REPORT

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Document Control

Title: Waitangi Wharf Coastal Process Monitoring Report					
Date	**Version**	**Description**	**Prepared by:**	**Reviewed by:**	**Authorised by:**
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Distribution:

Waka Kotahi New Zealand Transport Agency	1 copy
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Table of contents

1 Introduction

The Chatham Islands Port Limited (CIPL) obtained resource consents (CIC/2015/02) for the construction of an upgrade of Waitangi Wharf and related activities in 2015. The project was funded by the Department of Internal Affairs (on behalf of the New Zealand Government) and was designed and constructed by the Memorial Park Alliance (MPA) which consisted of the New Zealand Transport Agency (NZTA), Downer Construction, HEB, AECOM and Tonkin & Taylor Ltd. (T+T).

The project included reclamation and dredging activities which resulted in changes to the pre-construction coastal processes. The assessment of coastal processes (Appendix E) prepared as part of the resource consent application concludes that the wharf construction will not result in significant changes or adverse effects on coastal processes. To confirm this assessment and determine if mitigation measures are required, a Coastal Processes Monitoring Plan (CMP) was prepared (Appendix D).

The main areas of interest for coastal processes monitoring is the beach area in front of the Waitangi Hotel and Aotea Fisheries Factory and the area around the mouth of the Nairn River and north of the river.

The CMP set out procedures for the collection of relevant information and the analysis and reporting of results. The monitoring programme set out in the CMP includes:

- Photo-point monitoring
- Beach profile surveying
- Wave data analysis
- Satellite imagery analysis (shoreline analysis)

The granted resource consent requires ongoing beach monitoring in accordance with the above CMP. Following completion of the wharf, T+T took over the monitoring regime required from MPA.

This report outlines the results from the 2020 round of coastal monitoring and includes:

- A summary of the wave regime over the previous 12 months and commentary in respect the "normal" wave regime
- A summary of the photographic monitoring undertaken key observations and changes in the beach
- A summary of beach profile monitoring survey results
- Update on shoreline position if any new relevant satellite imagery has become available
- Comment on the monitoring results in comparison with the Summary and Conclusion section of the Waitangi Wharf Upgrade Coastal Processes Report.
- If necessary, suggestions on adaptive management such as additional sand transfer, coastal protection works or relocation of assets.

2 Beach monitoring description

2.1 Monitoring programme

The following programme has been undertaken to monitor potential changes in shoreline characteristics:

Table 1: Beach monitoring programme

Name	Description	Monitoring Requirement	Frequency		
			Baseline	**During Capital**	**Following Capital**
Photo-point Monitoring	Photographs taken from fixed locations and aspects.	Visually assess beach level change or fine sediment deposition.	June 2016	2 weekly	Annually for 2 years (until 2021) then bi-annually for remainder of consent
Beach Profile Survey	Beach profile survey from established benchmark[2].	Quantifies changes in profile geometry and/or location	June 2016	6 monthly	Annually for 2 years (until 2021) then bi-annually for remainder of consent
Wave Data Analysis	Wave climate data from NOAA Wavewatch III global numerical wave model at an output location 75 km offshore of Waitangi Bay.	Provide indication of ocean conditions occurring between surveys (i.e. magnitude and frequency of storms).	June 2016	Annually	Annually until 2021 then 5 yearly for the remainder of the consent[3]
Shoreline Analysis	Digitise and compare shoreline positions from aerial photographs/satellite imagery	Determines any changes in shoreline position.	June 2015	Annually (or as aerial photographs/satellite imagery become available if longer than this) to 2021 then 5 yearly for the remainder of the consent[3].	

[1]Monitoring frequency is broken into three stages,

- **Baseline** before works began; **During** capital works project; **Following** capital works project

[2]Surveys should be referenced to the benchmark and consist of horizontal and vertical offsets across the profile from the benchmark to the water edge at low tide. Acceptable survey methods include RTK GPS, theodolite, level and staff. Staff and tape and visual estimate are not acceptable.

[3]Shorter period if agreed by Council

2.2 Locations

Monitoring has been undertaken at the locations shown in Figures 1 and 2 for the types of monitoring shown in Table 2 below.

Table 2: Beach monitoring locations

Location		Photo-point monitoring	Beach profile survey
1.	Western end of Town Beach	✓ (P1)	
2.	Toe of boat ramp to south of Aotea Fisheries Factory	✓ (P2)	✓ (T2)
3.	In front of Waitangi Hotel accommodation block	✓ (P3)	
4.	75m east of Waitangi Hotel public bar	✓ (P4)	✓ (T3B)
5.	Eastern abutment Nairn River bridge	✓ (P5)	
6.	125m east of the eastern abutment Nairn River bridge	✓ (P6)	✓ (T4)
7.	710m east of the eastern abutment Nairn River bridge	✓ (P7)	✓ (T5)
8.	1500m east of the eastern abutment Nairn River bridge	✓ (P8)	✓ (T6)
9.	Northern end of diesel storage compound		✓ (T1)
10.	Boundary between Aotea Fisheries factory and the Waitangi Hotel		✓ (T3A)

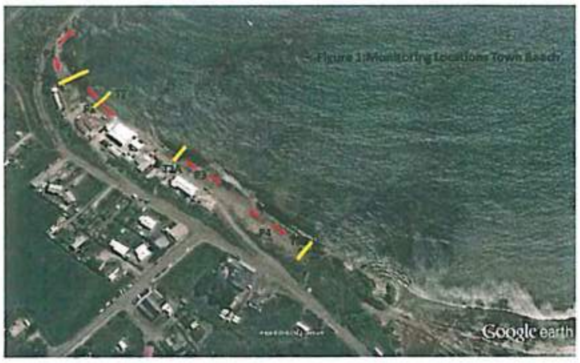

Figure 1: Monitoring Locations Town Beach


Figure 2: Monitoring Locations North of Nairn River

3 Monitoring results

3.1 Purpose

The purpose of the monitoring undertaken is to confirm that the conclusions stated in the Coastal Processes Report undertaken by MPA during the design phase stating that the capital works undertaken will not result in significant changes or adverse effects on coastal processes is correct. This report outlined the construction of the wharf and breakwater may result in some alterations to the beach processes within Waitangi Bay due to an altered wave climate from a north-west swell direction in Waitangi Bay namely:

- The swell wave climate was expected to be reduced by 20-80% at the western end of the bay (Town Beach) resulting in less sediment transport in front of Waitangi Town.
- The swell wave climate was likely to slightly increase up to 5% at the more northern end of Waitangi Beach (1-2km north-east of Nairn River) resulting in slightly increased erosion rates from those historically experienced (0.1-0.3m/year since 1969).

These actual effects were not expected to be noticeable given the background erosion rate in the area.

To combat the potential changes in sediment transport in the system and to increase the stability of the perched beach west of the Nairn River (Town Beach), dredged sand material was placed in this area to nourish the beach. It was anticipated before works began that approximately 3000m^3 material be placed along 120m of the shoreline started from the western end of the bay (refer Figure 3-1). This material was anticipated to be gradually transported eastward along the bay. It is noted that during construction dredged material was also placed further eastward of this location, extending along the Town Beach in front of the fish factory. Approximately 3300m^3 dredged sand was placed on the Town Beach during construction of the Wharf.

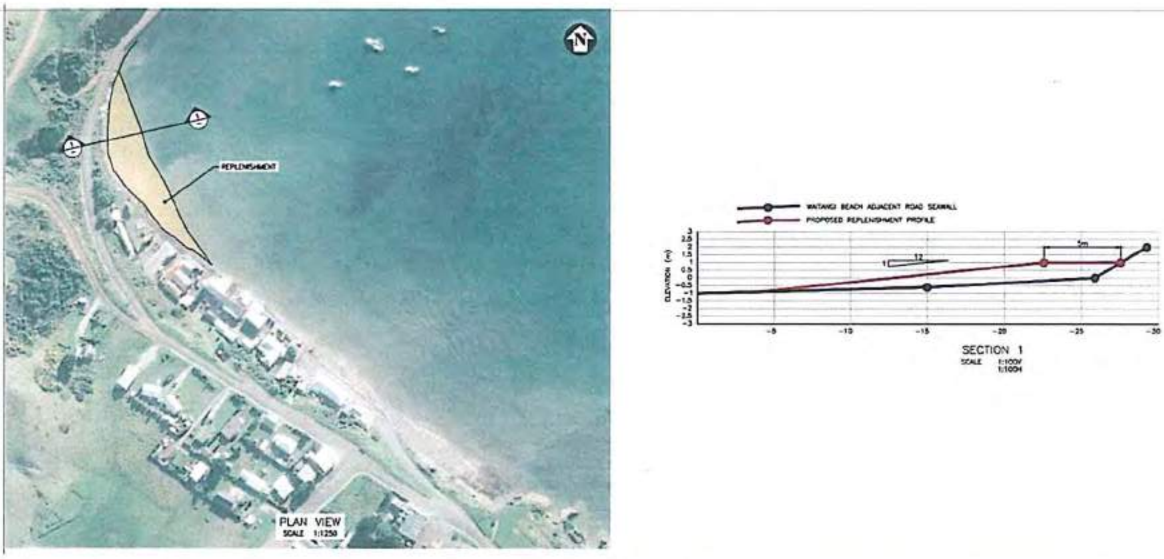

Figure 3-1: Initial beach sand nourishment design extent and profile concept (note sand was also placed further east along Town Beach during construction)

The purpose of each monitoring requirement is outlined in Table 1.

3.2 Photo-point monitoring

The photo-point and visual monitoring undertaken through the construction process and annually following completion of construction. The December 2020 photo-point monitoring round was undertaken between 6am and 9am on 8 December 2020. Photos were taken around low tide with overcast sky and negligible swell. In general these photos show only minor changes to the majority of the beach profile since beginning of works. Refer Appendix A for photos taken.

3.2.1 Town Beach

The sand levels at the far western end of Towns Beach, west of the concrete pile groyne appear unchanged between the December 2019 and 2020 monitoring rounds. There is erosion along the access road edge which will likely continue unless protected. The higher beach levels at this location (compared to pre-nourishment) will be somewhat aiding in mitigating the rate of erosion. Sand build-up on the western side of the groyne and reduction of the sand levels on the eastern side was noted during the 2018 monitoring rounds and this difference in beach levels was also evident during the 2020 monitoring round (refer Figure 3-2).

Figure 3-2: Beach photo-point monitoring at point 1 Dec 2018 (left) and Nov 2019 (right)

In 2019, the sand levels were noted to be visibly lower at the upper beach location at the western end of Town Beach adjacent to the boat haul out area and new ramp extension than in 2018. The beach levels at this location are currently similar to those in 2019 with the sand levels not rebounding. These changes are clear in photos taken at monitoring point P1 and P2 as well as in the survey profile at location T2 (refer Section 3.3). The gravel/sand interface at the top of the beach has retreated landward since 2019 see Figure 3-3. This gravel was exposed between 2018 and 2019.

Figure 3-3: Beach photo-point monitoring at point 2 Dec 2019 (left) and Dec 2020 (right). Red line shows gravel/sand interface.

Lowering of sand levels at this location was also noted in the December 2018 monitoring reports. This trend is ongoing and is likely the result of ongoing easterly transport of the dredged material placed here during construction.

There was evidence of displacement of armour rock along the edge of the access track at this location, this is also evident in survey profile T1 (refer Section 3.3.1).

The main difference noted in previous monitoring rounds since construction was to the sand levels at monitoring point 3 along central and eastern end of Town Beach. Here the underlying tuff rock was seen to be exposed during the latter half of 2016, then being covered by sand throughout 2018-2019 (refer Figure 3-4). This difference appeared to be a direct result of the dredged sand placed west of this location in front of the fish factory in the latter half of 2016 being transported along the length of beach in front of the Hotel. The volume of beach sand placed on the Town Beach averaged over the area of the western half of the beach (including the area in front of the fish factory) equates to an approximate average depth of 0.5m. This is approximately equal to the maximum depth of sand surveyed following the sand placement indicating a large volume of the placed material was relatively rapidly incorporated into the beach system and dispersed along the beach eastward.

It is still not clear what the natural beach fluctuation will be along this length of beach based on the monitoring to date as there is only two years of monitoring since nourishment was completed. However, the Tuff rock in front of the hotel is still not visible during the 2020 monitoring round (refer Figure 3-4) which indicates the sand has withstood over four year's climatic cycles.

Figure 3-4: Beach photo-point monitoring at point 3 June 2016 (left) and Dec 2020 (right)

Even though the Tuff rock in front of the Hotel is still covered, the beach levels to the west of monitoring point 3 have been lowering since 2017, exposing more gravel and Tuff rock seaward of the fish factory seawall (refer Figure 3-5). This ongoing trend is also evident between 2019 and 2020 with more Tuff rock exposed this year (refer Figure 3-17).

Figure 3-5: Exposed Tuff rock and gravel in front of the fish factory, Feb 17 (left) and Dec 20 (right).

3.2.2 North of Nairn River

There are no obvious areas of increased beach or dune erosion visible in the monitoring photos along the length of beach 1-2km to the north of the Nairn River which has experienced background erosion since 1969. There was, however, signs of dune accretion around P6 and P7. Dunal vegetation was noted establishing further seaward than the dune toe (refer Figure 3-6) which is evidence of a less disturbed environment along the dune edge. This could be the result of several factors including:

a Reduced storminess resulting in less erosion.

b Increased beach levels seaward of the dunes providing some additional buffer from wave runup.

c Increased sediment supply in the beach system at this location leading to increased deposition and dune accretion.

Figure 3-6: Dunal vegetation becoming established at the dune toe at P7

This is in line with a general dunal accretional trend noted at T4 and T5 discussed further in Section 3.3. There are also some additional minor changes noted along the beach north of the Narin River in the survey profile monitoring section.

3.3 Beach profile survey

The beach profile survey data undertaken since June 2016 is included in Appendix B. There are changes in the beach survey profiles over this time, however, long term trends are difficult to establish at this stage due to the relatively short monitoring period to date (6 years total, 3 years since construction completion).

3.3.1 Town Beach

Throughout construction, accretion occurred along Towns Beach due to the import and easterly migration of dredged sand placed on the beach during the works. However, since this placement stopped, the beach levels along the western half of Towns Beach at the location of monitoring profiles T1 and T2 have lowered (refer Figure 3-7 and Figure 3-8).

At monitoring location T1, beach levels are currently very similar to those surveyed in November 2019 with maximum lowering of 60mm recorded 5m offshore from the toe of the access ramp. The beach level at this location is currently up to 200mm lower than pre-construction levels. Beach level lowering at this location was also noted in the 2018 and 2019 monitoring reports. As stated in those reports, this drop in levels is expected due to increased reflection as a result of rock armour being installed seaward of its previous location to allow for the access track and the typical easterly migration trend of sediment at this end of the beach. As the 2020 profile is very similar to the 2019 profile, it is possible this profile is close to reaching an equilibrium position, it will continue to be monitored in future survey rounds.

The displacement of rock armour along the crest of the access track at this location mentioned in Section 3.2 is also evident in the T1 survey profile (refer Figure 3-7).

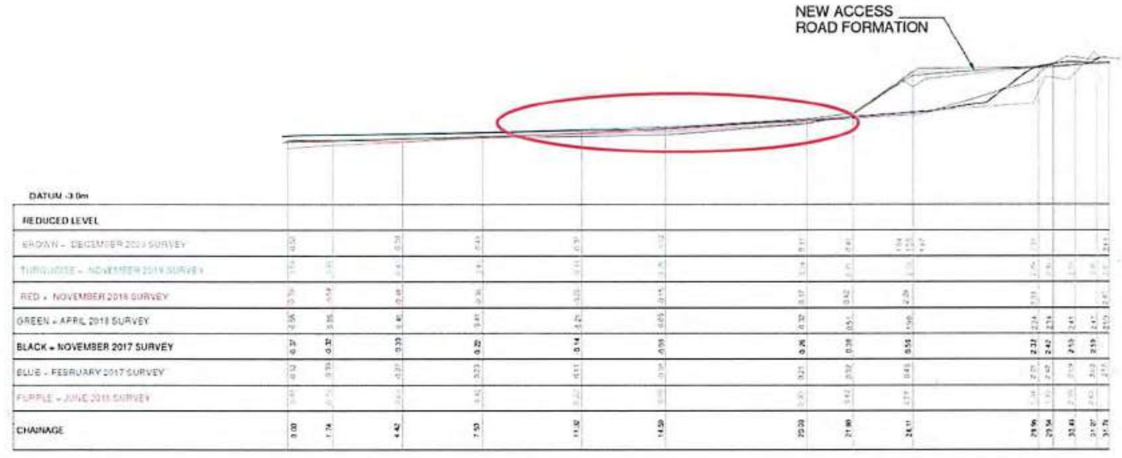

Figure 3-7: Decrease in beach levels at the western end of Town Beach (monitoring location T1)

Beach levels recorded at profile T2 show current beach levels are the lowest since monitoring started, in line with those recorded in 2019, up to 300mm lower than in June 2016. Previous surveys at this location all indicated negligible difference since the previous year. There appears to be a change in the general beach profile below high tide level. It is worth noting this monitoring location is seaward of the improved boat haul out area constructed towards the end of the wharf works in 2017.

As outlined in the 2019 report, this adjusted profile is likely the result of ongoing easterly sediment migration exacerbated by one or more of the following factors:

a The low tide beach profile may be adjusting due to a slight alteration in wave climate. However, at this location the wave climate was expected to reduce as a result of wharf construction, also reducing sediment transport rate and erosion risk.

b Increased use of the boat haul out may have resulted in a beach profile change along the alignment of boat launch and retrieval.

c Increased storminess during 2019 leading to more erosive conditions that year than noted in previous years monitored (refer Section 3.4).

d Human intervention at this location to improve usability of haul out area.

Given this profile is largely unchanged since 2019, it is possible this adjustment is close to establishing a new equilibrium profile. This location will be monitored going forward to determine if levels recover at this location or the beach is still adjusting to the recent construction of the access track, haul out and wharf.

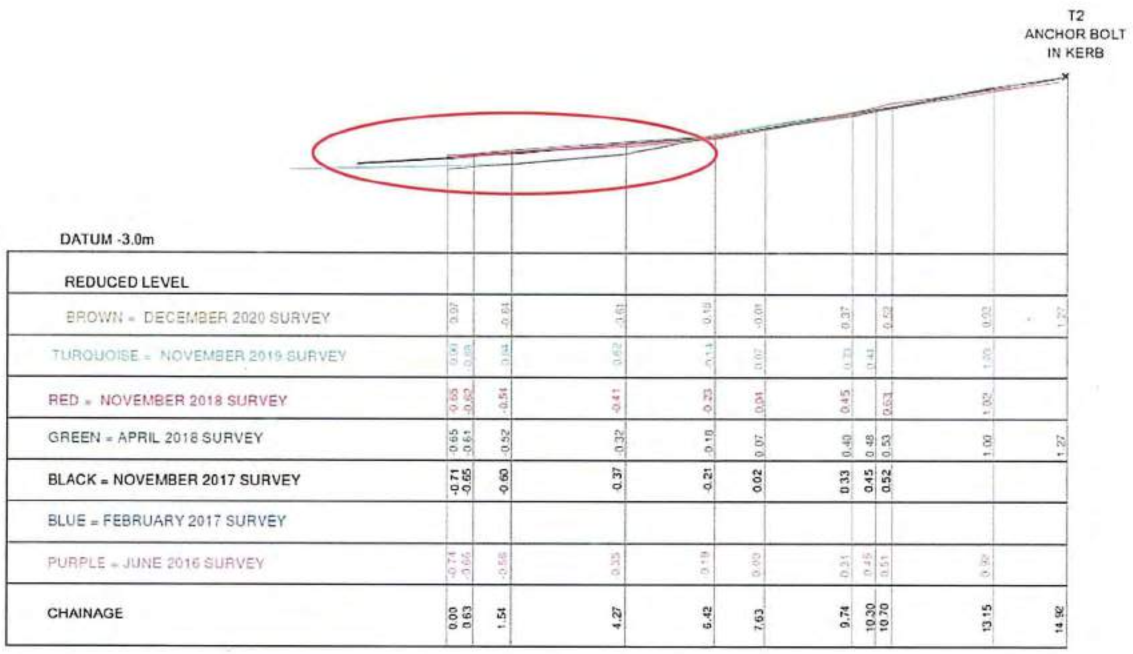

Figure 3-8: Lowering of low tide beach levels seaward of the boat haul out area (monitoring location T2).

There has also been a slight increase in the beach levels along Town Beach in front of the hotel since the 2019 report. This is evident in profile T3A (Figure 3-9). The current beach levels are generally 50mm higher than the 2019 levels. However, these are still generally 100-200mm lower than the April 2018 levels (following sand placement). The current beach levels are still up to 300mm higher than the June 2016 pre-nourishment levels, and the Tuff rock along this length is still covered. This indicates some of the placed material is still within the system. This is the first year since sand placement that the beach levels have increased and may indicate either or both of the following:

a Some easterly movement of material from the length of beach in front of the fish factory with lowering of sand levels evident here during photopoint monitoring (refer Section 3.3.1).

b A new equilibrium profile at this location is being established with the increase in beach levels part of the natural expected beach level fluctuation that may be expected around the equilibrium profile going forward.

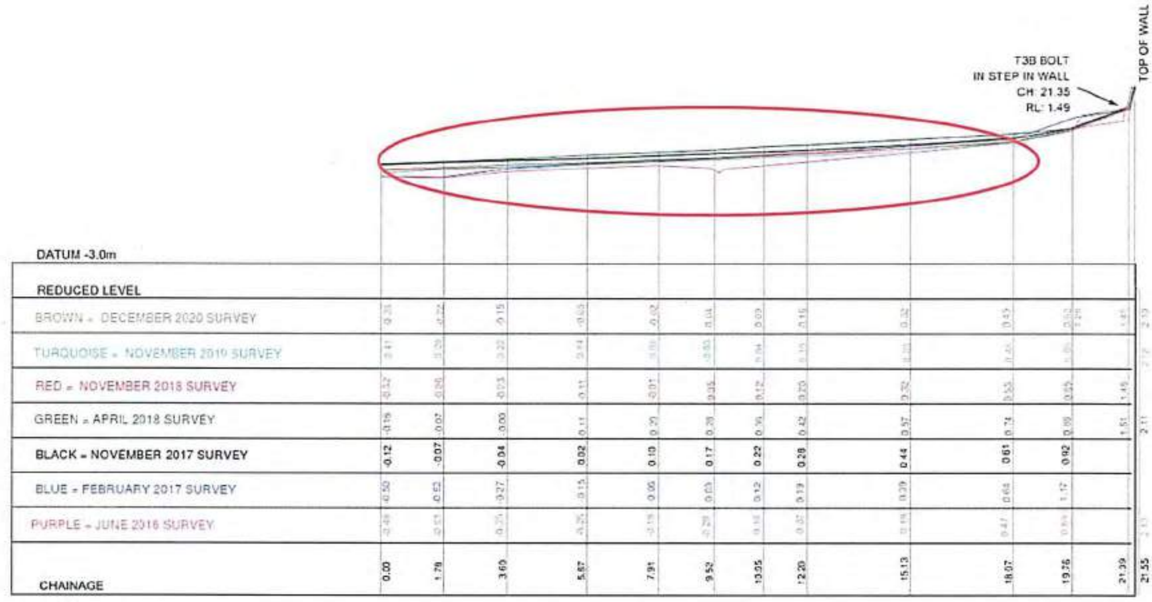

Figure 3-9: Decrease in beach levels in front of the hotel (monitoring location T3A)

Beach levels have generally decreased by 200mm at the eastern end of Town beach (profile T3B) since November 2019 (refer Figure 3-10). This is following the increase of up to 350mm noted between 2018 and 2019 in the 2019 report. This is expected as a result of the initial 'slug' of easterly sediment movement following sand placement. The headland at this location slowed the easterly sediment transport resulting in the accretion of sand following which this build-up of sand has likely continued its easterly movement trend around the headland towards the Nairn River.

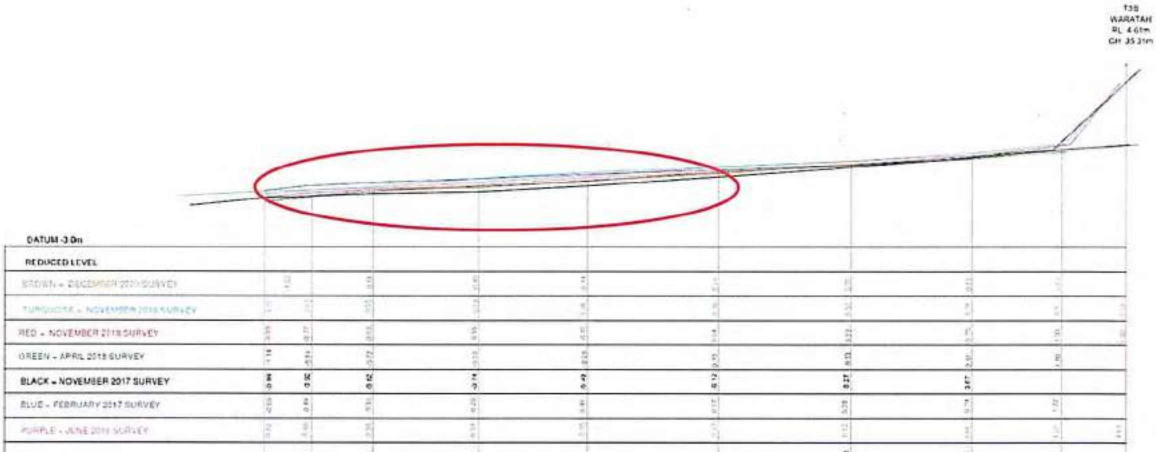

Figure 3-10: Increase in beach levels at the eastern end of Town Beach (monitoring location T3B).

It is worth noting that this area has experienced background erosion since 1969 (refer Figure 3-16) and it is likely that the recent erosional trend, following the short period of accretion, is more in line with the expected long term trend at this location moving into the future.

3.3.2 North of Nairn River

In general the beach levels are similar to those recorded in the December 2018 monitoring round. The wider, sandier beach at this location is subject to greater natural beach fluctuations than Town Beach and as per the 2019 report, the changes in level noted are generally within that expected due to natural beach fluctuation. However there are some general trends that can be established from the post construction monitoring:

- Dune/shoreline accretional trend at T4 and T5 since November 2017. This is particularly evident in the 2020 survey monitoring results for T4 (refer Figure 3-11). The dune toe has shifted approximately 6m seaward at this location since the 2016 pre-construction survey, an average trend of just over 1m per year (refer Figure 3-12).
- Increased lower beach profile levels of up to 500mm at T4 since preconstruction survey in 2016 (refer Figure 3-11).
- Erosional trend at the dune toe of approximately 2m at T6 since 2016. Note that this appears to be a localised issued which was attributed to anthropogenic influences as at this location during the 2020 visit. There is a track which appears to be used by local horse and motorbike riders immediately adjacent to the monitoring location which is the likely cause of this trend. This will be monitored in future but not discussed further in this report.

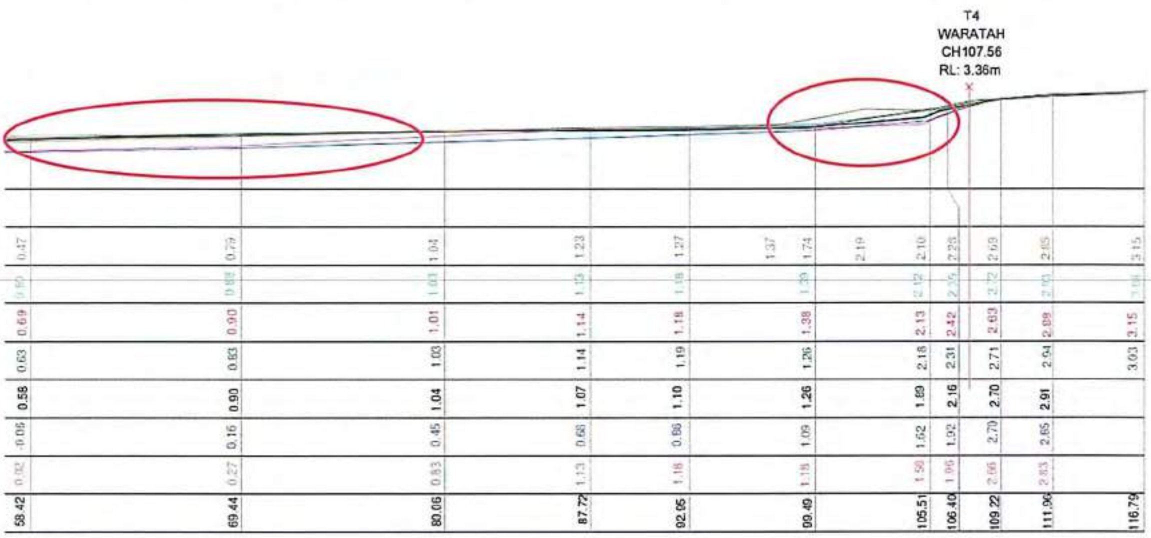

Figure 3-11: Dunal accretion and increased beach levels north of the Nairn River (monitoring location T4)

There is no evidence in the monitoring profiles (T4 to T6) of increased erosion rates along the lower beach profile since prior to construction.

The cause of the accretional trends may be associated partially with deposited material during the wharf construction, however more likely this is likely due to ongoing natural beach/dune fluctuation. Dunal accretion at this location is an indication that the modelled slight increase in wave climate at this location is not as yet having an effect on the beach and dune system at this location. There is no evidence of the background erosional trend at this location obvious through the monitoring undertaken to date. Future monitoring will be useful in establishing any changes in trends along this length of beach with a focus being on if any of these profiles drop below the pre-construction levels in future monitoring rounds.

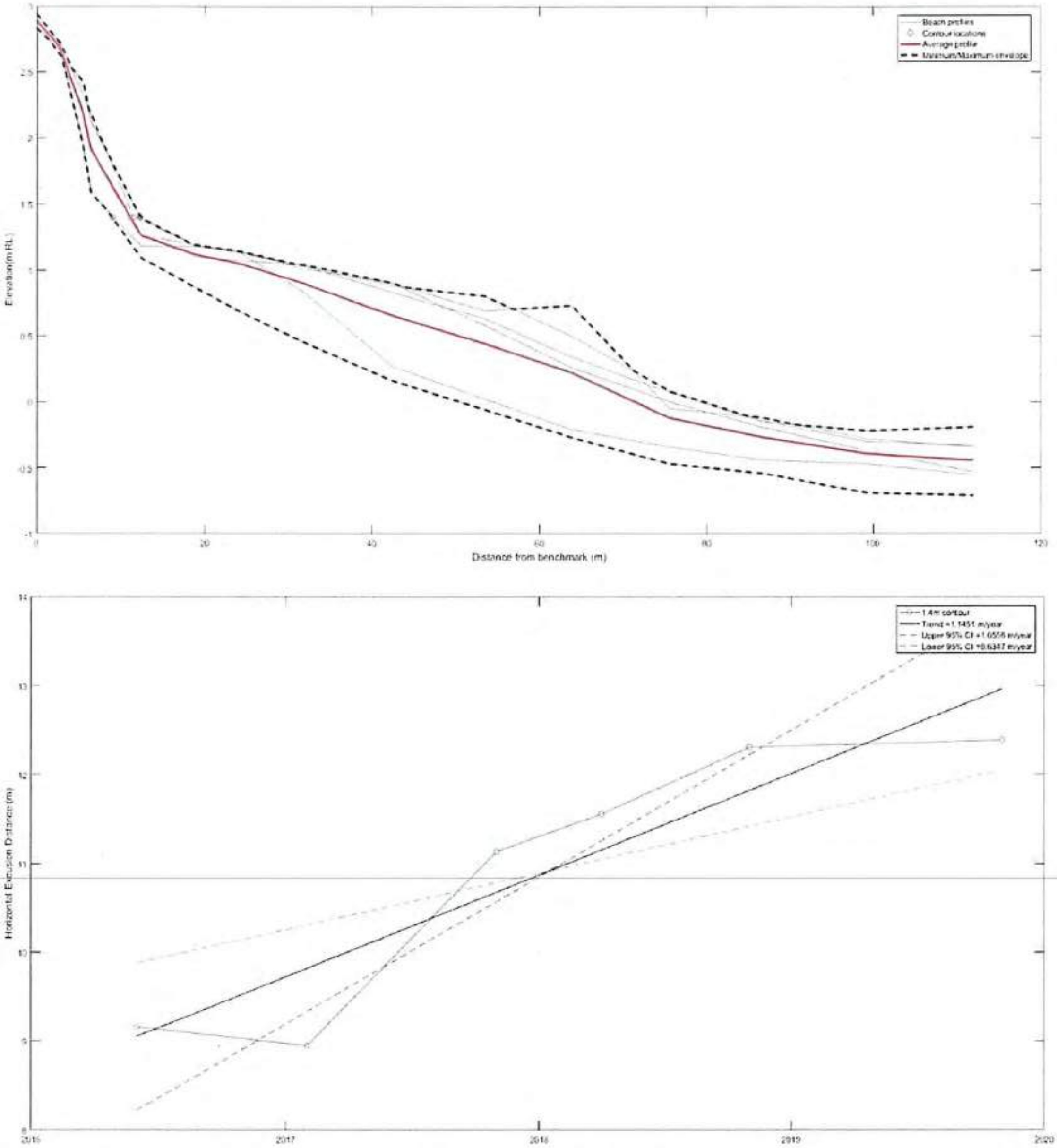

Figure 3-12: Profile analysis at T4 showing accretional trend of dune since monitoring began. Top image shows profile variation over monitoring period and bottom image shows profile trend analysis and average of 1.2m/year accretion at the 1.3mRL contour.

3.4 Wave data analysis

Wave data has been obtained from NOAA Wavewatch III (WW3) global numerical wave model at an output location 75km offshore from Waitangi Bay (refer Figure 3-13). The purpose of obtaining this wave data is to provide an indication of ocean conditions occurring between surveys (i.e. magnitude and frequency of storms). An understanding of these conditions is useful particularly when assessing any unexpected erosional or accretional trends that may be noted through any of the monitoring undertaken.

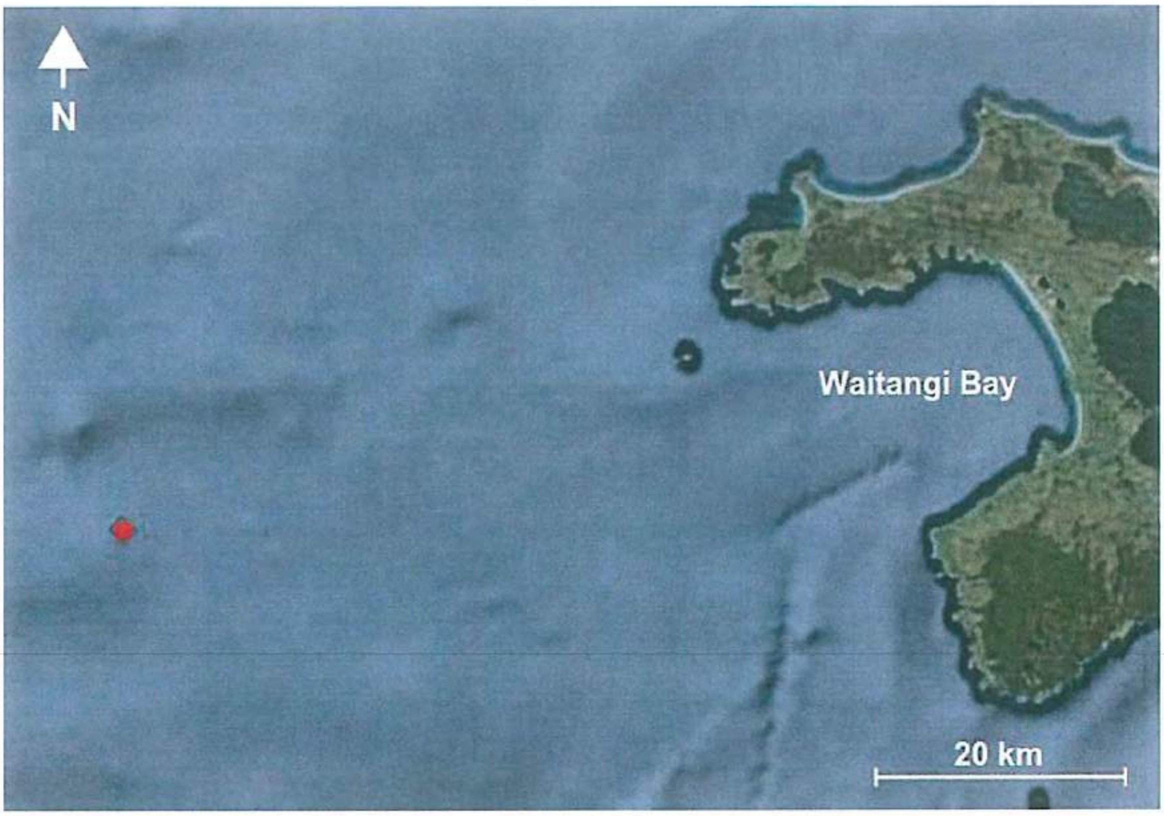

Figure 3-13: Location of numerical wave output point 75km offshore of Waitangi Bay

MetOcean Ltd produced a numerical wave hindcast model for a 35 year period until 2013 as part of the design phase of works. This model was also used to provide short term forecasting throughout construction through the metoceanview website. This model allowed for assessment of wave climate at locations within Waitangi Bay closer to the wharf location. This data can again be obtained in the future (at cost) if any significant, unexpected erosional trends are noted and a more detailed understanding of nearshore wave climate is required to assess these.

Offshore wave data was collected from WW3 and analysed to provide an indication of wave climate pre-construction (2005-2015), during construction (2016, 2017) and in the years following construction (2018-2020). Figure 3-14 shows comparison of the median significant wave height over these time periods with the box being the 25-75% confidence intervals and the red crosses indicating outliers (or extreme wave events) outside approximately 3 standard deviations from the median value (whisker length).

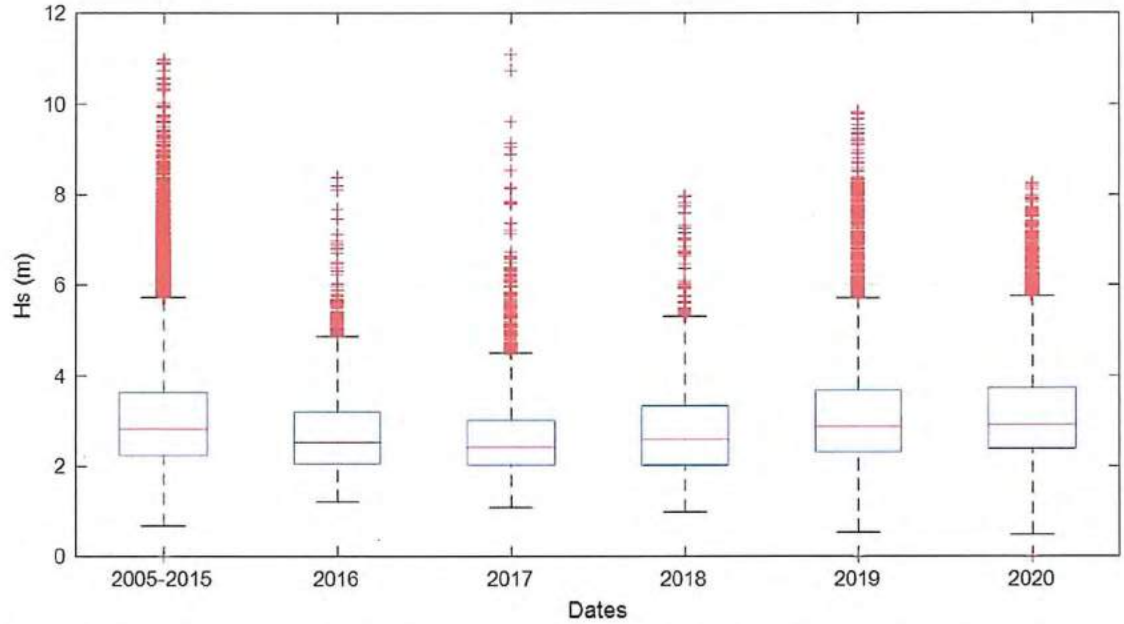

Figure 3-14: Offshore wave climate comparison between pre wharf construction (2005-2015), construction (2016-2017) and post construction monitoring years (2018-2020). Horizontal red line is the median significant wave height, the blue box is the 25-75% exceedance heights and the red crosses indicate extreme heights.

It can be seen from the figure that the median wave height during construction was lower than the pre-construction dataset indicating the wave climate was generally calmer than typical. However, there were two storm events where significant wave heights were above 10m (recorded in 2017), one of these was the largest in the 15 years of data. 2018 was a relatively calm year with median wave heights lower than pre-construction also experiencing the least number of, and lowest, extreme events recorded.

The 2019 and 2020 datasets are more in line with the pre-construction dataset, indicating these years was more indicative of the typical wave climate expected. The median significant wave height was 2.9m, compared with the pre-construction height of 2.8m. In 2020 there were no extreme events recorded above a 10m significant wave height and there was only one wave event over 8 in height. Compared to five in 2019, four during the construction period and 22 in the total pre-construction period of 2005-2015. So although the typical wave conditions were similar to 2019 and preconstruction, there were less extreme events in 2020 than in those periods.

A monthly wave climate analysis was undertaken with the dataset over the same time periods (refer Appendix C). November to March are generally the calmer months with May to July generally being the stormiest. It can also be seen that 2020 was relatively calm through the typically stormy months with no extreme events recorded between June and August. September was the stormiest month in 2020 with highest median wave height and largest extreme events.

An extreme value analysis (EVA) was undertaken using Weibull distribution on the pre-construction dataset to give an indication of return period associated with various significant wave heights experienced (refer Figure 3-15 and Table 3). This dataset contains only 10 years of wave height information and is unlikely to represent the true wave height return periods. It is likely to be an underestimate of extreme values as probability of a return period wave event greater than 10-50 years being included in a ten-year dataset is low and so should be used with caution. However for the purpose of this assessment this gives a good reference for discussion of the large wave events experienced during the analysis period.

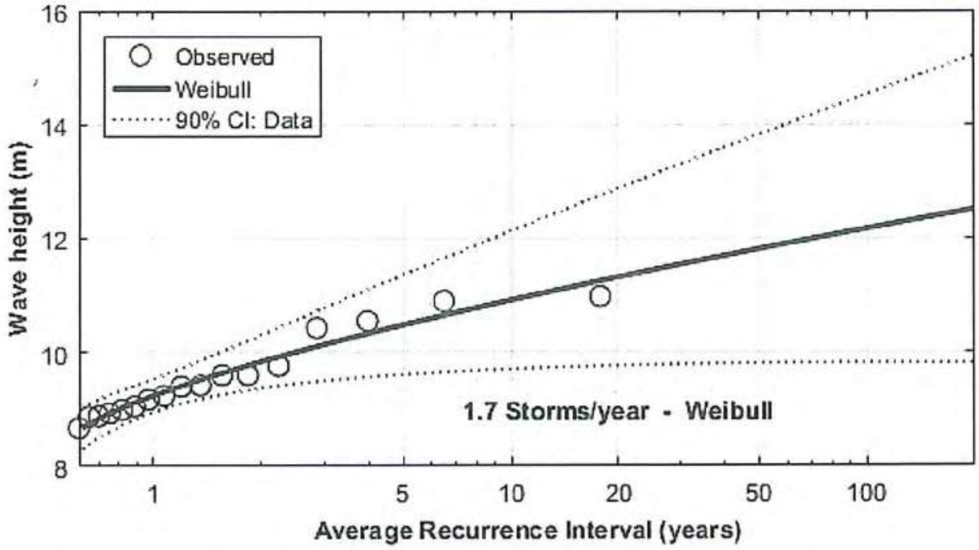

Figure 3-15: Weibull extreme value analysis undertaken on the pre-construction wave dataset (2005-2015).

Table 3: Weibull extreme value analysis undertaken on the pre-construction wave dataset (2005-2015)

Annual Recurrence Interval (ARI)	Significant wave height (Hs), m
1	9.2
2	9.8
5	10.5
10	10.9
20	*11.3*
50	*11.8*
100	*12.2*

There were no events recorded in 2020 with a significant wave height greater than 9.2m. This compared with two in 2019 (one between 1 and 2 year ARI and between a 2 and 5 year ARI). For reference there were also no events recorded with wave heights greater than 9.2m during 2018 and ten in the pre-construction period with three of these being greater than 10.5m.

Overall, this data set indicates that 2020, although stormier than 2016-2018, 2019 is still the stormiest year since before wharf construction began. This is likely the reason for the increased erosional trends noted during the 2019 not continuing into 2020 with trends noted typically more stable during this monitoring round than the previous one (refer Section 3.2.1).

3.5 Shoreline analysis

The currently available satellite imagery has been used to digitise the shoreline in relation to historic aerial photographs (refer Figure 3-16). The shoreline for this purpose is defined by the vegetation line or edge of sand (where it intersects a structure or cliff) and compared to previously available shoreline locations.

Satellite imagery was last updated in 2019, the first time since pre-construction in 2016. This was discussed in the 2019 report. Imagery will continue to be monitored and will be updated in future monitoring rounds if and when imagery becomes available.

IN the interim, Unmanned Aerial Vehicle (UAV) aerial photography has also been taken along Town Beach and part of the beach north of the Nairn River (2016-2020) and has been visually assessed for any notable changes in shoreline and beach profile (refer Figure 3-17). Comparing November 2019 and December 2020 aerial photography of Town Beach reflects the analysis in Sections 3.2 and 3.3 in that there is some lowering of sand level (or increased area of exposed Tuff rock) since the 2019 along the monitoring round in front of the fish factory.

Figure 3-16: Shorelines from 1969, 1989, 1996 and 2013 images superimposed on 2013 satellite image (source CNES/Astrium)

Figure 3-17: UAV photo analysis showing differences in Town Beach sand levels in front of the fish factory between Dec 2018 (top), Nov 2019 (bottom left) and Dec 2020 (bottom right). Arrow shows general easterly movement of sand.

4 Summary and conclusion

Beach monitoring has been undertaken at various locations around Waitangi Bay during the construction of the new Waitangi Wharf and continued following construction completion. The wharf modifies the wave climate refracting around the headland at the north-west end of the Bay. Beach monitoring is a resource consent requirement with the purpose of determining if there are any effects on sediment transportation and hence beach profile in the bay as a result of this modification.

To date the monitoring data is not sufficient to establish any significant trends post construction as it is dominated by construction activity, specifically sand nourishment. However, it is useful in showing any short term changes since construction began and for building a baseline dataset to build towards establishing longer term trends.

Trends noted to date include:

- Accretion along the majority of the Town Beach throughout the construction process. The accretion on Town Beach is a direct result of beach nourishment that has been undertaken during the construction works.
- Decreased sand levels in the years following nourishment along the majority of Town Beach, with levels at the western end at the lowest currently since pre-construction. There is exposed Tuff rock in places, likely linked to the easterly movement of the nourished sand material and establishment of the equilibrium beach profile.
- The low tide beach profile at the location of the boat haul out is currently lowest on record which may be linked to increase use of this haul out, increased storminess, human intervention or a combination of all the above. This location will be monitored in future rounds.
- An area of historic beach erosion at the eastern end of the Town Beach has previously been noted in the May 2018 monitoring report. This area however showed beach accretion in the December 2018 and 2019 reports which was evidence of eastward dredge sand migration. However 2020 monitoring indicates this end of the beach may be reverting to an erosional trend as sand migrates easterly around the headland.
- In general, the beach trends in 2020 appear to be more stable than those in 2019. This could be an indication that the effects of construction activity on Towns Beach (specifically sand nourishment) are diminishing with a new equilibrium being established.
- Accretion, erosion and beach level fluctuation along the length of Waitangi Beach to the north east of Nairn River is evident during and following the construction period. This is likely to be a result of natural beach fluctuation due to the scale at which this has occurred. The 2020 monitoring round has shown evidence of dune accretion along the ~1km length of beach north of the Nairn River. The background trend along this beach is erosion and therefore any ongoing accretion noted may be influenced by the altered wave climate.
- Wave climate analysis shows that 2020 was less stormy than 2019 which may have contributed to the reduced erosional trends noted in the previous reporting round.

These locations will be points of interest for ongoing data collection. The monitoring become biannual from next year, the next round of inspections being required in 2022 (refer Table 1).

In general, the monitoring undertaken to date does not provide any evidence to counter the conclusions presented in the 2015 Coastal Processes Report. As such, no adaptive management suggestions are made for the Waitangi Bay beach at this time. This will be reassessed as the monitoring regime progresses.

5 Applicability

This report has been prepared for the exclusive use of our client Waka Kotahi New Zealand Transport Agency, 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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Ed Breese

Project Director

Technical Review for T+T by: Dr Tom Shand, Coastal Technical Director

MAPP

\\ttgroup.local\corporate\wellington\tt projects\1008326\1008326.0200\issueddocuments\20201223_waitangi bay coastal processes monitoring report dec 2020.docx

Appendix A: Photo-point monitoring data

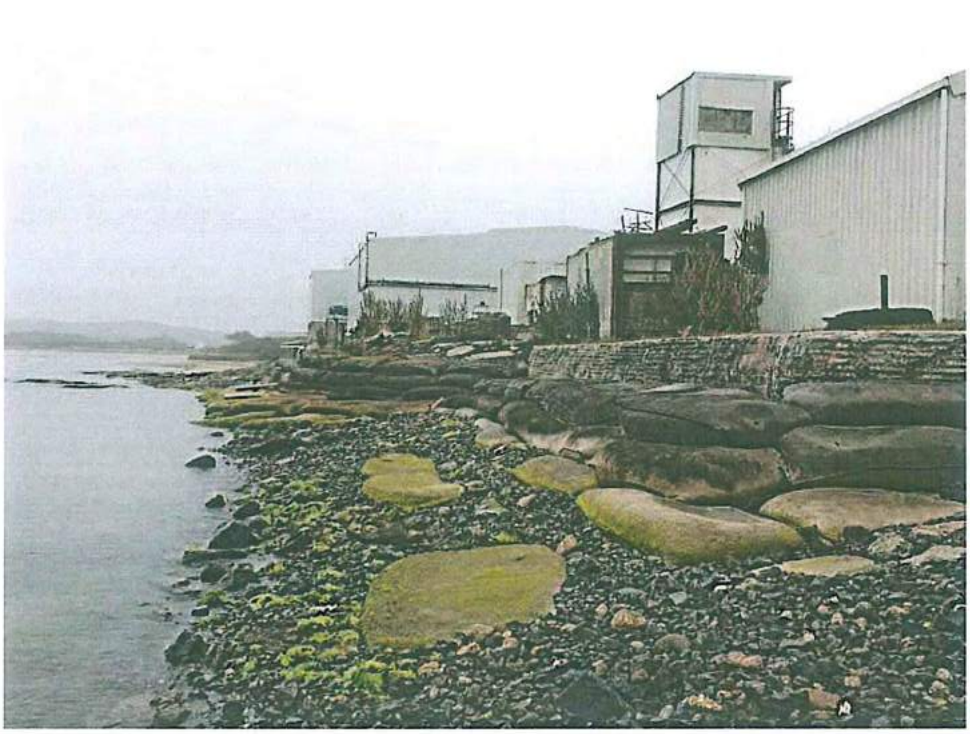
Monitoring location P1

Direction: East

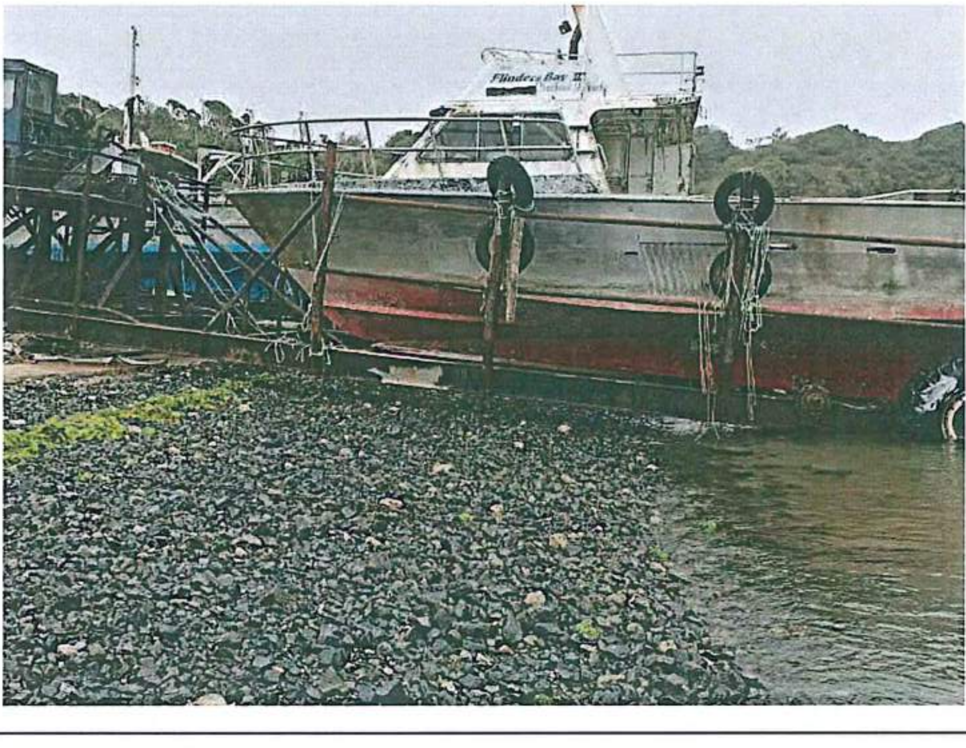
Direction: West

Monitoring location P2

Direction: East



Direction: West



Monitoring location P3

Direction: East

Direction: West

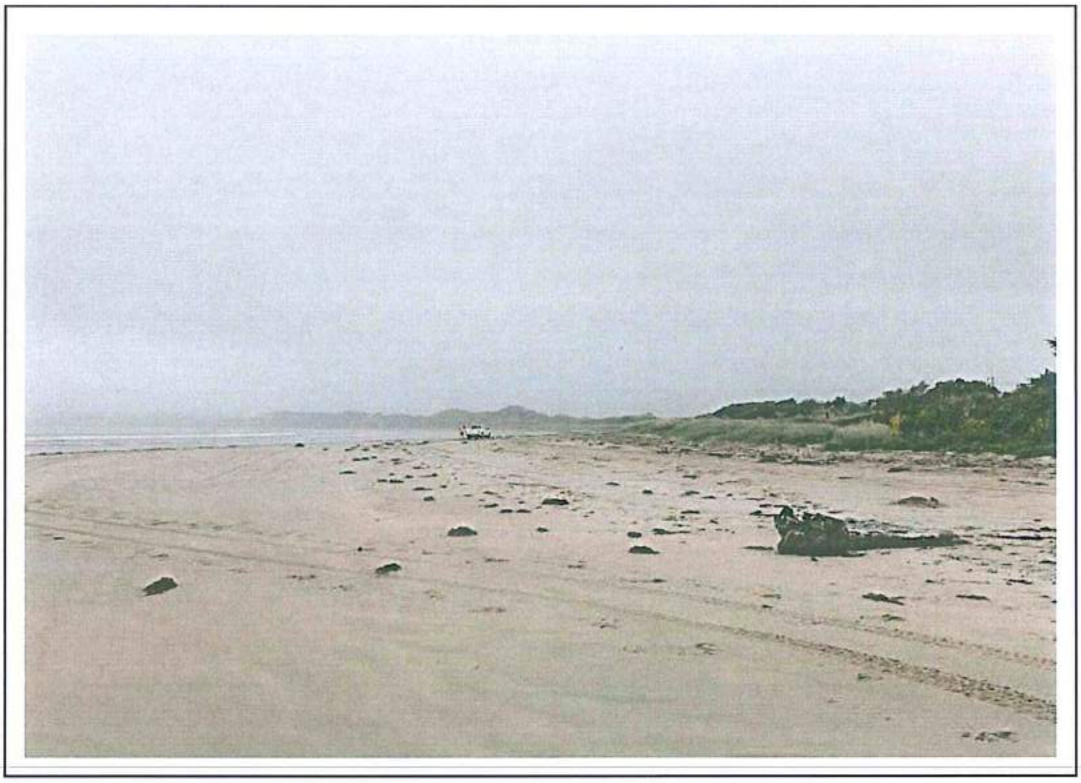
Monitoring location P4

Direction: East

Direction: West

Monitoring location P5

Direction: East



Monitoring location P6

Direction: East

Direction: West

Monitoring location P7

Direction: East

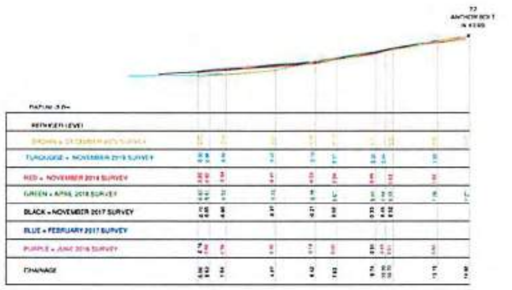
Direction: West

Monitoring location P8

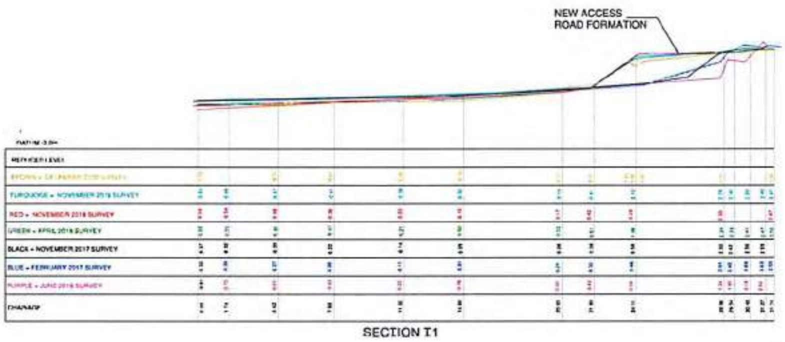
Direction: North

Direction: South

Appendix B: Beach profile survey data



SECTION T2



SECTION T1

COORDINATE DATUM - CHATHAM ISLANDS TM 2000

HEIGHTS ARE IN TERMS OF
- MEAN SEA LEVEL

ORIGIN OF LEVELS
- MARK NAME EHN1
- REDUCED LEVEL 2.35m
- SOURCE LINZ

SpencerHolmes
engineers - surveyors - planners

Level 6 - 8 Willis Street
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CLIENT
MEMORIAL PARK ALLIANCE

PROJECT
WAITANGI WHARF UPGRADE

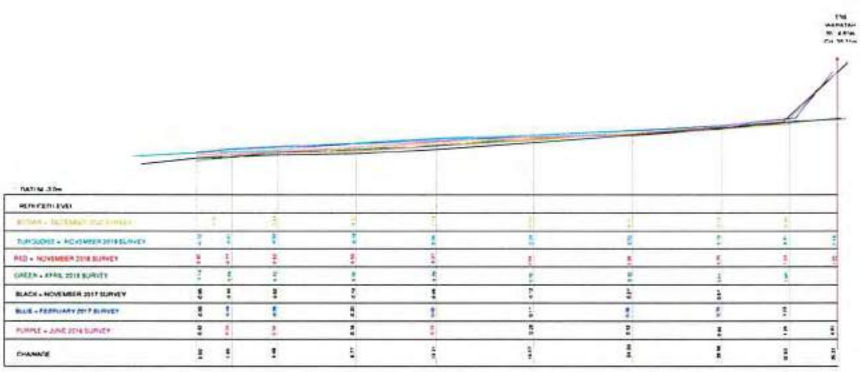
DRAWING TITLE
COASTAL MONITORING CROSS SECTIONS T1 & T2

DESIGN CONSULTANT

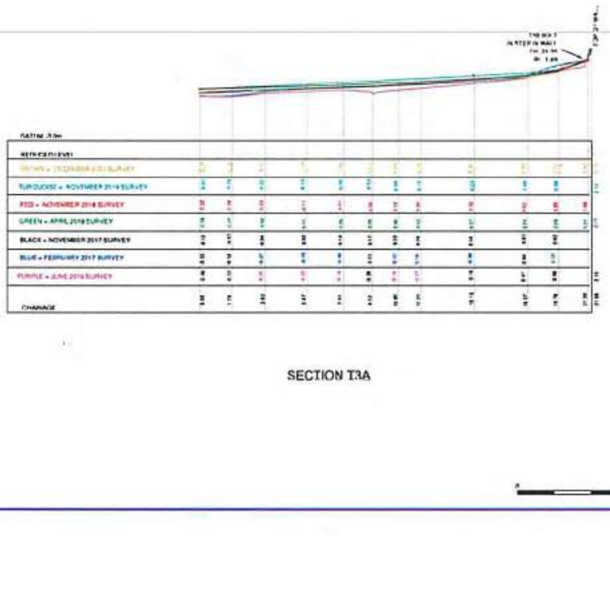
SCALE 1 : 100 | A1

DRAWING NUMBER S14-0871-M13 | REV D

RESOURCE MANAGEMENT - LAND PLANNING - SURVEYING - STRUCTURAL - CIVIL - FIRE



SECTION T3B



SECTION T3A

COORDINATE DATUM - CHATHAM ISLANDS TM 2000

HEIGHTS ARE IN TERMS OF
- MEAN SEA LEVEL

ORIGIN OF LEVELS
- MARK NAME EHN1
- REDUCED LEVEL 2.25m
- SOURCE LINZ

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PROJECT
WAITANGI WHARF UPGRADE

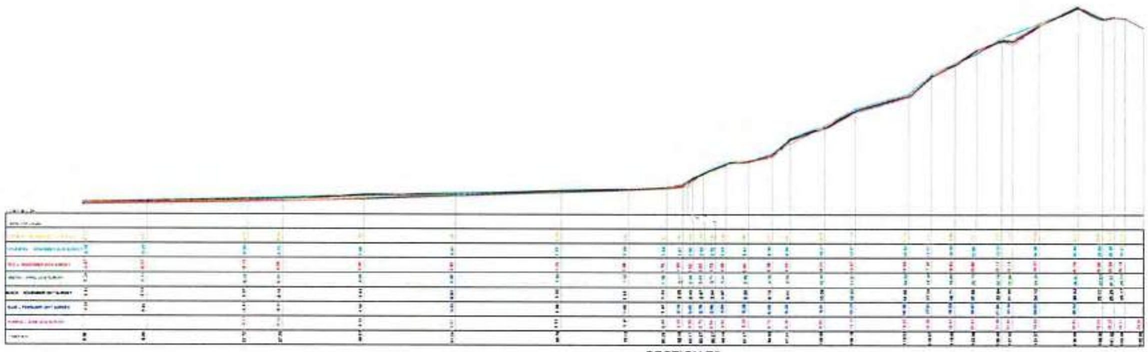
DRAWING TITLE
COASTAL MONITORING CROSS SECTIONS: T3A & T3B

DESIGN CONSULTANT

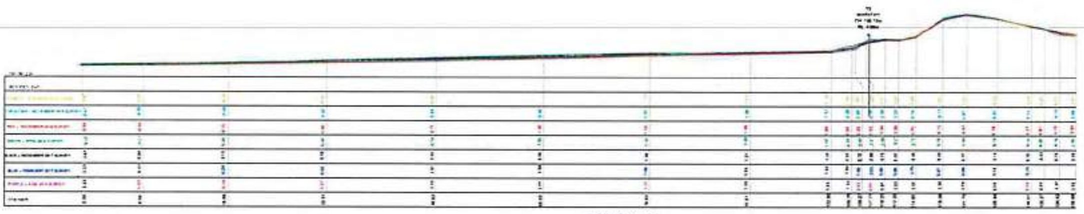
SCALE 1 : 100 | A1

DRAWING NUMBER S14-0871-M14 | REV D

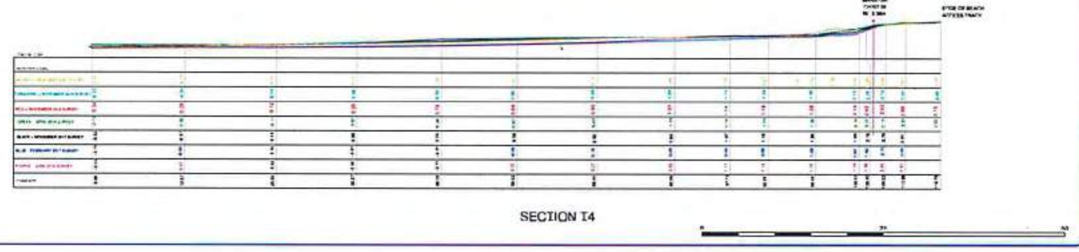
RESOURCE MANAGEMENT - LAND PLANNING - SURVEYING - STRUCTURAL - CIVIL - FIRE



SECTION T6



SECTION T5



SECTION T4

COORDINATE DATUM - CHATHAM ISLANDS TM 2000

HEIGHTS ARE IN TERMS OF
- MEAN SEA LEVEL

ORIGIN OF LEVELS
- MARK NAME EHN1
- REDUCED LEVEL 2.33m
- SOURCE LINZ

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CLIENT
MEMORIAL PARK ALLIANCE

PROJECT
WAITANGI WHARF UPGRADE

DRAWING TITLE
COASTAL MONITORING CROSS SECTIONS T4, T5, & T6

DESIGN CONSULTANT

SCALE 1 : 250 | A1

DRAWING NUMBER S14-0871-M15 | REV D

RESOURCE MANAGEMENT - LAND PLANNING - SURVEYING - STRUCTURAL - CIVIL - FIRE

Appendix C: Wave climate monthly plot

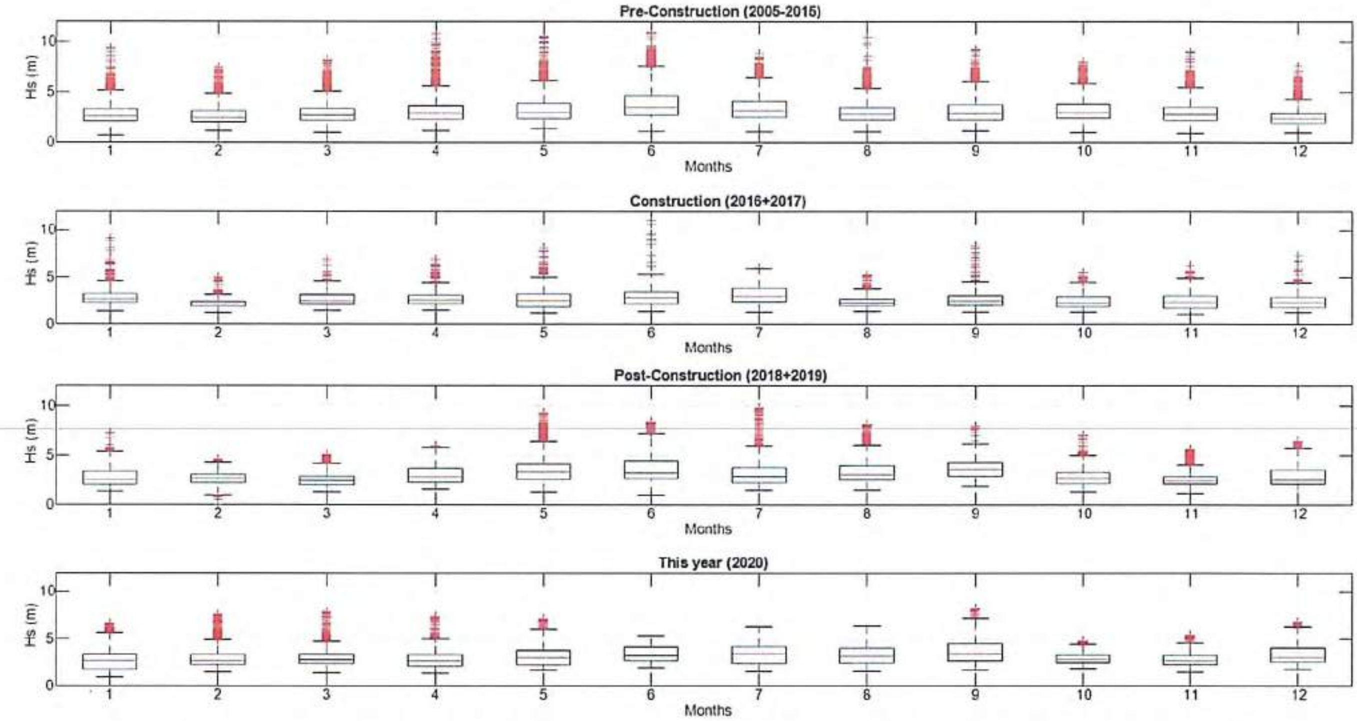


Figure C-1: Offshore wave climate monthly comparison between pre wharf construction (2005-2015), construction (2016, 2017), post construction monitoring years (2018, 2019) and this year (2020)

Appendix D: Waitangi Wharf Upgrade Coastal Processes Monitoring Plan

28 June 2016

COASTAL PROCESSES MONITORING PLAN

Waitangi Wharf Upgrade Project

Rev.	Status	Prepared by	Checked by	Date
1	Draft	Ed Breese	Tom Shand	17 May 2016
2	Final	Ed Breese	Tom Shand	28 June 2016

Name	Position	Date	Signature
Ed Breese	Environmental Manager	30/6/16	
Steve Croft	Alliance Manager	30/6/16	

1. INTRODUCTION

The Chatham Islands Port Limited (CIPL) has obtained resource consents (CIC/2015/02) for the construction of an upgrade of Waitangi Wharf and related activities. The project is funded by the Department of Internal Affairs (on behalf of the Government,) and is being designed and constructed by the Memorial Park Alliance (MPA) which consists of the New Zealand Transport Agency, Downer Construction, HEB, AECOM and Tonkin and Taylor.

The project will include the reclamation and dredging activities which will result in changes to coastal process. The assessment of coastal processes (Appendix 1) prepared as part of the resource consent application has concluded that these activities will not result in significant changes or adverse effects on coastal processes. To confirm this assessment and determine if mitigation measures are required, this Coastal Processes Monitoring Plan (CMP) has been prepared.

The main areas of interest for t coastal processes monitoring is the beach area in front of the Waitangi Hotel and Aotea Fisheries Factory and the area around the mouth of the Nairn River and north of the river.

The CPMP sets out procedures for the collection of relevant information and the analysis and reporting of results.

1.1 Statutory Requirements

Resource consent CIC/2015/02 sets out the following conditions in respect of coastal processes monitoring.

2 *The Consent Holder shall prepare a Coastal Processes Monitoring Plan (CMP), The CMP shall be submitted to Chatham Islands Council 20 working days prior to works in the CMA commencing for certification. The purpose of the CMP is to identify any impacts on coastal processes that are attributable to the construction works or the coastal structures once they are in place and the requirement for adaptive management to mitigate adverse effects. The CMP will include:*

- *a) use of a numerical wave model to record the wave climate within Waitangi Bay during the construction phase;*
- *b) fortnightly photo point monitoring of Waitangi Town Beach for the purpose of identifying measurable changes and causes of such changes for the period of construction;*
- *c) coastal profile surveys including at least six profiles at locations on Waitangi Town Beach, the Nairn River mouth and north of the Nairn River mouth,*
- *d) A description of survey intervals and duration;*
- *e) review of relevant satellite imagery as it becomes available; and*
- *f) an annual review of wave climate, beach profile and photo point monitoring data.*

3 *The Consent Holder shall report the monitoring results to the Community Liaison Group and the Chatham Islands Council together with any recommended adaptive management on an annual basis by 31 December each year.*

2. THE CMP

2.1 Roles & Responsibilities

The responsibilities in regard to the implementation of the CMP are shown in Table 1.

Table 1: Responsibilities for implementation of the CPMP.

Title	Name	Responsibilities
Alliance Manager	Steve Croft	Overall responsibility for the project
Stakeholder, Environmental and Compliance Manager	Ed Breese	Ensuring resource consent reporting requirements are met and engaging with Community Liaison Group
On Island Project Manager	Hugh Miliken	Ensure surveys and photo point monitoring is undertaken
Independent Surveyors	Spencer Holmes	Setting out survey control points and confirming baseline survey
Coastal Processes Expert	Dr Tom Shand	Data analysis, reporting and identification of mitigation measures if required.

2.2 Training

All people involved in monitoring activities will need to go through a training process. The objectives of the training will be to ensure the following:

- Health and safety procedures are clearly understood
- The procedures for photopoint monitoring are clearly understood and consistently applied
- The procedures for beach surveying are clearly understood and consistently applied

A record of this training will be kept.

3. MONITORING PROCEDURES

The coastal processes monitoring will involve maintaining a photographic record of the beach on a regular basis and a less regular surveying of the beach profile.

3.1 Photopoint monitoring

During the construction of the breakwater and wharf, two weekly photographic monitoring and visual observations will be undertaken. The photos and observations will be taken at the high water mark at the locations shown in Table 2. The position of these locations is also shown in Figures 1 and 2. The photos will be taken as close to time of low tide as practical.

At each photo location, a permanent off set marker will be established so the photo position can be easily replicated. Table 2 will be updated to provide the offset information. The permanent off set marker will be either a metal rod or timber pole buried at least 500mm into the ground or marked onto a fixed structure. The marker will also include a label such a metal tag to identify the location number.

Table 2: Photographic and visual monitoring locations

Monitoring point	Location	Off set marker description
P 1	Western end of Town Beach	
P 2	At toe of boat ramp to south of Aotea Fisheries factory	
P 3	In front of Waitangi Hotel accommodation block	
P 4	75m east of Waitangi Hotel public bar	
P 5	Eastern abutment Nairn River bridge	
P 6	125m east of the eastern abutment Nairn River bridge	
P 7	710 m east of the eastern abutment Nairn River bridge	
P 8	1500 m east of the eastern abutment Nairn River bridge	

A summary sheet for all photos will be prepared that covers the following information:

- Date
- Low tide time nearest to time of photo
- Weather conditions and sea state over the previous two weeks

For each photo the following information will be recorded;

- Time
- Location
- Photo direction
- Site observations particularly any differences from previous photos such as noticeable erosion or aggradation.

Following the first round of photos and visual observations the coastal processes expert will review the information collected and make recommendations if necessary on changes in the monitoring procedures.

Figure 1: Monitoring Locations Town Beach


Figure 2: Monitoring Locations North of Nairn River

3.2 Surveying

Prior to construction commencing a set of baseline beach profiles will be recorded. The transect locations are identified in Table 3. The position of these transects is also shown in Figures 1 and 2.

Surveys at the nominated transects will be undertaken on a 6 monthly basis during construction and then annually for 2 years after the breakwater and wharf construction is completed. Then bi- annually for the duration of the consent or a shorter term if approved by Council.

At each transect a permanent marker will be established so the survey can be easily replicated. Table 3 will be updated to provide information on the marker location, vertical elevation and transect bearing. All levels are reduced to local MSL (2.35 m below LINZ EHN1 survey mark), and all locations are referenced to Chatham Islands Transverse Mercator (CITM). The transect marker will be either a metal rod or pole buried at least 500mm into the ground. The marker will also include a label (such as a metal tag) to identify the transect number.

Table 3: Survey line locations

Transect	Location	Marker GPS position and transect bearing
T 1	At northern end of diesel storage compound	
T 2	At toe of boat ramp to south of Aotea Fisheries factory	
T 3	At boundary between Aotea Fisheries factory and the Waitangi Hotel	
T 3	75m east of Waitangi Hotel public bar	
T 4	125m east of the eastern abutment Nairn River bridge	
T 5	710 m east of the eastern abutment Nairn River bridge	
T 6	1500 m east of the eastern abutment Nairn River bridge	

The permanent marker will be located at least 5m (horizontally) landward of the beach or dune crest (or as agreed with the Coastal Processes Expert). The permanent marker will be moved inland and resurveyed if threatened by erosion.

Surveys should be referenced to the benchmark and consist of horizontal and vertical offsets across the profile from the benchmark to the water edge at low tide.The survey should pick up changes in grade, vegetation line, debris line (denoting high tide) and any other features of note. Acceptable survey methods include RTK GPS, theodolite, level and staff. Staff and tape and visual estimate are not acceptable.

Following the first round of surveying the coastal processes expert will review the information collected and make recommendations if necessary on changes to the monitoring procedures. If changes are required these will be undertaken before construction commences and another series of transects completed.

In the first year at least one set of transects will be surveyed by an independent surveyor.

4. PROGRAMME

The programme for the monitoring activities is shown below.

Activity	2016	2017	2018	2019	2020	2021 - End Consent*
2 weekly photo monitoring	■	■				
Beach profile baseline	■					
6 monthly beach profiling	■	■				
Annual beach profiling and photo monitoring			■	■	■	
Annual reporting	■	■	■	■	■	
Biannual beach profiling and photo monitoring						■
5 yearly reporting						■

* Shorter period if agreed by Council

5. WAVE DATA COLLECTION

Information will be collected on the wave climate reaching Waitangi Bay. This information will be obtained from NOAA Wavewatch III global numerical wave model at an output location 75 km offshore of Waitangi Bay (Figure 3). This output will provide indication of ocean conditions occurring between surveys (i.e. magnitude and frequency of storms).

Figure 3 Proposed location of numerical wave output point offshore of Waitangi Bay

6. SATELLITE IMAGERY

A review will also be undertaken to see if any new satellite imagery has become available. If so, the new image will be georeferenced with respect to historic aerial photographs (refer Waitangi Wharf Upgrade Coastal Processes Report) and the new shoreline digitised as defined by the vegetation line or edge of sand (where it intersects a structure or cliff) and compared to previous shoreline locations.

7. REPORTING

An annual report will be prepared upto 2021 and on a 5 yearly bais until the consent expires or Council agree reporting no longer required. The report will include the following;

- Summary of the wave regime over the previous 12 months and commentary in respect the "normal" wave regime
- A summary of the photographic monitoring undertaking covering frequency of monitoring and key observations and changes in the beach
- A summary of beach profile monitoring
- Update on shoreline position if any new relevant satellite imagery has become available
- Comment if the information changes the Summary and Conclusion section of the Waitangi Wharf Upgrade Coastal Processes Report.
- If necessary suggestions on adaptive management such as additional sand transfer, coastal protection works or relocation of assets.

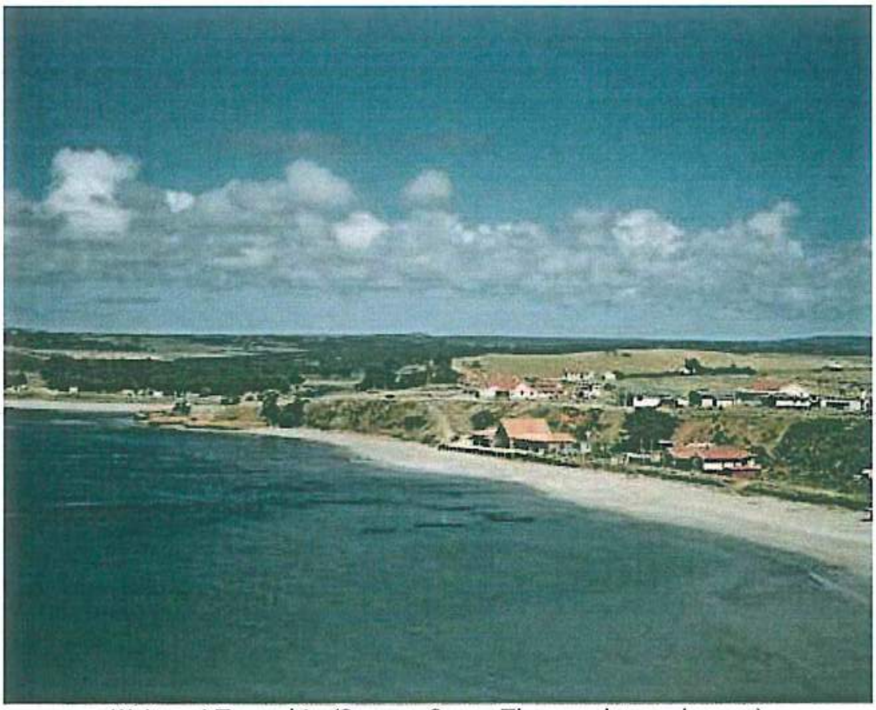
The report shall be provided to the Community Liaison Group and the Chatham Islands Council on an annual basis by 31 December each year. If significant change is observed during from the photopoint monitoring the Community Liaison Group and Chatham Island Council will be advised as soon as practical.

Appendix E: Coastal Process Report

- **No appendices – refer Consent Documentation if required**

24 June 2015

Waitangi Wharf Upgrade – Coastal Processes Report



Waitangi Township (Source: Susan Thorpe, date unknown)

Rev.	Status	Prepared by	Checked by	Date
5		Dr Tom Shand	Richard Reinen- Hamill	24.06.15

Name	Position	Date	Signature
Dr Tom Shand	Senior Coastal Engineer	24.06.15	
Richard Reinen- Hamill	Principal Coastal Engineer	24.06.15	

Contents

1 INTRODUCTION

The Chatham Islands Ports Limited (CIPL), in conjunction with its funder, the Crown, administered by the Department of Internal Affairs (DIA) is seeking resource consent applications to undertake the Waitangi Wharf Upgrade Project (WWUP) located in Waitangi Bay, in the Chatham Islands. The project seeks to improve the reliability and usability of the wharf, and enhance the port operations for the island. The key elements of the project include: –

- Creating a temporary landing area to enable the unloading and loading of construction equipment between New Zealand and the Chatham Islands;
- Constructing a breakwater up to 185 m long for protection of the wharves;
- Constructing new land for enhanced port operations including new buildings for cargo handling. The facings of this reclamation will create a new commercial wharf and a new fishing wharf;
- Dredging to enable the construction of the reclamation and wharves, and to improve vessel berthing;
- Beach replenishment of Waitangi beach using sand from the proposed dredging if it cannot be used within the reclamation; and
- Minor improvements to the existing livestock holding area and track.

This Coastal Report sets out the dominant coastal processes operating in the area surrounding Waitangi, describes the proposed works and evaluates their potential effect on coastal processes.

1.1 Site location

The Waitangi Bay coastline is situated at the southern end of Petre Bay and in the lee of a rocky headland, Tikitiki Hill (Figure 1- 1; Figure 1- 2). The dominant driver of the coastal processes are swell waves generated by the predominant westerly airstreams south of 40° latitude. These waves are refracted around the Tikitiki Hill headland before arriving in Waitangi Bay from a north westerly direction at a reduced height. The construction of the proposed physical works have potential to modify these wave processes and the resultant sediment transport dynamics within the bay.In regards to site datum's, all levels are reduced to local MSL (2.35 m below LINZ EHN1 survey mark), and all locations are referenced to Chatham Islands Transverse Mercator (CITM)

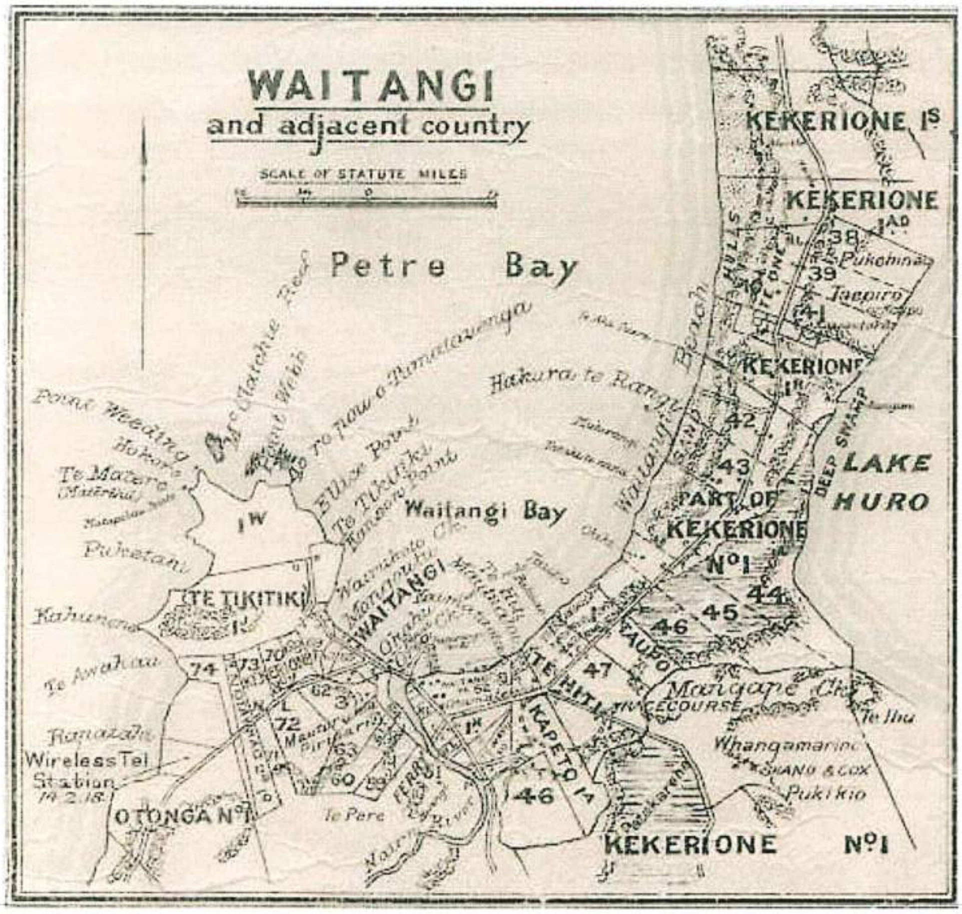


Figure 1-1 Waitangi Bay and surrounding land location figure

Figure 1-2 Waitangi Bay (left) with the existing wharf (right)

2 PHYSICAL ENVIRONMENT

2.1 Geological setting

The Chatham Rise is part of New Zealand's continental crust, extending east from the South Island. The Chatham Islands are the only emergent part of the rise and are located some 900 km east of Christchurch. The islands emerged within the last four million years and are comprised of Schist basement overlain by localised volcanic material (Figure 2- 1).

The largest Island, Chatham Island (Rekohu or Wharekauri), is comprised of a basement schist, emergent in the north, overlain by volcanic basalt, tuff and limestone accumulations. The Islands have been relatively unaffected by tectonic movement compared to the rest of New Zealand (Williams, 1995) with changes in sea level being the main drivers in the development of extensive marine cut surfaces and accumulation of marine sands. Following the most recent stabilisation of sea level (over the past 10,000 years) marine sediments have accumulated in the lee of the southern Chatham volcanic outcrops, joining northern and southern portions of the Chatham Islands and resulting in the formation of Te Whanga Lagoon system and new, low barrier beach systems on the eastern and western sides.

Waitangi Bay is located at the south- eastern corner of Petre Bay, and is defined on its western side by the rocky headland of Tikitiki Hill, and on its eastern side by Waitangi Beach which extends north to Red Bluff (refer Figure 2- 1). Both Tikitiki Hill and Red Bluff are comprised of Ecocene Volcanic Red Bluff Tuff. This material is highly to moderately weathered, weak rock that has varying degrees of calcite cementation. When weathered it typically breaks down into a fine to coarse sand with some silt.

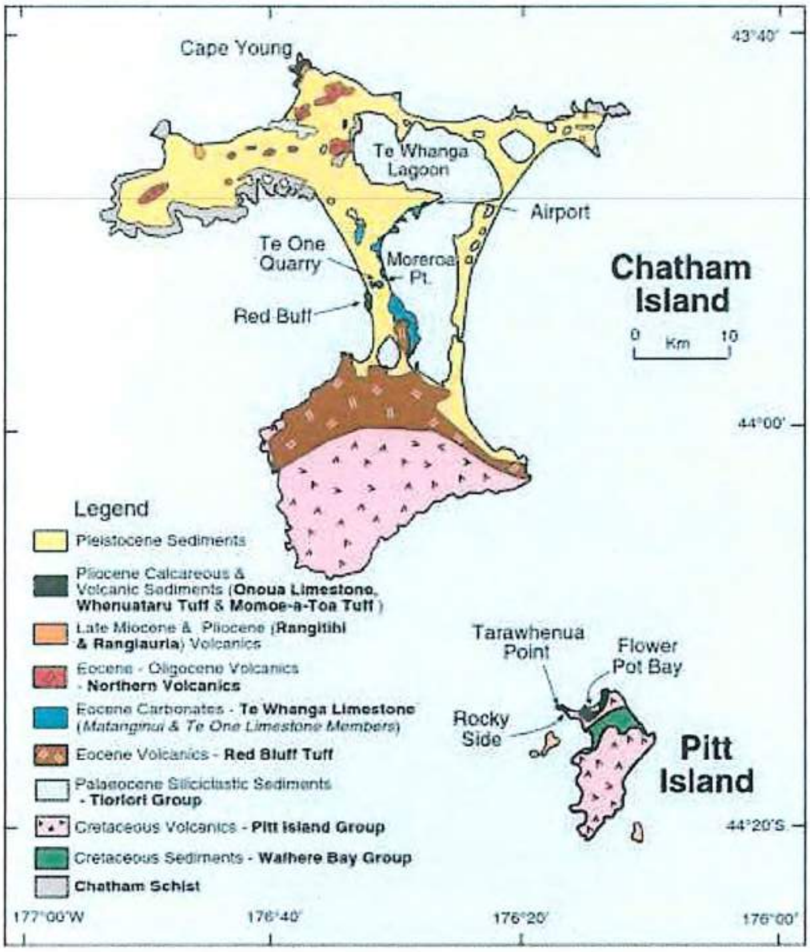


Figure 2-1 Chatham Islands Geology (James et al., 2011)

2.2 Topography and bathymetry

The headland adjacent the existing wharf comprises steep cliffs 20 – 30 m high overlain by rolling hills. To the northwest of the existing wharf the cliffs slope up from an intertidal rock platform at slopes of 2(V):1(H) flattening to 1(V):1(H) nearer the top of the cliff (Appendix A, Photograph 3). These cliffs appear over- steepened with recent slips evident. To the south of the wharf the cliff toe is retained by a seawall and roadway (Appendix A, Photograph 6) and the above cliff slopes up at around 1(V):1(H). While vegetation is becoming established on this cliff, minor slips remain evident. Buildings including the Moana Pacific fish processing factory and Hotel Chathams are located on a low terrace 1 to 1.5 m above the high water level backed by a 10 m high tuff embankment (Appendix A, Photograph 9). To the east of the hotel, the beach is backed directly by the tuff embankment (Appendix A, Photograph 9, 17).

Survey data from a number of sources were combined into a composite Digital Terrain Model (DTM) (Figure 2- 2). These sources include:

- LINZ Fairsheet Data (1977) covering South Petre Bay;
- LINZ Multibeam survey (2006) covering South Petre Bay with detail in Waitangi Bay;
- Spencer Holmes topographic survey (December 2014) of the wharf structure, road and eastern Tikitiki Hill; and
- Diver- collected seabed depths (December 2014) including depth of sand to basement rock.

The combined DTM shows that Waitangi Bay is a relatively shallow bay with depths of 10 m occurring 350 m offshore of the existing wharf and some 900 to 1000m from the shoreline. Between the 5 m and 10 m depth contour (nearshore shelf) the seabed slopes at s=0.007 before steepening to s=0.01 to 0.014 between the 5 and 0 m depth contour (within the surf zone). The beach fronting Waitangi Town slopes at between s=0.04 at the western end to s=0.08 at its eastern end. The longer Waitangi Beach that extends north to Red Bluff is at a flatter slope of s=0.037.

Several rocky outcrops comprised of Calcareous Tuff occur within Waitangi Bay. The most prominent being a 40 to 60 m wide intertidal reef platform at the toe of the cliff and road seawall. There is a 130 x 100 m reef in 4 to 5 m water depth approximately 50 m south of the existing wharf, and the 'hotel reef', a 200 x 150 m reef between 4 m and 0.5 m depth contours (approximately low tide), is in front of the Hotel Chathams in the centre of Waitangi Bay. A further reef extends 150 m offshore from the rocky headland between Waitangi Town Beach and the Nairn River.

A multi- beam survey outside the surf zone shows evidence of bed forms with wave length 5 to 10 m and amplitude up to 0.1 m. Higher frequency variations (length ~ 30 cm and amplitude up to 0.1 m) are also evident, although it is uncertain whether these are also bed forms or survey noise. It is inferred that fluctuations of ±0.2 m from the average seabed level are possible.

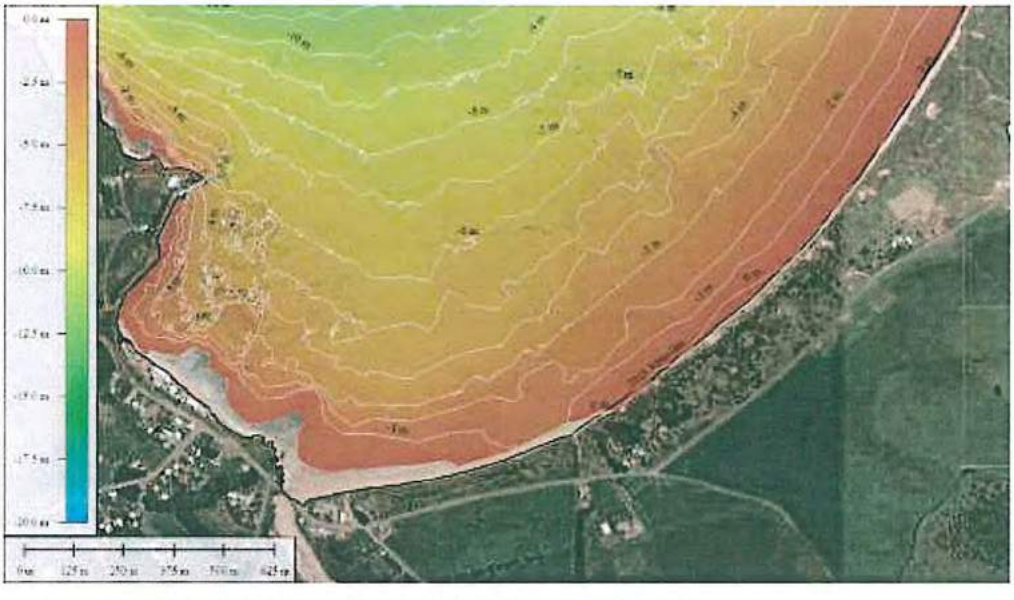


Figure 2-2 Combined digital terrain model of harbour bathymetry

2.3 Existing structures

The wharf was moved from its original location in the western corner of the bay to the present location at Hanson Point in the early 1930s. Previously surf boats would load cargo at a short jetty and then row out to the trading vessels offshore (Figure 2- 3). A 385 ft (117 m) long timber wharf with a 202 ft by 26ft (60 x 8 m) 'Tee' section was constructed (Figure 2- 4) to enable larger trading vessels to berth directly (King and Morrison, 1990). Depths off the berth at the time of construction were reportedly 15 to 17 ft (4.6 – 5.2 m). The wharf was upgraded to a reinforced concrete structure in 1979/1980 (Appendix A, Photograph 1) and a small previous reclamation at the base of the wharf was extended to approximately 65 m in length (2,800 m^2 total area) to allow construction of port facilities. This reclamation is protected on its seaward side by rock rip- rap comprised of both basalt and tuff material (Appendix A, Photograph 2- 4). The tuff riprap was placed during the formation of the reclamation. The basalt has been added as replenishment and repair. The basalt rock is relatively resistant to weathering and the tuff material reportedly rapidly deteriorates and is frequently replaced.

A road was excavated out of the cliff face to reach the new wharf with a stepped, vertical concrete seawall eventually constructed to protect the road. This road and seawall remain in use today (Appendix A, Photograph 6- 8) and appear in reasonable condition.

Early imagery of Waitangi Town Beach (Figure 2- 5 and Figure 2- 6) show buildings constructed on the low terrace at the base of the tuff embankment. Timber and brush seawalls fronting the land are also evident and were reported as being intended to prevent erosion (King and Morrison, 1990). This indicates that although a wider beach is evident than exists there today, periodic erosion was an issue then.

The present day foreshore at Waitangi Town Beach is backed almost continuously by seawalls. At the western end (Appendix A, Photograph 7, 10) rock and rubble has been dumped to prevent erosion at the end of the road seawall. This rubble wall continues towards the east fronting fuel tanks located on the backshore behind the beach (Appendix A, Photograph 11). The rubble does not appear to overlie an impermeable core (such as a geotextile filter layer) and so fine material is lost by hydraulic wave action. The boat pull- up area is fronted by a low, gravel and fill revetment (Appendix A, Photograph 12). Further east a range of vertical concrete walls front of the Moana Pacific fish processing factory (Appendix A, Photograph 13, 14, 15). These walls are in generally poor condition and have been undermined in some places with fill lost from behind the walls and collapse of concrete pavements. A sloped vertical seawall fronts the Waitangi Hotel (Appendix A, Photograph 16) and appears in reasonable condition, although the toe footing is becoming exposed in places indicating that the beach levels may have lowered since initial construction. Some loose rock has been placed at the eastern end of this hotel wall, though the adjacent cliff is located some 5 m landward indicating a trend of ongoing background erosion.

Figure 2-3 Waitangi Wharf 1907 (Source: Waitangi Museum)

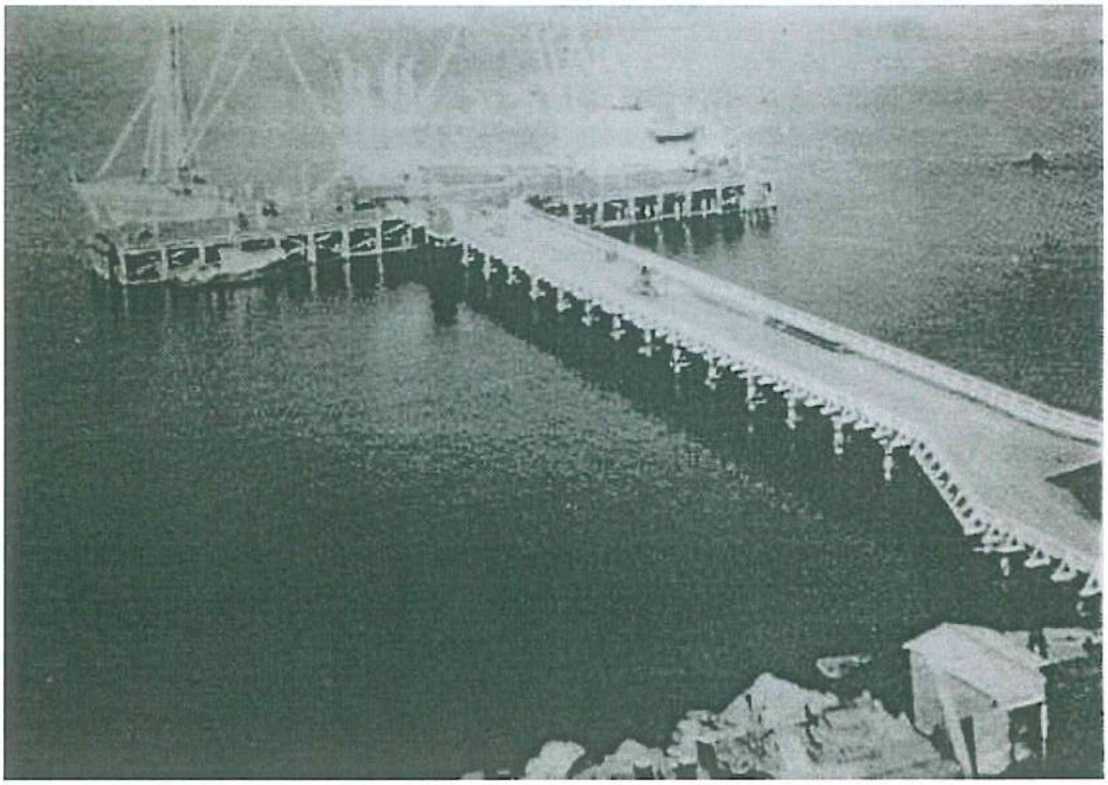


Figure 2-4 The wharf at Waitangi Bay before upgrades of the late 1970s. The M.V. Holmdale is berthed (Source: F. Holmes; Chatham Islands 1791 - 1984)

Figure 2-5 Waitangi circa 1870s (Source: Canterbury Museum in. King, M. and Morrison, R., 1990. A Land Apart)

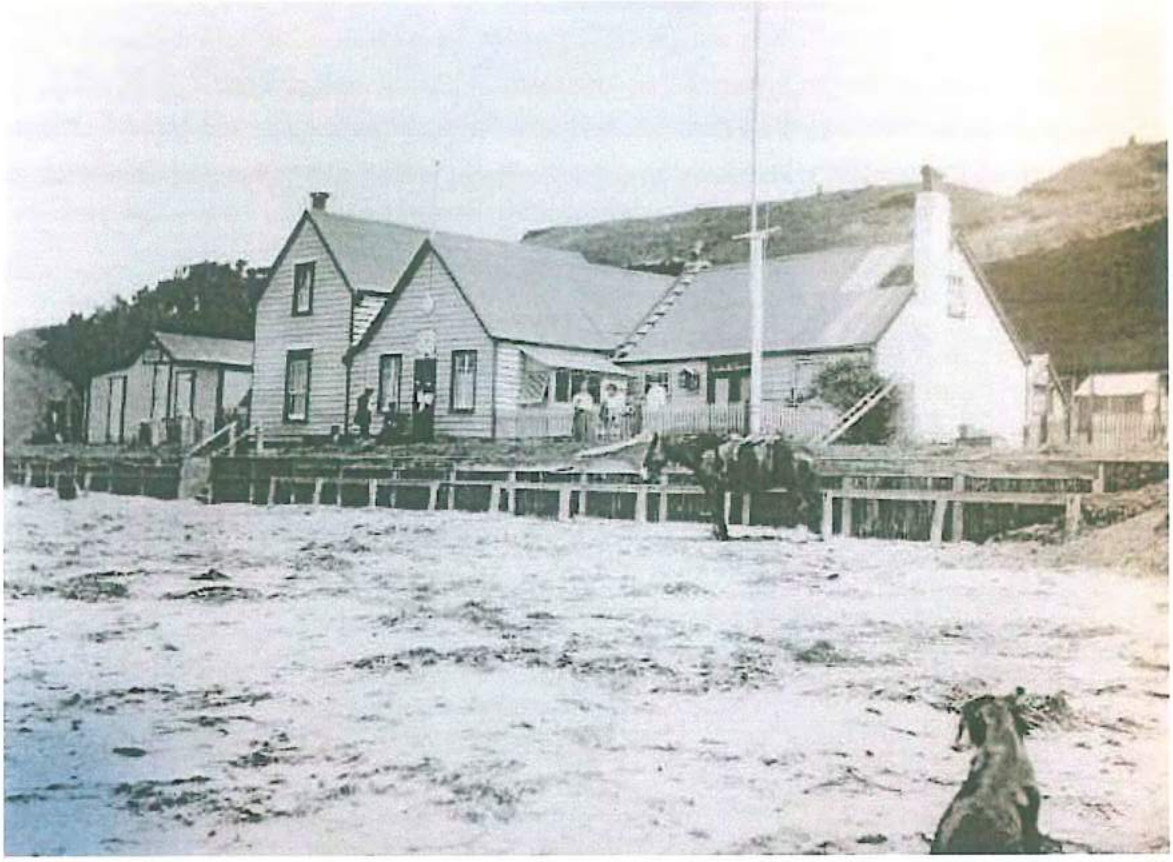


Figure 2-6 Beach fronting the Mangoutu Hotel circa 1910 (precursor to the Waitangi Hotel constructed in 1956) (Source: Guest Collection, Alexander Turnbull Library in. King, M. and Morrison, R., 1990. A Land Apart)

2.4 Historical shoreline changes

The position of the coastal shoreline, as defined by the intersection of the high tide level, may change over time through both erosional and accretionary processes, although cliff coastlines are generally subject to erosion only. Long- term rates of shoreline change are determined by comparison of the historic shoreline position. This is achieved by georeferencing historic aerials photographs to a consistent scale and datum and digitising the shoreline position. This corresponds to the cliff toe for cliff coastlines and vegetation line for beaches.

A list of historic aerial photographs and satellite imagery used in the analysis are provided in Table 2- 1. The accuracy of the georeferencing is estimated compared to the 2012 satellite image by comparing the location of ground control point such as buildings. Accuracy in the locating of the cliff to position is estimated based on the image resolution and contrast with low light and shadow over the cliff toe all potentially decreasing accuracy. The resultant potential error of these independent factors is derived using the sum of independent errors approach whereby $E_{sum} = \sqrt{E_1^2 + E_2^2 + \cdots + E_n^2}$.

An example of historic shoreline positions shown on the 2013 Satellite image is presented in Figure 2- 7 with all historical shoreline data presented in Appendix B. Software developed by Tonkin & Taylor has then be used to measure the distance to each shoreline from an assumed baseline at 50 m increments. A linear regression analysis is then undertaken on each set of shoreline measurements to estimate long- term retreat rates. Results are shown in Figure 2- 8 and, while some fluctuation in rates is apparent, general trends for the differing coastal compartments are evident.

Table 2-1 List of historic aerials and maps used in analysis

Year	Item	Scale or Ground Sample Distance	Source	Estimated georeference accuracy[1] (m)	Estimated shoreline accuracy[2] (m)	Resultant estimated error[3] (m)
9 Nov 1969	Aerial Photo SN2196 G/2	1:24,000	Opus	±5	±5	±7
24 Mar 1982	Aerial Photo SN8066 G/3	1:25,00	Opus	±5	±5	±7
7 Apr 2006	Satellite Image	0.4	Digitalglobe	±2	±2	±2.8
29 Nov 2013	Satellite Image	0.4 m	CNES/Astrium	-	±2	±2

[1]Relative to 2013 Image

[2]Shoreline accuracy estimated based on image resolution and ambient lighting causing contrast of cliff and beach

[3]Resultant estimated error derived using the sum of squares for summing independent parameters

To the northwest of the wharf (Figure 2- 7) the cliff coastline has been eroding at rates of up to - 0.25m/year, although average rates are - 0.08 m/year. Erosion of such cliff coastlines is often episodic, with no erosion over a long period followed by large amounts during a landslip. The artificial shoreline between the wharf and Waitangi Town Beach has been omitted as changes have been the result of human reclamation. Similarly at the western end of the Waitangi Town Beach the presence of seawalls has concealed any natural trends, although anecdotally beach levels (which are not picked up by aerial photo analysis) have dropped over this time. This is partially evident comparing Figure 2- 5 and Figure 2- 6 from the late 19th and early 20th centuries to today (Appendix A, Photograph 9, 16). Along the western end of the Waitangi Town Beach the base of the tuff embankment has eroded by up to 9 m since 1969 (up to - 0.2m/year) and the low tuff headland to the east has retreated by up to a similar amount. East of the Nairn River (to around 700 m), the shoreline eroded up to 10 m between 1969 and 1982. Since 1982 the shoreline has accreted by up to 17 m. This is evident in Appendix A, Photograph 22, where a low accretionary foredune is evident some 15 m in front of an older hind- dune. These changes result in long- term accretion rates of 0.3 to 0.6 m/year but realistically the trends are cyclical rather than constant with little movement having occurred since 2006.

It is therefore reasonable to assume that fluctuations of ± 10 to 15m are likely to occur along this beach. Such fluctuations are natural processes related to sequences of storms and calm periods and longer- term climatic cycles influencing sediment supply and average wave direction.

Further north, between 700 m and 1800 m from the Nairn, trends are erosive at rates of up to - 0.3 m/year (average - 0.1 m/year). This is evidenced by the high, over- steepened dunes in this area undergoing active toe erosion (Appendix A, Photograph 23). This trend appears more stable over the long- term and could be expected to continue. Further north trends become cyclical or negligible.

Figure 2-7 Example of historic shoreline positions superimposed on the 2013 Satellite Image

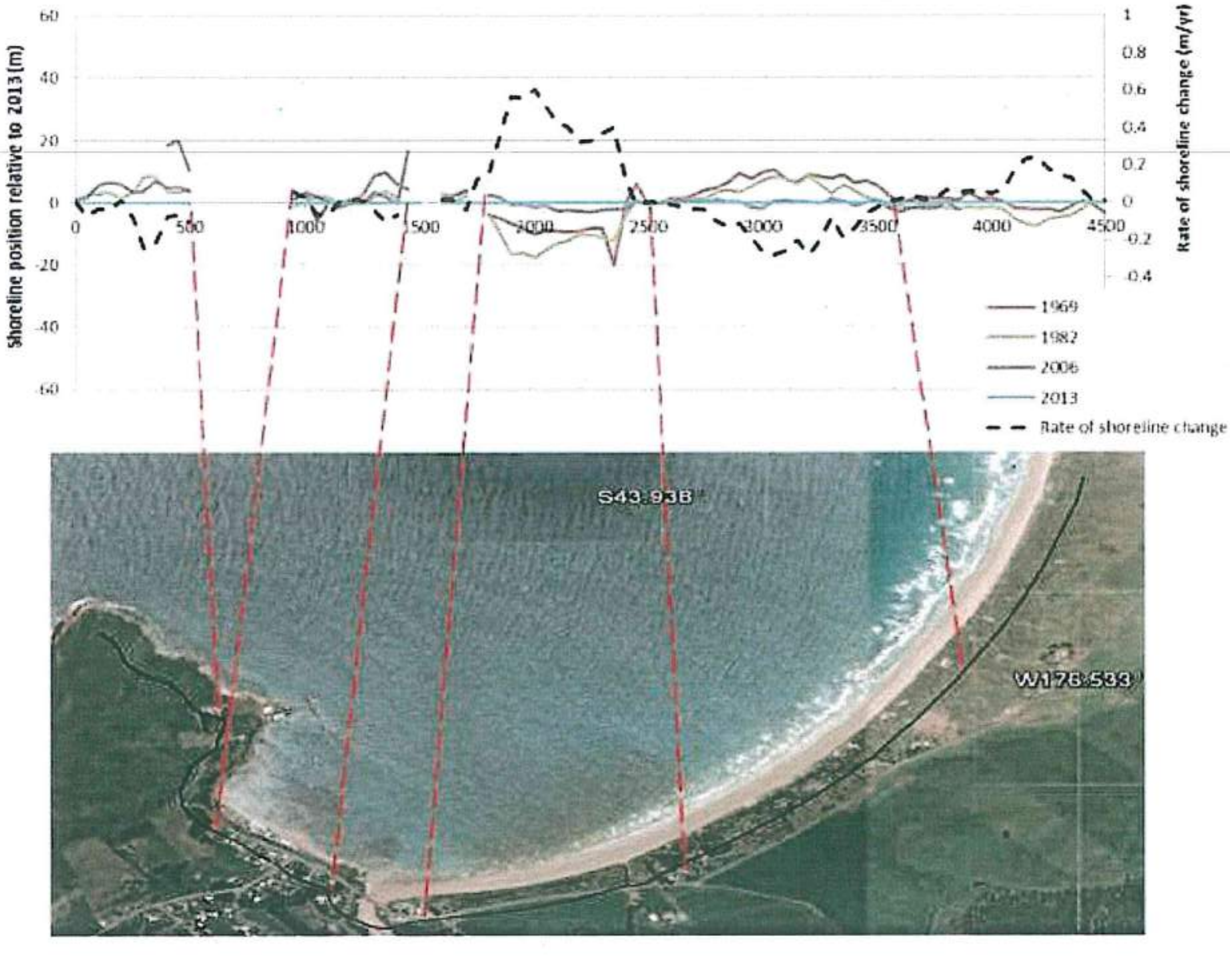


Figure 2-8: Historical shoreline position and rates of change

2.5 Sediments

Sediments on the Chatham Islands beaches are primarily of marine origin (Williams, 1995) and have accumulated in the lee of the southern Chatham volcanic outcrops as sea levels stabilised to their present stage.

During investigations, 20 samples were collected from the beaches, nearshore and offshore between 500m northwest of Waitangi Wharf to 1.5 km northeast of Waitangi with their locations shown in Figure 2- 9. Tests indicate material to be fine- medium white- grey sands with some shell. Size ranges from D_{50} = 0.15 to 0.3 mm with a medium grading offshore and adjacent the wharf to D_{50} = 0.125 mm with a more uniform grading on beach (Figure 2- 10). Solid density of the sediment ranges from 2.72 – 2.79 t/m^3.

Around the cliffs and wharf these marine sands are combined with small volumes (estimated at less than 5- 10% near the wharf, reducing to 1- 5% near the beach) derived from erosion of the adjacent Tuff cliffs. Based on 20 m high cliffs eroding at an average rate of - 0.08 m/year, up to 1120 m^3 of sediment could potentially be delivered into the nearshore system from the 700 m of cliff coastline northwest of the wharf.

Figure 2-9 Sediment sample numbered locations and median size (D_{50}) in mm.

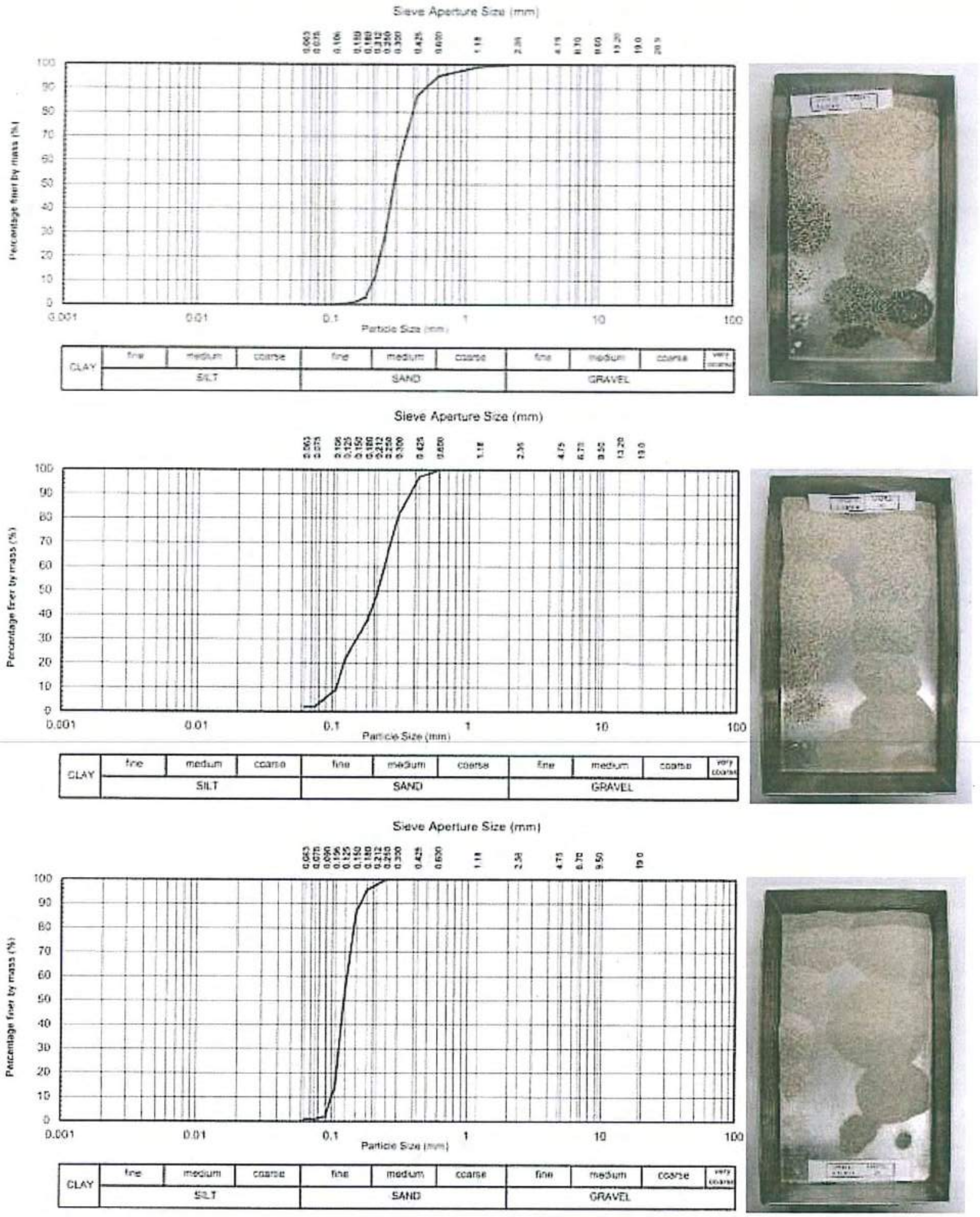


Figure 2-10 Example of sediment gradings from the base of the sea cliffs northwest of the breakwater (top; Sample 5, D_{50} = 0.28mm), at the end of the wharf (centre; Sample 15, D_{50} = 0.21 mm) and 500m along Waitangi Beach (bottom; Sample 25, D_{50} = 0.125m)

2.6 Wind

Observed wind data from Waitangi Bay is scarce with NZ Metservice (Thompson, 1983) compiling daily (9 am) records between 1972 and 1981 (Figure 2- 11). Records show wind direction is well distributed, although occurs most frequently from the northwest to southwest. A numerical analysis of a 35 year (1979 – 2013) wind field comprised of hourly data offshore of Port Webb was undertaken by Metocean (2015a). This data has been validated against coastal and open- ocean wind stations around the world with good agreement with 10 minute mean wind speed at 10 m elevation above sea level.

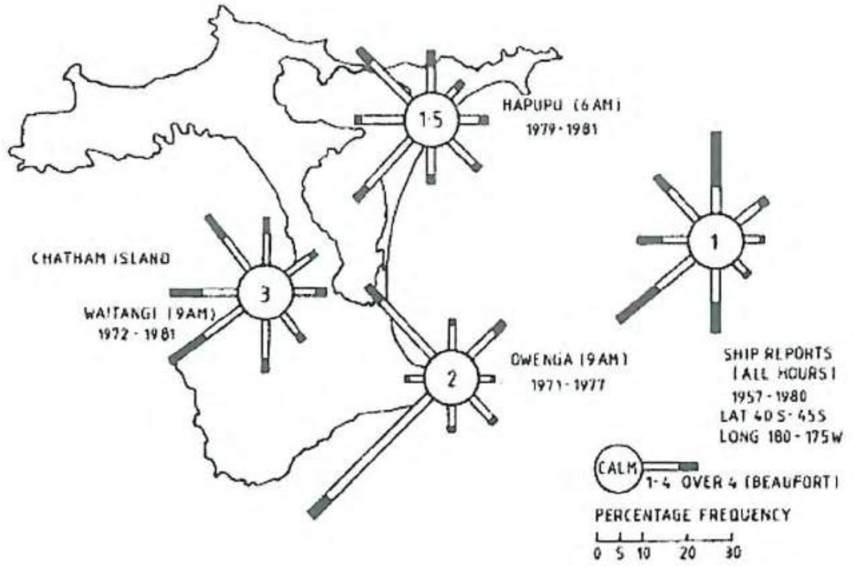
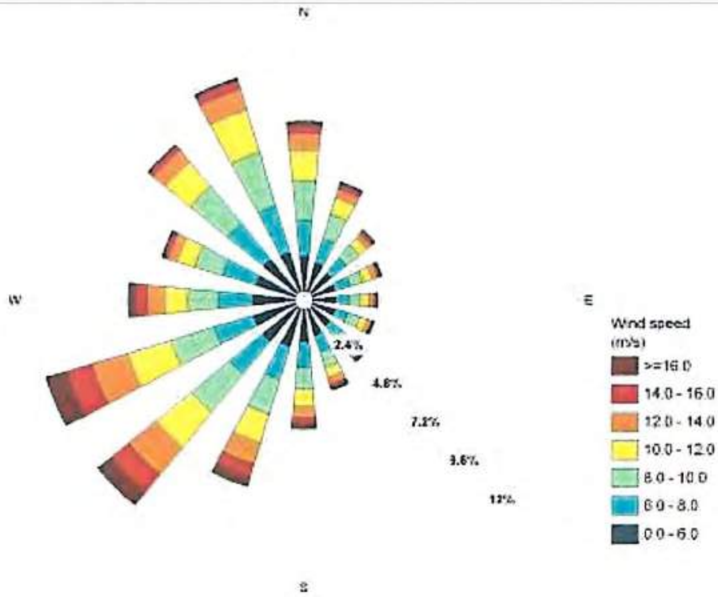


Figure 2-11: Wind frequency record (source: Thompson, 1983)



Results (

Figure 2- 12 and Table 2- 2) show similarly well distributed wind direction, although wind from the southwest quarter is more dominant. Peak wind speeds of 22- 24 m/s occur from the south to west, although can also occur from the north, potentially as the tail end of ex- tropical cyclones descend on the islands.

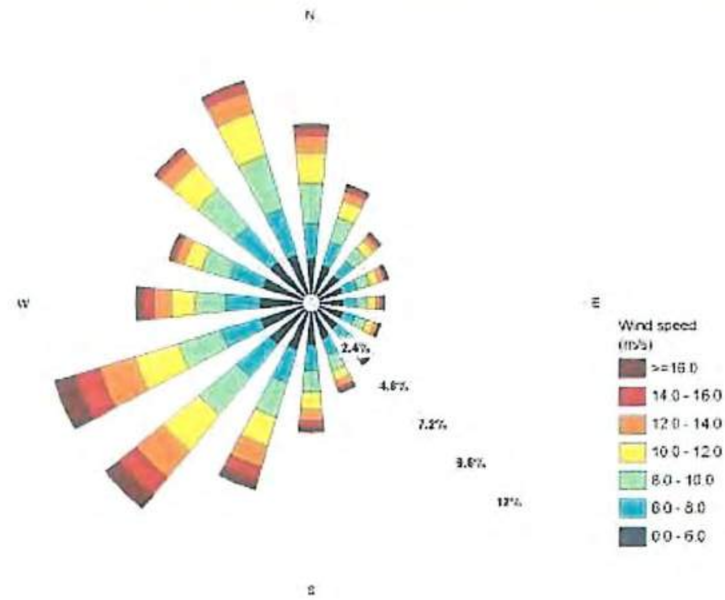


Figure 2-12: Wind rose offshore of Point Webb (source: Metocean, 2015a)

Table 2-2: Annual joint distribution (parts per thousand) of wind speed and wind direction offshore of Point Webb (source: Metocean, 2015a)

U (m/s)	Wind direction (degT)								
	337.5 - 22.5	22.5 - 67.5	67.5 - 112.5	112.5 - 157.5	157.5 - 202.5	202.5 - 247.5	247.5 - 292.5	292.5 - 337.5	Total
> 0 <= 2	3.0	2.3	2.5	2.7	2.8	3.0	3.1	2.9	22.3
> 2 <= 4	9.1	7.7	6.5	7.3	9.2	11.8	11.9	10.8	74.3
> 4 <= 6	19.1	12.8	10.7	11.3	17.3	23.7	22.7	24.1	141.7
> 6 <= 8	31.8	16.9	12.1	13.4	22.2	36.1	30.1	39.3	201.9
> 8 <= 10	36.3	15.9	10.9	12.1	21.1	40.5	29.0	40.9	206.7
> 10 <= 12	27.9	10.0	7.6	8.2	16.1	38.0	23.6	28.2	159.6
> 12 <= 14	14.8	5.7	4.1	4.9	10.5	29.7	17.4	13.8	100.9
> 14 <= 16	6.3	2.7	2.0	2.1	6.0	19.3	10.7	5.0	54.1
> 16 <= 18	2.2	1.3	0.8	1.1	3.0	9.3	5.3	1.4	24.4
> 18 <= 20	0.8	0.5	0.3	0.4	1.1	4.3	2.3	0.4	10.1
> 20 <= 22	0.2	0.2	0.1	0.2	0.3	1.3	0.6	0.1	3.0
> 22 <= 24	0.1	0.0	0.0	0.0	0.1	0.2	0.1	0.0	0.5
Total	151.6	76.0	57.6	63.7	109.7	217.2	156.8	166.9	1000.0

2.7 Water levels

The water level at any location varies across a range of timescales. Key components that determine water level are:

- Astronomical tides
- Barometric and wind effects, generally referred to as storm surge

- Medium term fluctuations, including El Nino Southern Ocean (ENSO) and Interdecadal Pacific Oscillation (IPO) effects
- Long- term changes in sea level
- Wave breaking can also contribute to water level through wave setup and runup. This is discussed in the following section.

A 14 year record (2001 - 2014) of hourly measured water level has been collected at Waitangi Port by the University of Hawaii Sea Level Centre (UHSLC). Metocean (2015a) analysed this data to provide astronomical tide and storm surge values.

2.7.1 Mean sea level

The mean level of the sea from year to year varies depending on cyclical changes such as; the 2- 4 year El Nino- Southern Oscillation (ENSO) cycle, the 20- 30 year Inter- decadal Pacific Oscillation (IPO) and long- term sea level changes.

LINZ (2012) give the present mean sea level (MSL) at 2.35 m below LINZ mark EHN1 (Waitangi BM1) and Chart Datum at 0.48 m below MSL.

2.7.2 Astronomical tide

Astronomical tide is the periodic rising and falling of the level of the sea surface caused by the gravitational interaction of the sun and moon on the earth's waters and harmonics of such interactions. A tidal table is derived for Port Waitangi based on Metocean (2015a) sea level analysis and information provided by LINZ (pers. comm. Jan 2015).

Table 2-3 Astronomical tidal levels for Port Waitangi (source: Metocean, 2015a)

Level Description	Tidal level (m) m CD[1]	MSL
Highest Astronomical Tide (HAT)	1.05	0.57
Mean High Water Spring (MHWS)	0.96	0.48[1]
Mean High Water Neap (MHWN)	0.79	0.31
Mean Sea Level (MSL)	0.48	0
Mean Low Water Neap (MLWN)	0.17	- 0.31
Mean Low Water Spring (MLWS)	0.13	- 0.35
Lowest Astronomical Tide (LAT)	0	- 0.48[1]

[1]Values provided by LINZ per. comm. (Jan, 2015)

2.7.3 Storm surge

Storm surge results from the combination of barometric setup from low atmospheric pressure and wind stress from winds blowing along or onshore which elevates the water level above the predicted tide (Figure 2- 13). The combined elevation of the predicted tide and storm surge is known as the storm tide. Storm- surge applies to the general elevation of the sea above the predicted tide across a region but excludes nearshore effects of storm waves such as wave setup and wave runup at the shoreline.

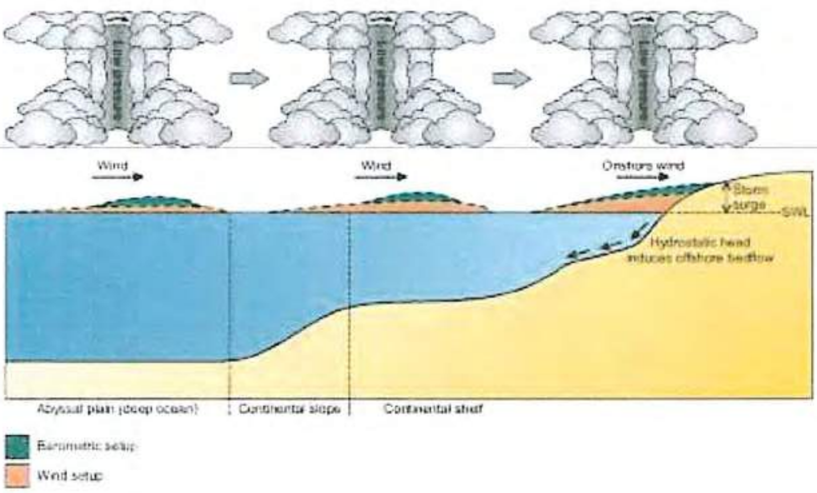


Figure 2-13 Processes causing storm surge

Metocean (2015a) analysed the 14 year sea level record separating storm surge from astronomical tide. Extreme value analysis showed that the 100 year average recurrence interval (ARI) storm surge at Waitangi was 0.6 m which is in general agreement with other New Zealand ports. The combination of astronomical tide and storm surge is known as storm tide and, as the components are independent, values are typically less than simply adding the storm surge to a given high tide value. Extreme value analysis shows a 100 year ARI storm tide of 0.86 m above MSL, approximately 0.4 m above the MHWS.

Table 2-4 Extreme value analysis of storm surge and total still water level (source: Metocean, 2015a)

Water level (m)	ARI (years)							
	5	10	20	50	100	200	500	1000
Storm surge (positive)	0.47	0.50	0.53	0.57	0.60	0.63	0.67	0.69
Storm surge (negative)	-0.37	-0.40	-0.43	-0.47	-0.50	-0.52	-0.56	-0.59
Total still water level (positive)	0.73	0.76	0.78	0.82	0.84	0.86	0.90	0.92
Total still water level (negative)	-0.64	-0.66	-0.68	-0.70	-0.72	-0.73	-0.76	-0.77

2.7.4 Tsunami

Tsunami are a series of waves generated when a large volume of water is rapidly displaced by such events as earthquakes (normally > M5) and their associated fault ruptures (especially dip- slip faulting of the seabed), volcanic eruptions, coastal landslides and submarine slides and meteor impact (GNS, 2005). The Chatham Islands may be vulnerable to Tsunami generated regionally (i.e. from the Hikurangi Margin) and from distance sources such as Tonga and South America.

The Chatham Islands has experienced Tsunami throughout its recorded history with a severe tsunami generated in Peru in 1868 devastating Tupuangi Village, a tsunami occurred in 1931 during the initial wharf construction with materials lost from the wharf surface, runup of 2.5 to 4 m was observed during the 1960 South Chile event and the Chatham Islands experienced the highest tsunami in New Zealand during the 2009 Tonga event of 0.89 m at Kaingaroa.

While Waitangi is more protected from these far field events, it remains vulnerable to regional events originating from New Zealand. Power (2014) estimates 100, 500 and 2500 year return period tsunami on the Chatham Islands west coast as 5 m, 9 m, 12 m+. Sea level rise

Sea levels have historically been rising around New Zealand (Hannah and Bell, 2012) at average rates of 1.3 mm/year (Dunedin) to 2.2 mm/year (Wellington) and with a NZ- wide average rate of 1.7 mm/year. While analysis has not been undertaken on the Waitangi sea level data, the landmass is tectonically stable (Williams, 1995) and so rates of sea level rise are expected to be comparable.

Ongoing changes in the global climate are predicted to result in acceleration of this sea level rise in coming decades. The Ministry of Environment (2008) guidelines recommends a base value sea level rise of 0.31 m at 2065 with consideration of the consequences of a rise of 0.45 m (relative to the 1980- 1999 average). Likewise, a base sea level rise of 0.5 m by 2100 is recommended with consideration of the consequences of sea level rise of at least 0.8 m with an additional sea level rise of 10 mm per year beyond 2100 (refer Table 2- 5).

Table 2-5: Baseline sea level rise recommendations for different future timeframes (MfE, 2008)

Timeframe	Base sea-level rise allowance (m relative to 1980–1999 average)	Also consider the consequences of sea-level rise of at least: (m relative to 1980–1999 average)
2030–2039	0.15	0.20
2040–2049	0.20	0.27
2050–2059	0.25	0.36
2060–2069	0.31	0.45
2070–2079	0.37	0.55
2080–2089	0.44	0.66
2090–2099	0.50	0.80
Beyond 2100	10 mm/year	

2.8 Waves

2.8.1 Wave hindcast

Waves occurring within Waitangi Bay are expected to be predominantly swell waves generated by the dominant westerly airstreams south of 40° latitude. Irregular west to north- west waves result from ex- tropical weather systems descending from the north and wind- waves are generated within South Petre Bay by north to northeast winds.

No instrumented wave data is available for the Chatham Islands. Metocean Ltd. have therefore produced a numerical wave hindcast using the numerical wind field described previously for a 35 year period between 1979 and 2013. The numerical model SWAN is a 3rd generation ocean wave propagation model which allows for wave growth, refraction and decay of wave fields. The resulting wave climate has been verified using satellite altimeter data recorded between 2010 - 2012.

Three levels of model downscaling are used to transform the wave fields from a global model domain (11 x 11 km resolution), to a domain including only the Chatham Islands (1 x 1 km resolution) and finally to a model domain including the south part of Petre Bay (50 x 50 m resolution). An example of the model bathymetry and an example wave field are shown in Figure 2- 16. Output points are provided in 20 m water depth 800m north of Point Webb, in 10 m water depth 300 m north of the existing wharf and in 6 m water depth 90 m north of the existing wharf.

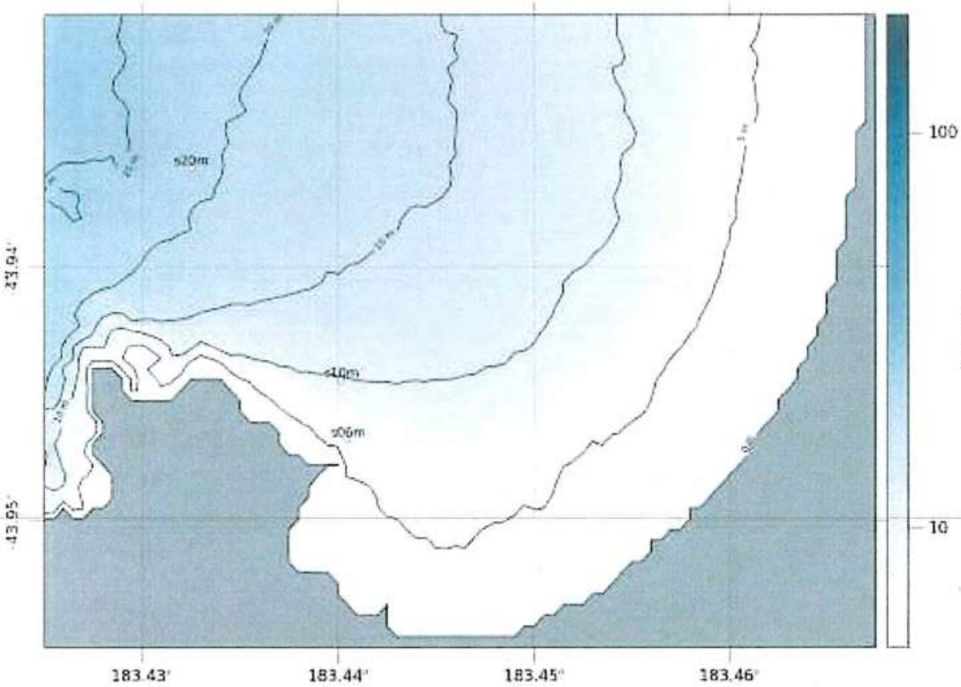


Figure 2-14: Swan model nearshore bathymetry (source: Metocean, 2015a)

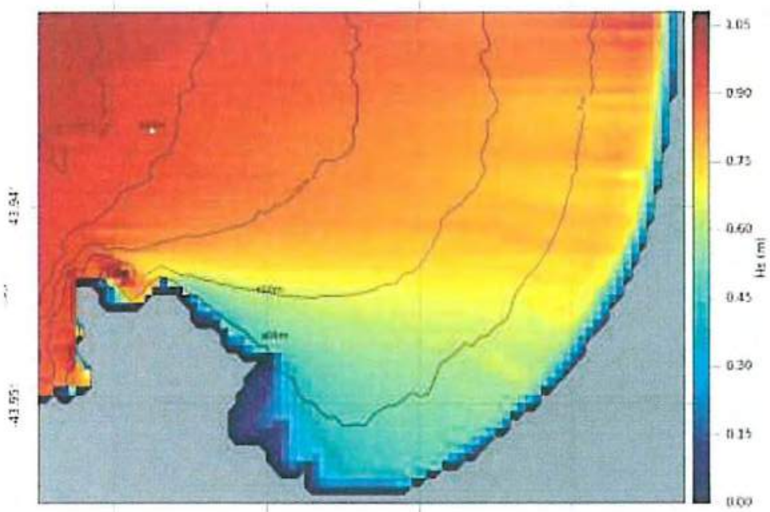


Figure 2-15: Example of modelled significant wave height in the nested Petre Bay domain (source: Metocean, 2015a)

Time series wave data are available for each of these locations as well as summary statistics which are provided in full within Appendix C. Result show that waves are transformed from a dominant westerly direction in 20 m depth to northwest by 10 m depth and north- northwest by 6 m depth near the existing wharf head. In 6 m depth the majority (90%) of waves occur from 332 to 335 degrees with periods from 11 to 15 s. A subset of wind waves generated locally within Petre Bay occur from 350 to 90˚ and reach heights of up to 1.6 m with periods up to 5 s.

Overall, over 50% of significant wave heights are less than 0.5 m with a further 30% between H_s = 0.5 - 0.75 m. Around 6% of waves exceed 1 m with a maximum significant wave height of 2.19 m occurring in August 1988. A greater proportion of larger waves occur during winter rather than summer, although waves exceeding 1.75 m can occur at any time of year.

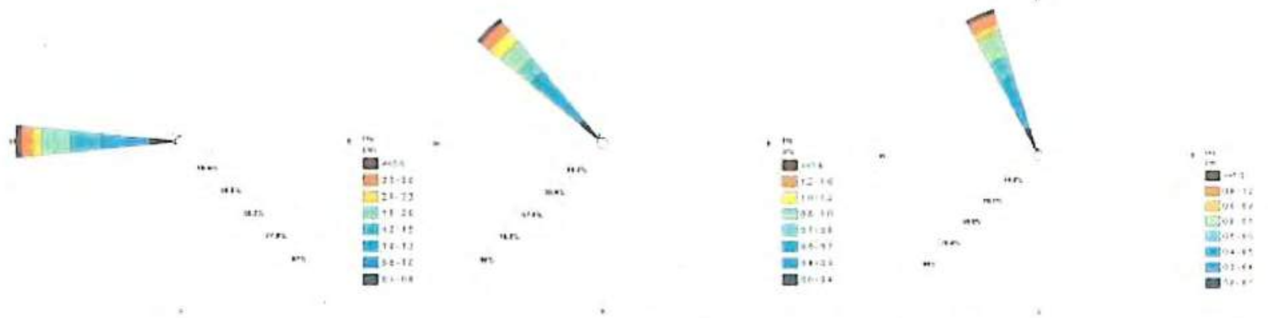


Figure 2-16: The transformation in the wave direction rose from 20 m depth (left) to 10 m (centre) to 6 m depth (right) (source: Metocean, 2015a)

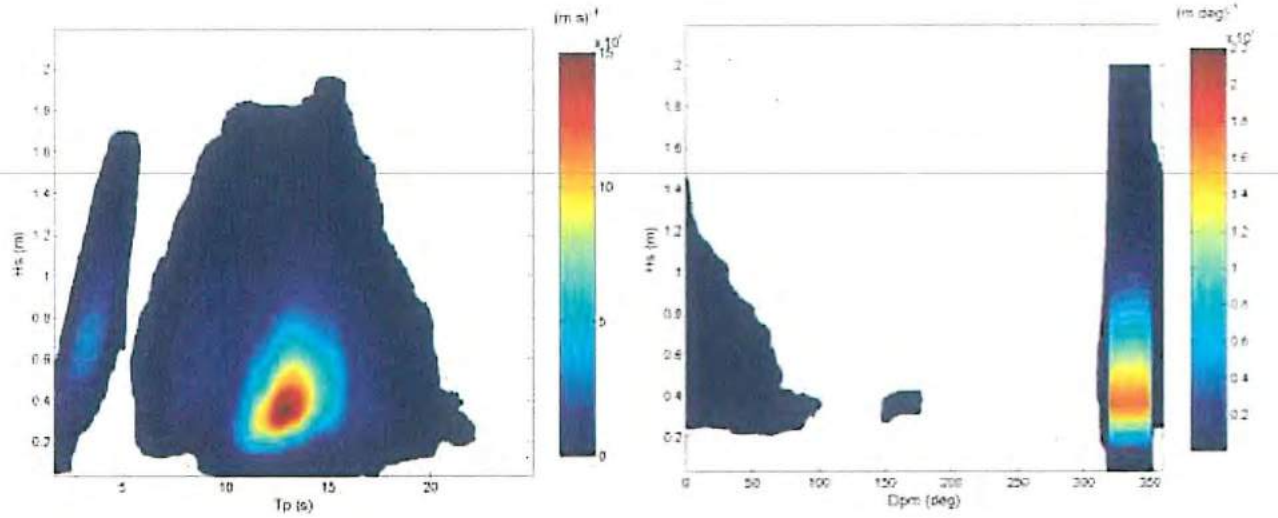


Figure 2-17: Density plot of total significant wave height vs peak wave period (left) and significant wave height vs peak wave direction (right) at outpoint point S06 (source: Metocean, 2015a)

Table 2-6: Annual significant wave height exceedance probabilities at S06

H_s (m)	Exceedance (%)	Average number of days per year exceeding
0 - 0.25	8.3	30
0.25 - 0.5	43.4	158
0.5 - 0.75	29	106
0.75 - 1.0	12.7	46
1.0 - 1.25	4.5	16.5
1.25 - 1.5	1.51	5.5
1.5 - 1.75	0.5	0.4 (10 hours)
1.75 - 2.0	0.01	0.04 (1 hour)

2.8.2 Extreme wave climate

An extreme wave climate including both swell and local wind waves has been produced for output point S06 in 6 m water depth approximately 80 m north of the existing wharf (refer Figure 2- 15). The results show a 100 year ARI swell- dominated extreme wave height of H_s = 2.22 m with an associated period of 13.8 s. A 100 year ARI wind- sea height of H_s = 2.01 m has an associated peak period of 5.5 s. Wave heights at the wharf structure are expected to be slightly smaller due to the increased sheltering nearer the shore. Note that waves of up to twice the significant height could occur during extreme conditions, i.e. over 4 m for the 100 year ARI swell event.

Table 2-7: Extreme wave climate at output point S06 (source: Metocean, 2015a)

Parameter	Units	ARI (years)							
		5	10	20	50	100	200	500	1000
Wind speed (10 min mean)	m/s	27.3	28.7	30.0	31.7	33.0	34.3	36.0	37.2
Significant wave height, H_s (swell dominated)	m	2.05	2.10	2.14	2.18	2.20	2.22	2.23	2.24
Peak wave period, T_p (swell dominated)	s	13.3	13.4	13.6	13.7	13.7	13.8	13.8	13.9
Significant wave height, H_s (wind- sea dominated)	m	1.71	1.79	1.86	1.95	2.01	2.06	2.12	2.15
Peak wave period, T_p (wind- sea dominated)	s	5.1	5.2	5.3	5.4	5.5	5.6	5.7	5.7
Maximum individual wave height, H_{max}	m	4.43	4.59	4.74	4.93	5.06	5.20	5.37	5.49
Maximum individual crest height, C_{max}	m	2.38	2.47	2.55	2.66	2.73	2.80	2.90	2.97

2.8.3 Nearshore processes

As waves move into the nearshore they interact with the seabed and begin to turn towards the seabed contours (known as refraction) and as they pass a headland, wave energy is transferred along the wave crest into the shadow region (known as diffraction). The spectral SWAN wave model does not resolve wave diffraction well and phase resolving (wave by wave) models are preferred.

The numerical refraction- diffraction model CGwave has been used to model wave propagation from 20 m water depth into south Waitangi Bay using a high resolution nearshore digital terrain model described previously (Metocean, 2015b). CGWAVE simulates the combined effects of wave refraction-diffraction within the mild- slope equation, and includes the effects of reflection, wave dissipation by friction, breaking, nonlinear amplitude dispersion, and harbour entrance losses (Panchang, and Xu, 1995) which means that it is ideal for resolving complex localised bathymetry and harbour walls in a numerically- efficient manner. This numerical model is an industry- standard tool for use in harbours and coastal regions with complex bathymetry.

Waves have been modelled for a range of incident directions at the boundary between 250 and 280° with periods between 10 and 16 s giving a total of 16 separate monochromatic simulations. Wave height is arbitrary with an adopted height used to find relative height elsewhere within the modelled domain. Initial modelling was undertaken using the existing bathymetry, reclamation and piled wharf structure. An example wave crest and wave height output is shown in Figure 2- 18 for 14 s waves occurring from 270° at the S20 offshore boundary (i.e. the average direction). Results show that waves refract in towards Waitangi Bay reaching the wharf from an almost north direction and wave energy moving into this shadow zone through diffraction. While the majority of wave energy reaches the beach north of the Nairn River, energy is focussed onto the reef offshore of the low headland adjacent the Nairn River and on the reef offshore of the hotel. Very little wave energy reaches the shadow zone west of the hotel reef.

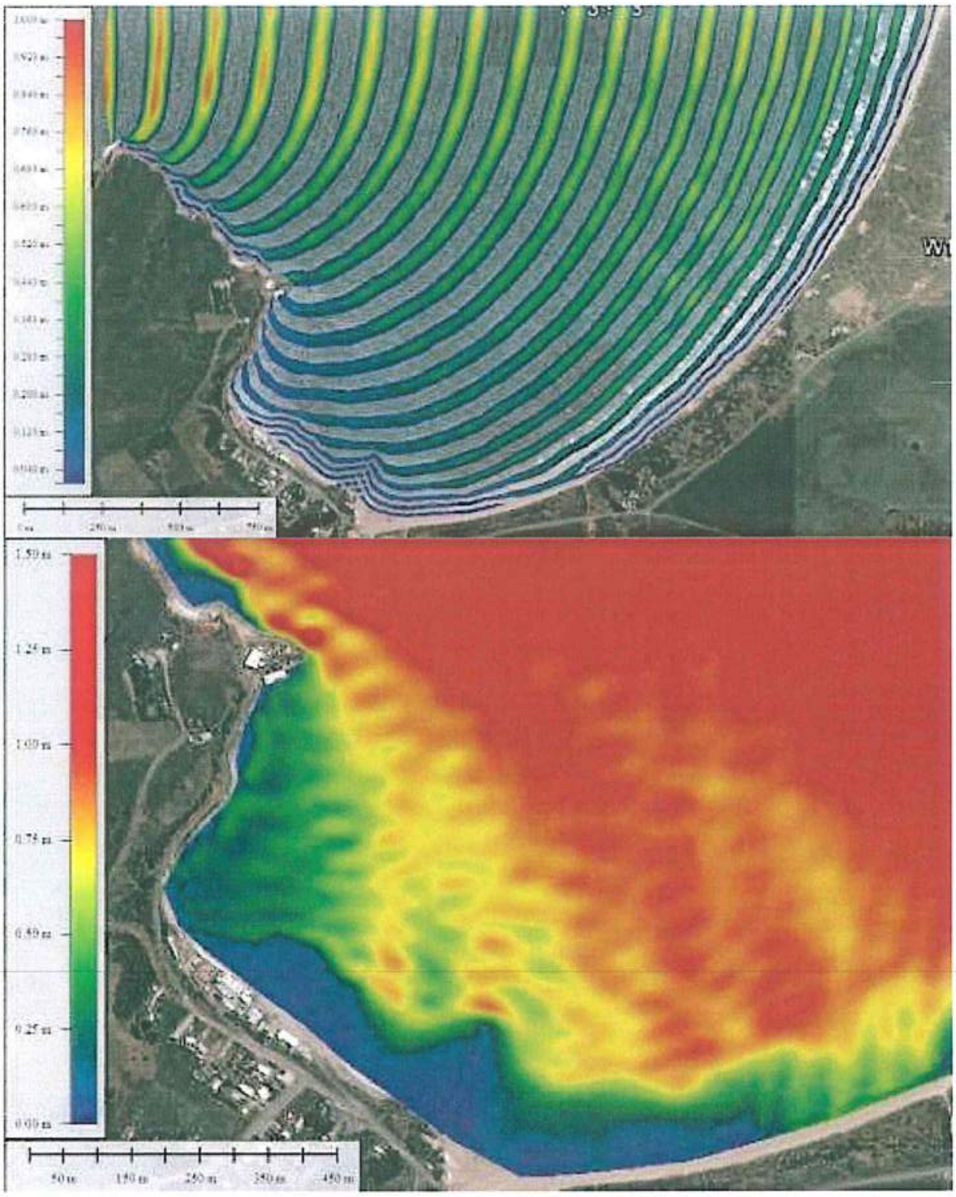


Figure 2-18 Example CGwave wave crest output (top) and wave height (bottom) for a 14 s wave occurring from 270° at the S20 offshore boundary

2.8.3.1 Wave reflection

Reflection off the seawall can be observed during high water levels. This reflection deflects waves that have refracted around the existing reclamation and are approaching from a northeast direction towards the beach from a northwest direction. While this has only been observed in relatively benign conditions, similar reflection reportedly occurs during high energy conditions.

Figure 2-19 Road seawall reflecting oblique incoming waves towards the beach

2.8.3.2 Wave run- up- and setup

Wave set- up is a super- elevation of the mean water surface over normal 'still' water level due to wave action alone. Following wave breaking, on- shore directed momentum flux or radiation stress is induced due to dissipation of wave energy. To balance this momentum flux, a pressure gradient is created by elevation of the water level. Water level is highest at the beach face, and drops towards the break point, creating an offshore gradient (Figure 2- 20). An associated process is wave run- up, which varies with breaking wave characteristics and beach and backshore slope and material. Wave run- up causes periodic wave swash above the inundation level and may contribute to flooding or cause damage to land and infrastructure within the impact zone.

Based on the extreme wave values derived above and the nearshore wave climate within Waitangi Bay, wave height during a 1% AEP event is assessed (Figure 2- 21) and wave setup and runup evaluated according to methods presented within the Coastal Engineering Manual (USACE, 2006). Results show wave setup to range from 0.1 to 0.4 m above still water level between the road seawall and open Waitangi Beach and wave run up to range between 0.6 and 1.3m.

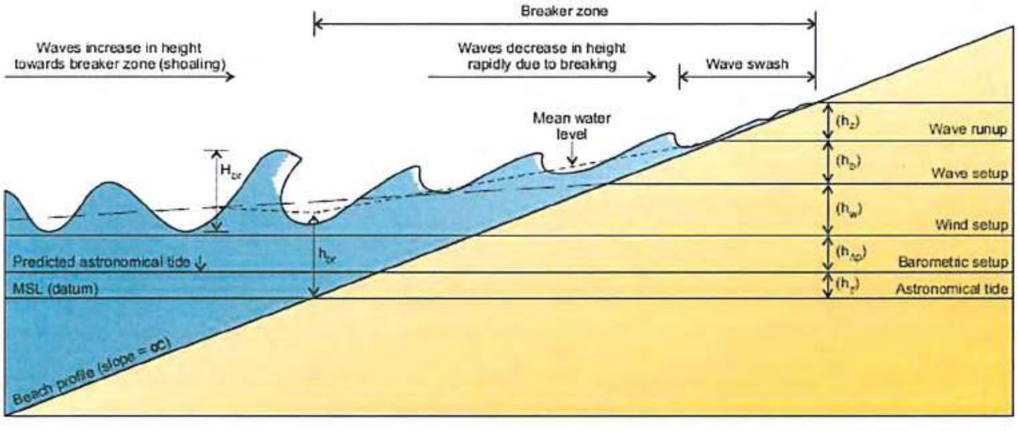


Figure 2-20 Schematic diagram showing components of wave runup level. (Frisby and Goldberg, 1981)

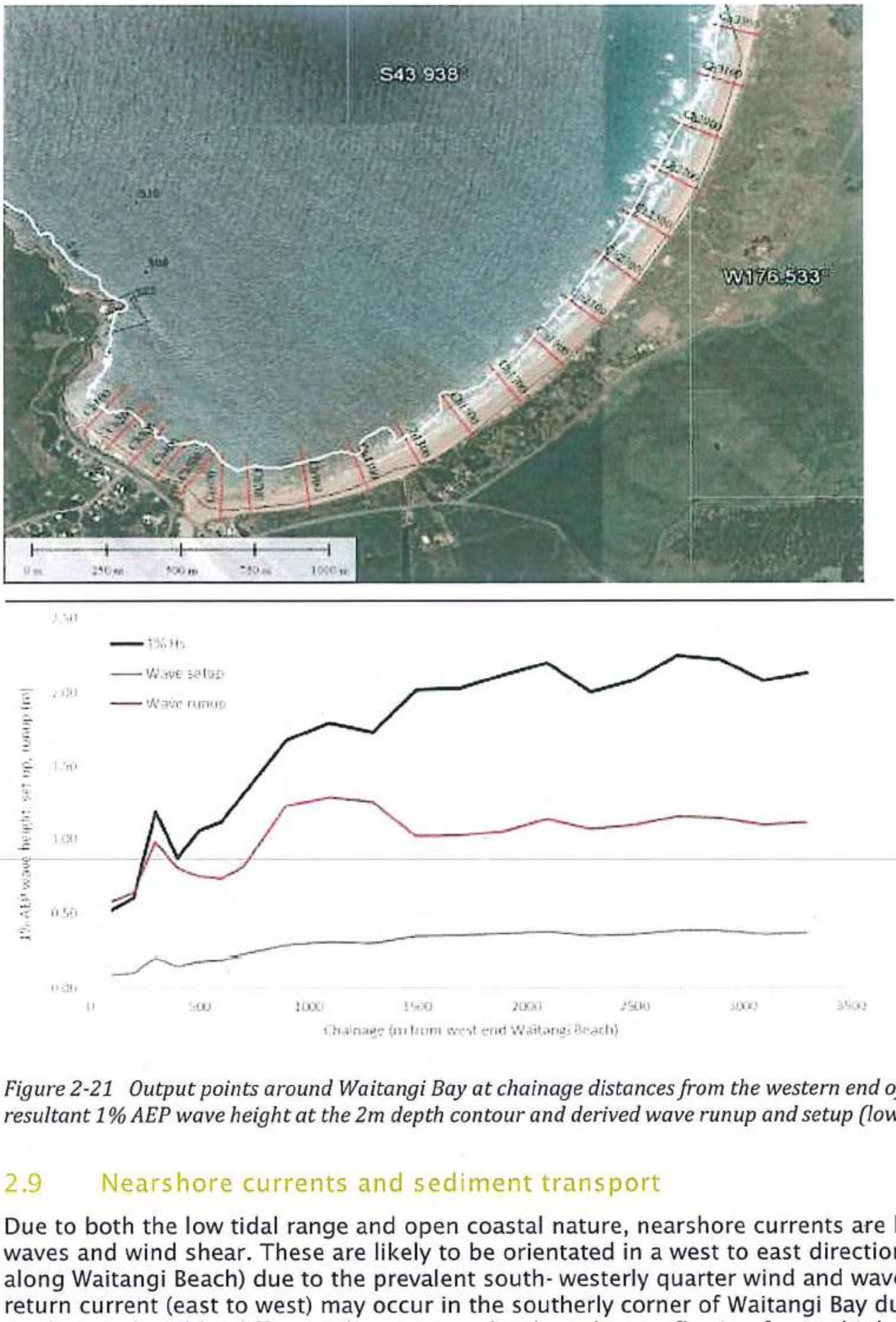


Figure 2-21 Output points around Waitangi Bay at chainage distances from the western end of the beach (top) and the resultant 1% AEP wave height at the 2m depth contour and derived wave runup and setup (lower)

2.9 Nearshore currents and sediment transport

Due to both the low tidal range and open coastal nature, nearshore currents are likely driven by waves and wind shear. These are likely to be orientated in a west to east direction (south to north along Waitangi Beach) due to the prevalent south- westerly quarter wind and wave climate. A small return current (east to west) may occur in the southerly corner of Waitangi Bay due to the water level gradient induced by differential wave setup levels and water flowing from a high to low elevation. It is unlikely that such return current would transport significant sediment volumes.

Sediment transport may occur in both the cross- shore and longshore directions with asymmetry of wave orbital velocities at the seabed driving cross shore transport and breaking wave orientation compared to shore normal driving longshore transport. Cross shore transport is likely to be offshore during storm events, returning onshore during calm periods. While large wave events may occur at any time of the year, they are more prevalent in winter. Erosion events are therefore likely to be concentrated during the winter months and rebuilding occurring over summer months. Beach profile data is not available to quantify the likely magnitude of storm- induced erosion events but based on similar low dissipative profiles along the west coast of New Zealand (i.e. Shand, 2008) the maximum potential horizontal retreat is likely to be in the order of 10- 15 m on the open Waitangi Beach and less than 5 m in the sheltered Town Beach (in areas not protected by seawall).Littoral drift is usually expressed in m^3/year of sand. For comparative purposes, the littoral transport rate under mean wave condition has been calculated for the existing situation using the Kamphuis/Queens sediment transport formula (Kamphuis, 2002, Eqn. 2- 1). A summary of the parameters used for littoral transport modelling is presented in Table 2- 8. These formulae calculate the sediment transport rate for the entire surf zone based on several physical parameters such as wave height, period and angle, sand grain size, surf zone slope, etc. The Kamphuis Model has been found in good agreement with physical and field studies without such parameter adjustment (Smith *et al.* 2003). Input wave height and direction parameters are obtained from CGwave modelling at intervals around Waitangi Bay (as shown in Figure 2- 21) using a mean wave height (1.4 m at location S20) and direction (270 degrees at S20). While this is an oversimplification of the actual processes where waves occur from a range of directions resulting in sediment transport in both directions, bulk transport formulae based on mean conditions have been found to provide good indication of general trends (Kamphius, 2010) and for the comparative purposes used here are deemed sufficient.

$$Q_s = \frac{7.3}{3600} H_b^2 . T_p^{1.5} . m_b^{0.75} . D_{50}^{-0.25} . \sin^{0.6}(2\alpha_b) \qquad (2\text{- }1)$$

Table 2-8 Sediment transport parameters

Parameter	Physical Description	Value
Q_s	Littoral transport rate	[m^3/s]
ρw	Density of sea water	1025 [kg/m^3]
ρs	Density of sand	2650 [kg/m^3]
γ	Breaker index	0.65 [-]
n	Porosity	0.40 [-]
Hb	Significant wave height at break point	Varies [m]
αb	Wave angle at break point	Varies [°]
Tp	Peak wave period	12.5 [s]
mb	Bed slope at break point	0.014 to 0.002 [-]
D50	median grain size	0.125 [mm]

Results (Figure 2- 22) show the mean significant wave height and peak period for swell, compared to shoreline orientation and the potential longshore sediment transport capacity under these swell conditions. Results show a general south to north transport, driven by the dominant south- west wave direction and increasing as the coastline becomes more exposed. However, the lack of supply from the western end of the bay and observed lack of sediment at the base of cliffs or on the beach indicates that the system contains a dearth of sediment. Therefore, while the potential sediment transport capacity can be calculated, the actual transport is likely to be significantly lower owing to this deficit. A reduction in transport rate is observed between chainage 500 and 700 m in front of the Nairn River Entrance. This reduction is likely due to the effect of the submerged reef fronting the low tuff headland which rotates the incoming waves to near shore parallel. While a large sediment accumulation is not evident at this location, the Nairn River is a sediment sink (refer following section) and accumulation has recently occurred over several hundred metres to the north.

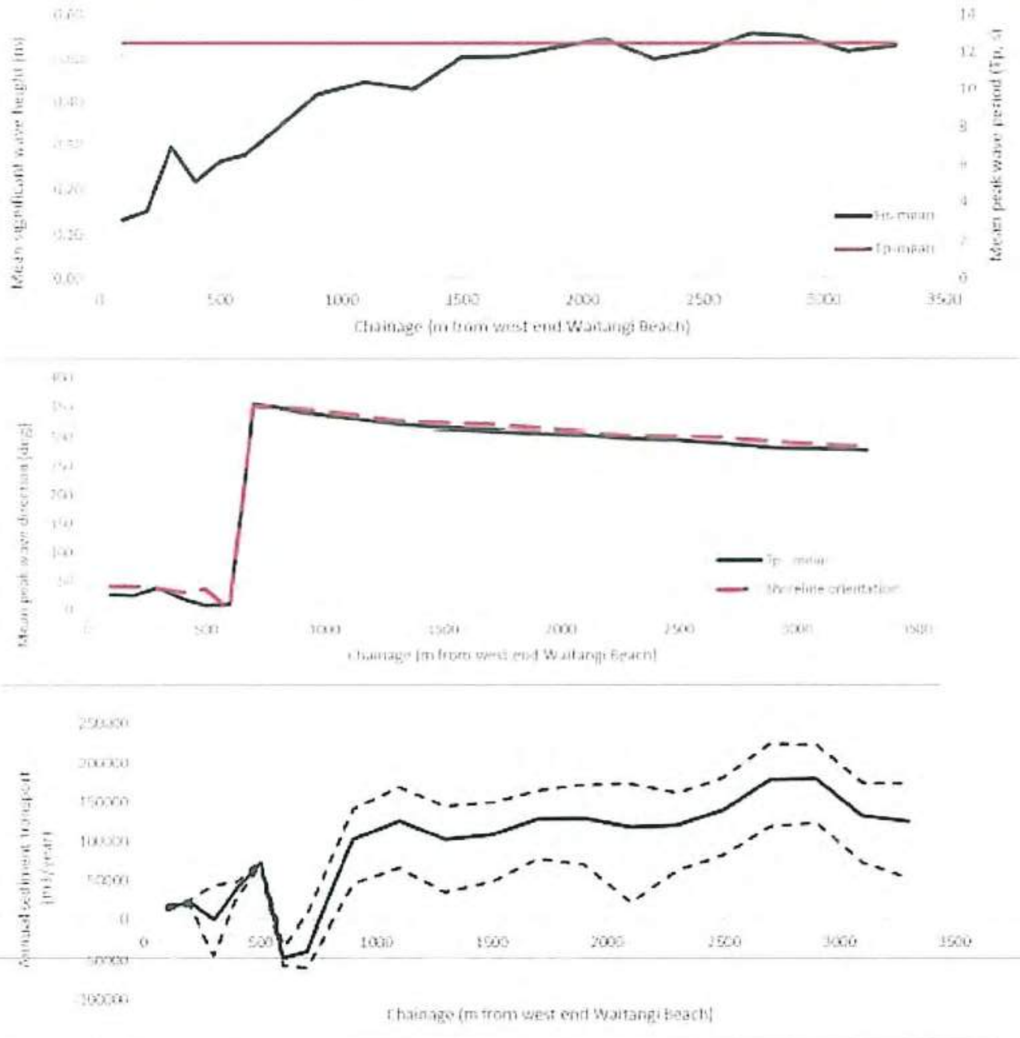


Figure 2-22 Annual mean wave height and period (top), direction and coastline orientation (middle) and potential longshore sediment transport capacity (lower) including the effect of ±5° wave directional error

2.10 Nairn River

The Nairn River discharges into Waitangi Bay adjacent a Tuff Headland (Appendix A, Photograph 21). Williams (1995) reports that the Nairn River has a total catchment area of 6500 ha with a mean annual flow in the upper river of 0.56 m^3/s. Due to the relatively low flows and the moderate wave energy and potential sediment movement at the mouth, the lower Nairn River is a sediment sink and contains large volumes of marine sediment (beach sands) that has been moved into the river mouth by wave processes and partially blocked the entrance. Such sediment transport dynamics are typical of high energy coastlines with entrances often becoming completely blocked until large rainfall events cause the waterway to break out with sediment redistributed back onto the beach. Once the flood water has drained waves begin pushing sediment back into the entrance and the refilling process begins again. These river systems are known as Intermittently Closed and Open Lagoons (ICOLL). Where flooding behind such entrances is problematic (particularly immediately before a breakout) it is often managed by manual excavation of sand from the river mouth.

The bridge over the lower Nairn River was initially constructed in 1947 and has reduced the width at the entrance from an estimated 70 m initially to 35 m today. Early imagery of the lower Nairn River (Figure 2- 23) showed similar coastal sediment accumulation indicating that this has been a long-term process. Sediment accumulates over a 15,000 m^3 area up to 300 m up- steam of the Waitangi Rd Bridge with historical imagery in Figure 2- 24 showing the extents and volume of sediment vary over time. While geotechnical investigation and long- term monitoring is required to make accurate estimates of sedimentation volumes and rates, it is likely that the lower river contains between 5,000 and 30,000 m^3 of marine sediment at any time.

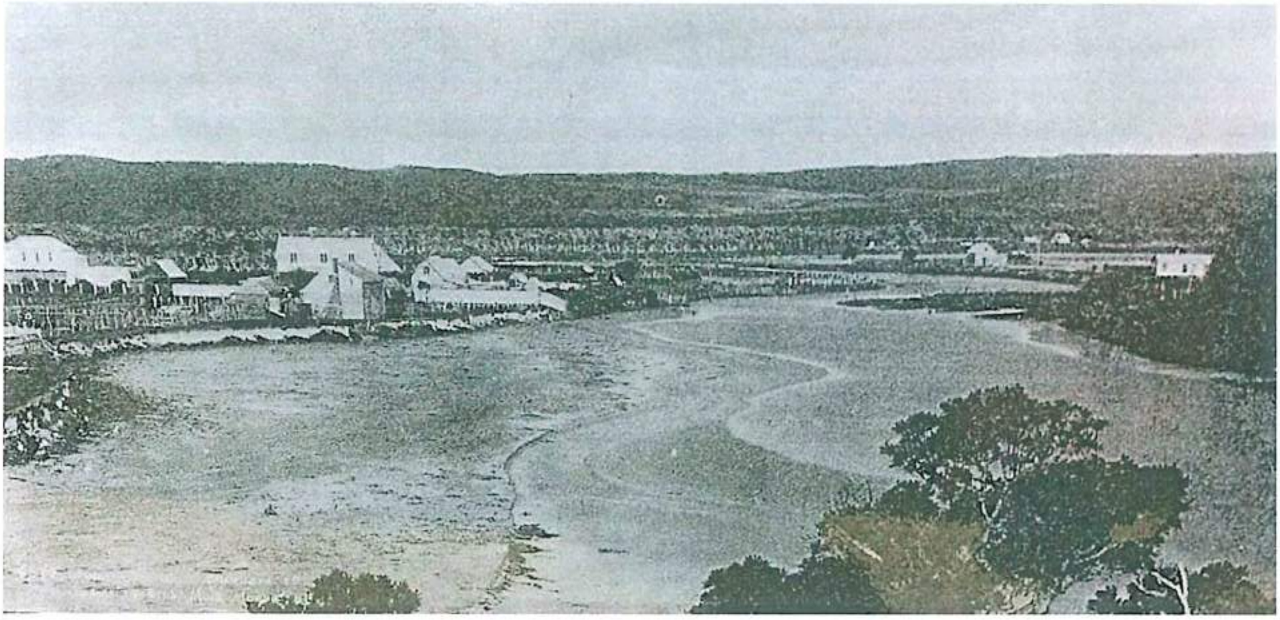


Figure 2-23 View of the Nairn River circa 1910. Source: Waitangi Museum

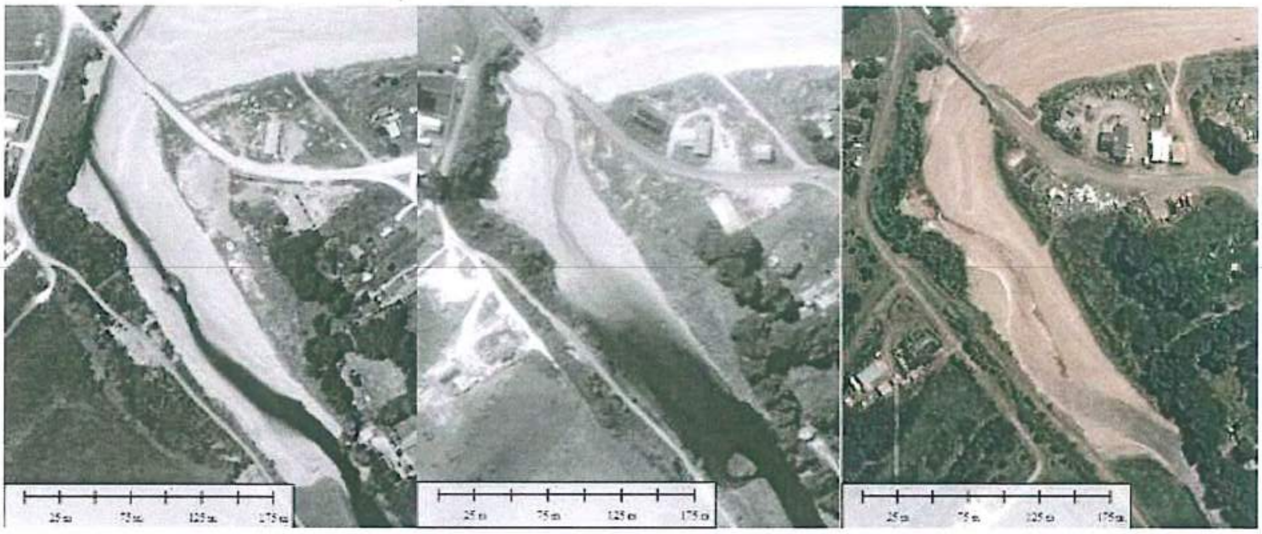


Figure 2-24 Example of sediment accumulated in the lower Nairn River in 1969 (left), 1982 (centre) and 2013 (right)

2.11 Coastal stability

2.11.1 Equilibrium planform

The equilibrium coast angle within southern Waitangi Bay was modelled using the software Mepbay (Klein et al., 2003) which is based on the parabolic bay model of Hsu and Evans (1989). The current port reclamation is assumed as the headland control point with incoming wave angle aligned to the dominant offshore direction. Results (Figure 2- 25) show that the beach geometry of southern Waitangi Beach closely approximates a parabolic bay shape which is typical of headland controlled beaches and indicates that the southern part of the bay is close to dynamic equilibrium.

The beach fronting Waitangi Township (termed the Waitangi *town beach*) is out of alignment from the equilibrium coast angle within the wider bay. This beach differs from that to the east in that it is a perched beach, with a layer of sand overlying a rock platform at the toe of a tuff bank. It is likely that this area is being maintained in its present position/ alignment by the offshore reefs and the small rock outcrop at the eastern end of the beach. Analysis of historical aerial photographs (Section 2.4) show that this outcrop, comprised of a weak tuff material, has eroded up to 9m since 1969. As this headland retreats, the control on the beach is lost and sand can migrate to the east.

Figure 2-25 Analysis of an equilibrium planform for southern Waitangi Bay

2.11.2 Climate change effects

As discussed previously, sea levels have been rising around New Zealand over the past century and are expected to continue to rise in the future. As sea level rises the morphology of the beach profile is expected to respond. The most widely known model for this beach response is that of Bruun (1962). The Bruun model assumes that as sea level is raised, the 'equilibrium profile' is moved upward and landward conserving mass and original shape (Figure 2- 26). This profile translation effectively results in a recession of the coastline, although the actual coastal response will depend on longshore transport and the wider sediment budget. The model may be defined by the following equation:

$$R = \frac{L_*}{B + h_*} S \qquad (4\text{-}2)$$

Where SL is the landward retreat, d_* defines the maximum depth of sediment exchange, L_* is the horizontal distance from the shoreline to the offshore position of d_*, B is the height of the berm/dune crest within the eroded backshore and S is the sea level rise.

For a given historic sea level rise of 85 mm over the past 50 years, the Bruun model predicts shoreline recession of 4 m at the Waitangi Town Beach, increasing to 7 m to the north along Waitangi Beach. Given the coastline here is close to a stable equilibrium angle (Figure 2- 25), historic sea level rise could explain the background erosion observed here.

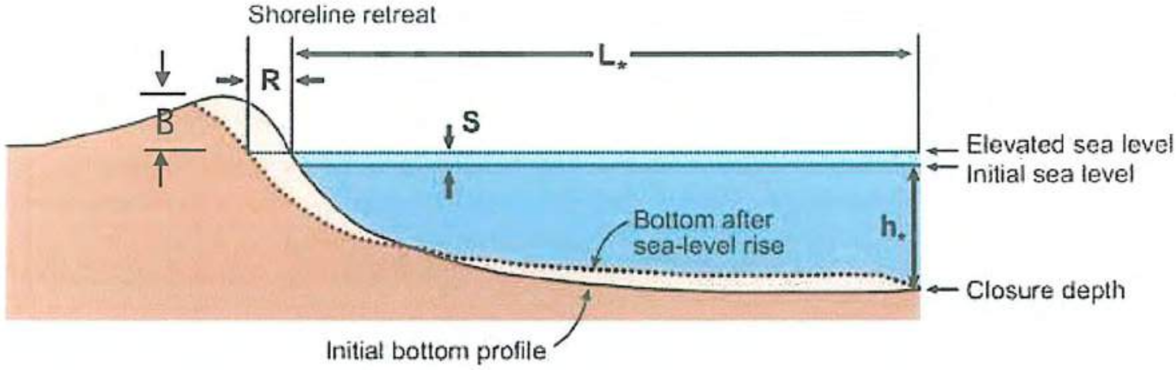


Figure 2-26 Bruun model of shoreline response to sea level rise

2.11.3 Effect of seawalls

Seawalls are constructed where the natural landward migration of the shoreline impacts on human assets. The seawall is intended to protect the land behind the structure only. They do not protect the fronting beach and, if the coast is in a state of long- term recession, the beach will gradually be lost in front of a wall (i.e. as shown in Figure 2- 27). Similarly, seawalls will not protect adjacent land from ongoing erosion and the erosion will continue adjacent to any constructed wall, potentially at an increased rate due to turbulence, reflected waves and the deficit caused by sediment trapped behind the wall.

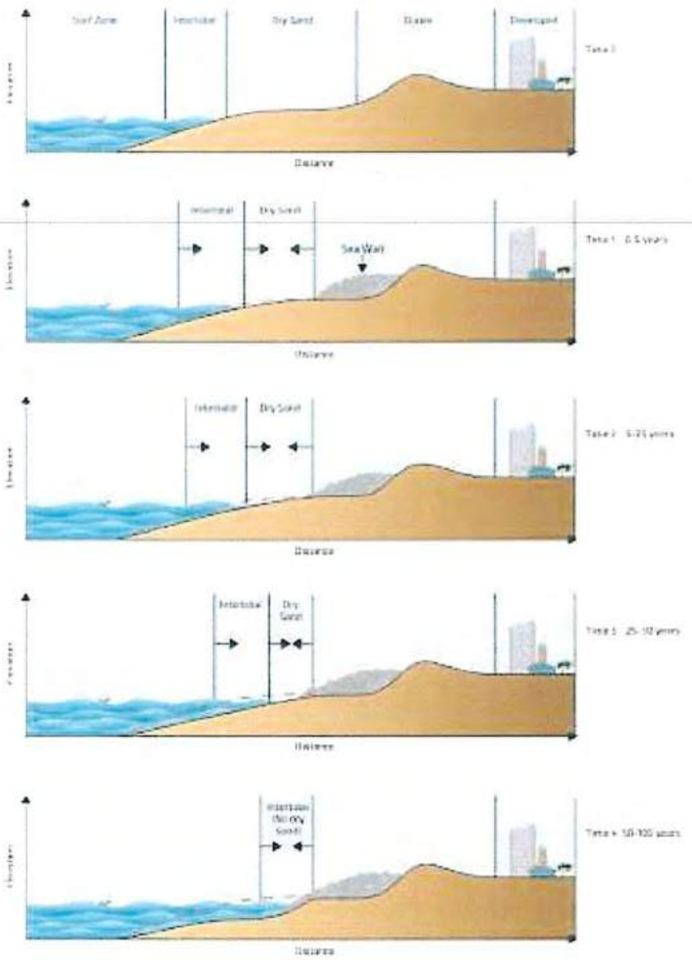


Figure 2-27 Example of the long-term shoreline response to a seawall on an eroding beach

2.12 Coastal process summary

Waitangi Bay is located at the south- eastern corner of Petre Bay, and is defined on its western side by the rocky headland of Tikitiki Hill, and on its eastern side by Waitangi Beach which extends north to Red Bluff. Waitangi Beach is an accretional feature formed by accumulation of Pleistocene aged marine sediment in the lee of the southern Chatham volcanic outcrops as sea levels stabilised to their present level and has resulted in the formation of the Te Whanga Lagoon system.

Sediment on the beaches are generally of marine origin (Williams, 1995) with small volumes derived from erosion of adjacent Tuff cliffs. Sediment movement along the coastline is expected to be typically south to north, driven by the dominant south- west wave direction. The beach geometry along southern Waitangi Beach to the Nairn River entrance fits a parabolic bay shape that is typical of headland controlled beaches indicating that the southern part of the bay is close to dynamic equilibrium. Analysis of historical aerial photographs (1969 and 1982) and recent satellite imagery shows that the beach adjacent the Nairn River has fluctuated up to 20 m over this time while the beach further north has been in a state of long- term erosion. Such fluctuations are natural processes related to sequences of storms and calm periods and longer- term climatic cycles influencing sediment supply and average wave direction.

The lower Nairn River is a sediment sink and contains large volumes of marine sediment (beach sands) that have been moved into the river mouth by wave processes and have partially blocked the entrance. Such sediment transport dynamics are typical of high energy coastlines with entrances often becoming completely blocked until large rainfall events cause the waterway to break out with sediment redistributed back onto the beach.

The beach fronting Waitangi township differs from that north of the Nairn River. It is a perched beach, with a layer of sand overlying a rock platform at the toe of a tuff bank. While the beach has been present as far back as settlement (refer Figure 2- 5), its low volume makes it more susceptible to erosion during storms or to changes in the sediment budget (i.e. the balance of sediment additions and losses) than the open coast beaches. The beach here is out of alignment with the wider bay and we believe is being maintained in its present position/alignment by the offshore reefs and the small rock outcrop at the eastern end. Analysis of historical aerial photographs show that this outcrop, comprised of a weak tuff material, has eroded up to 9 m since 1969. As this headland retreats, the control on the beach is lost and sand can migrate to the east. Early images show a wider beach than presently exists, but also show ponga breastwork constructed to combat coastal erosion. This indicates that cycles of erosion and accretion have long influenced this beach, although are likely to have become exacerbated recently by erosion of the adjacent headland to the west, ongoing sea level rise and potentially by wave reflection off the nearby Waitangi Wharf Rd seawall.

The wharf was moved from its original location in the western corner of the bay to the present location at Hanson Point in the early 1930s. Previously surf boats would load cargo at a short jetty and row out to the trading vessels offshore. A 385 ft (117 m) long timber wharf with a 202 ft by 26 ft (60 x 8 m) 'Tee' section was constructed to enable larger trading vessels to berth directly. Depths off the berth at the time of construction were 15 to 17 ft (4.6 – 5.2 m). A road was excavated out of the cliff face to reach the new wharf with a concrete seawall eventually constructed to protect the road. The wharf was upgraded to a reinforced concrete structure in 1979/1980 and a small reclamation at the base of the wharf was extended to approximately 65 m in length (2,800 m^2) to allow construction of port facilities.

Analysis of the historic aerial photographs and satellite images shows that the Tuff cliffs to the northwest of the wharf have been eroding at average rates of 0.1 to 0.25 m/year. While this represents up to 1100 m^3 of sediment input annually, the fine material is likely to be quickly lost offshore. The lack of sediment accumulation on the seaward side of the reclamation or in the sheltered lee adjacent to the fishing wharfs indicates a lack of sediment in the littoral system. Actual longshore sediment transport rates (i.e. northwest to southeast movement) are therefore likely to be substantially lower than the empirically- derived potential rates.

3 PROPOSED WORKS

Waitangi Wharf is nearing the end of its serviceable life and requires significant repairs to maintain freight services to the Chatham Islands. The wharf also requires an upgrade to improve the port operations facilities and berthing and usability at the wharf.

3.1 Design philosophy

The project is intended to

- Improve level of service

The existing concrete T- wharf used by commercial vessels currently protrudes into Waitangi Bay and is significantly exposed to weather and sea conditions. As a consequence, berthing vessels in all weather is not possible. From discussions with the commercial vessel operators, it is estimated that the vessels servicing the island each lose on average 40 days per year due to undesirable weather. As a consequence of exposed sea conditions the vessels often:

- Cannot manoeuvre into the wharf berth; and/or
- Cannot unload/load cargo (including any livestock which may be consolidated already in holding pens waiting to be loaded); and/or
- Cannot hold the boat on the berth due to unfavourable conditions; and/or
- Can suffer damage to vessels (and the wharf) when attempting to berth.

This project aims to decrease the number of days lost to undesirable weather by reducing the wave climate at the berth.

- Achieve compliance with current legislation

The project provides an opportunity to upgrade the existing wharf operations to meet the relevant legislative requirements and standards of similar facilities across New Zealand.

- Improve health and safety

The project has been designed to improve the health and safety for port and shipping company employees and the general public. The Health and Safety in Employment Act governs the operations proposed at the wharf.

- To minimise maintenance during the project design life

Due to the remote nature of the site and the difficulty and expense of mobilising plant for upgrade and repair, the design should seek to minimise maintenance requirements over the design life of the project where this is cost- effective.

3.2 Design Conditions

3.2.1 Environmental conditions

The likelihood of a design event impacting a structure is a function of both the probability of event occurrence defined by the annual exceedance probability (AEP) or its inverse an average recurrence interval (ARI) and the timeframe being considered. Given a 50 year design life, there is a 40% likelihood of a 1% AEP event (100 year ARI) occurring or a 5% likelihood of a 0.1% AEP event (1000 year ARI). Given the critical nature of the facility and the difficulty in undertaking repairs, a 0.1% AEP event is deemed appropriate.

Design criteria based on this are presented below:

Table 3-1 Design criteria for coastal protection works

Design criteria	Commentary	Value
Design life	Time period over which structure is expected to remain functional	50 years
Design event annual exceedance probability (AEP)	Probability of event being exceeded during any year	0.1%
Sea level rise to 2065	IPCC emission scenario A1F1 upper limit	0.5 m
Design water levels	MHWS Annual event at 2065 0.1% AEP at 2065	RL 0.5 m RL 1.30 m RL 1.55 m
Design wave heights	$P_{50\%}$ wave height Annual event inc 20% factor of safety Swell Wind-waves Design 0.1% AEP inc. 20% factor of safety Swell Wind-waves	H_s = 0.488m, T_p = 13.0s H_s = 2.3m, T_p = 13.0s H_s = 1.9m, T_p = 5.0s H_s = 2.7m, T_p = 13.8s H_s = 2.6 m, T_p = 6 s
Acceptable overtopping discharge	Typical (working) conditions at wharf (Hs<0. 5m) Design 0.1% AEP event	Not hazardous for pedestrians/vehicles ($q < 0.02$ l/s/m) No damage to pavement $q < 20$ l/s/m

3.2.2 Design vessel

The proposed works including wharf length and height, breakwater length and requirements for dredging of approaches and berthing have been sized to accommodate a design vessel (refer Pacific Marine Management Ltd. 2015 for details). This design vessel has been sized to accommodate future growth in cargo volumes and has the following characteristics:

- An overall vessel length of 68m;
- A beam width of 11.4m;
- Maximum displacement of 2,631 tons; and
- A maximum operating draft of 4.3m.

3.3 Reclamation

A key physical element of the proposed works is the creation of a new landform extending from the existing reclamation seaward to the extent (approximately) of the existing concrete T-wharf to provide improved port handling and access. This will involve creating 11,780m² of new land (encompassing the existing reclamation) protected by a concrete armour unit revetment to the north, a vertical concrete wall on the east and south-east and a sloping rock revetment to the southwest (Appendix D). The topography of the new reclamation will vary across the site, to accommodate overtopping and provide additional protection from storm surges. The height differences are summarised as follows:

- Port operations and commercial wharf height on the new reclamation: 3.0 m; and
- Fishing wharf height and area: 2.0 m, set lower than the remainder of the port area similar to the existing situation.

3.4 Breakwater

3.4.1 Planform

A breakwater is a structure designed to absorb wave energy on its seaward face providing a region of reduced wave climate in its lee. Wave energy may reach this sheltered region by being transmitted through a semi- permeable breakwater (rock or concrete armour structure) such as proposed for Waitangi (typically less than 5% wave height) or by refraction and diffraction around the end of the breakwater as shown in Figure 3- 1. The breakwater length should be sufficient to achieve the desired reduction in wave height behind the structure.

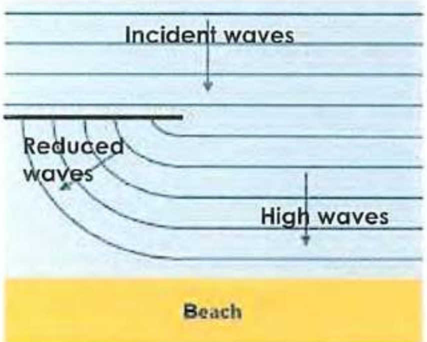


Figure 3-1 Diffraction around a breakwater head

The effect of breakwater lengths ranging from 40 to 90 m from the seaward end of the reclamation (measured to the RL0m contour) were tested. Final breakwater length remains to be finalised following more detailed mooring analysis but the effect of a 90 m long breakwater have been assessed as a maximum potential.

3.4.2 Material

A conventional breakwater contains a granular rock core, overlain by a filter layer to limit the loss of the smaller core material and covered by secondary and primary rock armour to protect the core material from wave attack. The armour layer needs to be large enough to withstand design wave heights without being displaced. Based on the assessed wave climate, a significant wave height of 2.7m with individual waves up to 5m has been design for. Rock sized using standard engineering methods would need to be 7- 15 Ton. Suitable rock is only available up to 300kg on island. This larger rock would need be imported from New Zealand and even these are difficult to source.

An alternative to rock are concrete armour units which can be manufactured in a range of designs from simple 'cubes' that replicate rock to more complex shapes that interlock. These interlocking units can be smaller than equivalent rock as they provide support to each other rather than acting as singular units. The disadvantage of these units is if they are damaged others can fail quickly. It is therefore important to oversize units to prevent failure.

After considering a range of potential armour units as described within PIANC (2005), Xbloc® units have been selected as most cost- effective and well- proven unit. Characteristics of Xbloc are:

- Single layer protection therefore required lower material volume;
- Highly interlocking therefore lower unit weight;
- Unreinforced;
- Used widely internationally since 1994 (20+ projects with 230,000 units placed);
- Sizing includes appropriate factors of safety for deep water and breakwater head.

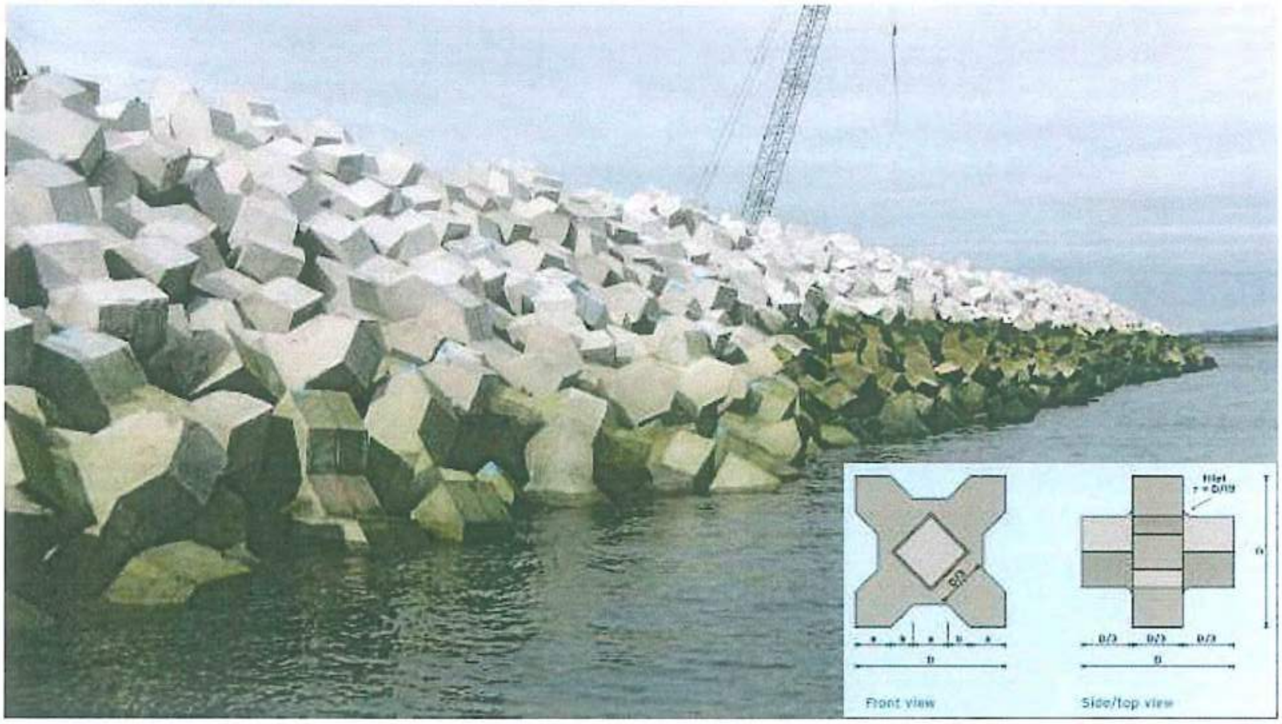


Figure 3-2 Example of breakwater constructed using Xbloc armour units (source: DMC, 2013)

3.4.3 Geometry

Cross- section of the proposed reclamation and breakwater are shown in Appendix D and have the following characteristics:

On northern side of reclamation and both sides of the breakwater (Appendix D: WAI- 15- 928, 929)

- Single layer Xbloc armour unit size = $0.75m^3$
- Side slope = 4(H):3(V)
- XBase unit at toe
- Toe either clad in 300kg armour rock and rafted on sand or excavated into the underlying rock
- A crest height of RL 4.6 m has been adopted to minimise overtopping to tolerable rates
- Along the reclamation, the crest is three Xbloc units wide backed by a concrete crown wall
- Along the breakwater, the crest is 6 m wide to allow width for construction plant
- Underlain by a 0.8m thick secondary armour layer of 60- 300kg local rock
- Underlain by filter layer up to 0.8m thick separating the core from the armour rock
- A Geotextile layer is used to separate the core from the armour layers above mean sea level

On southern side of reclamation (Appendix D: WAI- 15- 928)

- Two layer rock armour 60- 300 kg
- Side slope = 2(H):1(V)
- Toe clad in armour rock and rafted on sand
- Underlain by filter layer up to 0.8 m thick separating the core from the armour
- A Geotextile layer is used to separate the core from the armour layers above mean sea level
- Crest at RL2.4 m with concrete edge used to separate reclamation from revetment armour

3.5 Dredging

Dredging of seabed material is required to:

1. Excavate material for the breakwater toe
2. Remove potentially liquefiable material (sand) from the base of the vertical H- pile walls
3. Dredge an approach channel and berthing area to accommodate the design vessel. Details on required channel size and depths provided in Pacific Marine Management Ltd. (2015) and shown in Appendix D.

Dredging will be a combination of land- based (1, 2 and part of 3) and marine- based dredging (part of 3).

3.5.1 Land- based

Dredging

It is expected that dredging from land will be undertaken using a long- arm excavator operated from above the high water level. Anticipated volumes are as follows:

- Sand:
  - 5,000 - 10,000 m^3 from under breakwater toe
  - 750 m^3 in berthing pocket
  - 6500 m^3 from under H- pile wall
- Rock:
  - <200 m^3 where breakwater need to be toed into rock

Disposal

Disposal options for the dredged material are proposed as follows:

- Sand:
  - 3,000m^3 to the beach for replenishment,
  - Up to 14,500 m^3 into the reclamation if material is suitable,
  - Much of the material at the breakwater toe can likely be moved and replaced over the toe rather than removed from the system.
- Rock:
  - To be placed into reclamation.

3.5.2 Marine- based

Dredging

It is expected than marine- based dredging would be undertaken using a barge- mounted excavator or backhoe. The proposed dredge area is shown in Appendix D and covers a 4,750 m^2 area of sand and reef to the southeast of the reclamation. The current seabed in this area is of irregular height between RL- 4 and - 6 and will be lowered to a uniform RL - 6m. Anticipated volumes are as follows:

- Sand: 750 m^3 (though likely mixed with some rock)
- Rock: 2,250 m^3

Disposal

It is expected that marine- based dredging material will be disposed offshore or within the reclamation.

- Sand:
  - Clean sand may be deposited in the nearshore (in less than 4m depth) to the east of the dredge area and so will remain within the active beach system
- Rock: Options for disposal of rock material include:
  - Disposal ~400 m offshore in approximately 10 m depth. Using a 100x100 m disposal area, rock would average 0.25 m high or could be concentrated in more defined 'reef'
  - Disposal; in 30- 50 m depth 2.5 km to 7.5 km away
  - Land based. Could be potentially used in reclamation.

3.6 Beach replenishment

The beach fronting Waitangi township currently has very little dry beach in front of near continuous seawalls. While this beach has likely been continually subject to periods of erosion and accretion (as the existence of historical seawalls suggests), comparisons with of historic photographs indicates that the beach has lost significant sediment volume over the past 140 years. This has likely been caused by a combination of ongoing sea level rise and erosion of the controlling headland to the east combined with the presence of backing seawalls behind and adjacent the beach causing wave reflection along the beach.

One solution to ongoing erosion is beach replenishment whereby sediment is placed either along the beach or at the updrift end where it will eventually migrate along the beach. Such replenishment results in a wider beach and can absorb the effects of storm erosion demand without further erosion occurring at the backshore.

It is proposed to replenish the beach at Waitangi with sediment dredged from around the wharf structure as part of construction of the reclamation and breakwater. This dredged sediment has a similar mean diameter (D_{50} = 0.125mm) to the existing beach sand (Figure 3- 3) meaning that the beach profile grades are likely to be similar. The grading of the existing beach sand is more uniform than the dredged replenishment material indicating that the replenishment material is better sorted and that some of the placement material is likely to be more mobile and lost from the beach system. An overfill ratio of 1.7 has been initially assessed meaning that approximately 60% of the material placed on the beach will be retained with the remainder migrating alongshore or offshore over time.

In general, the replenished beach profile will have a 5 m wide berm at RL 1 m (approximately 0.5 m above high tide), sloping down at 1(V):12(H) to the existing foreshore (Figure 3- 4). This elevation should minimise wave overtopping during most tide/wind combinations. While a 1(V):12(H) slope is steeper than the existing profiles (ranging from 1:12 to 1:20), the replenished profile is expected to flatten over time as the imported material achieves a stable angle that will result in a lowering of the replenished beach crest. Longshore transport will also continue to act on this area reducing total volumes over time.

It is expected that 3,000m^3 will be available to be placed on the beach. At an average rate of 25 m^3/linear m the material will extend approximately 120 m along the shoreline. A larger volume will be placed at the western (updrift) end, tapering towards the east and it is expected that the material will gradually migrate towards the east. To retain the sand on Waitangi beach would require a coastal control structure such as a groyne at the eastern end of the beach. This is beyond the scope of the present works but could be considered by the community at a later date, potentially during the management of the Nairn River entrance.

The material is expected to be back tipped from the western end of the beach or from the seawalls below Waitangi Wharf- Owenga Road, before being spread along the beach by hydraulic excavator or similar plant.

Figure 3-3 Comparison of proposed replenishment sediment (left) with existing beach material (right)

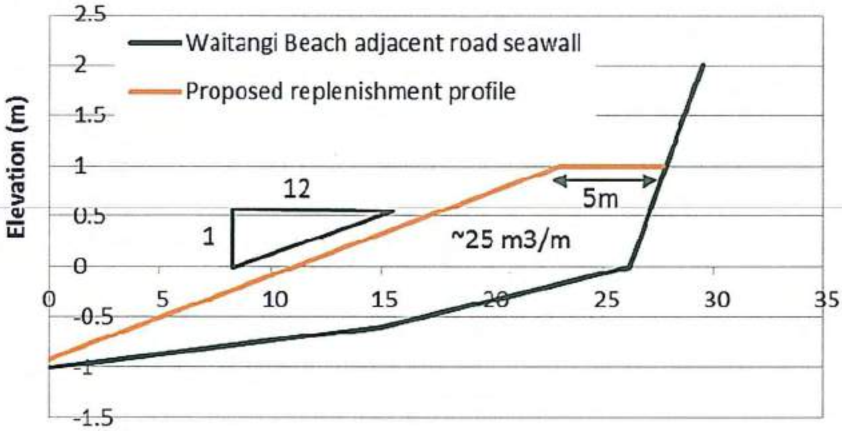


Figure 3-4 Proposed typical replenishment profile

4 ASSESSMENT OF EFFECTS ON COASTAL PROCESSES

4.1 Shoreline location

Construction of a reclamation and breakwater over 9,000m² will shift the existing shoreline position, as defined by MHWS, offshore.

Placement of dredged sand material on the beach will temporarily move current MHWS location offshore by average of 10 m.

4.2 Waves processes

The construction of a breakwater would shift the headland control point further offshore, modifying incoming swell waves. Wave modelling has been undertaken using the refraction- diffraction model CGwave for both the existing situation and with the addition of the reclamation and various breakwater lengths to assess the change in typical and extreme wave climate in the breakwater lee.

Results are presented within Appendix C and summarised for a particular typical wave case for the existing situation and with the inclusion of a reclamation and 60 m long breakwater in Figure 4- 1 and Figure 4- 2. Results show that wave climate in the lee of the structures and reaching the Waitangi Town Beach is substantially reduced.

Figure 4- 3 shows the change in mean wave climate and direction at the 2 m depth contour. Results show that a 90 m breakwater would reduce the wave climate along Waitangi Town Beach by between 20 and 80% while a 40 m breakwater would reduce the climate between 10 and 70%. A slight increase (up to 5%) in wave height is noted outside the shadow region which is typical along refraction edges. This increase would occur between Chainage 700m (for 40 m breakwater) and 1700 m (for 90 m breakwater) which is along the southern end of Waitangi Beach.

While these changes are very small and may not result in noticeable effects, the change in wave climate may modify longshore and cross shore sediment transport processes as the reduced wave climate transports less sand to the east and causes less movement across the surf zone. These are discussed below.

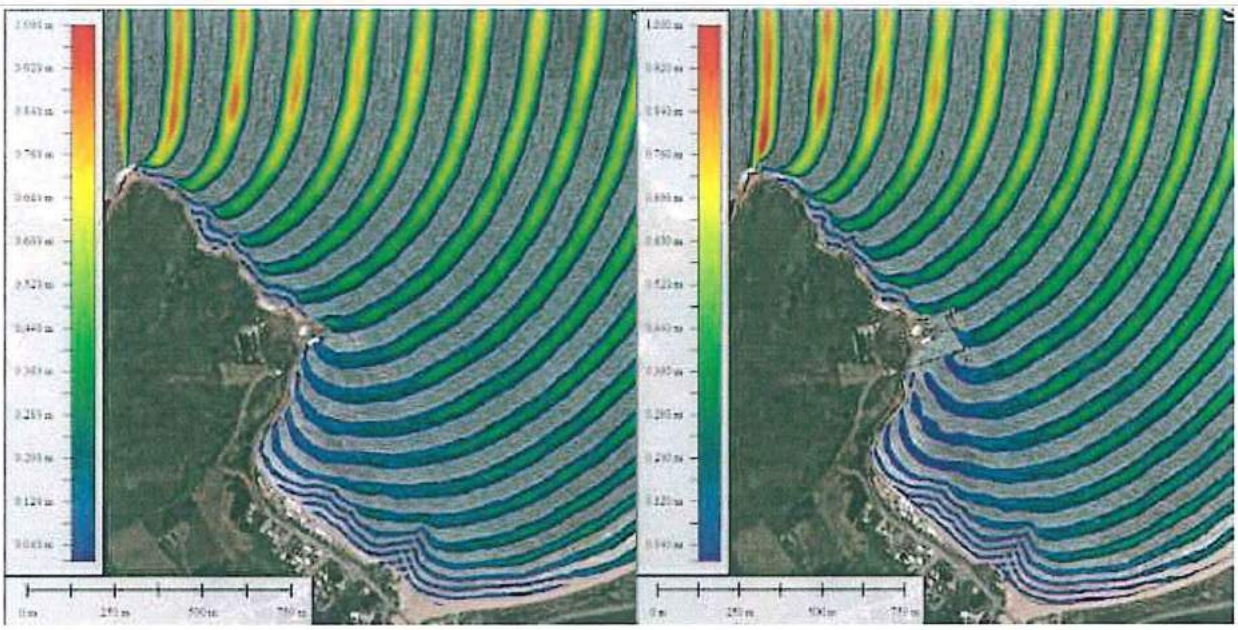


Figure 4-1 Change in wave crest patterns construction of the enlarged reclamation area and breakwater

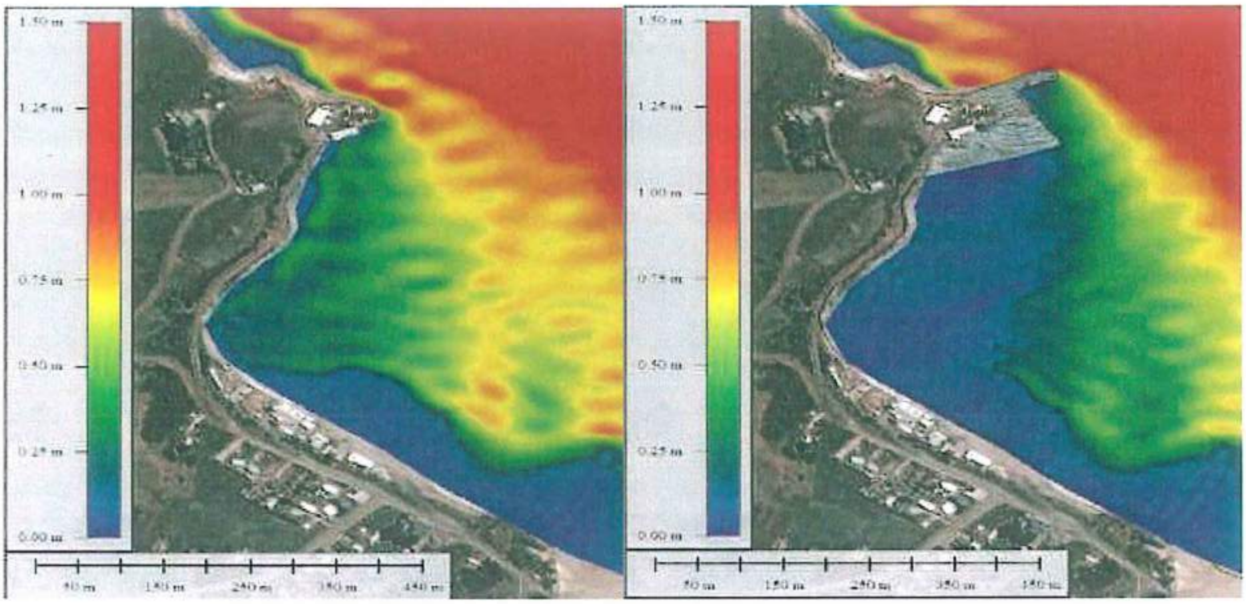


Figure 4-2 Expected effects of the construction on wave processes

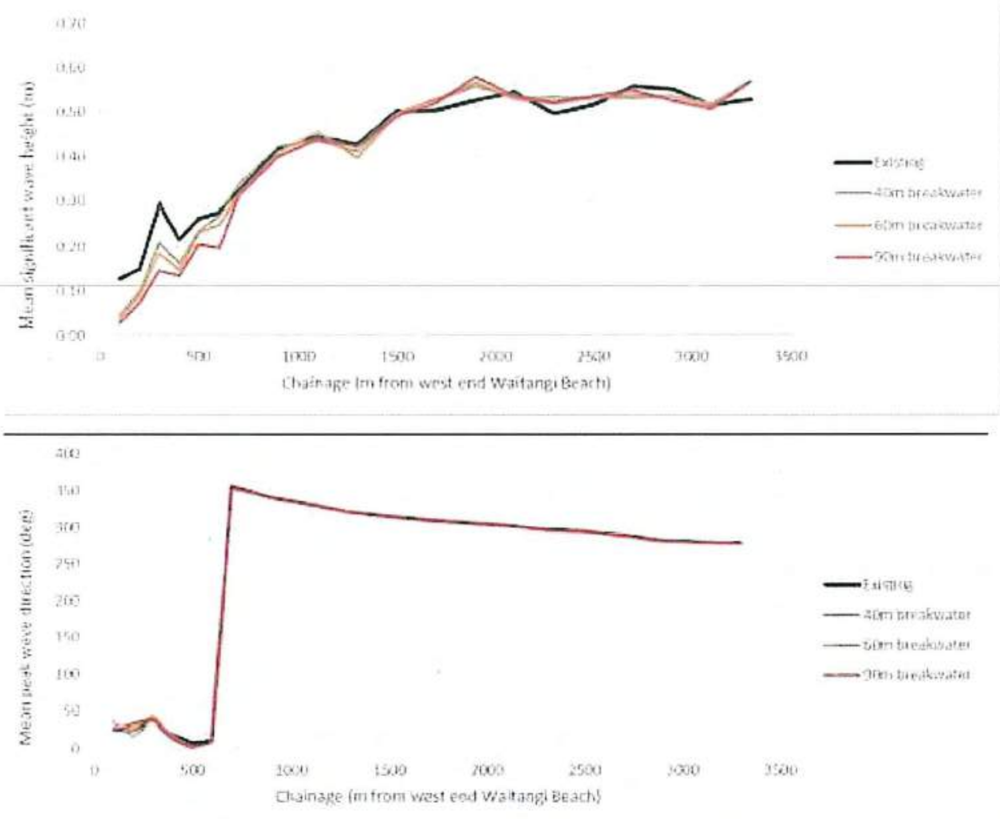


Figure 4-3 Change in annual mean wave height (top), wave direction at the coastline (lower) for a range of potential breakwater lengths.

4.3 Nearshore currents

The effect of the proposed works on nearshore currents are expected to be minimal as tidal currents are not expected to be present within Waitangi Bay due to the low tidal range and open coast nature of the site. The differential reduction in wave climate across Waitangi Town Beach could induce a slight east to west current but is not expected to be sufficient to transport sediment.

4.4 Sea levels

Proposed works are not expected to have an effect on sea levels. Future sea level rise (SLR) has been accounted for in design of the physical works.

Physical works will assist in offsetting future SLR effects on the western corner of Waitangi Beach by manually placing additional sand on the upper beach to help offset expected SLR- induced erosion.

4.5 Sediment processes

4.5.1 Scour

Scour may occur in front of breakwaters due to increased sediment suspension and transport due to wave turbulence. Van Rijn (2006) presents a number of methods to evaluate scour at the toe of a rubble mound structure. Based on the four standard empirical methods, toe scour in sand under 100 year ARI swell (H_s = 2.2 m, T_p = 13.7 s) and wind- wave conditions (H_s = 2.0 m, T_p = 5.5 s) was assessed. Results incorporating a safety factor of 1.3 are shown in Figure 4- 4 and show that under design swell conditions, average predicted scour depths range from 1 to 2.2m with a maximum prediction of 3.2m. Under wind- wave conditions, average predicted scour depths range from 1 to 1.7m. It is not known how long these scour depths take to develop and it is likely that these maximum depths could not be achieved during the storm peak (i.e. before wave height begins to reduce).

These scour depths may reach the underlying rock depending on sand depths at the time and have been allowed for in design with larger volumes of rock used along the breakwater toe. This 'toe protection' rock is sized to limit the potential for toe scour damage to the structure.

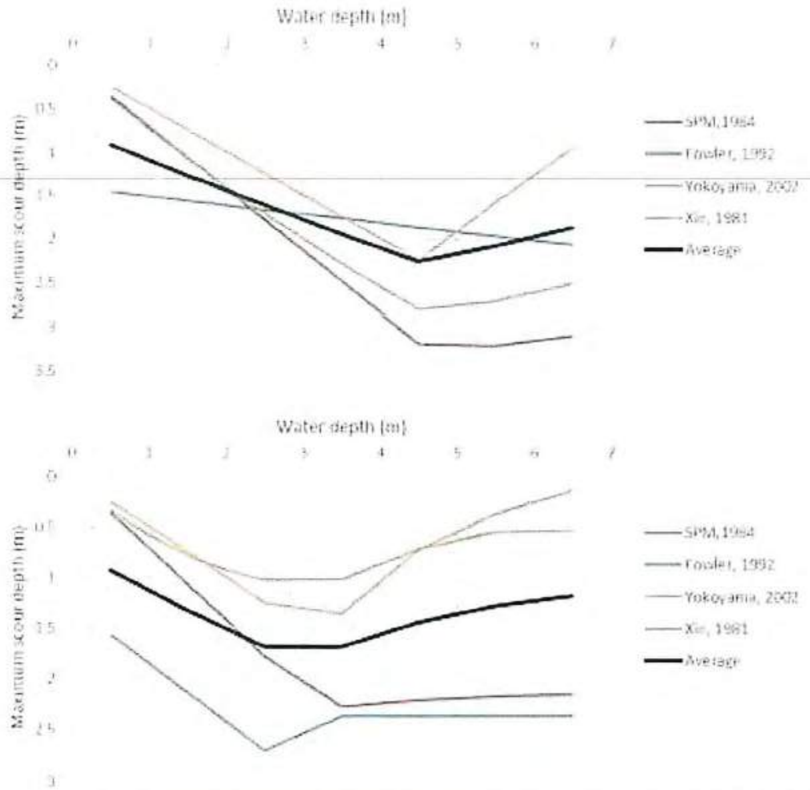


Figure 4-4 Maximum theoretical scour depth for the 100 year ARI swell (top) and wind-wave (bottom) conditions

4.5.2 Longshore processes

Under wave conditions, sediment is transported along the sea bed and in suspension. Sediment transport along the seabed is highest within the inner surf zone where wave velocities at the seabed are highest and reduces with distance offshore. The longshore transport model Unibest CL+ (Version 7.1, Deltares 2011) was used to compute the annual sediment transport *potential* at the proposed breakwater location. The model (example in Figure 4- 5) shows that sediment transport is highest in 0 – 2 m water depth, decreasing to zero transport offshore of 4 m. This model computes the sediment transport *potential* with actual transport dependent on the availability of sediment in the system. Given the rocky nature of the nearshore to the northwest of the breakwater, actual transport is likely to be significantly lower. The key finding, however, is that the potential for longshore transport seaward of the 4m depth contour is effectively zero indicating that sediment is unlikely to be transported around the end of the breakwater that could accumulate in the breakwater lee.

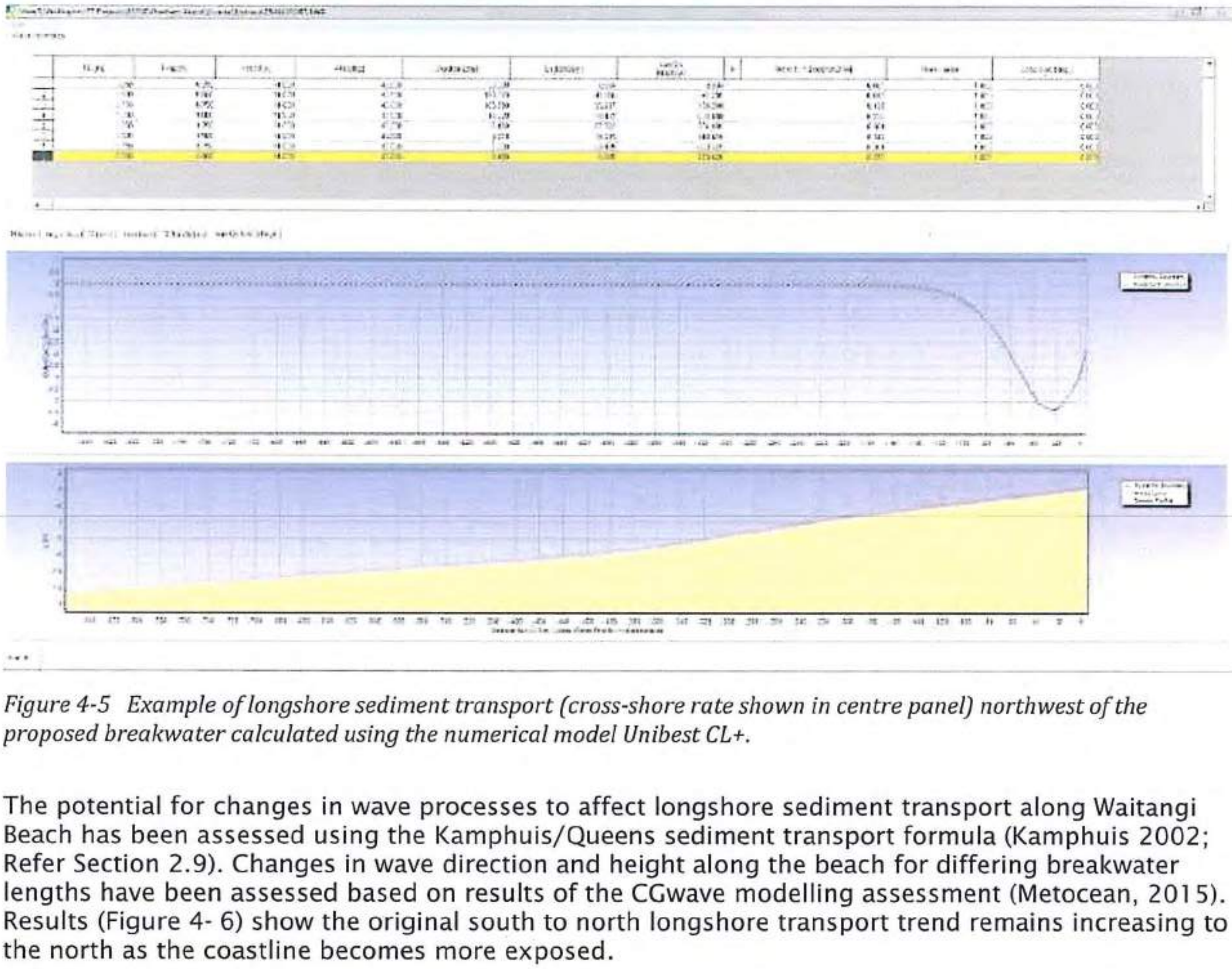


Figure 4-5 Example of longshore sediment transport (cross-shore rate shown in centre panel) northwest of the proposed breakwater calculated using the numerical model Unibest CL+.

The potential for changes in wave processes to affect longshore sediment transport along Waitangi Beach has been assessed using the Kamphuis/Queens sediment transport formula (Kamphuis 2002; Refer Section 2.9). Changes in wave direction and height along the beach for differing breakwater lengths have been assessed based on results of the CGwave modelling assessment (Metocean, 2015). Results (Figure 4- 6) show the original south to north longshore transport trend remains increasing to the north as the coastline becomes more exposed.

Results show, similar to wave climate, a slight reduction in sediment transport capacity west of Chainage 1500m (west of approximately 900m east of the Nairn River mouth) with longer breakwater lengths resulting in more reduction in sediment transport. This trend reverses further east with greater transport potential. These results indicate that less longshore transport may occur in front of Waitangi town, potentially increasing beach stability or maintaining the replenishment material for longer. Sediment potential in front of the Nairn River is similarly reduced. This may or may not result in decreased sediment accumulation at the mouth but any changes will likely be negligible. Model results show sediment transport rates could increase slightly (10- 20%) between 1 to 2 km northeast of the Nairn River, although these small changes are likely well within the model's margin of error (refer Figure 2- 22). Given this area has experienced a background erosion rate of 0.1 to 0.3m/year since 1969, this increased transport could potentially increase erosion pressure here. As described previously, while the potential sediment transport capacity can be calculated, the actual transport is likely to be significantly lower owing to the lack of sediment apparent in the system (Section 2.9).

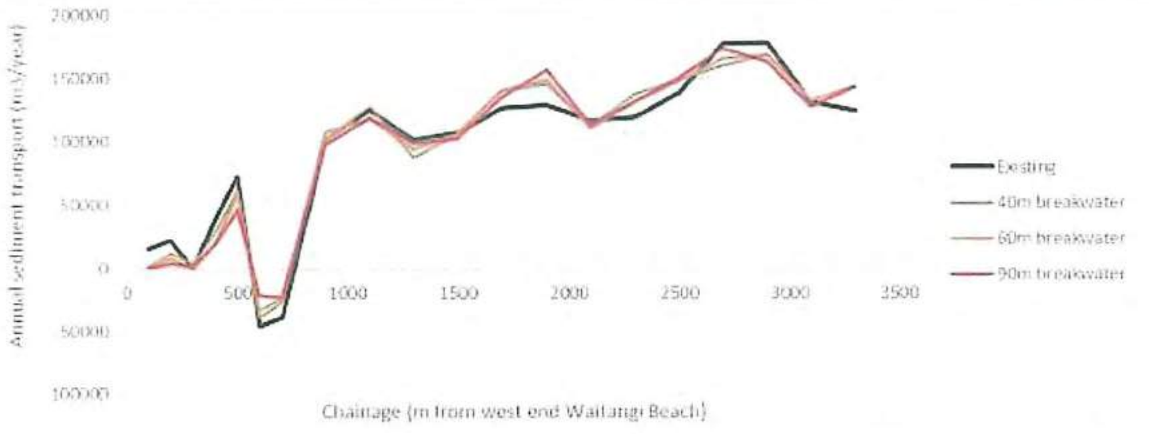


Figure 4-6 Change in potential longshore sediment transport for a range of potential breakwater lengths

4.5.3 Cross-shore processes

To minimise initial and potential ongoing dredging requirements, the wharf has been rotated anti-clockwise to remove the requirement for an approach channel within the bay. Some initial dredging is still required for vessel manoeuvring immediately landward of the wharf. This initial dredging includes removing the upper part (up to 1 m thick) of an irregular reef (estimated at 2,200 m^3 rock) and some sandy material (estimated at 750m^3).

The rocky nature of the existing reef indicates a lack of sediment in this area. As the dredging is effectively levelling the reef to the elevation of the adjacent seabed, we do not anticipate additional sedimentation on the reef top requiring ongoing dredging.

Sedimentation of the sandy dredge area could occur by cross shore transport from inshore. This later sediment transport mechanism may have additional adverse effects by removing sediment from the upper part of the cross-shore profile, potentially inducing beach erosion. The cross shore sediment transport model, SBeach, has been used to assess the potential for cross shore sediment transport during storm events to move sediment across the profile and cause infilling of the dredged areas. A cross-shore profile extending from Waitangi town beach offshore through the dredge area has been tested. Figure 4-7 shows the location and the proposed dredge profile including the required dredge areas. As is evident, the required dredging is an incision into the profile rather than a deep dredge channel.

The June 1988 storm event was initially tested being the largest on record. Results showed sediment to be removed from the upper beach and deposited on the lower profile, extending to RL-4 m to -5 m but no accumulation in the dredge area. A 100 year return period event based on Metocean analysis was similarly run through the model with results again showing less than 0.1 m sediment deposited in the dredge area (Figure 4-8). We therefore recommend over-dredging by 0.2m to allow for potential offshore sediment transport due to storm events but based on our analysis we do not expect significant levels of ongoing dredging to be required as a result of cross-shore processes.

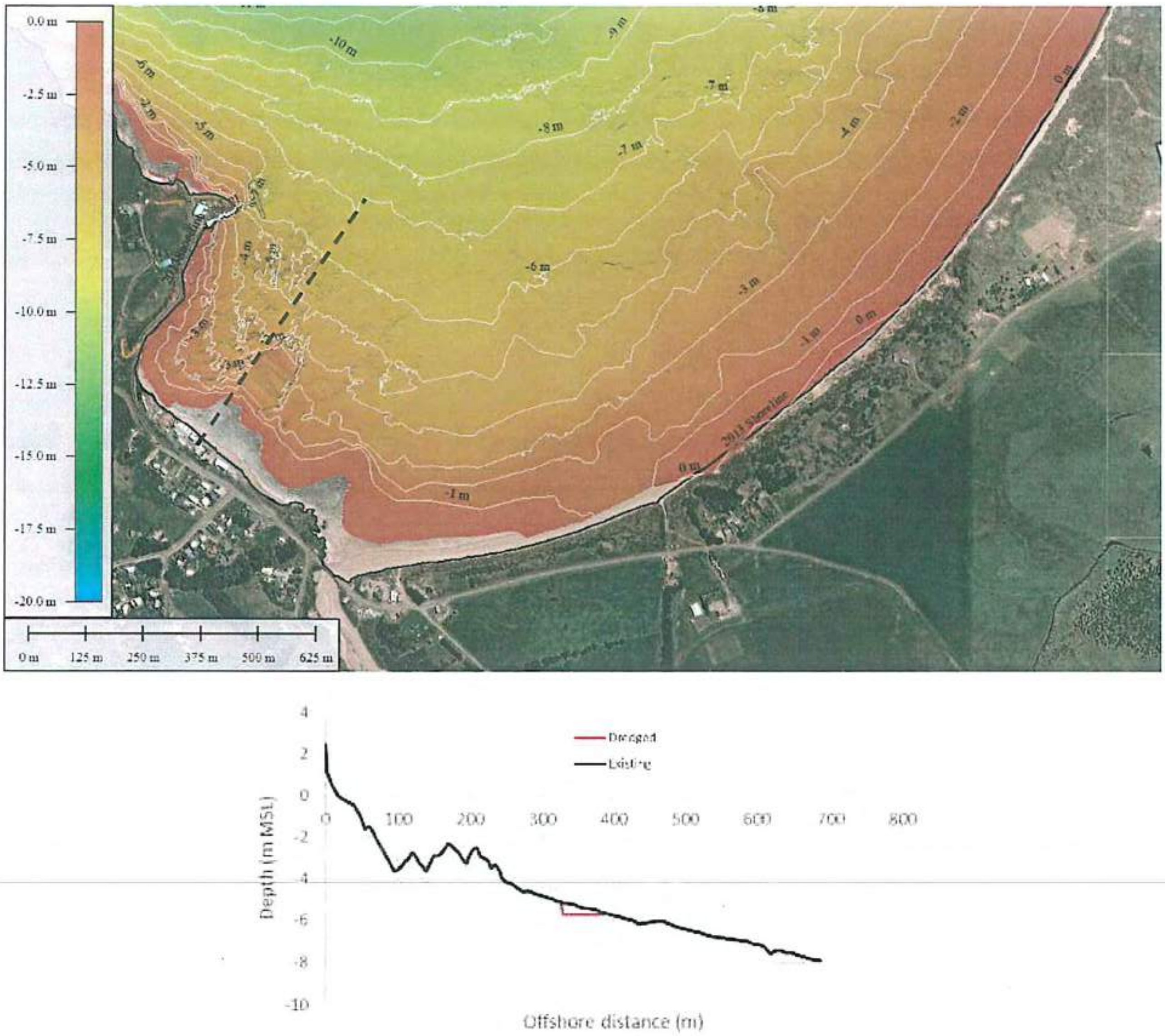


Figure 4-7 Location and tested profile used to test potential cross-shore transport

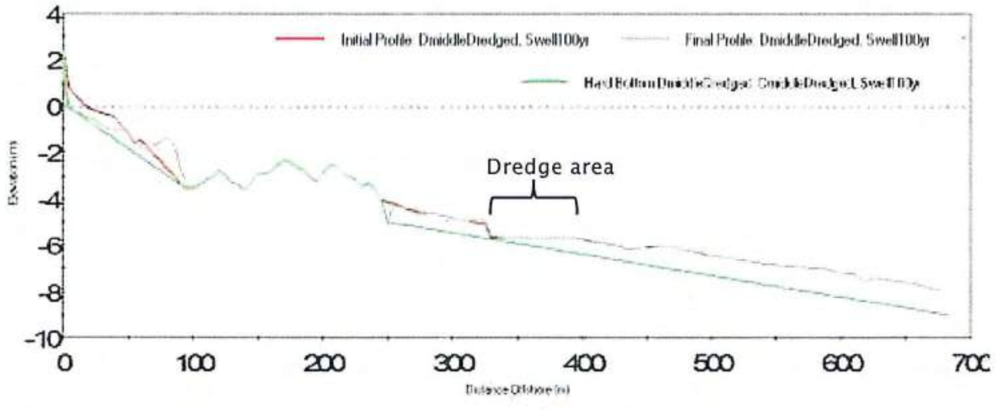


Figure 4-8 Initial and final profiles after a 100 year Return Period storm event. Material is eroded off the upper shoreface and deposited offshore. Minimal accumulation in the dredge areas is evident.

4.5.4 Suspended sediments

In deeper water sediment may still be transported as the orbital velocities beneath non- breaking waves suspend and move sediment. Once the wave climate is reduced, suspended sediment may settle. The computational software TRANSPOR (Van Rijn, 2004) has been used to assess the *potential* suspended sediment transport offshore at the breakwater and within the shadow zone behind the breakwater. Sediment characteristics used in the model (D_{10}, D_{50} and D_{90}) have been obtained from sediment grading curves and wave characteristics from the numerical SWAN and CGwave modelling. Assessment shows that suspended sediment transport under wave processes may range from 0 to 0.143 kg/s.m depending on wave height. Based on the annual wave climate at the breakwater head, the total sediment transported in suspension in this area may range from 4 to 24 m^3/year.m^2. Given a 130 m long potential shadow zone (i.e. the length of the wharf) where this material may drop from suspension, approximately 500 to 3,000 m^3/year may potentially accumulate in the breakwater lee.

However, the existing 65m long reclamation effectively acts as a breakwater inducing a shadow zone in its lee and therefore provides some indication of the *actual* existing sedimentation rates. Anecdotally, since construction of this reclamation in 1979/1980 no dredging has been required adjacent the Fishing Wharf in the lee of the reclamation. This indicates that *actual* suspended sediment loads are low and it may be inferred that sedimentation behind the proposed breakwater is also likely to be low. We recommend that an allowance is made for 500- 1,000 m^3 of sediment accumulating annually in the breakwater lee. This would likely equate to at between 0.05- 0.1 m/year at the outer edge of the breakwater increasing to 0.1- 0.3 m/year adjacent the wharf.

4.6 Coastal stability

The effect of moving the refraction control point seaward on the equilibrium planform of Waitangi Bay has been assessed using the software Mepbay (refer Section 2.11 for details on initial calibration). Assuming a breakwater length of 60 m, the control point is shifted offshore by 150 m. Results show that the equilibrium planform is extended seaward of the current coastal edge (Figure 4- 9).

While Waitangi town beach is not likely affected be this change in equilibrium planform as it is maintained by different controls (i.e. the small eastern headland and offshore reefs), the result indicates that increased erosion pressure on the town beach as a result of the development is unlikely. Some additional accumulation of sediment may, however, occur at the southern end of Waitangi Beach in front of the Nairn Rivermouth (refer section 4.8 for discussion).

Figure 4-9 Analysis of an equilibrium planform for southern Waitangi Bay with reclamation and 60 m breakwater

4.7 Coastal water quality

4.7.1 Construction effects

Based on sediment samples, dredged material is expected to be clean sand or rock with small amount of fine material. Based on the model Transpor (Van Rijn, 2006) for typical sand material with D_{50}= 0.138mm, D_{90} = 0.28mm, D_{10} = 0.1mm and 3% fines (<0.063mm), average fall velocity for the suspended components is 0.01m/s

The fall duration for this material can be calculated for a range of depths (Figure 4- 10). This material may be transported by waves due to the asymmetry in the wave orbital velocities caused by Stokes drift. This has been calculated using Transpor (Van Rijn, 2006) and the distance travelled by suspended sediment particles in falling 4m (typical depth at landward edge of dredge area) is shown for a range of wave heights in Figure 4- 10. This figure shows that distances travelled are generally less than 20m for wave heights below 1m. It can be assumed that dredging will only occur in wave heights of less than 1m and therefore any sediment suspended during the dredging process is likely to have reached the seabed within 20m. Wind- and any tidal induced currents are not considered here but, as described previously, tidal currents are expected to be low and prevalent wind directions are from the SW, directing any suspended sediments towards the NE, away from Waitangi Bay. Any turbidity plume is expected to be confined to a 50 m area around the dredge area.

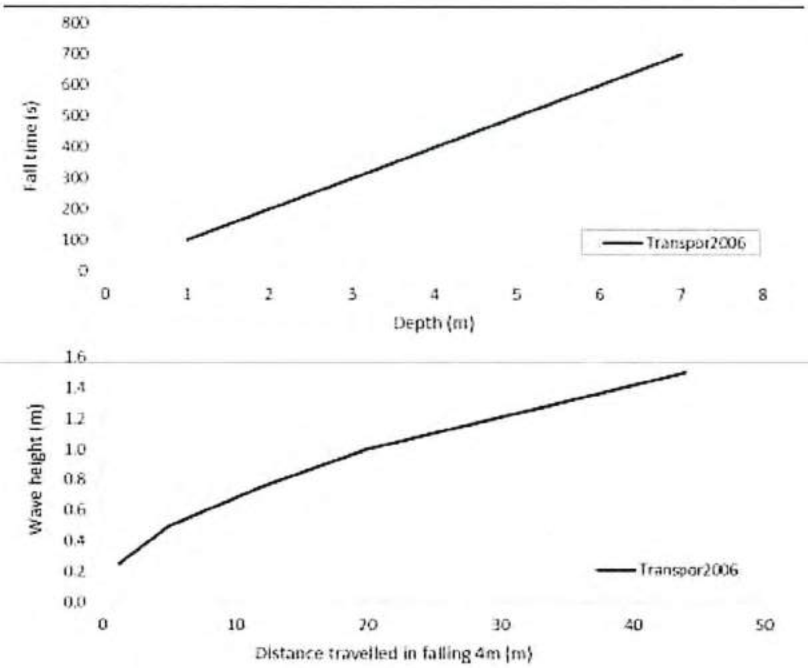


Figure 4-10 Sediment fall duration (s) for range of depths and distance travelled while falling 4m for a range of wave heights

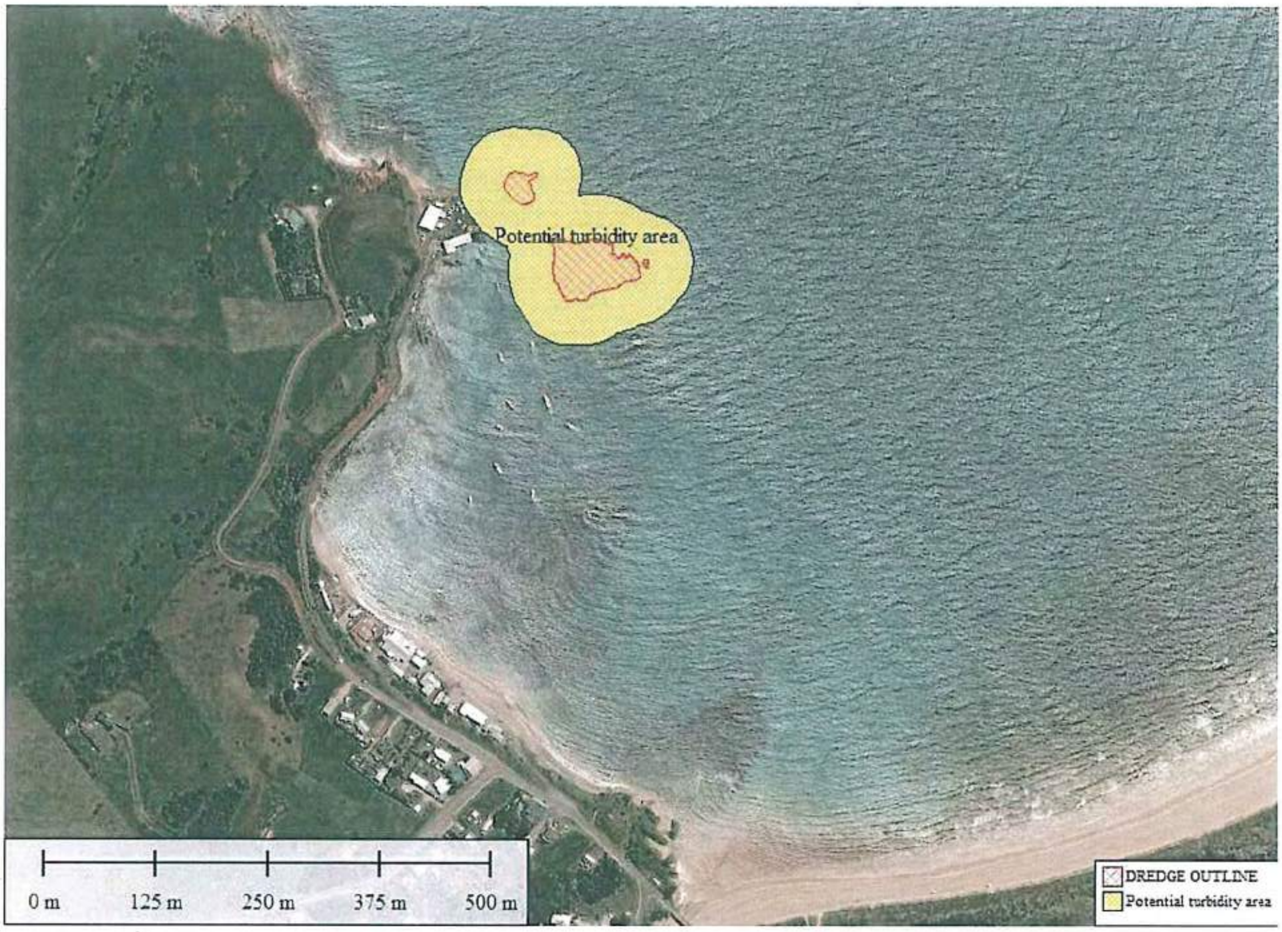


Figure 4-11 Area anticipated to be affected by turbidity during dredging

4.7.2 Long-term

The breakwater extension is likely to result in slightly more enclosure of the beach and nearshore at Waitangi. However, the bay is still largely 'open' with an exposure of 100° from mid beach following construction (c.f. 109° at present). Given the majority of wind is from a westerly quarter which will drive surface water towards the NE and bring in water from the deeper parts of the bay, construction of the proposed works are not expected to affect water exchange or seawater residence time within the Bay.

The water intake for Moana Seafood factory is likewise not expected to be adversely affected.

4.8 River discharge

Wave modelling shows that the wave climate near river mouth may be reduced by up to 25% (Figure 4-3) depending on the breakwater length adopted. Sediment transport potential in front of the Nairn River is likely to be similarly reduced (Figure 4-6). However, as discussed in Section 4.5.2, this may or may not result in increased sediment accumulation at the river mouth. Trends show that sediment transport occurs towards the river from both directions so a reduction in the transport potential may slow accumulation and infilling of the river mouth. Any changes are expected to be minor compared to natural fluctuations.

The periodic, partial blockage of the river mouth is a natural process as described previously and will continue to occur. It is recommended that a river management plan is developed to monitor and periodically open the entrance to minimise the incidences of upstream flooding that currently occur. Dredged material could be placed on Waitangi Town Beach to migrate back or could be placed further north to migrate up Waitangi Beach.

4.9 Effect on existing structures

The foreshore today at Waitangi Town Beach is backed almost continuously by seawalls. At the western end (Appendix A, Photograph 7, 10) rock and rubble has been dumped to prevent erosion at the end of the road seawall. This rubble wall continues towards the east fronting the fuel tanks (Appendix A, Photograph 11). The rubble does not appear to overlie filter layer of rock (or geotextile filter layer) and so fine material from the slope behind is easily lost by hydraulic wave action. The boat pull- up area is fronted by a low, gravel and fill revetment (Appendix A, Photograph 12). Further east a range of vertical concrete walls in front of the Moana Pacific fish processing factory (Appendix A, Photograph 13, 14, 15). These walls are generally in poor condition and have been undermined in some places with fill lost from behind the walls and collapse of concrete pavements. A sloped vertical seawall fronts the Waitangi Hotel (Appendix A, Photograph 16) and appears in reasonable condition, although the toe footing is becoming exposed in places indicating that the beach levels have lowered since initial construction.

A range of existing structures exist within Waitangi Bay including the stepped, vertical concrete seawall below the wharf road, a rubble revetment at the west end of the beach, a concrete boat pull up area fronted by rock and rubble and a range of vertical concrete seawall in poor to average condition.

While the breakwater is likely to provide additional wave sheltering from swell waves, wind- waves from the north to north- east are likely unchanged. There is therefore unlikely to be significant reduction in damage to coastal structures during north to northeasterly storm conditions. However, the sand placed on the beach will provide some protection to the toe of the structures, decreasing the likelihood of the structures being undermined and/or losing material from behind the wall.

Water discharging from the Moana Pacific fish processing factory is likely to cause additional scour to any beach replenishment material placed or that has migrated in front of the factory. The replenishment material is not expected to have an adverse effect on the discharging of water, although the low level pipes may become blocked if flow is not continuous.

5 SUMMARY AND CONCLUSIONS

The Chatham Islands Port Limited, in conjunction with the Department of Internal Affairs are seeking resource consent applications to undertake the upgrade of Waitangi Wharf in the Chatham Islands. The project seeks to improve the reliability and usability of the existing wharf operations and its facilities, and enhance the resilience of the port infrastructure for the island.

Waitangi Bay is located at the south- eastern corner of Petre Bay, and is defined on its western side by the rocky headland of Tikitiki Hill, and on its eastern side by Waitangi Beach which extends north to Red Bluff. Waitangi Beach is an accretional feature formed by accumulation of Pleistocene aged marine sediment in the lee of the southern Chatham volcanic outcrops as sea levels stabilised to their present level and has resulted in the formation of the Te Whanga Lagoon system.

Sediment movement along the coastline is expected to be typically south to north, driven by the dominant south- west wave direction. The beach geometry along southern Waitangi Beach to the Nairn River entrance fits a parabolic bay shape that is typical of headland controlled beaches indicating that the southern part of the bay is close to dynamic equilibrium, although it may fluctuate by up to 20 m. The lower Nairn River is a sediment sink and contains large volumes of marine sediment (beach sands) that have been moved into the river mouth by wave processes and have partially blocked the entrance. Such sediment transport dynamics are typical of high energy coastlines with entrances often becoming completely blocked until large rainfall events cause the waterway to break out with sediment redistributed back onto the beach.

The beach fronting Waitangi township differs from that north of the Nairn River. It is a perched beach, with a layer of sand overlying a rock platform at the toe of a tuff bank. While the beach has been present as far back as settlement, its low volume makes it more susceptible to erosion during storms or to changes in the sediment budget (i.e. the balance of sediment additions and losses) than the open coast beaches. The beach here is out of alignment with the wider bay and we believe is being maintained in its present position/alignment by the offshore reefs and the small rock outcrop at the eastern end. Analysis of historical aerial photographs show that this outcrop, comprised of a weak tuff material, has eroded up to 9 m since 1969. As this headland retreats, the control on the beach is lost and sand can migrate to the east. Early images show a wider beach than presently exists, but also show a ponga breastwork constructed to combat coastal erosion. This indicates that cycles of erosion and accretion have long influenced this beach, although it is likely to have become exacerbated recently by erosion of the adjacent headland to the west, ongoing sea level rise and potentially by wave reflection off the nearby Waitangi Wharf Rd seawall.

Analysis of the historic aerial photographs and satellite images show that the Tuff cliffs to the northwest of the wharf have been eroding at average rates of 0.1 to 0.25 m/year. While this represents up to 1100 m^3 of sediment input annually, the fine material is likely to be quickly lost offshore. The lack of sediment accumulation on the seaward side of the reclamation or in the sheltered lee adjacent to the fishing wharf indicates a lack of sediment in the littoral system. Actual longshore sediment transport rates (i.e. northwest to southeast movement) are therefore likely to be substantially lower than the empirically- derived potential rates.

Works are proposed to improve the reliability and usability of the existing wharf operations and its facilities, and enhance the resilience of the port infrastructure for the island. These works are expected to include reclamation of land for enhanced port operations, construction of rock and armour revetments to protect the land, construction of a breakwater to protect the wharf berth area, dredging of an approach and berthing area and replenishment of the town beach using dredged material.

These physical works will affect the natural environment in the following ways:

- The shoreline as defined by MHWS will be moved seaward
- The construction of a breakwater would shift the headland control point further offshore, modifying incoming swell waves. This is likely to reduce the swell wave climate along the Waitangi Town Beach by between 20 and 80% and slightly increasing the swell wave climate further north along Waitangi Beach (up to 5%). Local wind- waves are unlikely to be affected.
- The effect of the proposed works on nearshore currents are expected to be minimal as currents are not expected to be present within Waitangi Bay due to the low tidal range and open coast nature of the site.
- The changes in wave climate may modify the longshore transport rates along Waitangi Beach resulting in less sediment transport in front of Waitangi Town and around the Nairn River and slightly increased transport rates further north along Waitangi Beach (1- 2 km north of the River). Given this area has experienced a background erosion rate of 0.1 to 0.3m/year since 1969, this increased transport could potentially increase erosion pressure here, however, given the small rates of change, actual effects may not be noticeable.
- Modelling of cross- shore sediment transport indicates than negligible sediment is exchanged between the beach and area where dredging is proposed meaning that any dredging is not likely to have adverse effects on the beach
- Sediment transport rates in front of the Nairn River mouth are likely to be reduced. While this may or may not result in increased sediment accumulation at the river mouth, changes are likely to be minor compared with natural fluctuations. The natural periodic blockage of the mouth is likely to continue and could be managed through periodic manual opening.
- Beach replenishment is proposed for Waitangi Town Beach using sand excavated from the seabed during the construction process. This replenishment will provide additional amenity for beach users, will provide increased protection to the backshore and existing structures from wave processes and will offset ongoing erosion due to sediment deficits and ongoing sea level rise.
- Some sediment, suspended by wave action, may settle in the sheltered lee of the breakwater. While this could theoretically amount to 500 to 3,000m^3 annually, the lack of sediment accumulation adjacent to the Fishing Wharf since the construction of the existing 65m long reclamation in 1979/1980 indicates that the actual suspended sediment loads are low and it may be inferred that future sedimentation behind the proposed breakwater is also likely to be low.
- Dredge material is expected to be primarily clean sand and rock and will fall out of suspension quickly (within 10 minutes in typical 4- 6m water depth). Turbidity plumes driven by wave processes are therefore likely to be limited to within 50 m of the dredge area. Tidal currents are expected to be low and prevalent wind directions are from the SW, directing any suspended sediments towards the NE, away from Waitangi Bay.

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