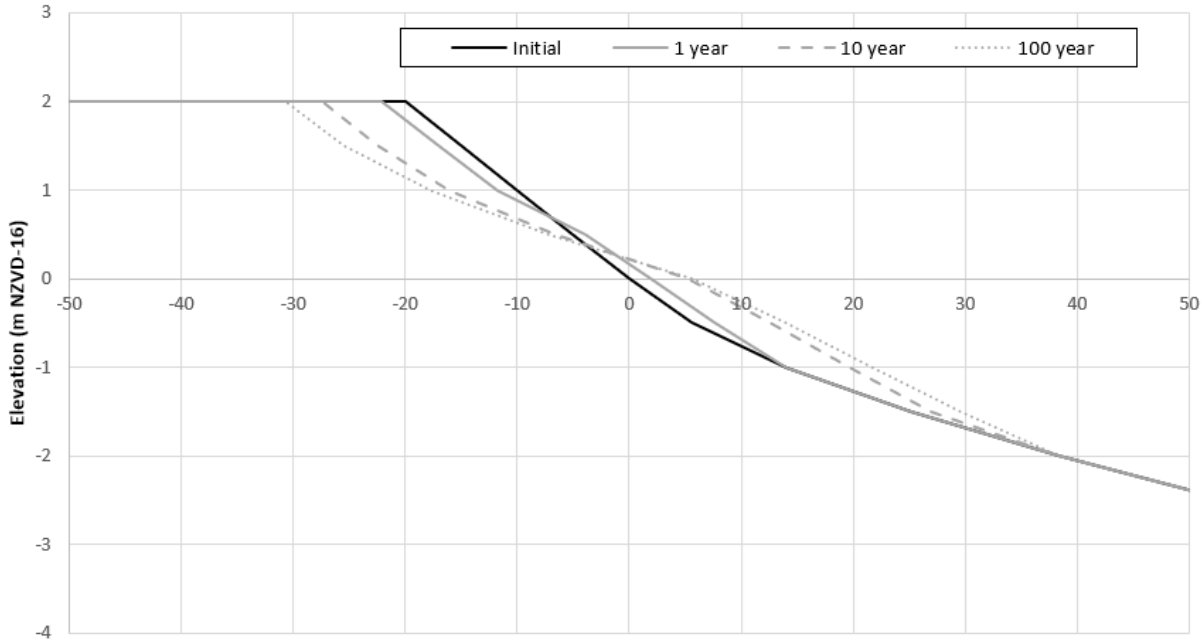

Figure 4.35: Example of beach response under different storm conditions within Lyttelton Harbour.

4.5.2 Long-term trends (LT)

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

In areas where protection structures exist it is difficult to determine long-term rates, however, in absence of the structures, erosion is likely to occur and hence the structure exists. Site observations show evidence of scour and overtopping around structures which also implies that in absence of the structure, shoreline retreat is likely to occur (Figure 4.37). Similarly, some of the unprotected shorelines show minimal erosion in the historic aerials, however based on site observations there is

evidence of undercutting and erosion, for example at Takamatua (Figure 4.38). In contrast, there are some sites which appear stable from the site observations and as expected show minimal movement in the historic aerials, for example Purau Bay and Cass Bay (Figure 4.39).

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

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

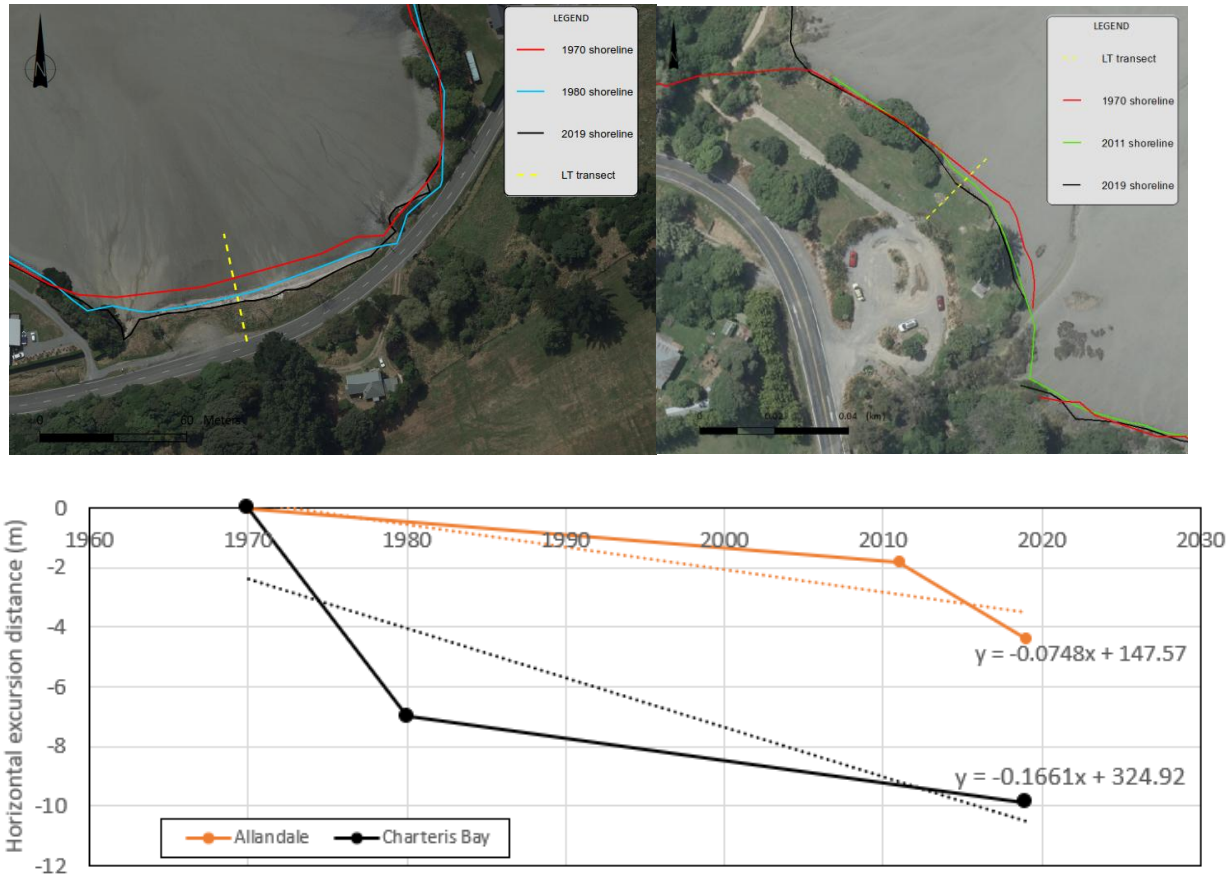

Figure 4.36: Example of shoreline analysis along transects at Charteris Bay (left) and Allandale (right).

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

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

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

4.5.3 Dune and bank stability (DS)

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

to 1(V):3(H) is appropriate. Adopted dune/bank heights and stable angles for each cell are shown in Table 4.26 and Table 4.27.

4.5.4 Response to sea level rise (SLR)

4.5.4.1 Harbour beaches

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

4.5.4.2 Harbour banks

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

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

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

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

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

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

4.5.5 Summary of components

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

4.5.6 Uncertainties

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

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

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

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

[1] Closure slope applicable for the harbour beach morphology

[2]SLR factor applicable for the bank morphology.

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

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

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

[2]SLR factor applicable for the bank morphology.

4.6 Banks Peninsula (regional sites)

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

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

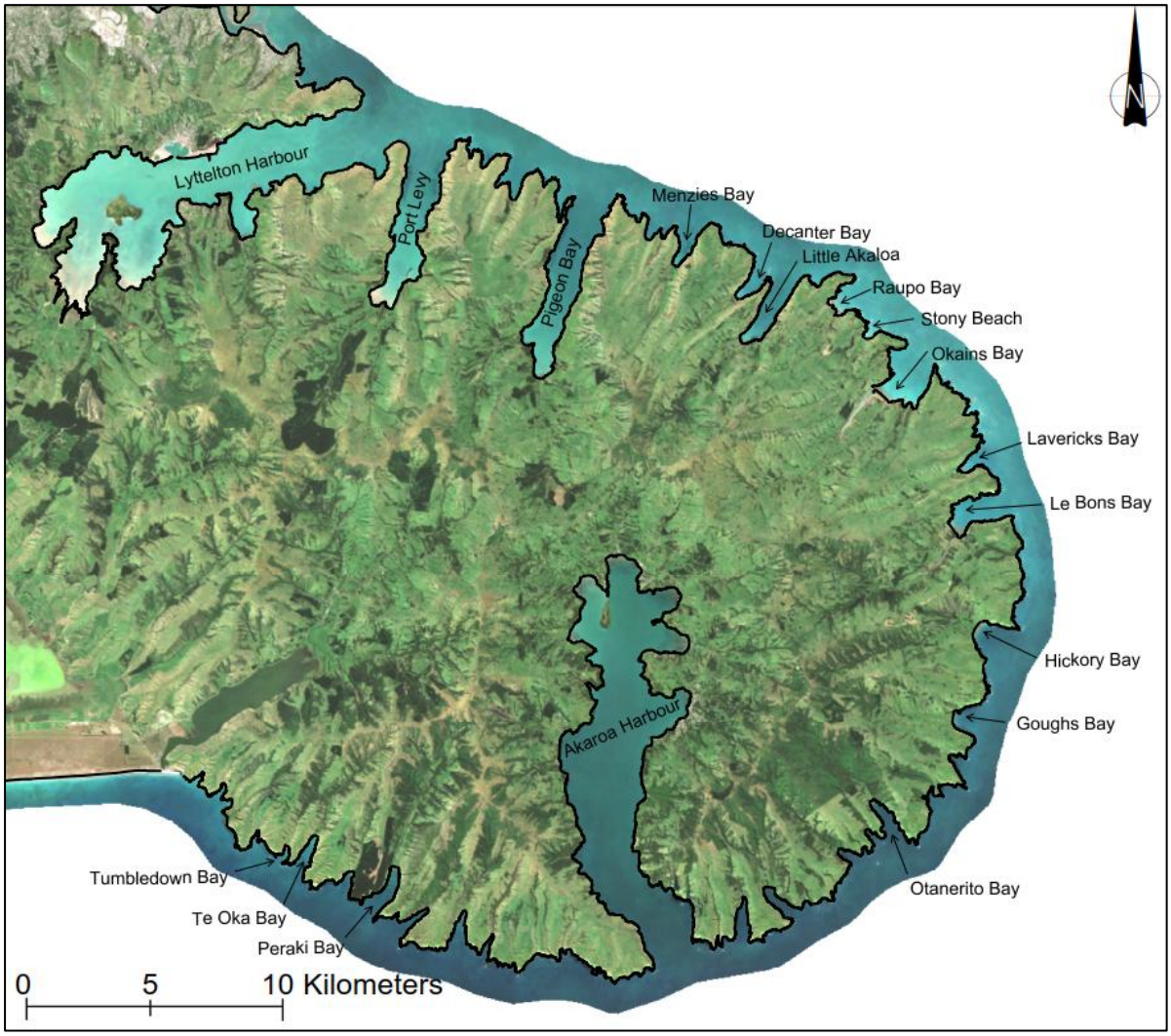

Figure 4.40: Overview of the bays and harbours around Banks Peninsula.

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

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

4.6.1 Short term component (ST)

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

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

assumed that storm cut along the moderately exposed beaches is between the sheltered and exposed beach storm cut distances. Adopted short term values for each beach are provided in Table 4.29.

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

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

4.6.2 Dune and bank stability (DS)

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

4.6.3 Long term component (LT)

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

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

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

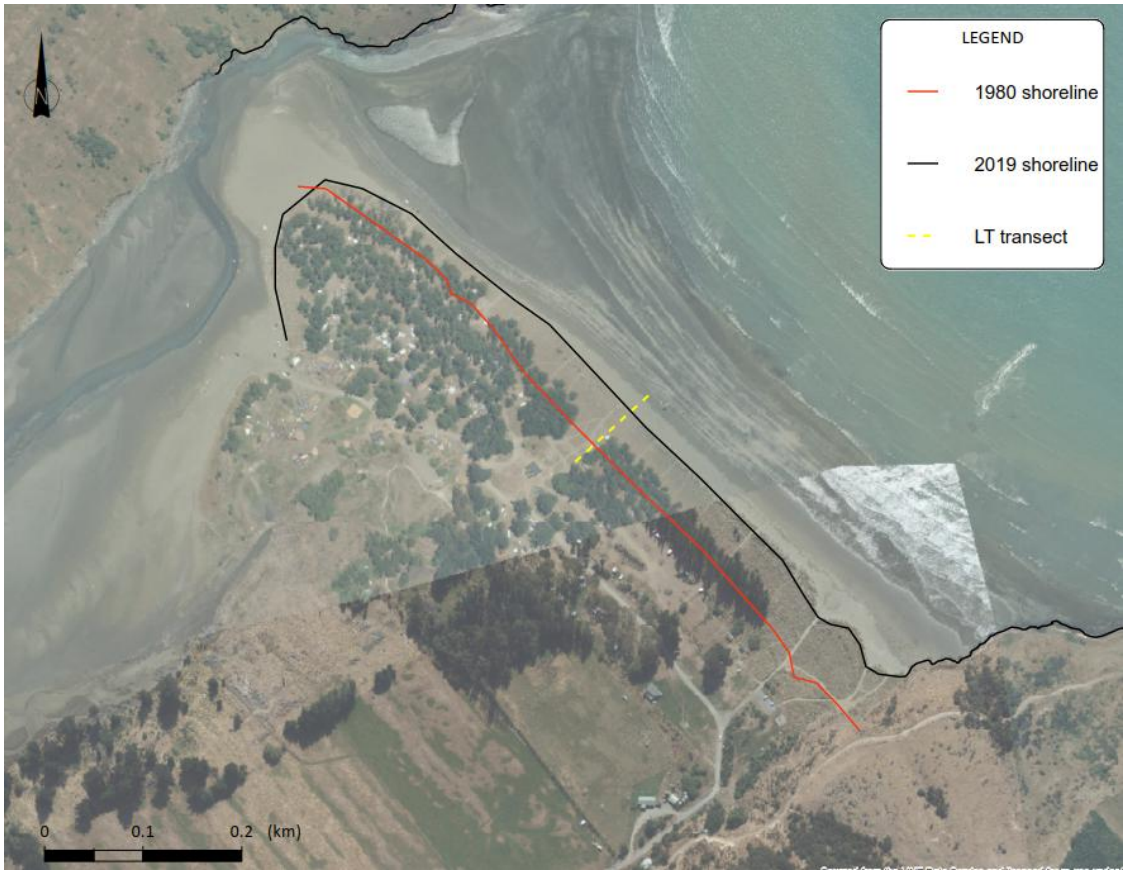

Figure 4.43: Example of shorelines used to assess long term trends in Okains Bay. Approximately 23 m of accretion measured between 1980 and 2019.

4.6.4 Response to SLR (SLR)

4.6.4.1 Beach response

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

4.6.4.2 Bank response

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

4.6.5 Cliff instability

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

Figure 4.44: Example of cliff shorelines around Banks Peninsula.

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

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

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

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

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

While this method is not a detailed cliff projection method, it is suitable for a regional coastal hazard screening assessment. It is emphasised that cliff collapse hazard is highly dependent on site-specific details and a can include a range of potential triggers in addition to coastal processes. These details

cannot be incorporated into this regional-scale assessment. If more detailed information is required about the cliff instability hazard for a specific location (e.g. as part of proposed development or hazard management activities in future) then a site-specific assessment should be undertaken, which may indicate that the hazard area is narrower or wider than mapped in this regional assessment.

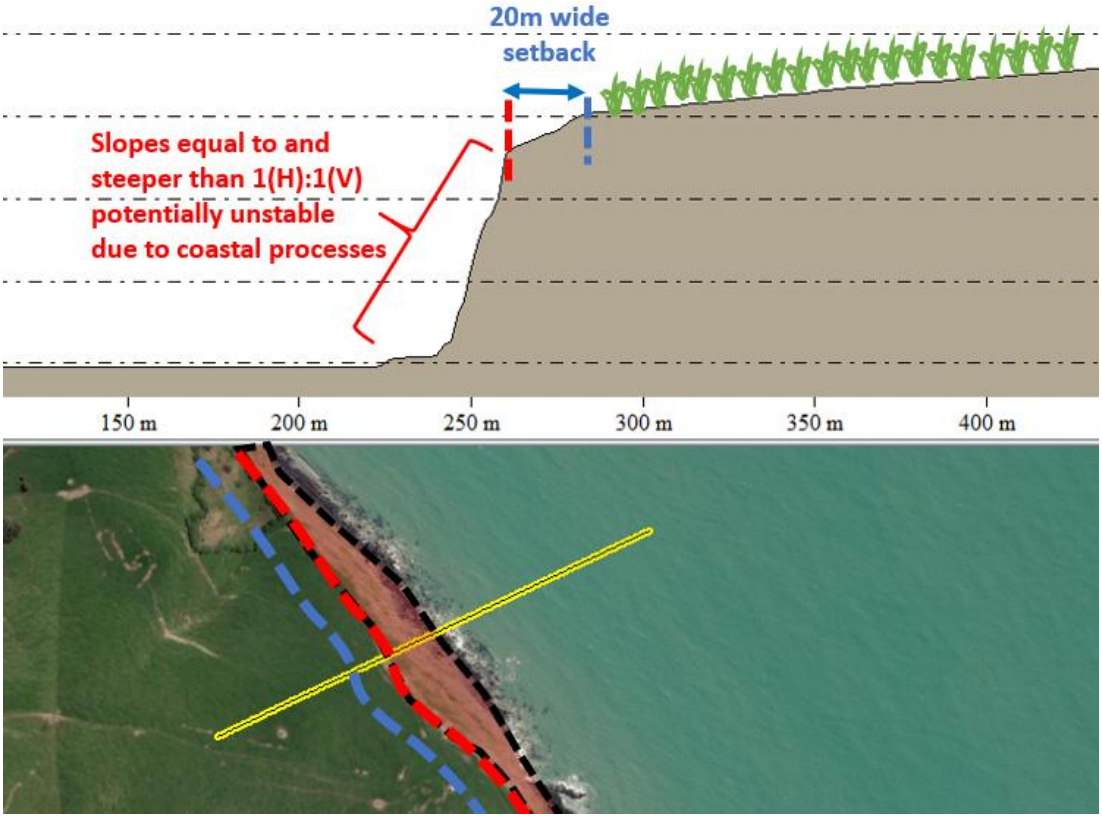

Figure 4.45: Example of cliff ASCE.

4.6.6 Summary of components

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

4.6.7 Uncertainties

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

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

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

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

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

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

4.7 Kaitorete Spit

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

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

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

4.7.1 Cell splits

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

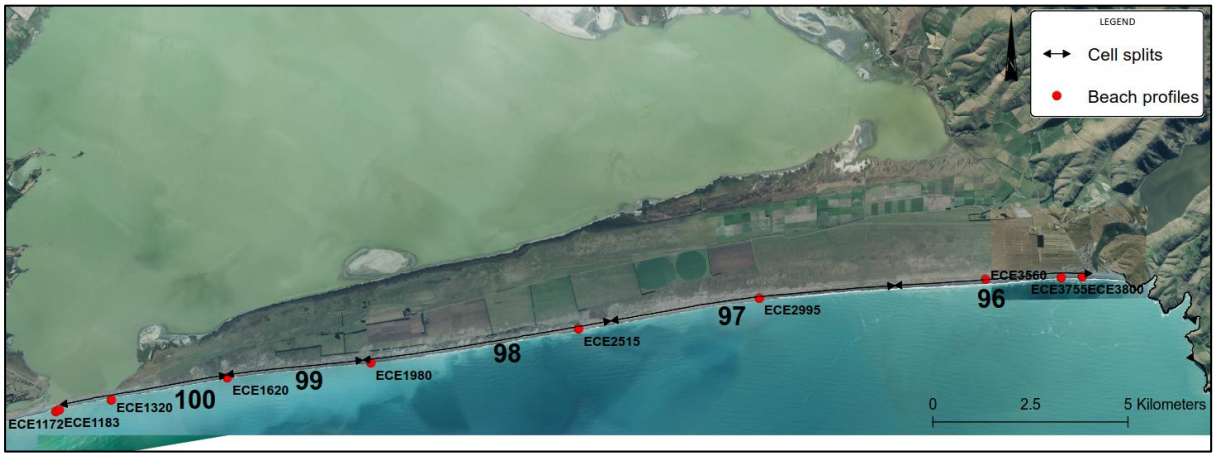

Figure 4.47: Overview of cell splits and profile locations along Kaitorete Spit.

4.7.2 Short term component (ST)

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

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

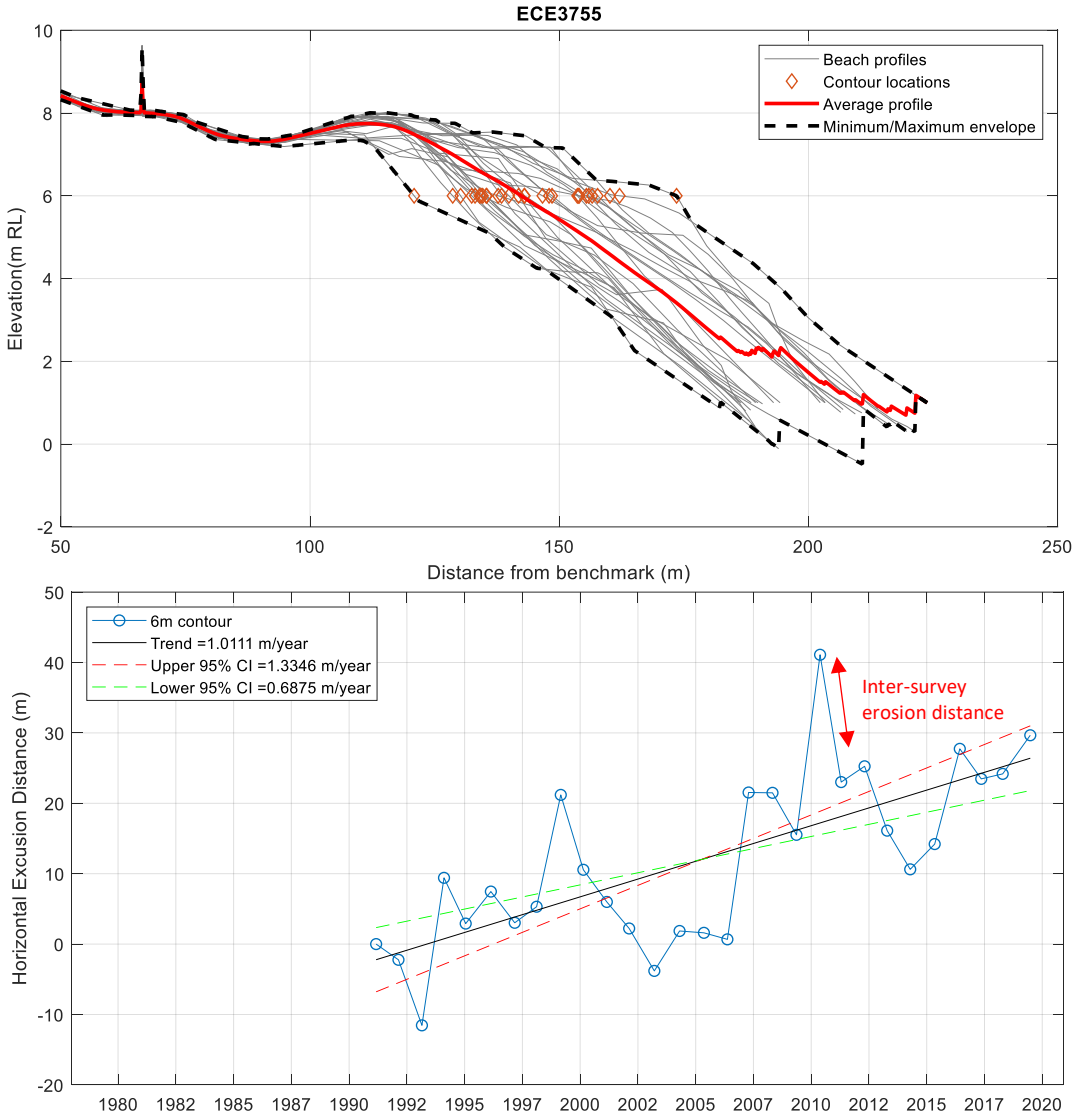

Figure 4.48: Example of beach profile ECE3755 used to assess short term erosion along the Kaitorete Spit.

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

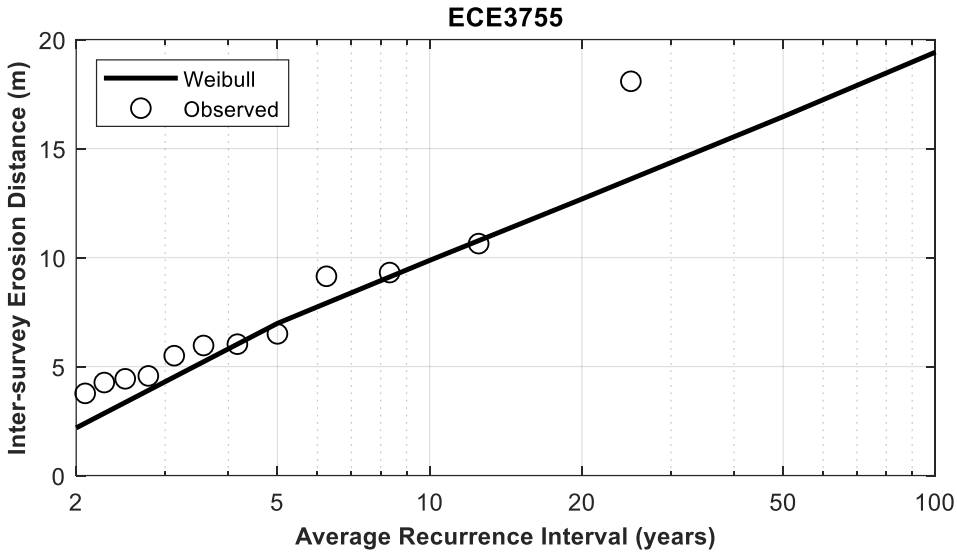

Figure 4.49: Example of extreme value curve for inter-survey storm cut distances at profile ECE3755 on Kaitorete Spit.

4.7.3 Long-term trends (LT)

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

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

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

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

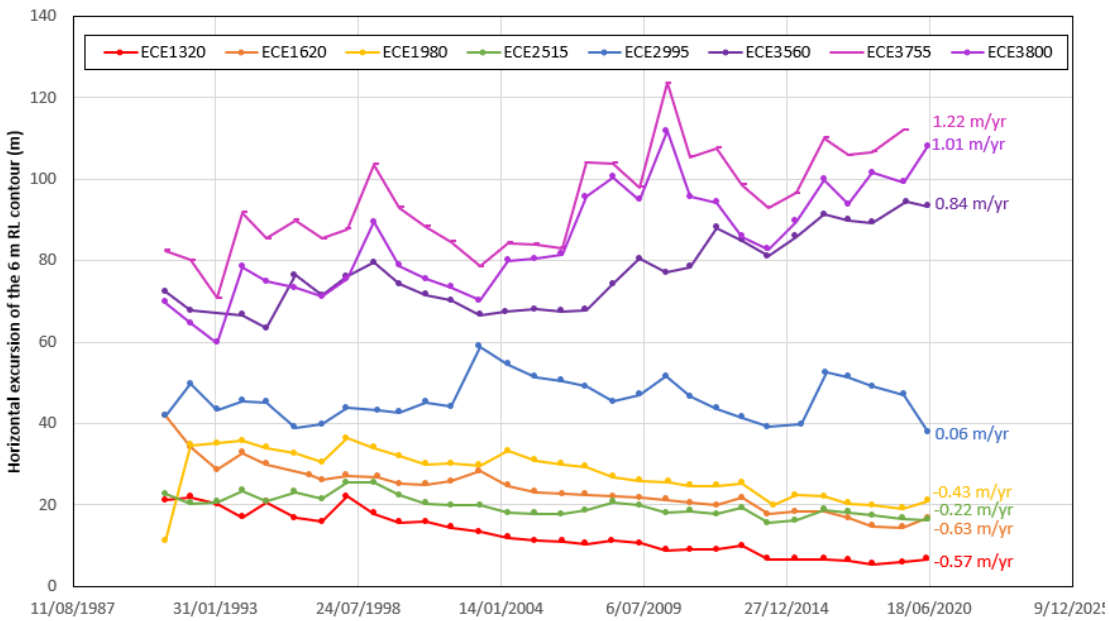

Figure 4.50: Horizontal excursion distances measured at the berm toe for profiles along Kaitorete Spit.

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

4.7.4 Response to sea level rise (SLR)

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

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

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

Where:

SLR = SLR (m).

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

h = height of the berm above the beach step.

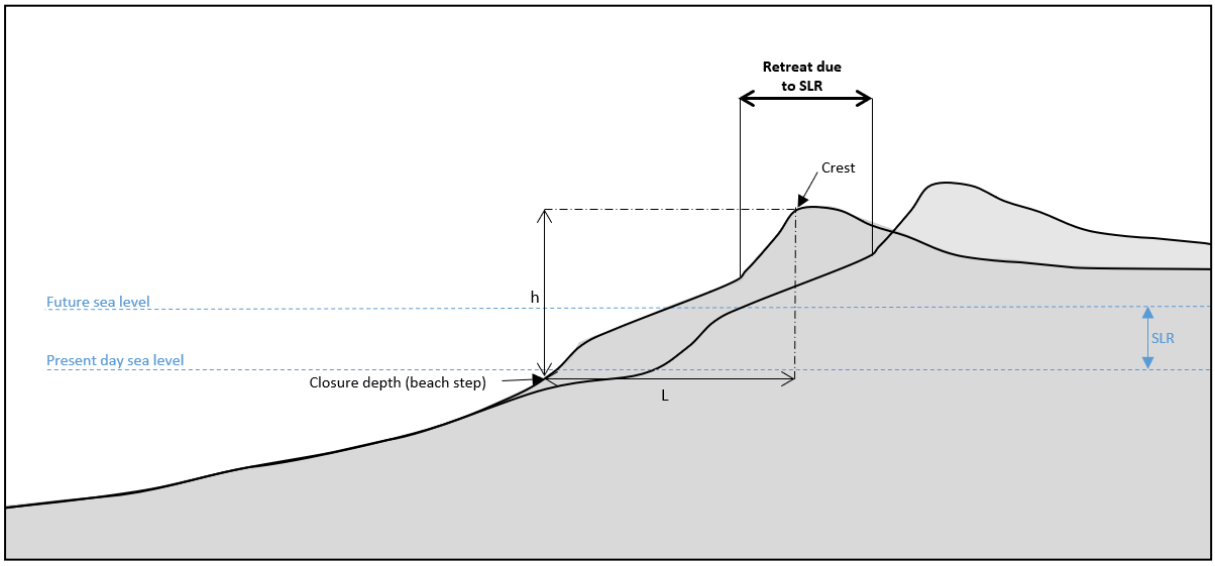

Figure 4.52: Conceptual diagram of SLR response along mixed sand gravel beach.

4.7.5 Summary of components

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

4.7.6 Uncertainties

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

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

Table 4.30: Summary of adopted components along the Kaitorete Spit

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

5 Coastal erosion results

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

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

5.1 Christchurch open coast

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

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

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

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

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

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

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

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

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

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

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

*Negative values are erosion and positive values are accretion.

5.2 Sumner

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

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

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

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

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

5.3 Taylors Mistake

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

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

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

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

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

5.4 Avon-Heathcote Estuary

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

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

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

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

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

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

5.5 Harbours (detailed sites)

5.5.1 Lyttelton Harbour

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

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

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

under high SLR is up to -32 m. For the harbour banks, the 2050 P5% ASCE under low SLR ranges from -6 to -27 m and by 2130 under high SLR, is up to -36 m.

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

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

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

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

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

5.5.2 Akaroa Harbour

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

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

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

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

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

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

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

5.6 Banks Peninsula (regional hazard screening sites)

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

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

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

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

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

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

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

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

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

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

5.7 Kaitorete Spit

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

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

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

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

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

6 Coastal inundation methodology

6.1 Conceptual approach

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

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

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

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

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

Where:

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

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

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

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

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

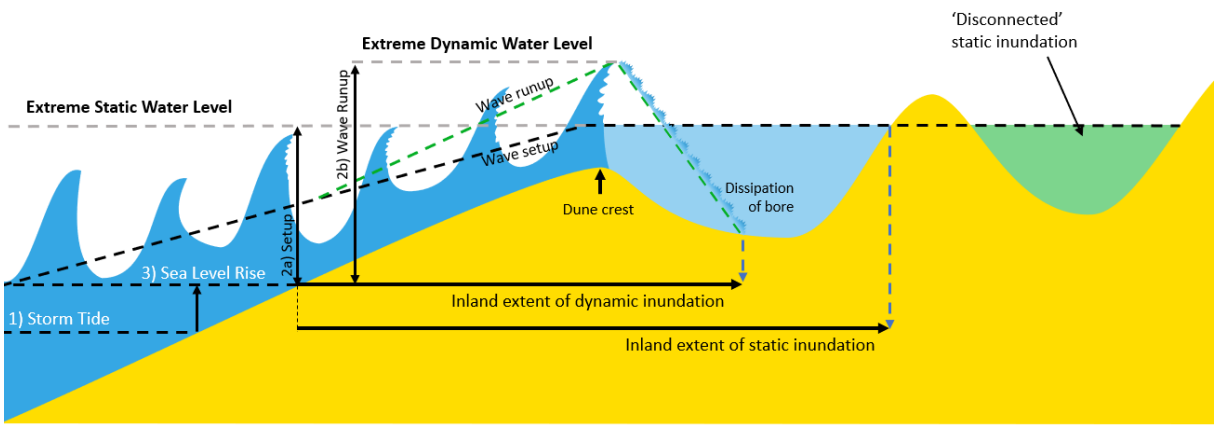

Figure 6.1: Schematisation of extreme water level components and combined extreme water levels.

Assessing and mapping coastal inundation takes a two part approach:

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

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

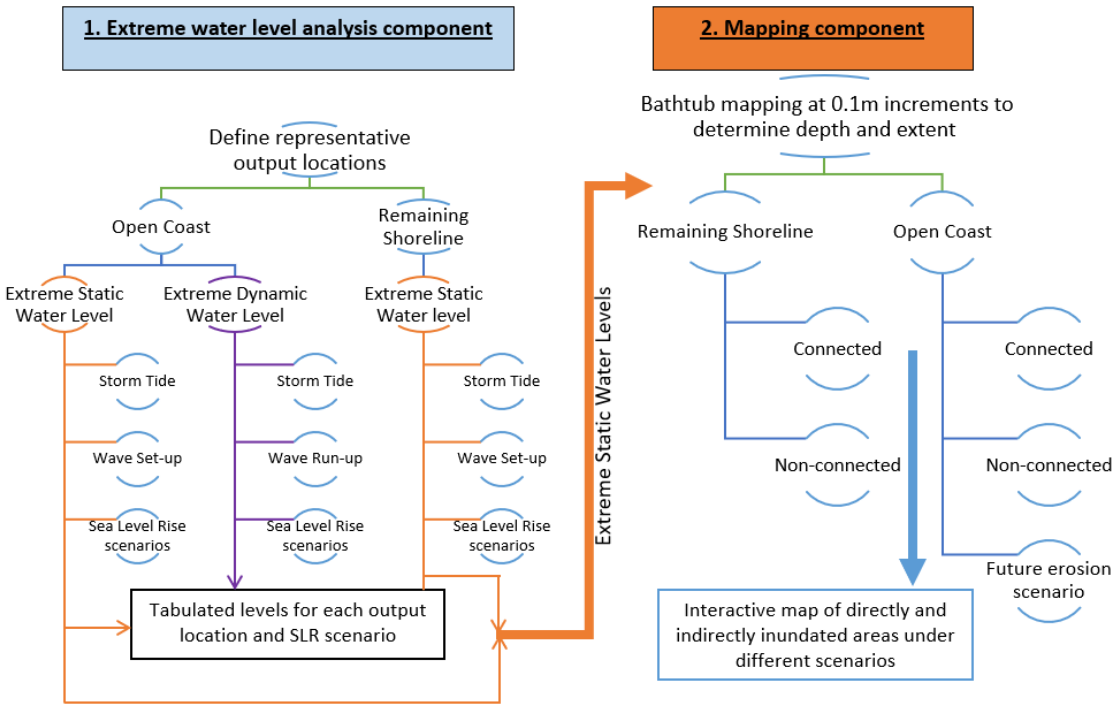

Figure 6.2: Proposed conceptual approach for inundation assessment and mapping.

6.2 Assessment level

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

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

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

NIWA (2015) also includes information on joint occurrence of storm tide and wave height and provides methods for calculating wave set up and run up for output locations along the open coast (excluding the Banks Peninsula). However, these levels are based on a hindcast from 1970 to 2000

and does not consider the effects of the 2018 storm events. As subsequent analysis of tide gauges by Goring (2018) and GHD (2021) show that 100-year ARI storm tide levels for Sumner are 0.2 m higher, the NIWA (2015) data has not been used for this study. Note that NIWA (2015) did not derive extreme levels from the Sumner tide gauge due to wave events affecting the quality of the water record (now resolved), however, the 100-year ARI level for Sumner is included in the report based on the Coastal Calculator.

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

6.2.1 Detailed inundation hazard assessment

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

Full probabilistic approach

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

- Christchurch open coast.

Quasi-probabilistic approach

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

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

6.2.2 Regional inundation hazard screening assessment

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

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

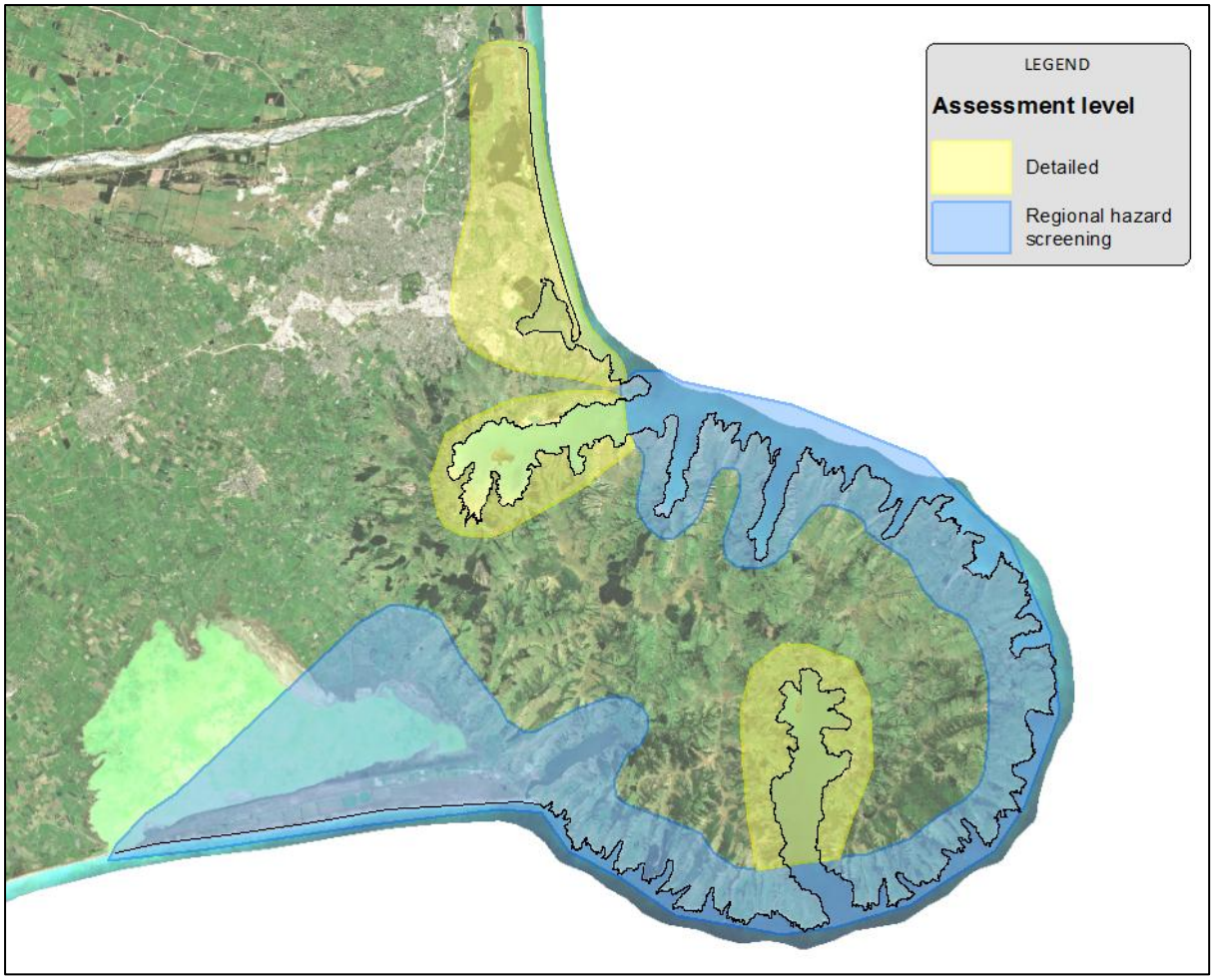

Figure 6-3 Christchurch district showing adopted extents and level of detail for the coastal inundation hazard assessment.

6.3 Scenarios

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

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

Table 6.1: Proposed assessment scenarios for inundation look up tables

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

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

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

[3] Christchurch open coast only.

[4] Both full probabilistic and quasi-probabilistic.

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

6.4 Mapping to determine inundation extent and depth

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

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

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

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

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

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

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

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

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

6.4.1 Inundation protection structures

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

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

7 Coastal inundation analysis

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

7.1 Christchurch open coast

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

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

7.1.1 Input data

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

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

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

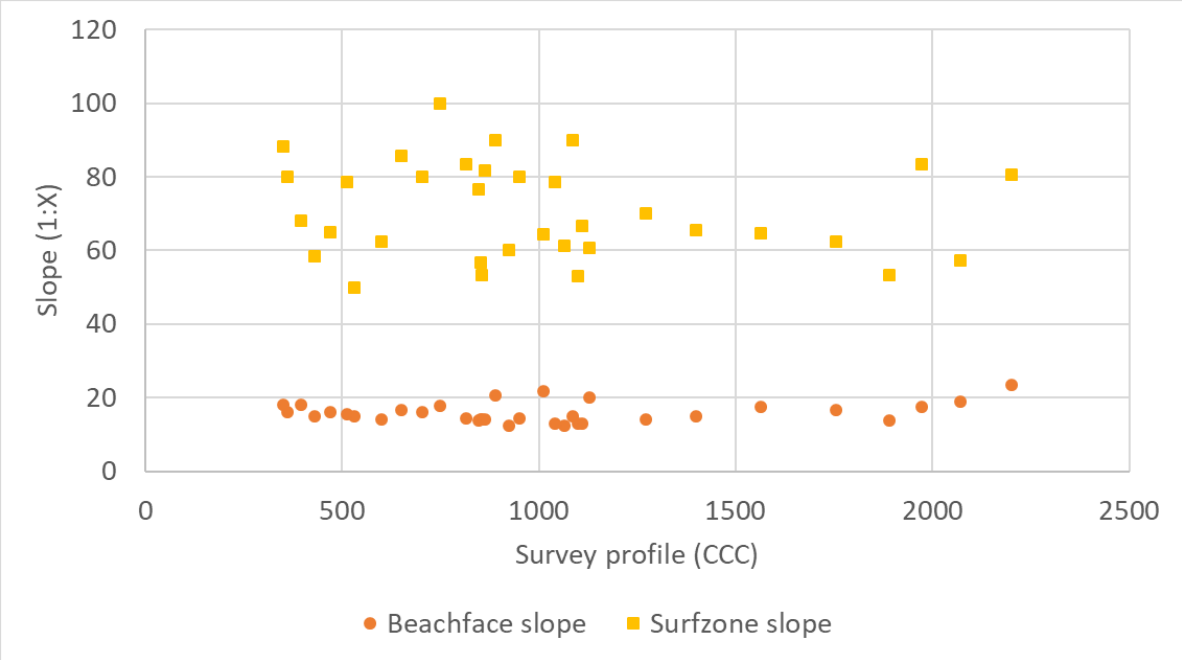

Figure 7.1: Alongshore beachface and surfzone slopes based on averaged surveyed profiles (note that CCC beach profiles run from south to north, e.g. CCC2000 is farthest north).

7.1.2 Analysis of extreme water levels

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

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

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

7.1.2.1 Validation of XBeach model

For the selection of appropriate wave setup formula, the numerical model XBeach NH (Deltares, 2015) has been utilised. XBeach NH (non-hydrostatic) is a numerical model that is able to transform offshore waves to the nearshore and simulate wave-induced set up and wave run up (see example of model in Figure 7.2). Two historic storms have been simulated by XBeach to extract wave run up and setup levels to compare with field data and values calculated by the empirical formulas. These

storm events were selected based on NIWA (2015), which include recorded storm event dates and surveyed debris lines following storms at several locations along the open coast beach. Only two storm events were found to be suitable based on available data and recorded levels (i.e. no further representative data points were available).

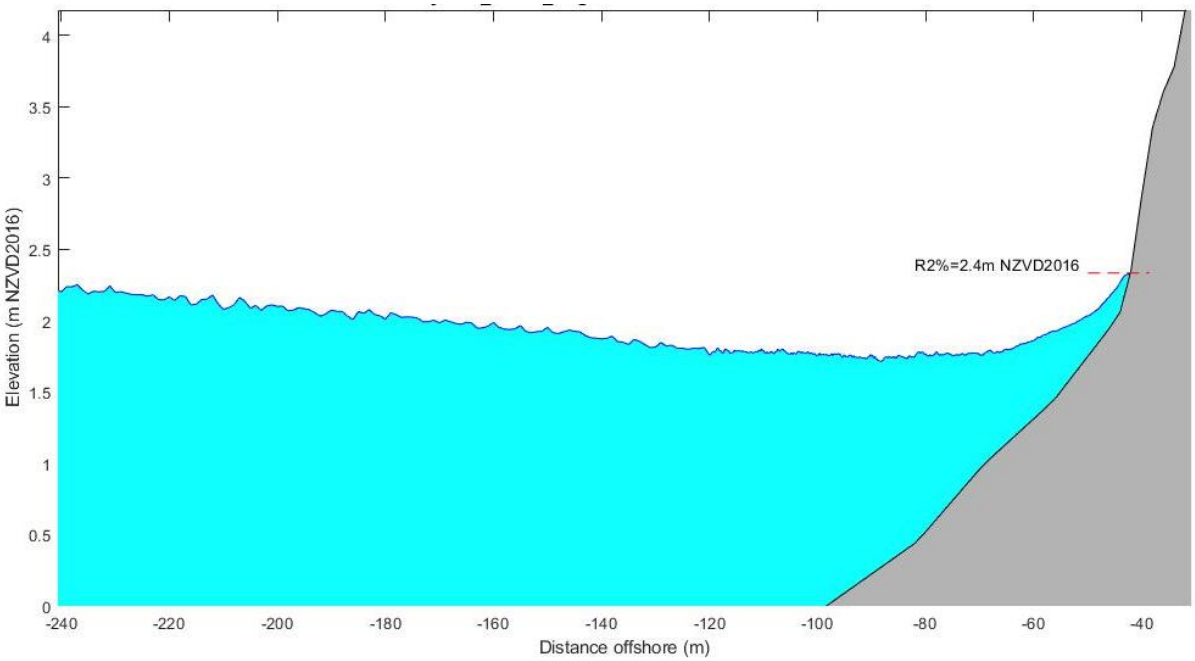

Figure 7.2: Example of XBeach NH model output.

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

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

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

[1]Source: NIWA (2015).
[2]$R_{2\%}$ (wave run up exceeded by 2% of wave run up events).

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

7.1.2.2 Calibration of empirical models

Wave set-up

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

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

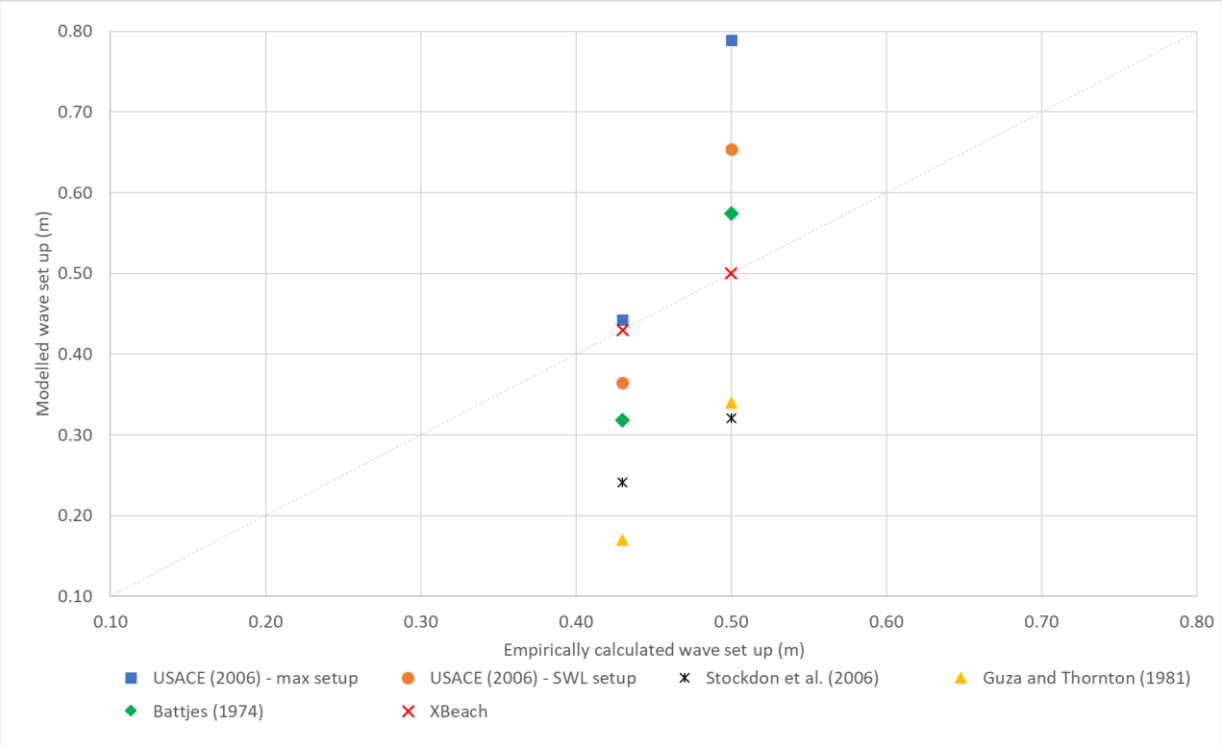

Figure 7.3: XBeach modelled versus empirically calculated wave set up.

Wave run-up

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

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

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

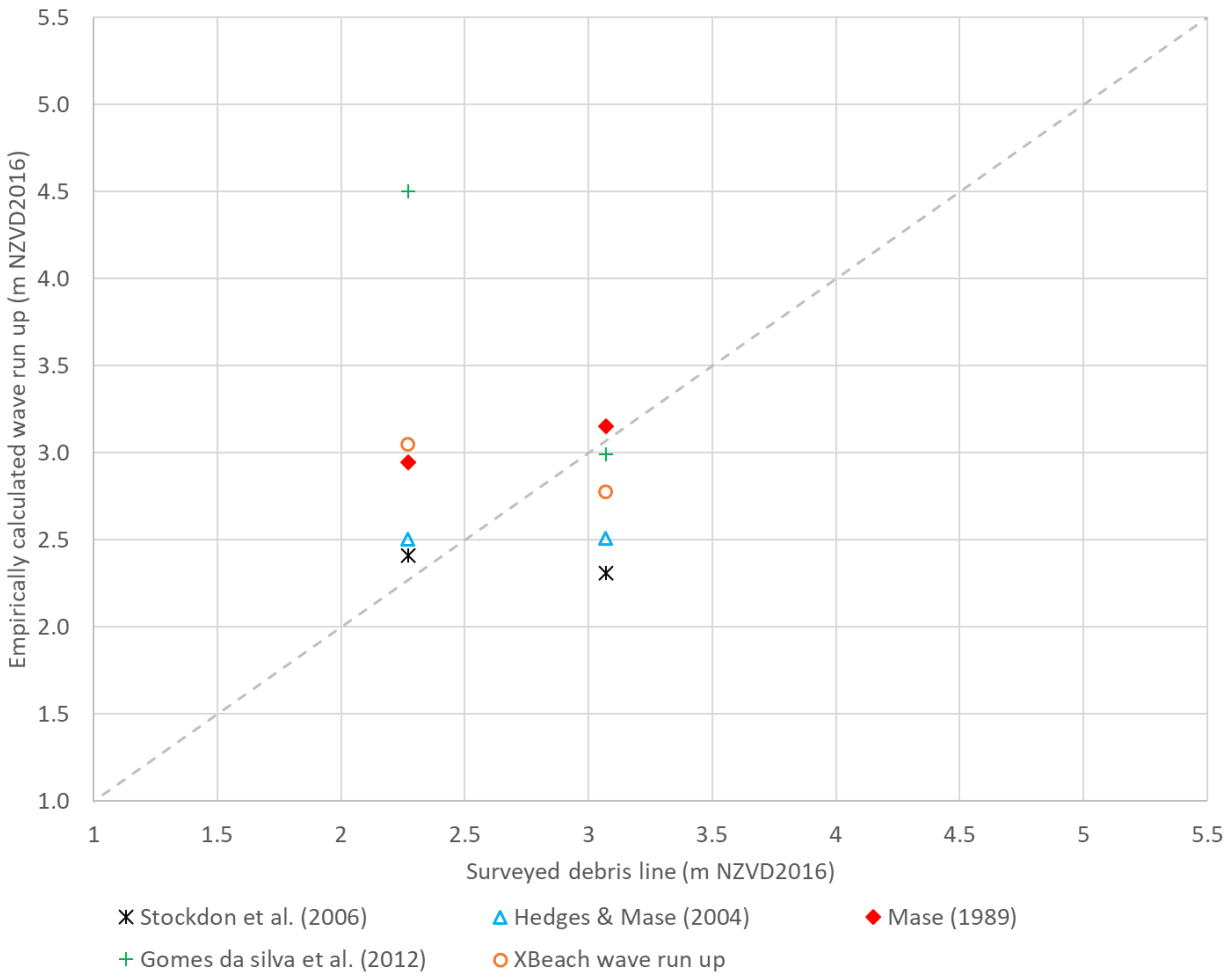

Figure 7.4: Comparison of surveyed debris line (assume wave run up extent) and empirically calculated wave run up.

7.1.2.3 Combined storm tide and wave effects

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

1. Develop hourly timeseries of nearshore wave heights based on the 1979-2019 wave hindcast data at the -10 m depth contour at each location along the shoreline provided by MetOcean.
2. Develop an equivalent hourly timeseries of water levels based on the 1994-2020 Sumner tide gauge record, which is expected to be representative of storm tide for the open coast. This water level includes the effect of the astronomical tide, storm surge and any medium-term sea level fluctuations.
3. Calculate wave effects (i.e. either set-up or run-up) for each timestep (1 hour) for the overlapping wave and water level timeseries (i.e. 1994-2019) and add to water level producing an extreme water level timeseries (i.e. either static or dynamic). As wave effects are dependent on wave height and beachface or surfzone slope, extreme water level timeseries have been created separately for the open coast from Waimakariri to Southshore, Sumner and Taylor's Mistake.
4. Undertake an extreme value analysis (EVA) to derive the 'structural' or combined extreme values based on the created timeseries. Analysis has been undertaken using a peaks-over-threshold method and a Weibull distribution which was found to represent wave-dominated extremes most accurately (Shand et al., 2010). The thresholds were selected to suit each individual area such that only extreme storms are included, with the EVA giving a reasonable fit through the data without the confidence intervals becoming too wide.

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

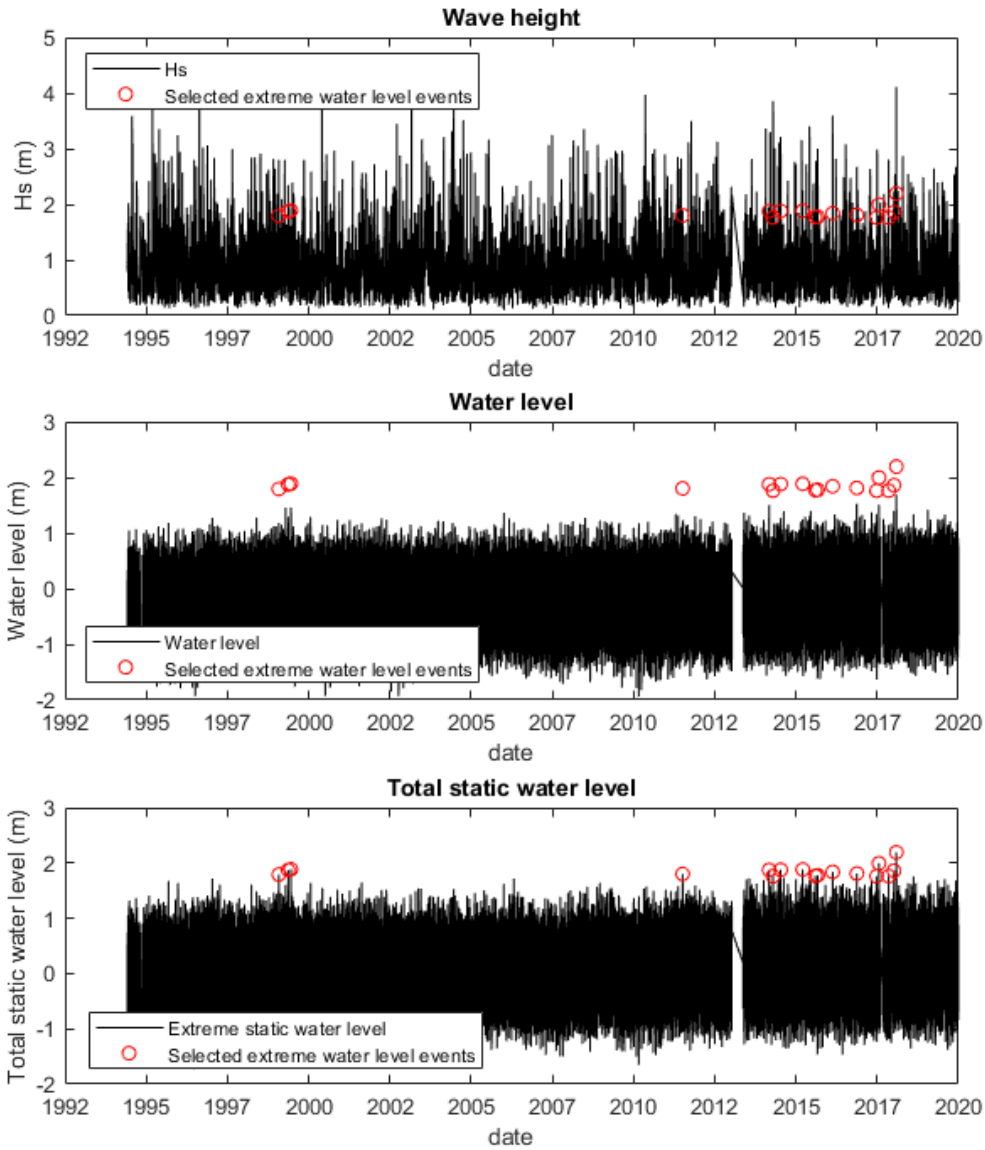

Figure 7.5 Example of extreme water level timeseries derived from a full wave height and water level timeseries.

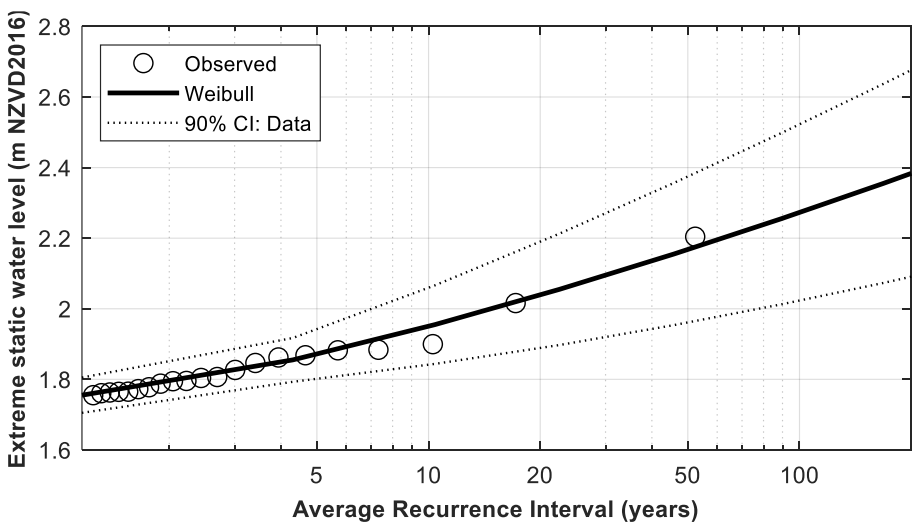

Figure 7.6 Example of extreme values analysis for static inundation level for the Christchurch open coast.

7.1.3 Attenuation of run-up

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

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

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

Where:

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

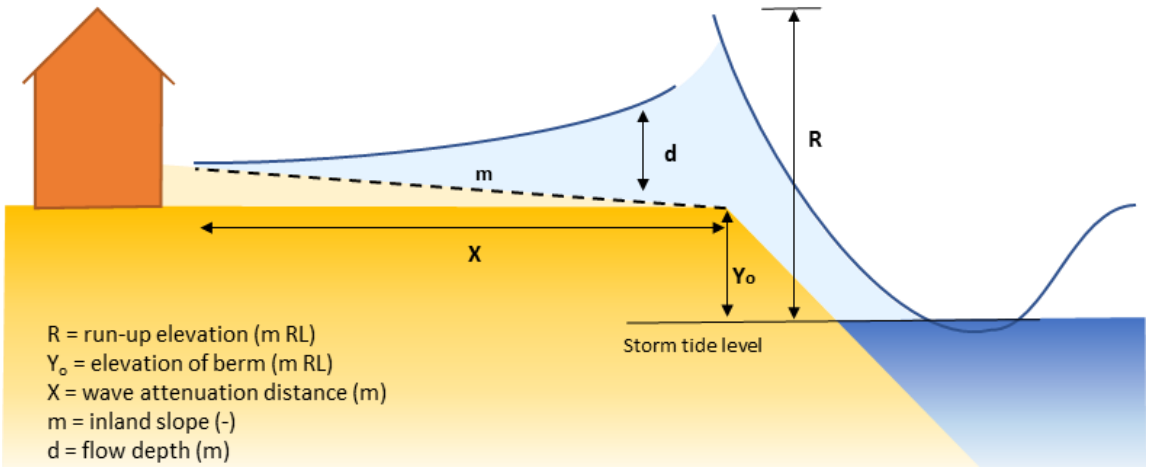

Figure 7.7: Run-up attenuation definition sketch (modified from Cox and Machemehl, 1986).

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

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

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

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

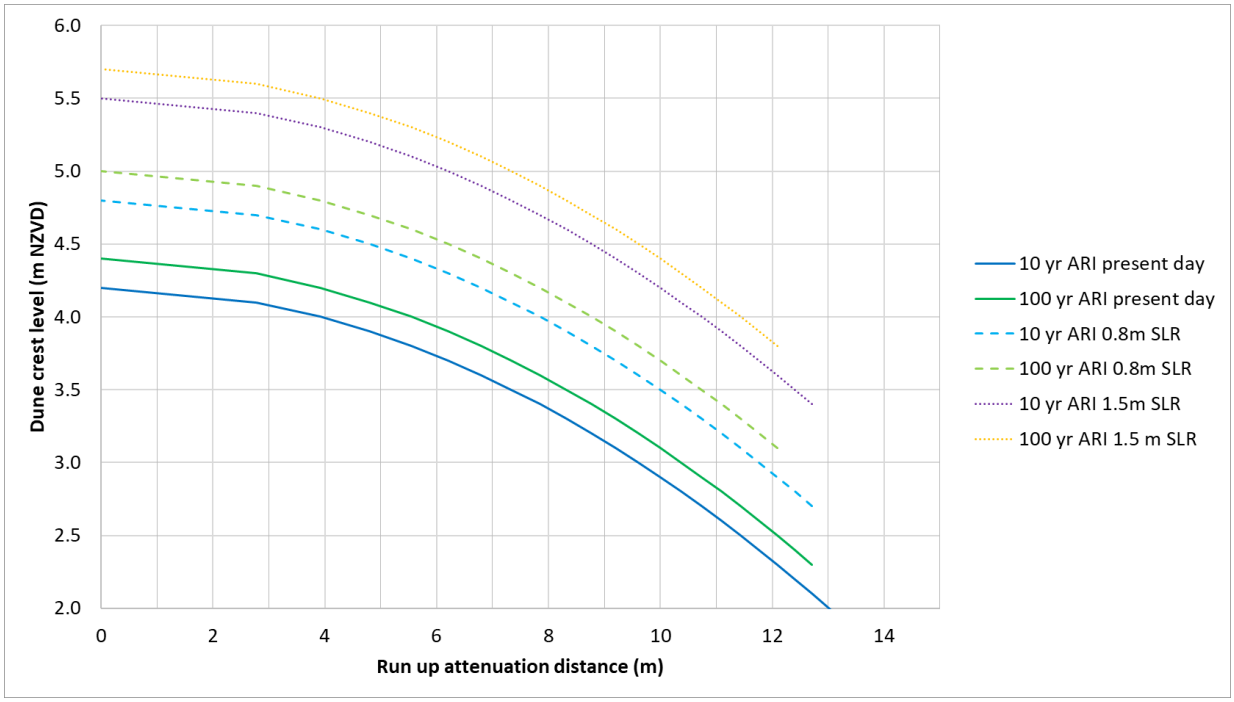

Figure 7.9: Run up attenuation distances from dune crest for a range of dune/seawall crest levels for a range of scenarios based on the modified Cox and Machemehl (1986) method.

7.1.4 Future erosion effects on static inundation

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

1 Original beach profile (C1065), assuming no beach response.

2 Profile maintaining original dune shape, retreated to predicted shoreline position at 2130.

3 Profile with dunes removed by erosion, retreated to predicted shoreline position at 2130.

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

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

retains its current position and geometry. However, when dunes are removed either due to erosion or anthropogenic interventions the backshore may become susceptible to static inundation as sea level rises.

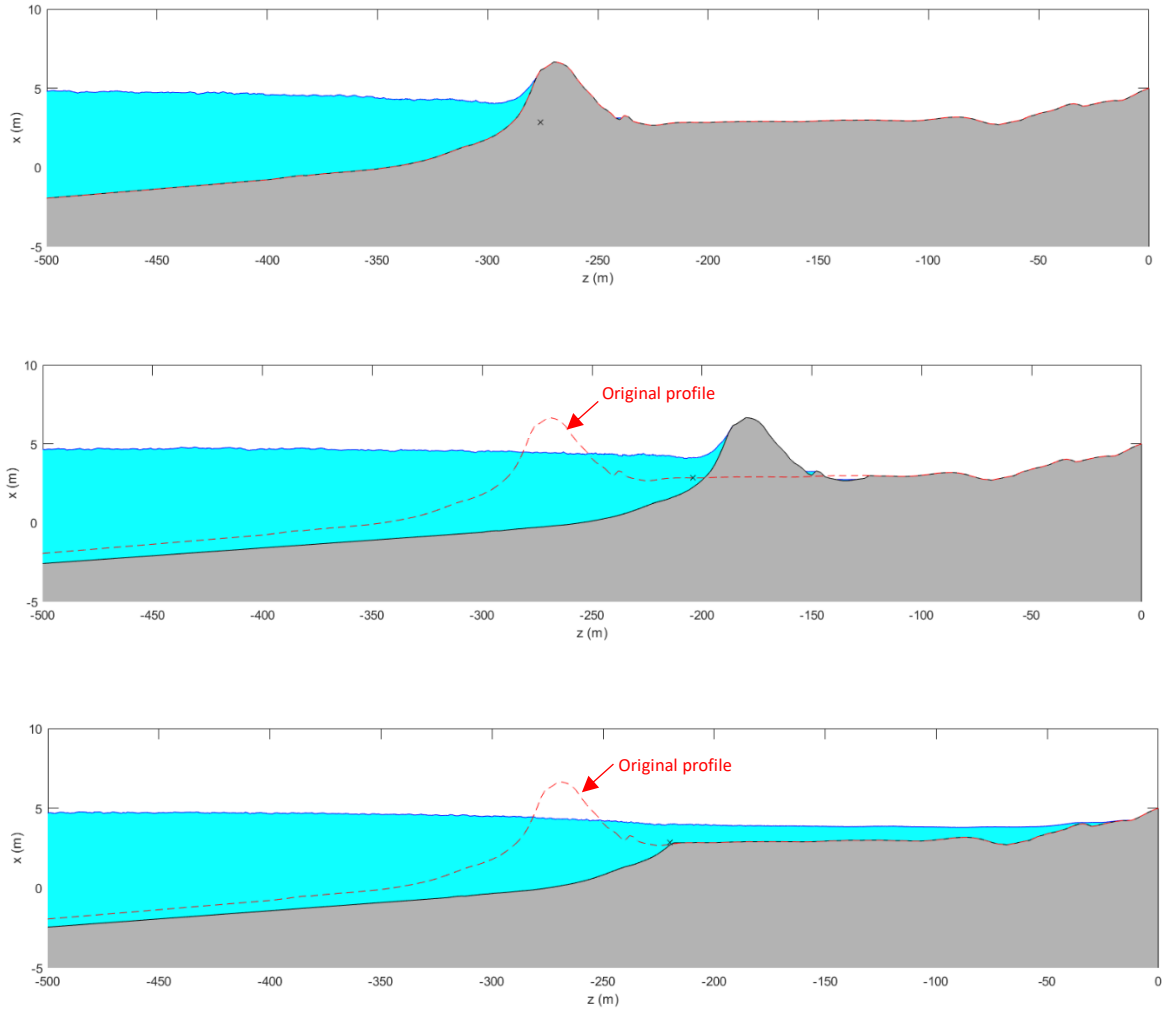

Figure 7.10: Xbeach results for original profile C1065 (1), retreated profile while maintaining dune shape (2) and retreated profile with dunes eroded (3).

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

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

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

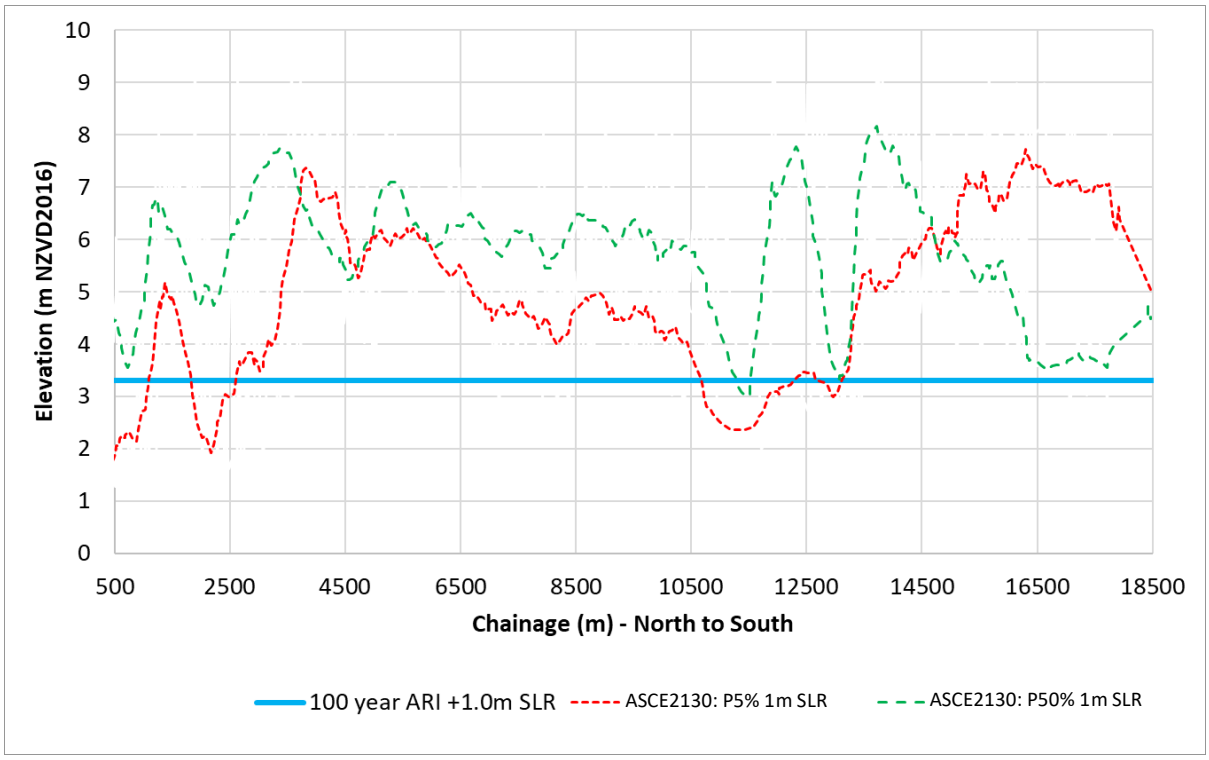

Figure 7.11: Alongshore backshore elevations at 2130 ASCE lines (both P50% and P5%) compared with 100 year ARI + 1 m SLR water level (chainage north to south)

7.1.5 Output locations

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

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

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

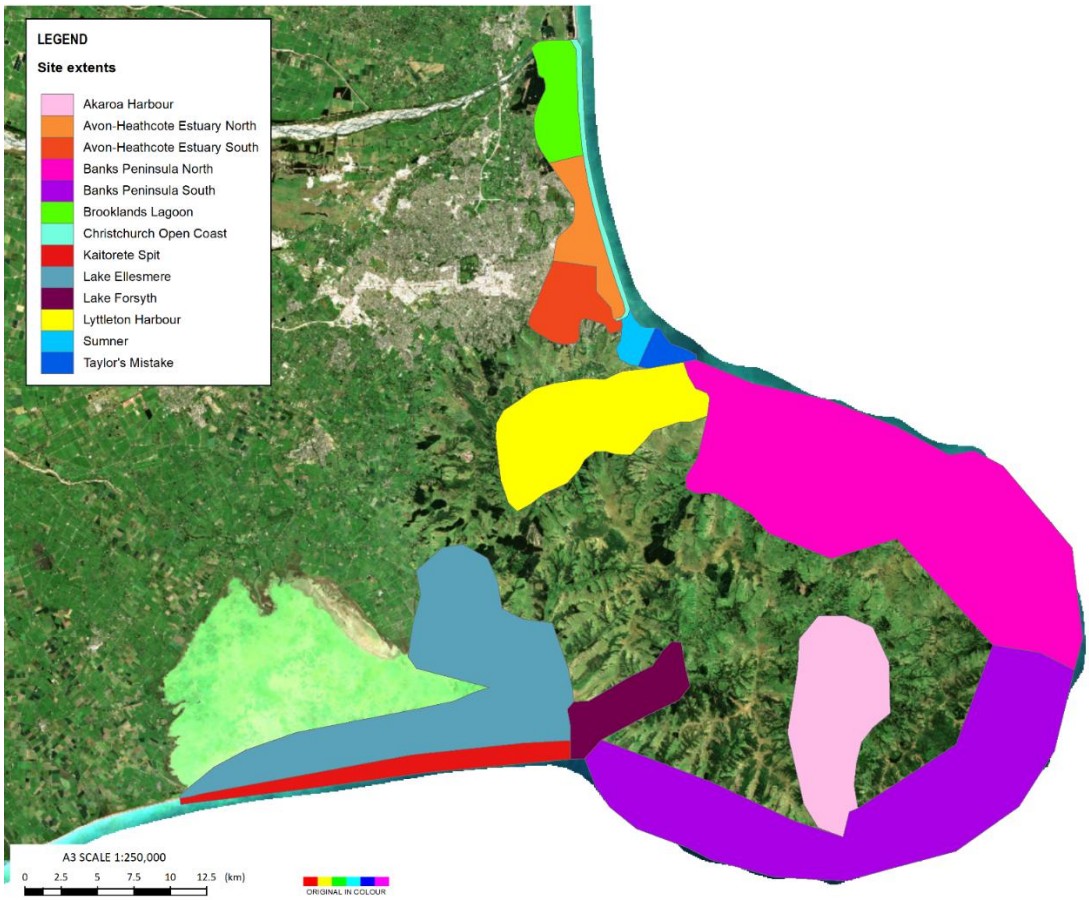

Figure 7.12: Site extents and location for Christchurch open coast, major harbours and estuaries and regional hazard screening sites.

7.2 Major harbours and estuaries

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

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

7.2.1 Input data

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

No water level data is available from the GHD (2021) report for Akaroa Harbour. Therefore, water levels for the Akaroa Harbour have been based on water levels from GHD (2021) at the Lyttelton

gauge, with an offset of the MHWS difference between the Lyttelton gauge (0.84 m NZVD2016) and Akaroa Harbour gauge (1.08 m NZVD2016) based on LINZ (2021). Table 7.2 shows the extreme water levels for the Akaroa Harbour. It should be noted that due to the difference in location, geometry and orientation of Lyttelton Harbour and Akaroa Harbour the exposure to storm surge may vary as well. NIWA (2015) suggest that the 100 year ARI storm tide levels at Birdlings Flat (southern side of Banks Peninsula) are approximately 0.1 m lower compared to Sumner (northern side of Banks Peninsula. However, as the 100 year ARI storm tide level at Lyttelton Port are in the order of 0.2 m lower than the 100 year ARI storm tide level at Sumner, it is reasonable to assume that storm surges within Lyttelton Harbour and Akaroa Harbour are similar.

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

	ARI							
Site	**1 yr**	**2 yr**	**5 yr**	**10 yr**	**20 yr**	**50 yr**	**100 yr**	**200 yr**
Akaroa Harbour	1.55	1.60	1.65	1.69	1.73	1.78	1.82	1.86

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

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

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

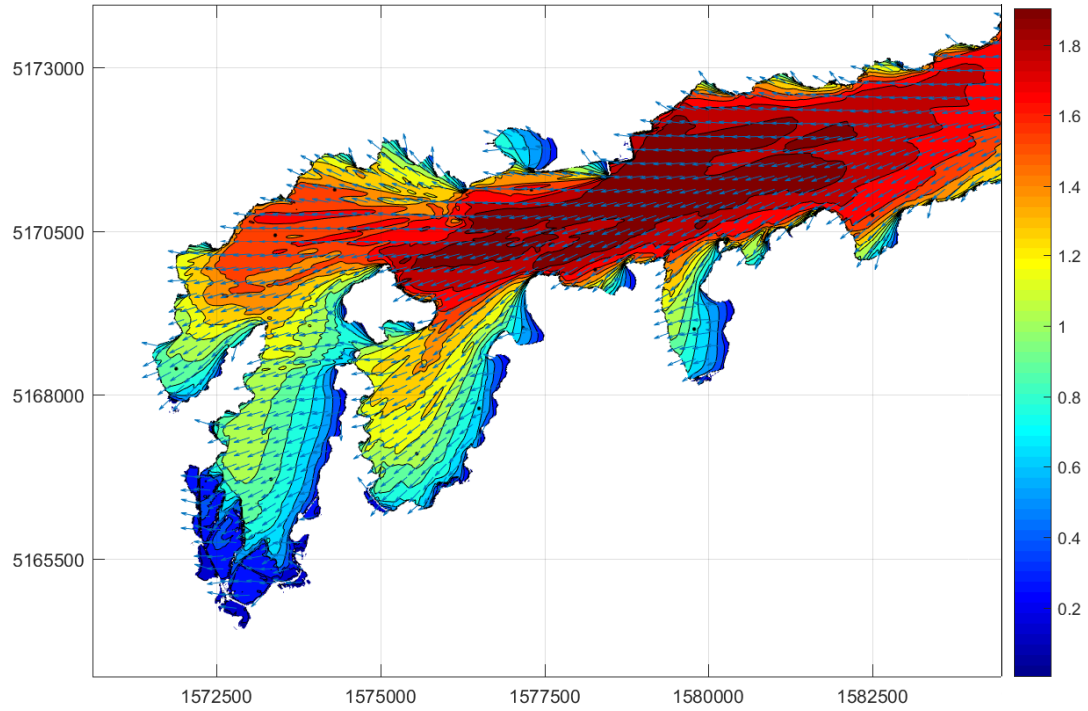

Figure 7.13: SWAN model results for Lyttelton Harbour using 100-year ARI easterly wind as input showing resulting significant wave height (H_s) in metres.

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

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

7.2.2 Analysis of extreme static water levels

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

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

$$\bar{\eta} = 0.17 \cdot H_s \qquad (7.2)$$

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

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

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

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

Avon-Heathcote	Akaroa Harbour	Lyttelton Harbour
0.15	0.25	0.25

7.2.3 Output locations

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

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

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

7.3 Regional hazard screening sites

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

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

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

7.3.1 Input data

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

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

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

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

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

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

7.3.2 Analysis of extreme water levels

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

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

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

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

7.3.3 Output locations

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

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

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

8 Coastal inundation results

8.1 Christchurch open coast

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

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

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

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

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

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

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

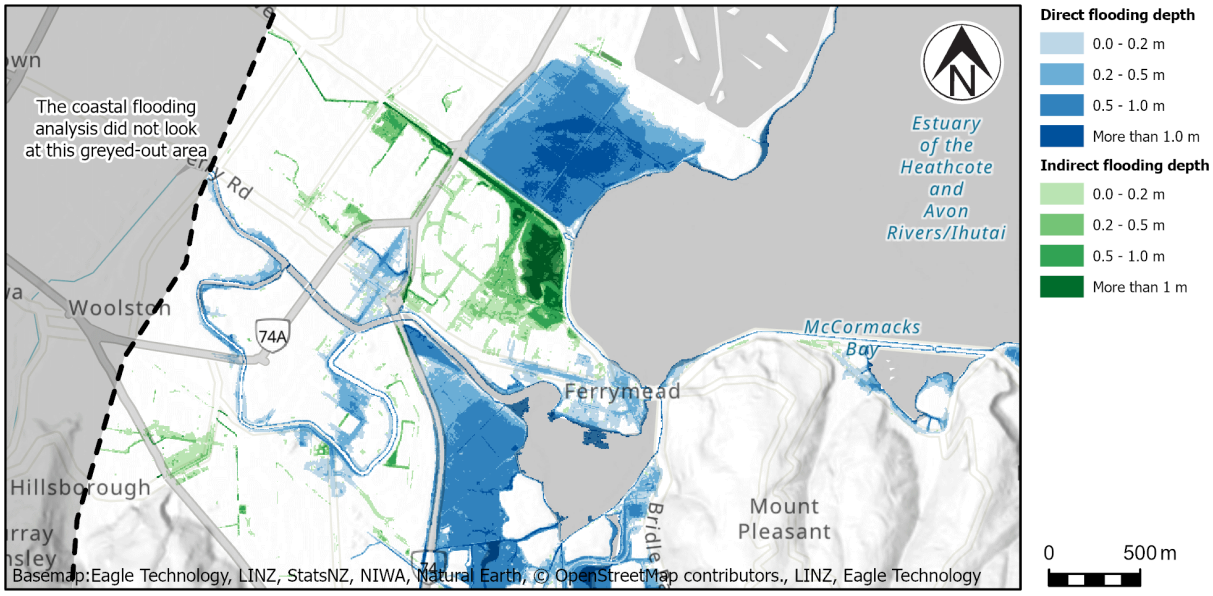

Figure 8.1: Example of static inundation depths for 100 year ARI water levels with for current-day sea level including areas connected to the shoreline (blue shading) and separate inundation areas that are not connected to the coast (green shading).

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

8.2 Major harbours and estuaries

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

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

Return period	Brooklands Lagoon	Avon-Heathcote North	Avon-Heathcote South	Lyttelton Harbour	Akaroa Harbour
1 year ARI	1.4	1.5	1.5	1.6	1.8
10 year ARI	1.6	1.7	1.6	1.7	2.0
100 year ARI	1.8	2.0	1.8	1.8	2.1
100 year ARI +0.4 m SLR	2.2	2.4	2.2	2.2	2.5
100 year ARI +1.5 m SLR	3.3	3.5	3.3	3.3	3.6

Table 8.2 shows similar static inundation levels within the Brooklands Lagoon, Avon-Heathcote Estuary and Lyttelton Harbour ranging from 1.4 to 2.0 m NZVD2016 for present-day static inundation levels. This is a result of various factors influencing the water levels, such as exposure to waves, river discharge effects, wind set-up effects or exposure to storm surges. The static inundation levels within the Akaroa Harbour are 0.2-0.5 m higher compared to the other harbours, which is a result of

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

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

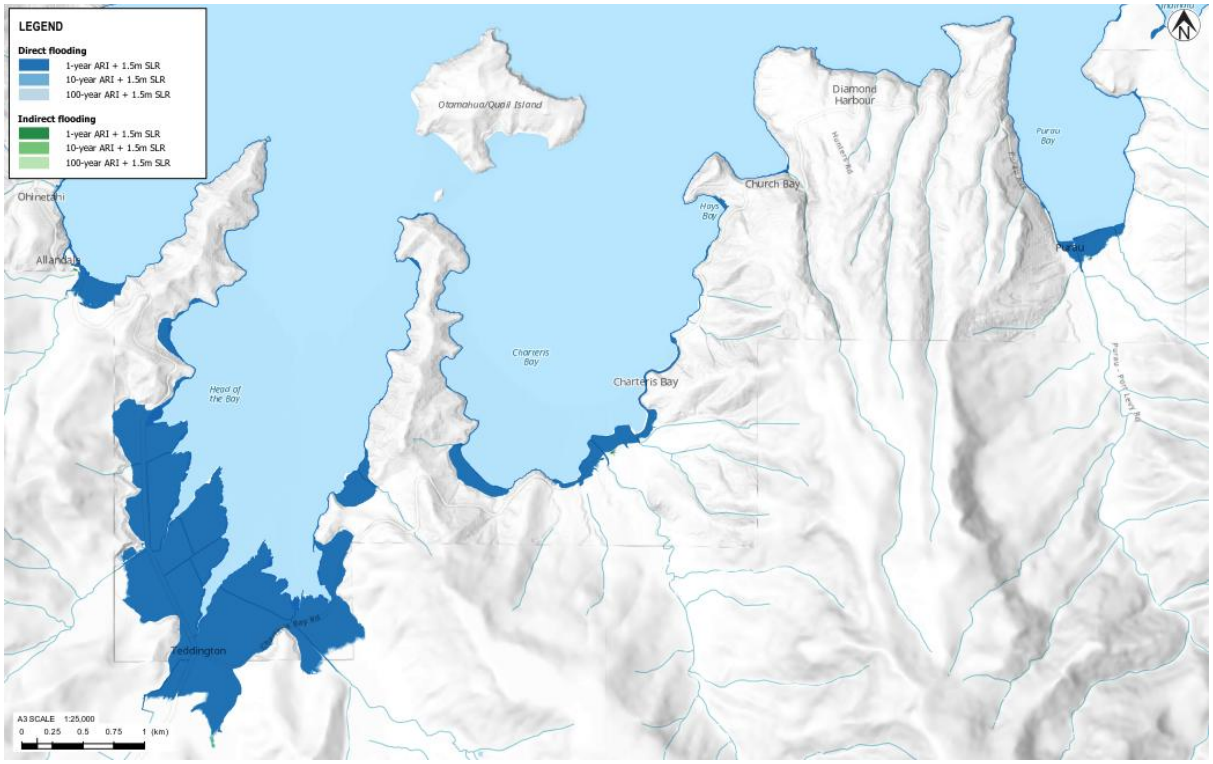

Figure 8.2: Example of static inundation extent map for Lyttelton Harbour (refer Appendix E for full map).

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

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

8.3 Regional hazard screening sites

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

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

Return period	Banks Peninsula North	Banks Peninsula South	Wairewa (Lake Forsyth)	Kaitorete Spit	Te Waihora (Lake Ellesmere)
1 year ARI	2.2	2.9	2.3	2.6	1.1
10 year ARI	2.5	3.4	2.7	2.9	1.4
100 year ARI	2.8	3.9	2.9	3.3	1.7
100 year ARI +0.4 m SLR	3.2	4.3	N/A	3.7	N/A
100 year ARI +1.5 m SLR	4.3	5.4	N/A	4.8	N/A

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

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

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

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

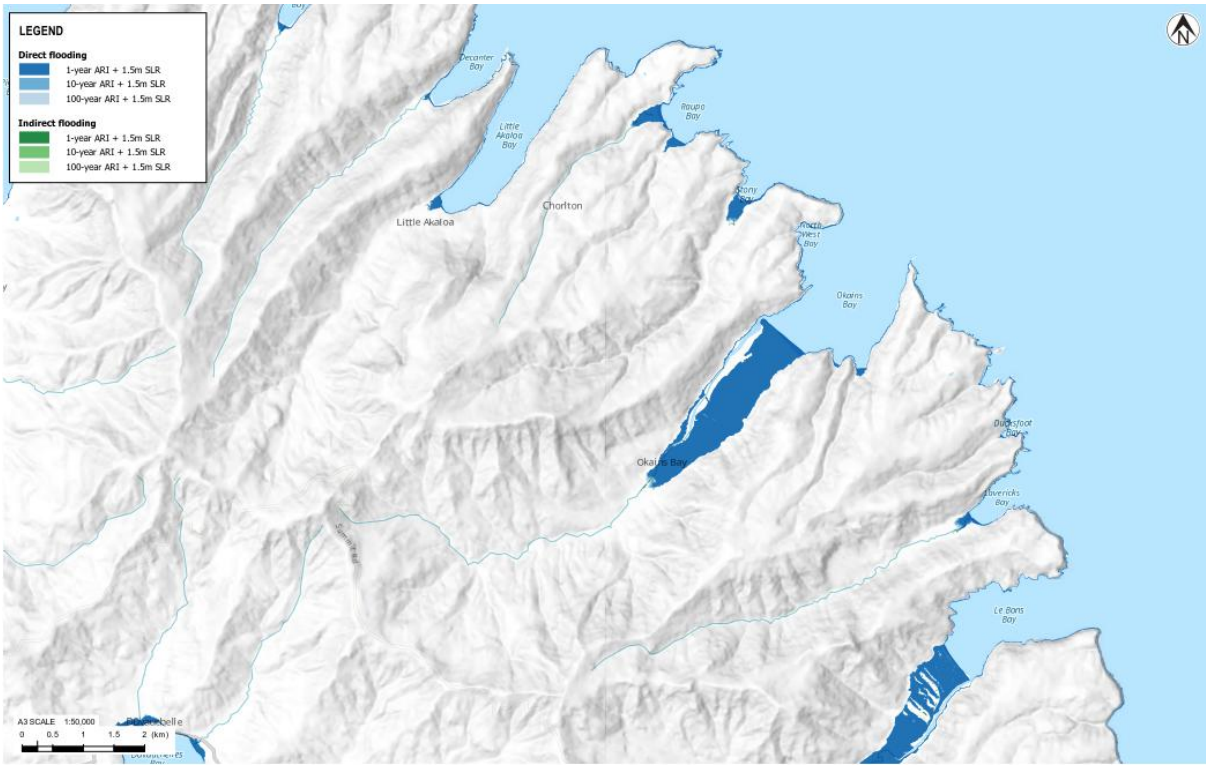

Figure 8.3: Example of static inundation extent map for Banks Peninsula (refer Appendix E for full map).

9 Rising groundwater assessment

9.1 Background

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

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

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

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

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

9.2 Christchurch urban flat-land area

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

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

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

9.3 Banks Peninsula

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

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

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

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

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

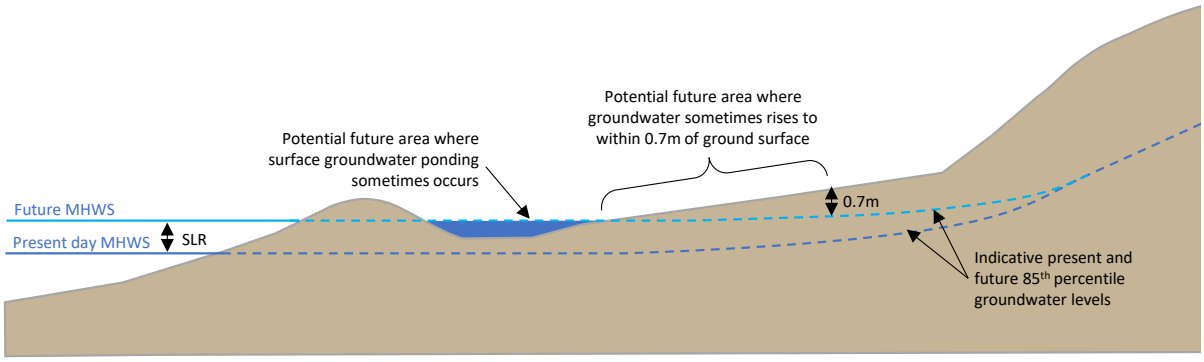

Figure 9.1: Conceptual model for indicative present-day and future groundwater levels for low-lying areas close to the coast around Banks Peninsula.

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

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

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

10 Applicability

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

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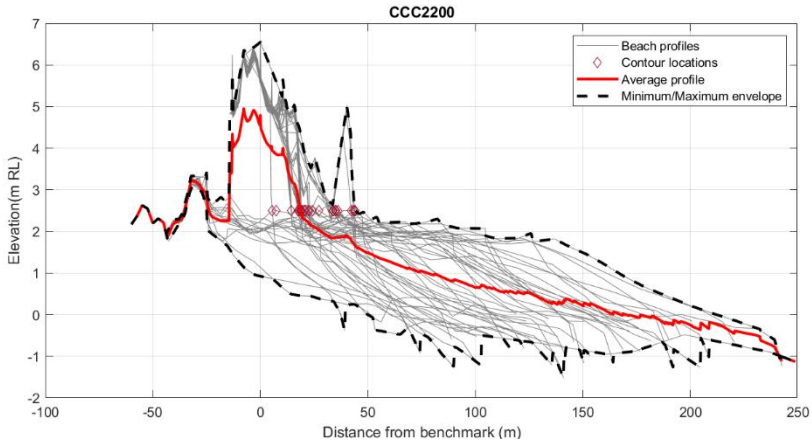
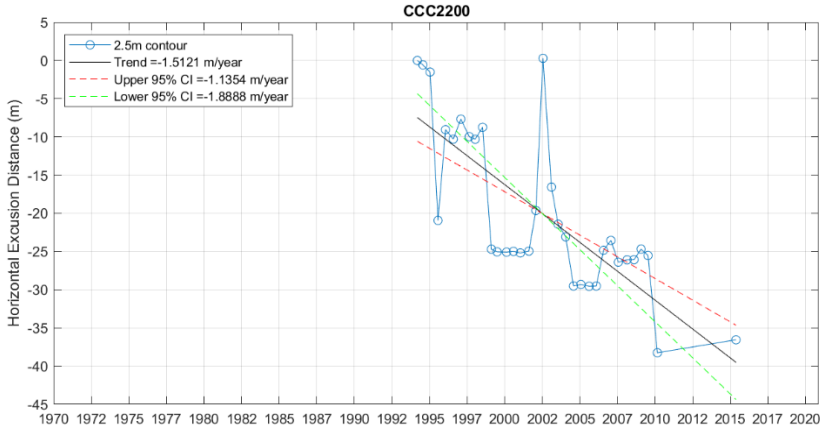
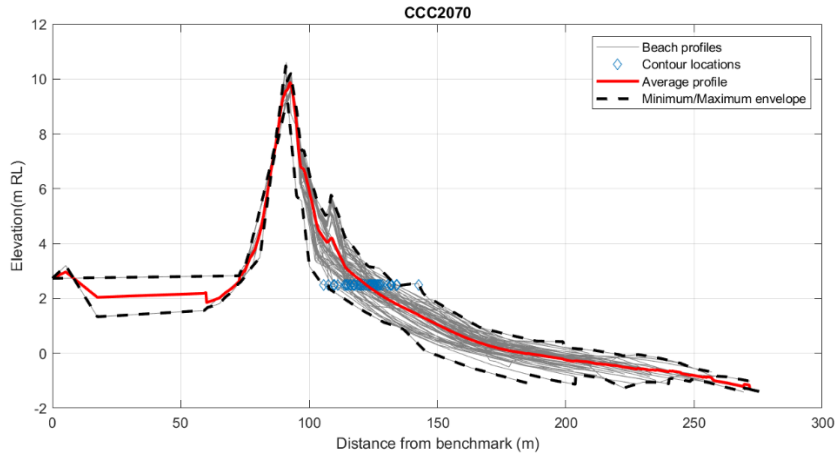
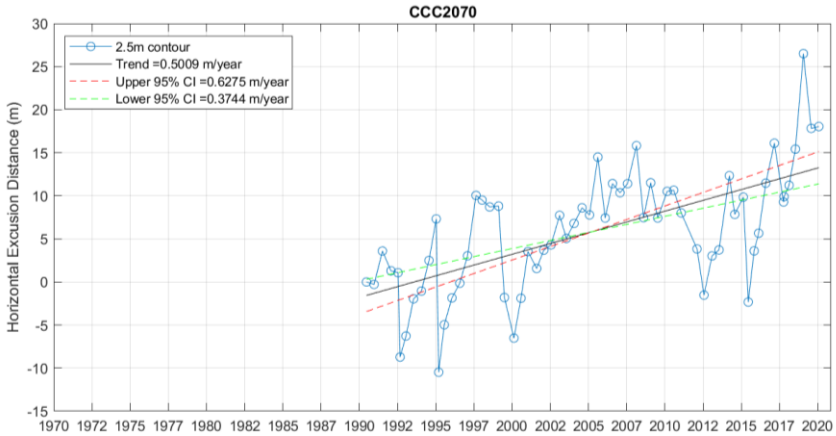
Appendix A: Beach profiles


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


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


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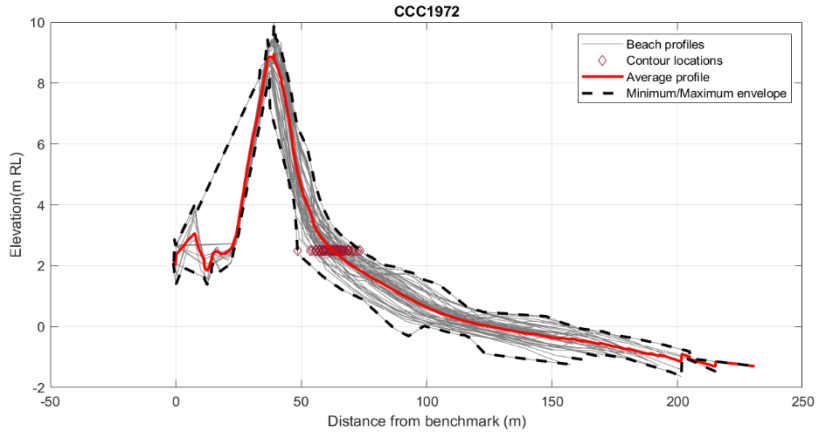
Horizontal Excusion Distance (m)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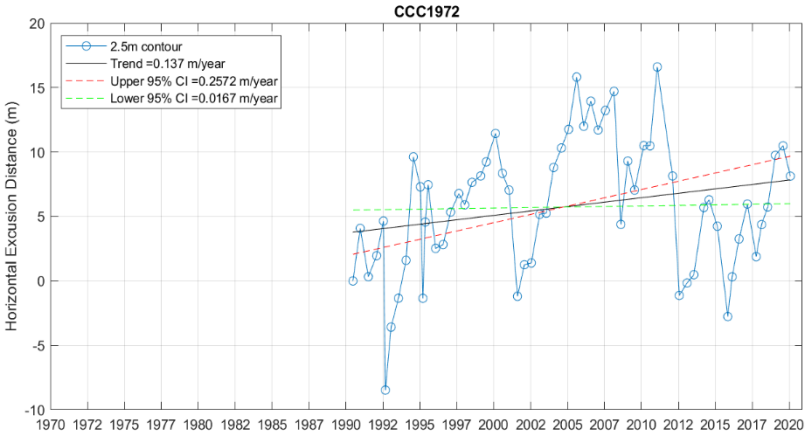
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


CCC1972
2.5m contour
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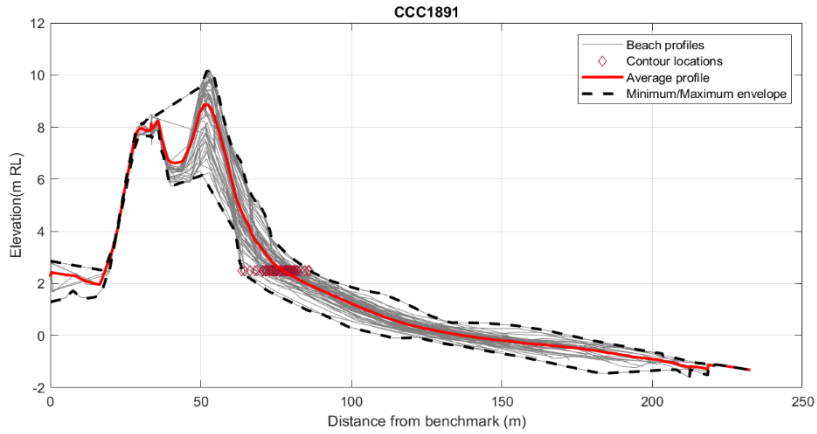
Lower 95% CI =0.0167 m/year
Horizontal Excusion Distance (m)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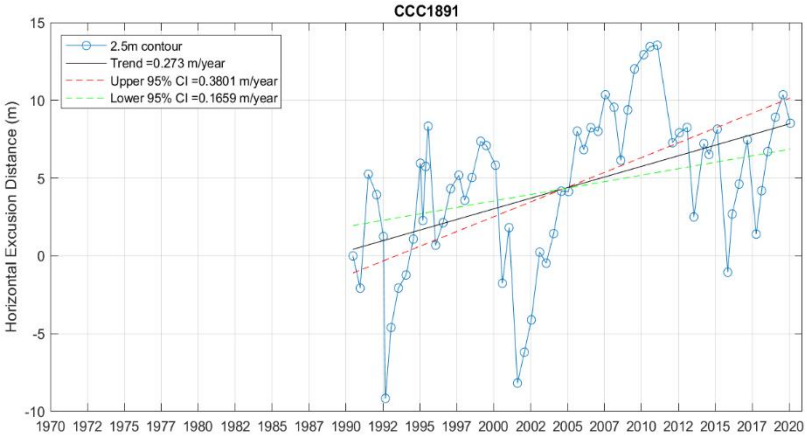
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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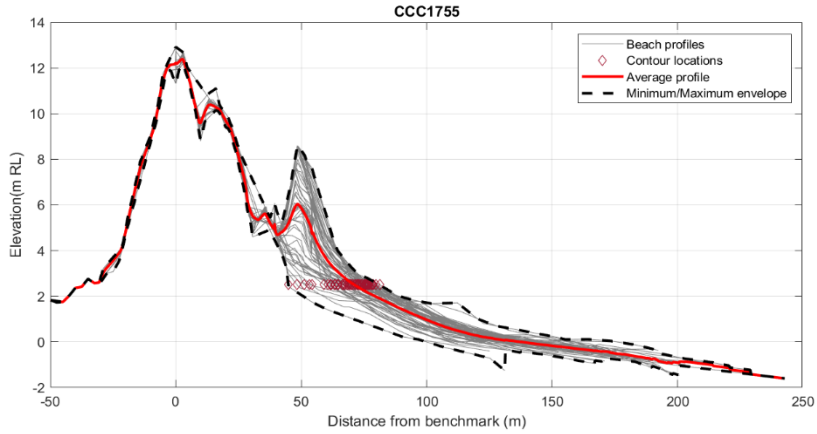
Lower 95% CI =0.1659 m/year
Horizontal Excusion Distance (m)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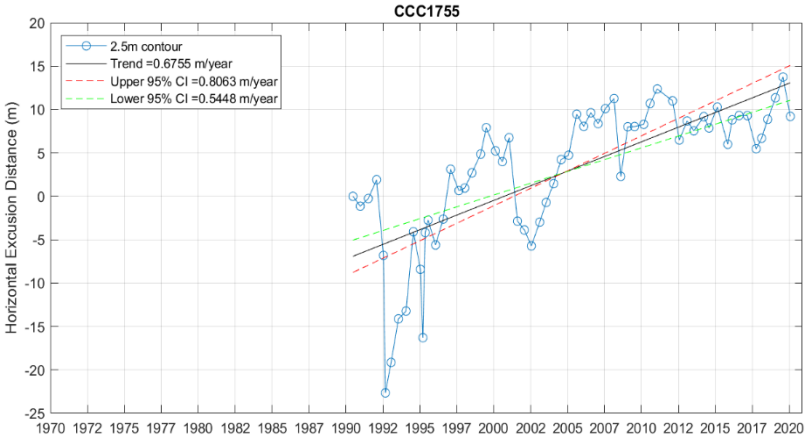
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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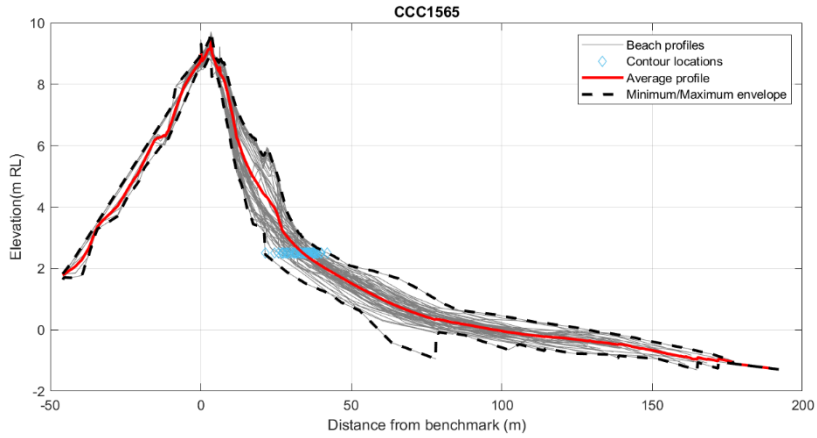
Lower 95% CI =0.5448 m/year
Horizontal Excusion Distance (m)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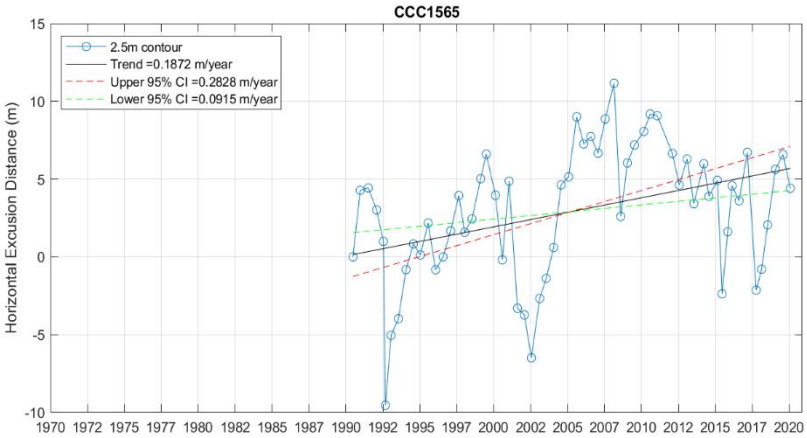
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


CCC1565
2.5m contour
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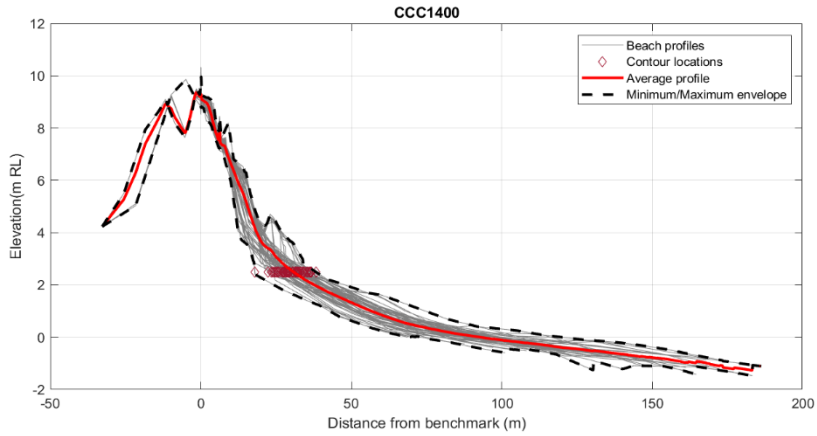
Upper 95% CI =0.2828 m/year
Lower 95% CI =0.0915 m/year
Horizontal Excusion Distance (m)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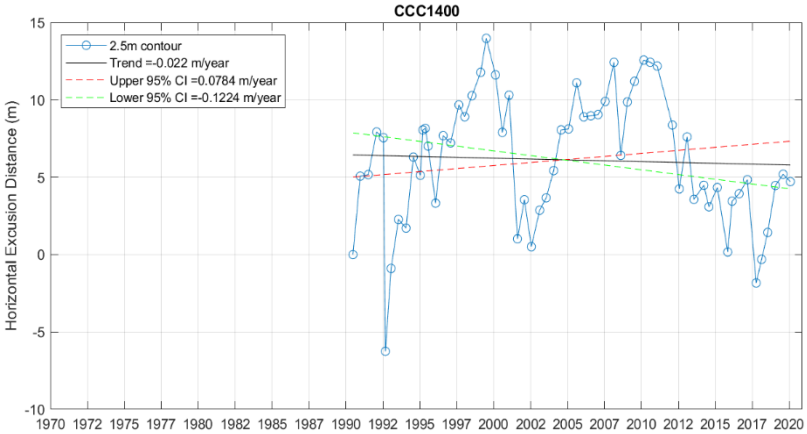
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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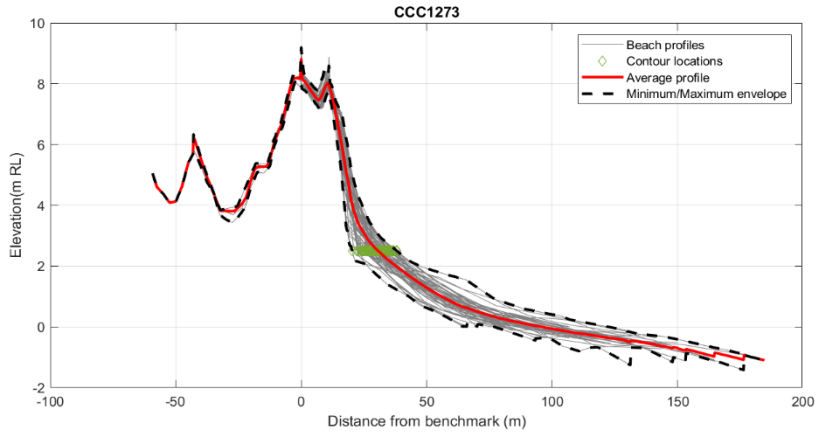
Lower 95% CI =-0.1224 m/year
Horizontal Excusion Distance (m)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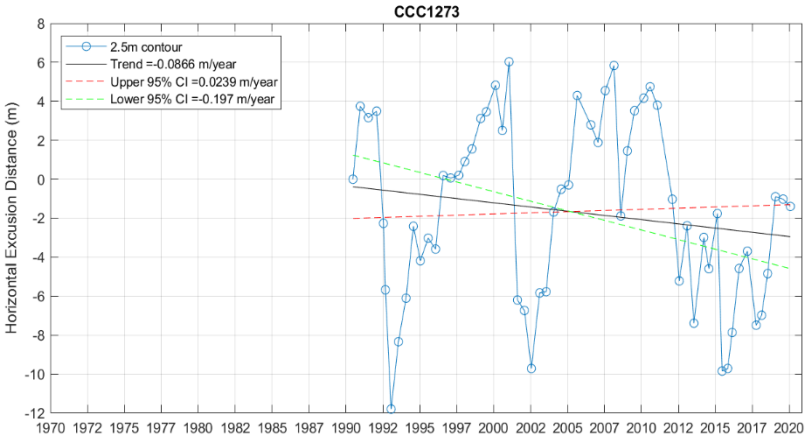
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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Upper 95% CI =0.0239 m/year
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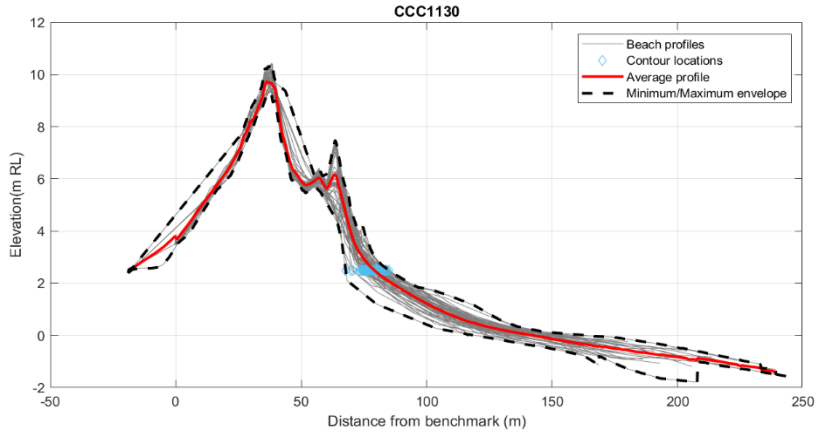
Horizontal Excusion Distance (m)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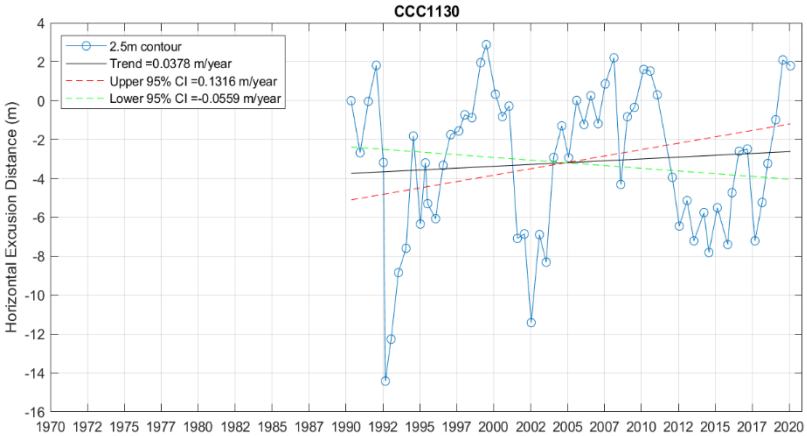
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


CCC1130
2.5m contour
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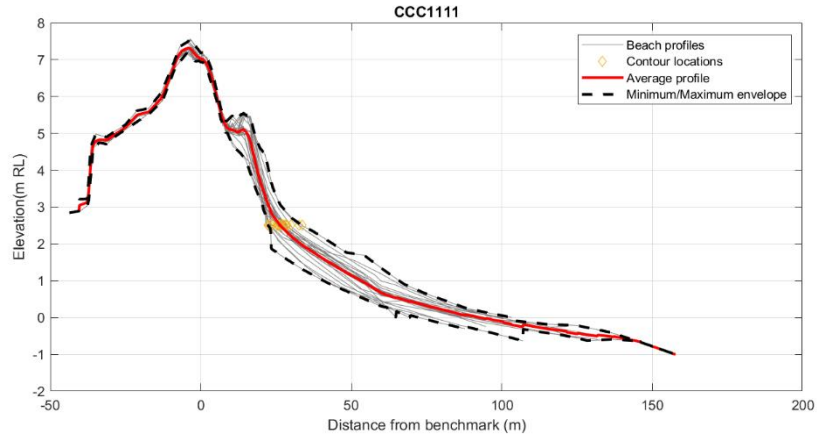
Upper 95% CI =0.1316 m/year
Lower 95% CI =-0.0559 m/year
Horizontal Excusion Distance (m)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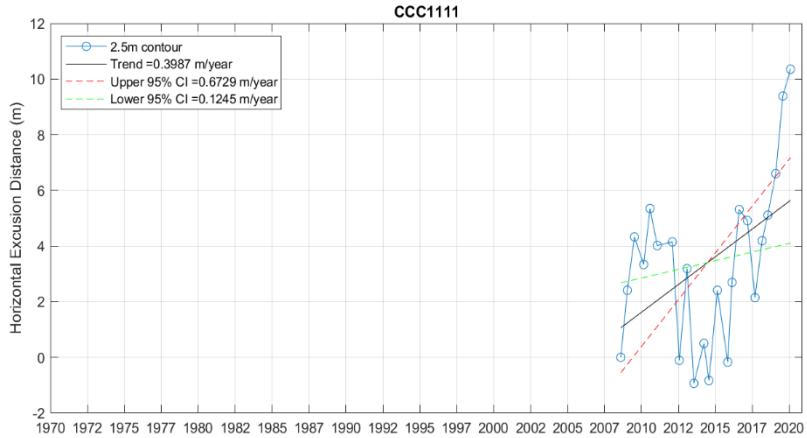
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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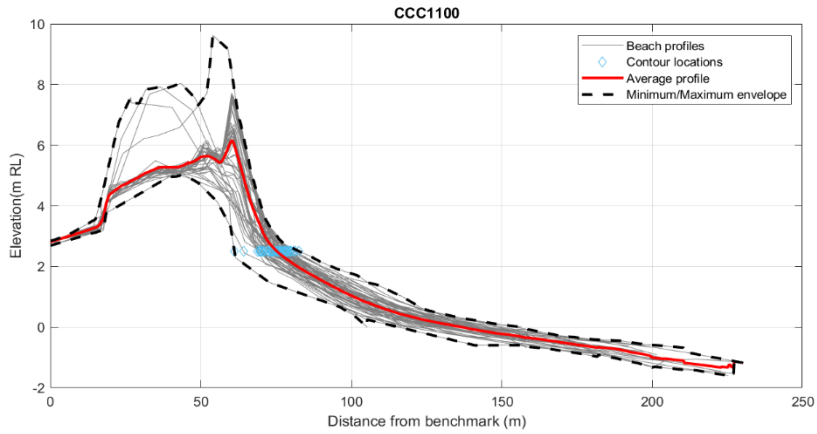
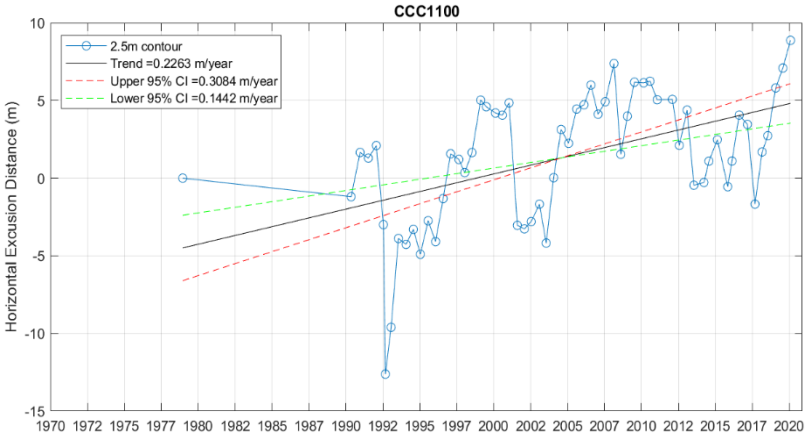
Lower 95% CI =0.1245 m/year
Horizontal Excusion Distance (m)


CCC1100
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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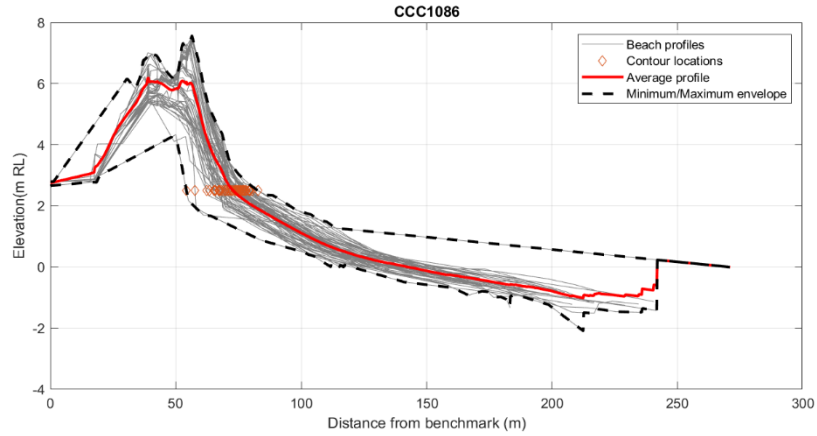
Upper 95% CI =0.3084 m/year
Lower 95% CI =0.1442 m/year
Horizontal Excusion Distance (m)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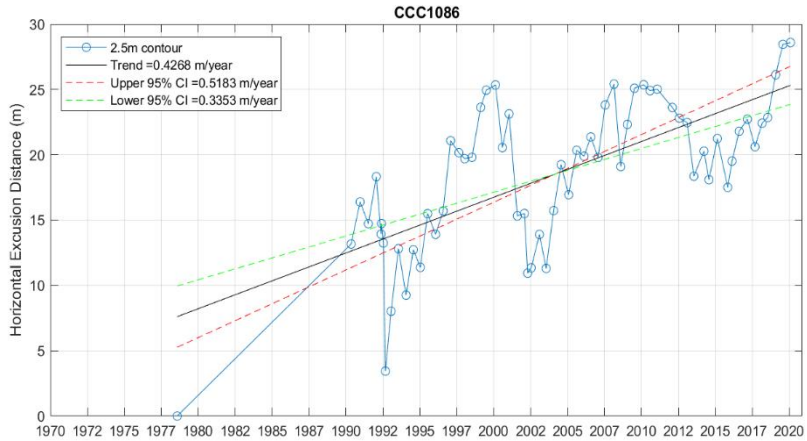
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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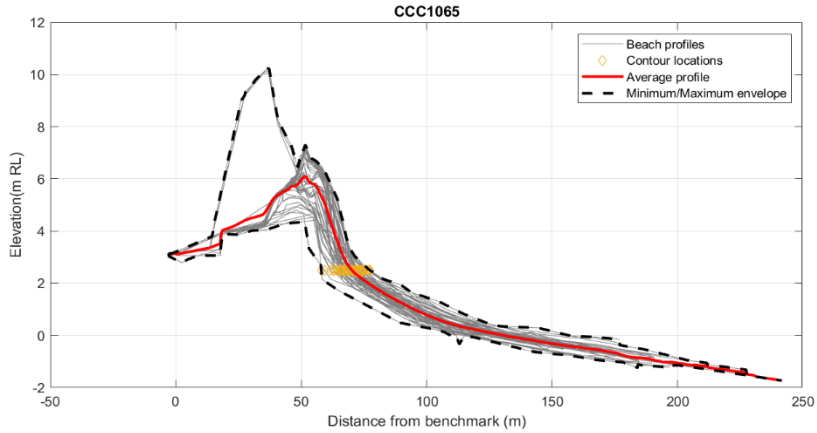
Upper 95% CI =0.5183 m/year
Lower 95% CI =0.3353 m/year
Horizontal Excusion Distance (m)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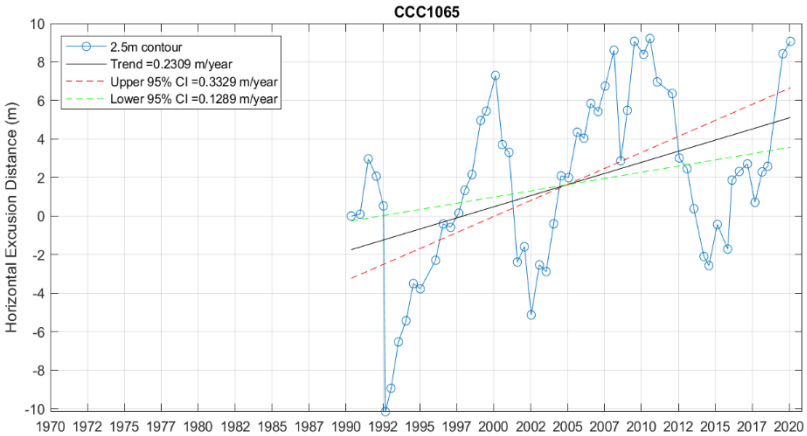
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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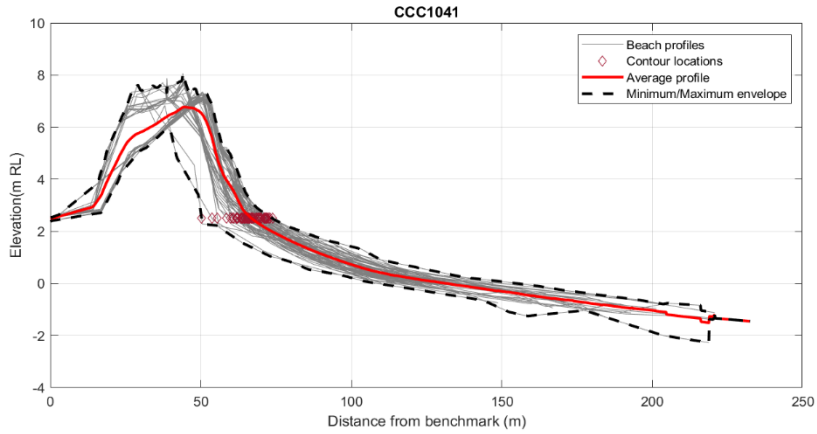
Lower 95% CI =0.1289 m/year
Horizontal Excusion Distance (m)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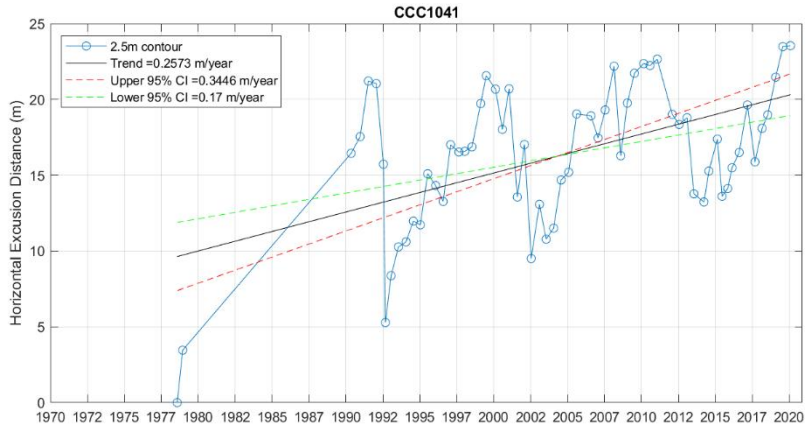
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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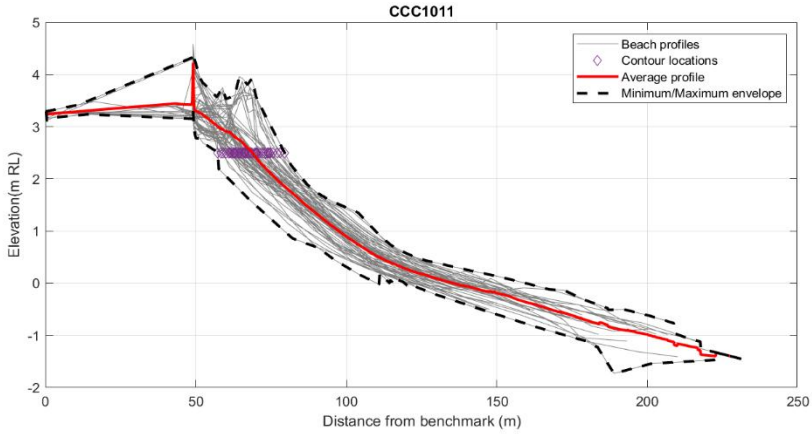
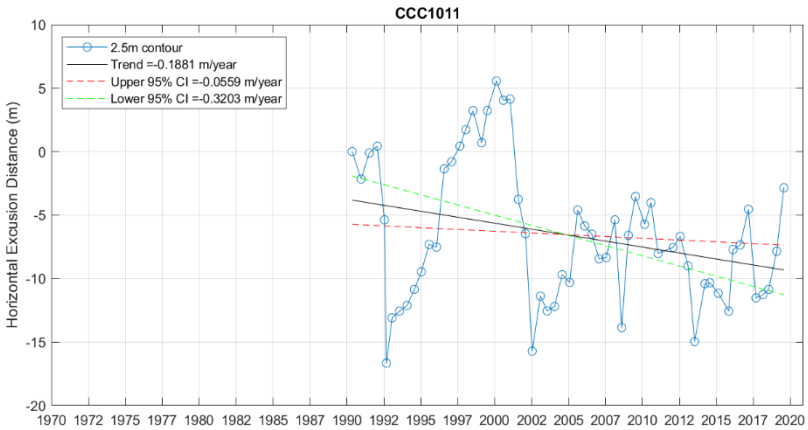
Lower 95% CI =0.17 m/year
Horizontal Excusion Distance (m)


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


CCC1011
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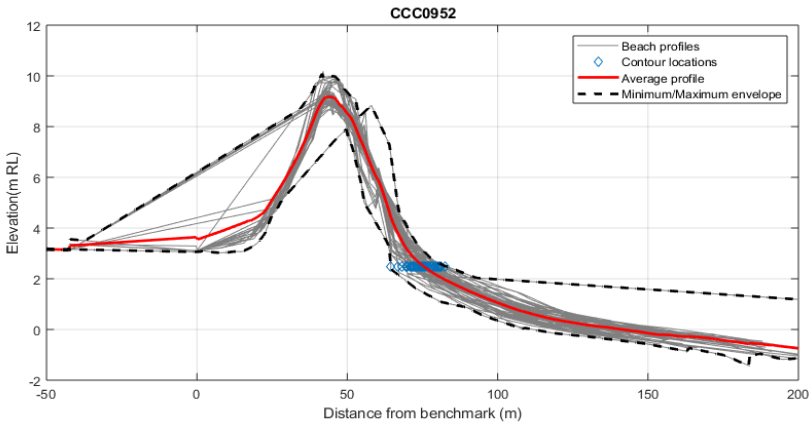
Lower 95% CI =-0.3203 m/year
Horizontal Excusion Distance (m)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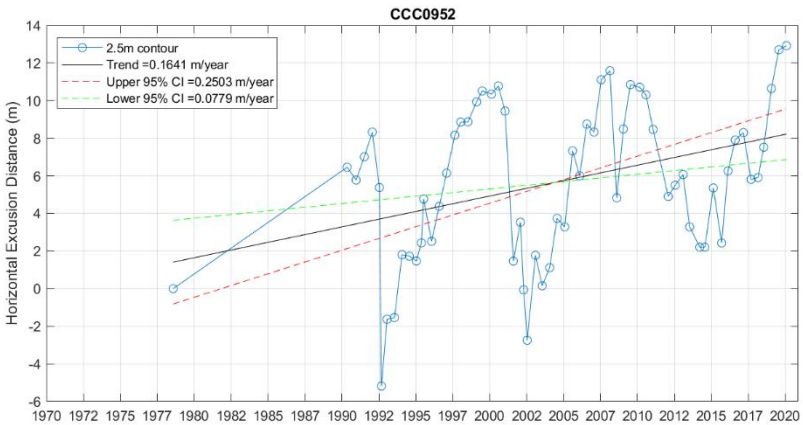
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


CCC0952
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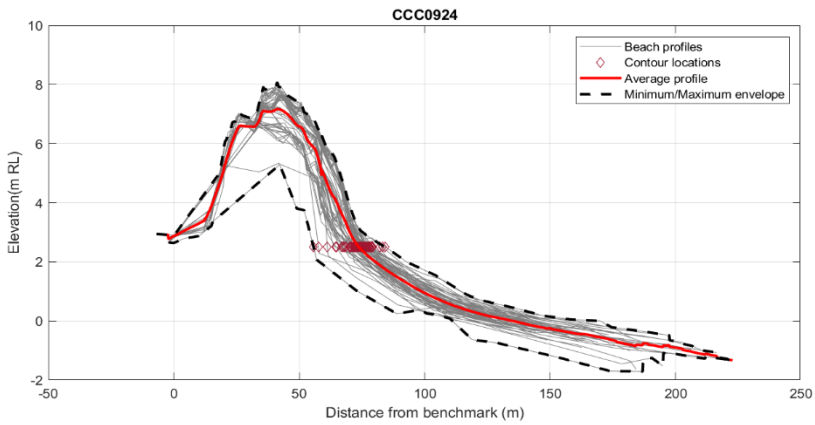
Upper 95% CI =0.2503 m/year
Lower 95% CI =0.0779 m/year
Horizontal Excusion Distance (m)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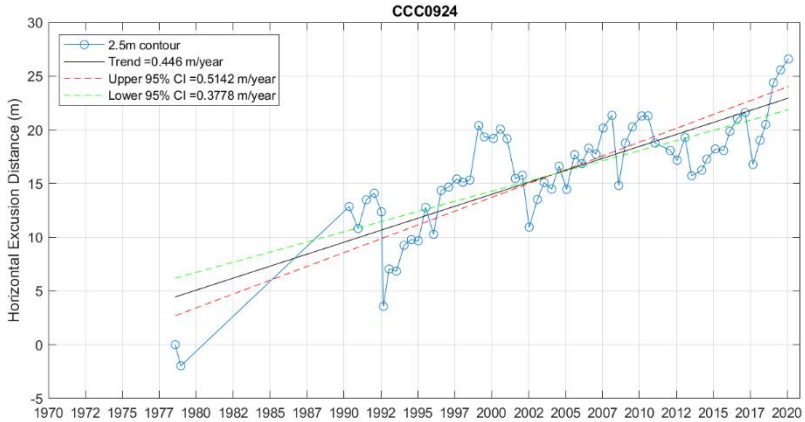
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


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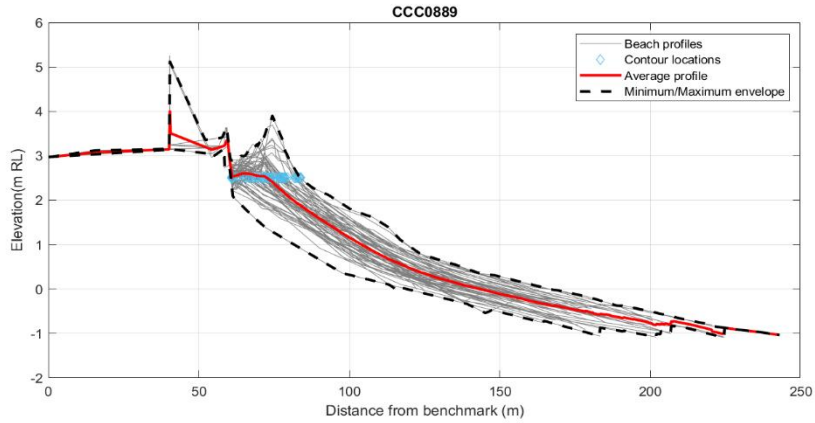
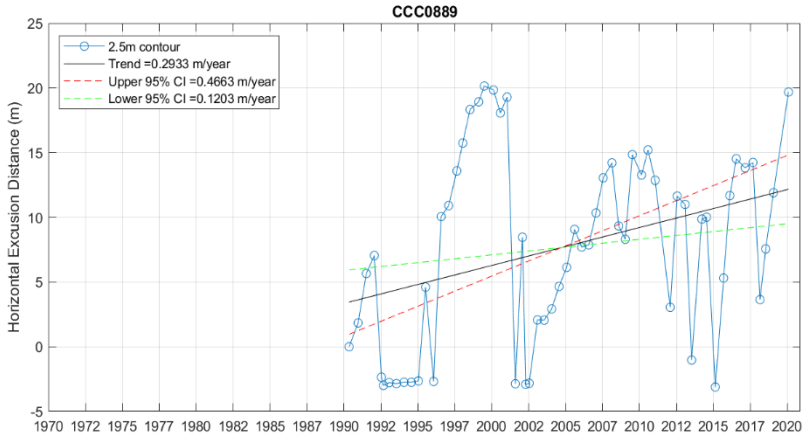
Upper 95% CI =0.5142 m/year
Lower 95% CI =0.3778 m/year
Horizontal Excusion Distance (m)


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


CCC0889
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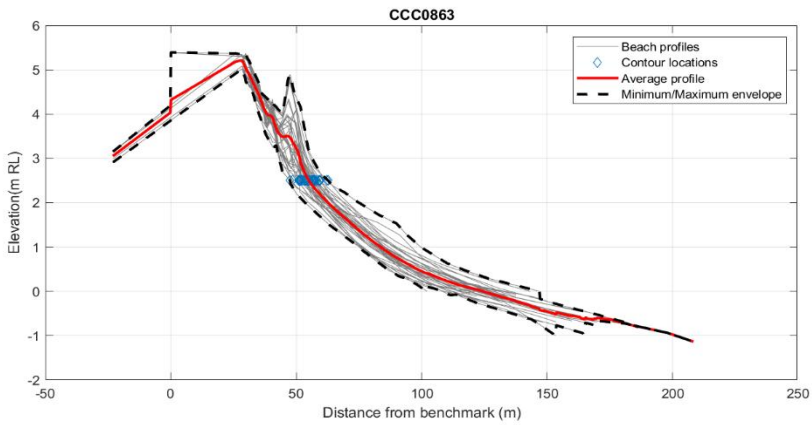
Upper 95% CI =0.4663 m/year
Lower 95% CI =0.1203 m/year
Horizontal Excusion Distance (m)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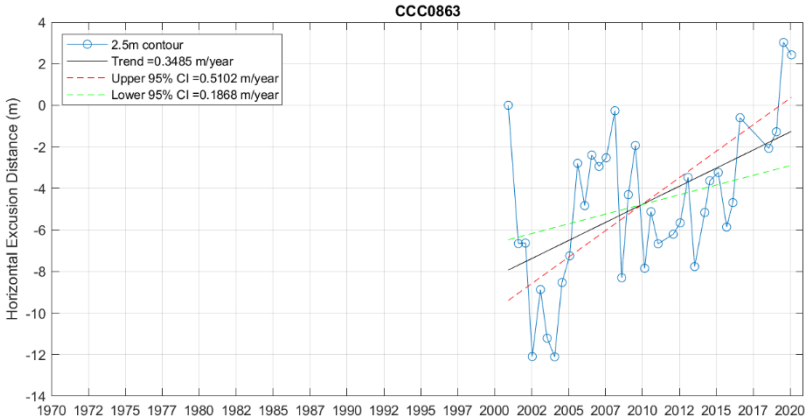
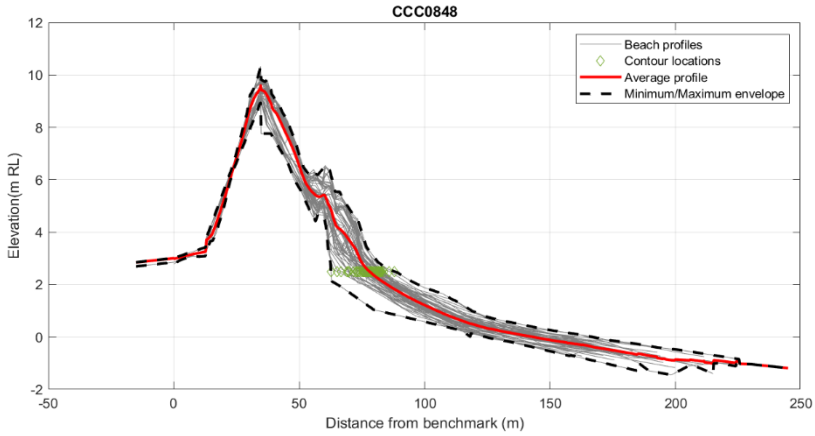
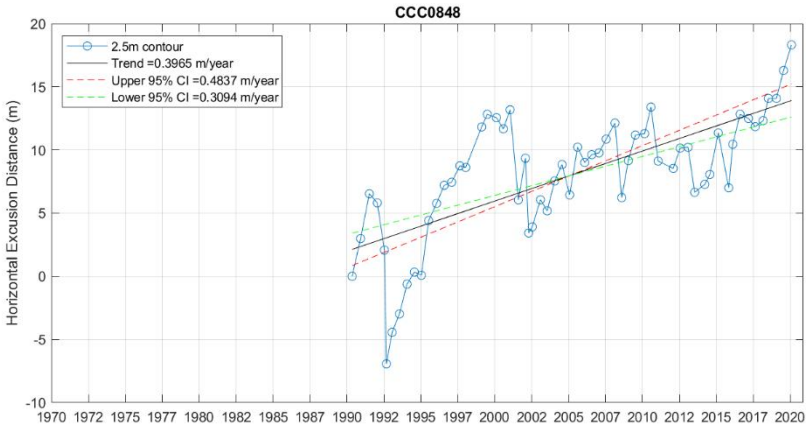
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


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


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


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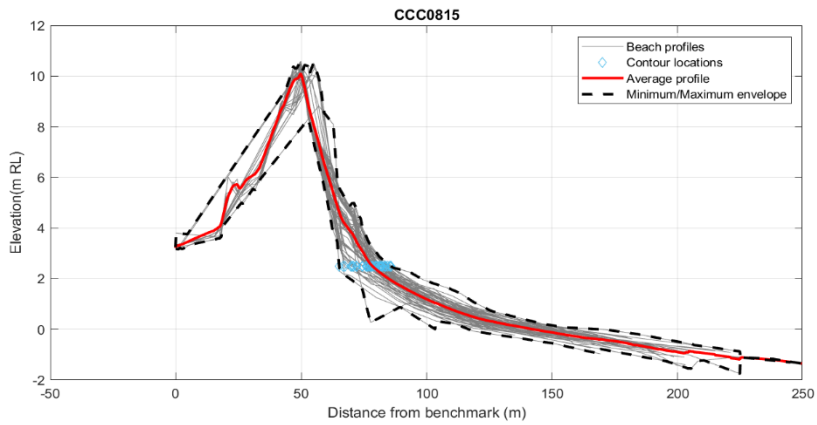
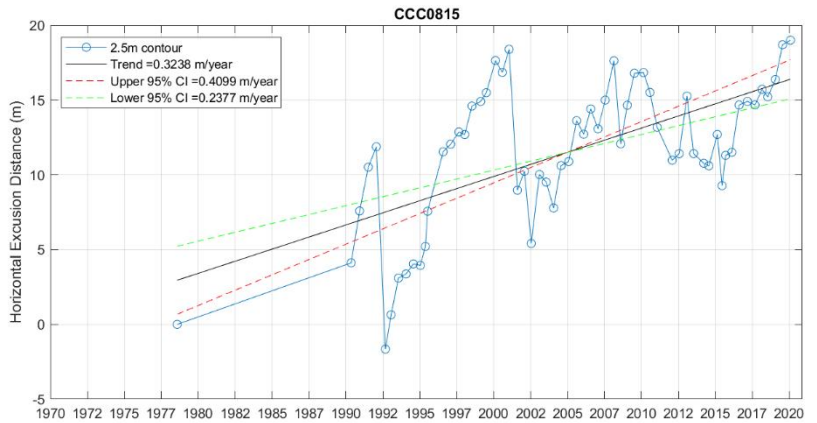
Upper 95% CI =0.4837 m/year
Lower 95% CI =0.3094 m/year
Horizontal Excusion Distance (m)


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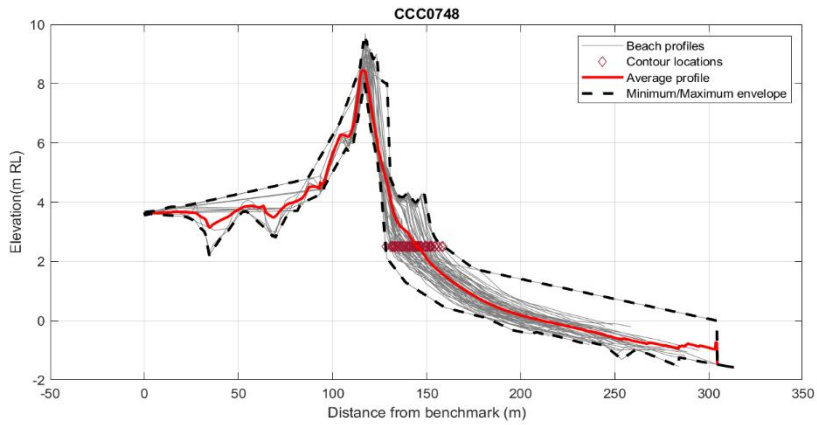
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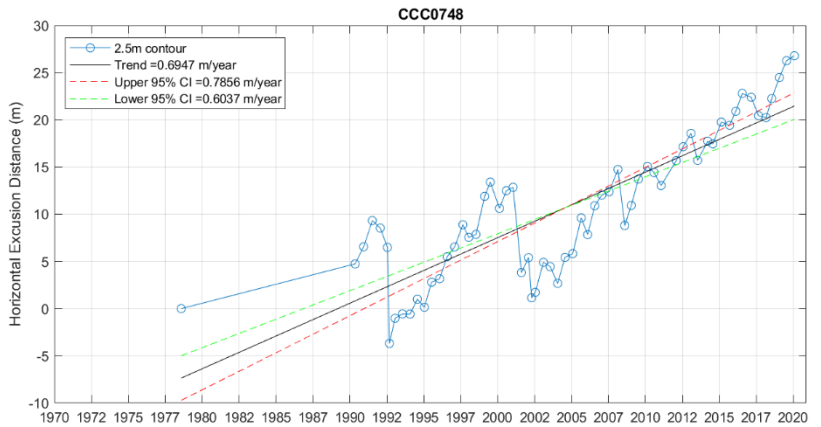
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)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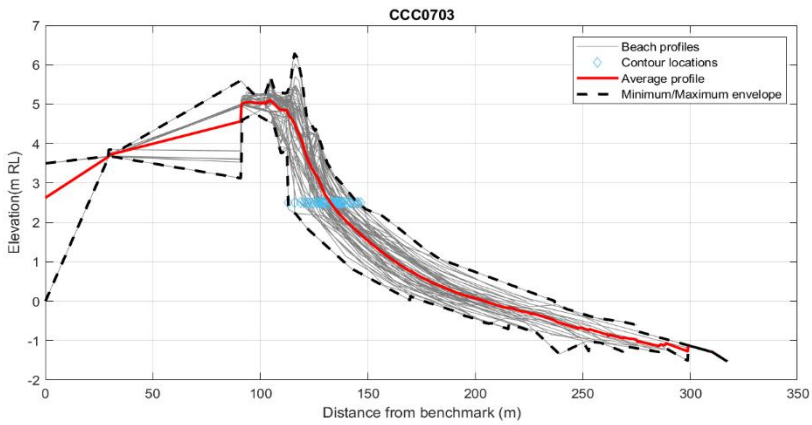
Horizontal Excusion Distance (m)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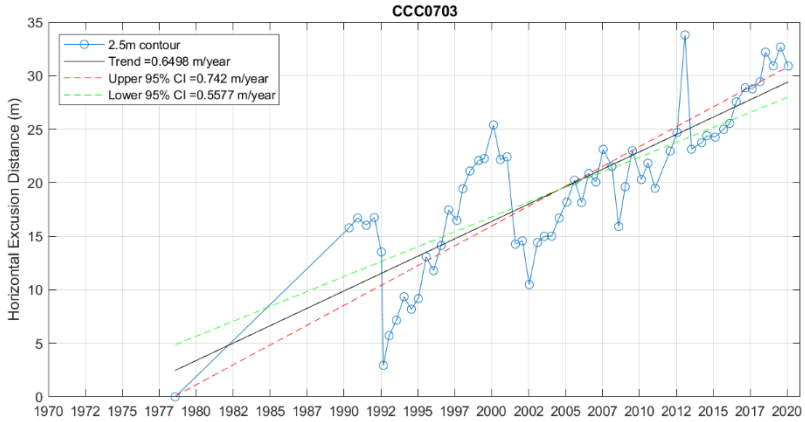
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


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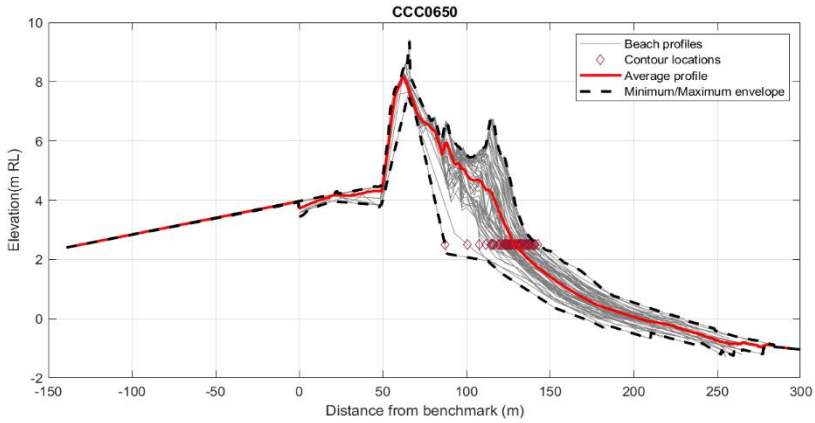
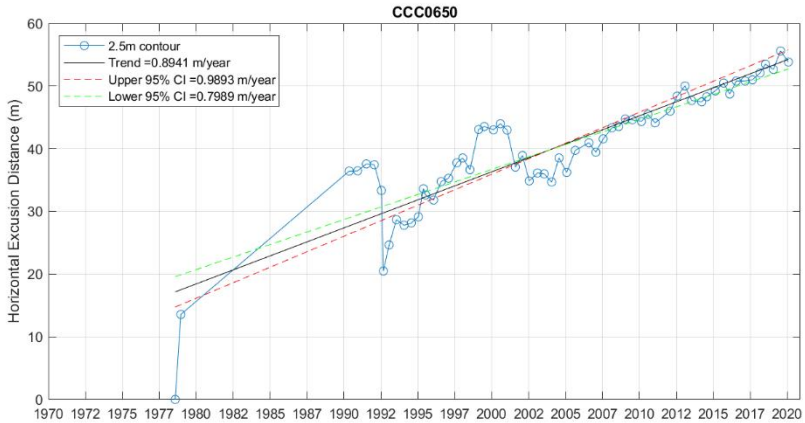
Upper 95% CI =0.742 m/year
Lower 95% CI =0.5577 m/year
Horizontal Excusion Distance (m)


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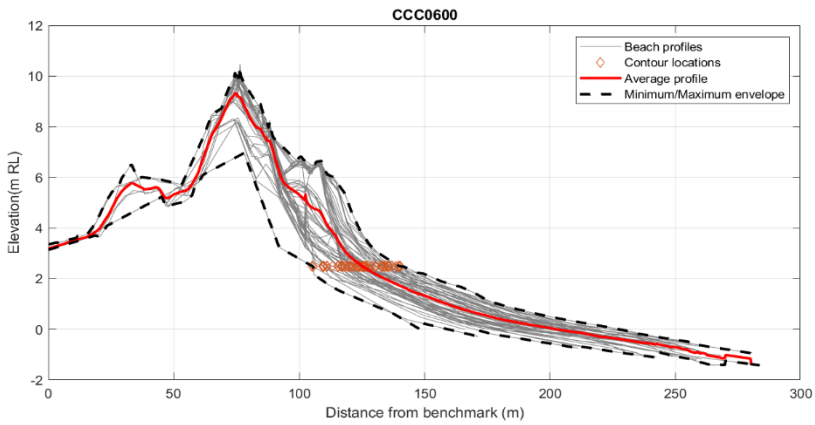
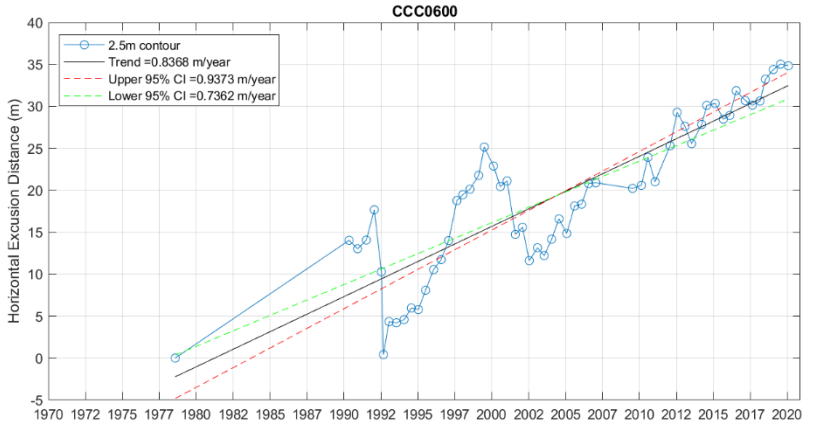
Horizontal Excusion Distance (m)


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


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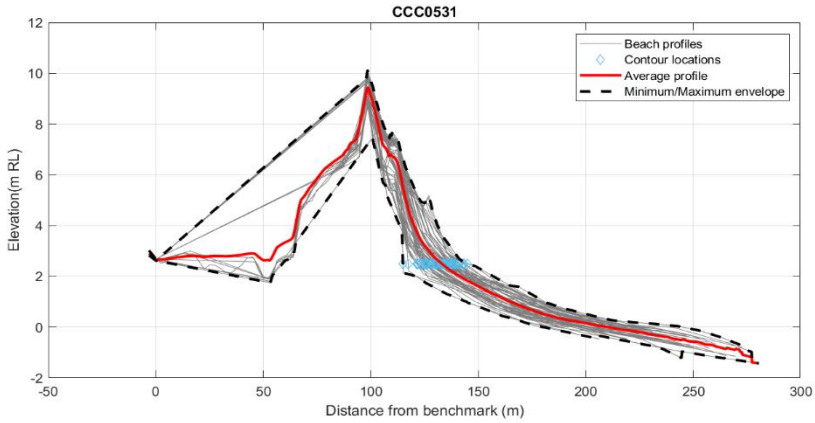
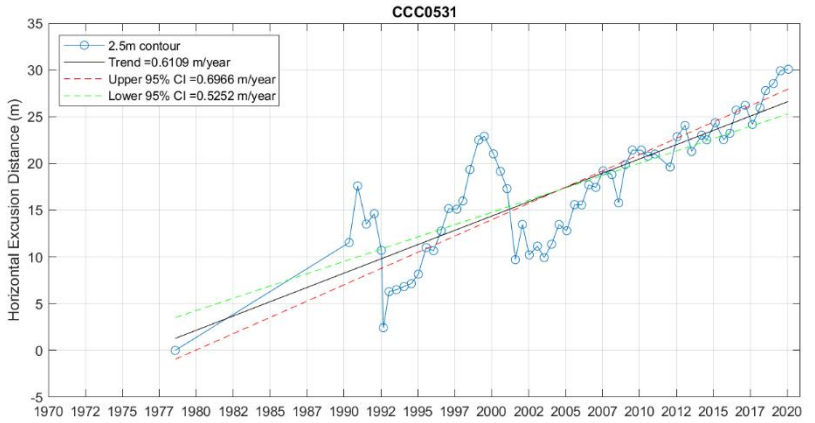
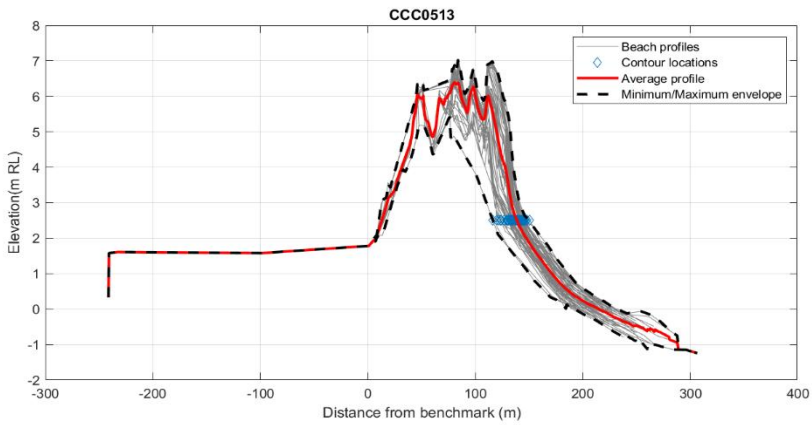
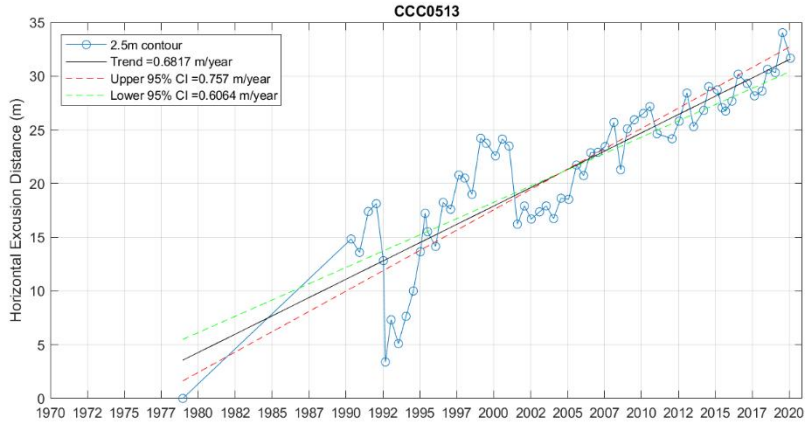
Upper 95% CI =0.9373 m/year
Lower 95% CI =0.7362 m/year
Horizontal Excusion Distance (m)


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


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


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


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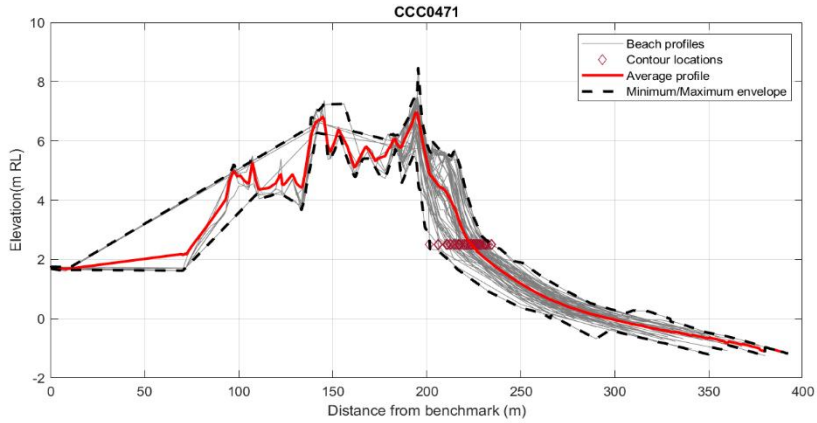
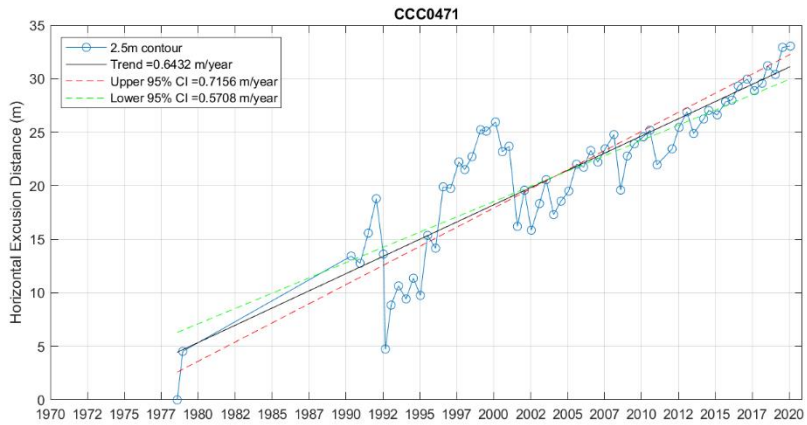
Upper 95% CI =0.757 m/year
Lower 95% CI =0.6064 m/year
Horizontal Excusion Distance (m)


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


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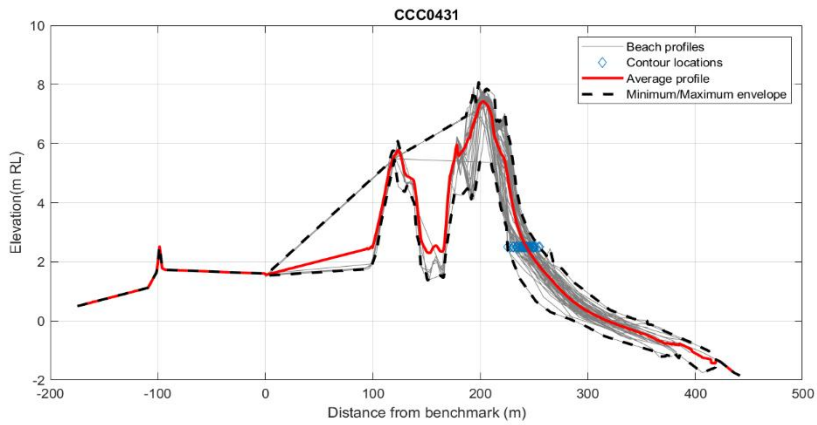
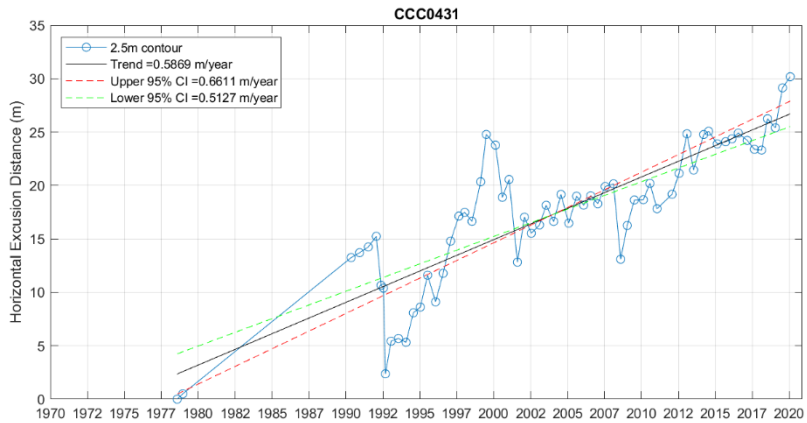
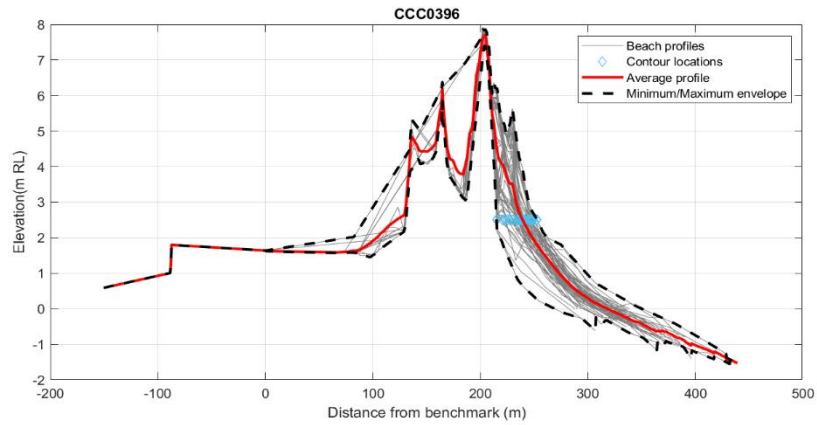
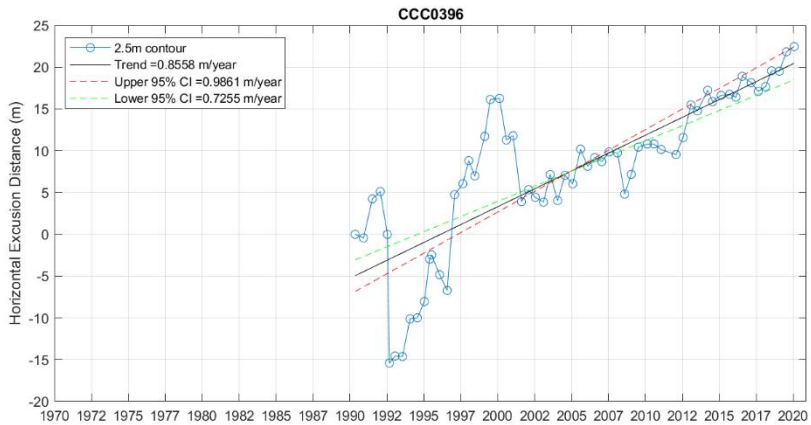
Upper 95% CI =0.7156 m/year
Lower 95% CI =0.5708 m/year
Horizontal Excusion Distance (m)


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


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


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


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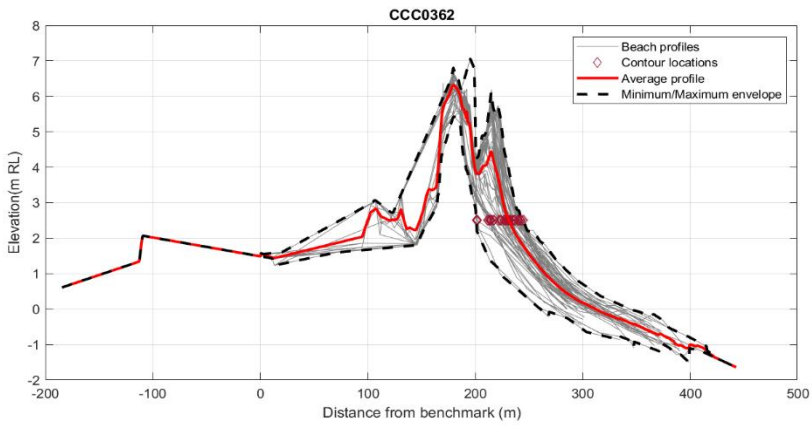
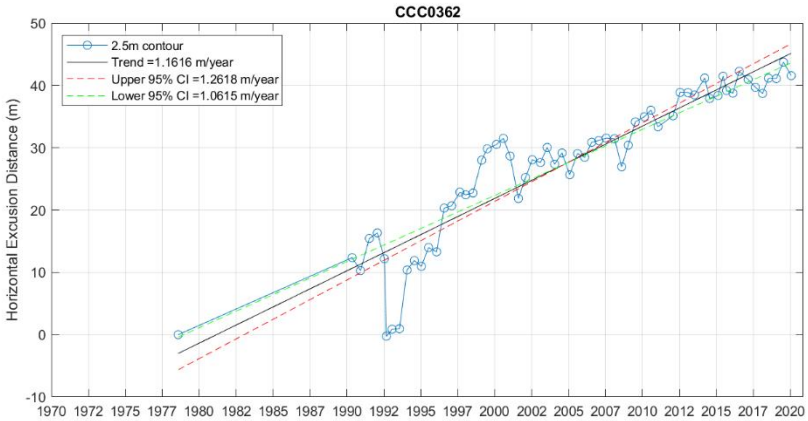
Upper 95% CI =0.9861 m/year
Lower 95% CI =0.7255 m/year
Horizontal Excusion Distance (m)


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


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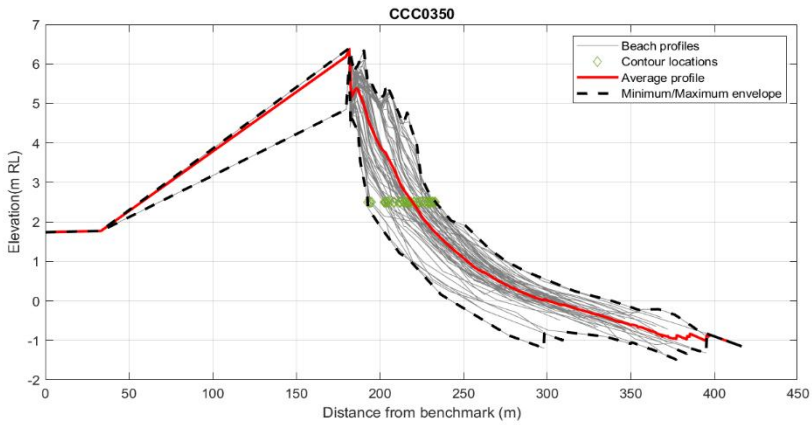
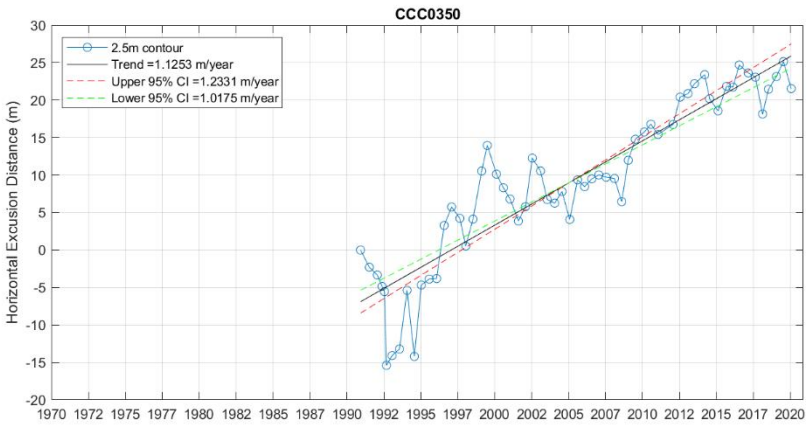
Upper 95% CI =1.2618 m/year
Lower 95% CI =1.0615 m/year
Horizontal Excusion Distance (m)


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


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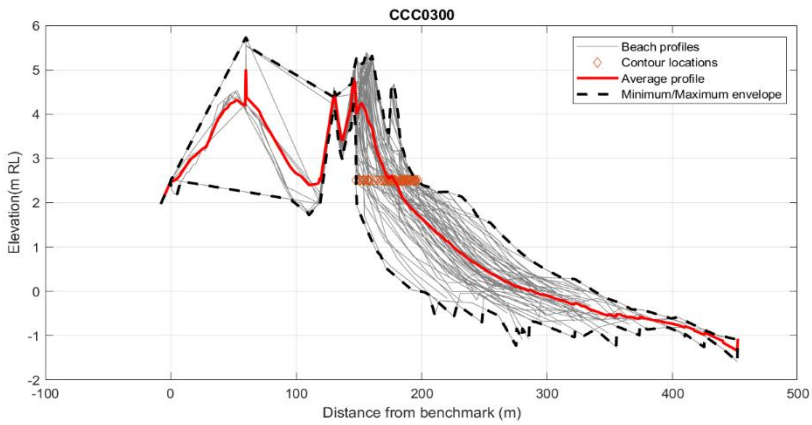
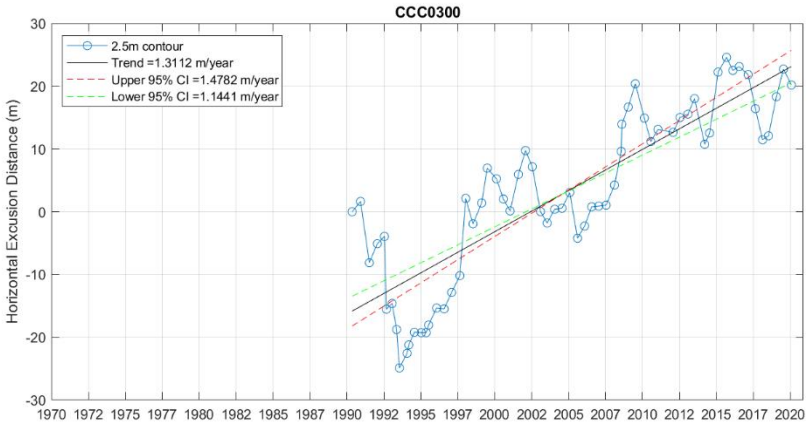
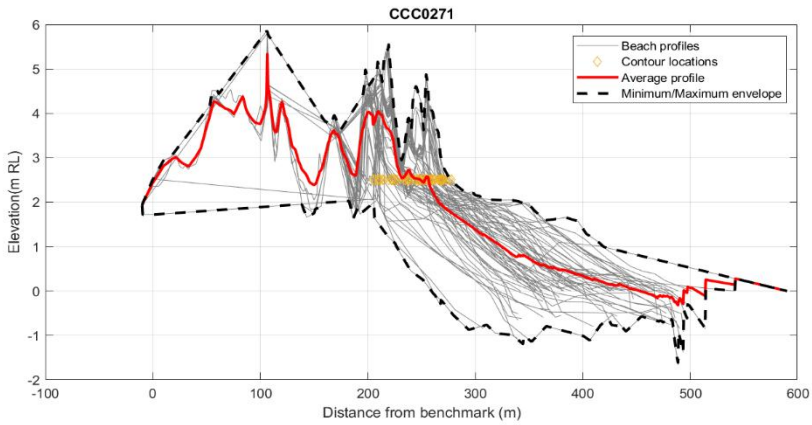
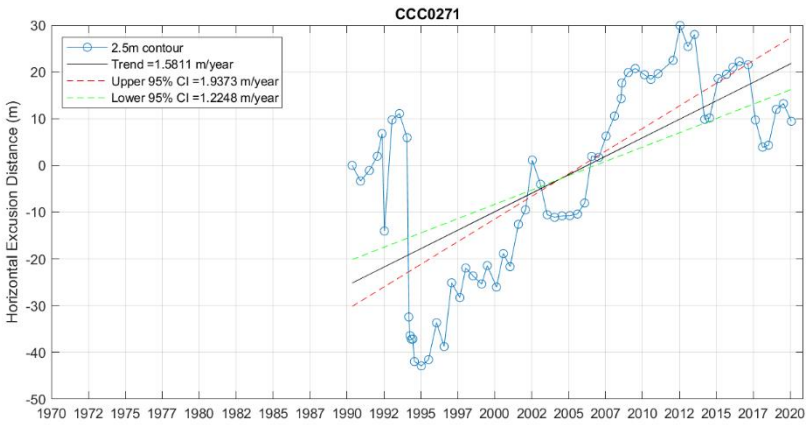
Trend =1.1253 m/year
Upper 95% CI =1.2331 m/year
Lower 95% CI =1.0175 m/year
Horizontal Excusion Distance (m)


CCC0300
Beach profiles
Contour locations
Average profile
Minimum/Maximum envelope
Elevation(m RL)
Distance from benchmark (m)


CCC0300
2.5m contour
Trend =1.3112 m/year
Upper 95% CI =1.4782 m/year
Lower 95% CI =1.1441 m/year
Horizontal Excusion Distance (m)


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


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

Appendix B: Wave transformation using numerical SWAN model

B1 Estuary and harbour sites

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

B2 Model description

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

B3 Model domains

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

Appendix B Table 1: Model Domains

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

B4 Wave transformation modelling

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

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

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

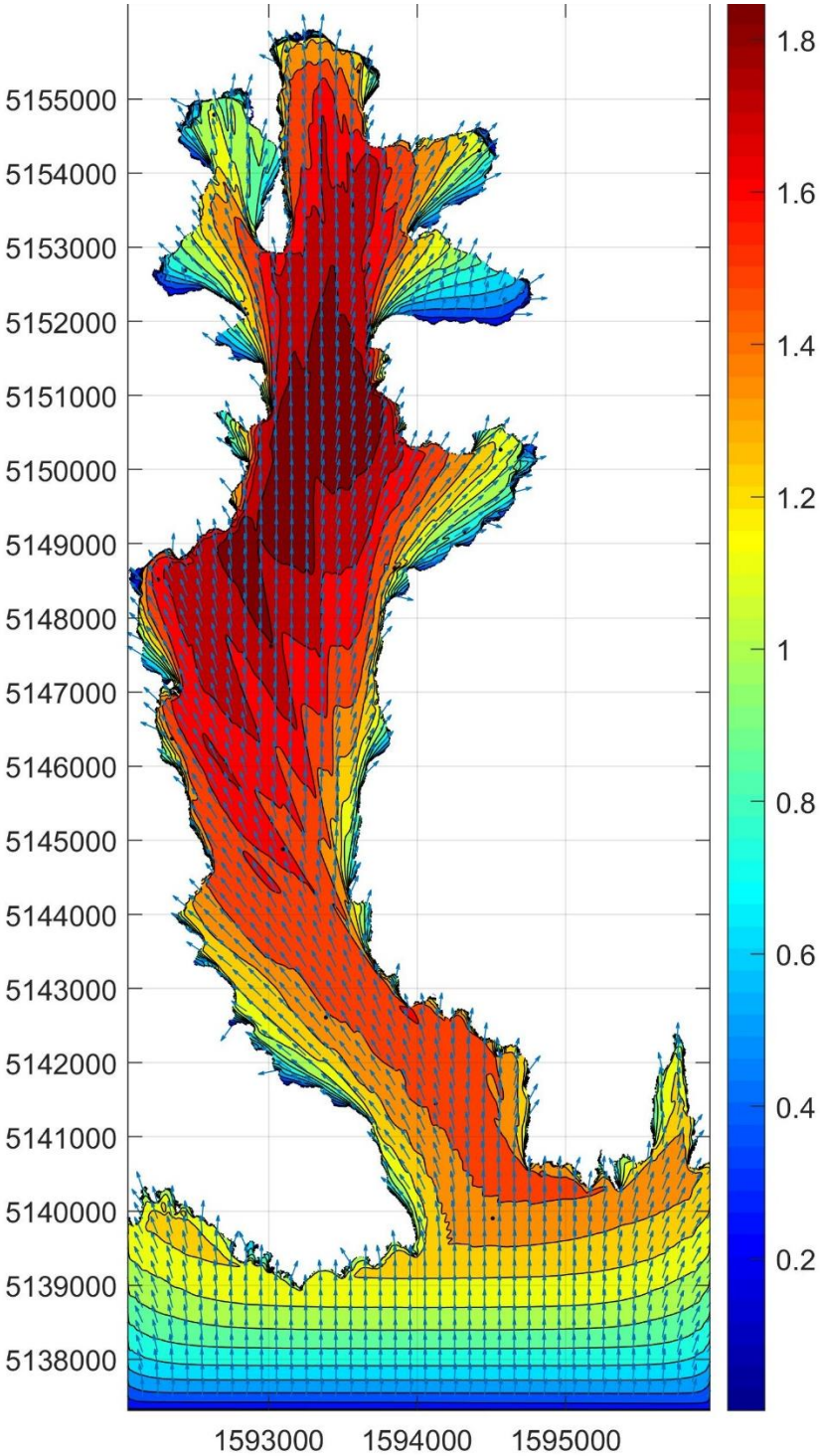


Figure Appendix B.1: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South - Wind generated waves.

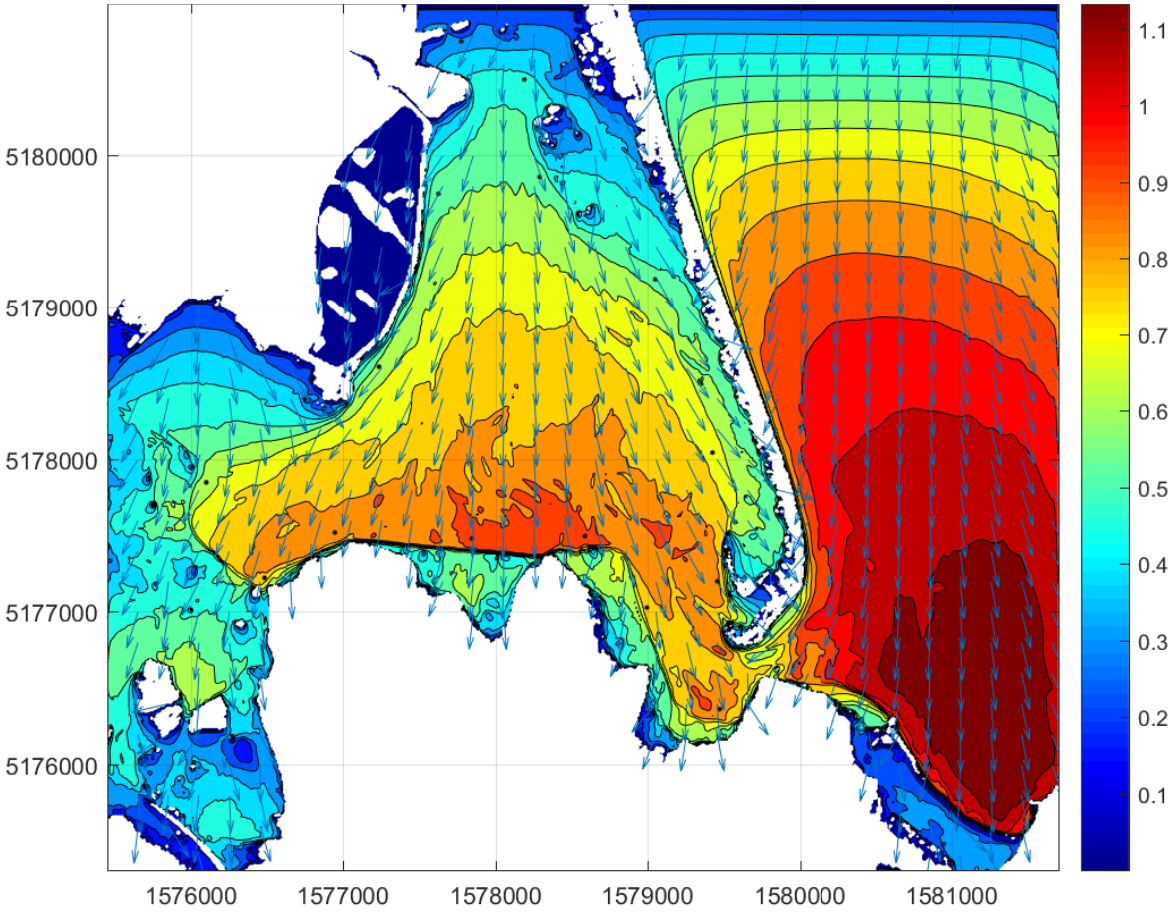


Figure Appendix B.2: SWAN model results for the Avon Heathcote domain – Significant wave height and direction during a 100-year ARI storm from the North – Wind generated waves.

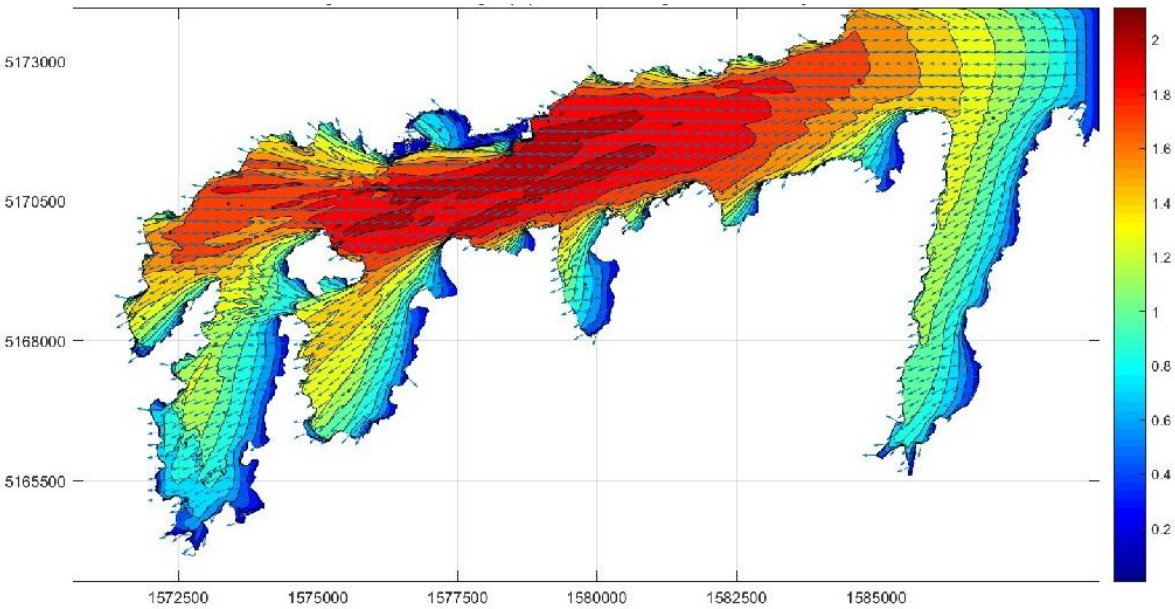


Figure Appendix B.3: SWAN model results for the Lyttleton domain – Significant wave height and direction during a 100-year ARI storm from the East – Wind generated waves.

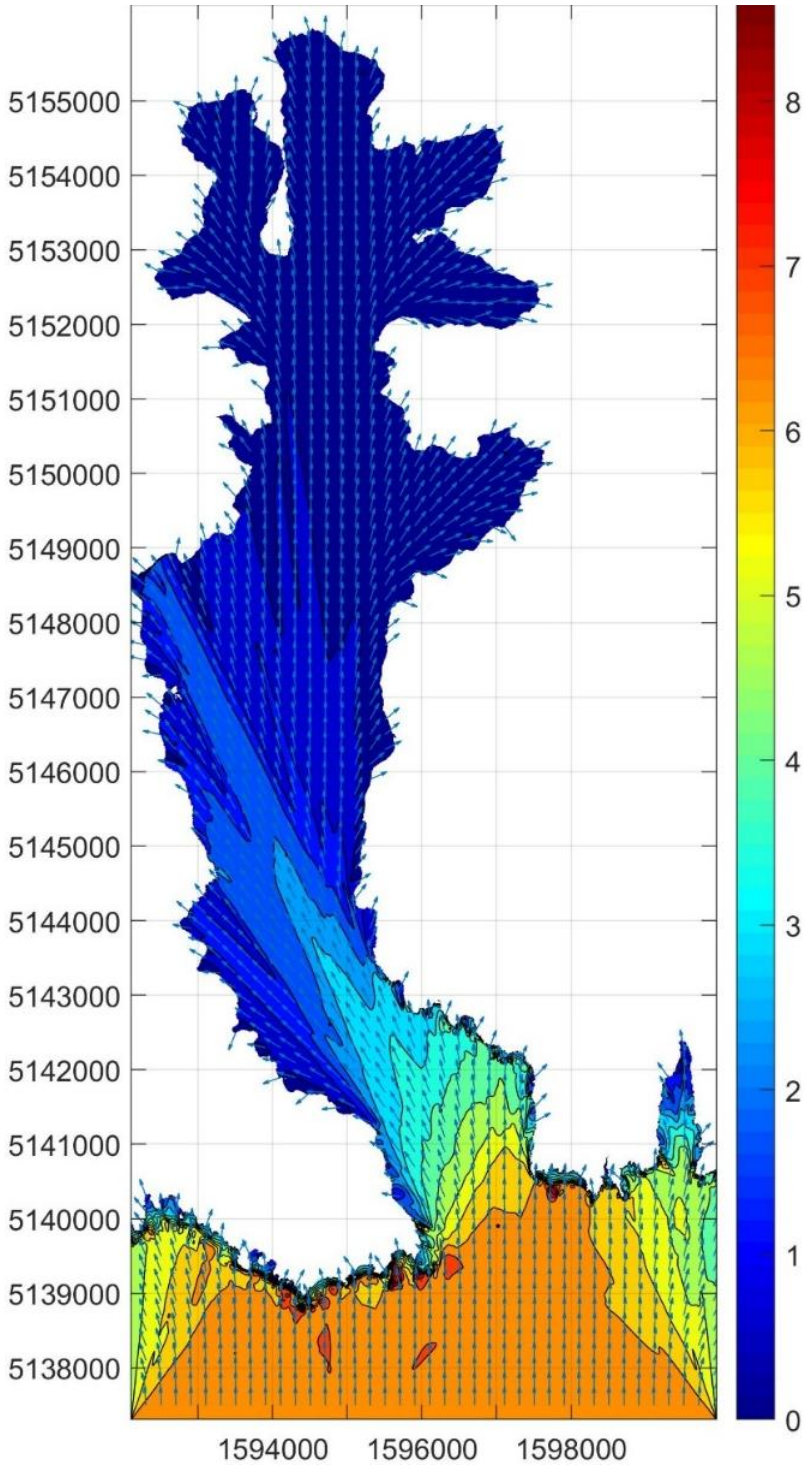


Figure Appendix B.4: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South – Swell.

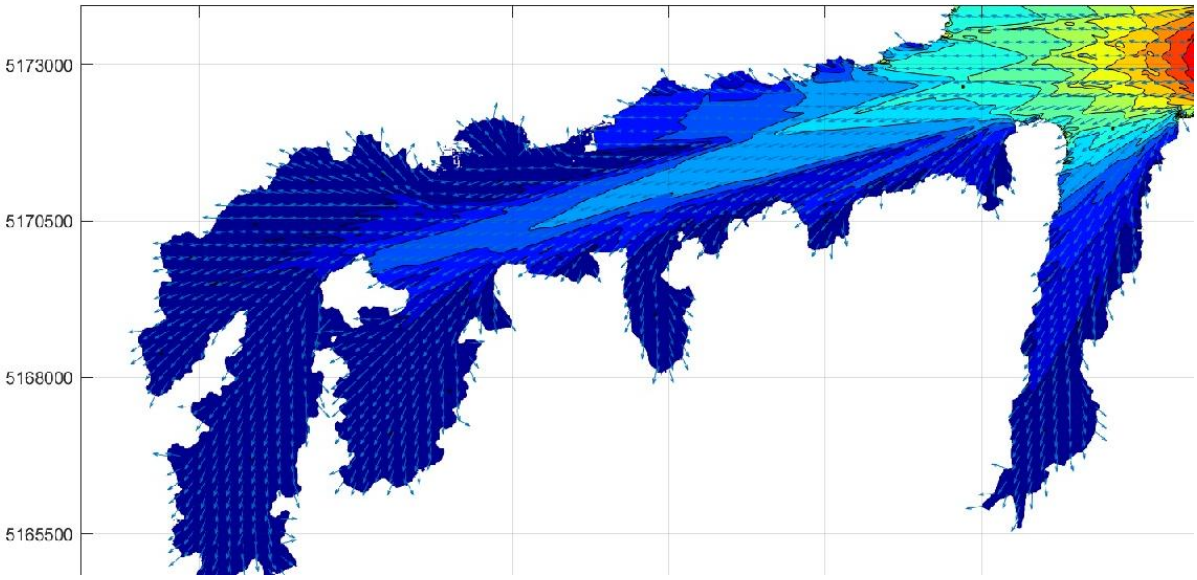


Figure Appendix B.5: SWAN model results for the Akaroa domain – Significant wave height and direction during a 100-year ARI storm from the South- Swell.

Appendix C: Sensitivity assessment of bathtub approach

Tonkin+Taylor

Memo

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

1 Introduction

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

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

2 Modelling approach

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

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

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

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

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

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

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

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

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

3 Model results

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

3.1 T+T (2017) TUFLOW model

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

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

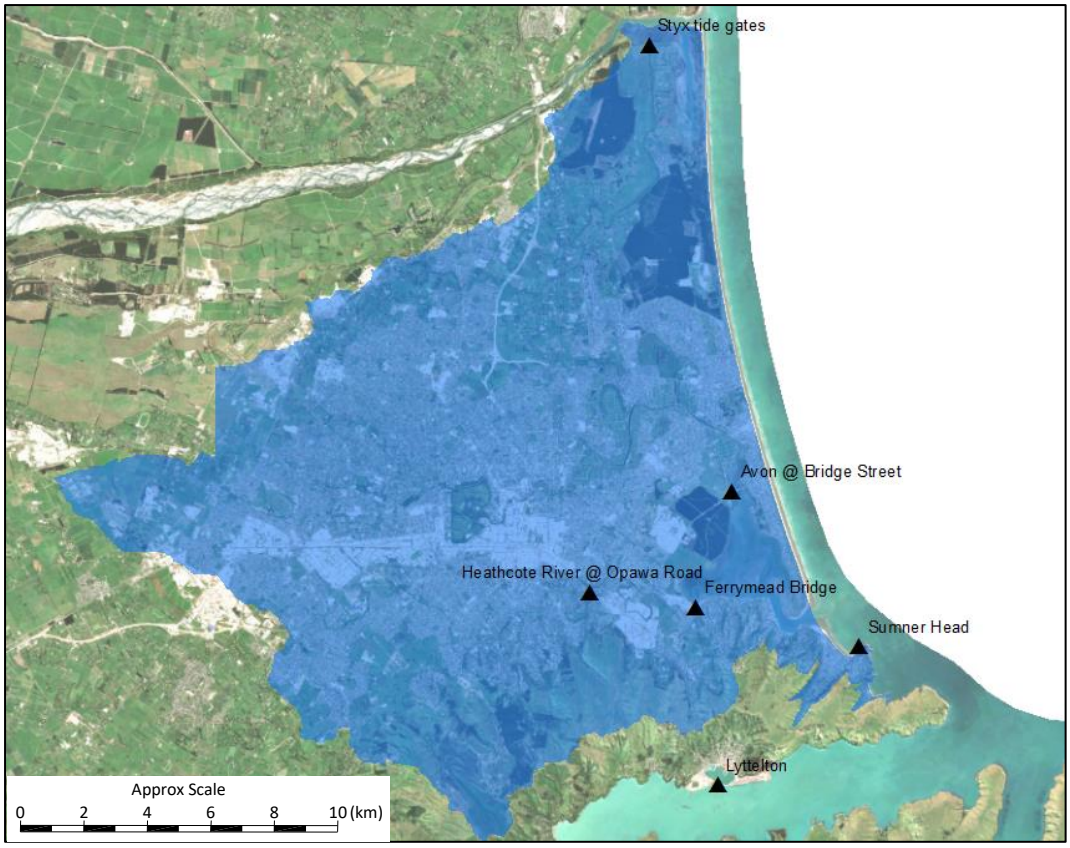


Figure 3.1: Extent of T+T (2017) TUFLOW model.

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

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

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

3.2 CCC city-wide flood model

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

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

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

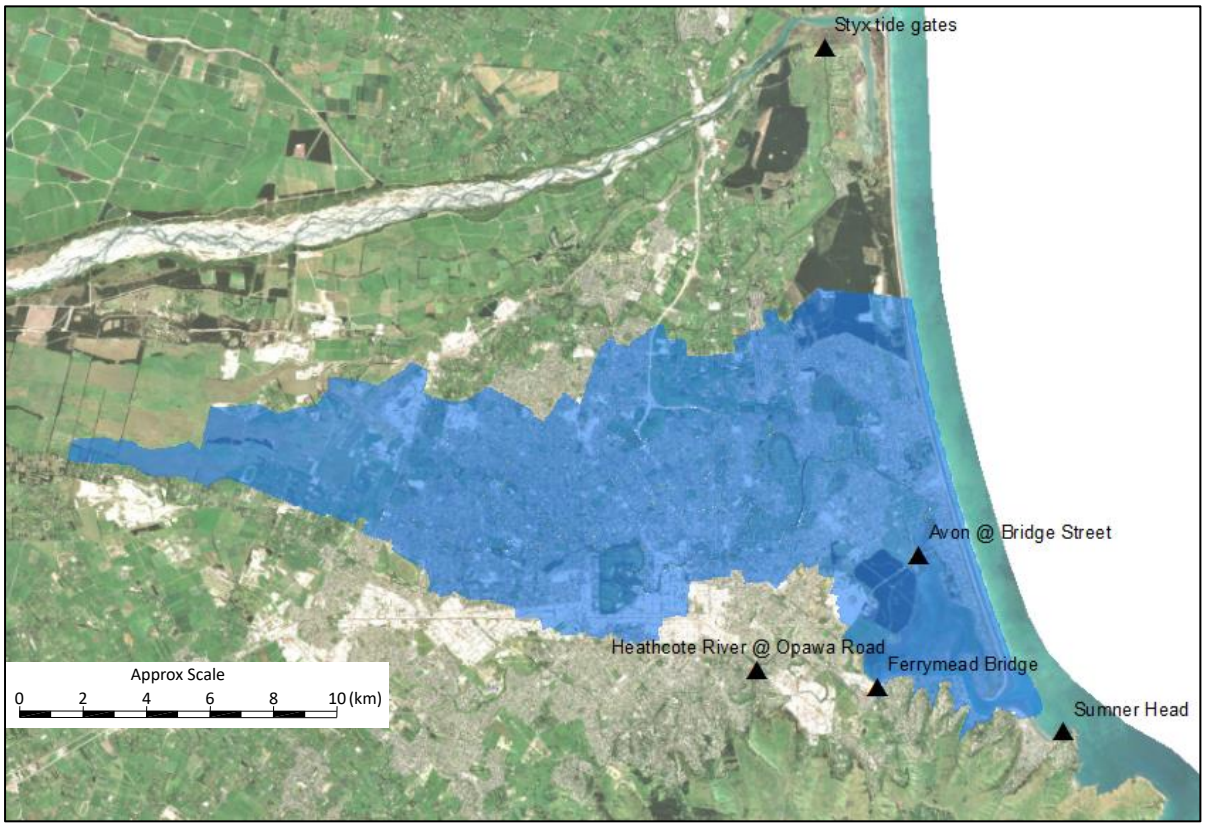


Figure 3.2: Extent of Avon catchment within the city-wide flood model.

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

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

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

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

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

3.3 Bathtub model

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

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

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

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

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

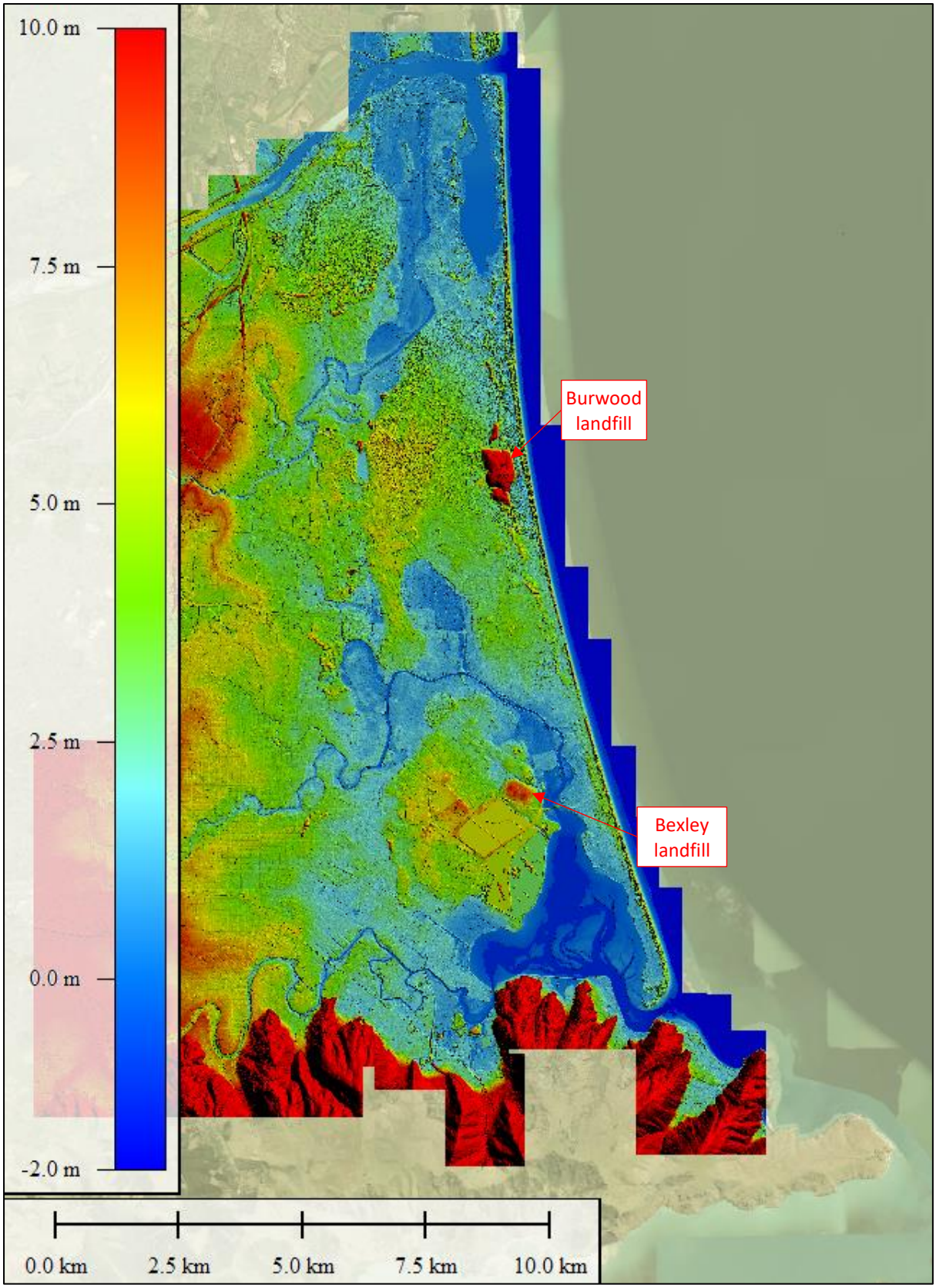


Figure 3.3: Example of 2018 DEM used for bathtub model.

4 Model comparisons

4.1 Comparison methodology

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

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

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

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

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

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

[1]3.2 m bathtub scenario adopted for Heathcote catchment

4.2 DEM comparison

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

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

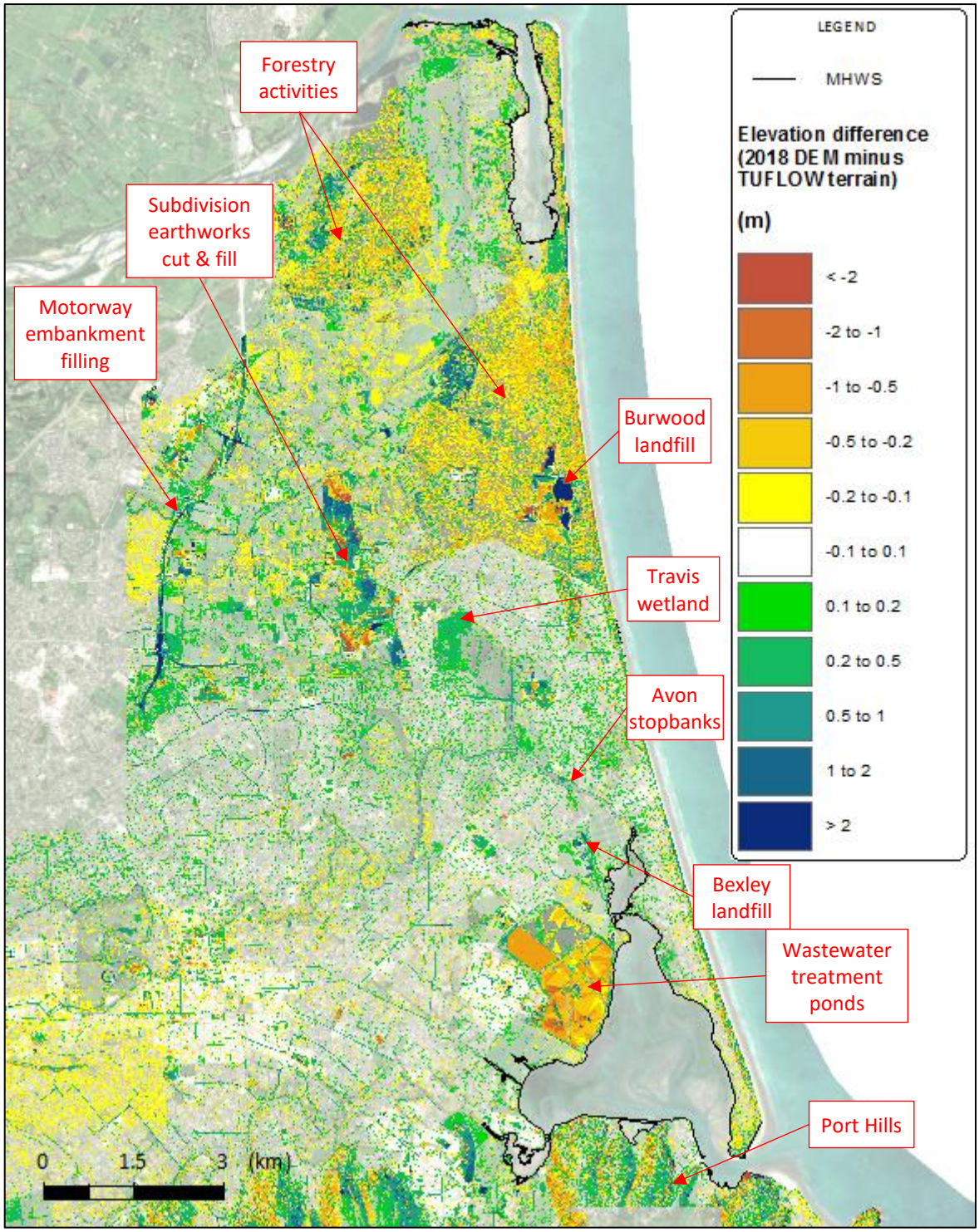


Figure 4.1: Elevation difference between the 2018 DEM used for the bathtub modelling and the terrain grid used within the TUFLOW model.

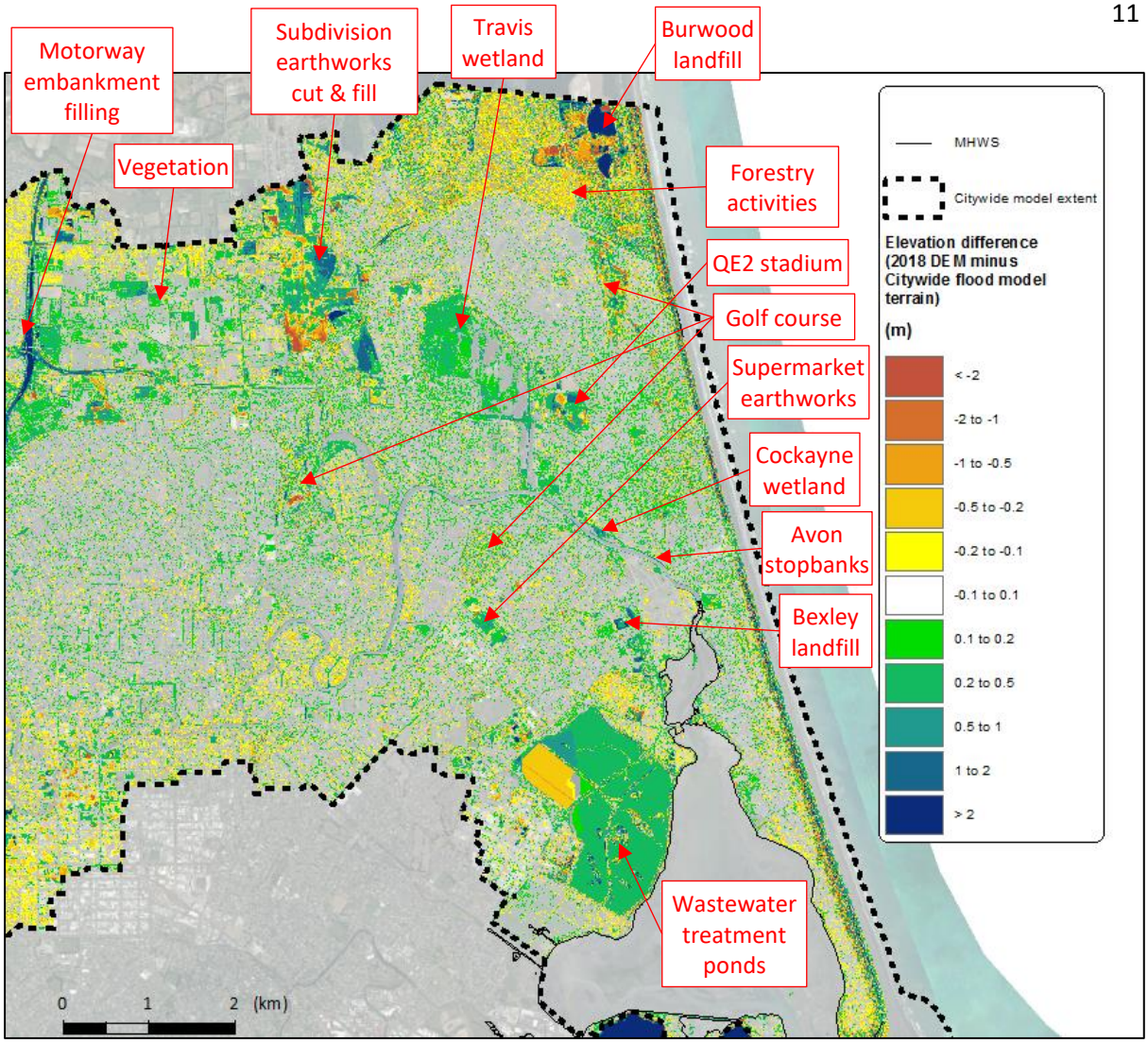


Figure 4.2: Elevation difference between the 2018 DEM used for the bathtub modelling and the terrain grid used within the CCC city-wide flood model.

5 Results

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

5.1 Comparison with T+T (2017) TUFLOW model

5.1.1 Low sea level rise scenario

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

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

the water level at Bridge St and subsequently the level used for the bathtub (i.e. the bathtub overestimates inundation depth by up to 0.5 m in some areas upstream of Wainoni Rd).

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

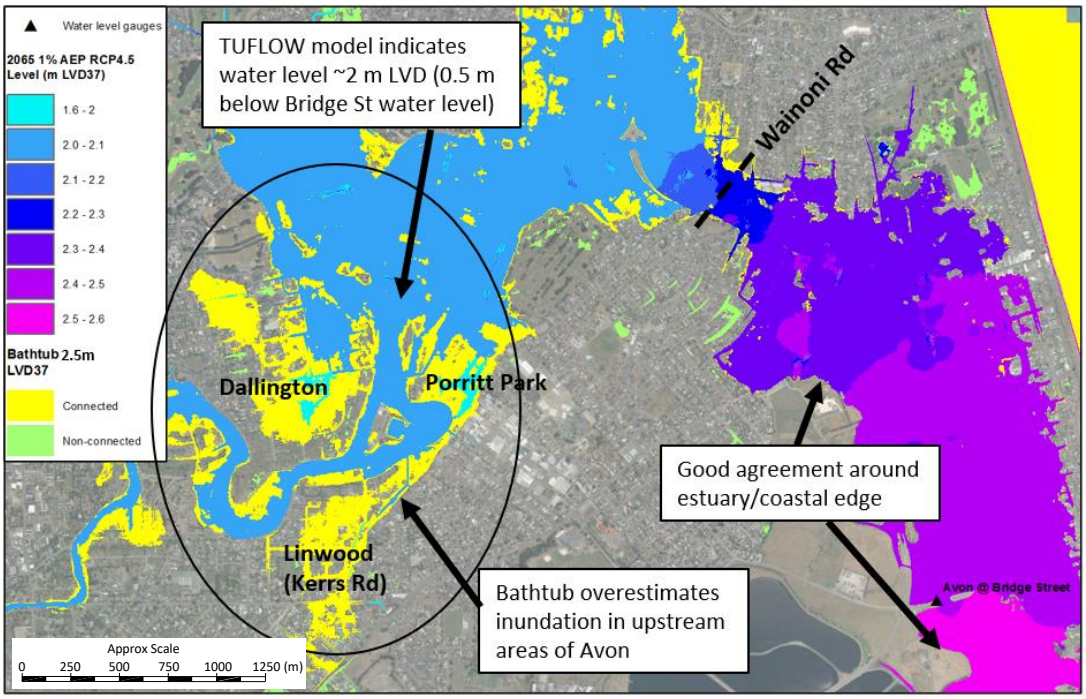


Figure 5.1: Comparison of bathtub and T+T TUFLOW results for a low SLR scenario (2065 1% AEP RCP4.5). Key areas of difference along Avon River (yellow and green shading are where bathtub flood extent is larger).

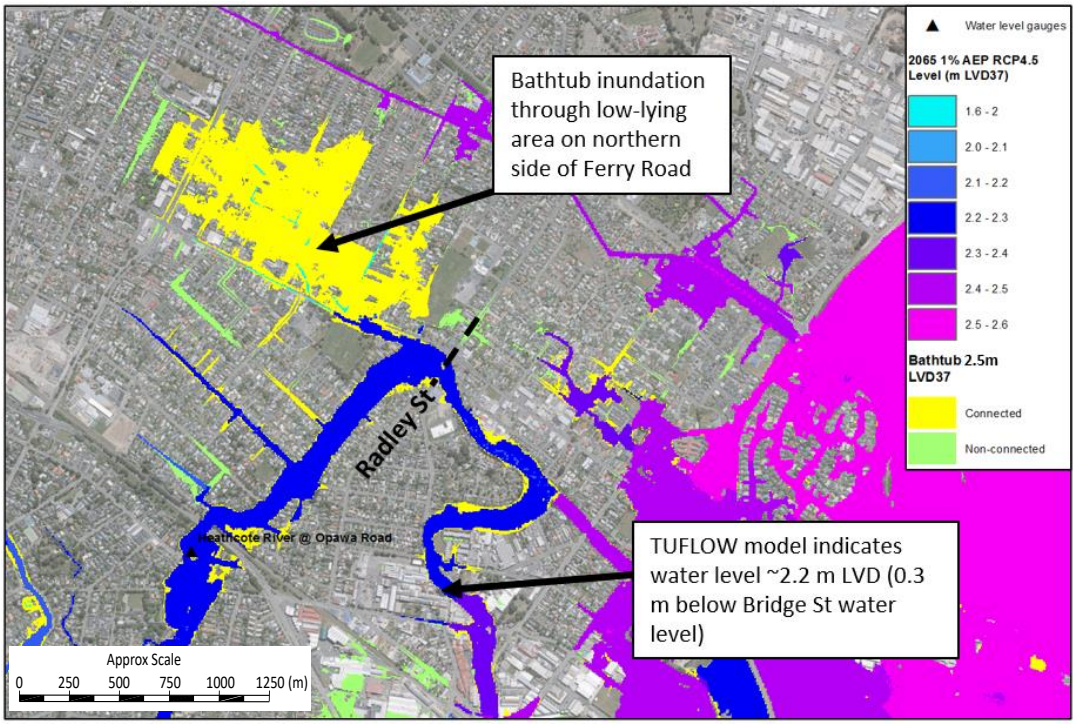


Figure 5.2: Comparison of bathtub and T+T TUFLOW model results for a low SLR scenario (2065 1% AEP RCP4.5). Key areas of difference along Heathcote River (yellow and green shading are where bathtub flood extent is larger).

5.1.2 High sea level rise scenario

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

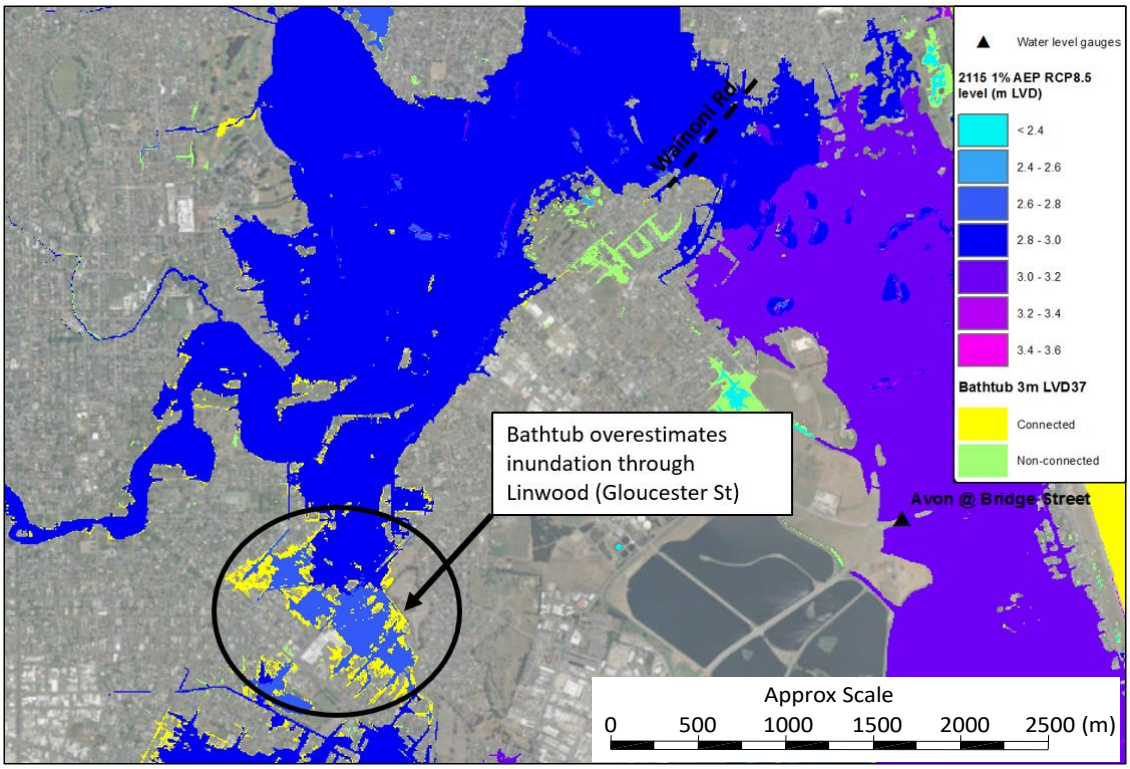


Figure 5.3*: Comparison of bathtub and T+T TUFLOW model results for a high SLR scenario (2115 1% AEP RCP8.5). Key areas of difference along the Avon River (yellow and green shading are where bathtub flood extent is larger).*

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

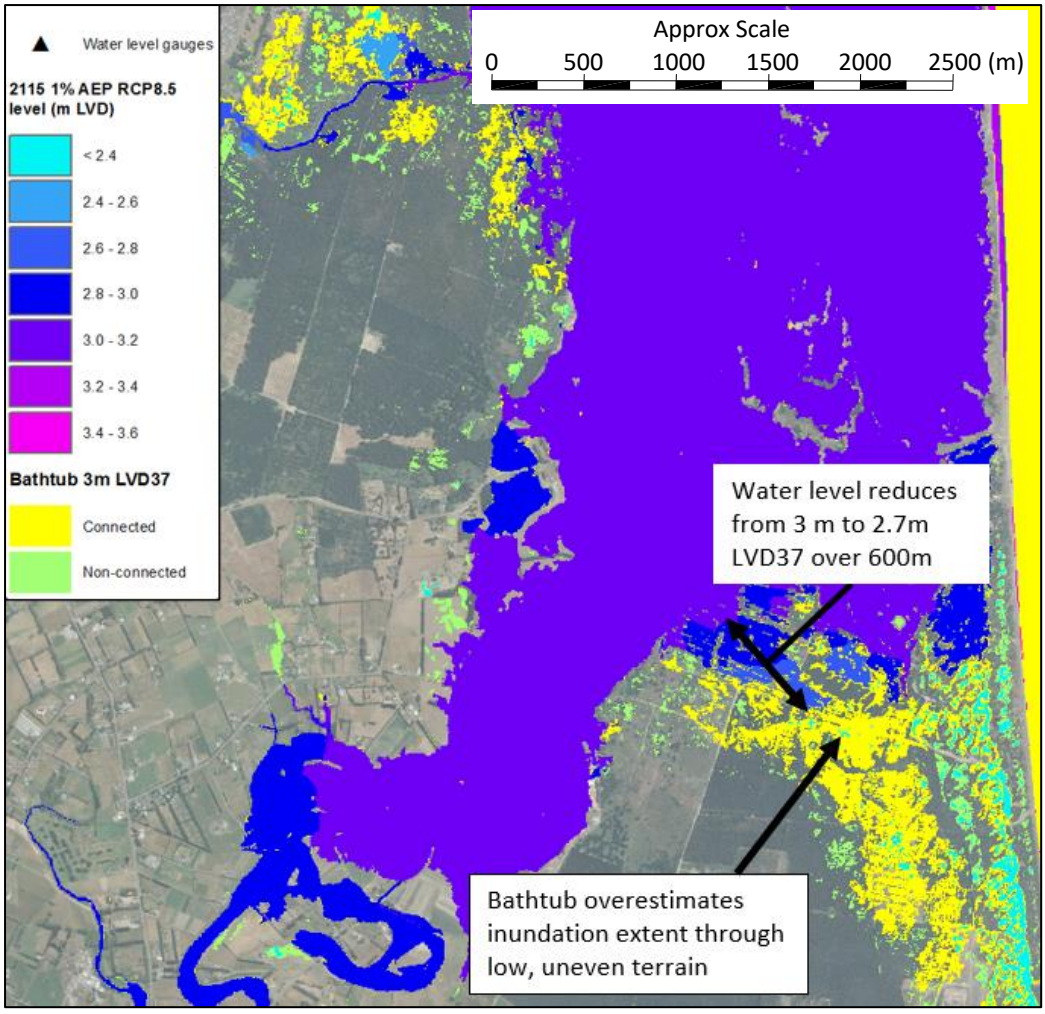


Figure 5.4: Comparison of bathtub and T+T TUFLOW model results for a high SLR scenario (2115 1% AEP RCP8.5). Key areas of difference near Brooklands Lagoon (yellow and green shading are where bathtub flood extent is larger).

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

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

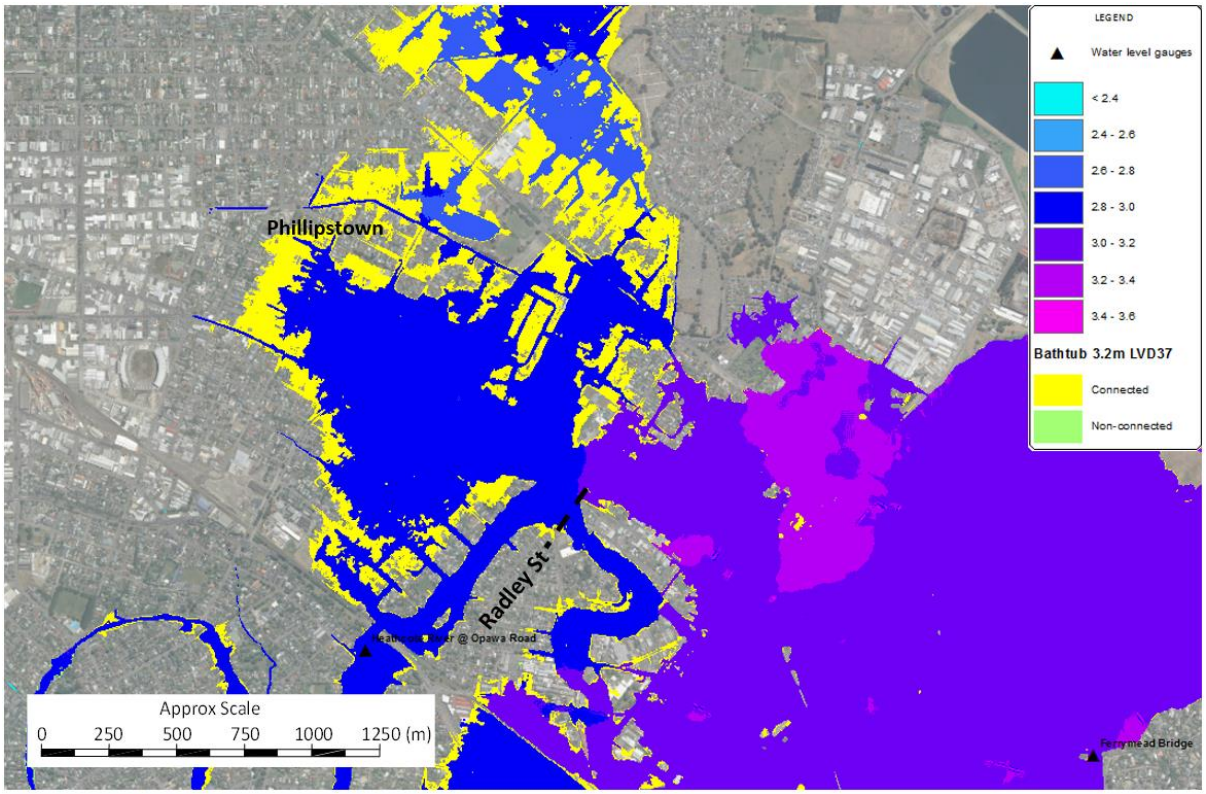


Figure 5.5: Comparison of bathtub and T+T TUFLOW model results for a high SLR scenario (2115 1% AEP RCP8.5). Key areas of difference along the Heathcote River (yellow and green shading are where bathtub flood extent is larger)

5.2 Comparison with CCC city-wide flood model

5.2.1 Present day scenario

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

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

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

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

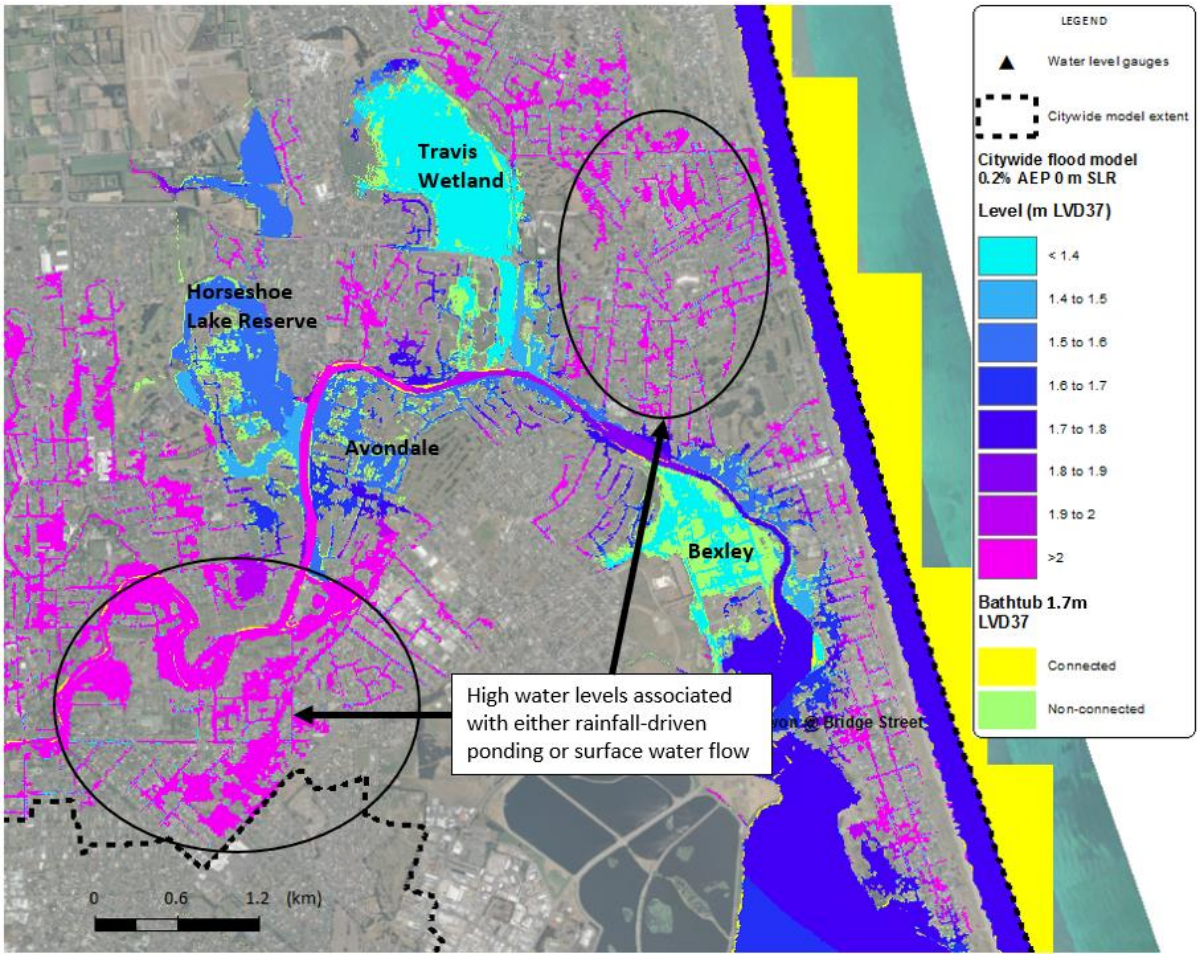


Figure 5.6: Comparison of bathtub and CCC city-wide flood model results for a present-day scenario (0.2% AEP 0 m SLR). Key areas of difference for Avon catchment (yellow and green shading are where bathtub flood extent is larger).

5.2.2 High sea level rise scenario

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

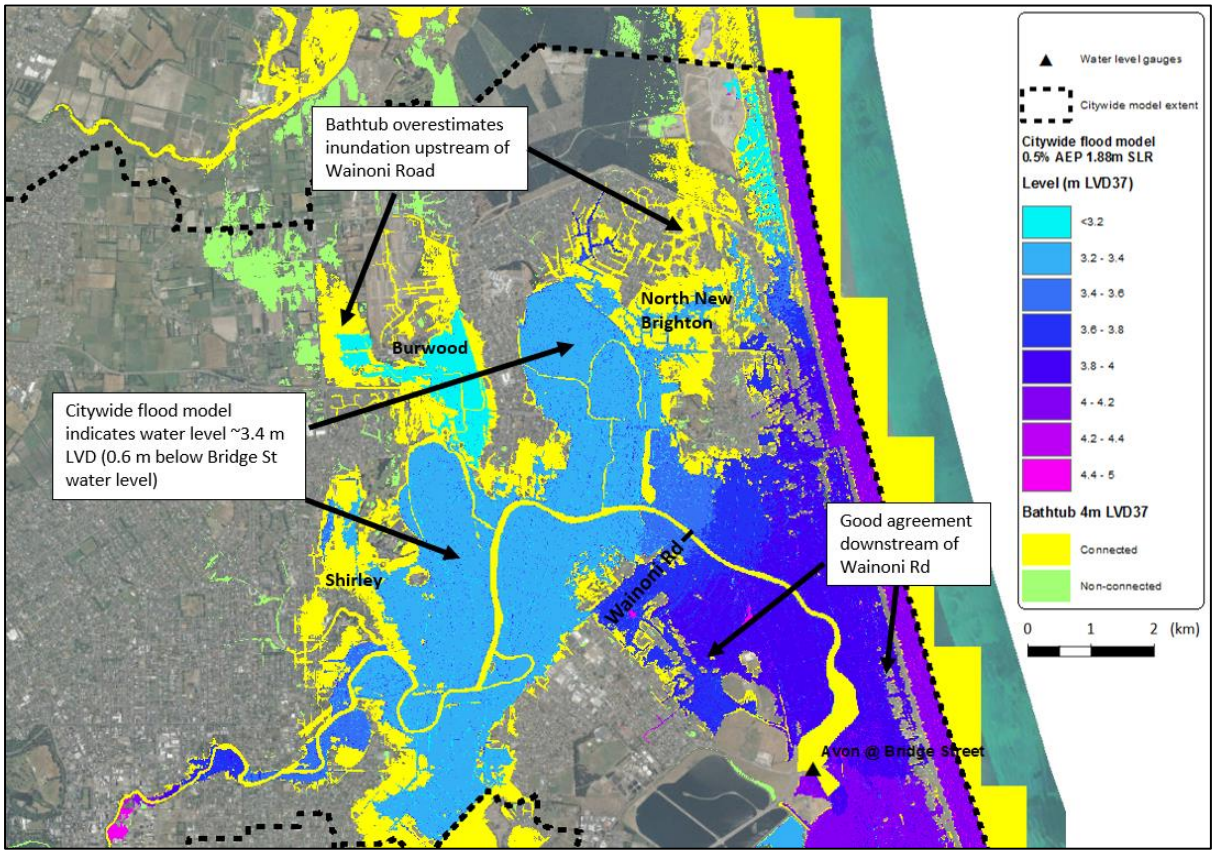


Figure 5.7: Comparison of bathtub and CCC city-wide flood model results for a high SLR scenario (2150 0.5% AEP 1.88 m SLR). Key areas of difference for Avon catchment (yellow and green shading are where bathtub flood extent is larger).

6 Conclusions

6.1 Recommended boundary for bathtub model output

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

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

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

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

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

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

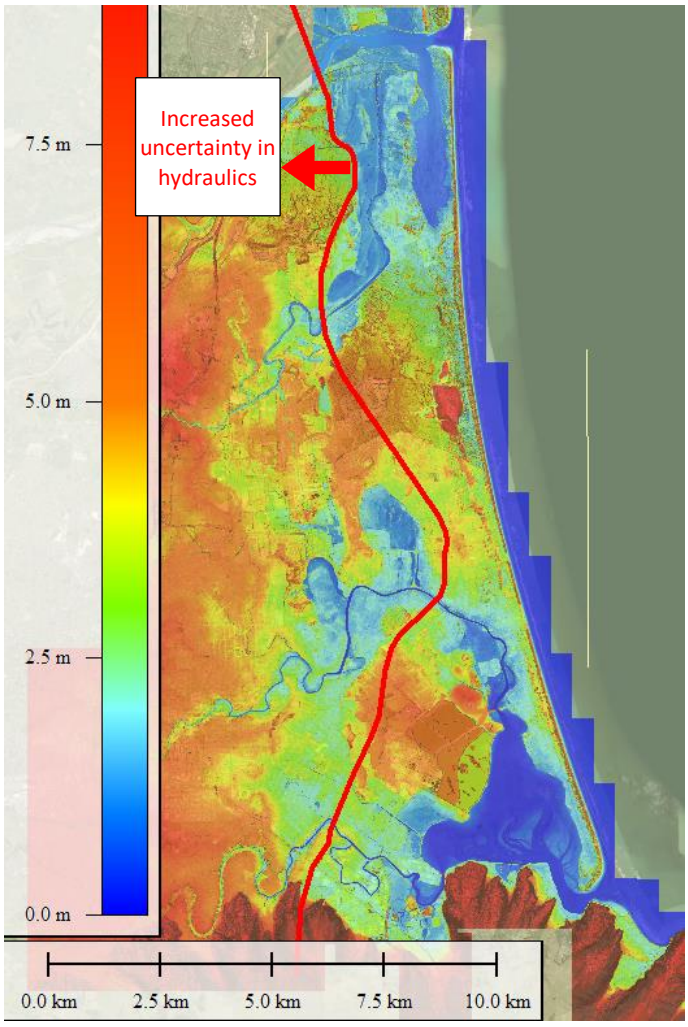


Figure 6.1: 2018 DEM used for bathtub model with the recommended bathtub boundary shown in red.

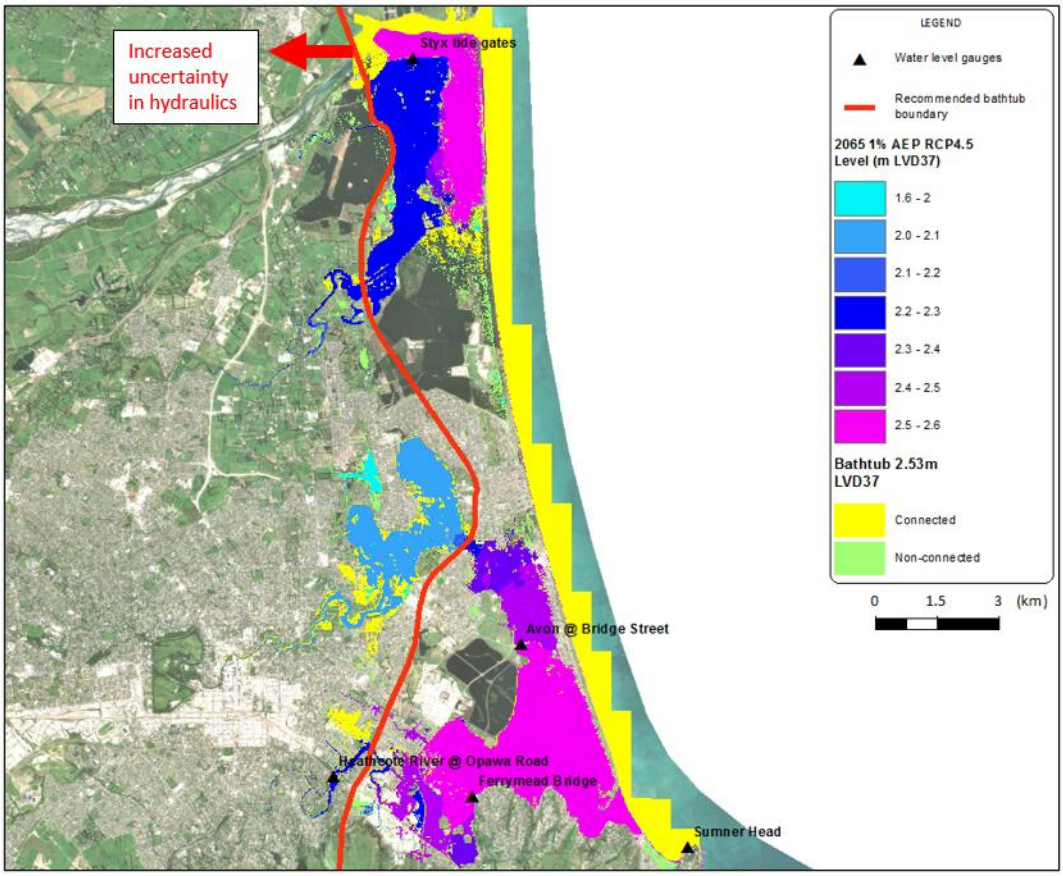


Figure 6.2: TUFLOW and bathtub model comparison with the recommended bathtub boundary shown in red

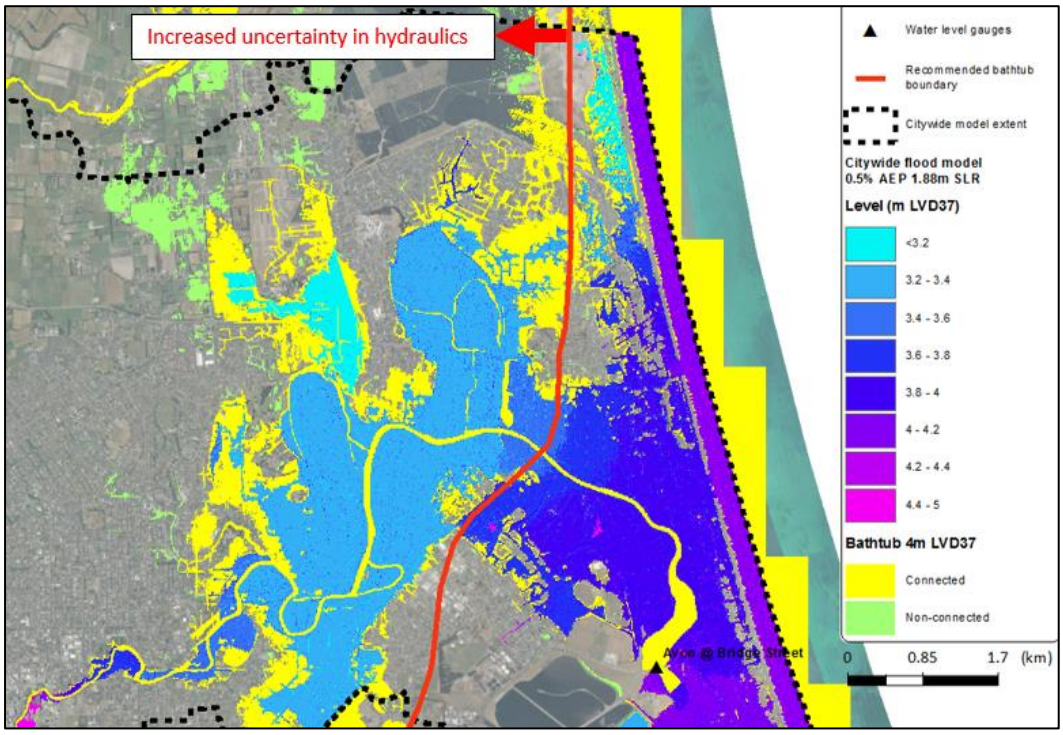


Figure 6.3: CCC city wide flood model and bathtub model comparison with the recommended bathtub boundary shown in red

6.2 Use of bathtub model for current adaptation planning purposes

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

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

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

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

6.3 Summary of recommendations

We recommend that:

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

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Appendix D: Coastal inundation levels

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Return period	Present day	Relative sea level rise (m)							
		0.2	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.8	2.0	2.2	2.4	2.6	3.0	3.2	3.3	3.8
10 year ARI	2.0	2.2	2.4	2.6	2.8	3.2	3.4	3.5	4.0
100 year ARI	2.1	2.3	2.5	2.7	2.9	3.3	3.5	3.6	4.1

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

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

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

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

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

Return period	Present day	Relative sea level rise (m)							
		0.2	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	2.3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10 year ARI	2.7	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
100 year ARI	2.9	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

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

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

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

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

Appendix E: Example maps

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

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

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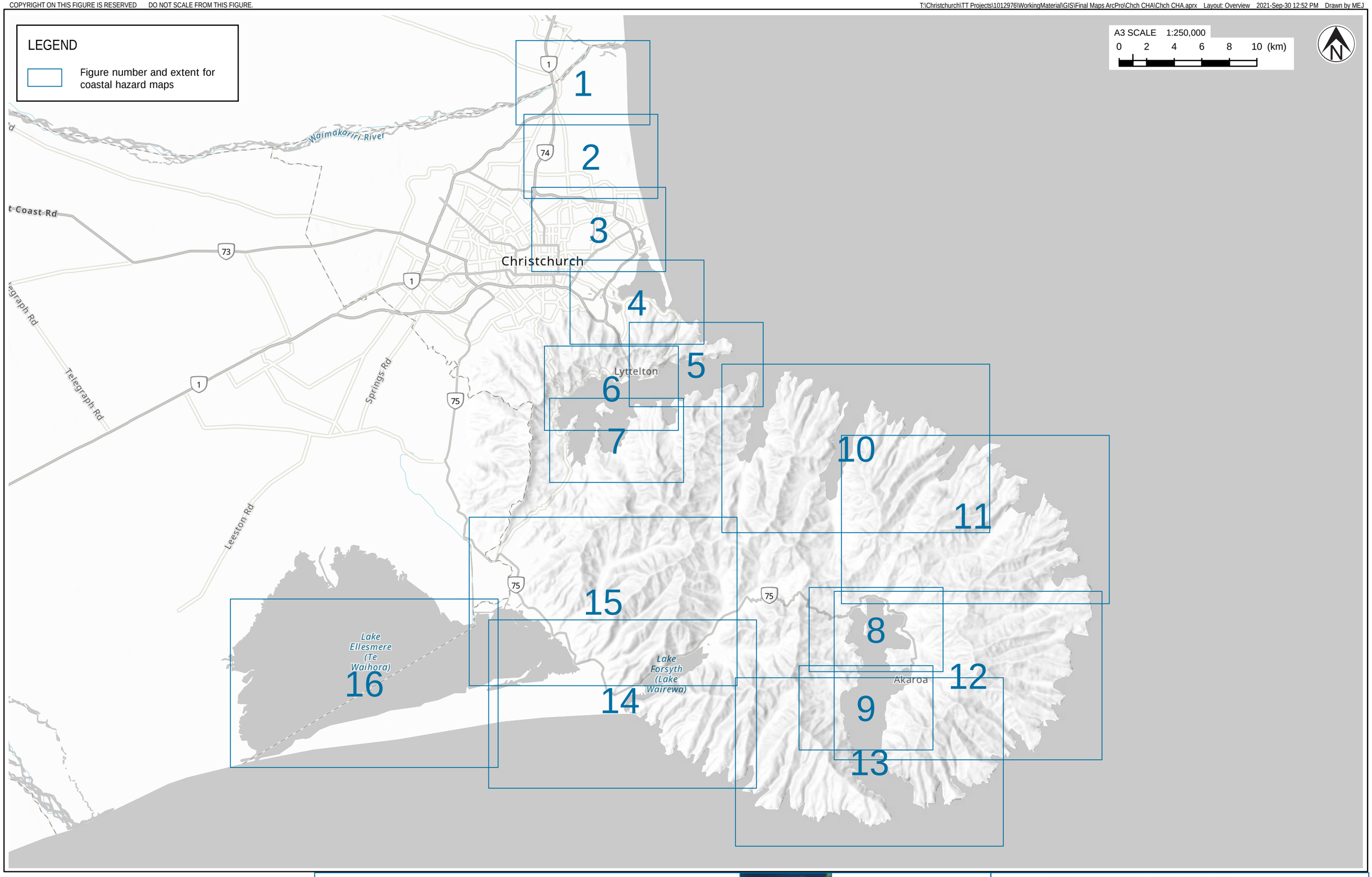

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NOTES:

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PROJECT **COASTAL HAZARD ASSESSMENT**

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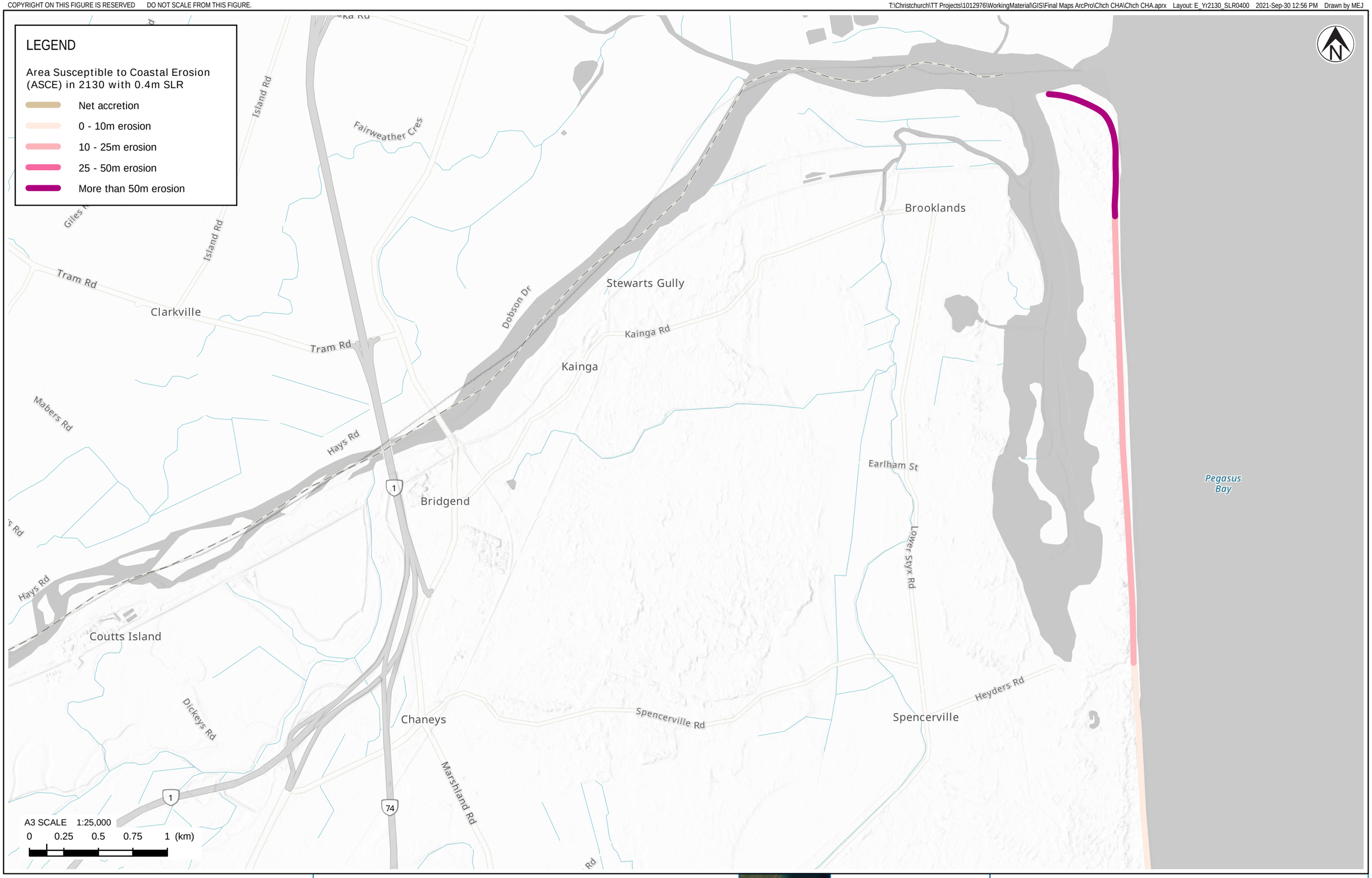

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NOTES:
1. For detailed analysis areas, this map shows the P5 erosion distance (5% probability that erosion will be greater for this scenario).
2. For regional screening analysis areas, this map shows the upper envelope erosion distance.
3. Basemap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

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PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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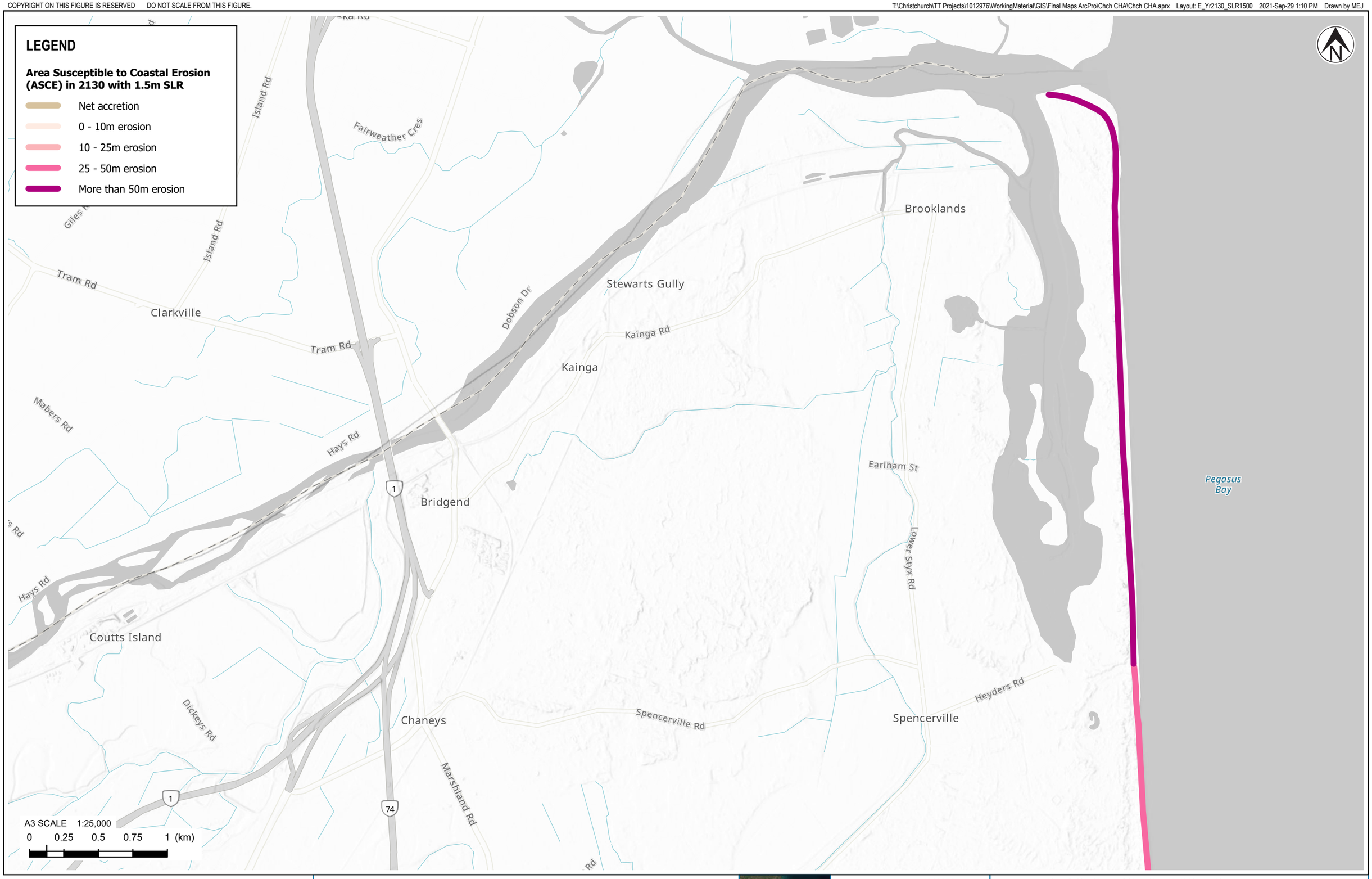

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NOTES:
1. For detailed analysis areas, this map shows the P5 erosion distance (5% probability that erosion will be greater for this scenario).
2. For regional screening analysis areas, this map shows the upper envelope erosion distance.
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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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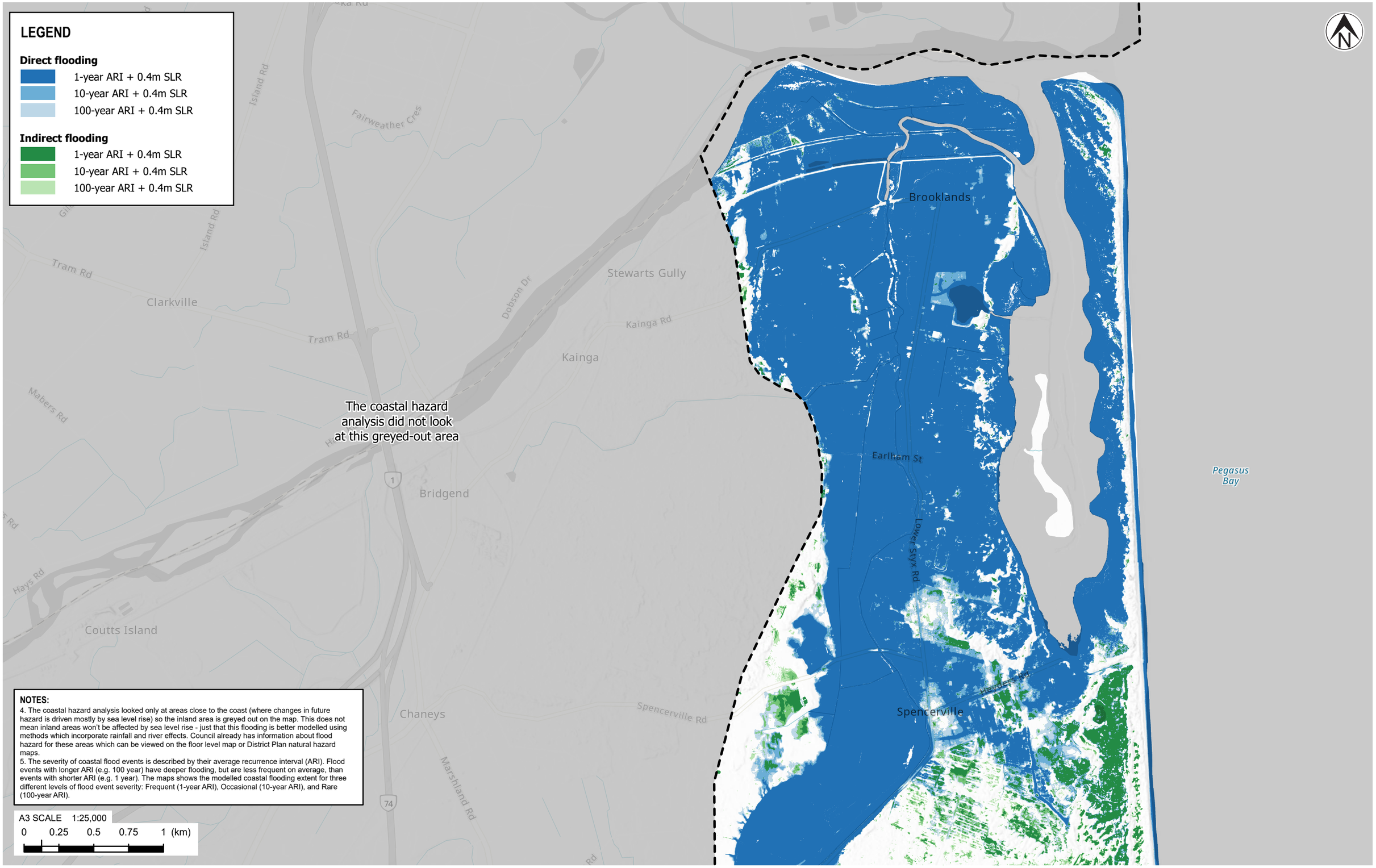

NOTES:

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

NOTES:

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

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TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

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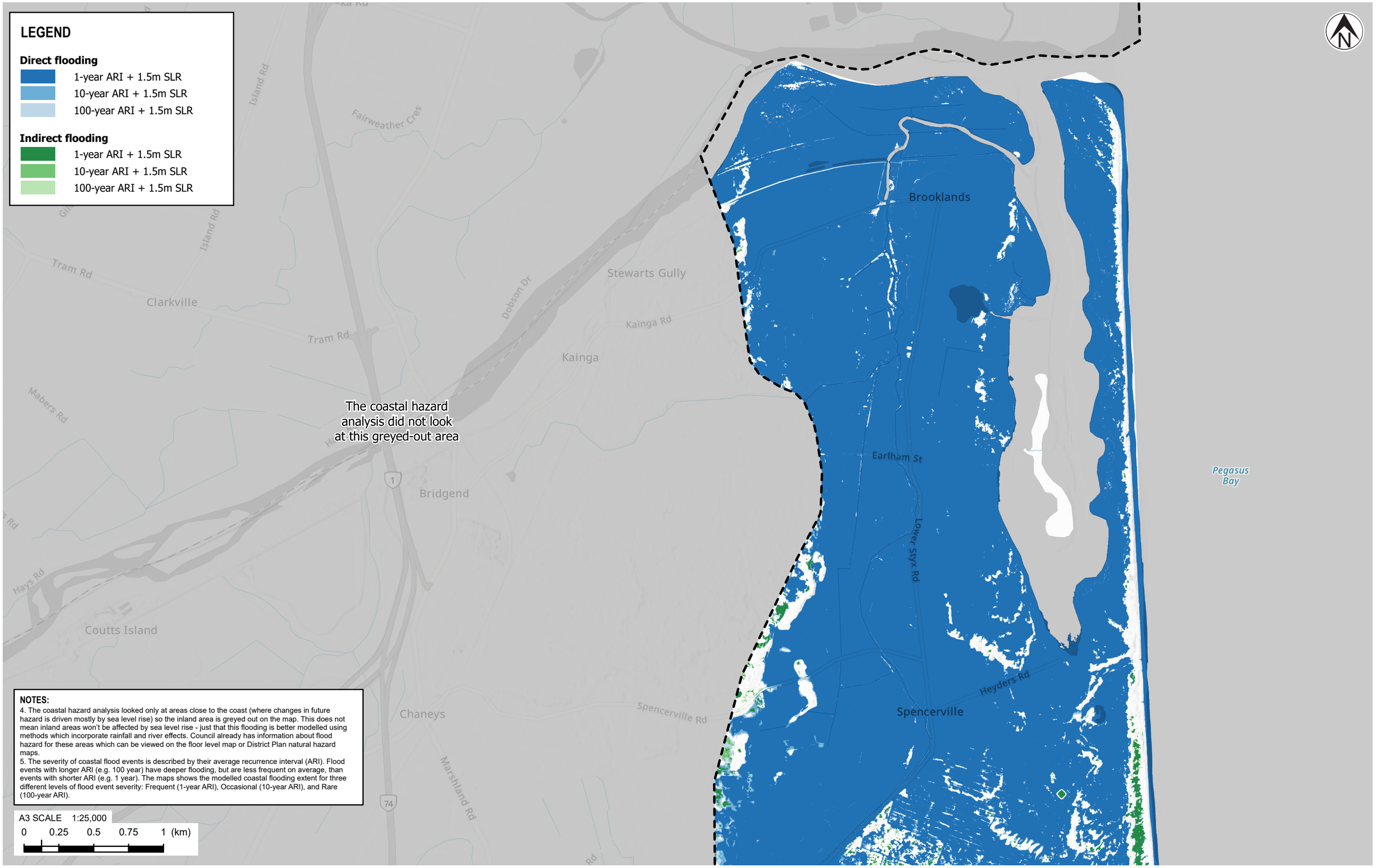

NOTES:
4. The coastal hazard analysis looked only at areas close to the coast (where changes in future hazard is driven mostly by sea level rise) so the inland area is greyed out on the map. This does not mean inland areas won't be affected by sea level rise - just that this flooding is better modelled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

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

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TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE
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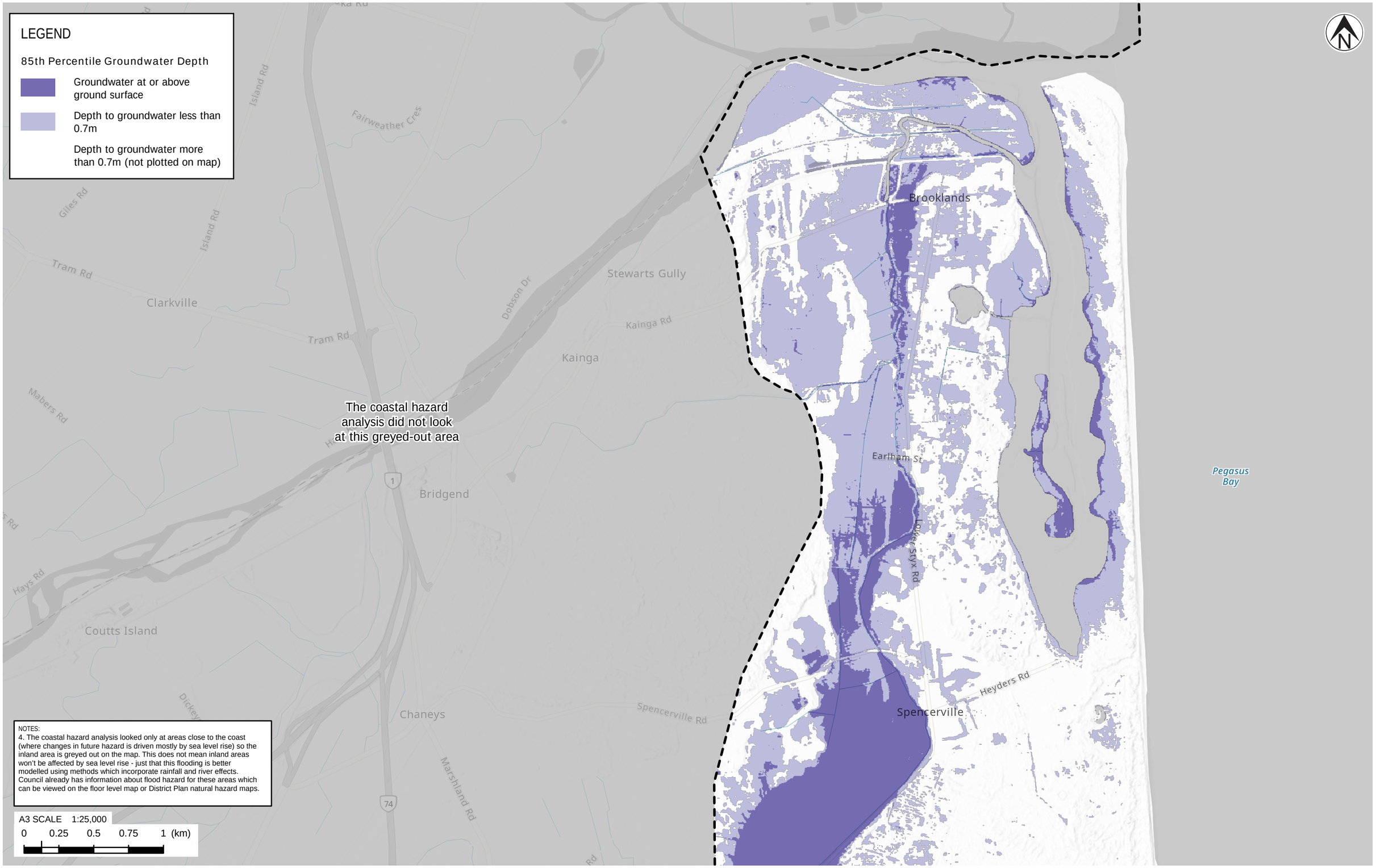

NOTES:
4. The coastal hazard analysis looked only at areas close to the coast (where changes in future hazard is driven mostly by sea level rise) so the inland area is greyed out on the map. This does not mean inland areas won't be affected by sea level rise - just that this flooding is better modelled using methods which incorporate rainfall and river effects. Council already has information about flood hazard for these areas which can be viewed on the floor level map or District Plan natural hazard maps.

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

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE

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

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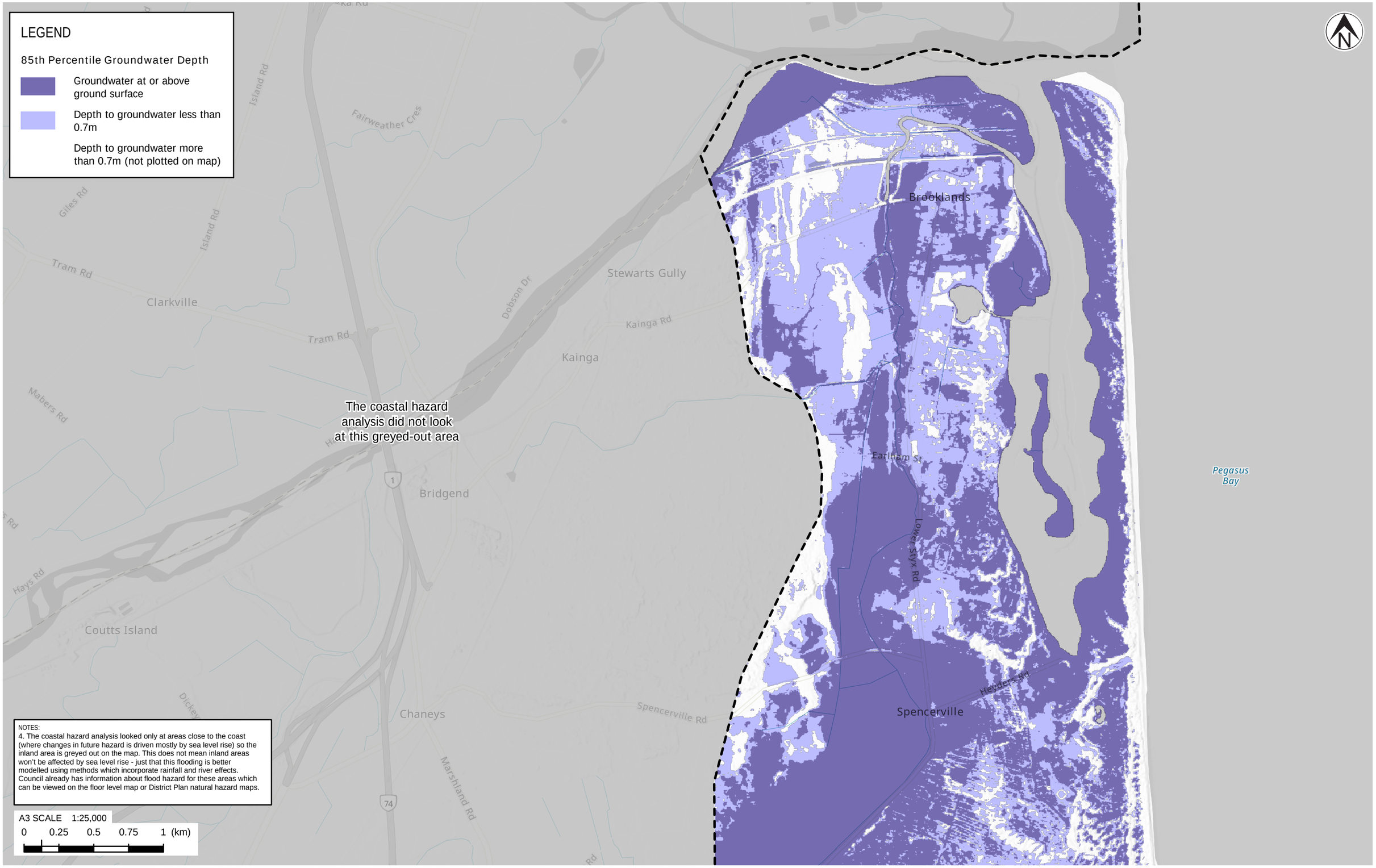

NOTES:
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A3 SCALE 1:25,000
0 0.25 0.5 0.75 1 (km)

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1	Report issued	MEJ	PPK	29/09/21

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

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

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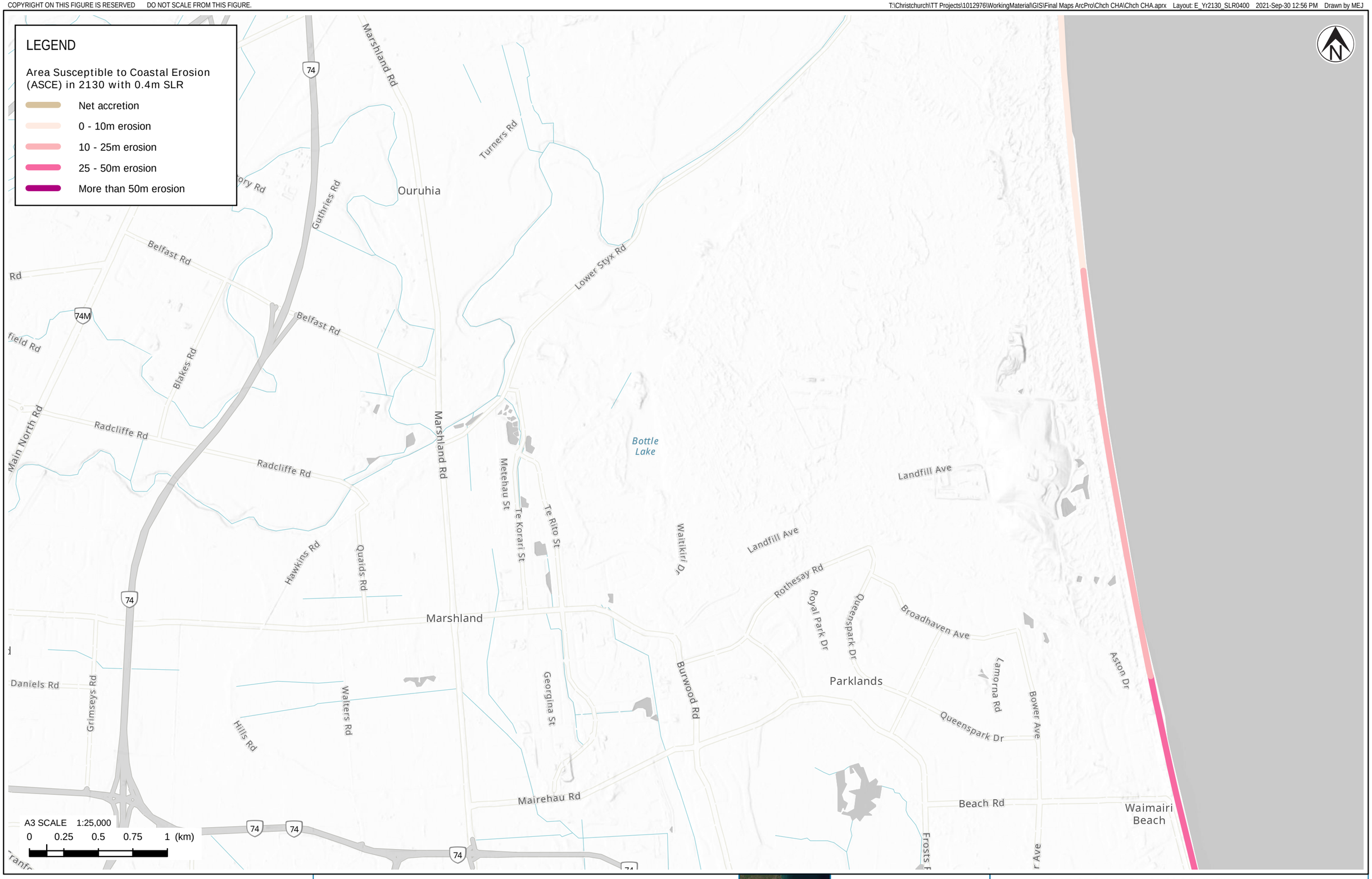

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CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:25,000 FIG No. FIGURE 2A REV 1

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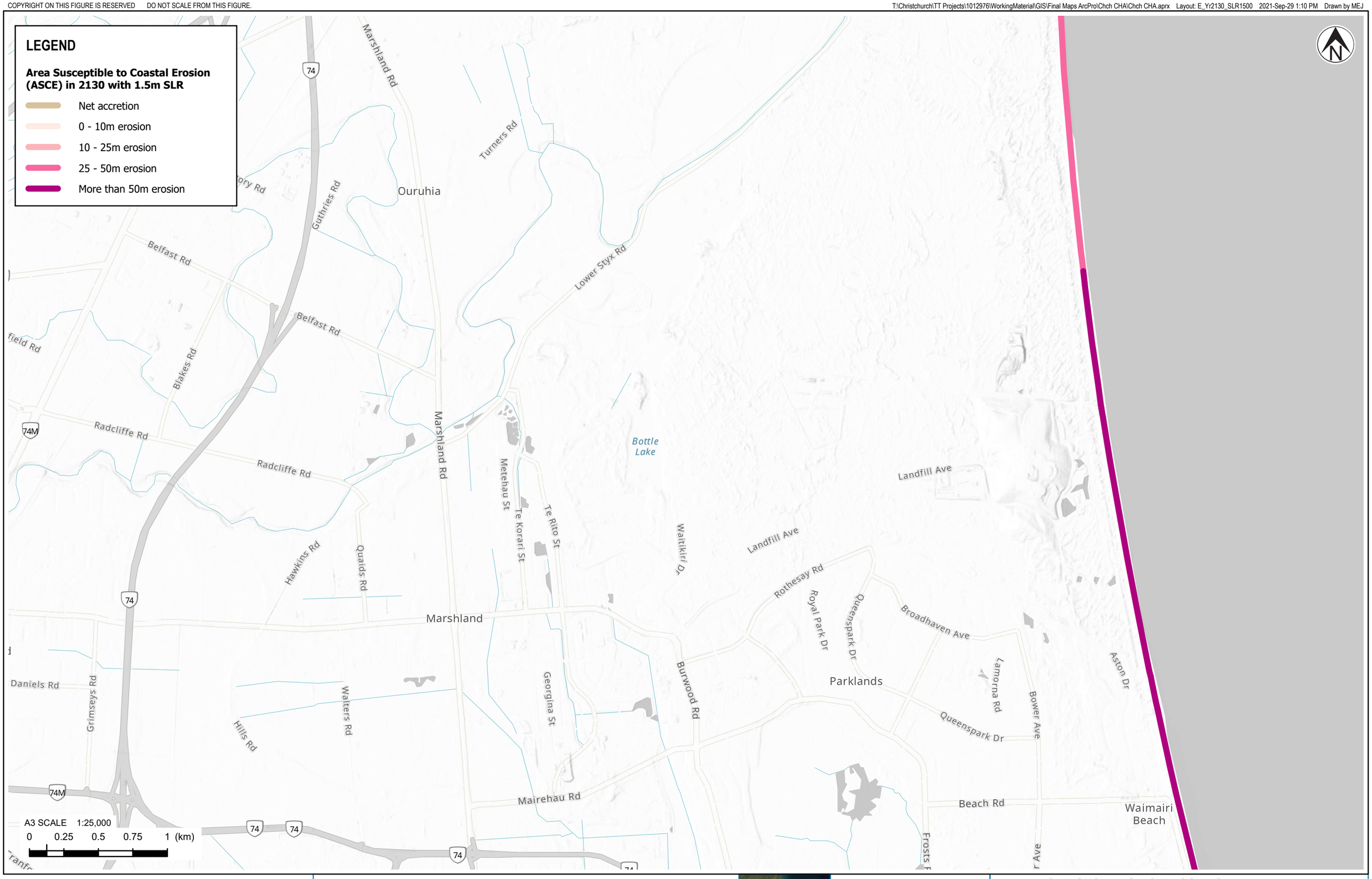

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:25,000 FIG No. FIGURE 2B REV 1

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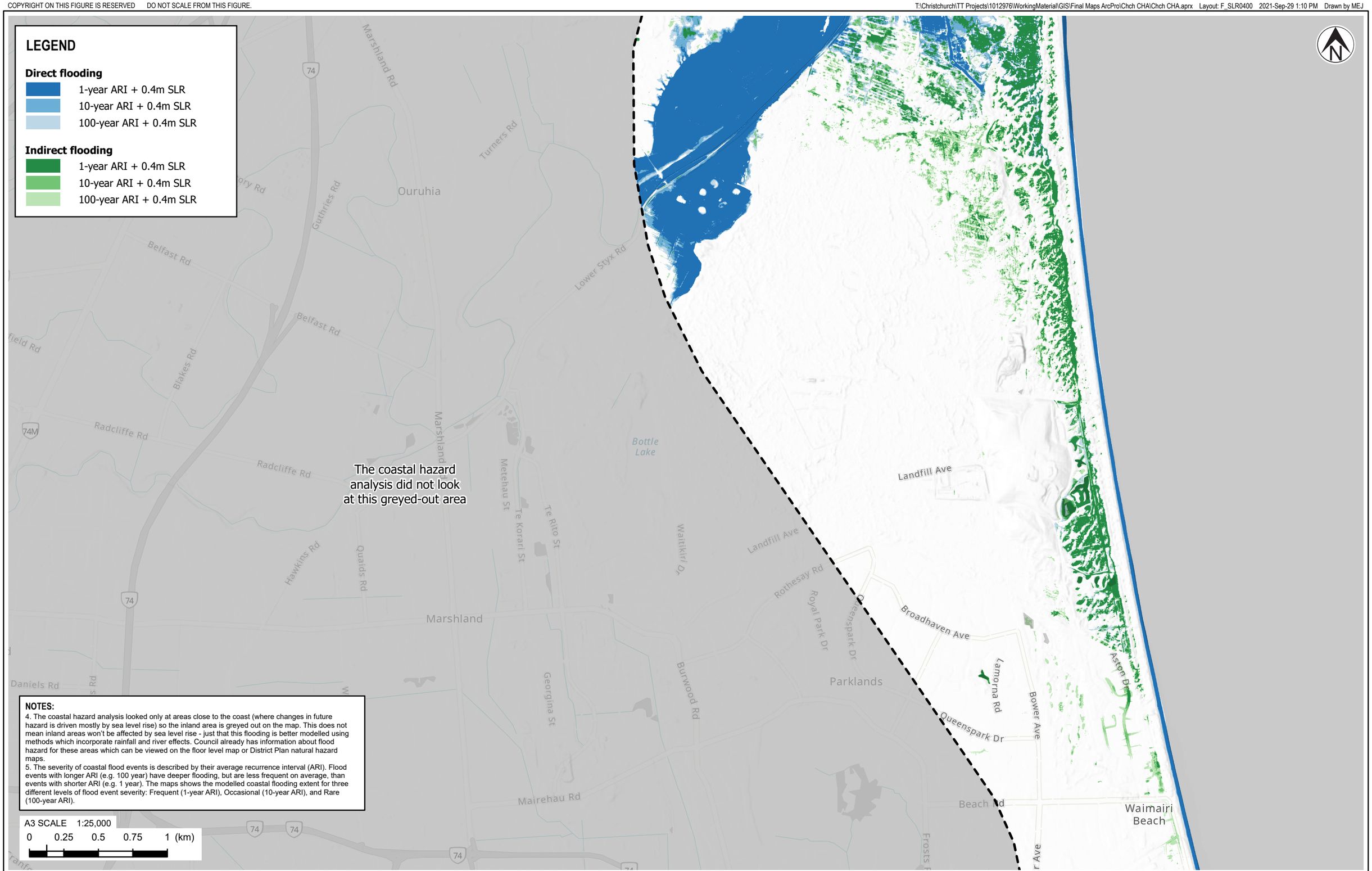

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APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 2C REV 1

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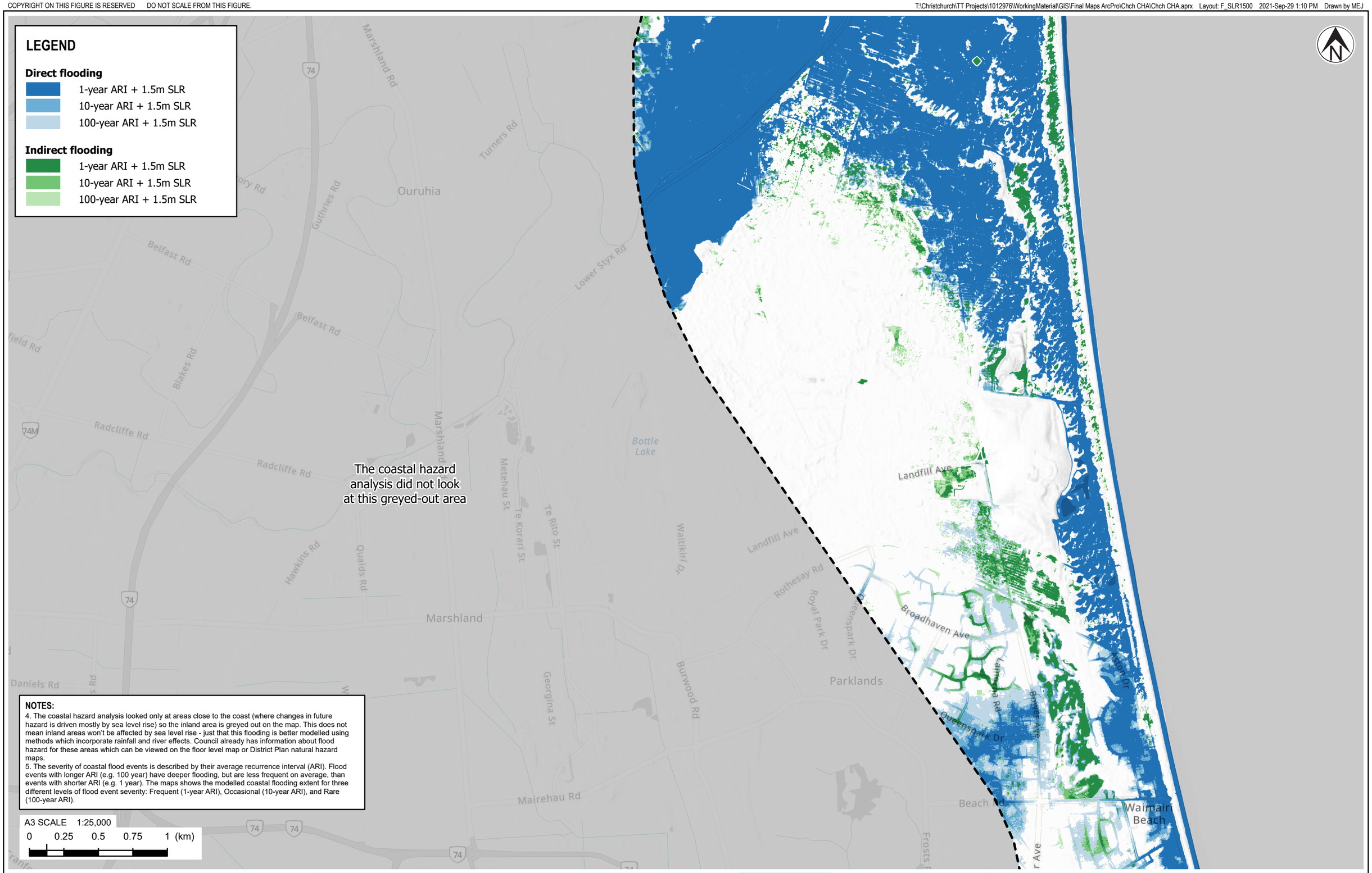

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0 0.25 0.5 0.75 1 (km)

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APPROVED	P. COCHRANE	DATE 29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 2D REV 1

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CLIENT **CHRISTCHURCH CITY COUNCIL**
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TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 2E REV 1

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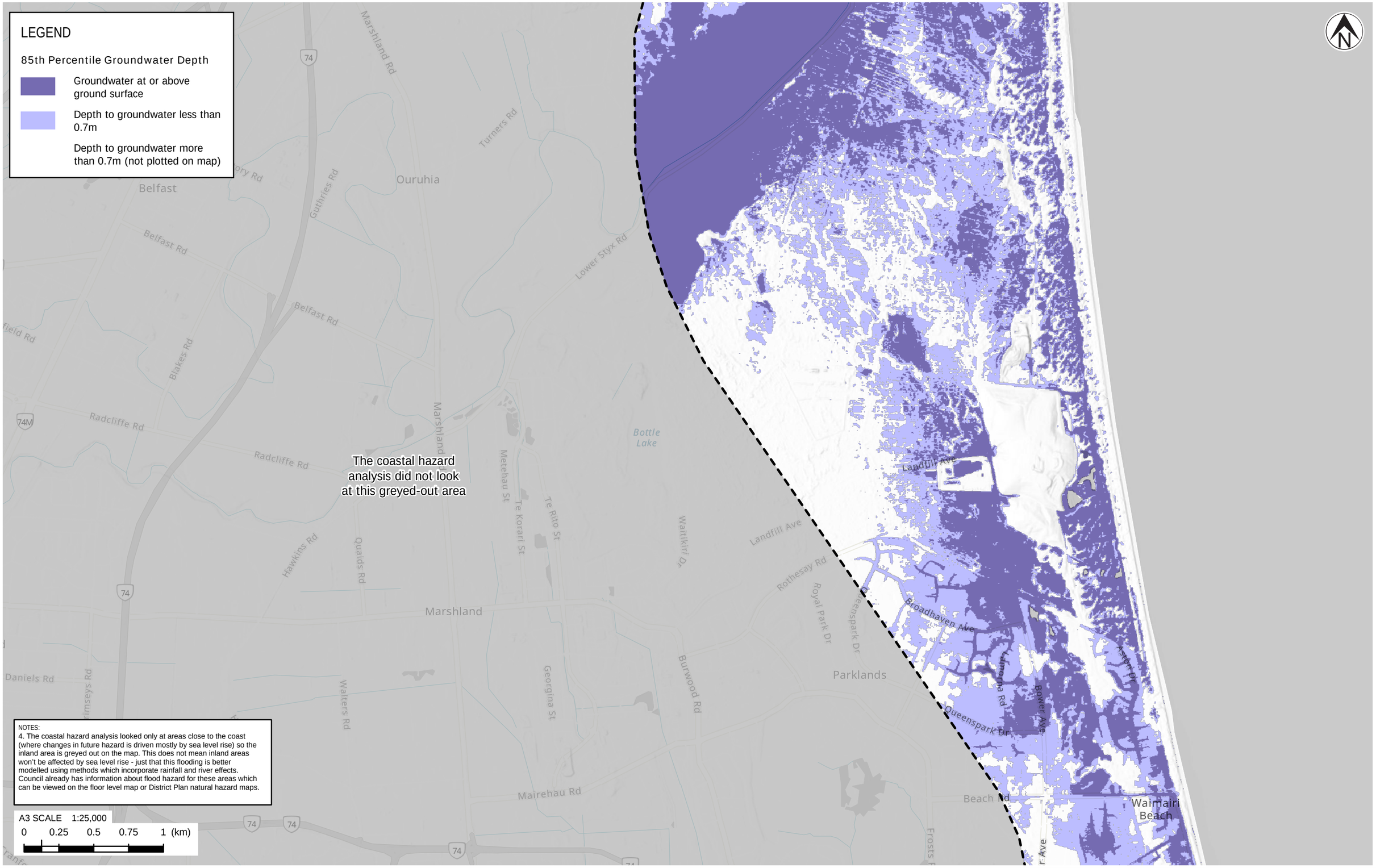

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 2F REV 1

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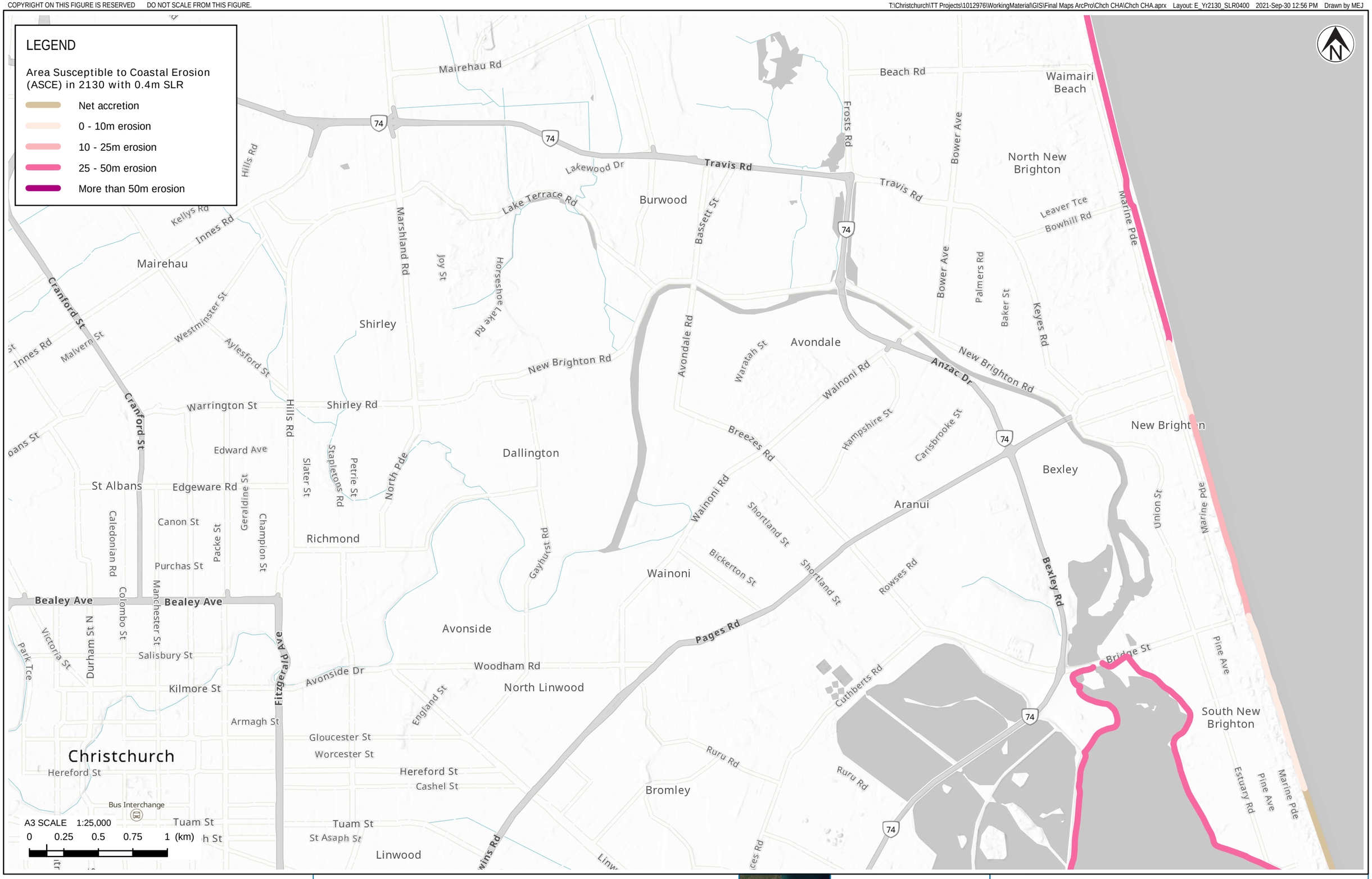

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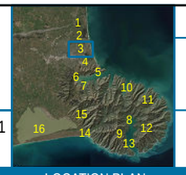

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DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	RHAU	SEP.21
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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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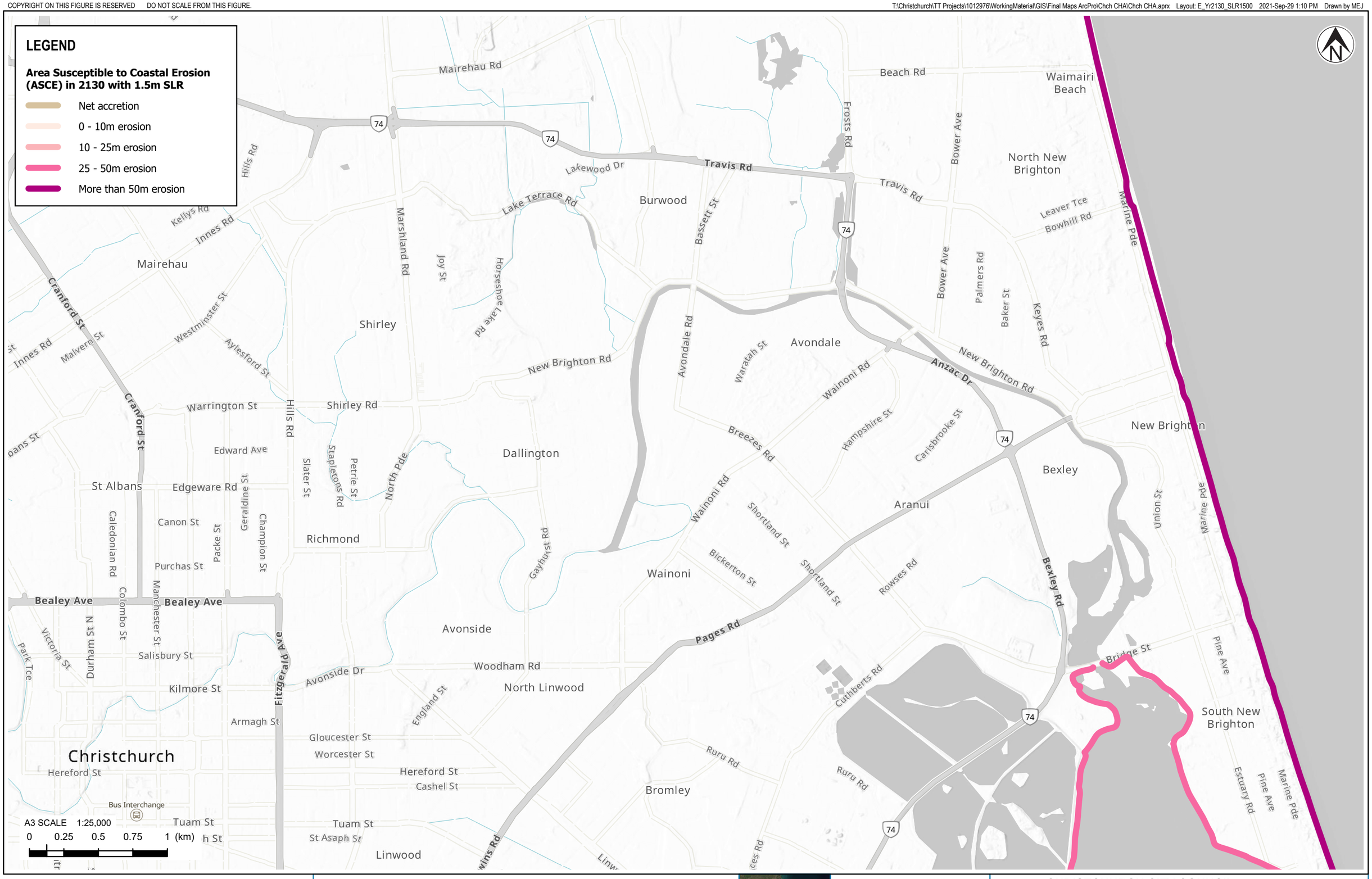

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1	Report issued	MEJ	RHAU	29/09/21

LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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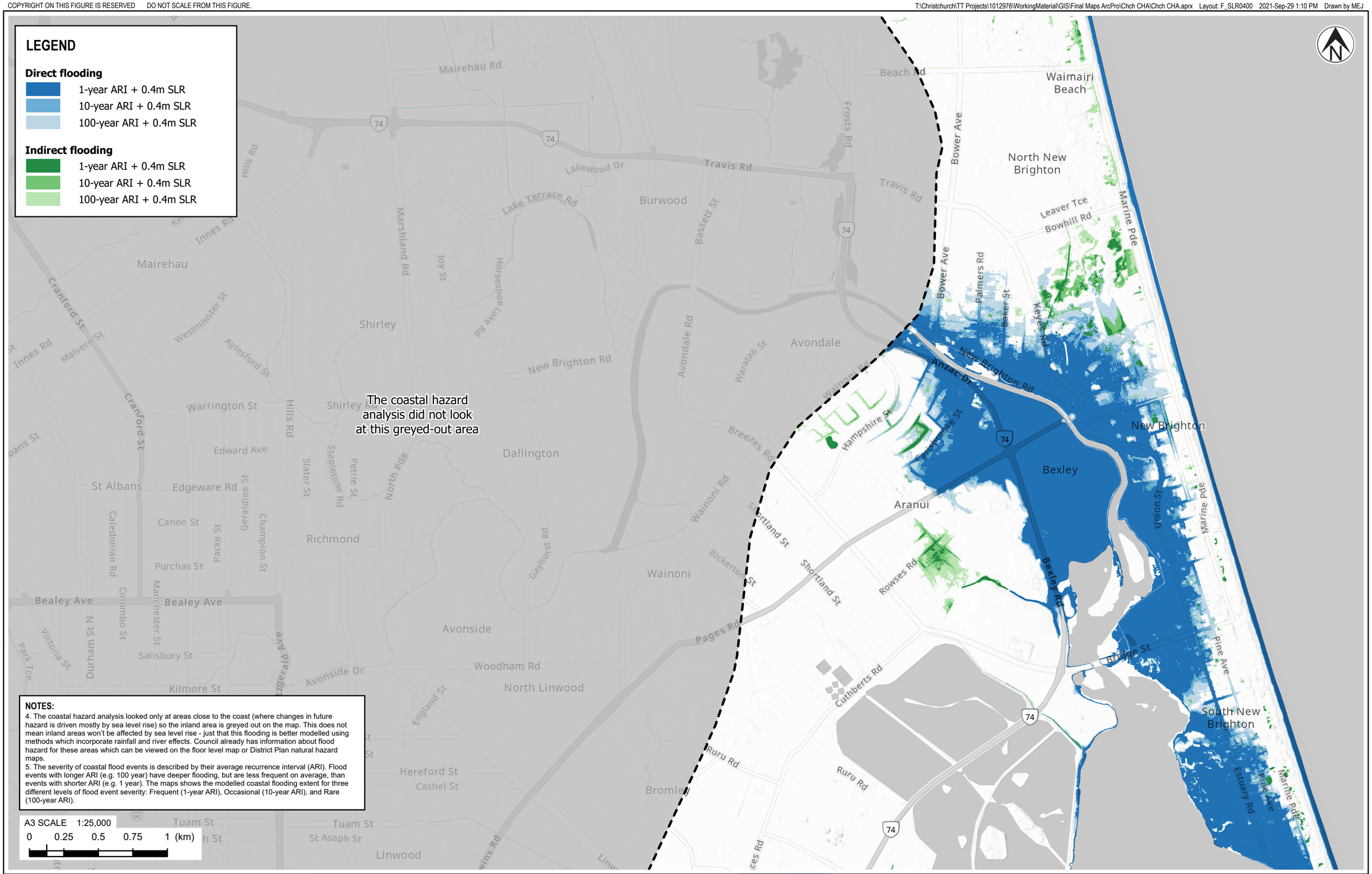

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CLIENT **CHRISTCHURCH CITY COUNCIL**
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TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 3C REV 1

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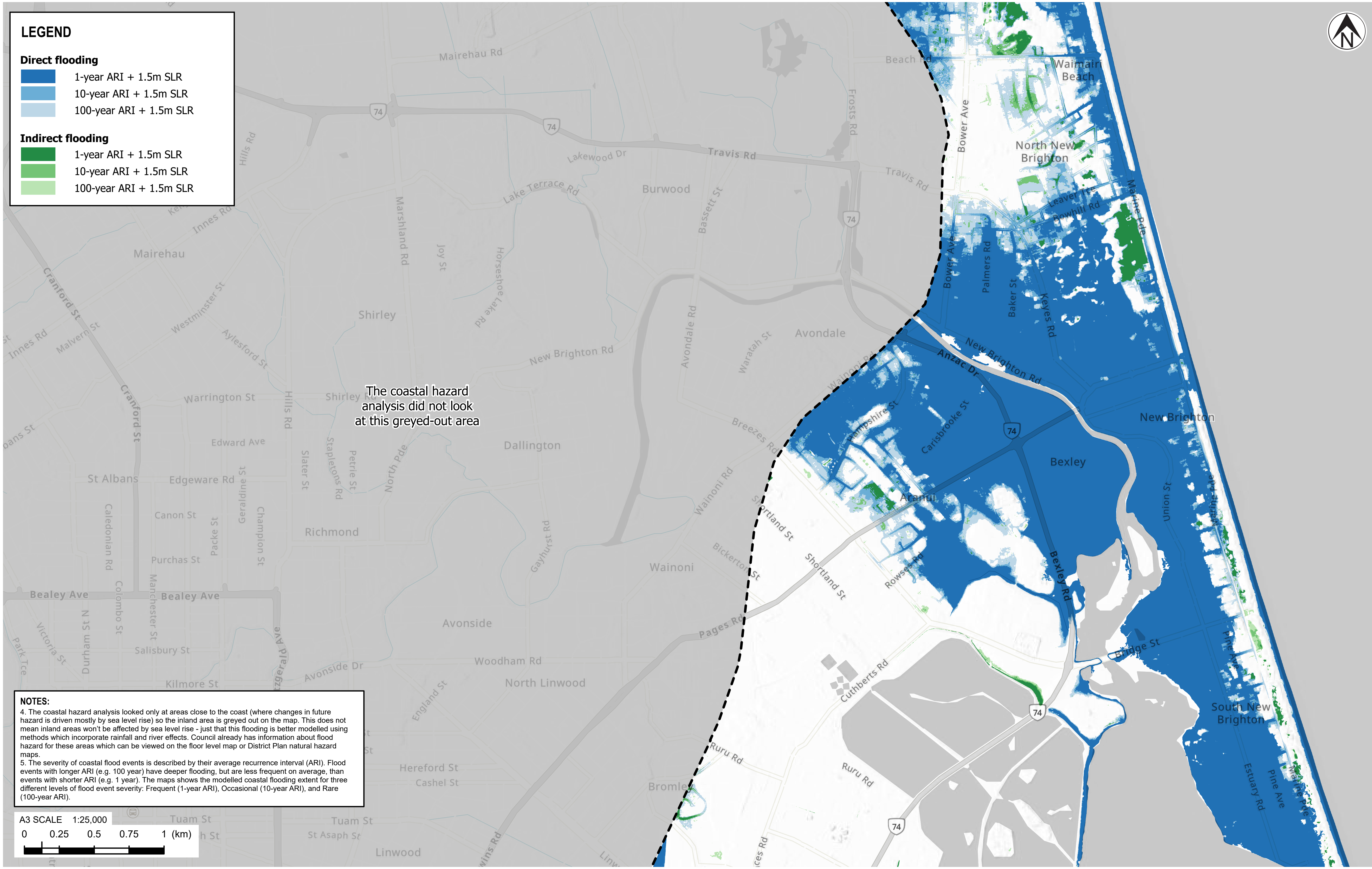

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE

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

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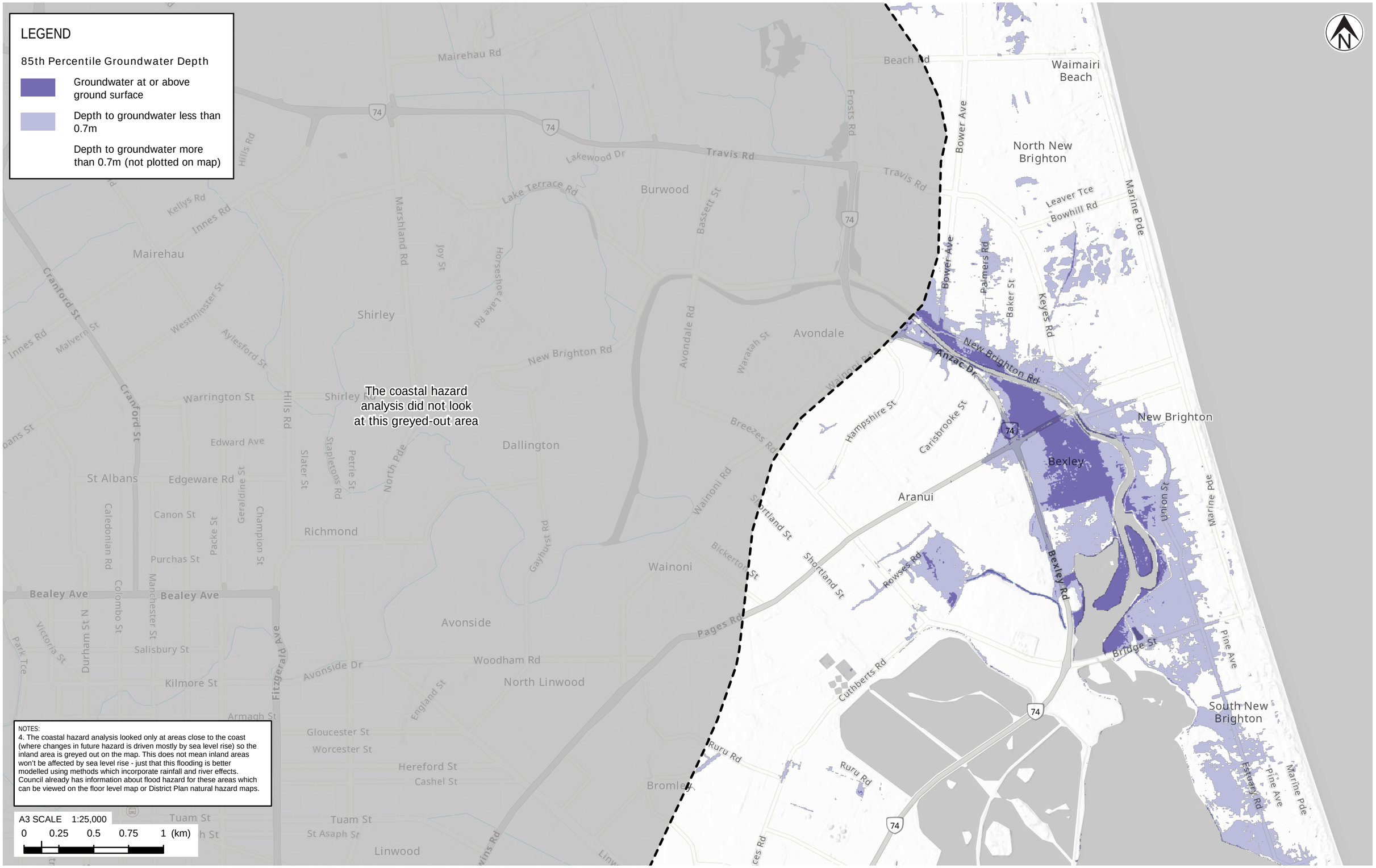

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TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 3E REV 1

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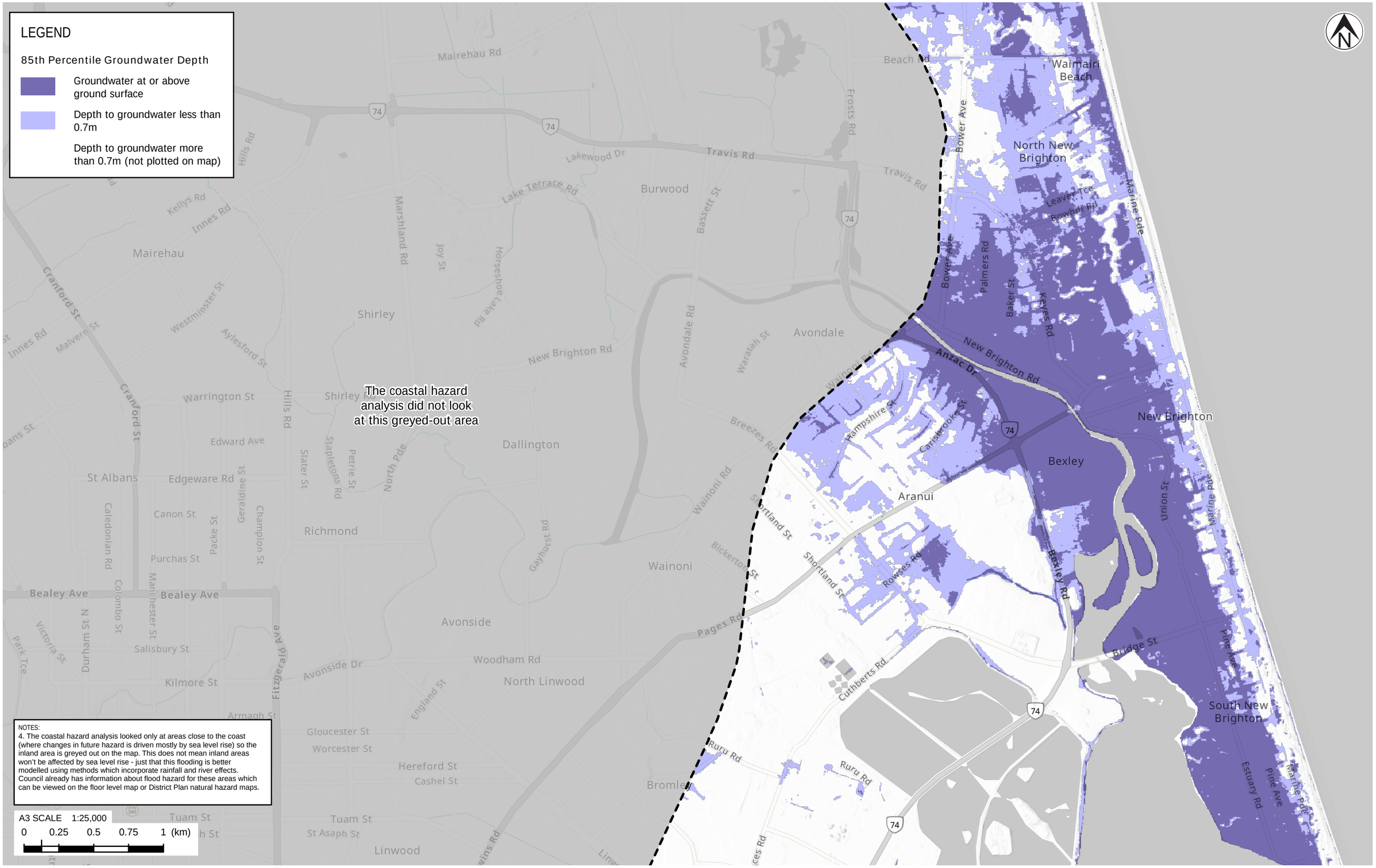

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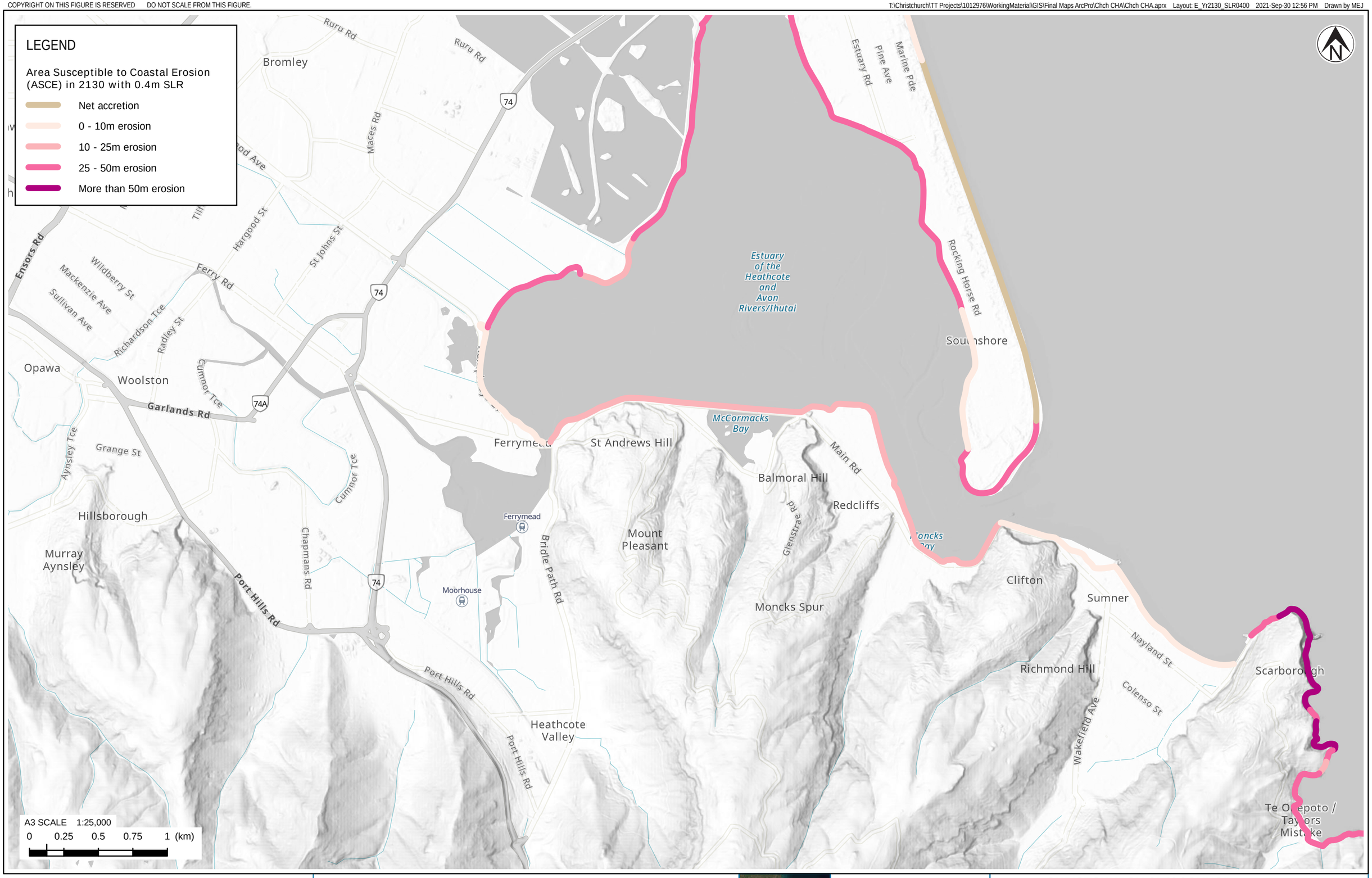

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PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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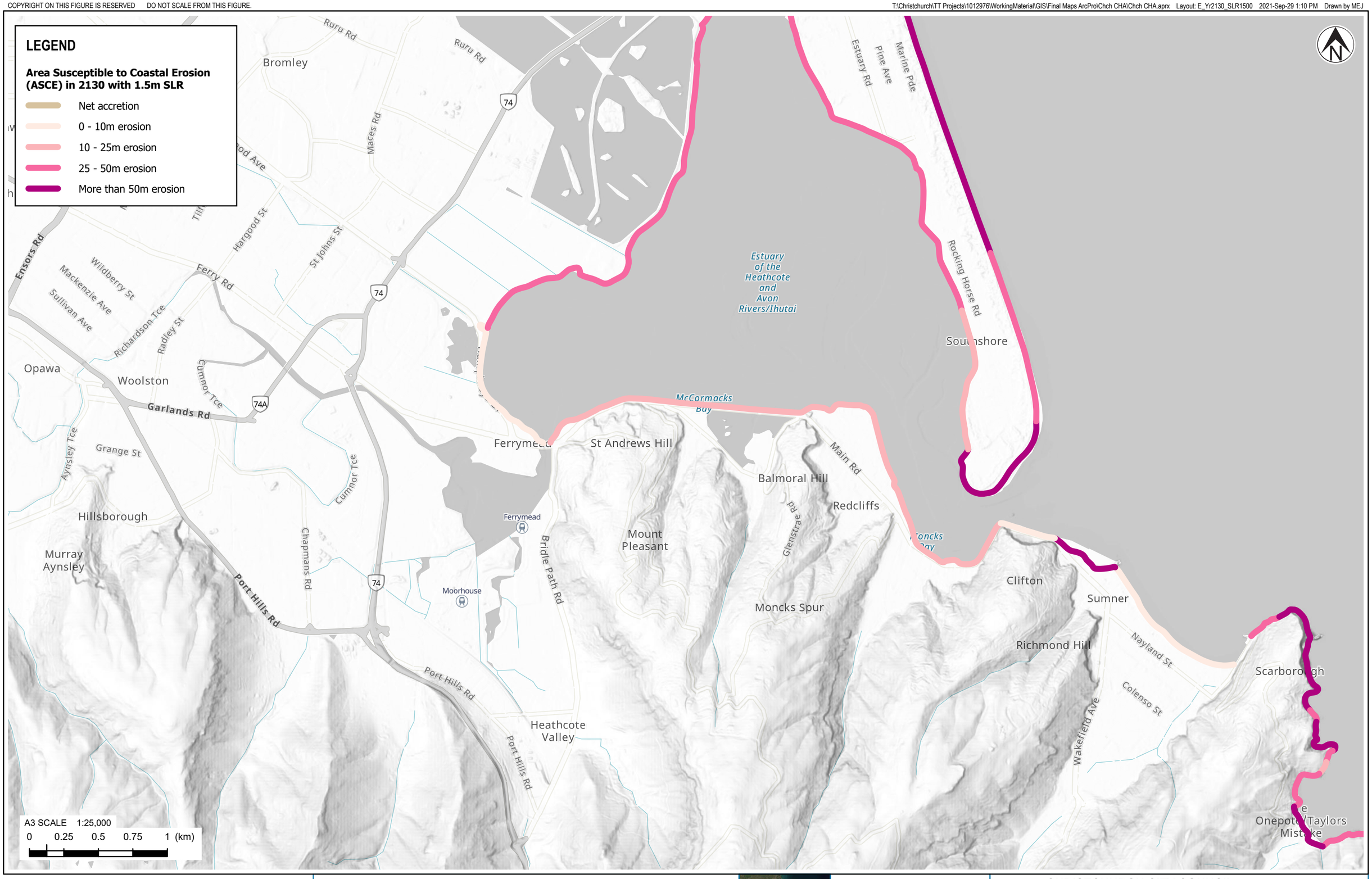

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NOTES:

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

LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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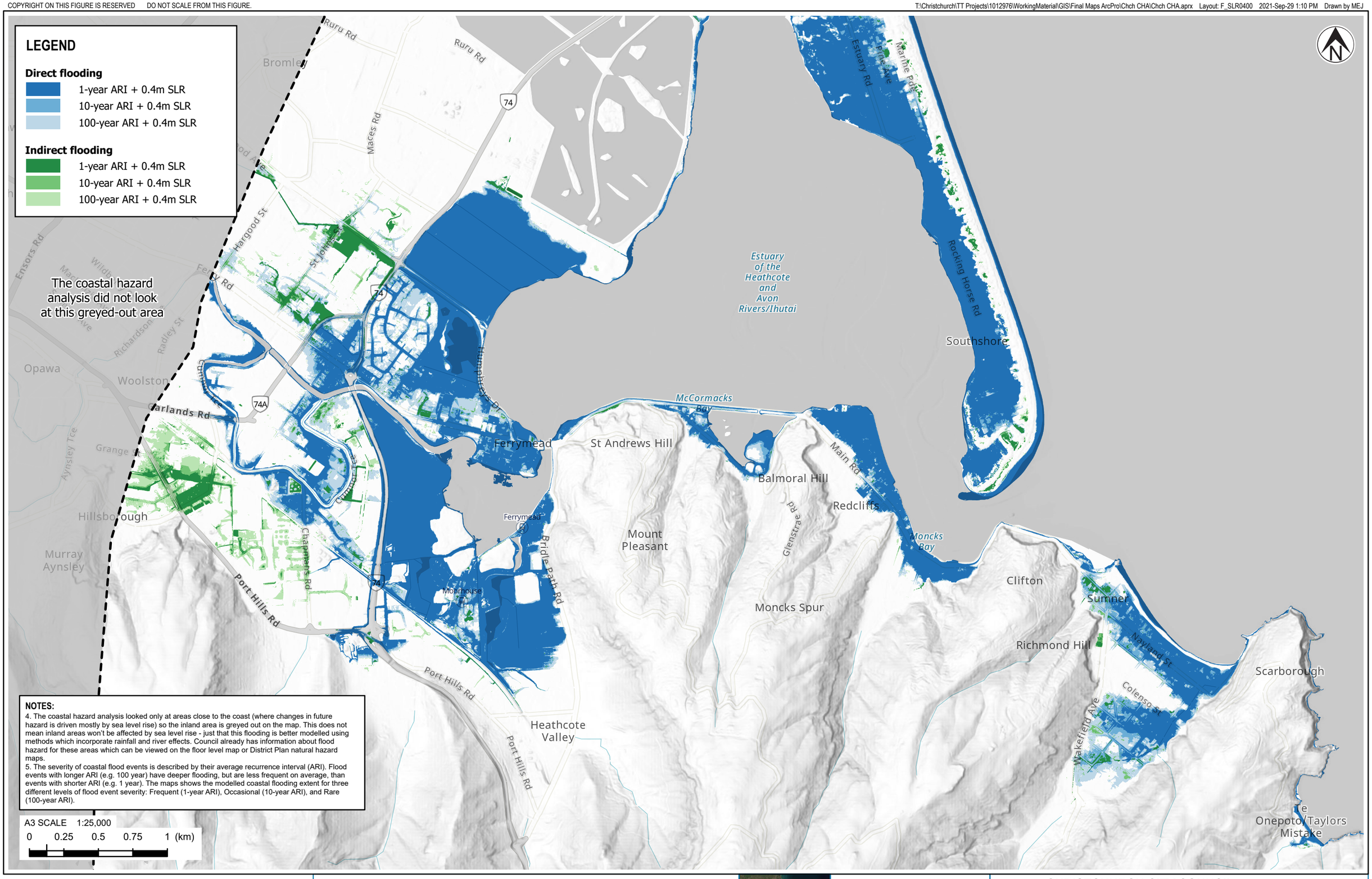

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A3 SCALE 1:25,000

0 0.25 0.5 0.75 1 (km)

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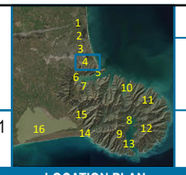

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PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

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

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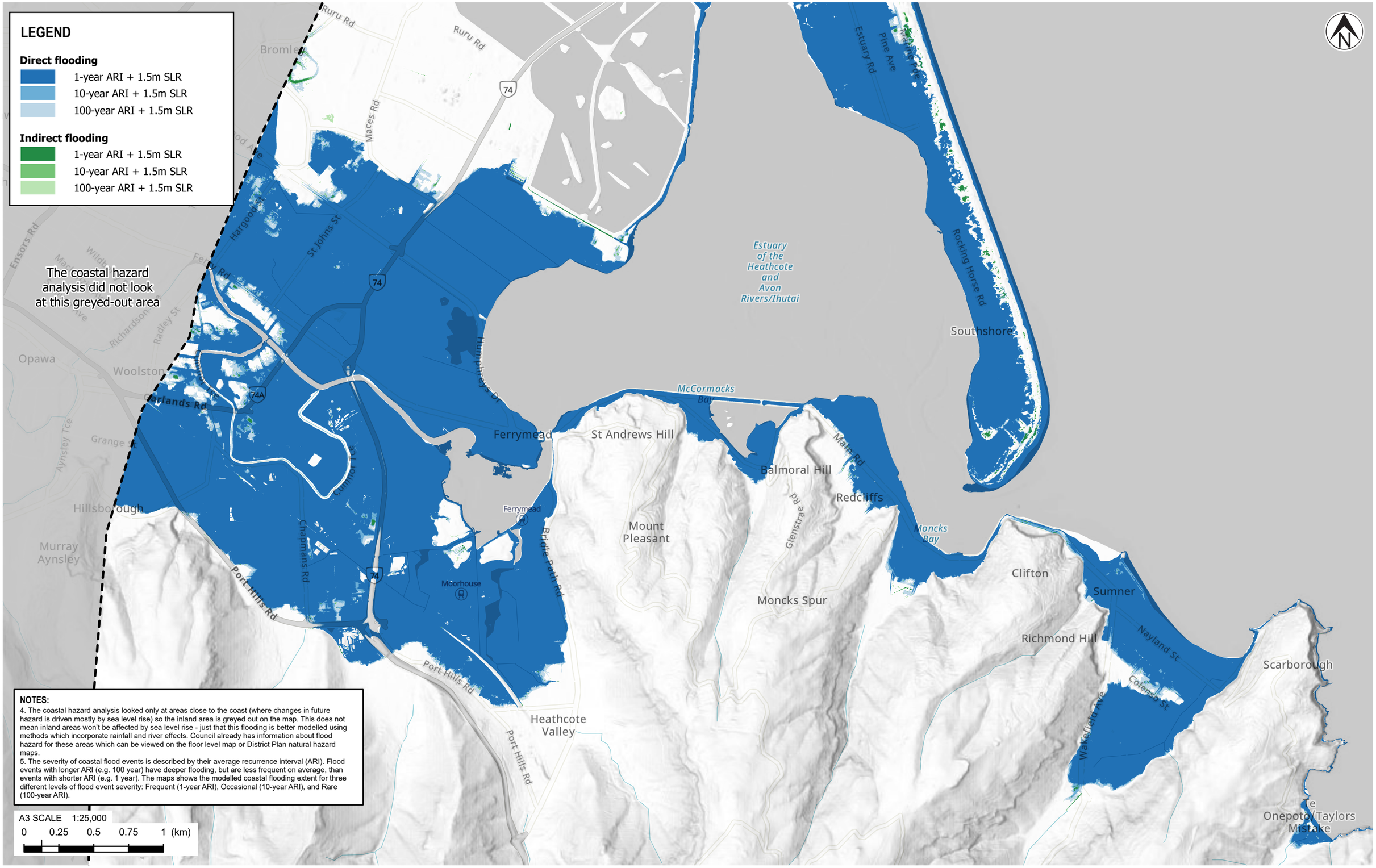

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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

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

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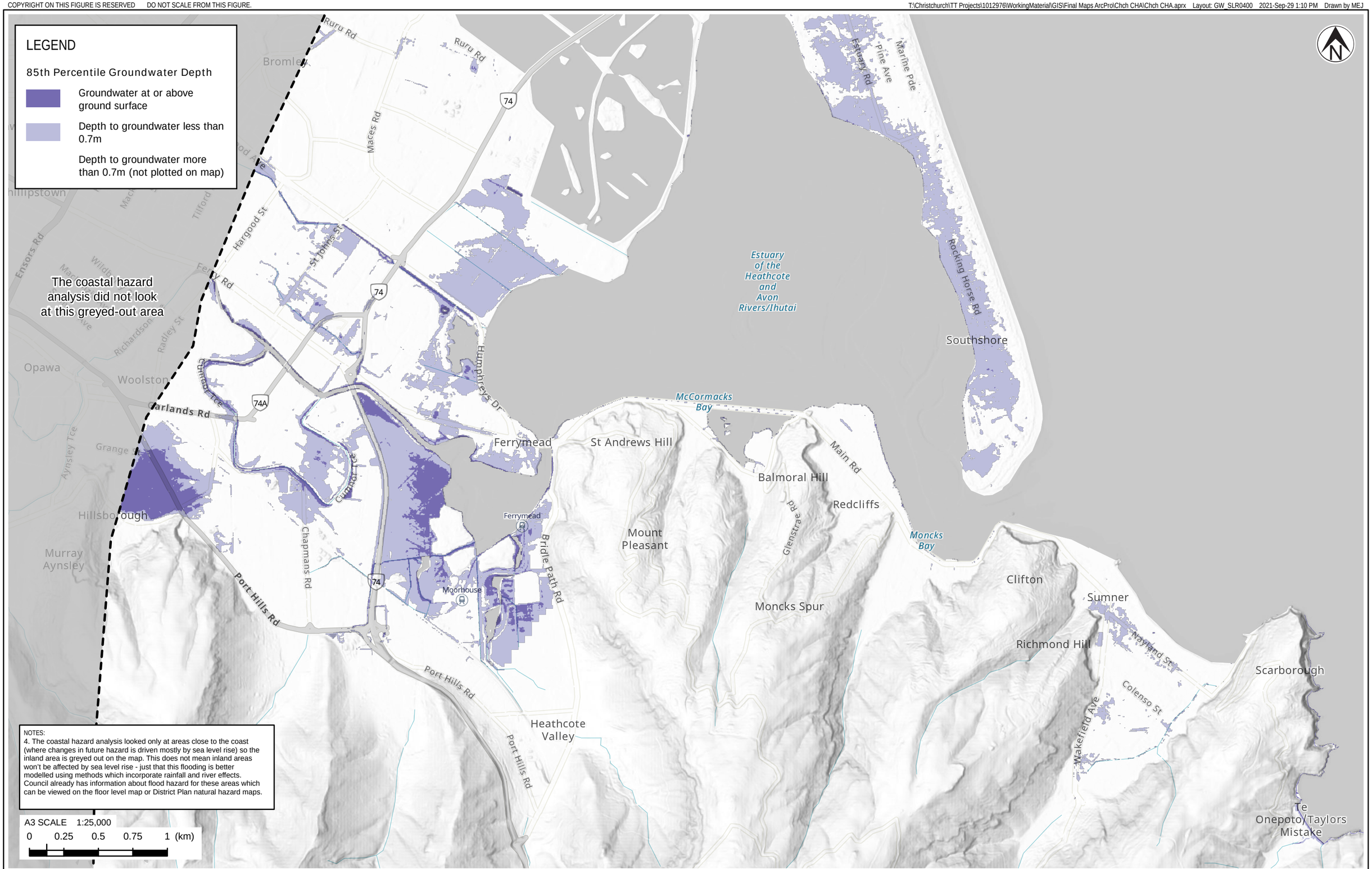

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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE

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

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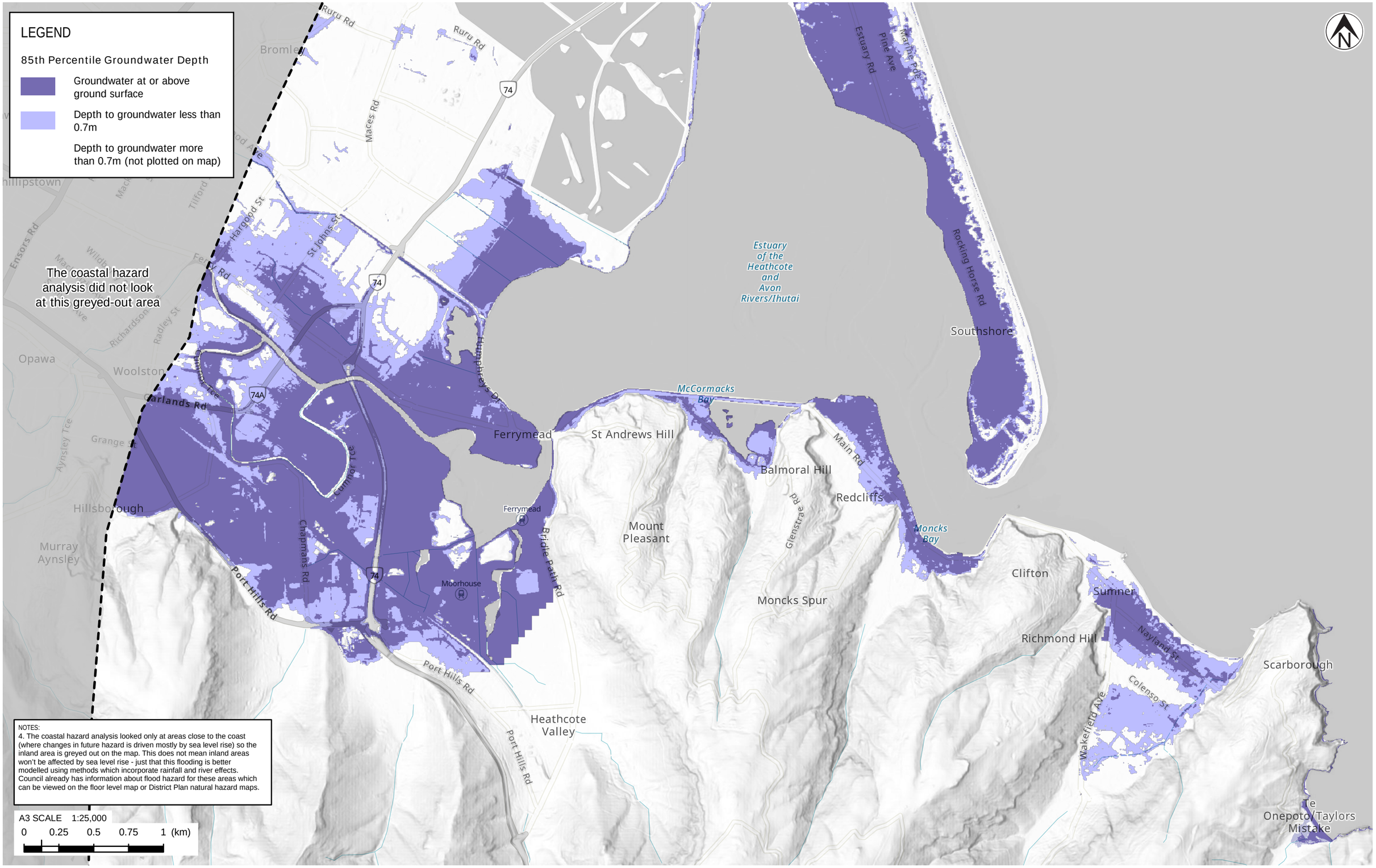

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 1.9M SEA LEVEL RISE

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

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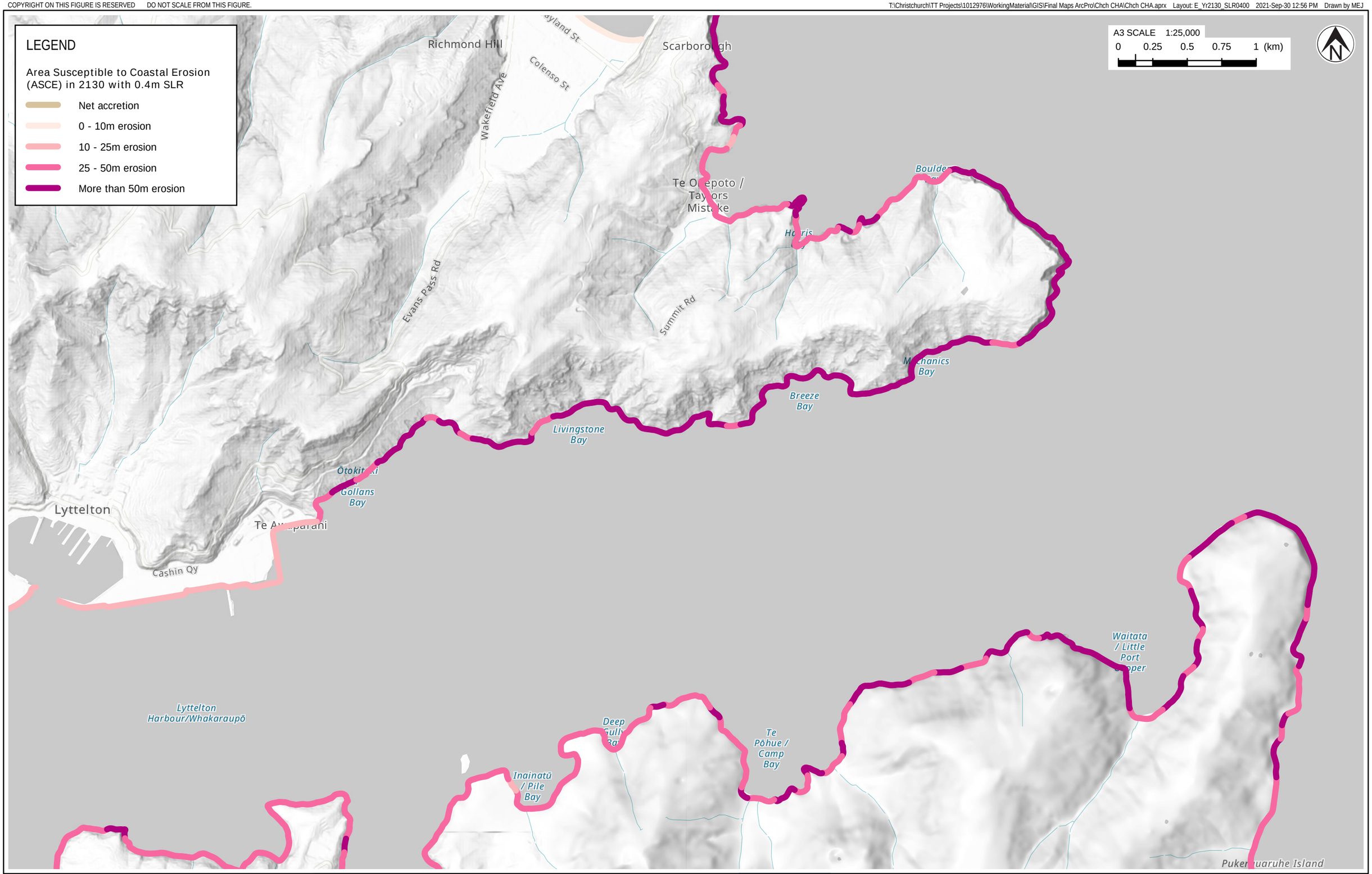

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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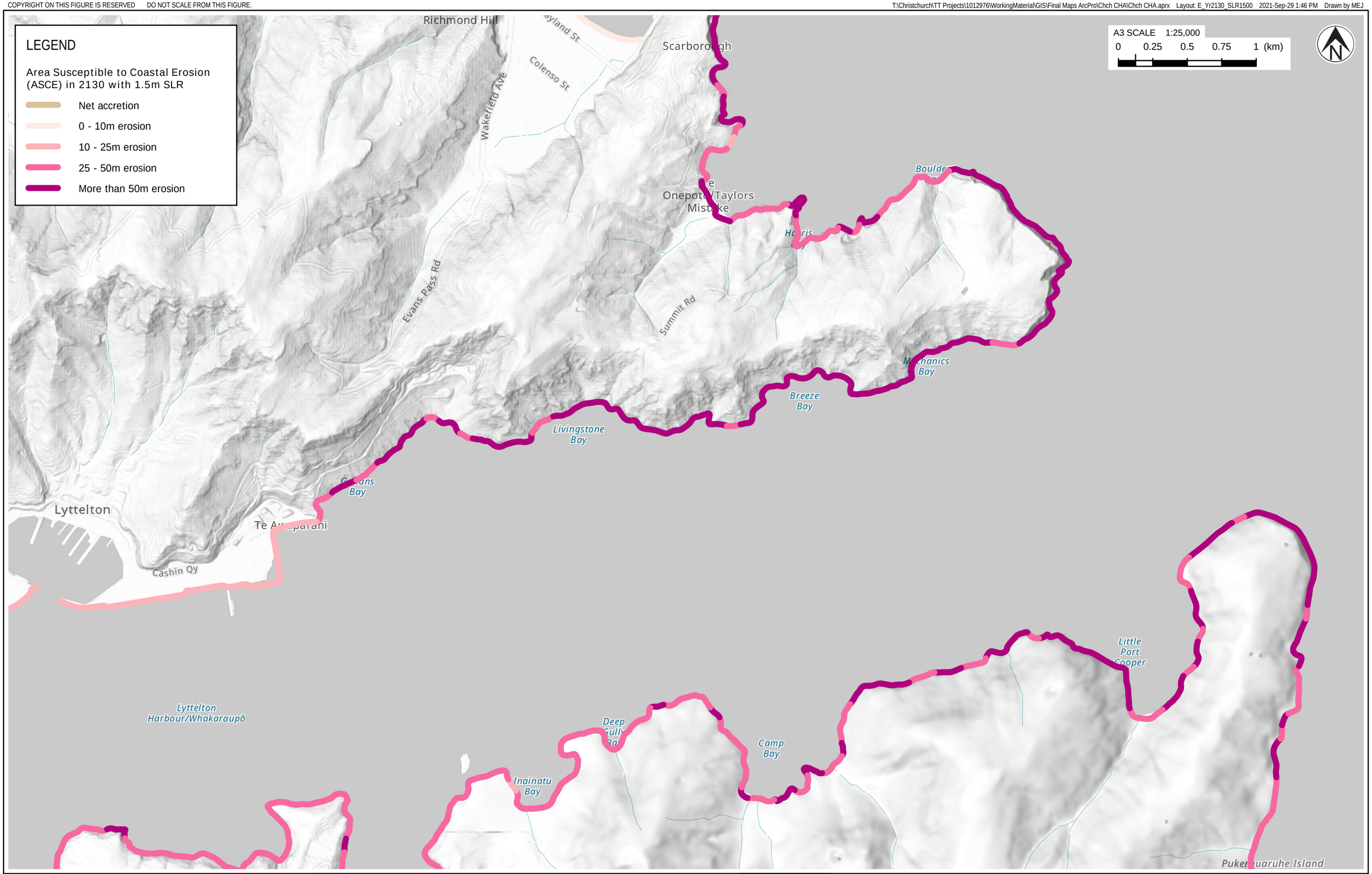

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PROJECT No.	1012976	
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CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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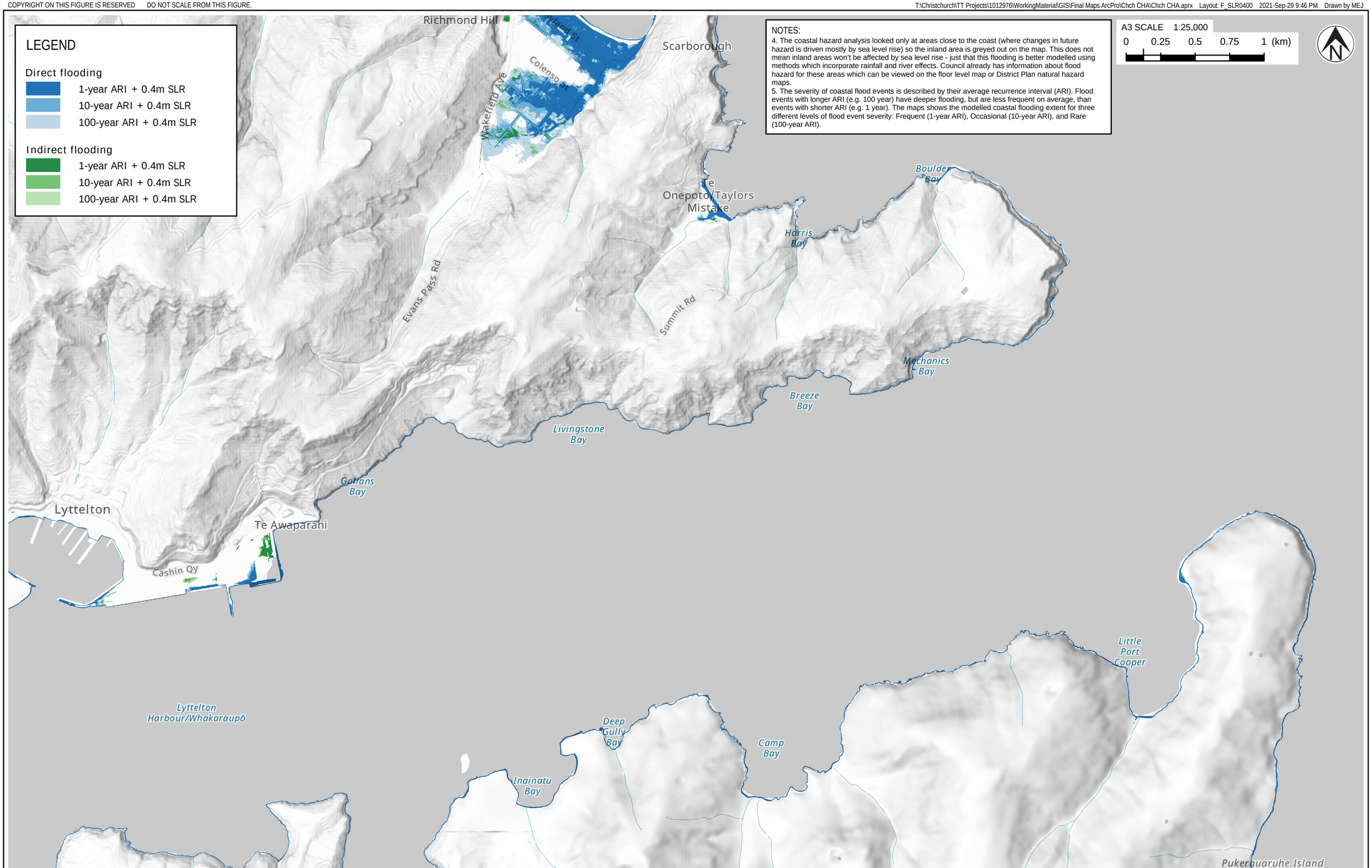

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1	Report issued	MEJ	PPK	29/09/21

LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

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

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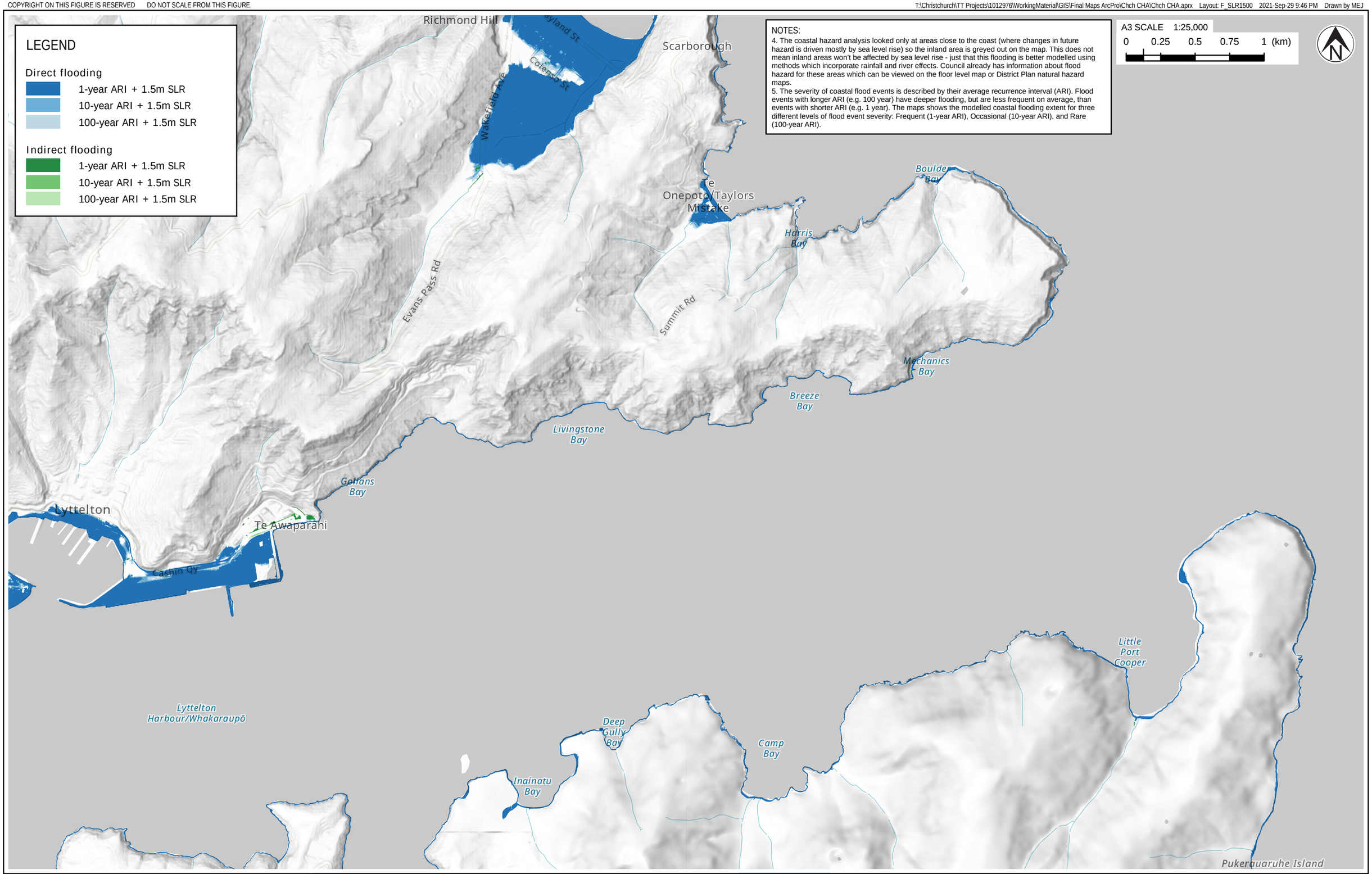

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PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE

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

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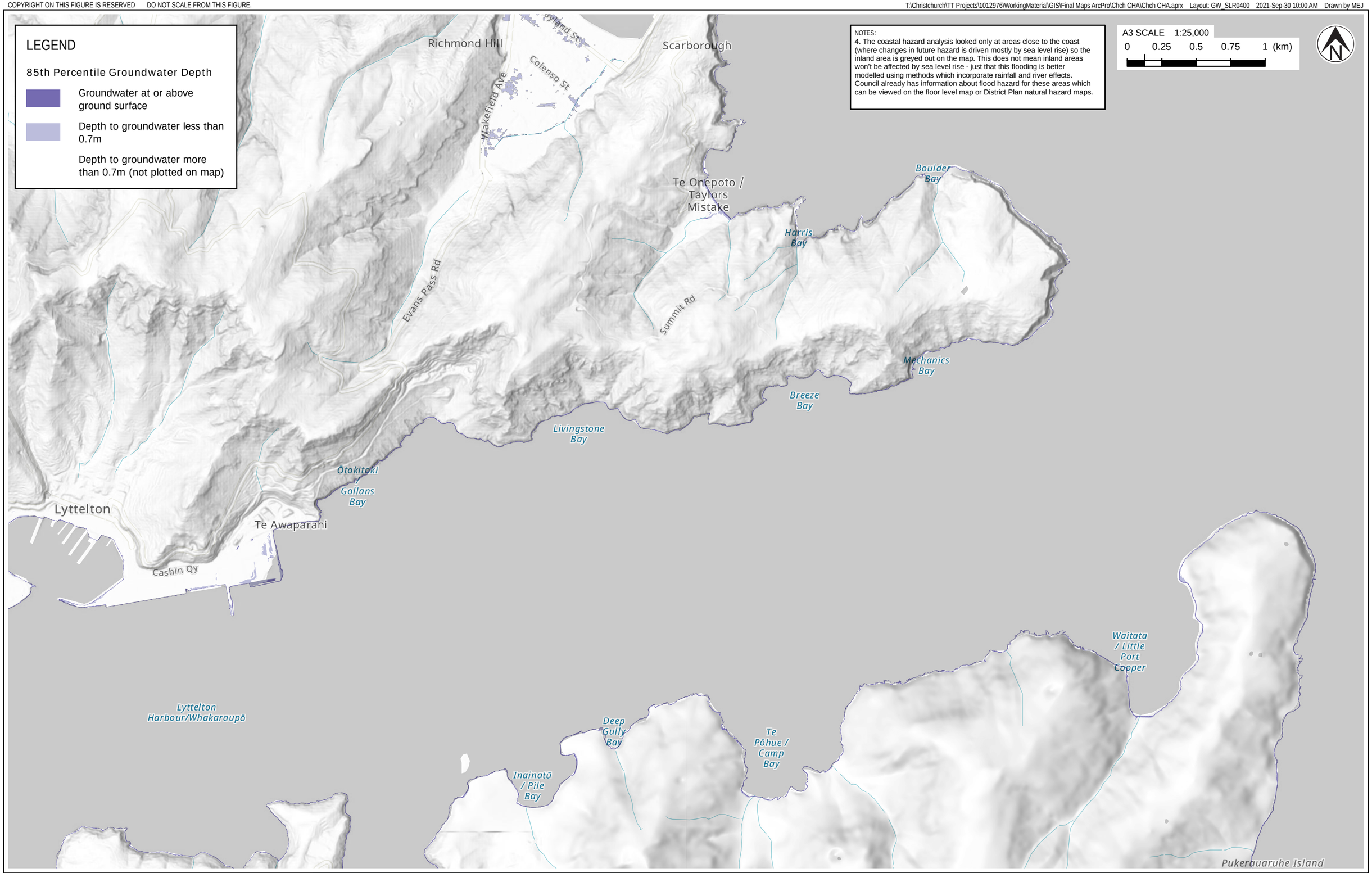

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PROJECT No.	1012976	
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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE

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

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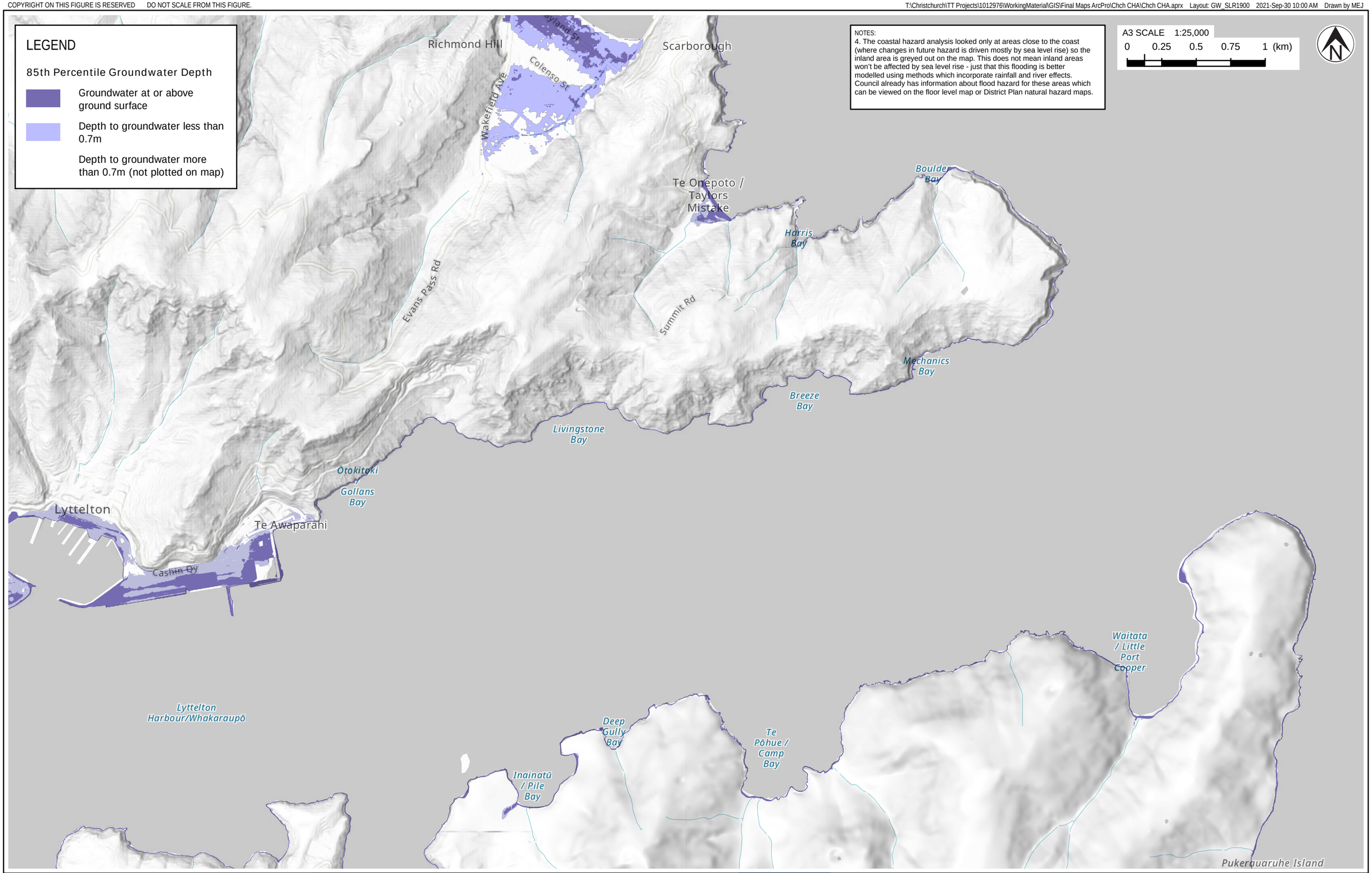

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 1.9M SEA LEVEL RISE

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

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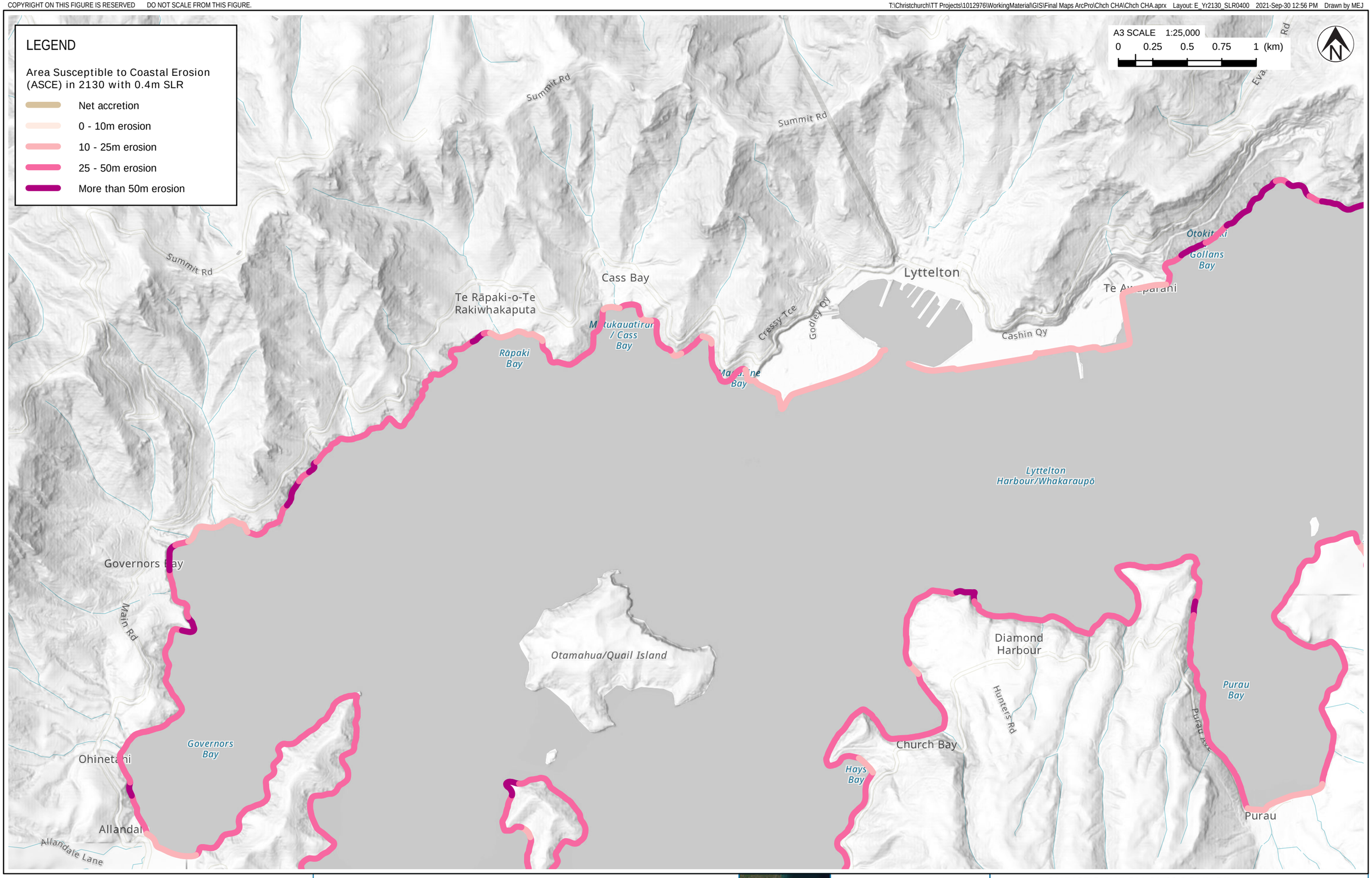

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DESIGNED	MEJ	SEP.21
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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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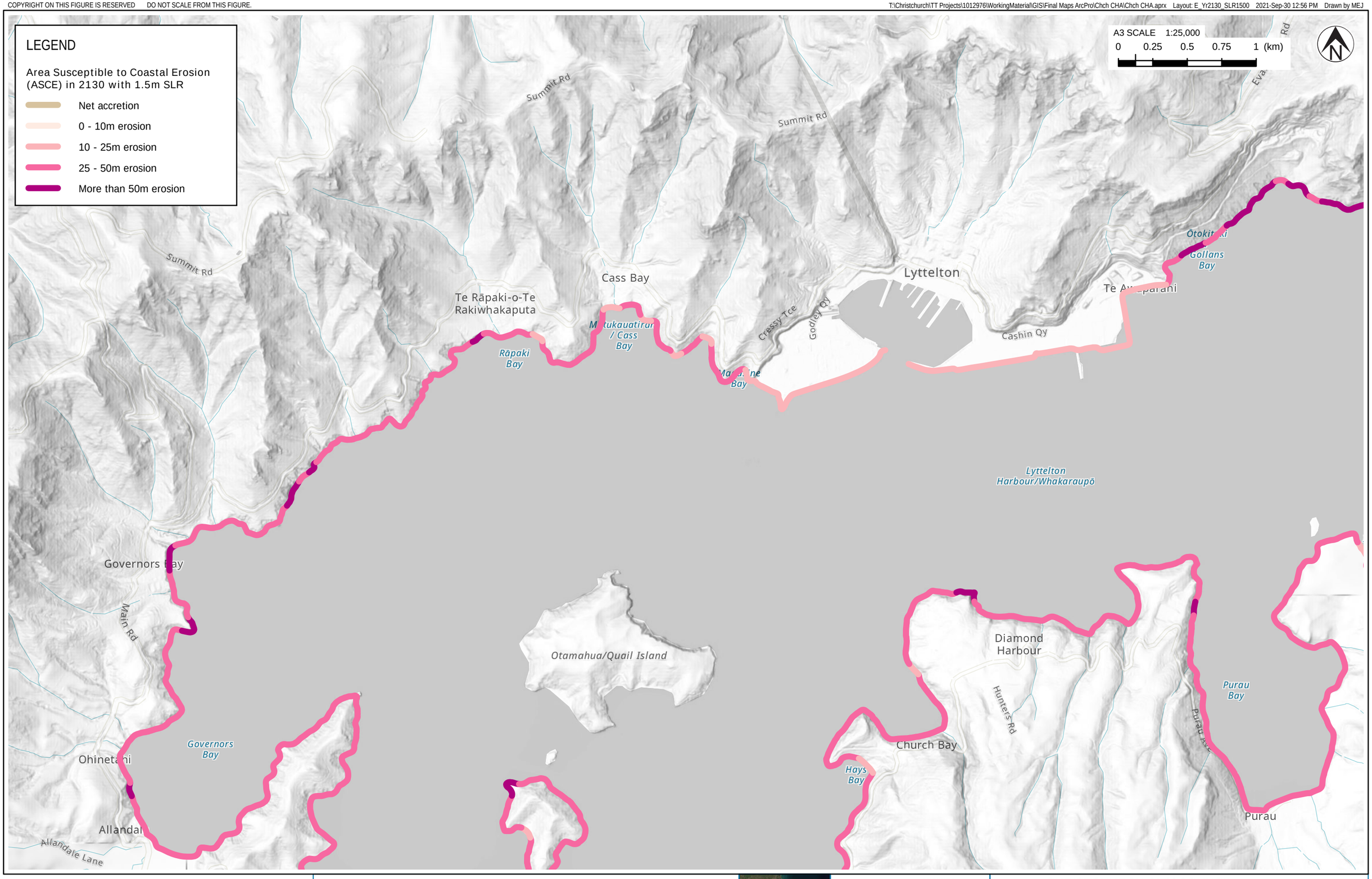

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PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 1.5M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 6B REV 1

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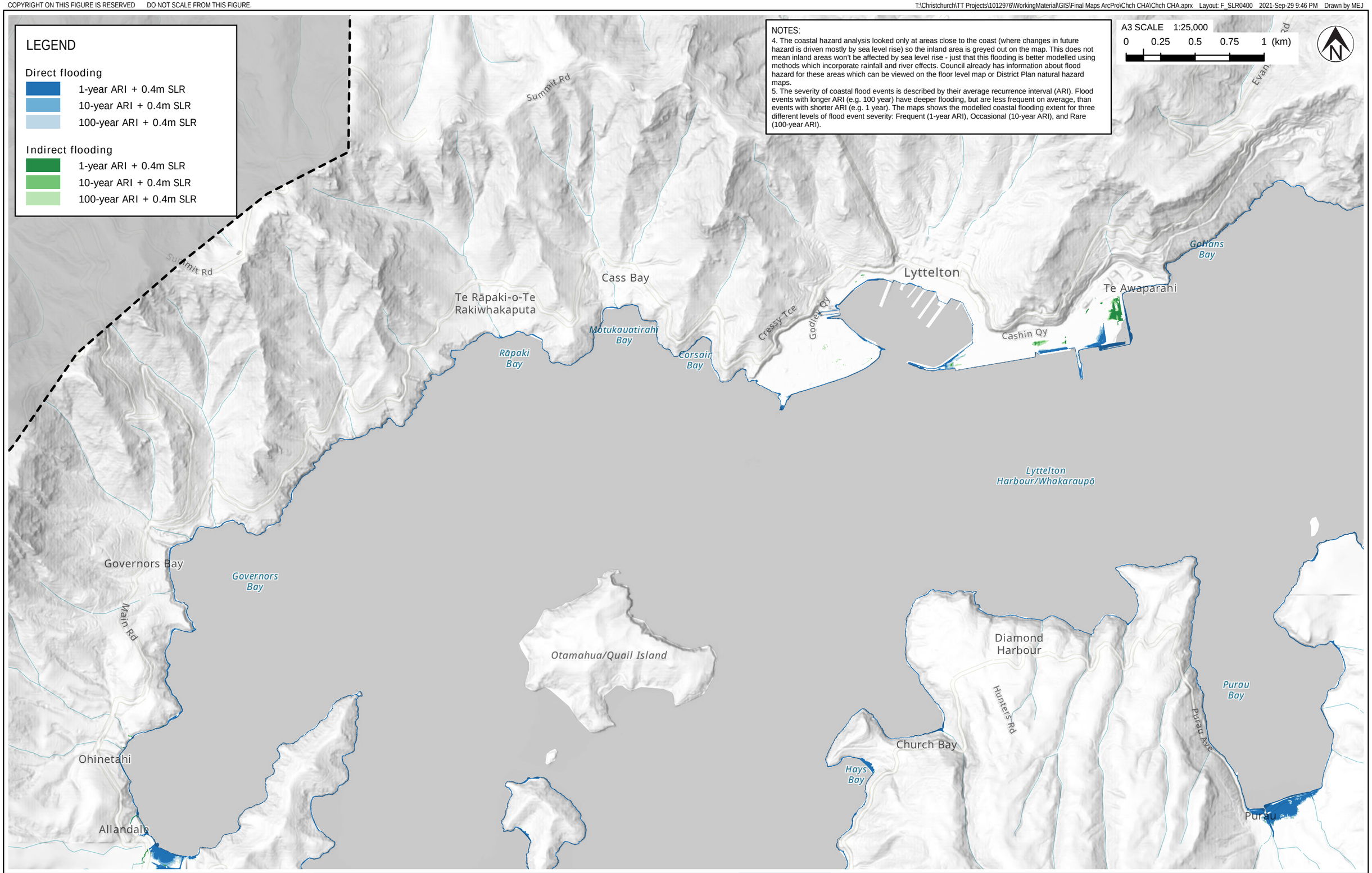

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

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

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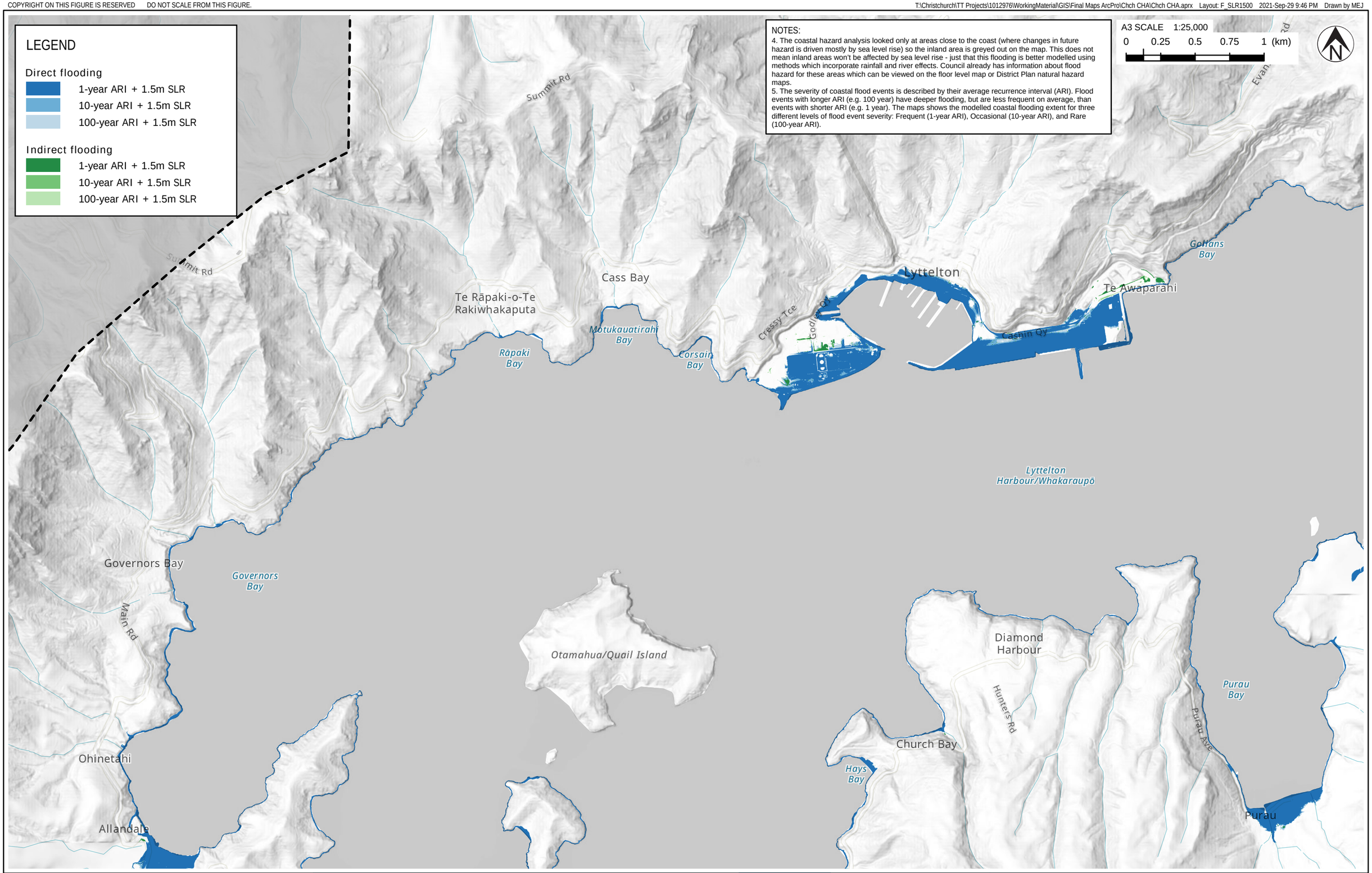

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SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:25,000 FIG No. FIGURE 6D REV 1

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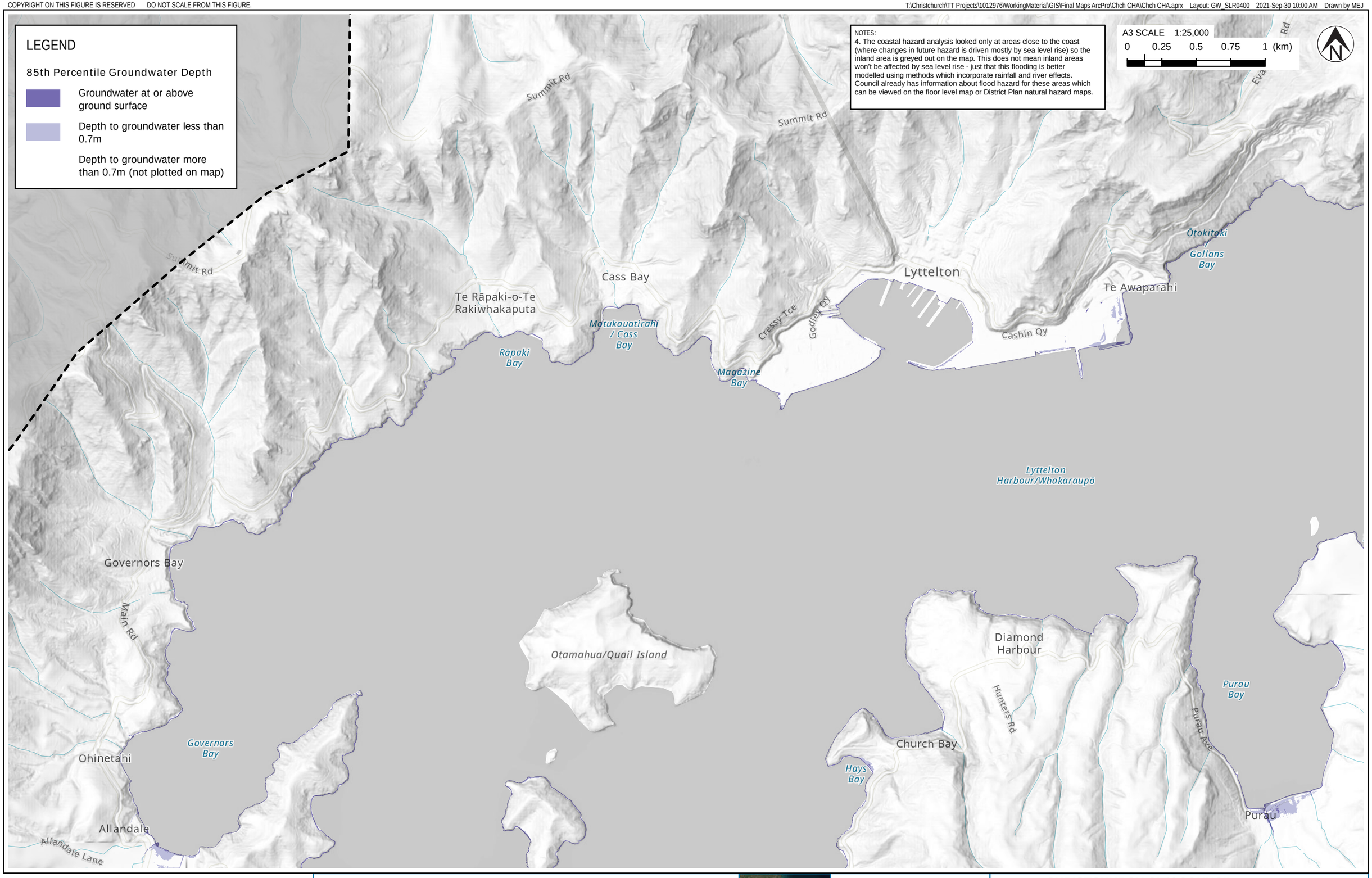

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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE

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

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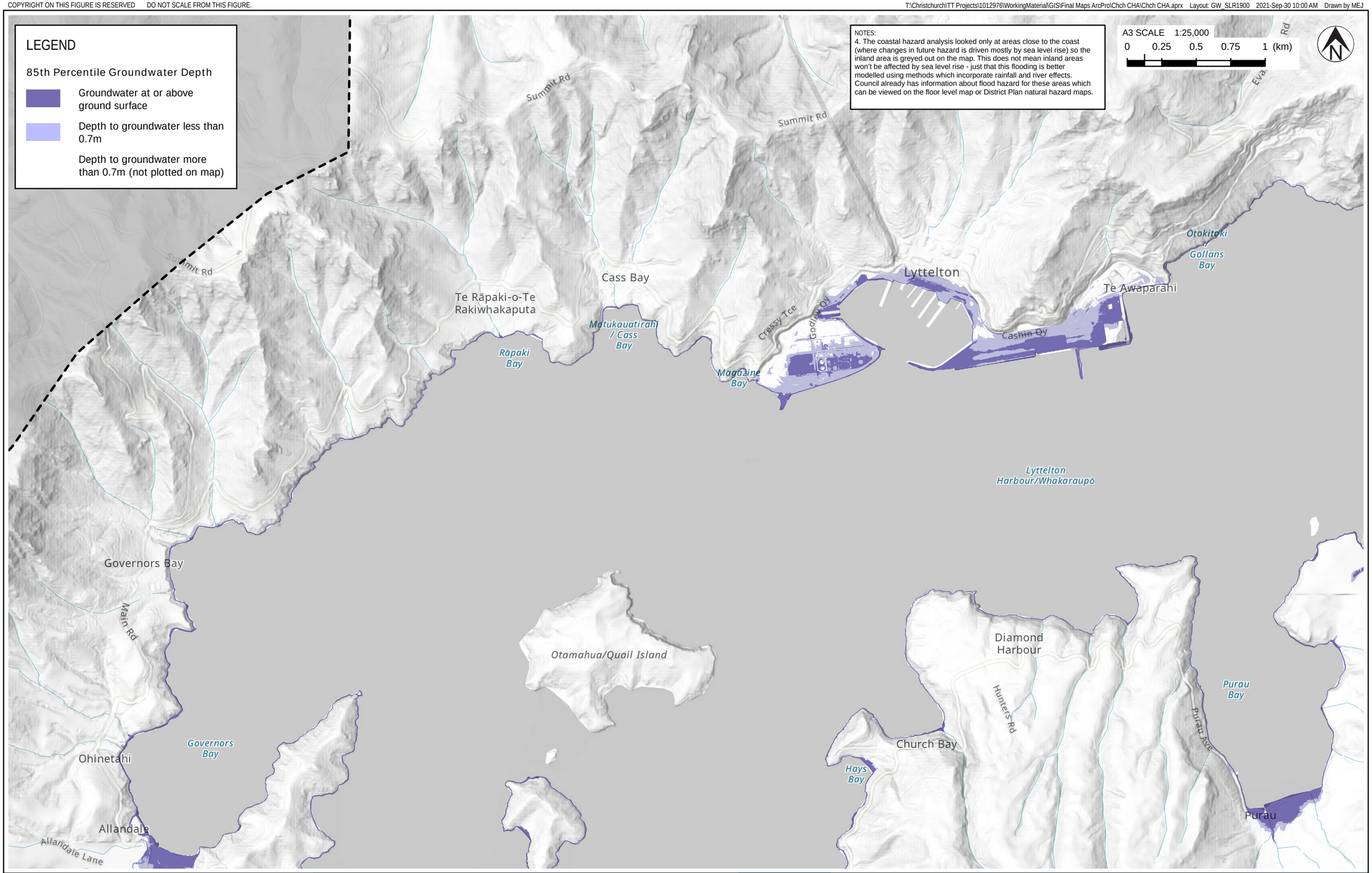

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 1.9M SEA LEVEL RISE

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

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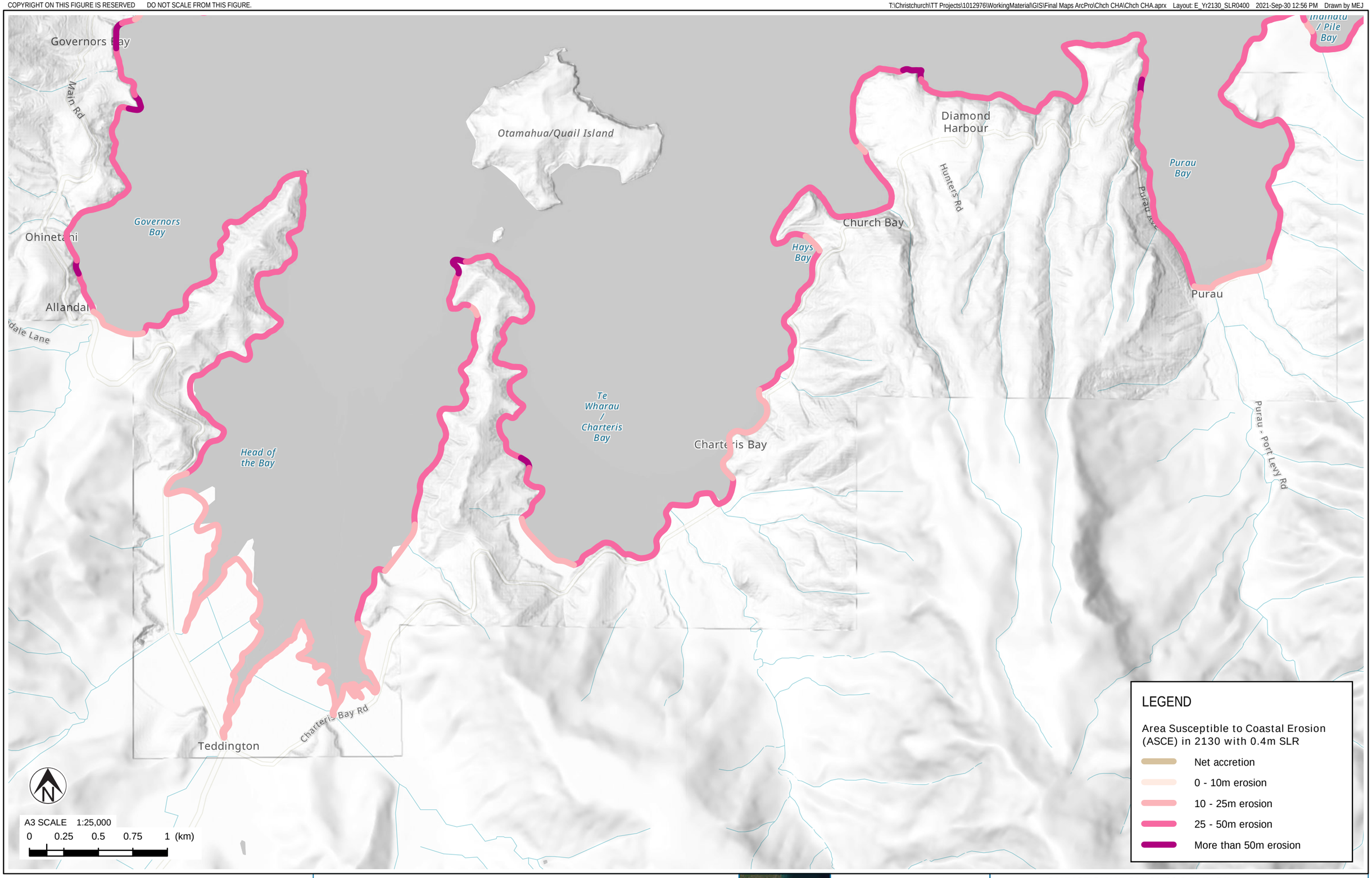

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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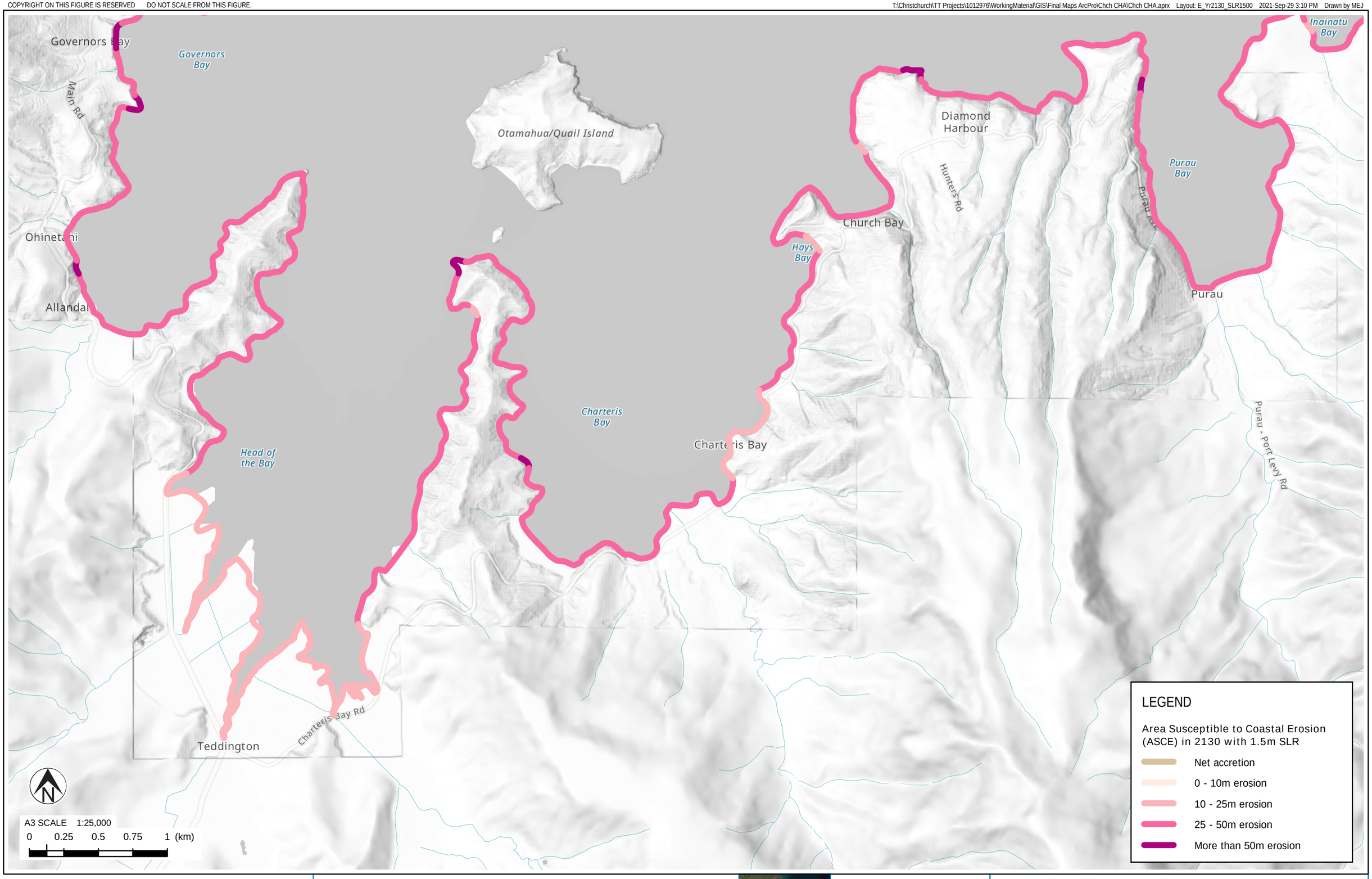

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

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

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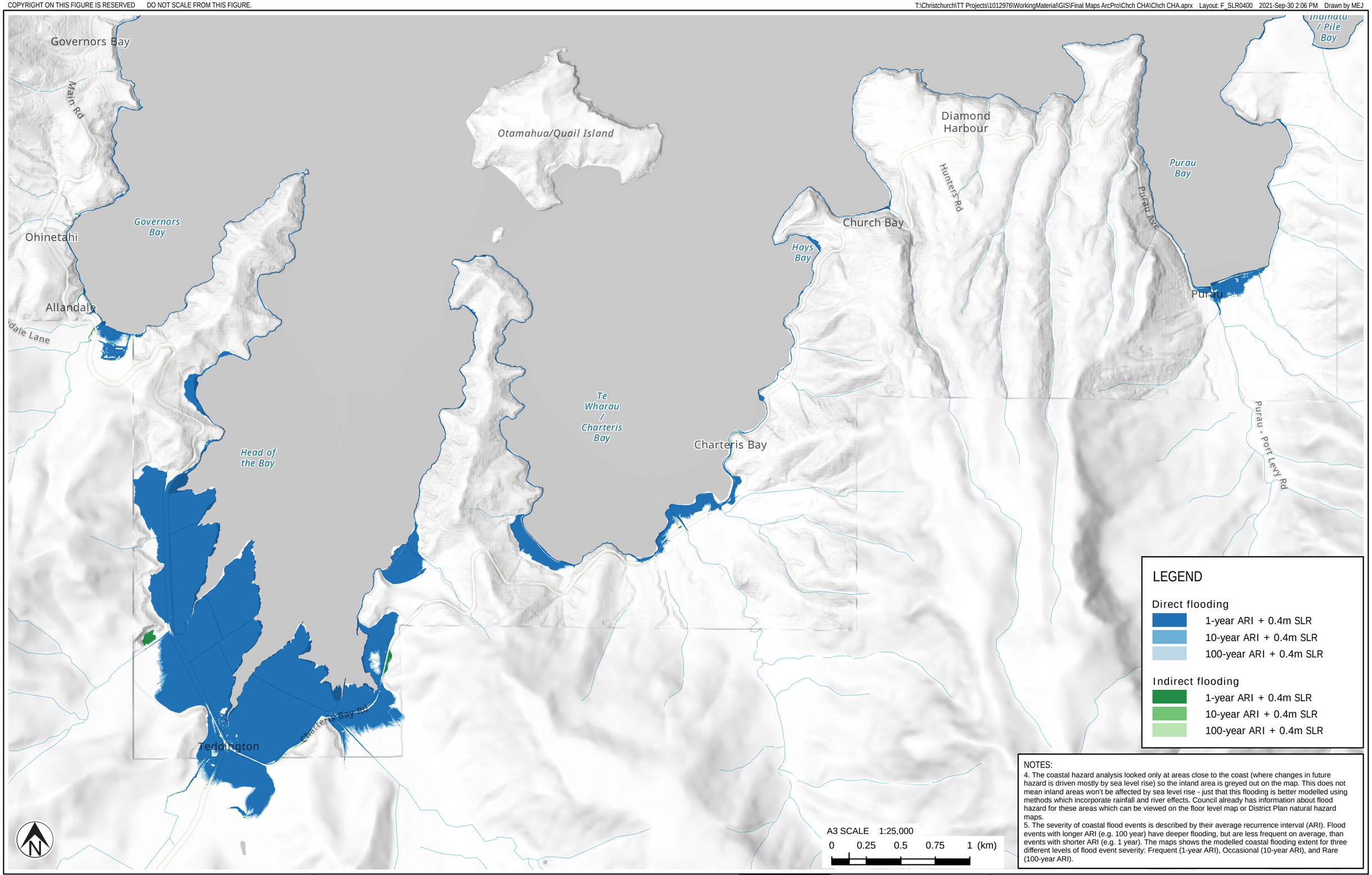

NOTES:
4. The coastal hazard analysis looked only at areas close to the coast (where changes in future hazard is driven mostly by sea level rise) so the inland area is greyed out on the map. This does not mean inland areas won't be affected by sea level rise - just that this flooding is better modelled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

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

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

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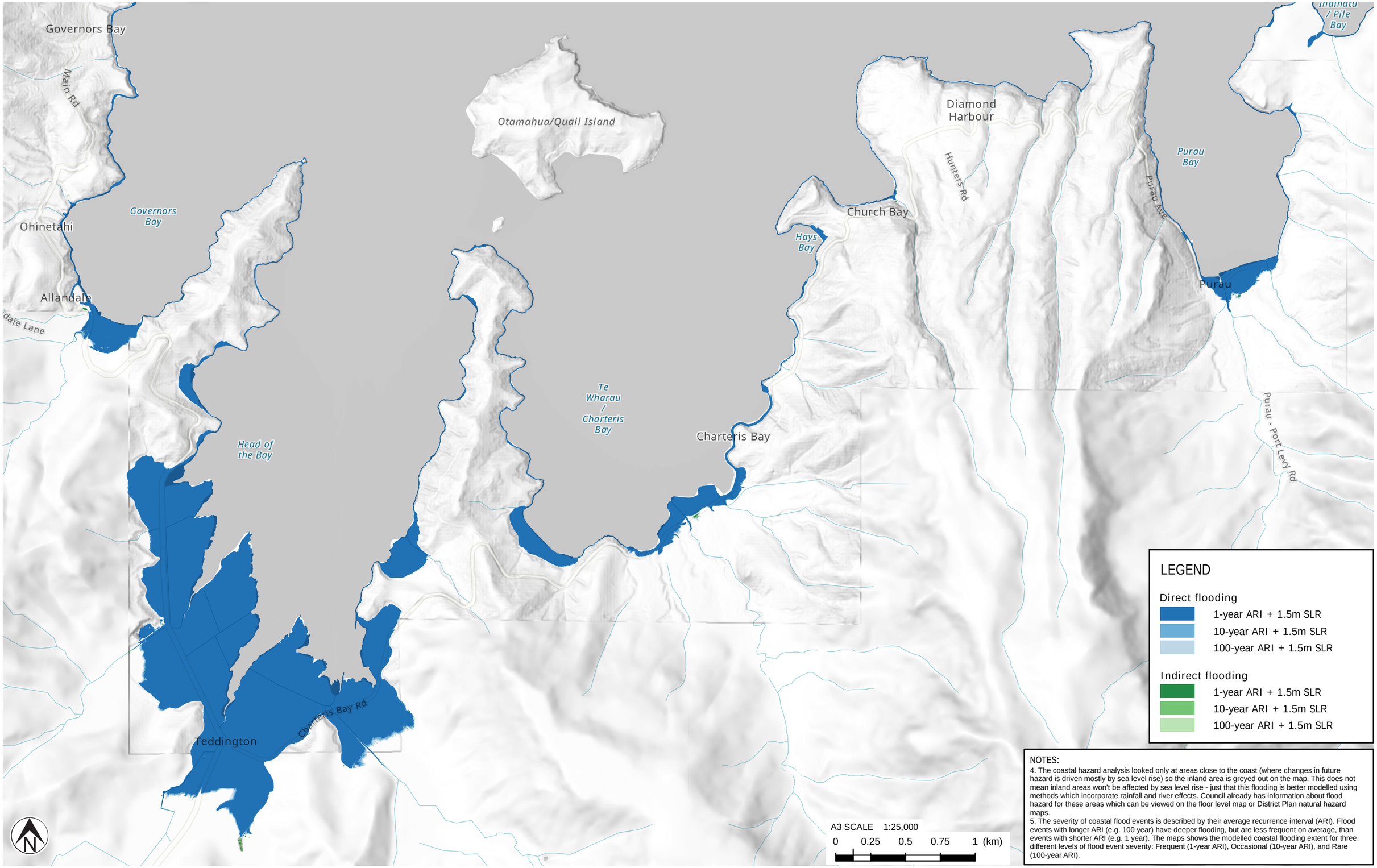

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE

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

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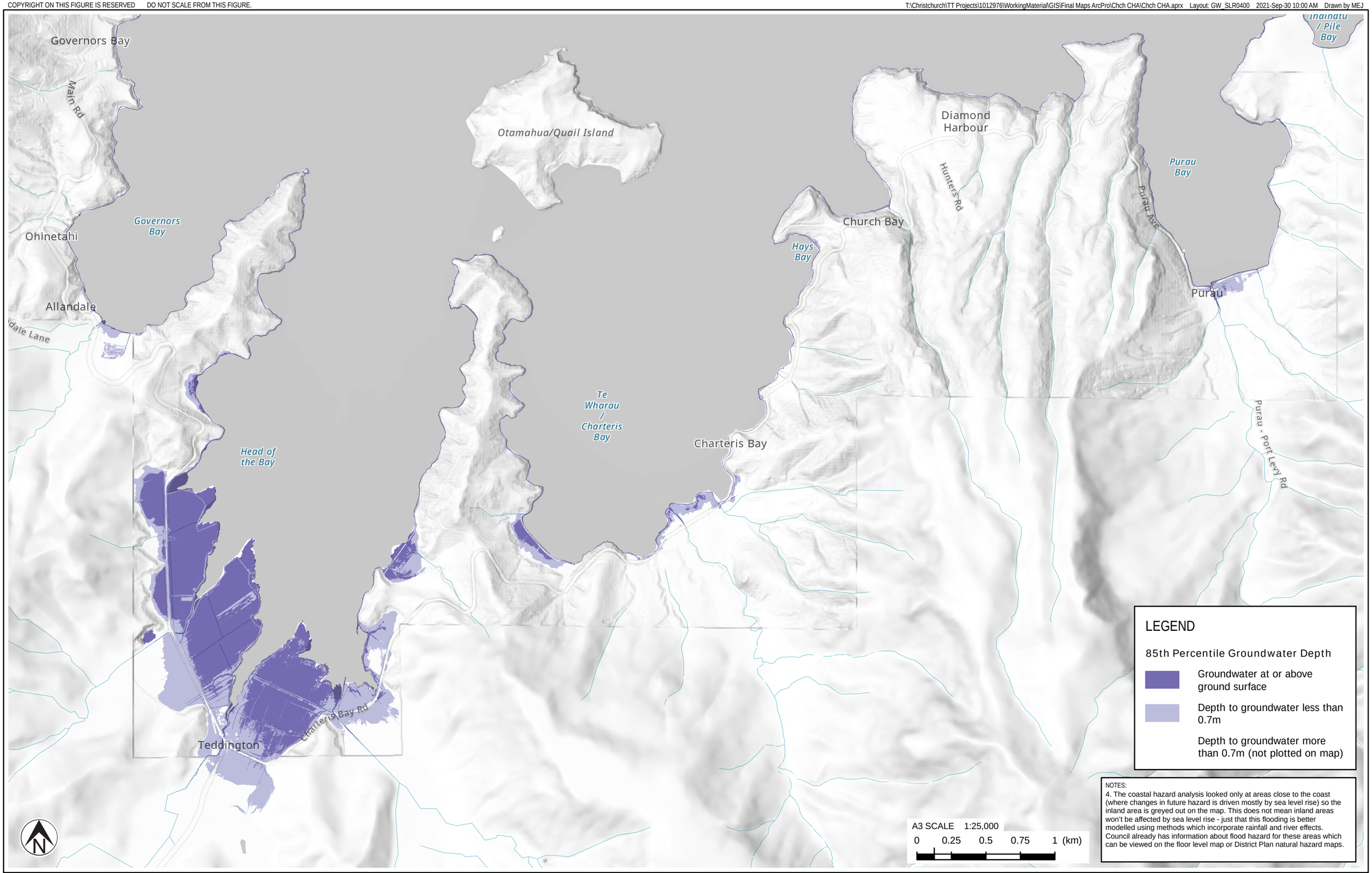

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

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

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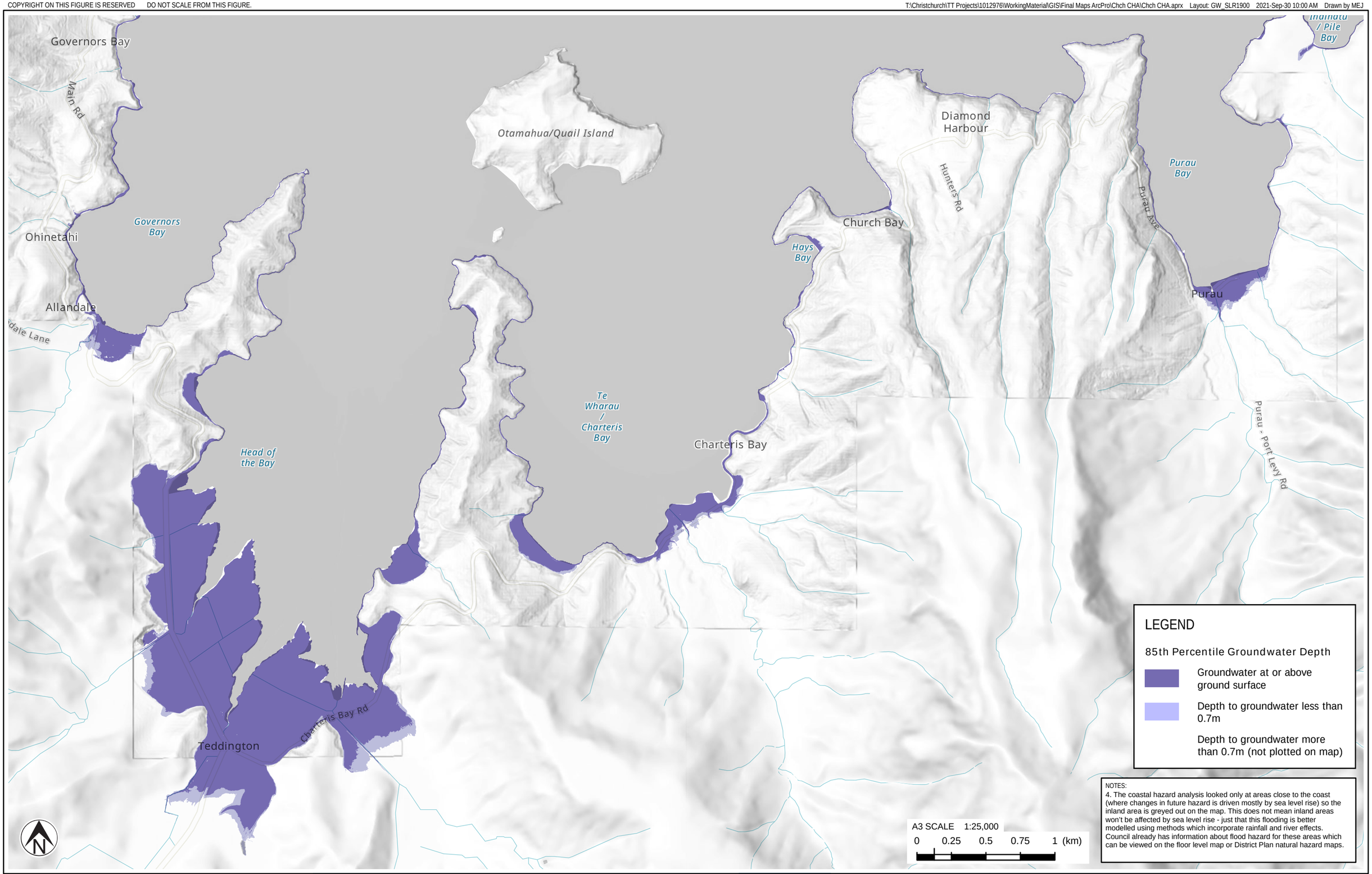

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

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

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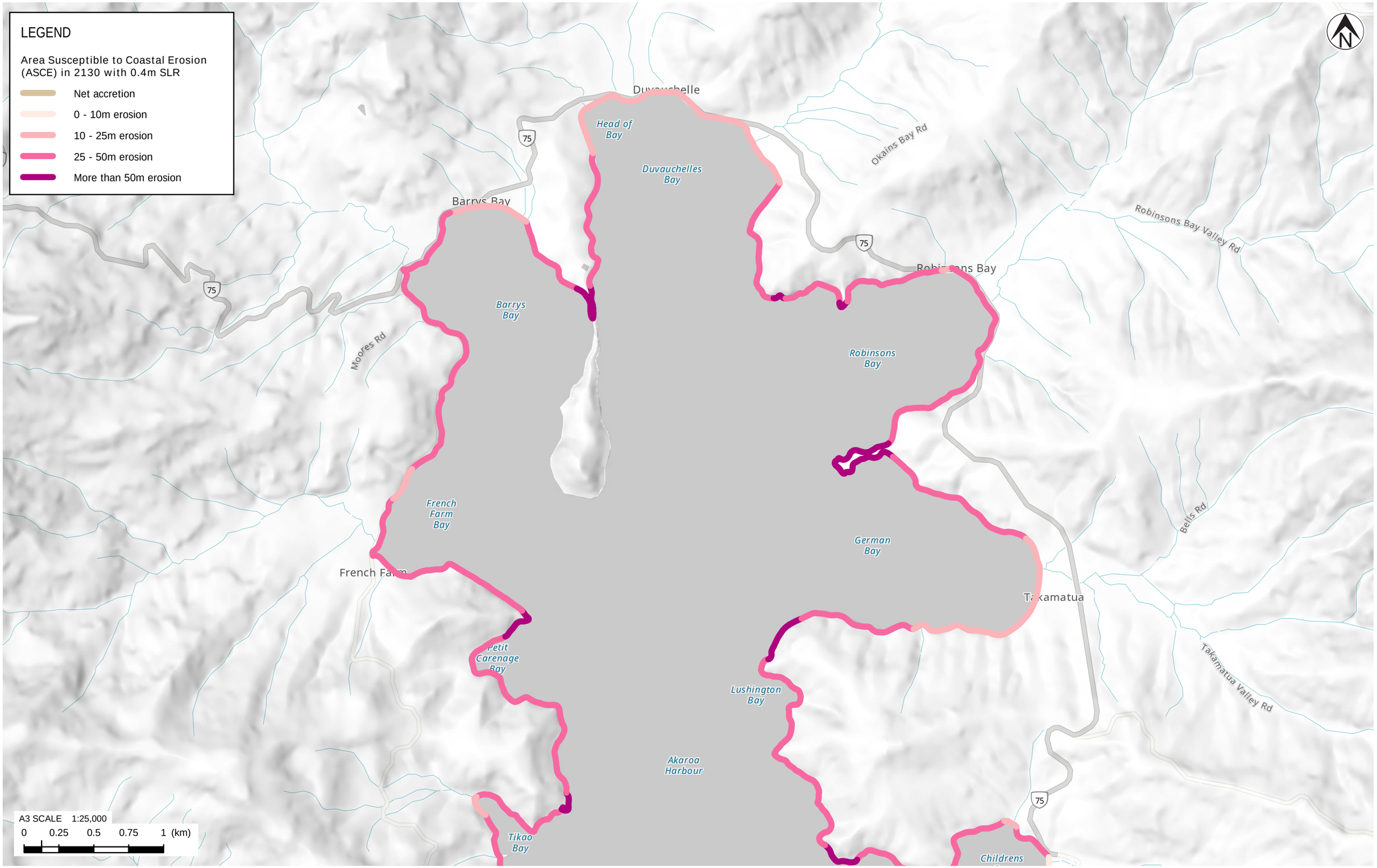

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CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:25,000 FIG No. FIGURE 8A REV 1

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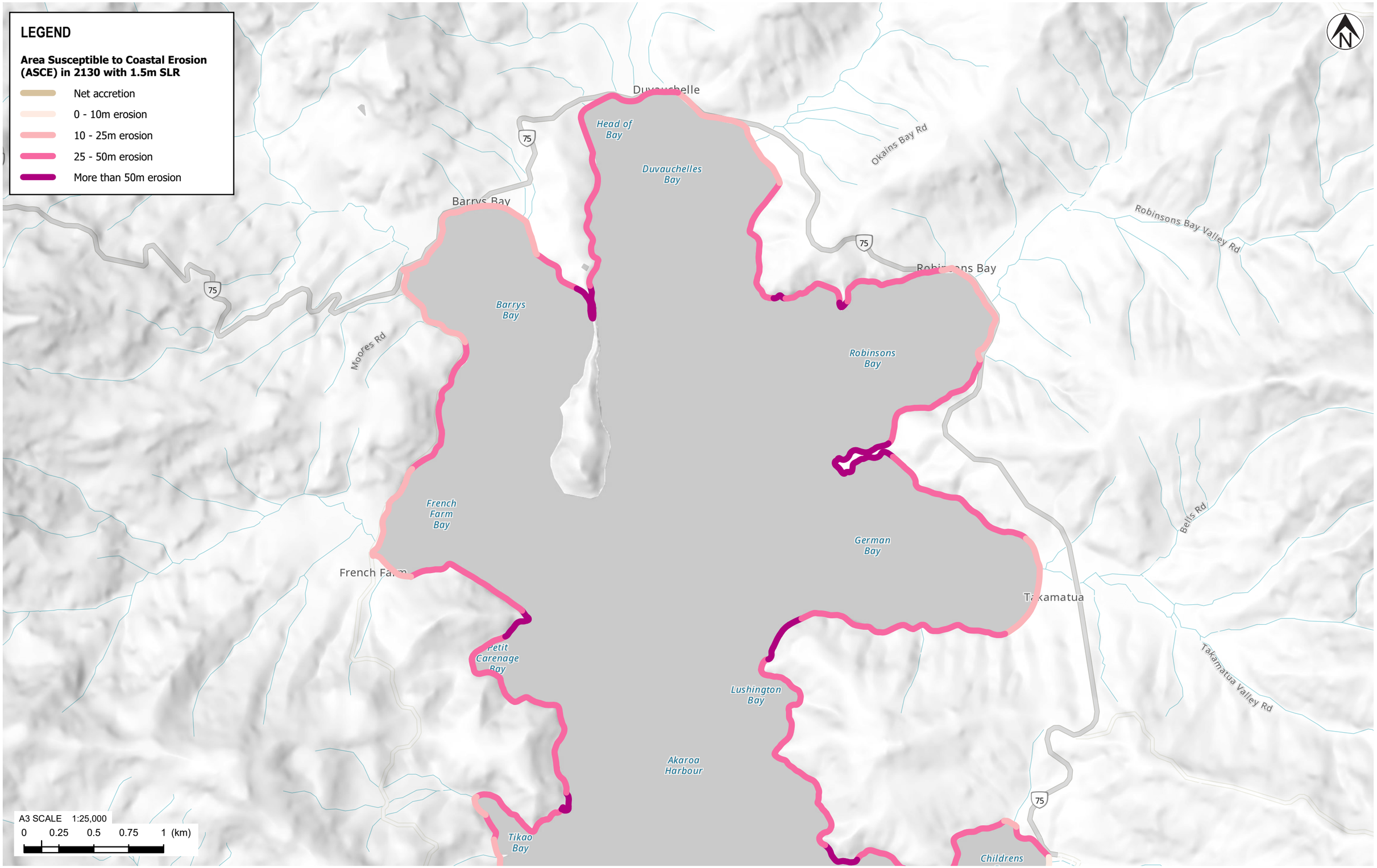

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:25,000 FIG No. FIGURE 8B REV 1

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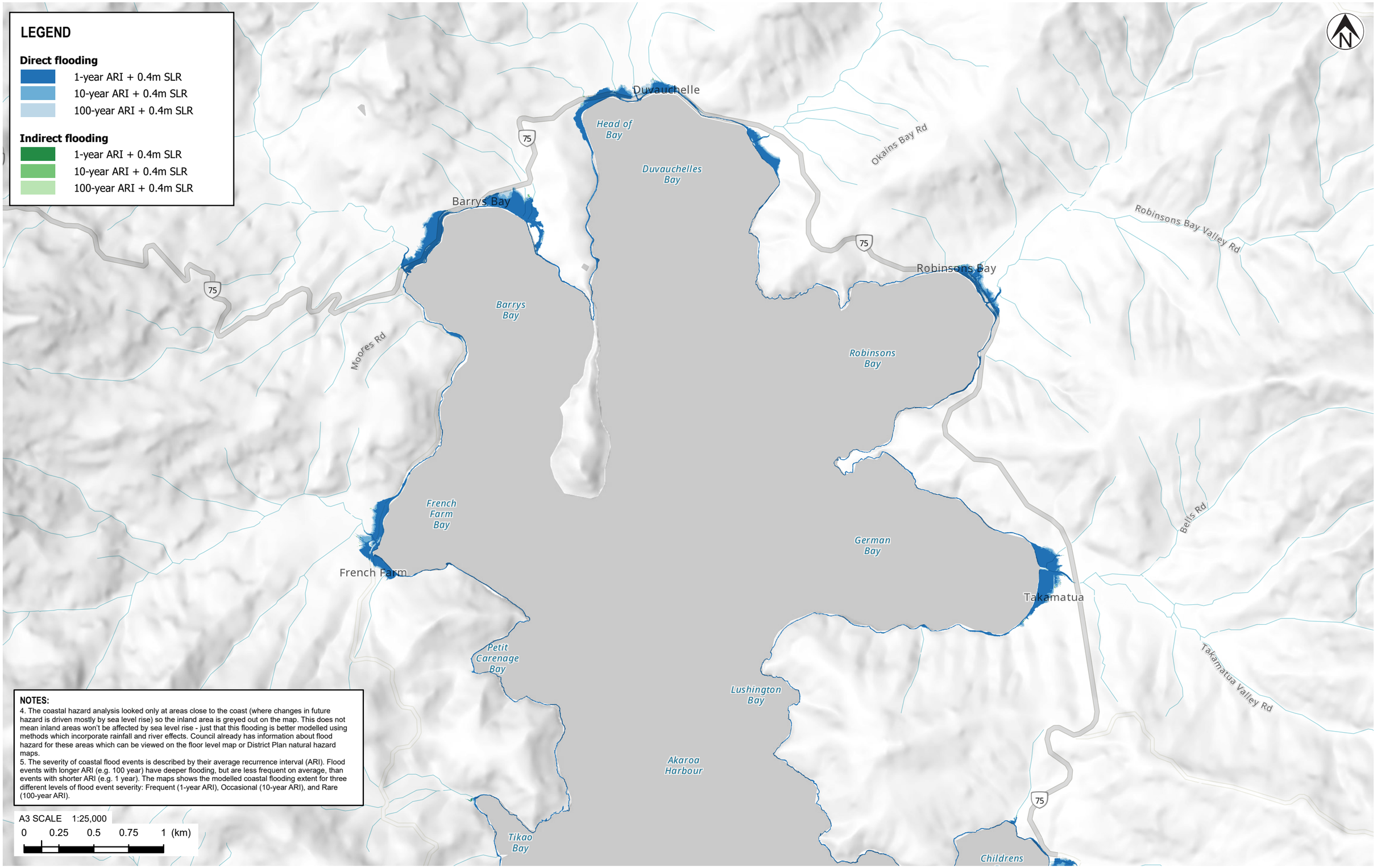

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A3 SCALE 1:25,000

0 0.25 0.5 0.75 1 (km)

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 8C REV 1

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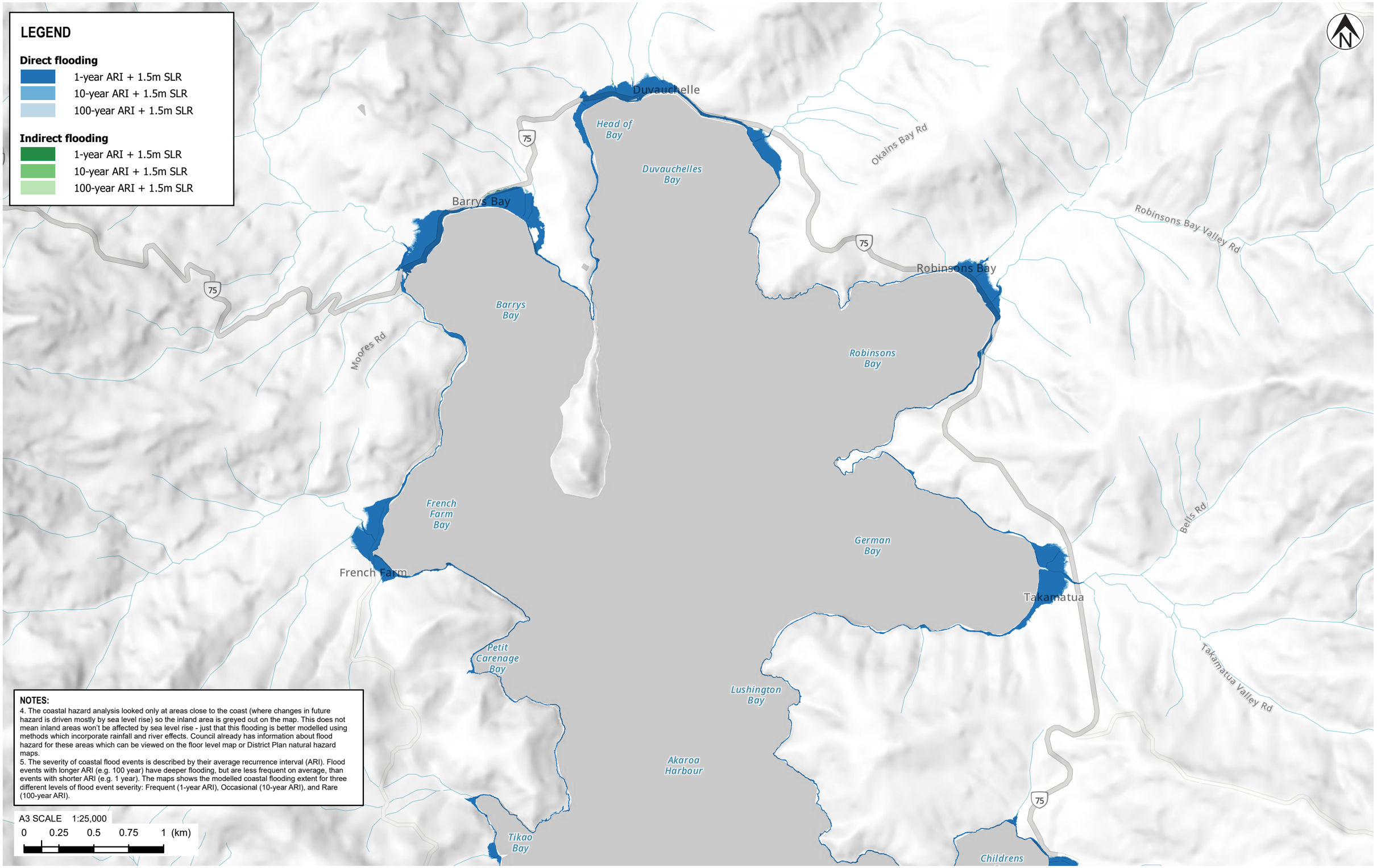

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PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE

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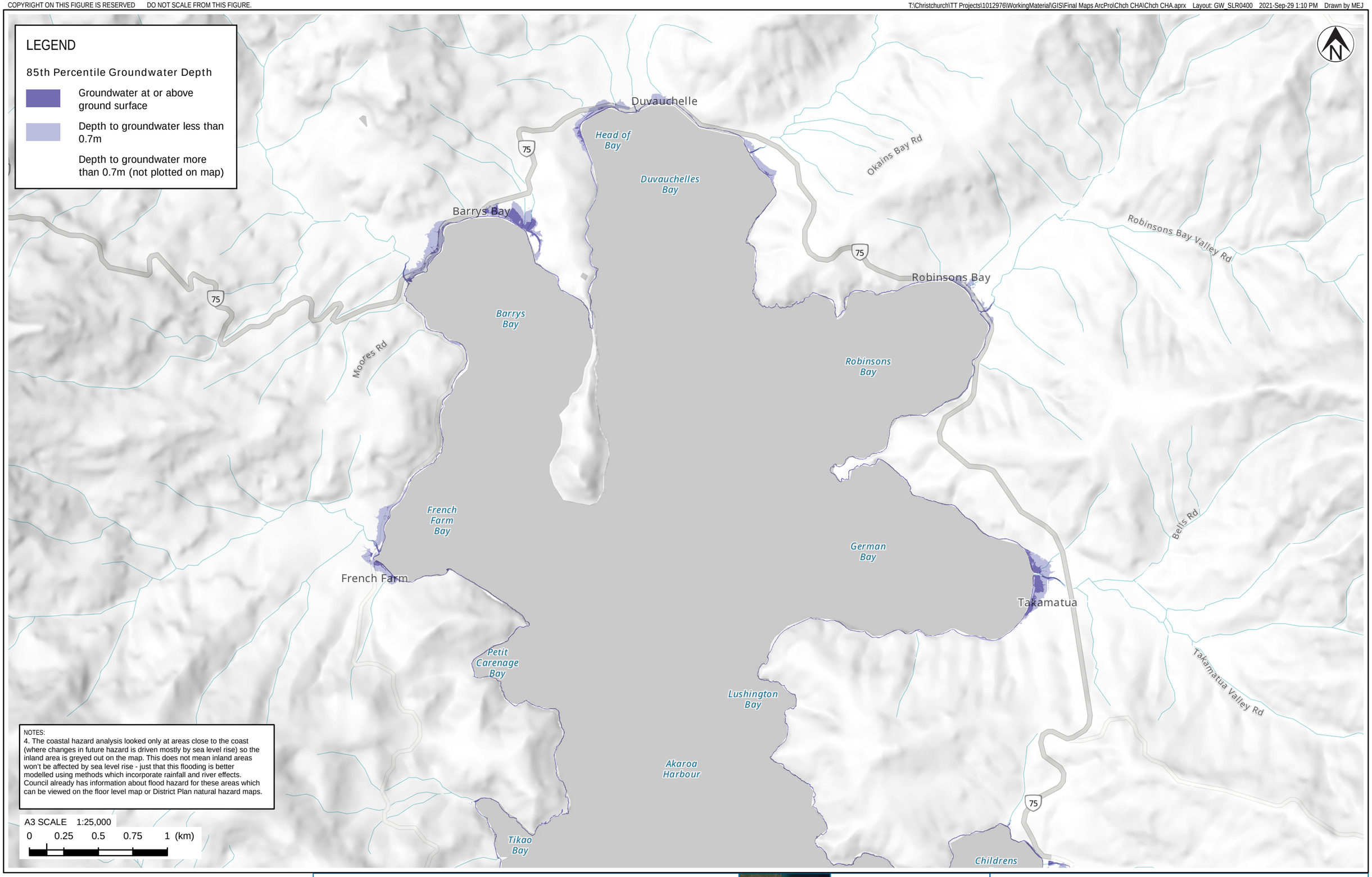

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

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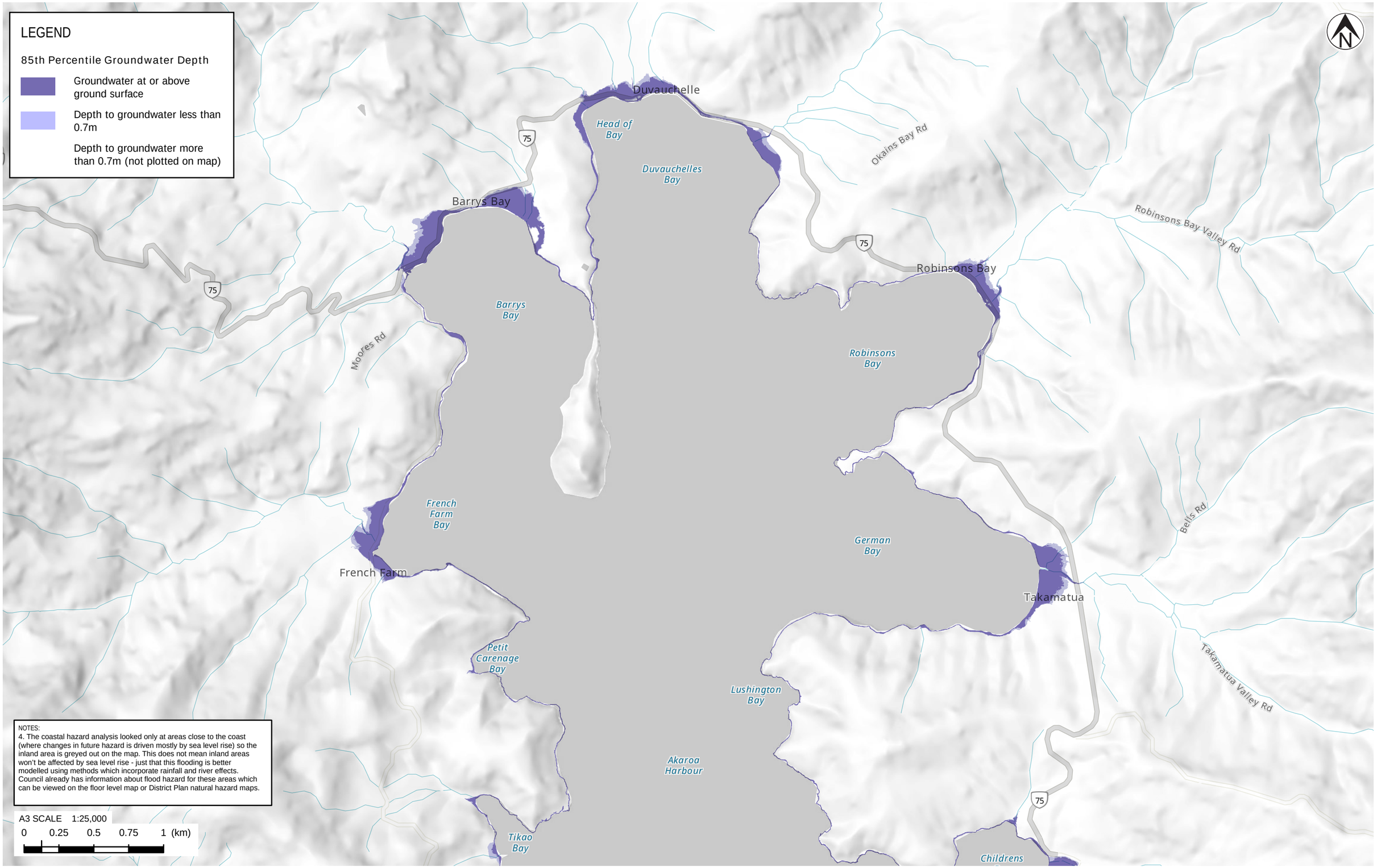

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PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 1.9M SEA LEVEL RISE

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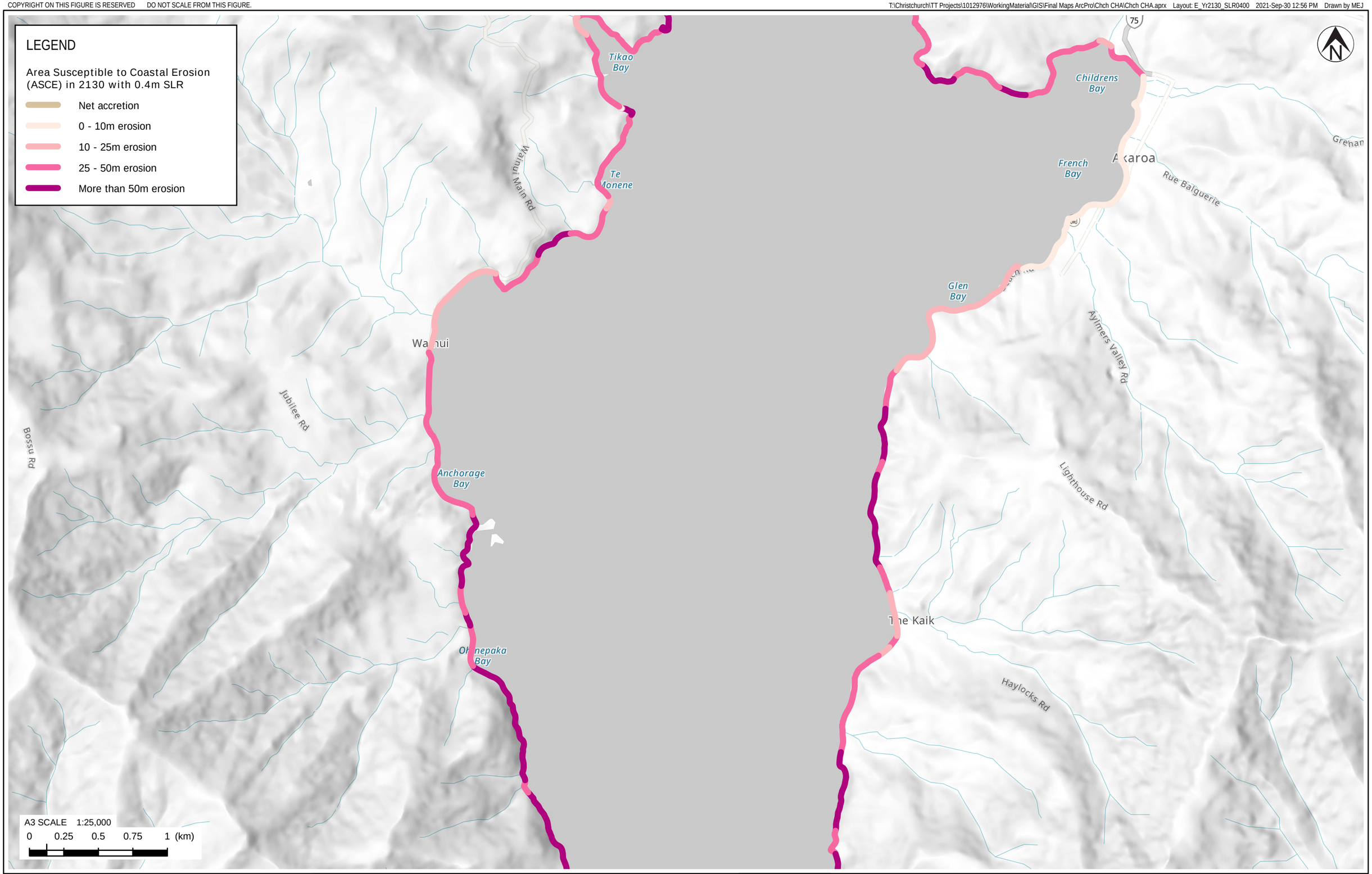

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DRAWN	MEJ	SEP.21
CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:25,000 FIG No. FIGURE 9A REV 1

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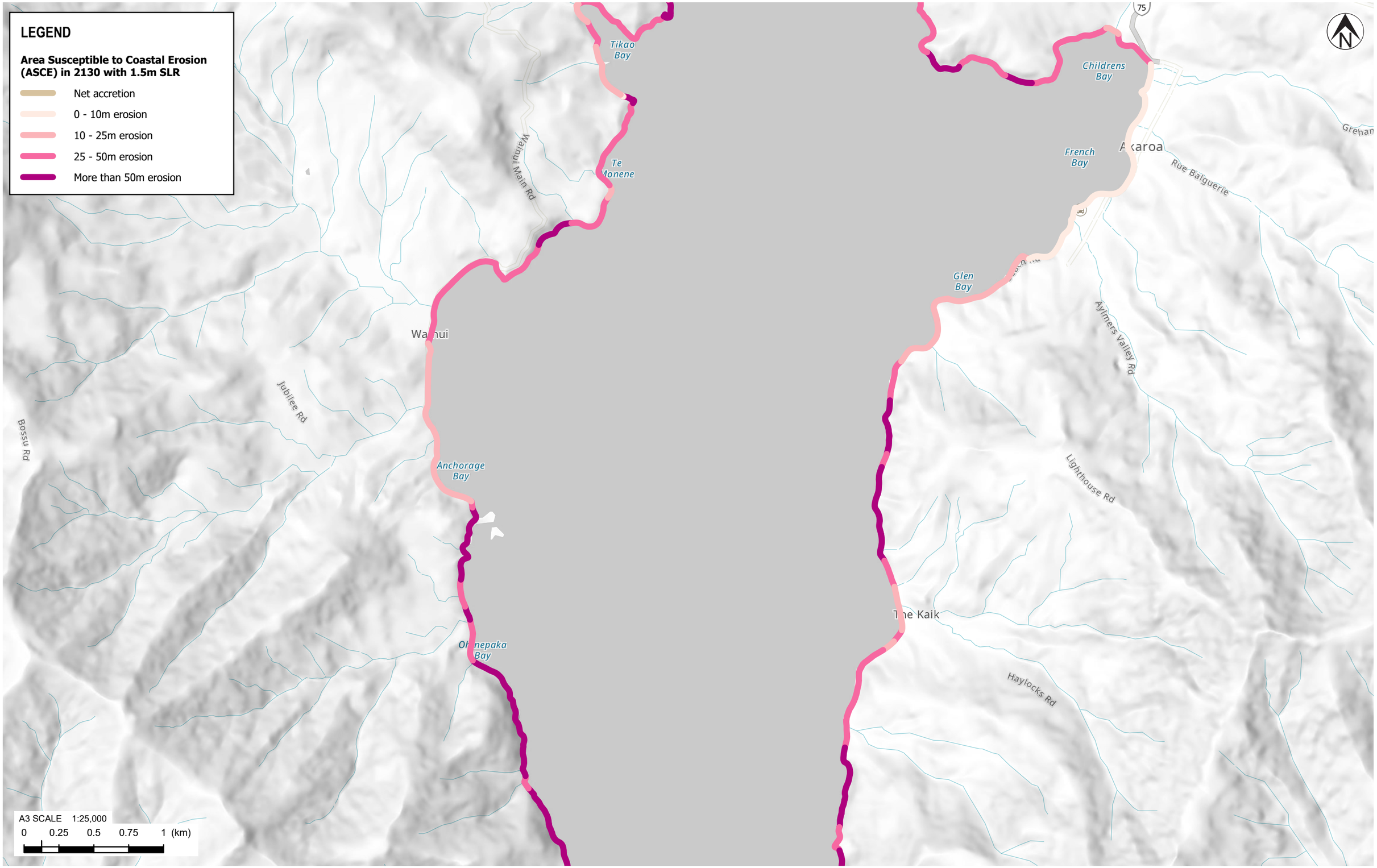

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LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:25,000 FIG No. FIGURE 9B REV 1

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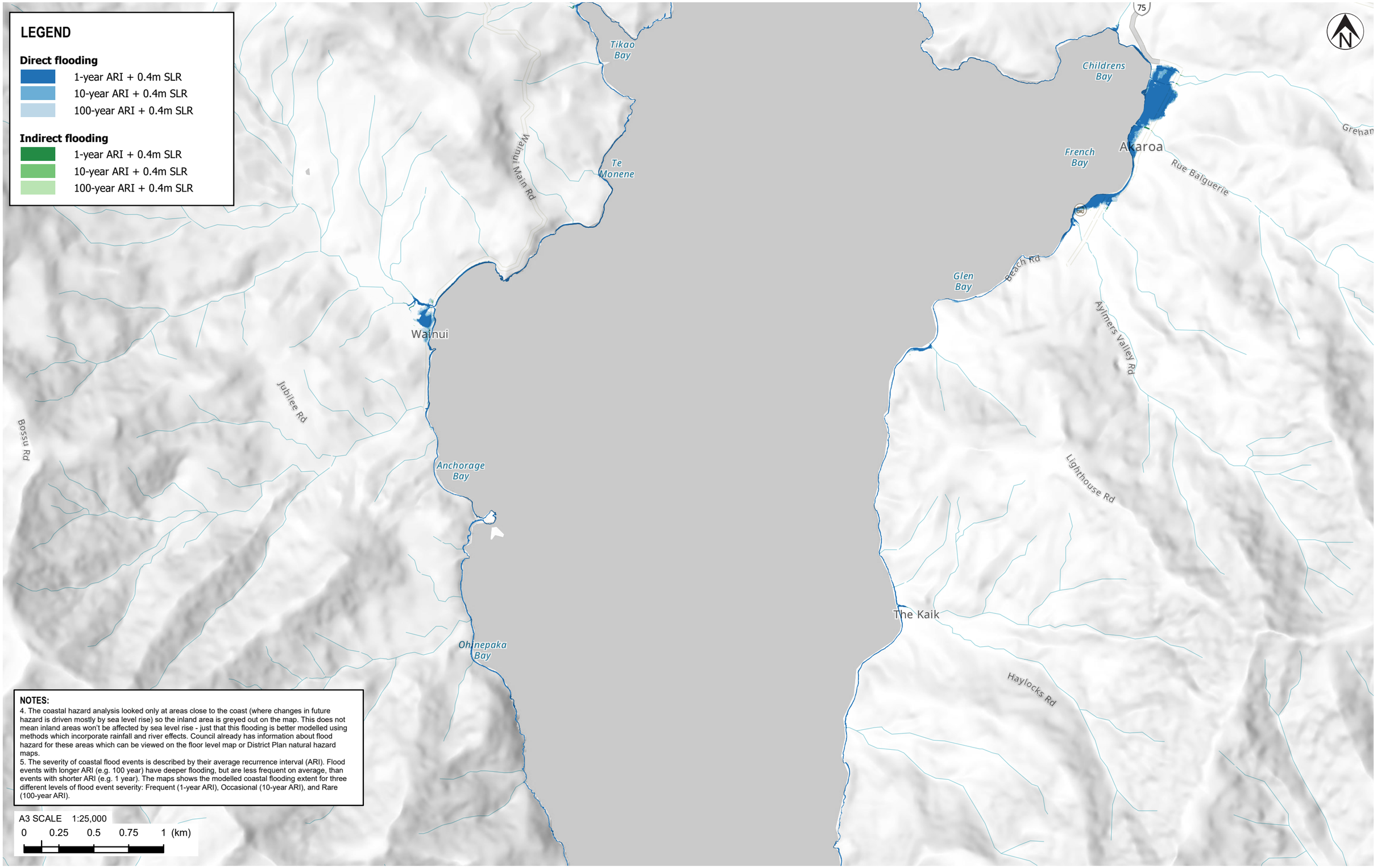

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REV	DESCRIPTION	GIS	CHK	DATE
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LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 9C REV 1

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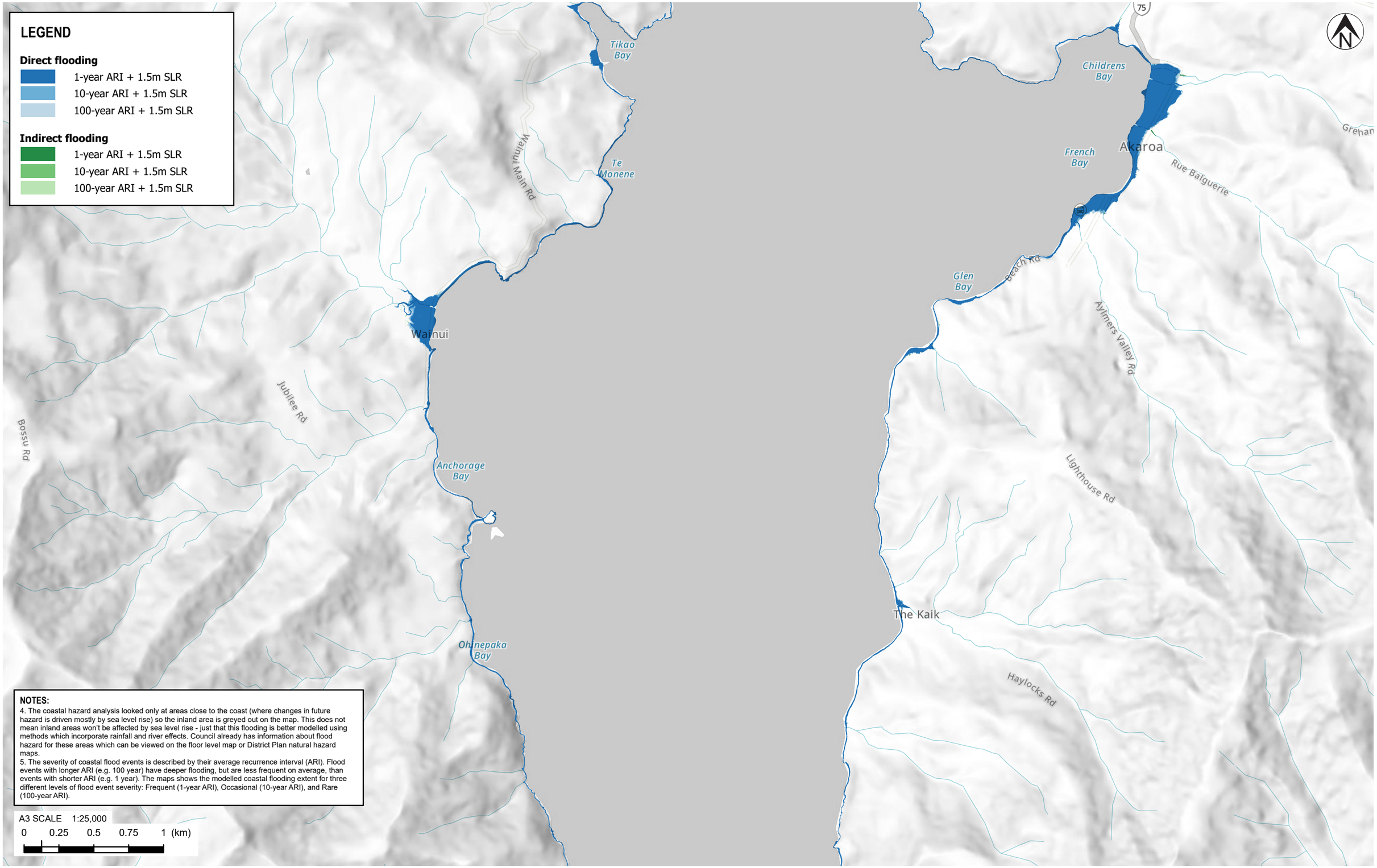

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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 9D REV 1

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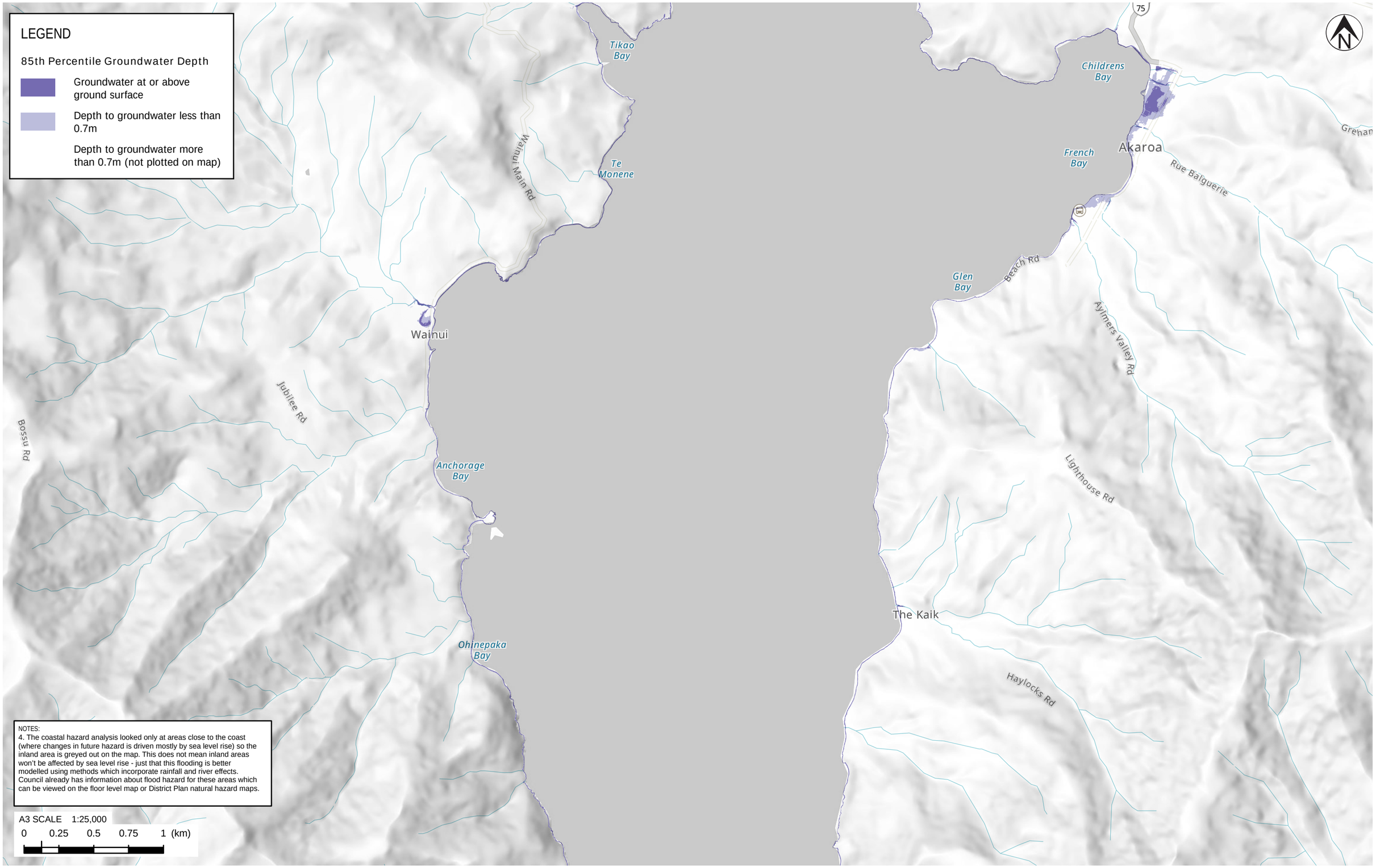

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 9E REV 1

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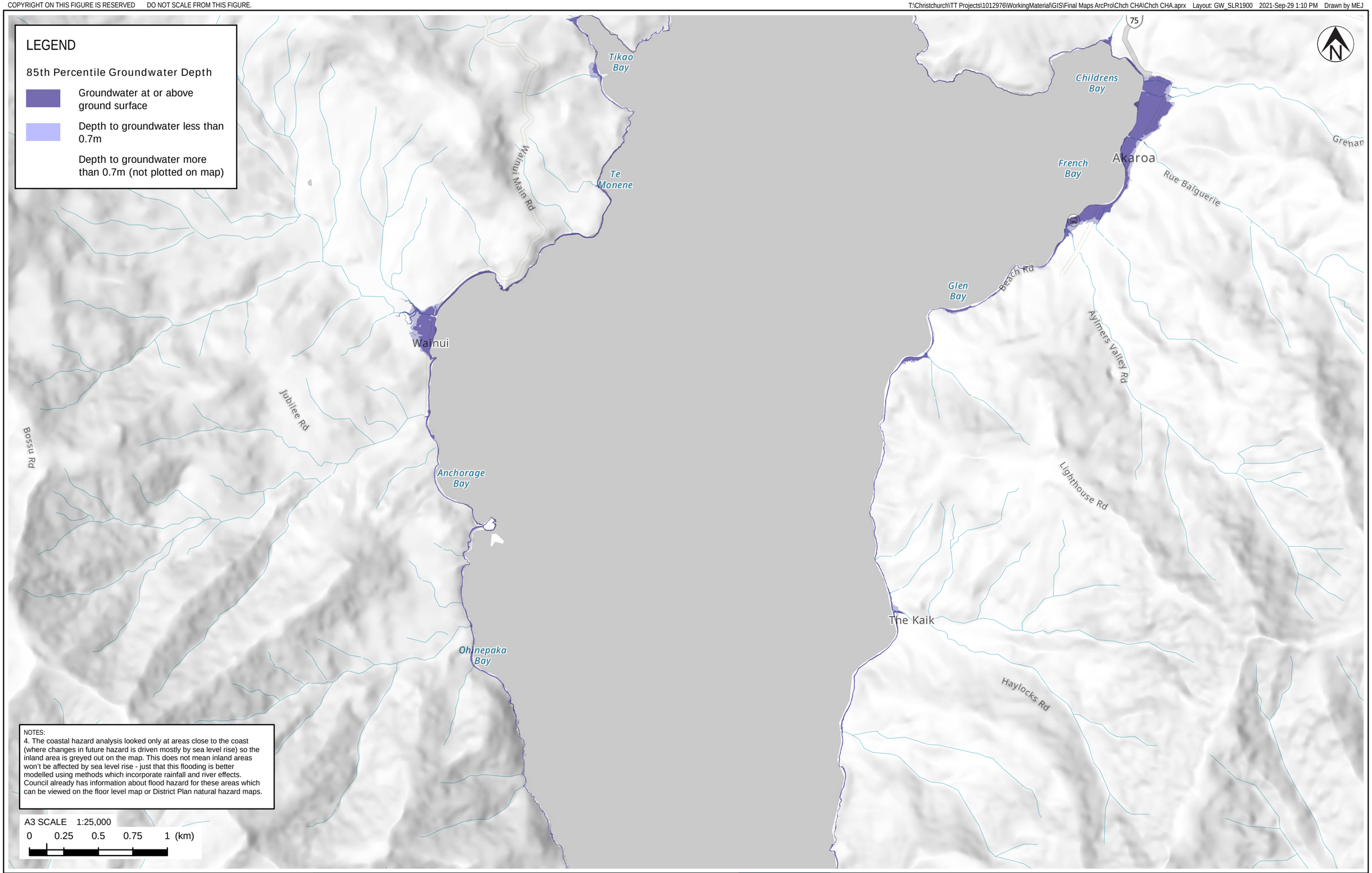

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PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:25,000 FIG No. FIGURE 9F REV 1

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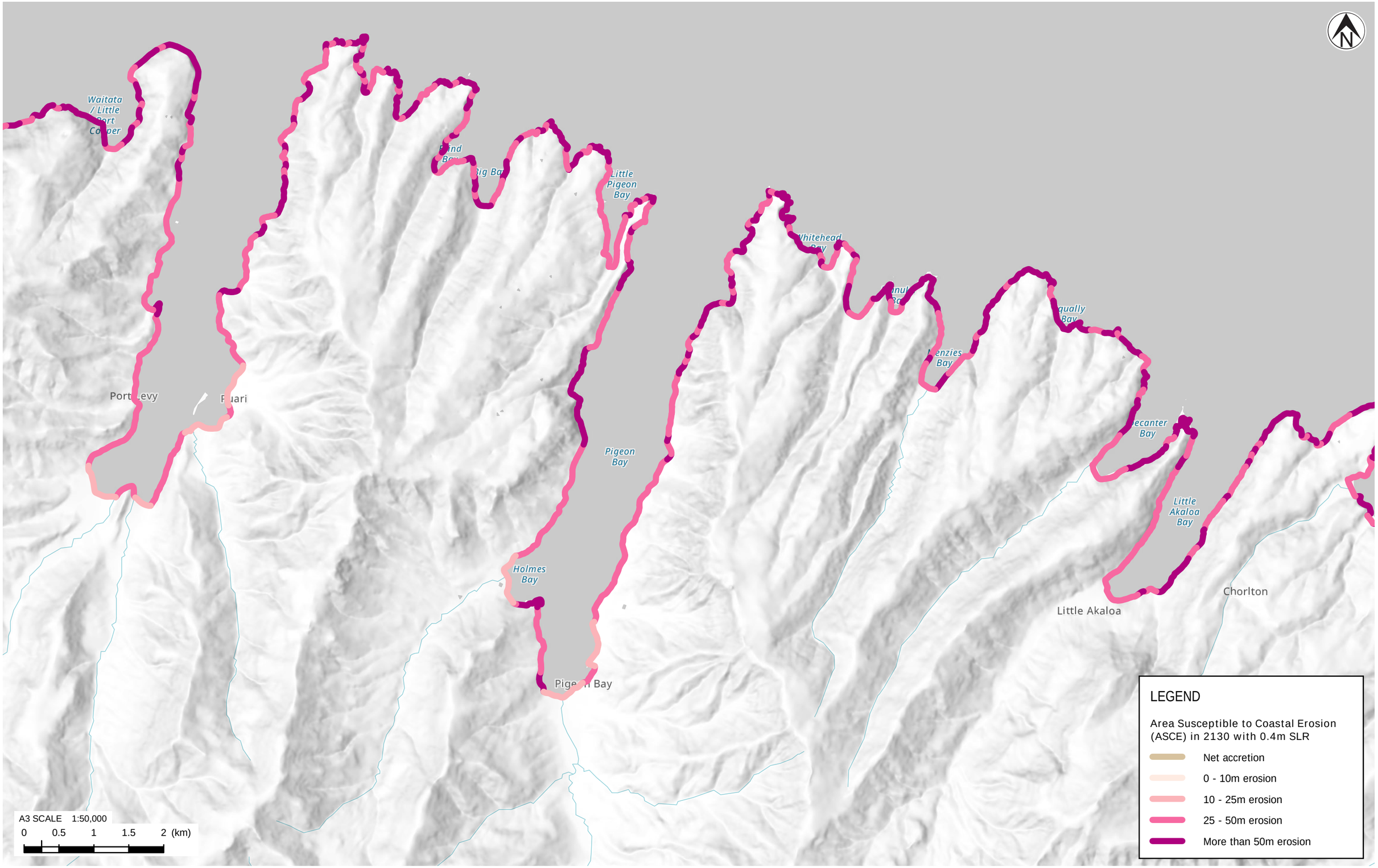

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CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 10A REV 1

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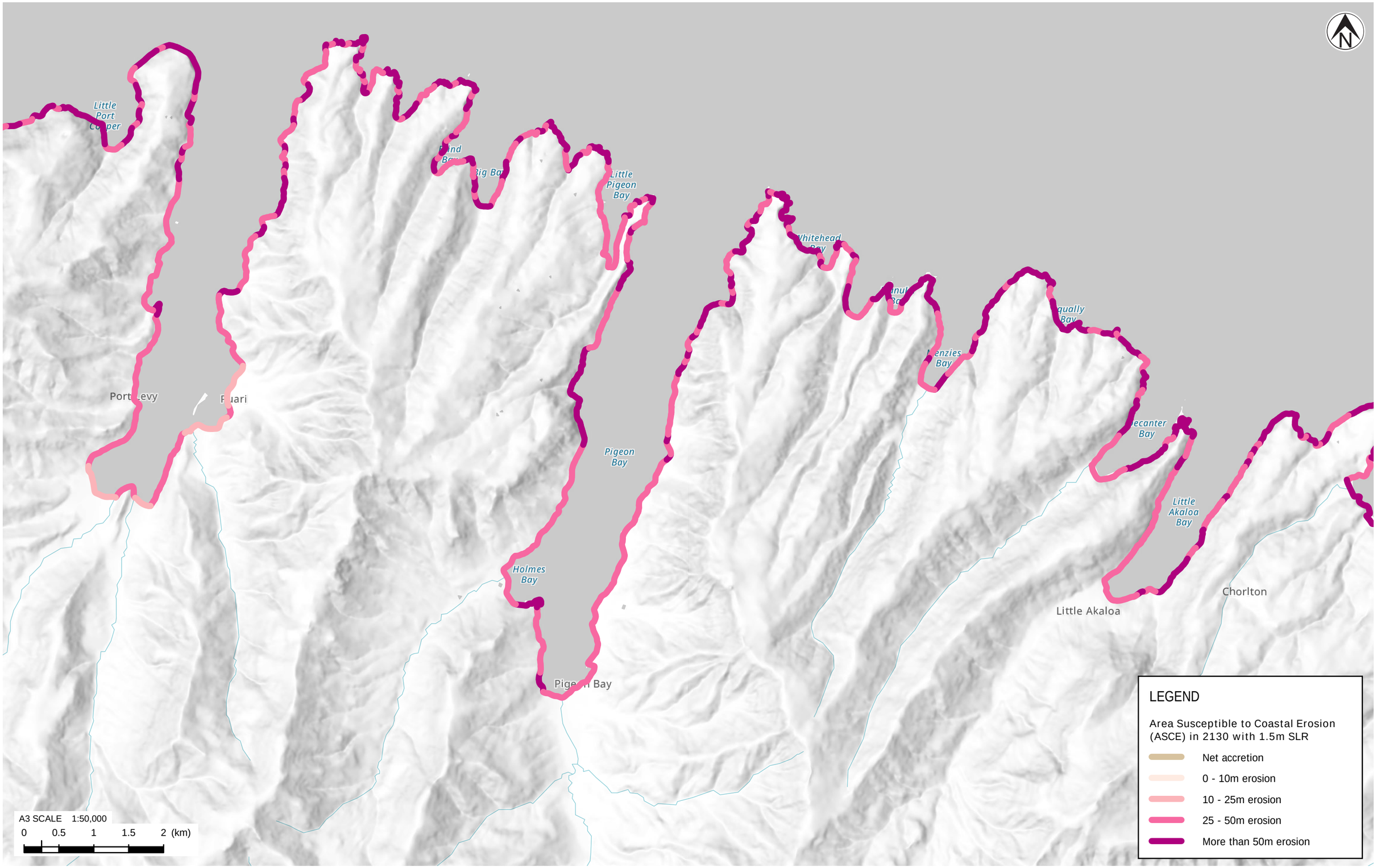

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 10B REV 1

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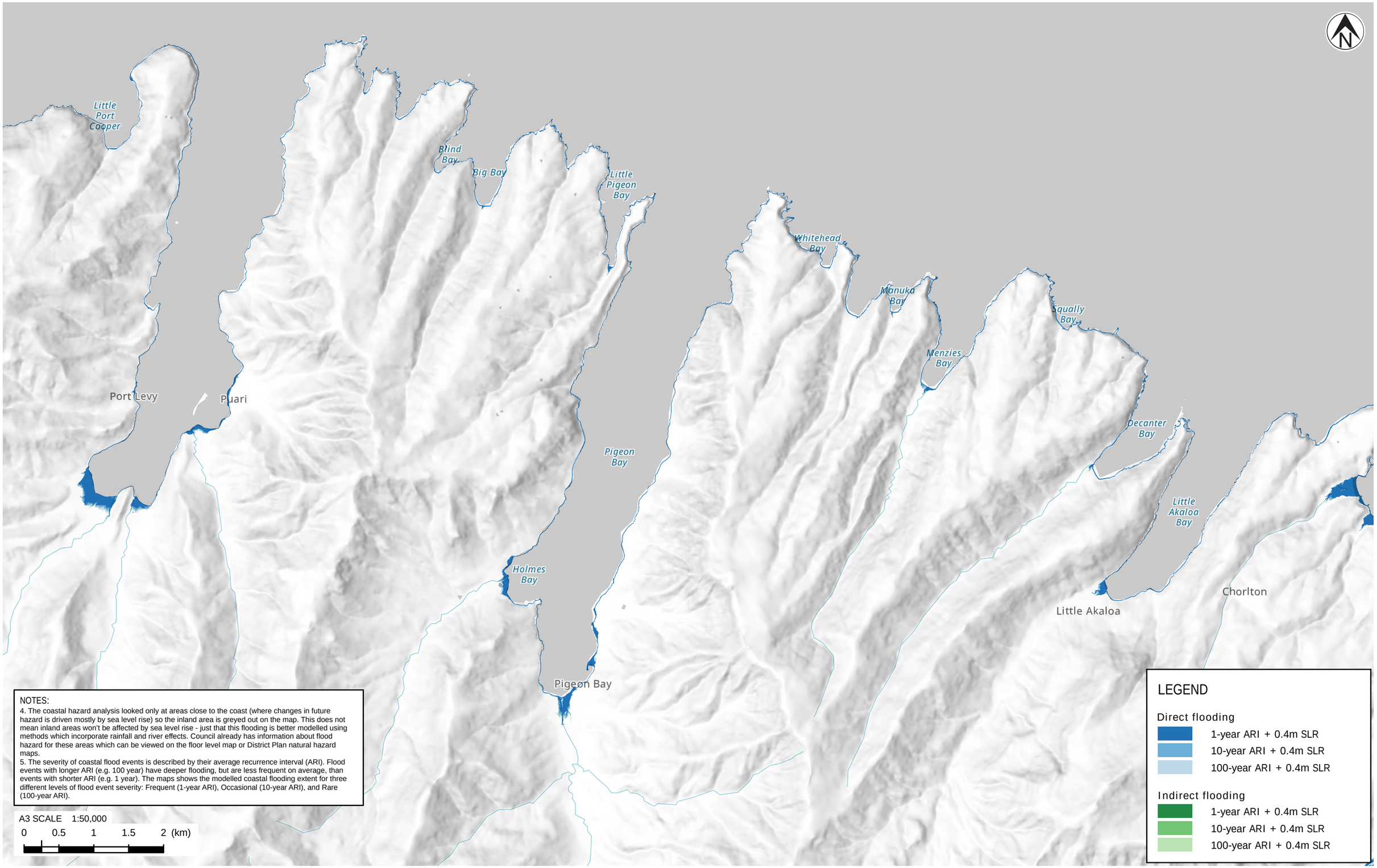

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LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 10C REV 1

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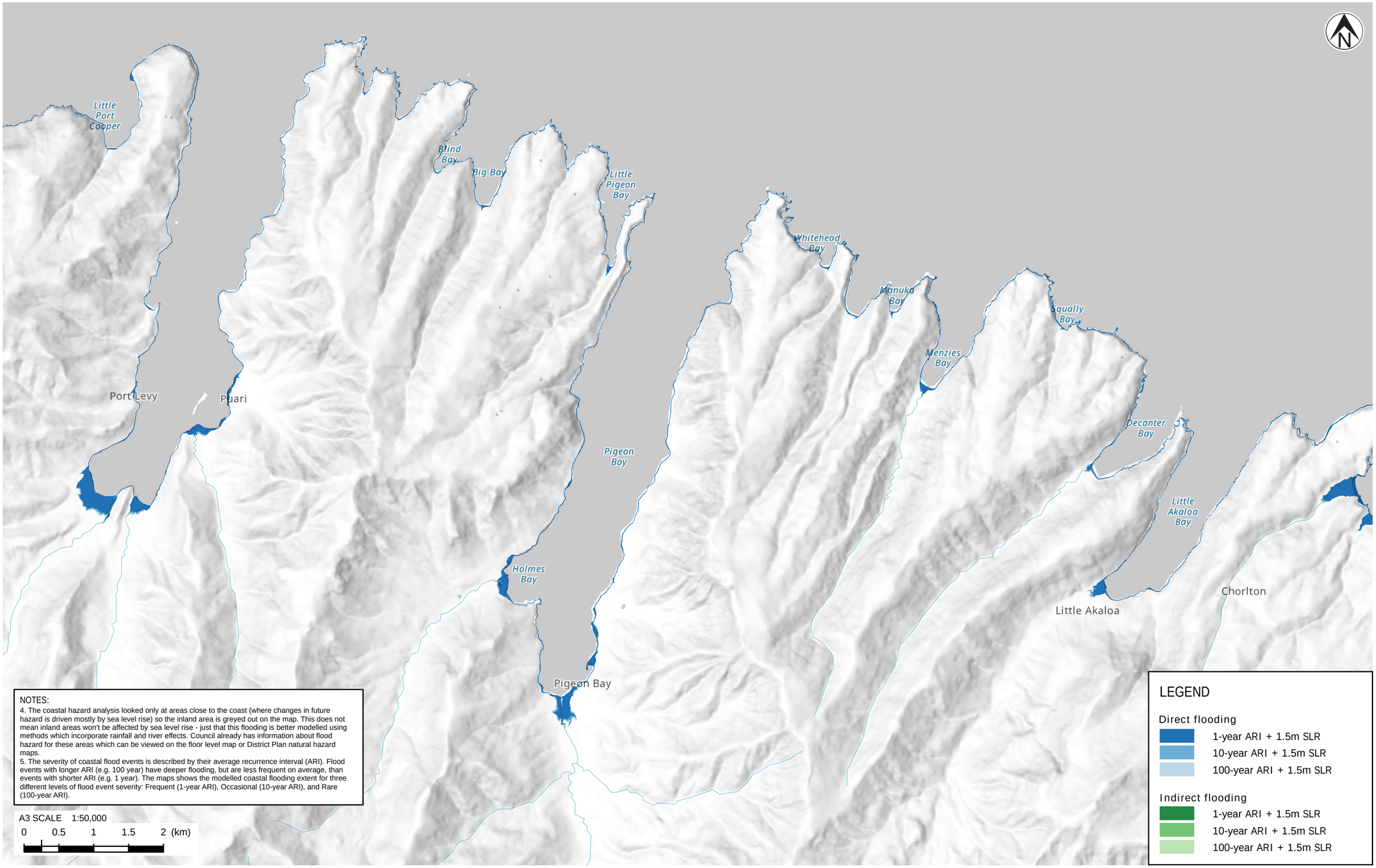

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LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 10D REV 1

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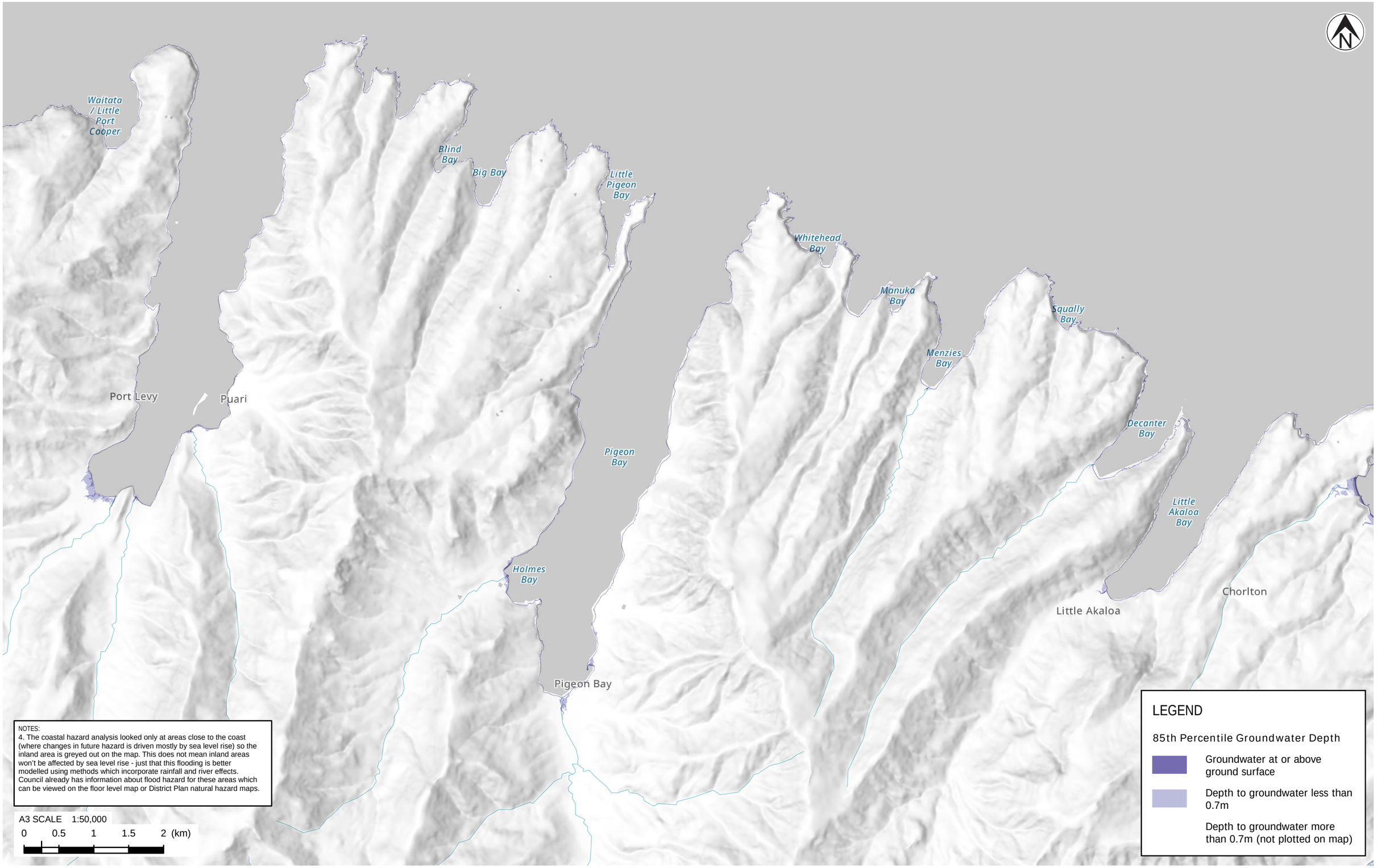

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 10E REV 1

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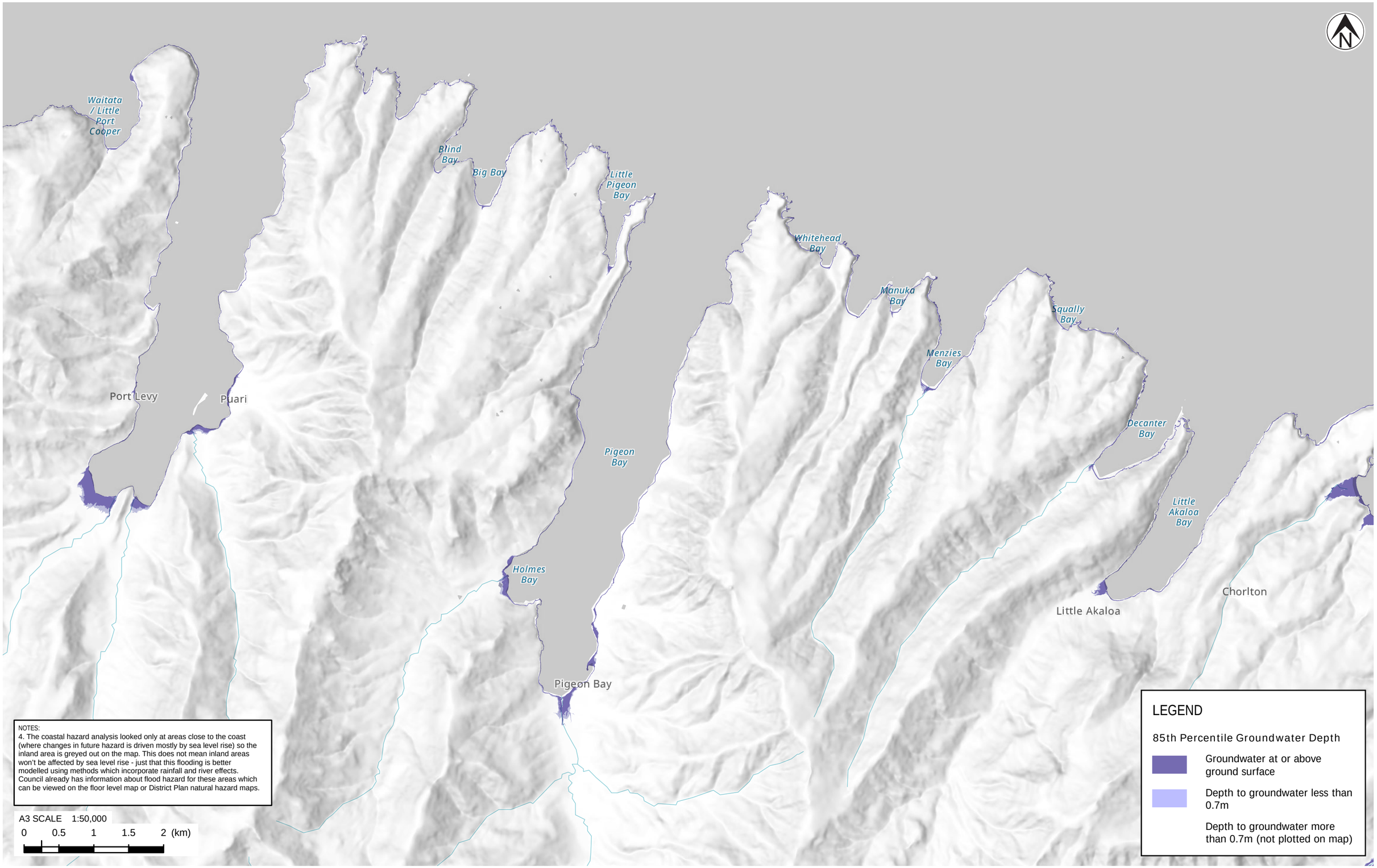

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 10F REV 1

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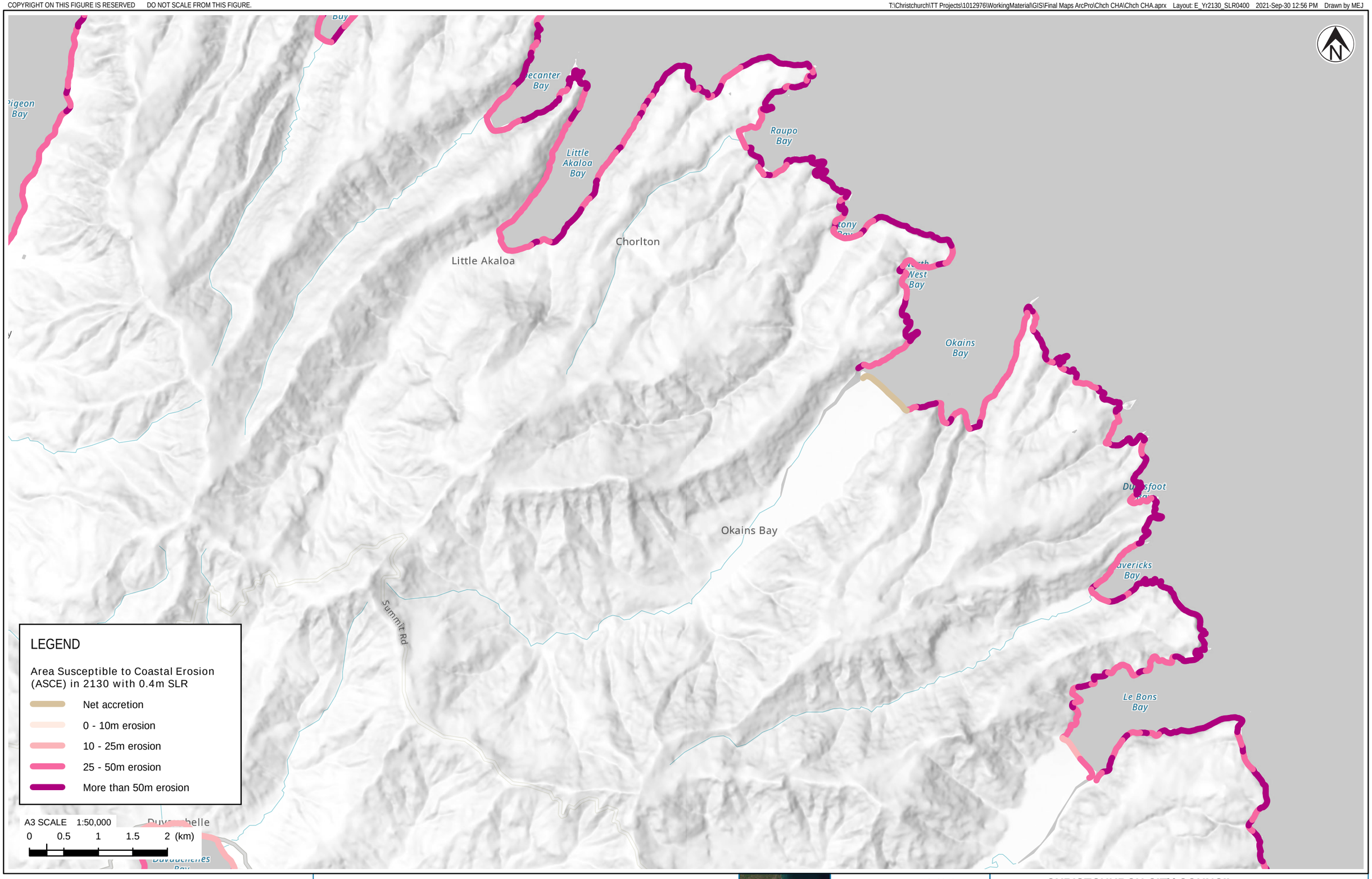

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 11A REV 1

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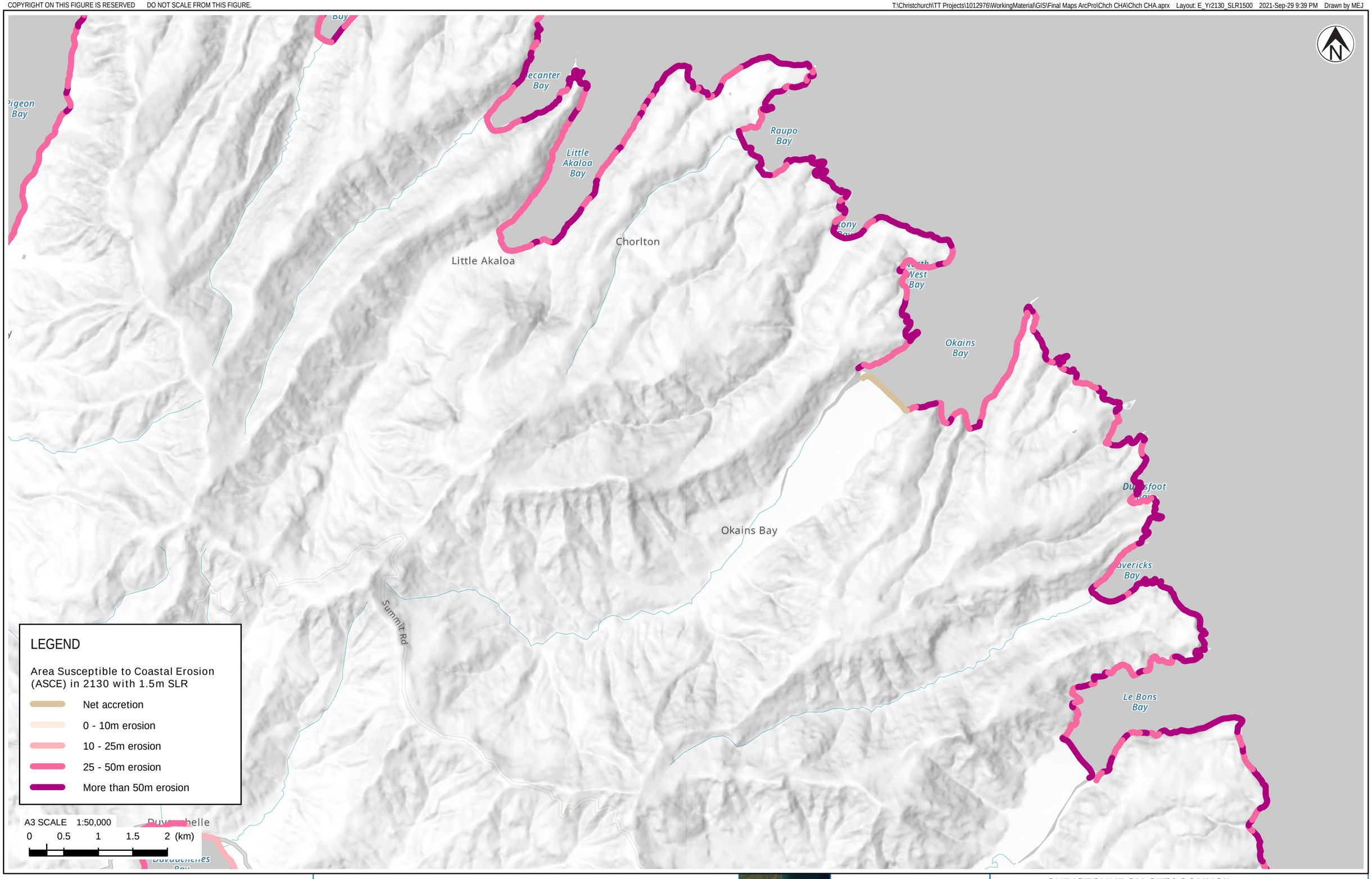

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CLIENT **CHRISTCHURCH CITY COUNCIL**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 11B REV 1

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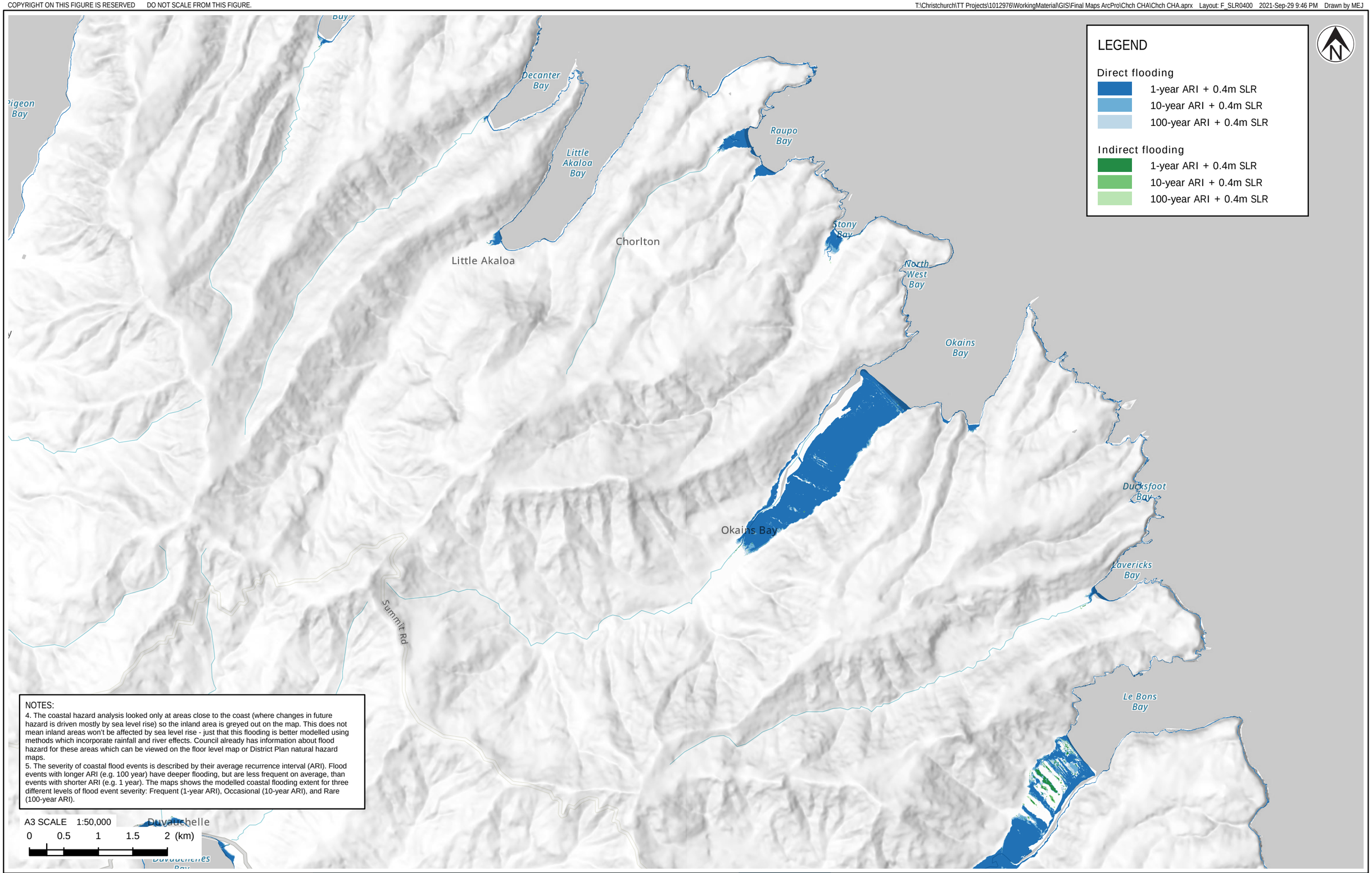

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LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 11C REV 1

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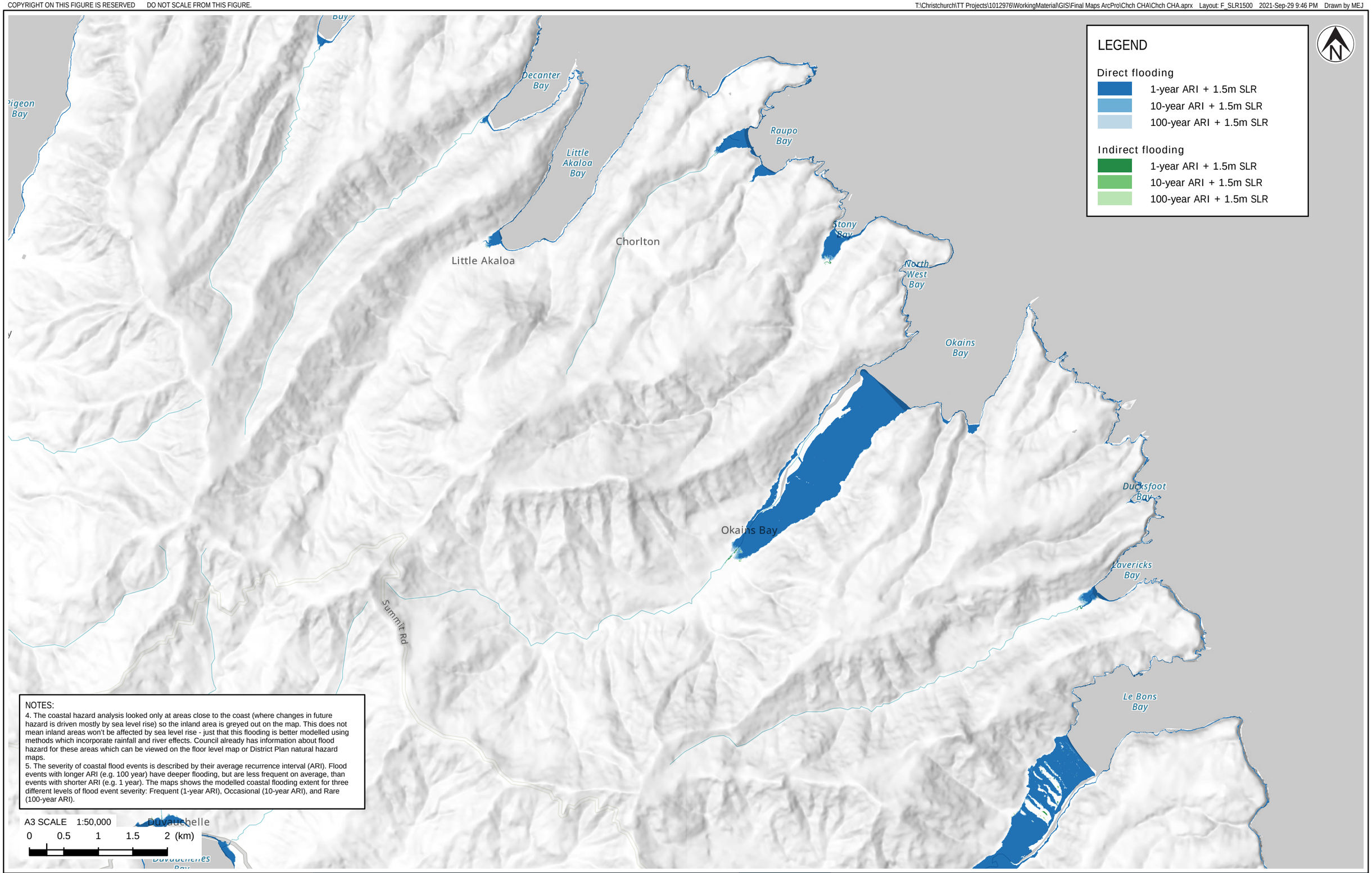

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PROJECT No.	1012976	
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CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 11D REV 1

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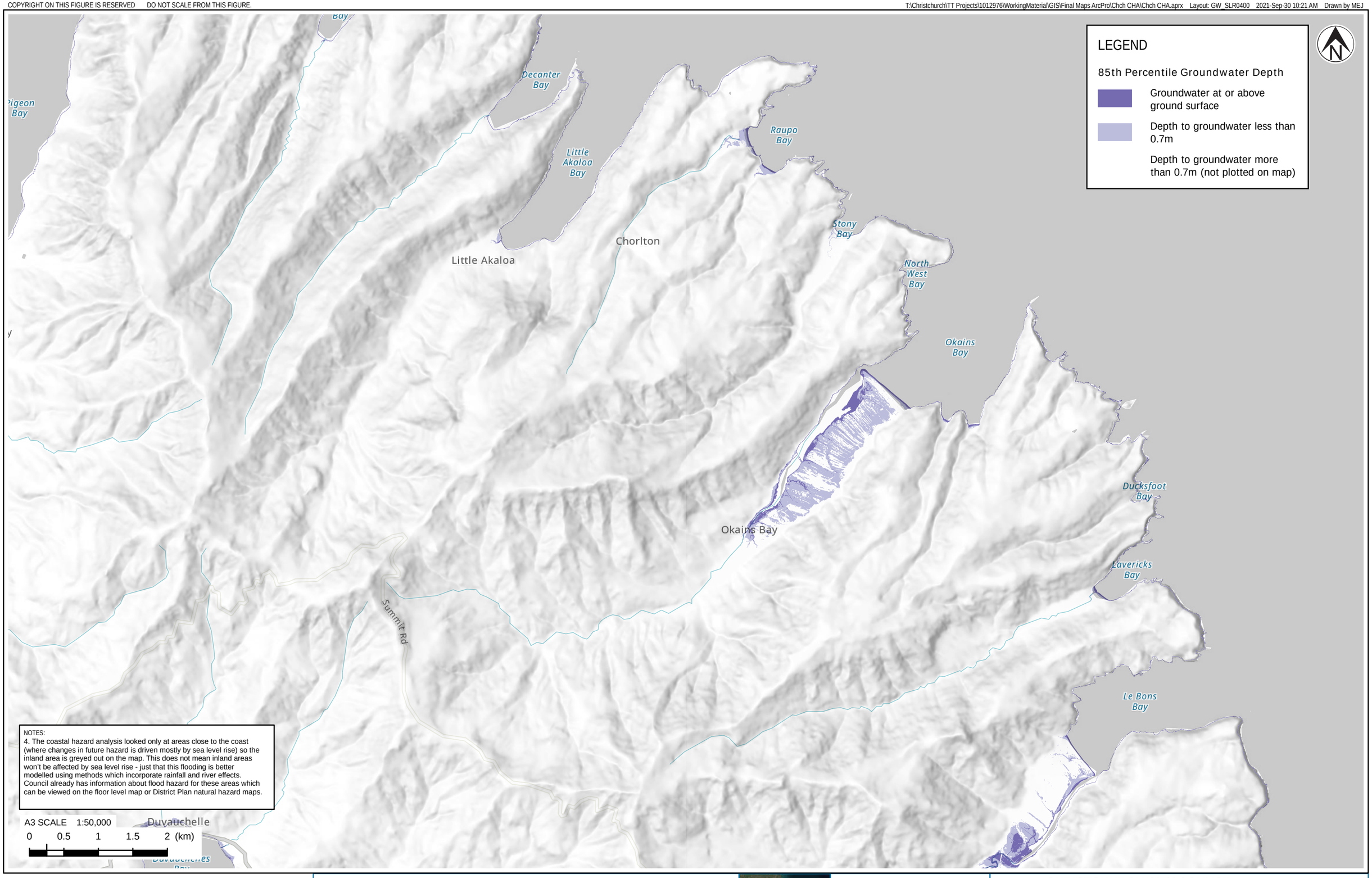

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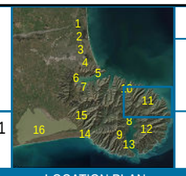

LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 11E REV 1

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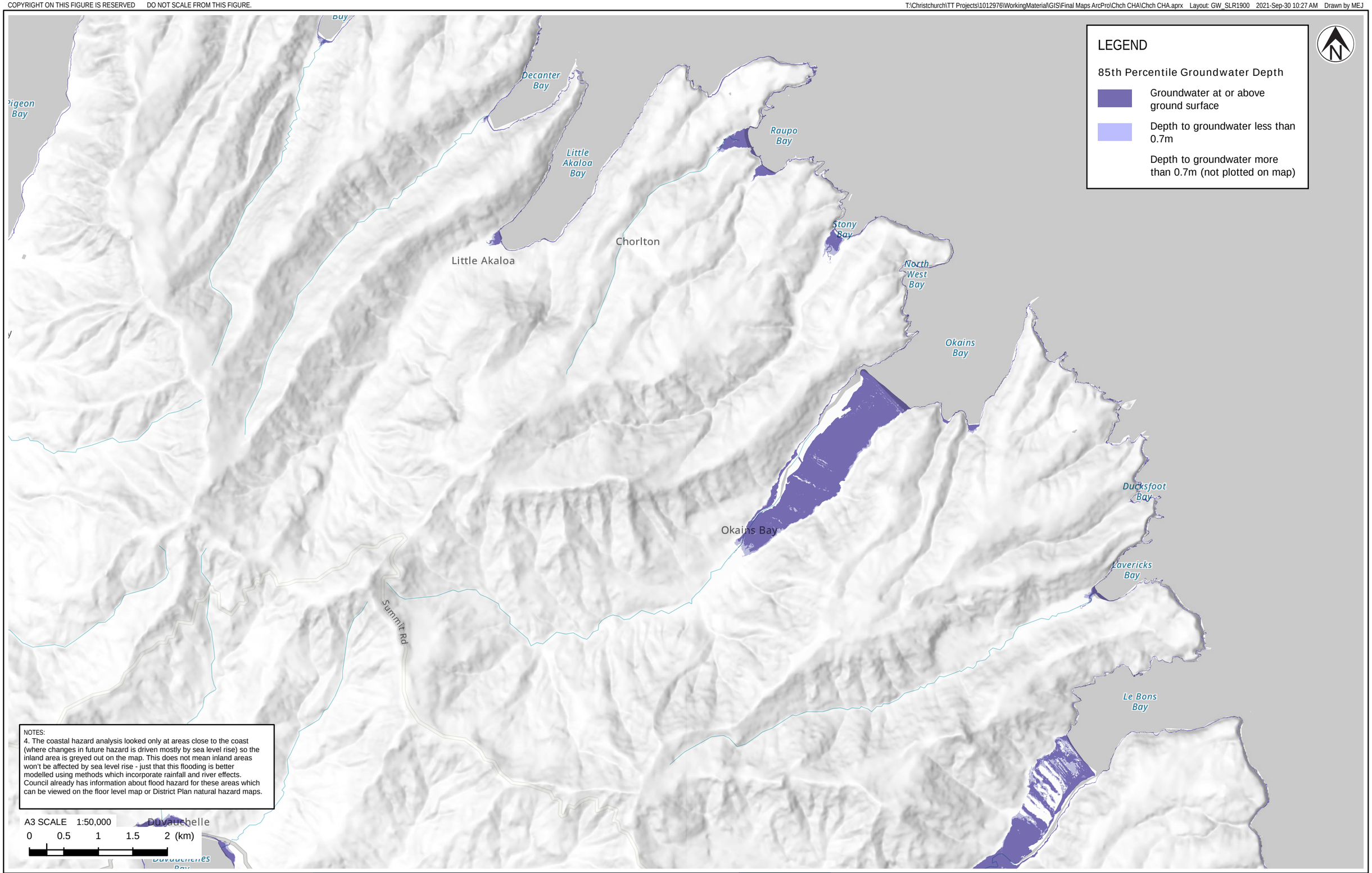

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 11F REV 1

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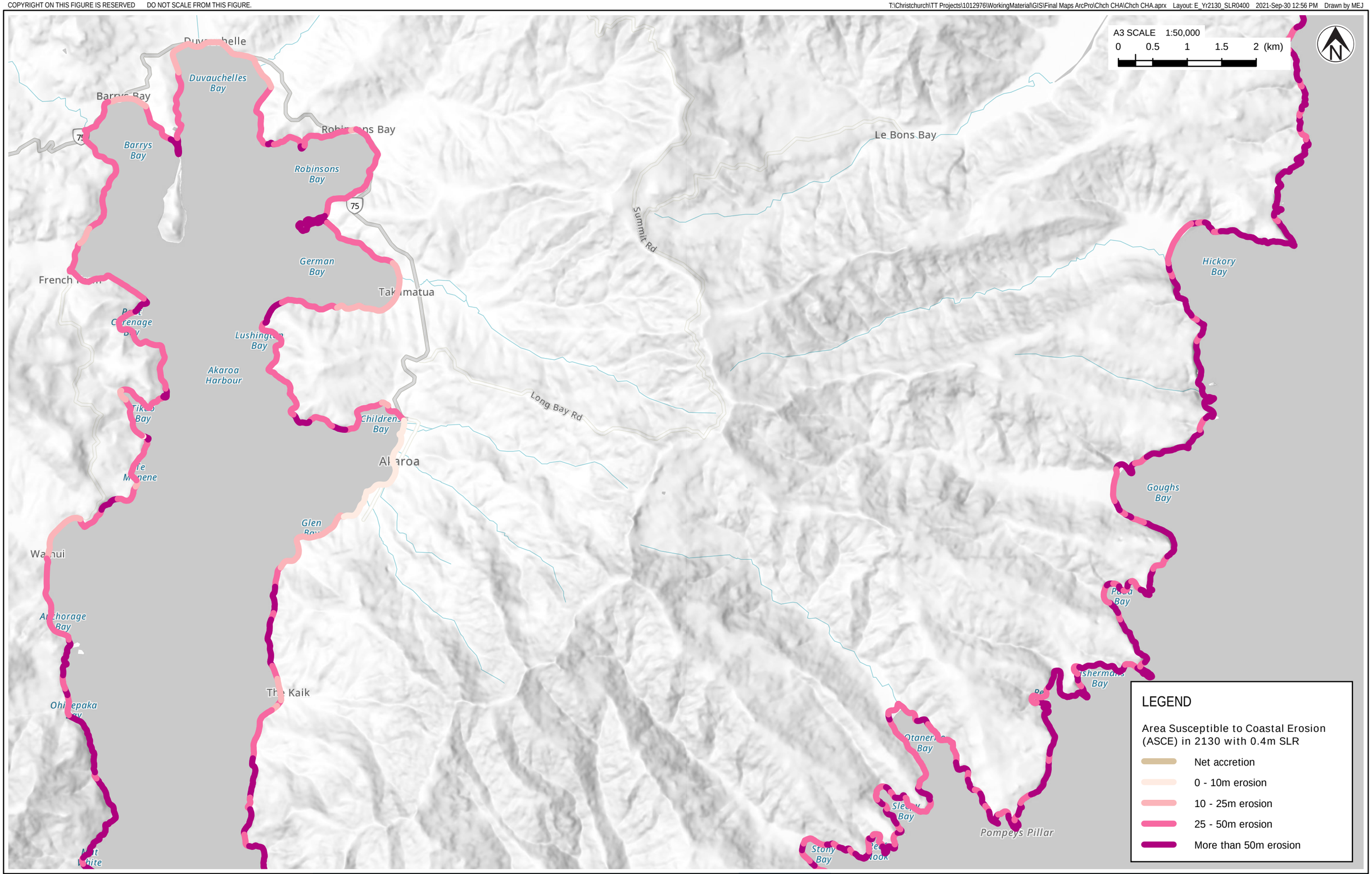

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NOTES:
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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 12A REV 1

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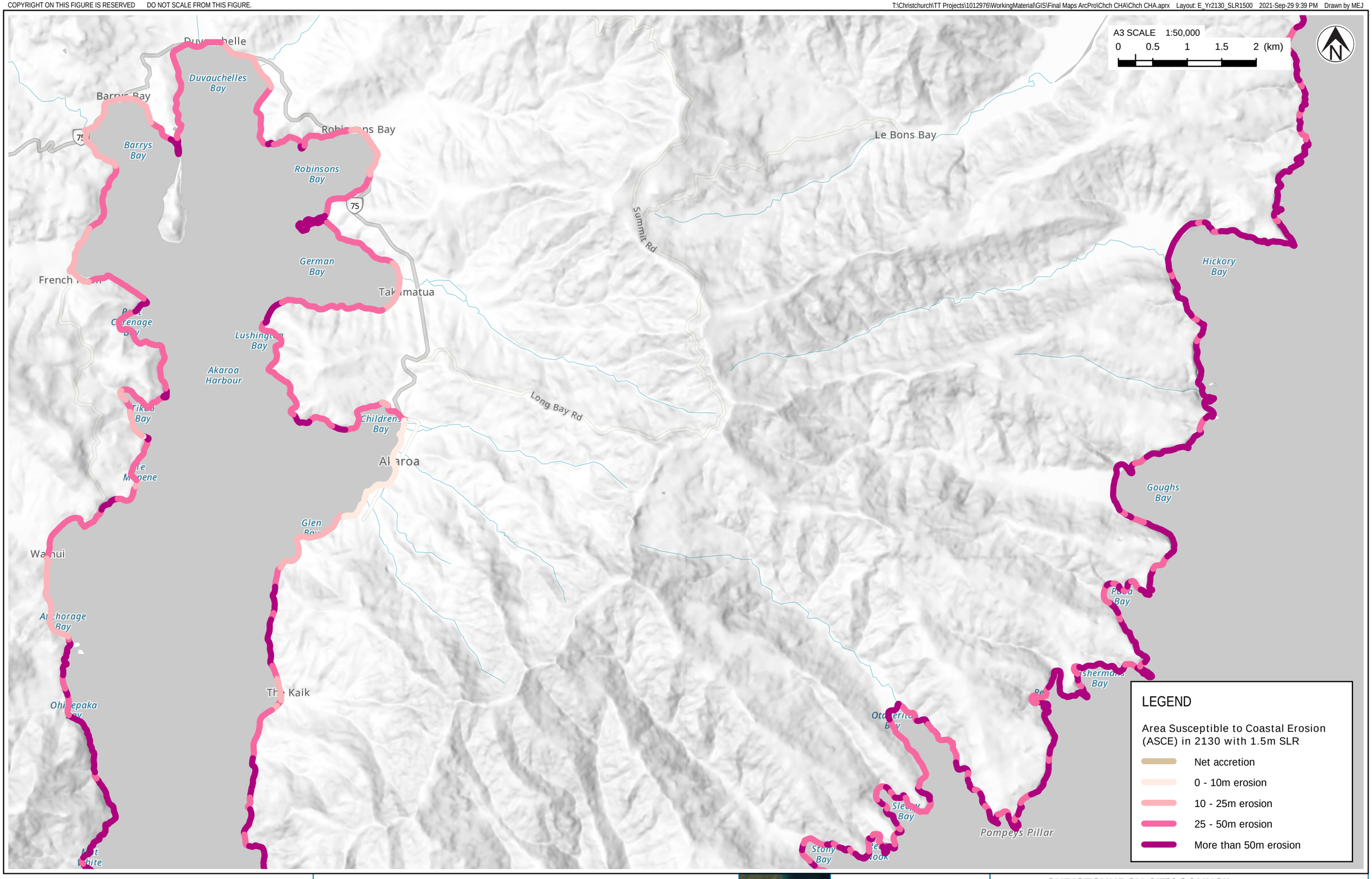

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 12B REV 1

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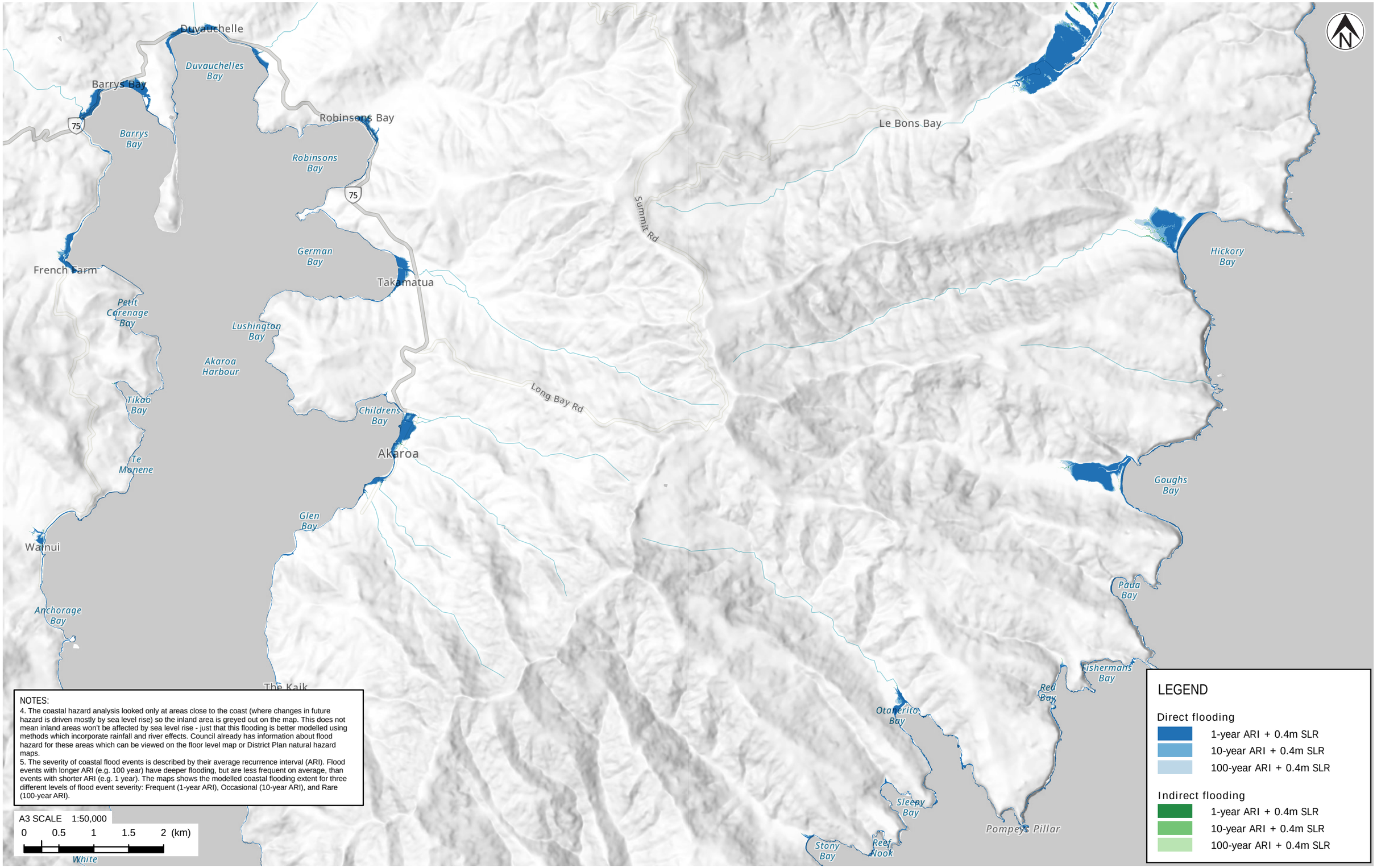

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 12C REV 1

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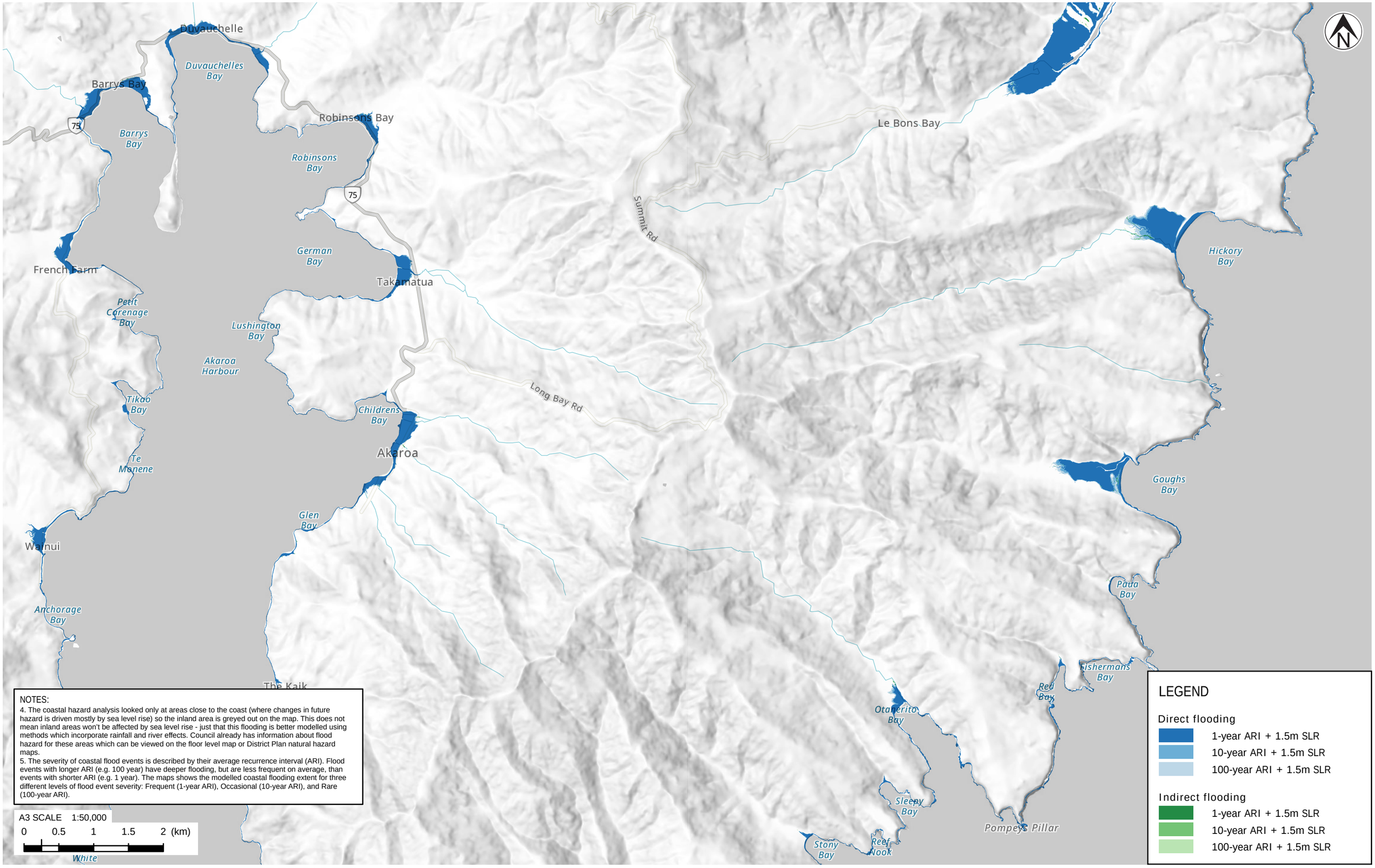

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 12D REV 1

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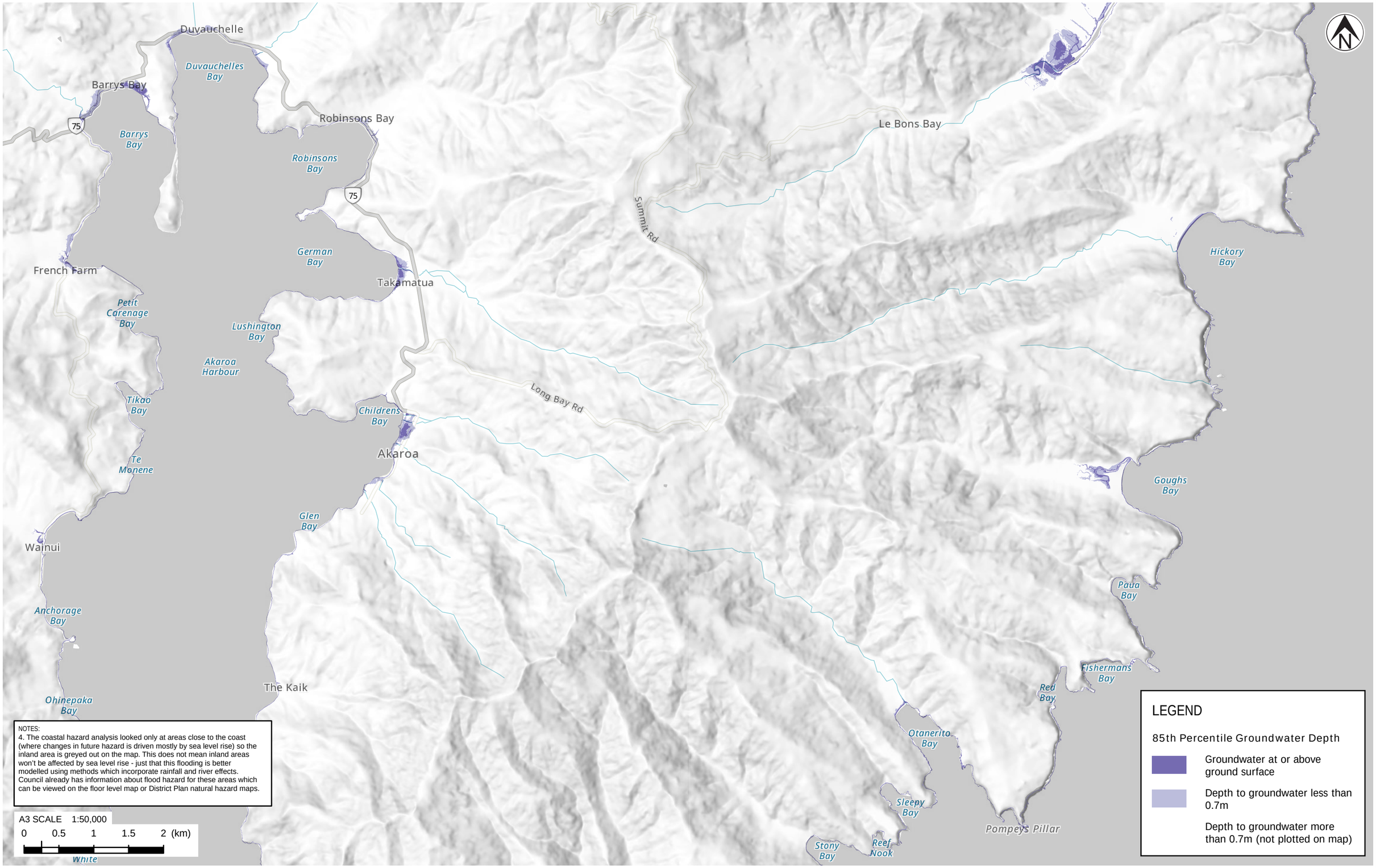

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 12E REV 1

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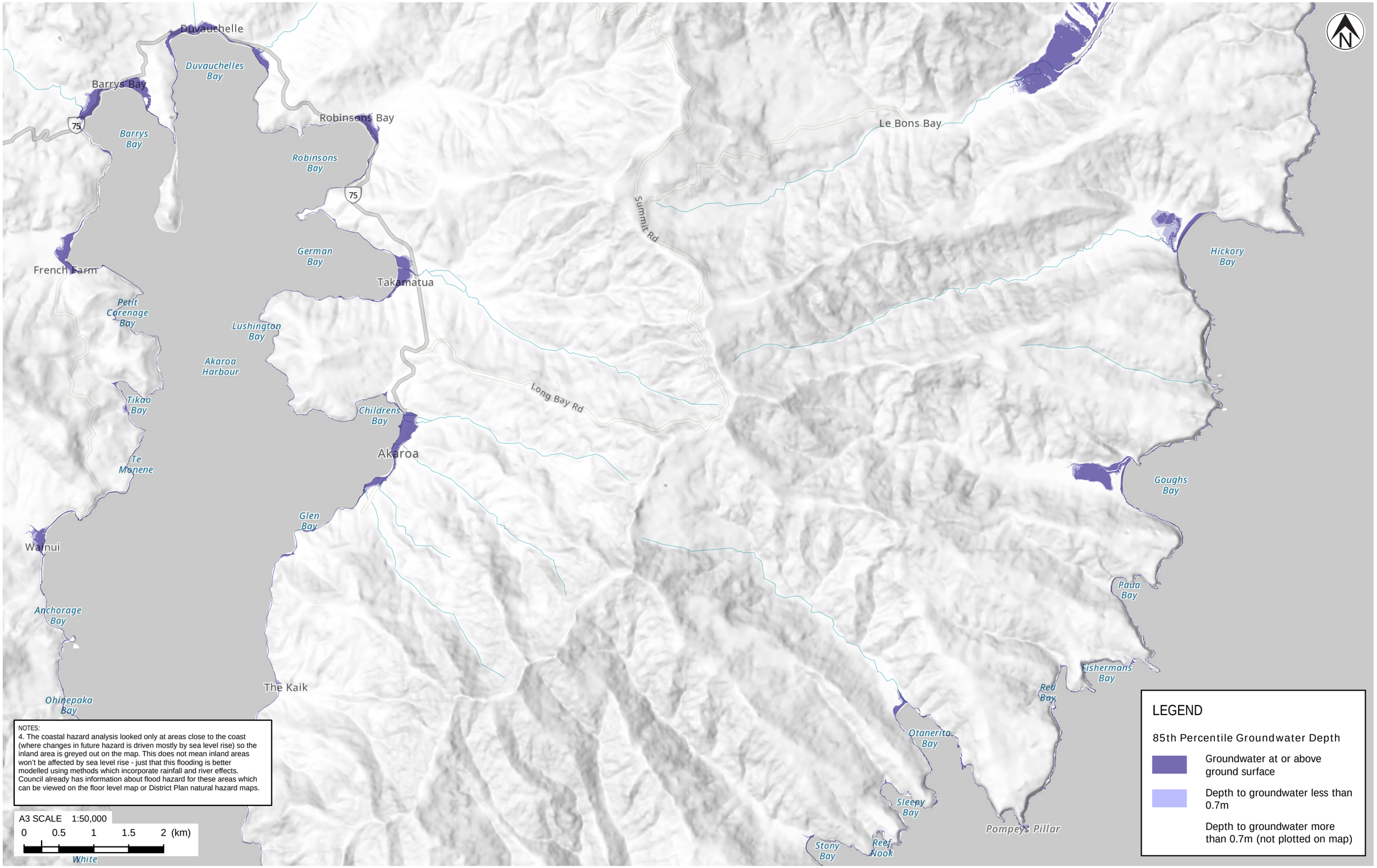

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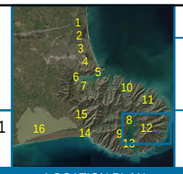

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 12F REV 1

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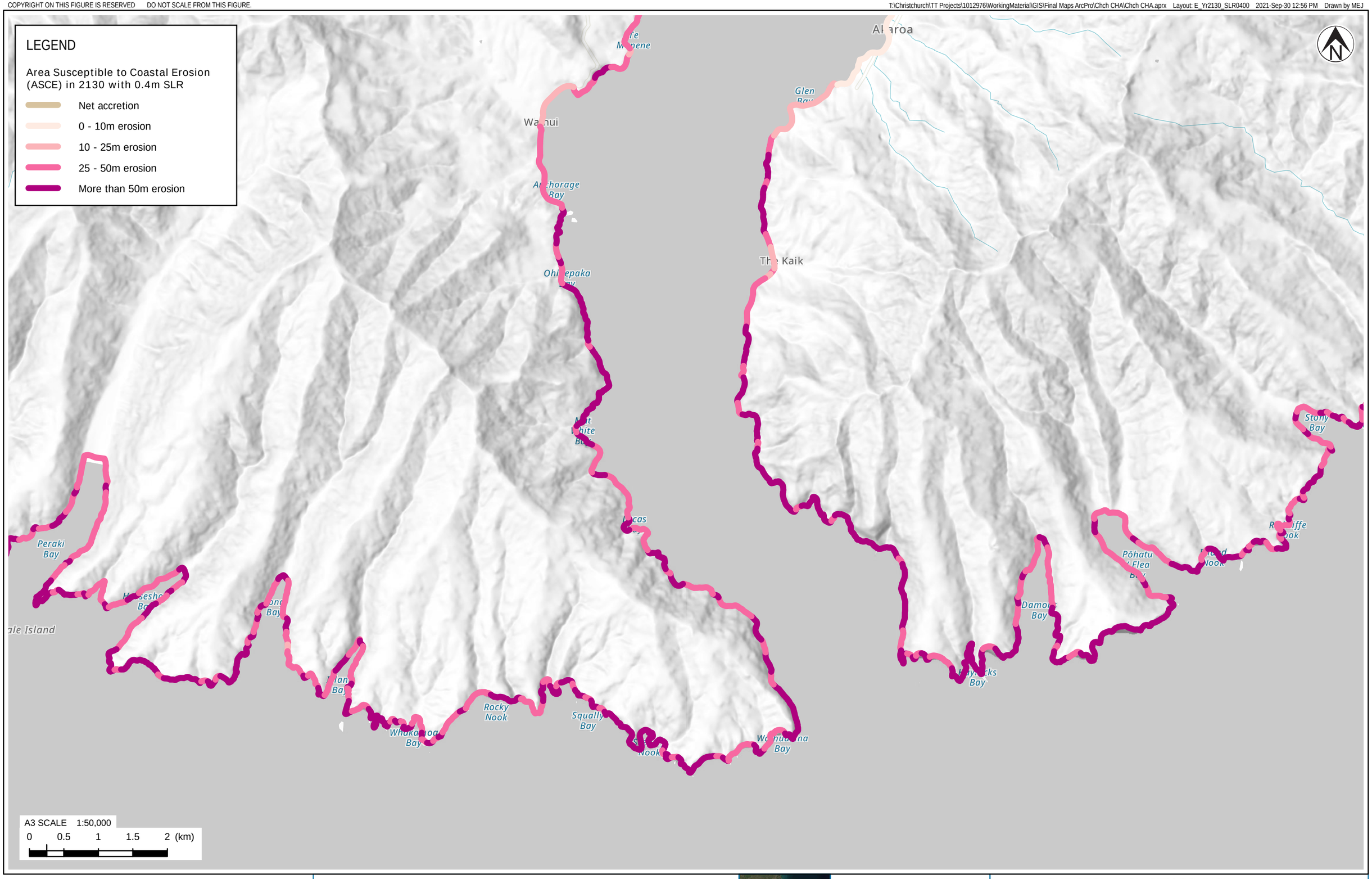

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 13A REV 1

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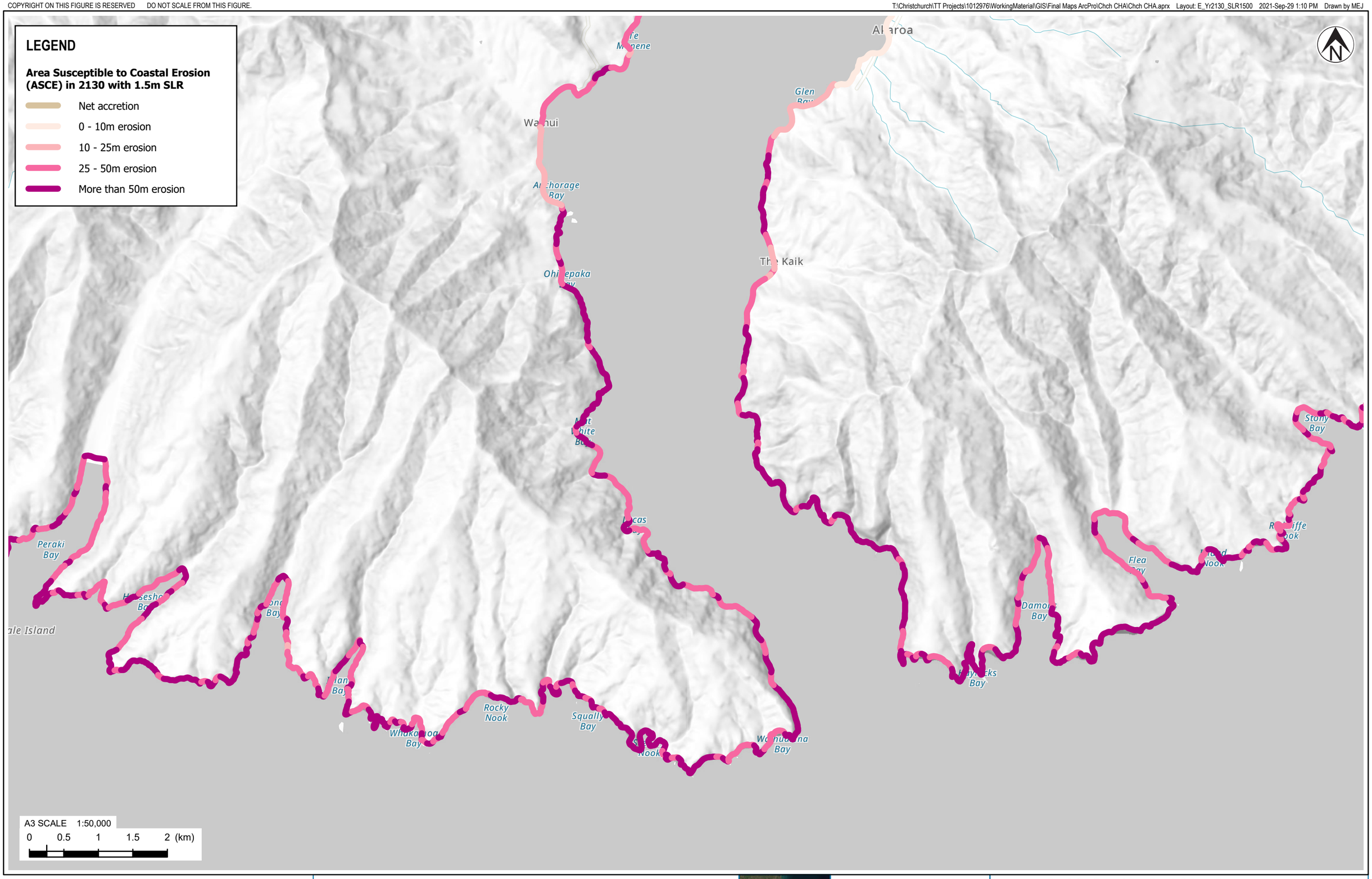

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CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 1.5M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 13B REV 1

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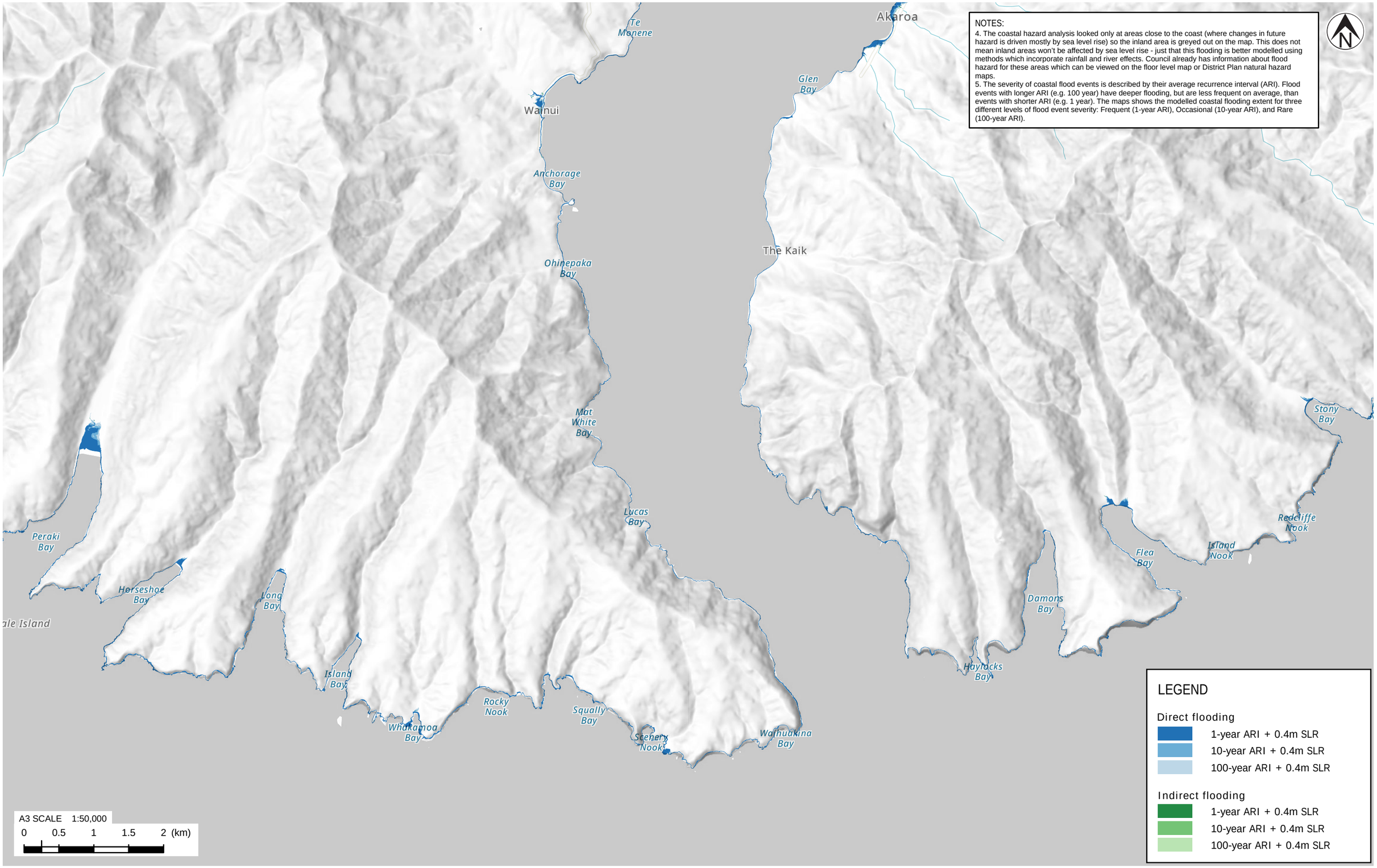

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LEGEND

Direct flooding
- 1-year ARI + 0.4m SLR
- 10-year ARI + 0.4m SLR
- 100-year ARI + 0.4m SLR

Indirect flooding
- 1-year ARI + 0.4m SLR
- 10-year ARI + 0.4m SLR
- 100-year ARI + 0.4m SLR

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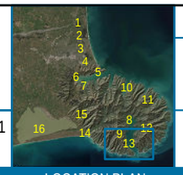

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 13C REV 1

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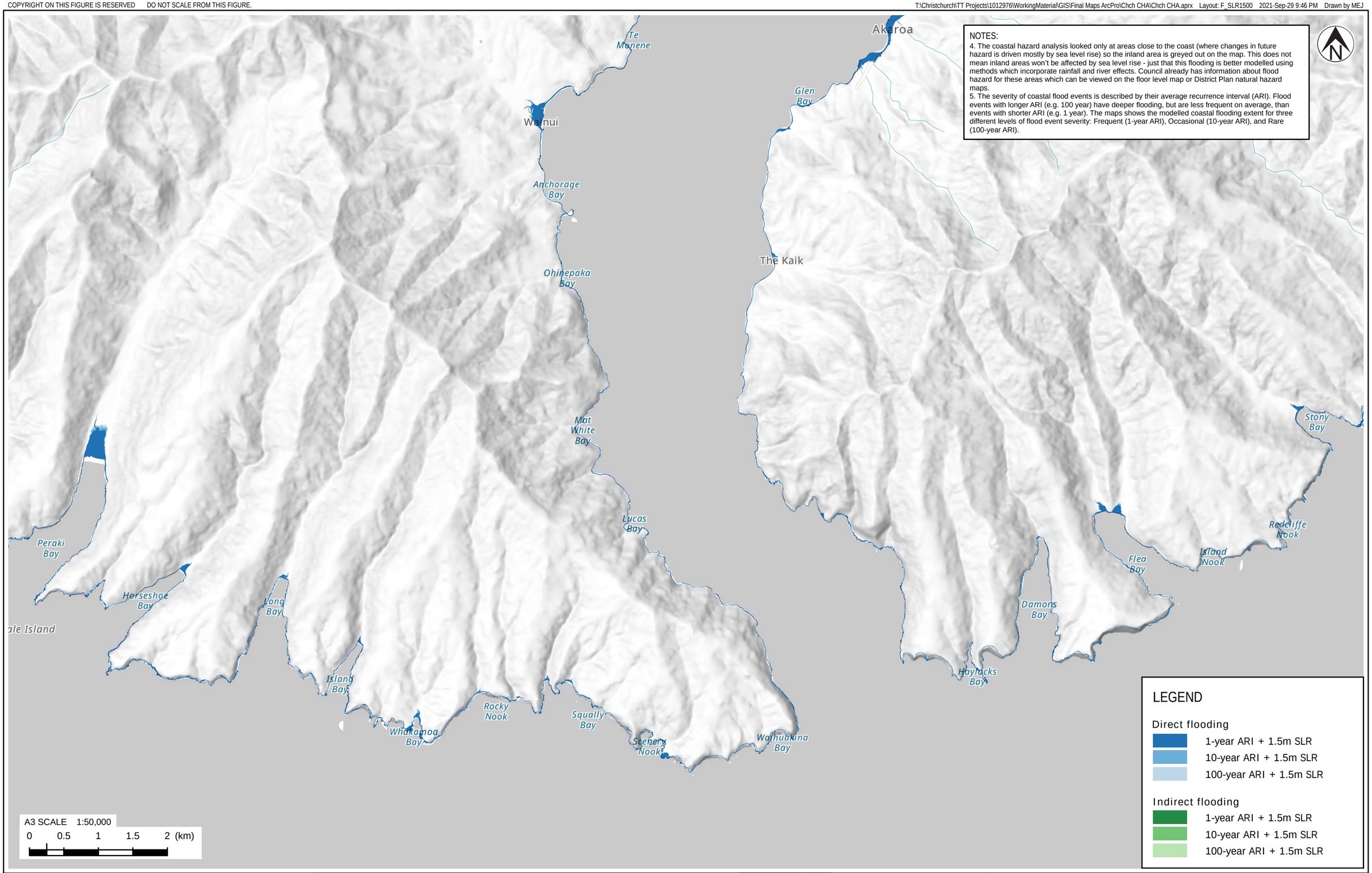

NOTES:
4. The coastal hazard analysis looked only at areas close to the coast (where changes in future hazard is driven mostly by sea level rise) so the inland area is greyed out on the map. This does not mean inland areas won't be affected by sea level rise - just that this flooding is better modelled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

LEGEND

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

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

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NOTES:
1. 'Direct flooding' is where there is a direct path for water to flow overland from the coast.
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3. Basemap NZ Hillshade (Alpha): LINZ, Eagle Technology. NZ Topographic Map for use with relief - Grey: Eagle Technology, LINZ, StatsNZ, NIWA, Natural Earth, © OpenStreetMap contributors.

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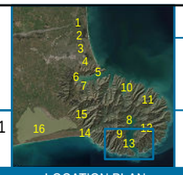

PROJECT No.	1012976	
DESIGNED	MEJ	SEP.21
DRAWN	MEJ	SEP.21
CHECKED	PPK	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 13D REV 1

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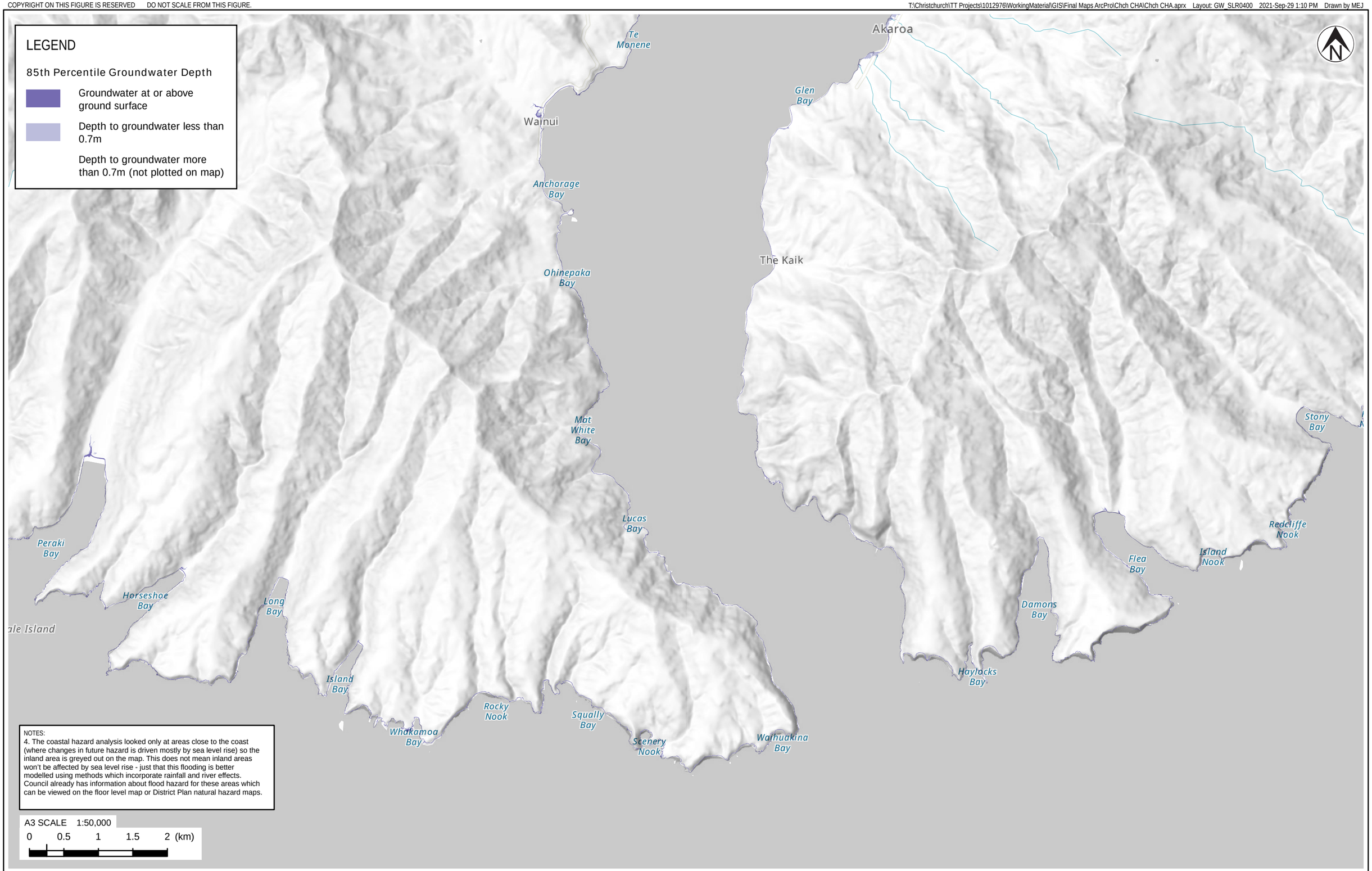

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3. Basemap NZ Hillshade (Alpha): LINZ, Eagle Technology. NZ Topographic Map for use with relief - Grey: Eagle Technology, LINZ,

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 13E REV 1

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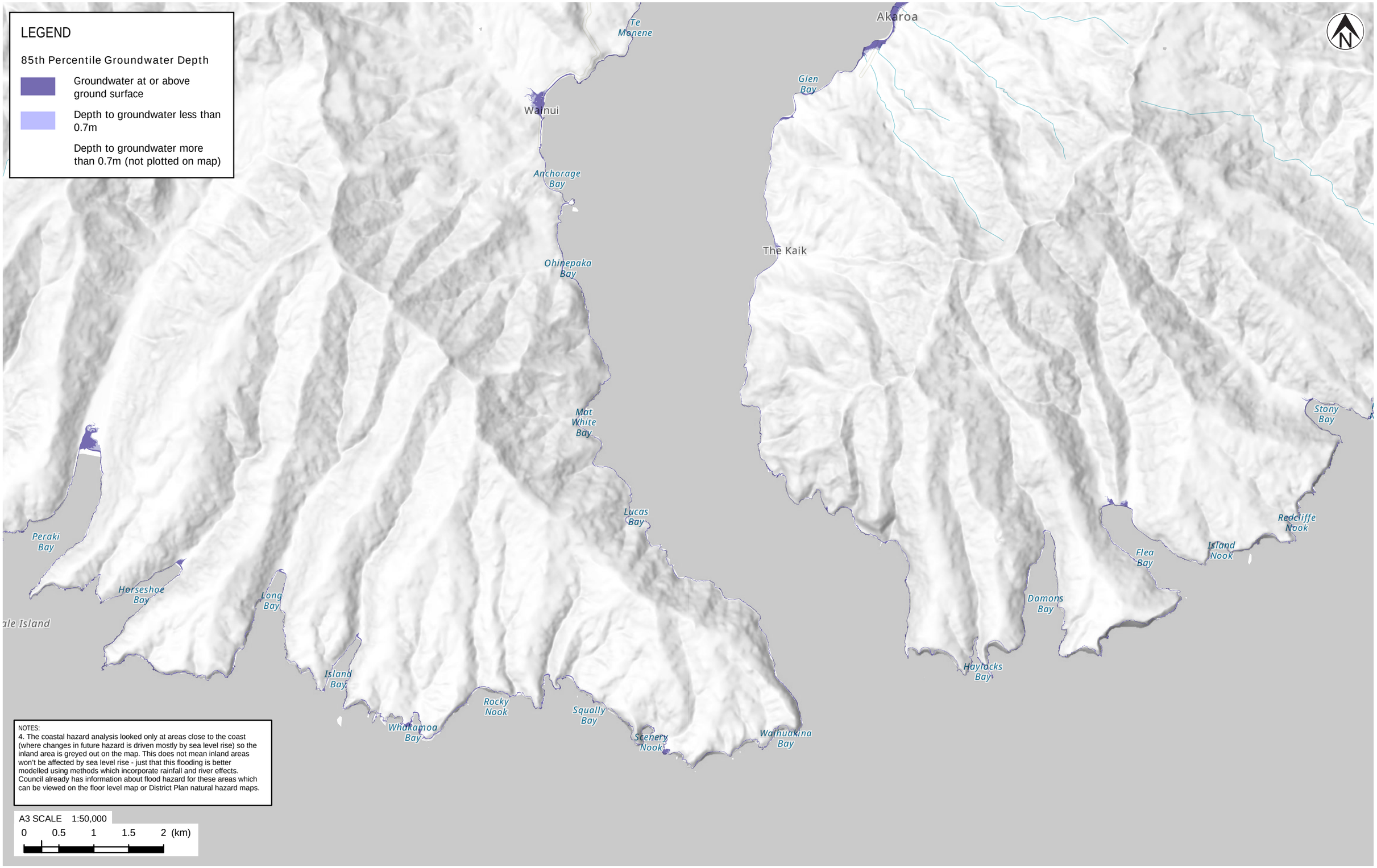

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 13F REV 1

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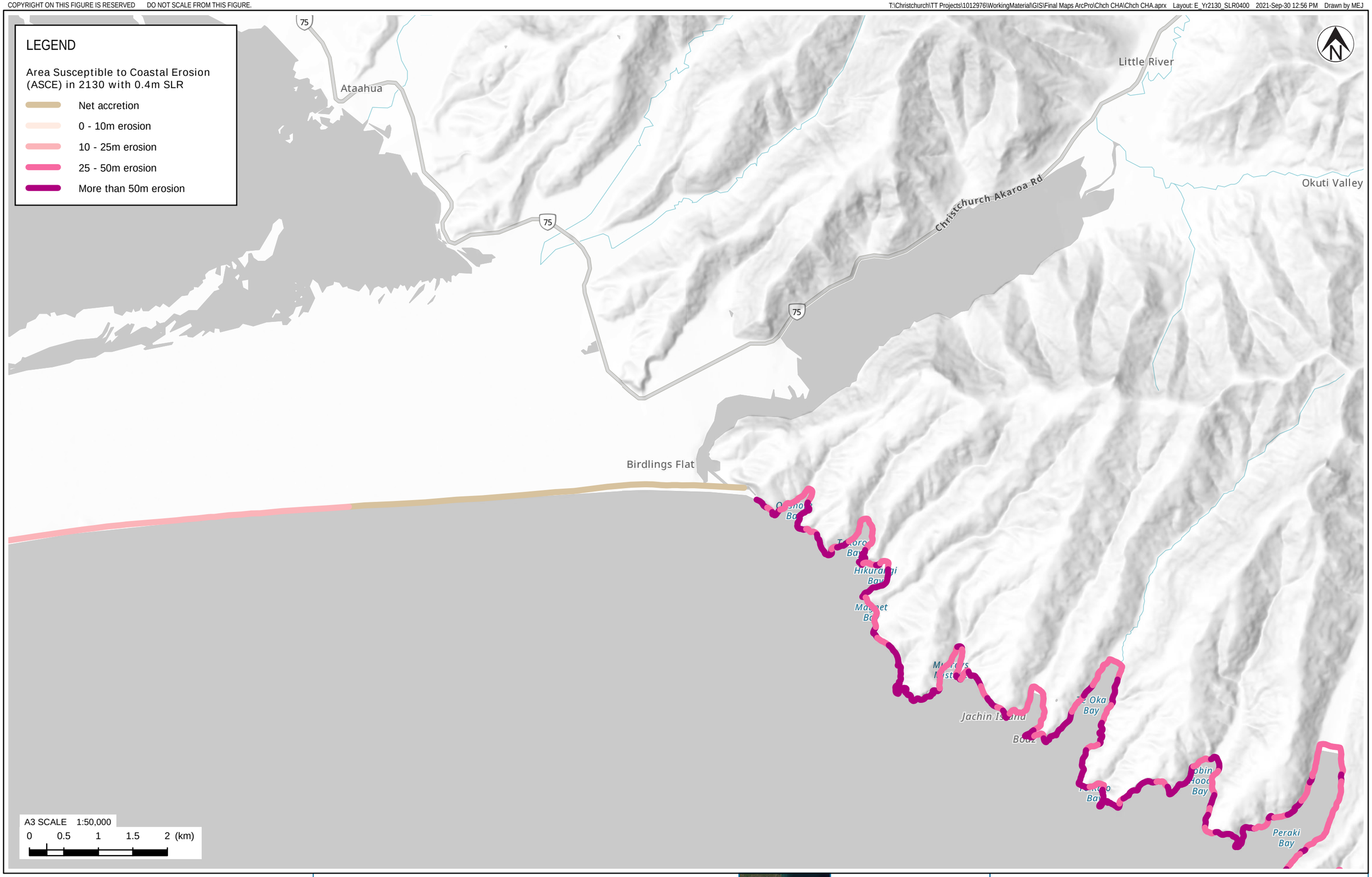

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 14A REV 1

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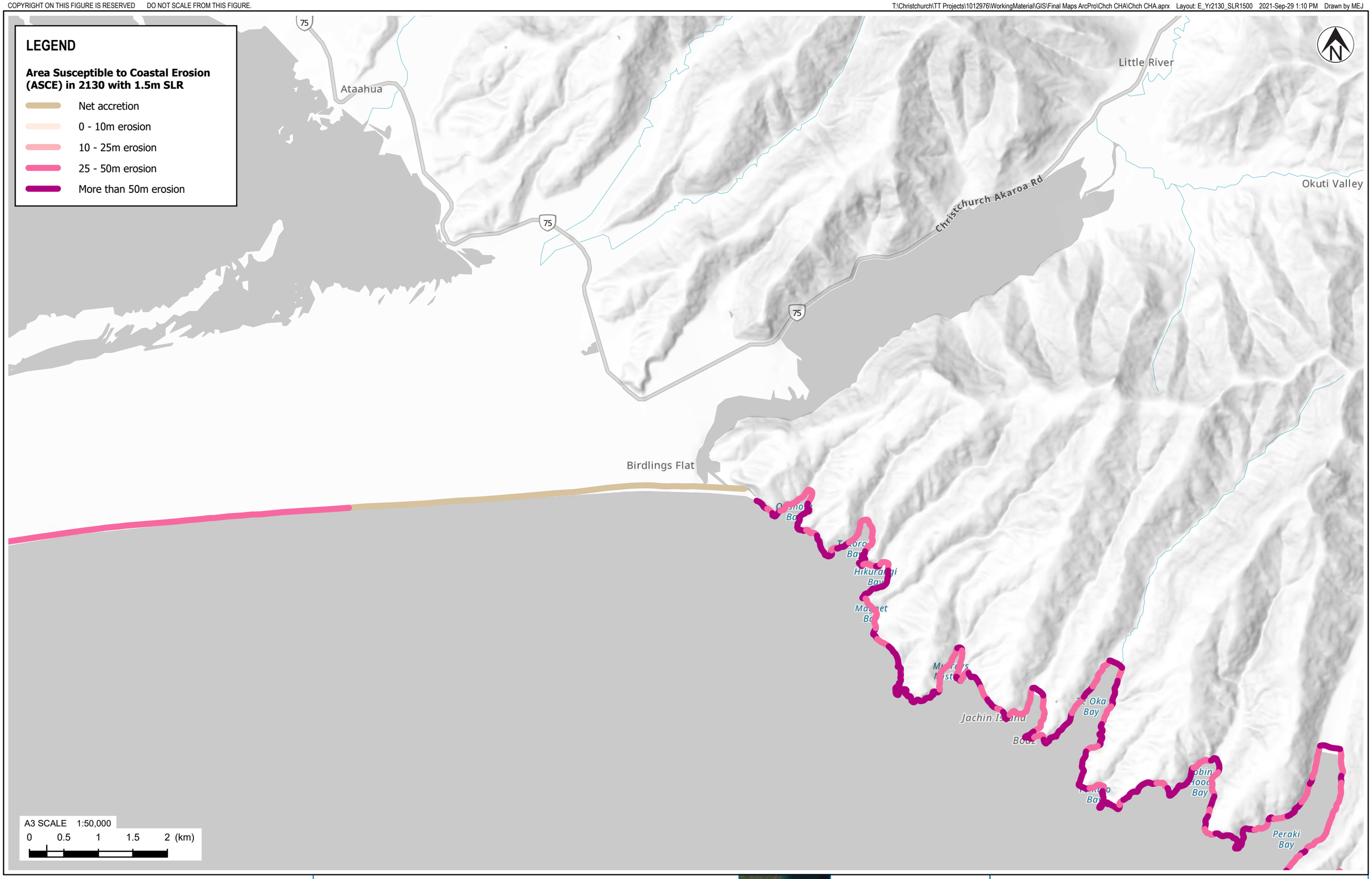

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CHECKED	RHAU	SEP.21
APPROVED	P. COCHRANE	29/09/21

CLIENT CHRISTCHURCH CITY COUNCIL
PROJECT COASTAL HAZARD ASSESSMENT
TITLE COASTAL EROSION ANALYSIS
SCENARIO: YEAR 2130 WITH 1.5M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 14B REV 1

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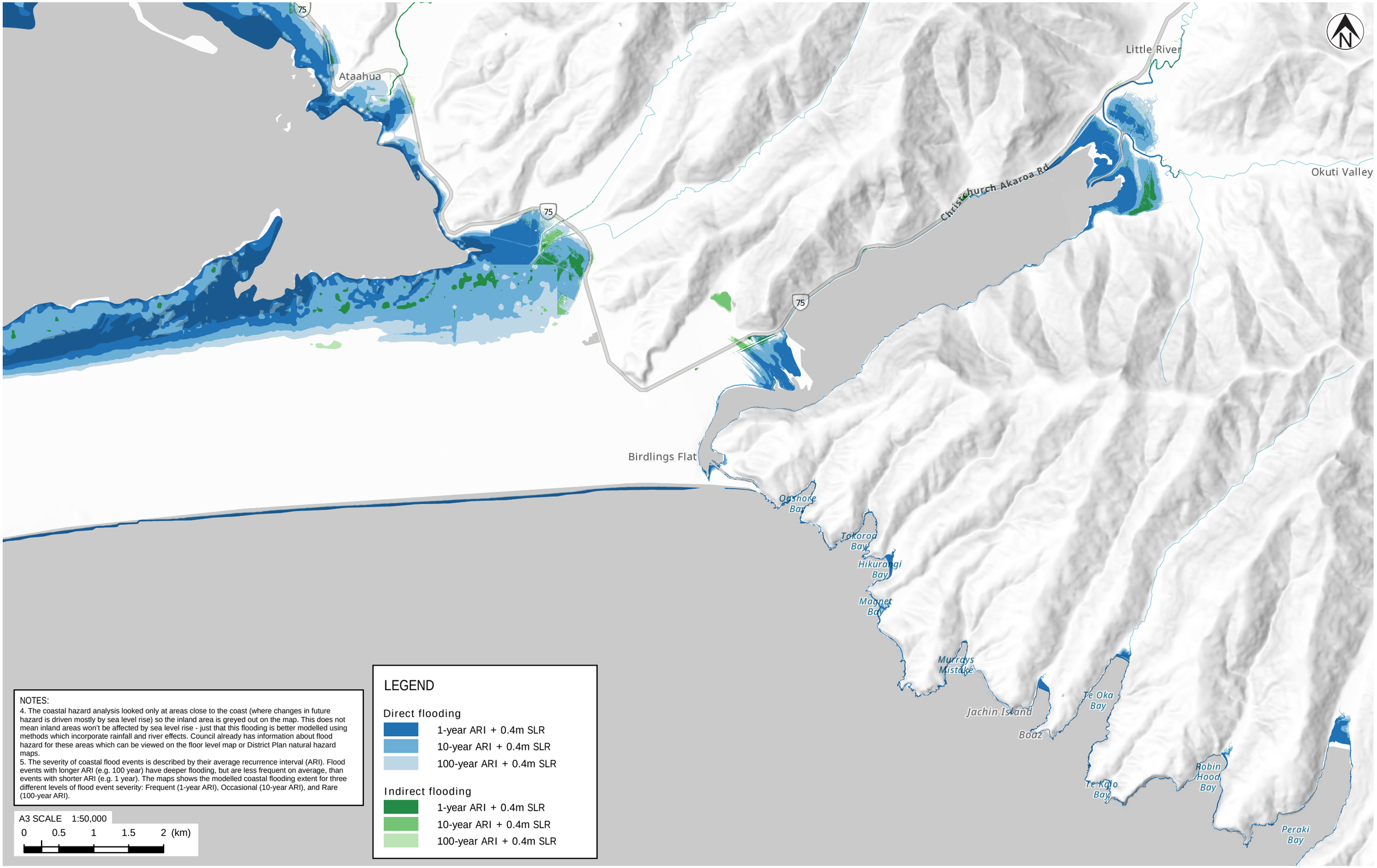

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A3 SCALE 1:50,000

0 0.5 1 1.5 2 (km)

LEGEND

Direct flooding
- 1-year ARI + 0.4m SLR
- 10-year ARI + 0.4m SLR
- 100-year ARI + 0.4m SLR

Indirect flooding
- 1-year ARI + 0.4m SLR
- 10-year ARI + 0.4m SLR
- 100-year ARI + 0.4m SLR

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 14C REV 1

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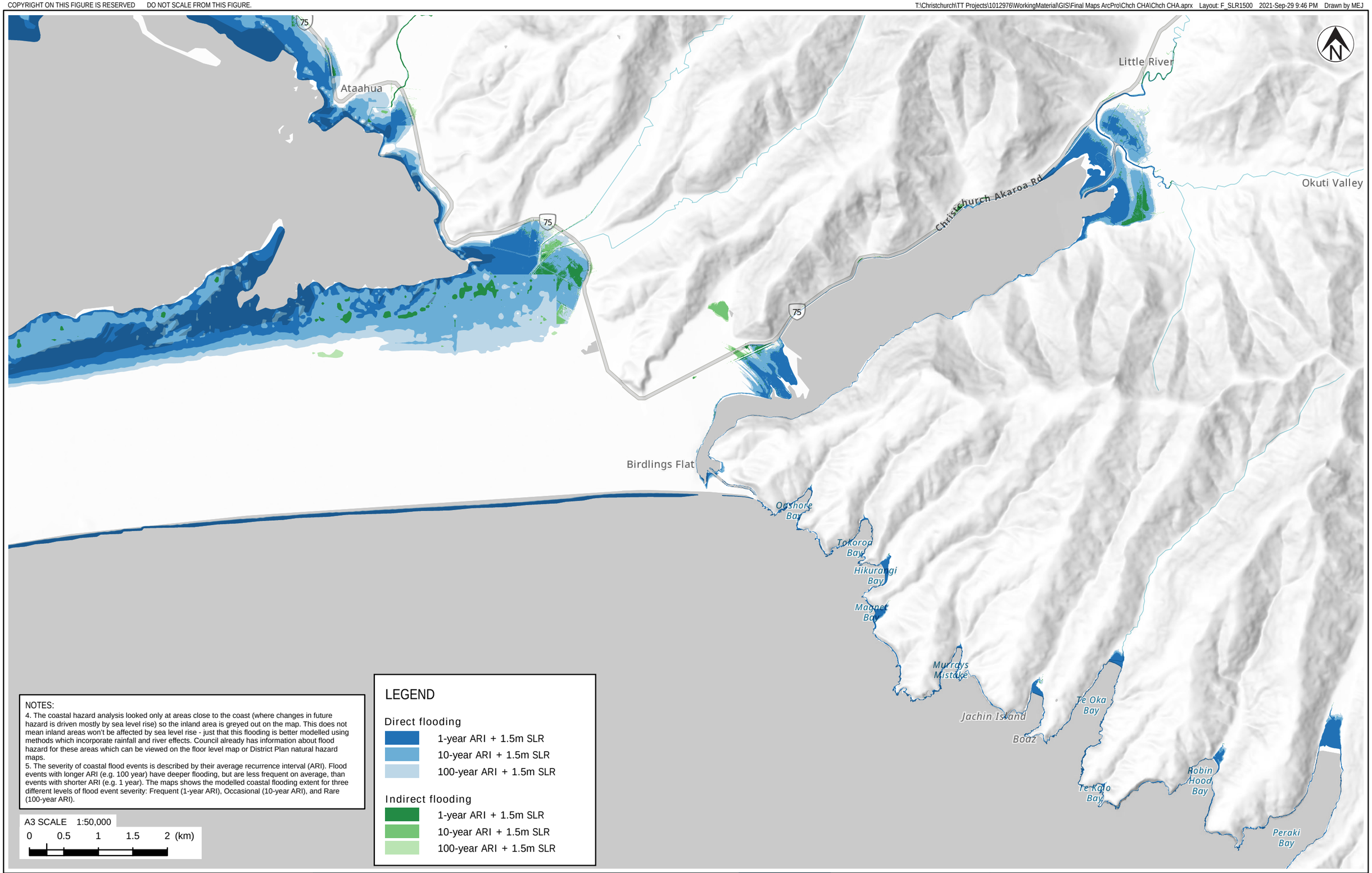

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A3 SCALE 1:50,000
0 0.5 1 1.5 2 (km)

LEGEND

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

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

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

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 14D REV 1

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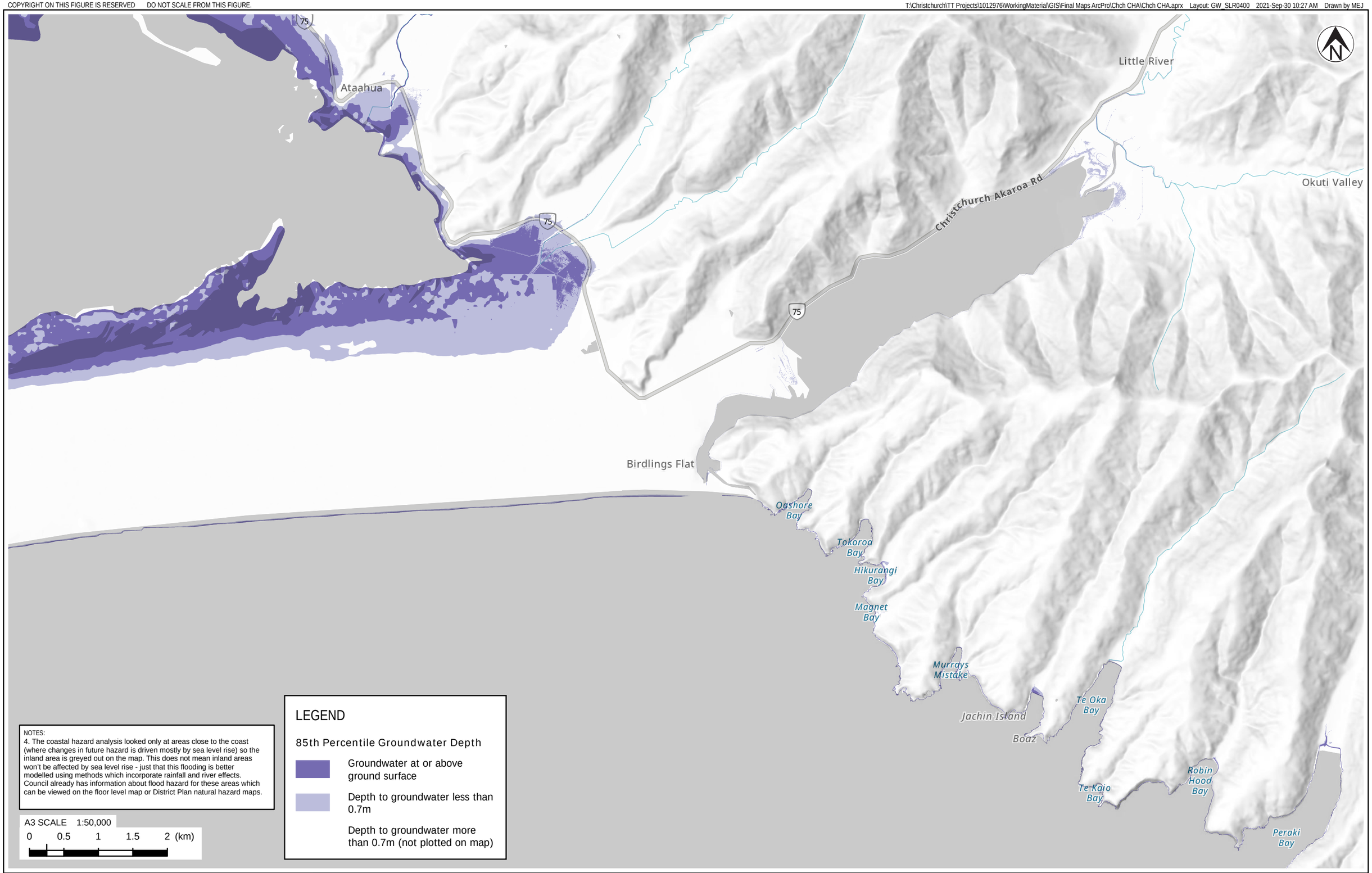

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A3 SCALE 1:50,000

0 0.5 1 1.5 2 (km)

LEGEND

85th Percentile Groundwater Depth

- Groundwater at or above ground surface
- Depth to groundwater less than 0.7m
- Depth to groundwater more than 0.7m (not plotted on map)

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NOTES:
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LOCATION PLAN

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

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 14E REV 1

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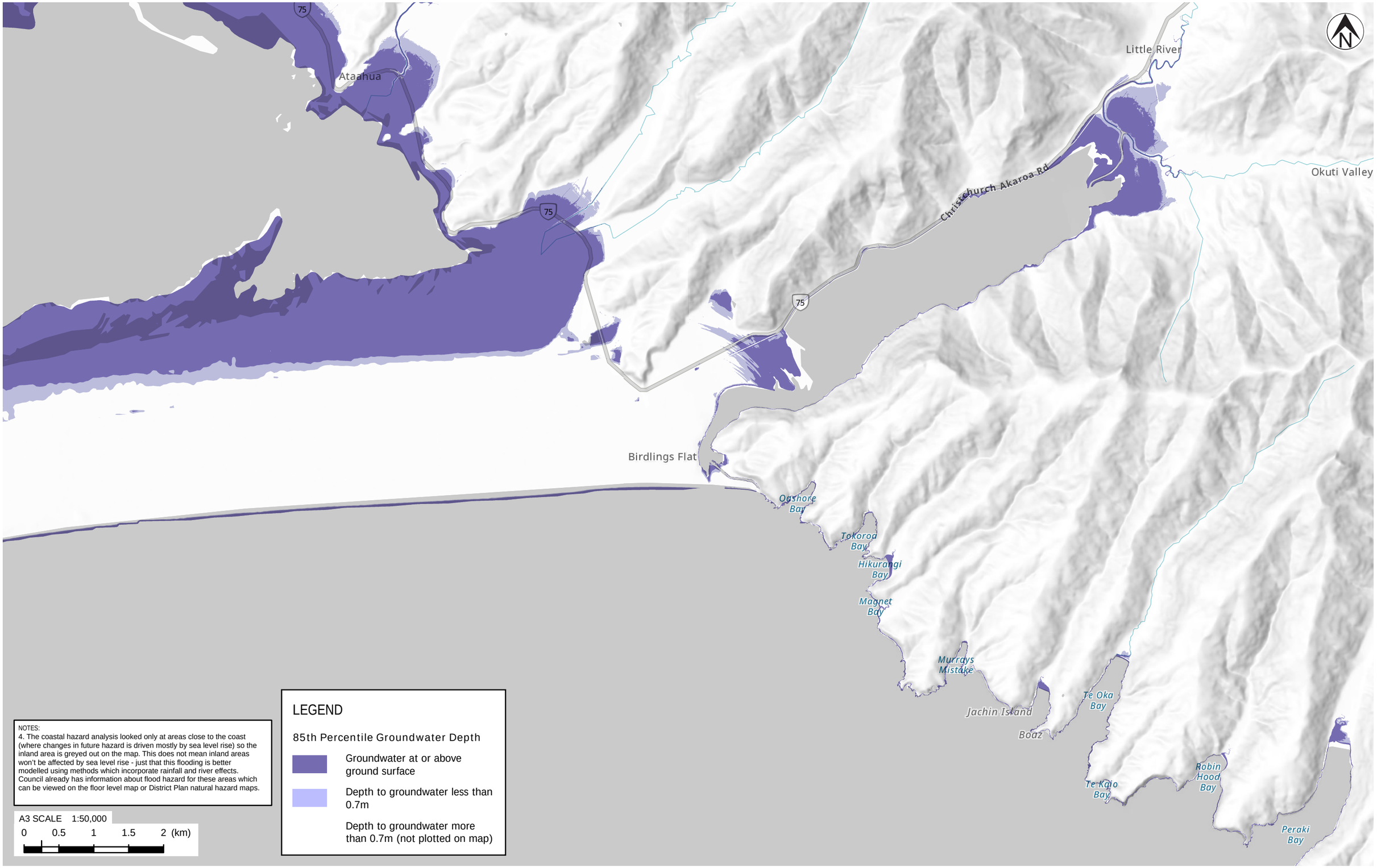

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A3 SCALE 1:50,000

0 0.5 1 1.5 2 (km)

LEGEND

85th Percentile Groundwater Depth

- Groundwater at or above ground surface
- Depth to groundwater less than 0.7m
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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 14F REV 1

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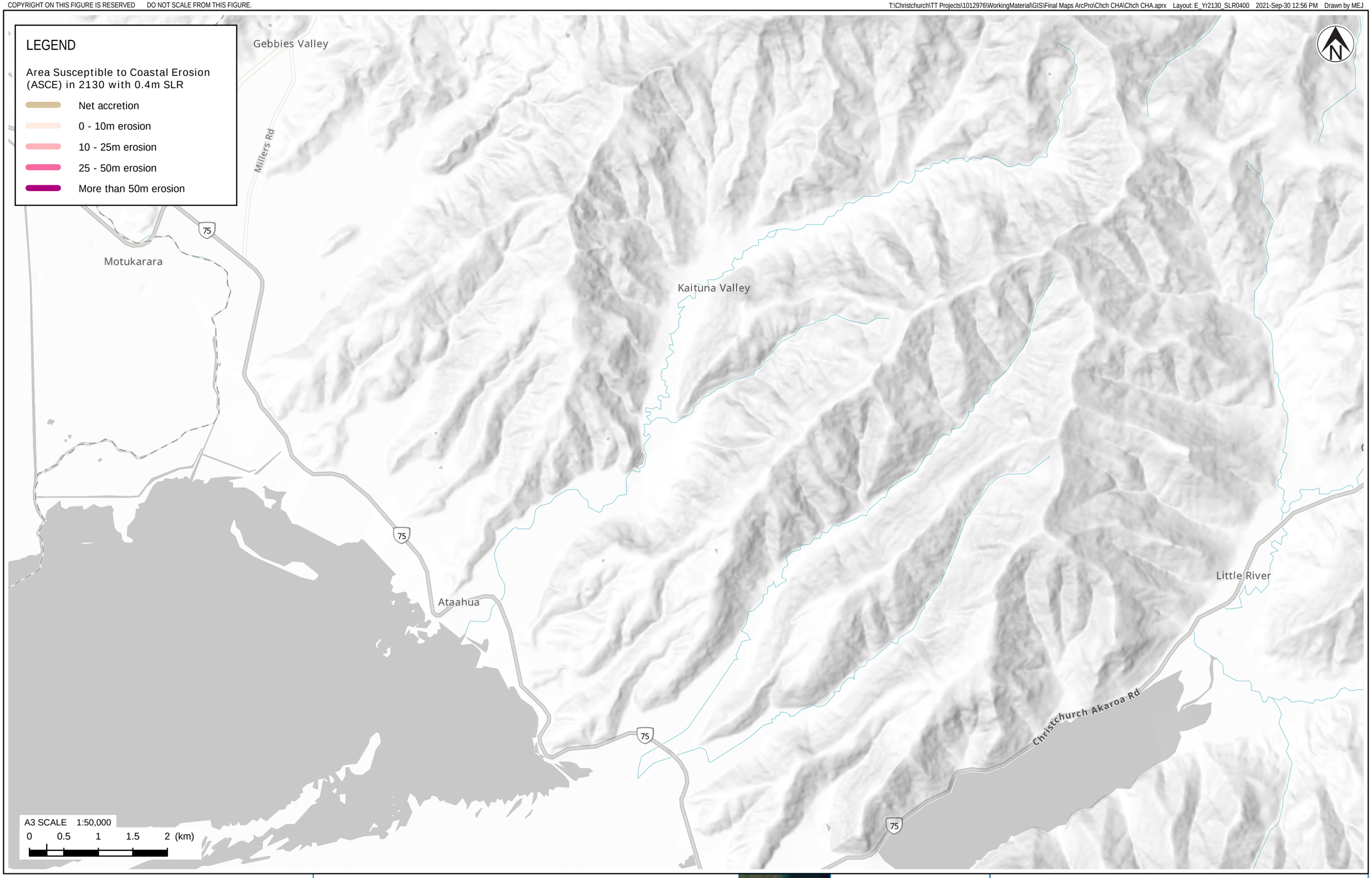

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LOCATION PLAN

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 15A REV 1

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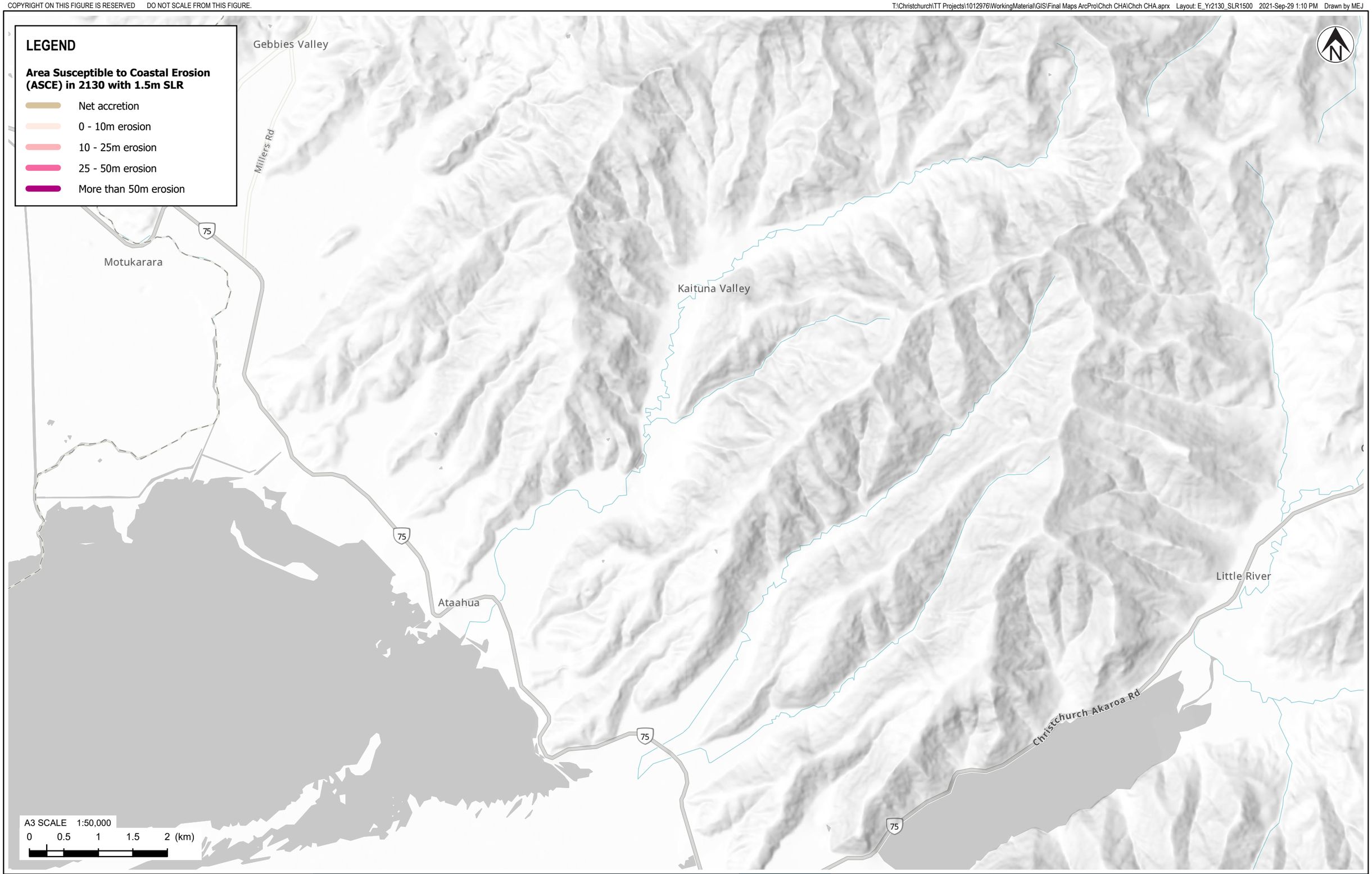

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 15B REV 1

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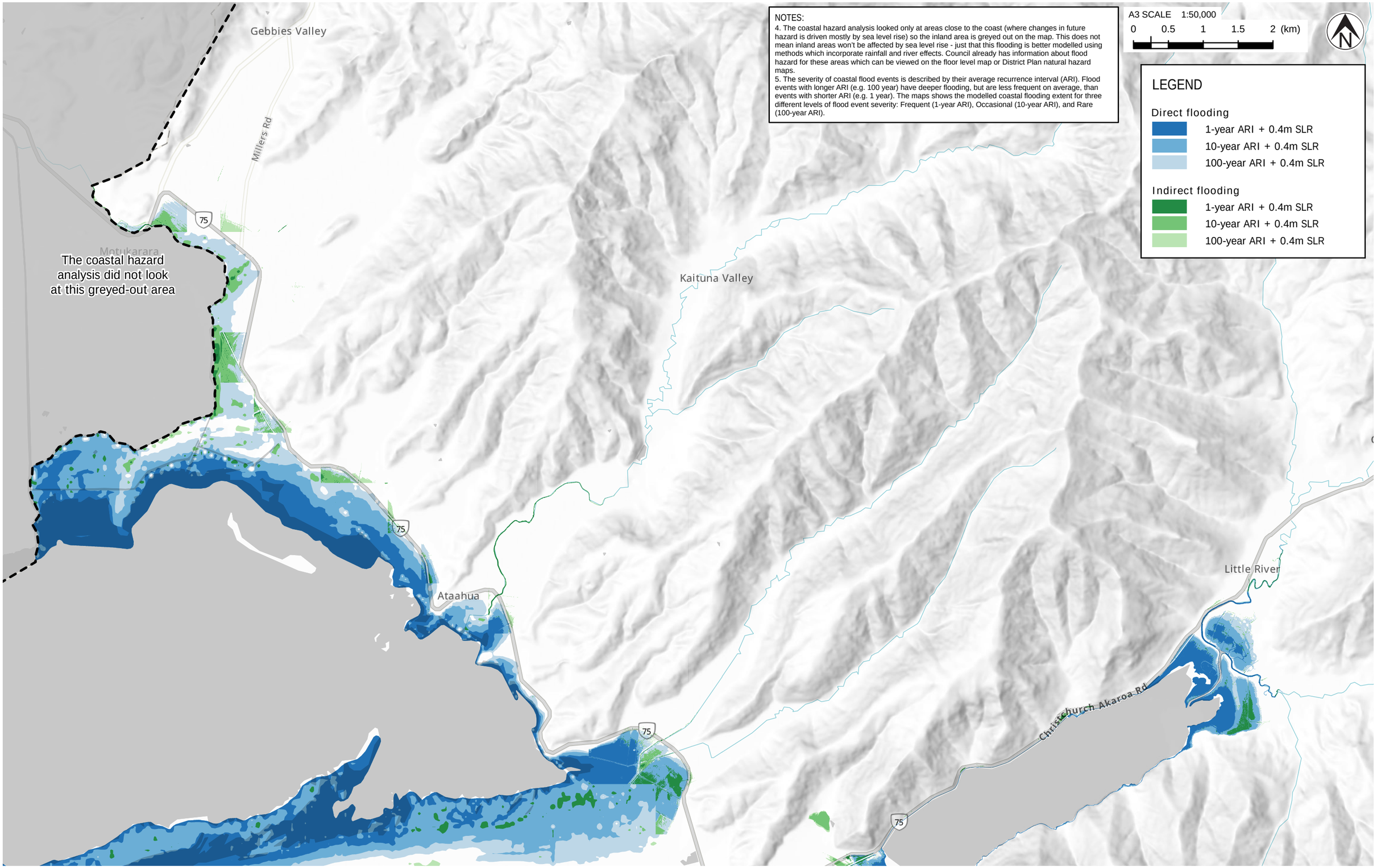

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 15C REV 1

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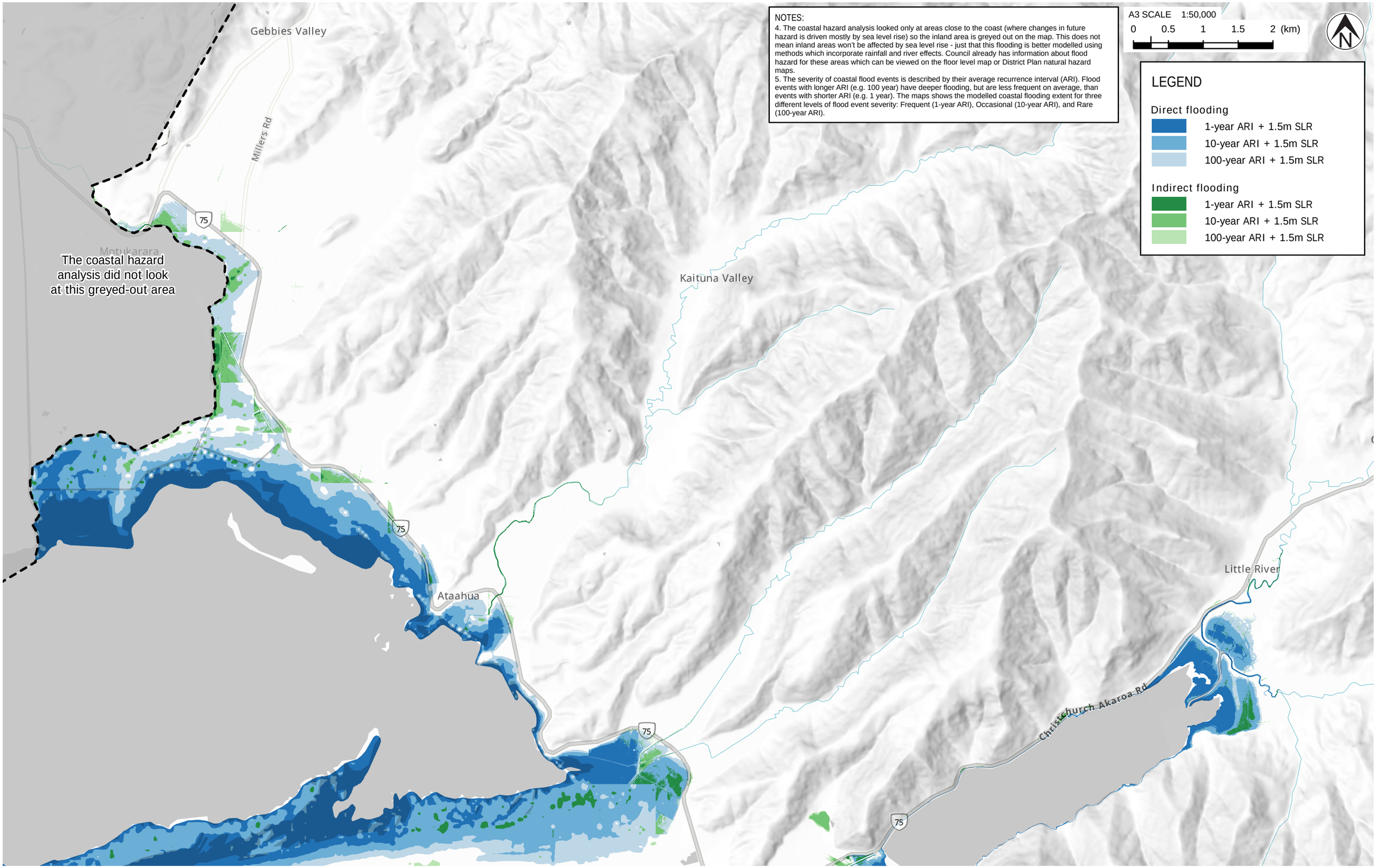

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APPROVED	P. COCHRANE	29/09/21

CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 15D REV 1

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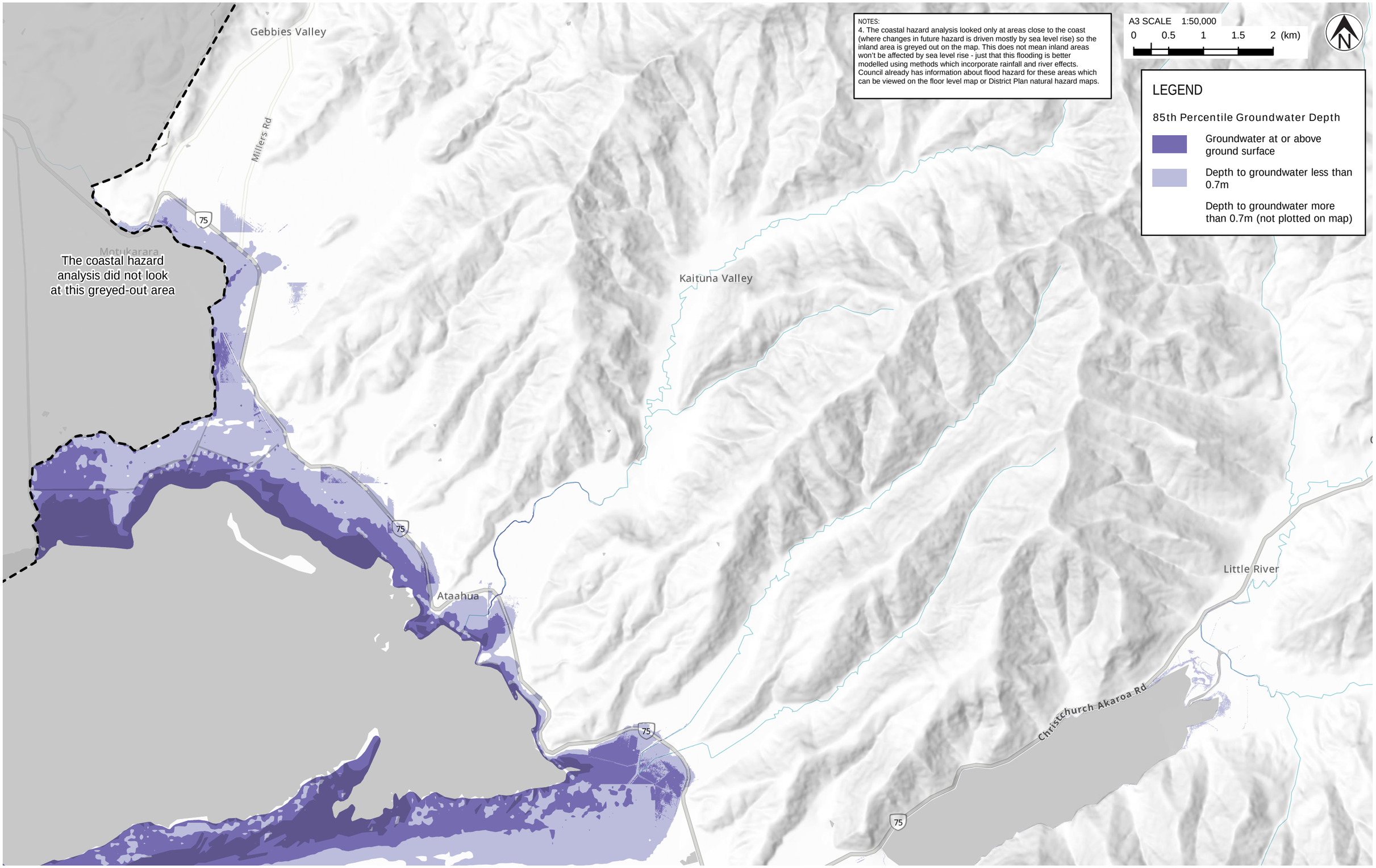

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 15E REV 1

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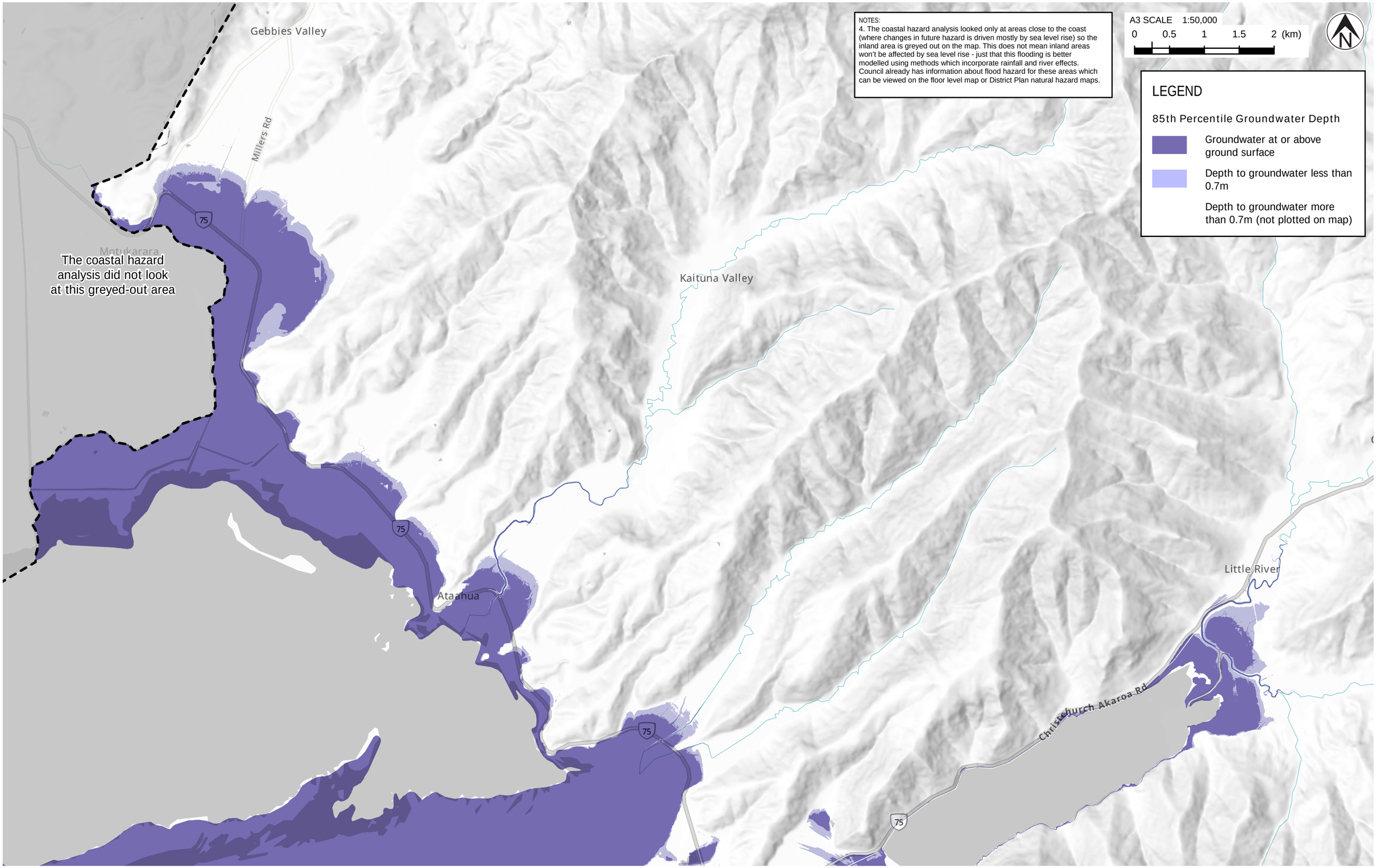

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 15F REV 1

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NOTES:
1. For detailed analysis areas, this map shows the P5 erosion distance (5% probability that erosion will be greater for this scenario).
2. For regional screening analysis areas, this map shows the upper envelope erosion distance.
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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 16A REV 1

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

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

SCALE (A3) 1:50,000 FIG No. FIGURE 16B REV 1

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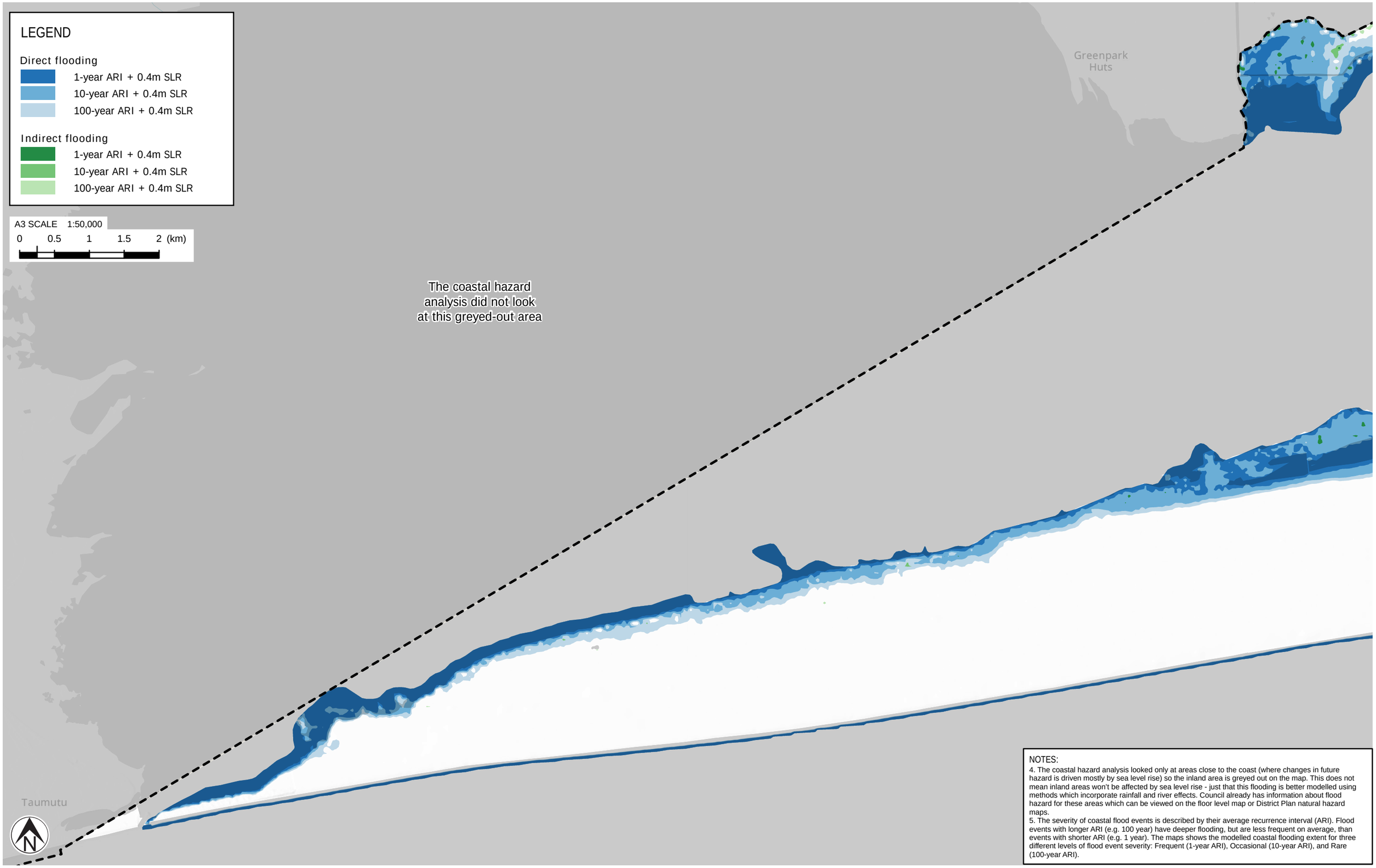

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CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 0.4M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 16C REV 1

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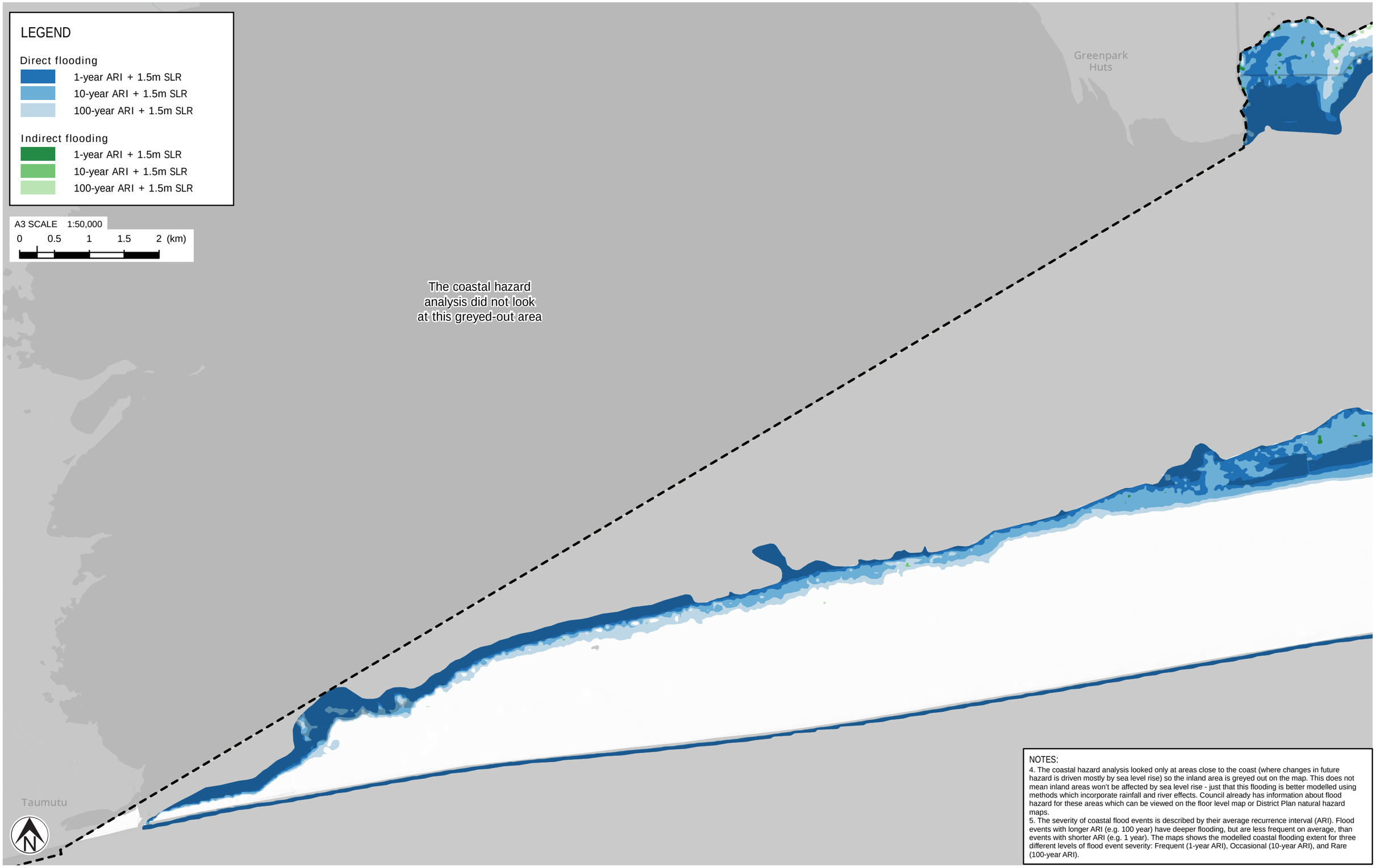

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CLIENT **CHRISTCHURCH CITY COUNCIL**
PROJECT **COASTAL HAZARD ASSESSMENT**
TITLE COASTAL FLOODING ANALYSIS
SCENARIO: 1.5M SEA LEVEL RISE
SCALE (A3) 1:50,000 FIG No. FIGURE 16D REV 1

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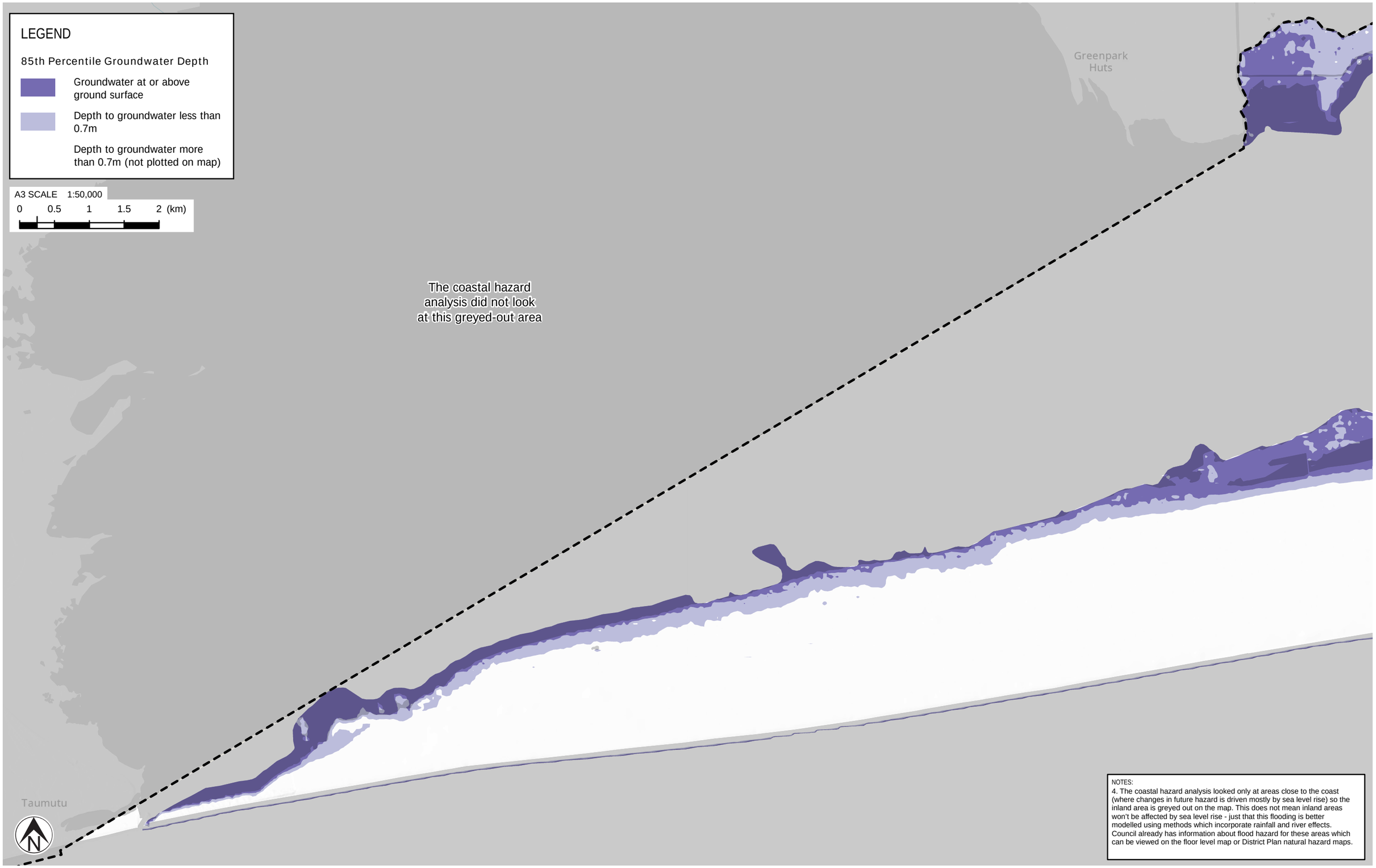

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CLIENT **CHRISTCHURCH CITY COUNCIL**

PROJECT **COASTAL HAZARD ASSESSMENT**

TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 0.4M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 16E REV 1

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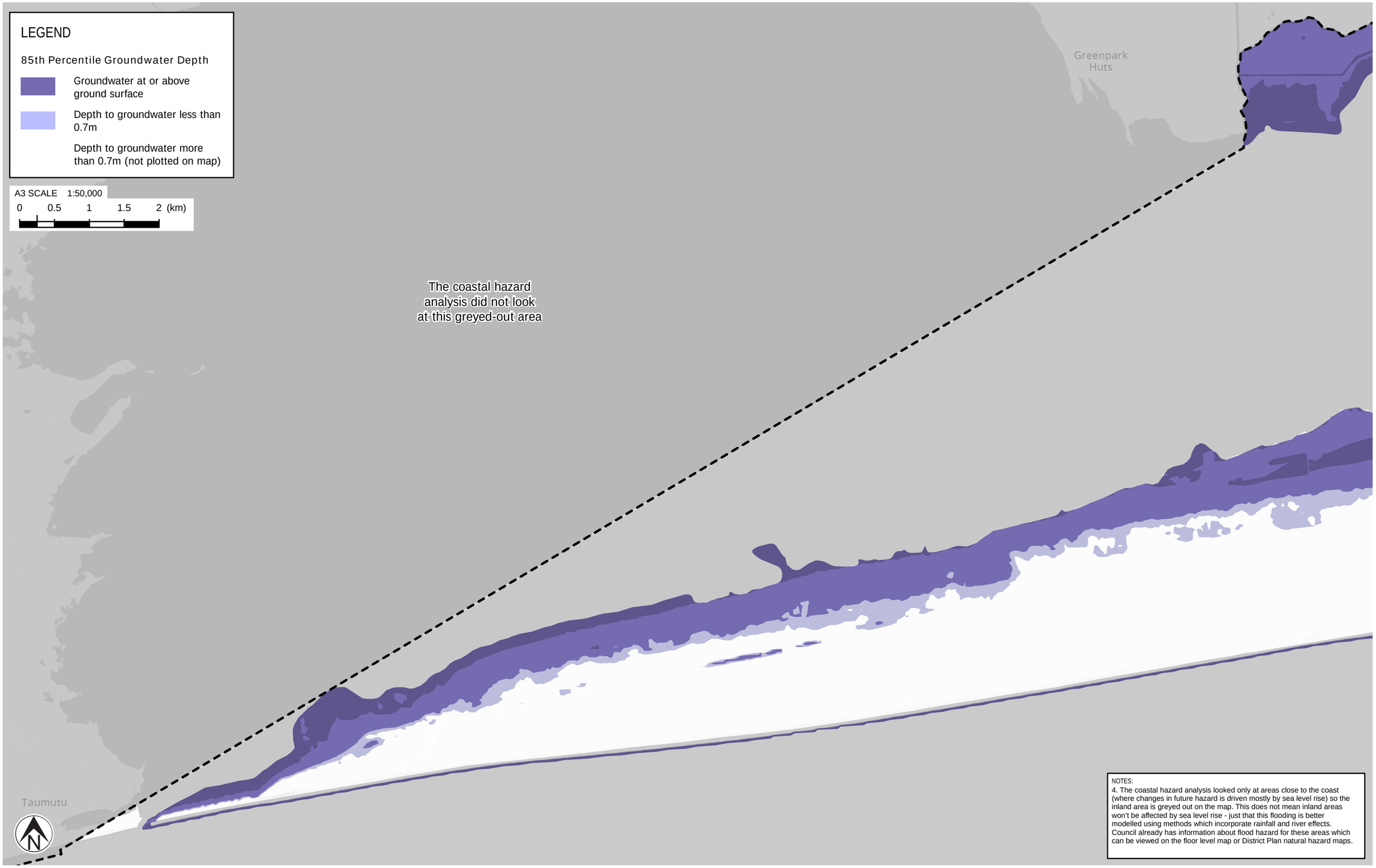

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TITLE COASTAL GROUNDWATER ANALYSIS SCENARIO: 1.9M SEA LEVEL RISE

SCALE (A3) 1:50,000 FIG No. FIGURE 16F REV 1

Appendix F: Addendum Report 2022

Tonkin+Taylor

25 November 2022
Job No: 1012976.2000

Christchurch City Council
PO Box 73011
Christchurch 8154

Attention: Tom Simons-Smith

Dear Tom

Coastal Hazard Assessment for Christchurch District
Addendum Report

1 Introduction

This report is an addendum to the Coastal Hazard Assessment for Christchurch District: Technical Report (T+T, 2021) and should be read in conjunction. This addendum report sets out the following:

- Additional clarifications related to consideration of the sediment supply/budget within future predicted ASCE (Areas Susceptible to Coastal Erosion) along the Christchurch open coast.
- Confirmation of discounting historic sea level rise for derivation of future ASCE.
- Updated coastal inundation levels for Akaroa and Wairewa (Lake Forsyth).

2 Clarifications on sediment supply scenarios

Potential climate change effects on sediment supply that may influence the long-term shoreline erosion component along the open coast are set out in Section 4.1.4.3 of T+T (2021). As per Table 3.1 (Section 3.5 of T+T, 2021), both a reduced and increased sediment supply scenario were assessed for the 2130 timeframe and high end (+1.5 m) relative sea level rise scenario. Both adjusted sediment supply scenarios consider changes in the amount of sediment discharged by the Waimakariri River and changes in sediment volumes transported southwards along the coast to the Christchurch Beaches.

The two adjusted sediment supply scenarios that have been considered by T+T (2021) were selected to stress-test potential large positive and negative changes in sediment supply on the long-term shoreline trends and resultant hazard extents. Therefore, the reduced sediment supply scenario was based on Scenario A (worst case; -11% supply) as per Table 3.9 of Hicks (2018), see Table 1 below, and the increased sediment supply scenario based on Scenario D (+28% supply).

Changes were applied by considering the percentage change in the net rate of shoreline shift (Δy) between the stress test scenario (Scenario A and D) and the 'status quo' (Scenario J) based on changes in sediment supply and profile characteristics as set out in Table 3.9 and using Equation (6) in Hicks (2018). The percentage changes in the net rate of shoreline shift (-20% for Scenario A and +37% for Scenario D) were applied to the individual long-term change (LT) distributions derived for

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each cell. Where one of the existing LT values within a distribution was 0 m/yr, the value was adjusted based on the other distribution values to maintain the overall distribution shape.

The most likely change in river load sediment supply scenario in combination with wave climate effects (i.e., Scenario G) has not been considered as part of the stress-test exercise as this resulted in similar shoreline changes as the status quo scenario (i.e., Scenario J).

Table 1: Impacts of future sand budget scenarios on alongshore-averaged shoreline shift rate, Waimakariri to Sumner (Table 3-9 from Hicks, 2018)

Scenario label	A	B	C	D	E	F	G	H	I	J	K
Scenario description	Worst case independent combination	Worst case independent combination	RCP8.5 83 percentile SLR	RCP8.5 83 percentile SLR, no wave change, inlet loss	RCP8.5 Median* + ebb-delta losses	RCP8.5 Median*	RCP6.0	RCP2.6*	Landslide doubles river load	Status quo	Landslide doubles river load, no CC
River load scenario	Reduced 8% by CC, 3% by irrigation	Reduced 8% by CC, 3% by irrigation	Increased 28% by CC, reduced 3% by irrigation	Increased 28% by CC, zero irrigation effect	Increased 28% by CC, reduced 3% by irrigation	Increased 28% by CC, zero irrigation effect	Increased by 9%	Reduced 8% by CC, 3% by irrigation	Load doubled	Baseline	Load doubled
Waves scenario	B2	A2	A2	Baseline	A2	A2	B2	Baseline	A2	Baseline	Baseline
Sea-level rise by 2120 (m)	1.36	1.36	1.36	1.36	1	1	0.63	0.55	1.36	Baseline	Baseline
River sand load to tidal reach (million m^3/yr)	0.664	0.664	0.933	0.955	0.933	0.955	0.813	0.664	1.492	0.746	1.492
River sand load to coast (million m^3/yr)	0.653	0.653	0.922	0.944	0.924	0.947	0.807	0.659	1.481	0.744	1.490
Nearshore trap efficiency (%)	36%	36%	36%	36%	36%	36%	36%	36%	36%	36%	36%
Proportion moved south (%)	0.610	0.637	0.637	0.724	0.637	0.637	0.600	0.680	0.637	0.680	0.680
Losses to A-H ebb-delta (million m^3/yr)	0.015	0.015	0.015	0	0.015	0	0	0	0.015	0	0
Shore span (km)	20.65	20.65	20.65	20.65	20.65	20.65	20.65	20.65	20.65	20.65	20.65
Average profile height (m)	15.10	14.80	14.80	15.35	14.75	14.75	14.84	14.70	14.80	15.07	15.07
Average profile width (m)	758	719	719	789	710	710	724	700	719	751.5	751.5
Sea-level rise rate at 2120 (m/yr)	0.015	0.015	0.015	0.015	0.011	0.011	0.007	0.005	0.015	0.002	0.002
Derived Shore shift rate at 2120	-0.341	-0.288	-0.086	0.005	0.117	0.183	0.228	0.275	0.333	0.485	1.072
SLR rate for a stable shore (m/yr)	0.008	0.009	0.013	0.015	0.013	0.015	0.012	0.011	0.022	0.012	0.023

3 Discounting historic sea level rise

The response to sea level rise (SL) component for the open coast is described in Section 4.1.5 of T+T (2021). This component assesses the potential for the shoreline retreat from a baseline (i.e., 2018) due to future sea level rise (refer to Equation A1) . As the long-term (LT) component already implicitly includes historic sea level rise (i.e., by extrapolating derived historic trends into the future), historic sea level rise has been discounted from the future sea level rise in order to assess the 'additional' response to future sea level rise.

$$SL = \frac{B+h_*}{L_*} * (SLR_{Future} - SLR_{Historic}) \qquad \text{(Equation A1)}$$

In which:

- SL = horizontal movement of the beach face due to sea level rise.
- B = vertical distance between dune crest and Mean Low Water Springs (MLWS).
- h_* = vertical distance between MLWS and bed level at depth of closure (d_i).
- L_* = horizontal distance between dune crest and depth of closure.
- SLR_{Future} = future sea level rise over period of interest.
- $SLR_{Historic}$ = historic sea level rise over same period/duration.

4 Updated coastal inundation levels for Akaroa and Lake Waiwera

Extreme storm tide water levels (i.e., excluding wave effects) for the Akaroa Harbour are set out in Table 7.2 of T+T (2021). These levels were inadvertently offset from the Sumner gauge instead of the Lyttelton gauge as described in the report. These water levels have been updated and offset from the Lyttelton gauge. The updated present-day extreme storm tide levels are set out in the updated Table 7.2 (see below). The updated static inundation levels for Akaroa Harbour are set out in the updated Appendix D Table 11 (see below).

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

Site	ARI							
	1 yr	**2 yr**	**5 yr**	**10 yr**	**20 yr**	**50 yr**	**100 yr**	**200 yr**
Akaroa Harbour	1.55	1.60	1.65	1.69	1.73	1.78	1.82	1.86

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

Return Period	Present Day	Relative sea level rise (m)							
		0.2	**0.4**	**0.6**	**0.8**	**1.2**	**1.4**	**1.5**	**2**
1 year ARI	1.8	2.0	2.2	2.4	2.6	3.0	3.2	3.3	3.8
10 year ARI	2.0	2.2	2.4	2.6	2.8	3.2	3.4	3.5	4.0
100 year ARI	2.1	2.3	2.5	2.7	2.9	3.3	3.5	3.6	4.1

The extreme static inundation levels for Wairewa (Lake Forsyth) are set out in Appendix D Table 14 of T+T (2021) and are a combination of the extreme storm tide water levels (Table 7.5 of T+T, 2021) and wave effects. Section 7.3.2 of T+T (2021) describes that 0.1 m wave set up has been adopted for Waiwera, however this wave setup was inadvertently omitted from Appendix D Table 14. The updated static inundation levels for Wairewa are set out in the updated Appendix D Table 14 below.

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

Return Period	Present Day	Relative sea level rise (m)
1 year ARI	2.3	N/A
10 year ARI	2.7	N/A
100 year ARI	2.9	N/A

5 Applicability

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

Tonkin & Taylor Ltd

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
.....	.....
Patrick Knook Senior Coastal Engineer	Peter Cochrane Project Director

Report technically reviewed by:

Dr Tom Shand – Technical Director Coastal Engineering

Mike Jacka – Project Manager

25-Nov-22
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