ATTACHMENT F

ENGINEERING DESIGN STATEMENT PREPARED BY TONKIN AND TAYLOR

PLANNERS PLUS LIMITED

Land Development, Resource Consent and Planning Specialists.

Tonkin+Taylor

Job No: 851872
7 September 2016

Paul Dimock and Jim Nolan
C/- Paul Dimock
PO Box 9
Whitianga 3542

Attention: Paul Dimock and Jim Nolan

Dear Paul and Jim

Homeowners Wall - Coastal Erosion Protection

In our January 2016 report, Tonkin & Taylor Ltd (T+T) presented a seawall concept design involving reconstructing the existing seawall (with the possibility of selecting a new alignment). Further to that report T+T have been requested by the Homeowners, to consider an option which doesn't require removal of the existing seawall. We have now evaluated this, and this report presents a concept which involves leaving the majority of the existing seawall structure in place. This is achievable by strengthening the seaward face of the seawall with additional larger rock armour, sized to withstand a 100 year return period storm (1% AEP) allowing for storm surge and predicted sea level rise over the next 40 years, (note that this does not mean that the design waves will occur only once in 100 years, these conditions or worse could occur at any time and more frequently than once in the life of the seawall).

1 Site meeting and discussions

Following the presentation of our January report (including the option to construct a new purpose designed seawall), a meeting took place on the 4th of March to review and discuss other potential options which might have less impact and disturbance on the existing seawall and property frontages. The Homeowners representatives provided additional information on the construction of the existing seawall including a design sketch, and photos during the construction. This has clarified some of the concerns with the existing construction, and allowed options to be considered where much of the existing seawall could potentially remain in place.

While there is a strong desire from the Homeowners to keep the majority of the existing construction, albeit with improvement to extend its expected life, there is also a concern, by the Homeowners, about erosion at the end of the seawall, and a desire to protect the seawall end from being out-flanked by erosion of adjacent sand-dune to the south.

Should the existing seawall remain in its current location, the likely impact of the revised concept on the beach would be a widening of the seawall footprint and reduction of beach width by approximately 2m.

We understand the location of the seawall is being discussed with TCDC, who own the recreational reserve on which some of the existing seawall has been built. We note that while the Homeowners desire a solution involving strengthening of the existing seawall, rather than rebuilding a new seawall, the final design may be influenced by consultation with the Consent Authorities and any

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resulting Resource Consent conditions and the final option may differ from either concepts presented to date.

2 Existing Seawall

2.1 Site location

The site of the proposed coastal erosion protection structure is located at the northern end of Buffalo Beach. The site is bounded by Buffalo Beach Road/State Highway 25 to the west and the coastal marine area (CMA) to the east. Residential housing is located immediately landward of the site. Figure 1-1 provided below identifies the location of the proposed coastal erosion protection structure.

We understand that the proposed coastal erosion protection structure is around 360m in length.

Figure 1-1: Site location plan

2.2 Site description

The site is fronted by an existing rock riprap structure. The structure, along its length, is shown[1] to be predominately located within a Recreational Reserve (survey plan included in Appendix A), or straddling the seaward property boundary of the neighbouring residential area, through the central section. The structure extends into the neighbouring Macrocarpa Reserve, to the north, connecting to an existing structure administered by the Thames-Coromandel District Council.

Mean High Water Springs is shown on the survey plan to be seaward of the existing coastal protection structure, at the date of the survey.

[1] RMS Surveyors Ltd, July 2014. Topographical Survey of Existing Revetment Structure, Buffalo Beach Road, Whitianga.

3 Site inspection

Inspection of the existing coastal erosion protection structures initially took place in August 2015, and subsequently following our March site meeting, and again more recently in July 2016. The purpose of these site inspections was to assess the existing rock size and estimate the quantity of rock that may be suitable for reuse, and to consider alternatives incorporating the majority of the existing seawall.

Selected photos of the existing structure are included within Appendix B, with the following observations noted:

- **Structure condition**: the existing structure includes both imported rock and 'Massblocs'. The Massblocs are around 1.0 x 1.0 x 0.9m in size and made of concrete. The rock along the beach, fronting the Massblocs, varies in size at diferent locations along the wall, with the majority of rock smaller than desirable. There is no apparent displacement of rock into the intertidal zone which indicates a general stability of the rock. Some cleavage plans, cracks and fissures were evident in the larger rock located fronting the Massblocs. In areas along the length of the structure smaller rock has been placed above the Massblocs, likely to help protect the backing land from wave run-up and overtopping flows.
- Note that no inspection of the size or quantity of rock beneath the formed beach was undertaken. It is noted that during erosion periods this rock may be exposed.
- **Access Structures**: a number of access structures exist along the length of the erosion protection works. These structures have been made from either concrete, timber or grouted rock.

In summary the existing coastal protection structure appears to been providing a reasonable level of protection to the backing land when compared to adjacent areas that do not have a protection structure. As a result the beach width fronting the structure is reduced compared to adjacent land area to the south where the beach has migrated inland (refer Figure 1).

However, due to the limited rock size, particularly on the upper slope, and given the predicted sea level rise and probable ongoing erosion and retreat of the shoreline to the south of the seawall, the existing seawall construction is unlikely to provide acceptable protection for a design storm of 2% AEP, unless improvements are implemented.

3.1 Existing seawall construction

As discussed above, information on the existing seawall construction was provided to T+T at our March site meeting by the Homeowners representatives. Details of the existing seawall are attached in Appendix C, and include B. F. Bolt and Associates drawings of the Massbloc wall, together with a series of 12 photos taken during the wall construction. Key details of the construction include the following:

- The Massbloc wall is a minimum 2 units high, however in the central and northern section where the beach was lowest, this height was increased to 2.5 units high. This is shown in the photos and the half units were observed in our recent site inspection. Based on this evidence we believe the Massbloc wall is 2.5m high in the central and northern sections, and 2m high for the southern section.
- The seawall survey (refer Appendix A), surveyed the top of the Massblocs generally at an elevation of +2.5m AVD-46 (Auckland Vertical Datum 1946). This implies that the toe of the Massbloc units are at RL 0.0m AVD-46 for the central and northern sections of the wall, and at +0.5m AVD-46 for the southern section of the wall.

- The design drawings and the construction photos both show a geotextile behind the wall and extending up above the height of the Massblocs to close to the elevation of the ground above the Massblocs.
- A small rock toe appears to have been constructed in front of the lower Massbloc layer, at the time of construction. This has subsequently been enhanced with additional rock, which for the majority, but not all properties, this rock now has a crest level with the top of the Massblocs. This now provides a rock armour face to the Massblocs. We understand, the additional rock has been placed following occasions when the rock toe was close to being undermined and settled. At the time of our inspections, only the top metre or so was exposed, and the remainder of the toe rock was covered by sand. We do not know the elevation of the base of the toe.
- The rock size in the toe armour varies along the wall, with sizes in the central and northern area typically 400mm to 1000mm diameter rock, while the rock in the southern section was larger and typically ranged between 500mm and 1200mm diameter. The slope of the face of the toe rock was measured at three locations and is estimated to be approximately 1V:2H.
- Above the Massbloc level, the top of the seawall consists of an armoured berm to an elevation varying between RL3.1m and 3.6m AVD-46, with most surveyed levels at RL3.3m or above. The rock size in the central and northern sections of the crest section of the seawall appear to be approximately 300mm to 400mm diameter, whereas in the southern section of the seawall the size increased to be typically 400mm to 700mm diameter rock. We were informed that, on many properties, this upper rock protection extends well back into the crest berm fronting the properties.

4 Concept

A description of coastal engineering design parameters for the seawall is included in Appendix D, and effectively relates to both the concept presented in our January report, and the alternative concept in this report.

Typical sketch cross-sections of the option to strengthen the existing seawall are included in Appendix E. The proposed seawall upgrade works involve placing additional larger armour rock over the outer face of the existing seawall. This larger armour provides protection against larger waves anticipated to occur during extreme storm events and allowing for future sea level rise, and also has the benefit of reducing wave overtopping during extreme storm events.

Two options have been presented, with the difference between the two options being that Option A simply encases the existing seawall in larger rock, whereas Option B includes installing additional geotextile layers beneath the existing rock armour to provide greater protection against sand being eroded through the existing rock armour, during extreme storms when the beach levels are low.

We understand that the existing toe armour has settled during storms in the past. The placement of a geotextile beneath the existing rock, will assist in reducing any future settlement, however we do not have any information on the level of the underside of the existing toe rock, so do not know if any future settlement risk still exists. If Option A is selected by the Homeowners, there may be a need to top-up the Armour rock crest level in the future with additional rock, if settlement occurs. This could be done as a maintenance activity in the future if and when required.

Other key aspects of the concept designs are as follows:

4.1 Crest height

The general elevation of the rock crest has been set at RL 3.5m, which allows for crest rock of typical 1.0m diameter to be placed on top of the Massbloc wall units. As discussed above, typical existing rock berm elevations at individual properties vary between 3.1m and 3.6m RL, with most property levels surveyed as 3.3m RL or higher. The crest level of the new rock will therefore be approximately similar or slightly above existing property levels, depending on the individual properties. Note that as rocks are irregular, individual rocks will protrude above this level.

4.2 Overtopping

While the new seawall rock crest elevation will be 3.5m RL, storm waves during extreme storm events will surge through the large crest rocks reaching the land behind the rock. The level to which each individual property is protected against wave overtopping and inundation is dependent on the level of the existing protected ground immediately behind the seawall crest. Based on the existing levels shown in the survey, overtopping and inundation of the backshore is likely to occur during significant storm events (as potentially occurs currently), with the risks greater for the lower lying properties. This overtopping is likely to result in damage to the land and assets behind the seawall unless the ground surface is armoured and protected for a distance back behind the seawall. The risk of damage will increase with future sea level rise.

We have completed initial overtopping assessments using procedures based on the EurOtop Manual ("Wave Overtopping of Sea Defences and Related Structures: Assessment Manual"). For this assessment we have assumed beach levels in front of the seawall are at historic low levels of -0.5m AVD-46, and have considered a 1%AEP storm and allowed for predicted sea level rise of 0.4 m over the next 40-50years.

The EurOtop Manual gives some guidance of the erodibility from wave overtopping of grass covered surfaces, but only for grass cover over clay substrate. We anticipate that for most Homeowners the ground surface behind the existing seawall or steps is likely to be grass cover over sandy soil, and in our experience this is likely to be more erodible than the guidance in the EurOtop Manual indicates. Guidance is also for in the US Army Corp of Engineers Coastal Engineering Manual EM 1110-2-1100 (Part V1), for "grass covered sea dikes".

The Coastal Engineering Manual indicates that damage may start for average overtopping flows of between 1 to 10 litres/second/metre of seawall, and damage is expected for overtopping flows above 10 litres/second/metre of seawall.

We have assessed a number of existing property ground levels (between levels of RL 3.1m to 3.5m, as shown on the survey Drawing), and estimated overtopping flows, as shown in the table below:

Ground Level behind seawall (m AVD46)	**Estimated overtopping flows (l/s/m)**	
	Current sea level, 1% AEP storm	**1% AEP storm, and 0.4m Sea Level Rise**
RL = 3.1	5.2	16.0
RL = 3.3	3.0	9.2
RL = 3.5	1.7	5.3

From the above, some damage can be expected to grassed surfaces behind the seawall for significant storms coinciding with spring tides and storm surge with current sea levels. For the lower property backshore elevation of 3.1m RL, the risk of damage increases with predicted sea level rise. There is also a high risk of wave overtopping and flooding of the backshore area during significant storms, and consideration should be given to raising the backshore berm levels to reduce this risk.

While we have been informed that for many properties the existing backshore berm includes a lot of rock, and therefore may be erosion resistant, we have not carried out a detailed assessment of property erosion risk at this stage, but recommend that this takes place prior to final design of the additional protection works.

4.3 Existing concrete/rock stairs:

Most of the existing stairs are concreted rock construction, and these types of stairs allow more wave run-up and overtopping at the stair location than occurs at the rock seawall location.

We have assessed a number of existing property ground levels (between levels of RL 3.1m to 3.5m, as shown on the survey Drawing), and estimated overtopping flows for concreted stairs in place of a seawall, as shown in the table below:

Ground Level behind seawall (m AVD46)	Estimated overtopping flows (l/s/m)	
	Current sea level, 1% AEP storm	1% AEP storm, and 0.4m Sea Level Rise
RL = 3.1	16.2	29.9
RL = 3.3	11.9	22.1
RL = 3.5	8.8	16.2

Our calculations show that in severe storms with high storm surge coinciding with high tides, we currently expect all backshore ground levels to be overtopped, and damage to grassed areas to occur for ground elevations of 3.3m RL and below. If sea level rise is taken into account we would expect damage to occur for all ground elevations.

There currently exists a risk of crest berm overtopping and land inundation behind the berm from wave action at the stair locations. This situation will get worse with predicted sea level rise in the future increasing the frequency, and the wave flow volume of potential concrete stair overtopping events. Our suggestions for stairs are as follows, in order of preference:

1. Remove all private stairs and construct a continuous seawall to provide the best long term property protection. Property owners use public beach access locations, and these Council owned beach access locations are upgraded to resist surface erosion from wave overtopping.
2. Remove all private stairs and construct a continuous seawall to provide the best long term property protection. Council owned beach access locations are upgraded to resist surface erosion from wave overtopping. A limited number of shared private timber stairs could be constructed over the upgraded seawall, with the timber stairs being similar to the Council owned timber stairs near the end of Halligan Rd at the southern end of Buffalo Beach. Note that timber stairs can be expensive, and will require maintenance over their life, but will result in less overtopping than concrete or rock stairs. Shared private access options will probably require agreement between property owners regarding access, cost sharing and maintenance of the structures.
3. Remove all stairs and construct a continuous seawall to provide the best long term property protection. Council owned beach access locations are upgraded to resist surface erosion from wave overtopping. Adjacent properties share a timber set of stairs constructed to straddle the shared property boundary so that each property has direct access to the beach from a corner of the property. The timber stairs being similar to the Council owned timber stairs near the end of Halligan Rd at the southern end of Buffalo Beach. Note that timber stairs can be expensive, and will require maintenance over their life, but will result in less overtopping than concrete or rock stairs.

4 Some form of the existing stairs are retained, with design improvements made on a property by property basis to suit the owners requirements, and the owners accept the inundation and erosion risk now and in the future, with all future maintenance and property repair costs being the responsibility of the property owner. Note we do not recommend this option, as it provides a lower standard of protection, and is likely to result in potential flooding of the front of sections, and possible erosion damage to the ground surface from wave action, unless the ground is additionally protected against erosion.

4.4 Council access ways:

Council own two walkways with beach access. The walkway between Lots 4 and 5 (near the end of Kawakawa Road) includes a set of timber stairs built immediately in front of the Massbloc wall, and providing access from the top of the Massbloc to the beach. There appeared to be less toe rock in front of the wall at the stair location than elsewhere, and the walkway above the Massbloc was grassed and will be susceptible to higher erosion unless additional protection against erosion is provided.

The walkway between lots 5 and 6 (toward the southern end of the site, includes a set of grouted rock stairs below the Massbloc wall, and is grassed above the wall. As discussed above concreted rock stairs allow greater wave overtopping flows and result in greater land erosion risk above the wall than a rock seawall allows.

Our suggestion for these two Council access ways, is the same as for the private stairs, and involves extending the rock seawall through to absorb wave energy, and reduce wave overtopping and erosion risks during extreme storm event. Each access way is likely to require specific design in conjunction with Council.

4.5 Seawall end erosion

Approximately 18m north of the southern end of the existing seawall, the planform of the seawall changes alignment and the last 18m section of wall is oriented so that any shore normal waves striking the wall, will be reflected to the south. This may possibly be contributing to long-shore transport of sand away from in front of this section of wall, with a slight increase in beach erosion likely as a result.

The RMS survey (2014), shows the beach to the south of the seawall has eroded back to the point where the southern end of the seawall is exposed and is outflanked, with a risk of erosion behind the seawall. At the time of our July 2016 inspection, Council appeared to have recently completed beach dune regrading and planting in this area, and seawall outflanking was not evident to the extent indicated by the RMS survey.

Assuming the end of the seawall was originally constructed with the end buried slightly behind the adjacent beach dune, then it appears that the beach may be in a slow state of erosion in the area south of the seawall, although we haven't completed a detailed study of beach changes as part of this project.

In order to improve the long term stability of the end of the seawall, we suggest the southern 18m section is removed and reconstructed curving back toward the dune, and is buried well into the beach backshore dune area as shown on Sketch SK04 (refer Appendix E). We also recommend Council continue with beach dune management in this area in an attempt to reduce the risk of the new end of the seawall being outflanked in the future.

5 Applicability

This report has been prepared for the exclusive use of our client Paul Dimock and Jim Nolan, 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.

Our reports and other deliverables will be provided on the basis that T+T accepts no liability or responsibility for, or in respect of, any use or reliance upon any of them by any person other than Paul Dimock and Jim Nolan as our client. While T+T understands that Paul Dimock and Jim Nolan may be engaging T&T on behalf of the individual property owners they represent, those individual property owners shall have no direct right of action against T+T, and any claim or demand by an individual property owner must be brought by Paul Dimock and Jim Nolan as our client in the client's name and shall otherwise be subject to the terms of engagement between T+T and Paul Dimock and Jim Nolan.

Tonkin & Taylor Ltd

Environmental and Engineering Consultants

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Keith Dickson

Project Director

GGN

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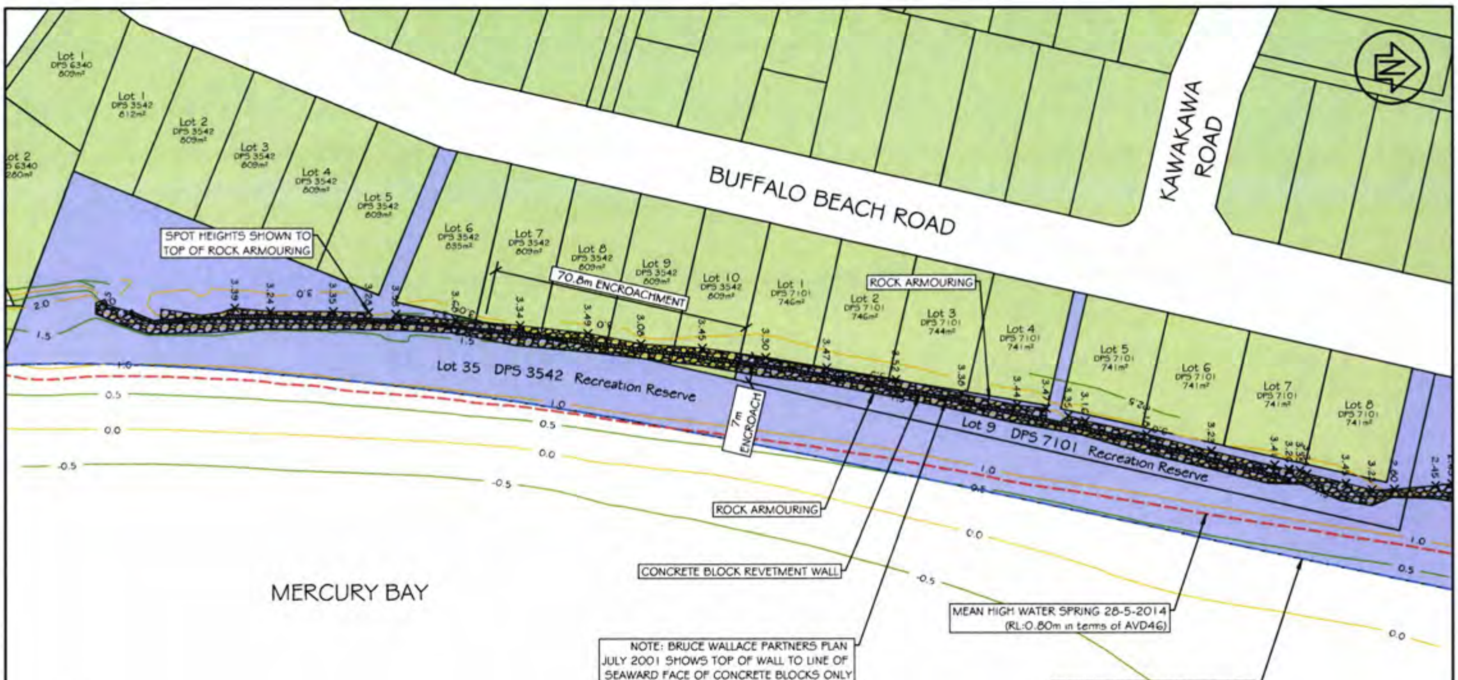
Appendix A: Survey plan



Notes:

1) Levels are in terms of Auckland Vertical Datum 1946
2) Origin of levels OBLK I SO 51500 RL: 1.99m
3) Source: TCDC Benchmark Booklet 2008
4) Contour interval: 0.5m
5) Mean High Water Springs = RL: 0.80m (sourced from Whitanga Waterways level datum chart ref: 3808)

Date: 24 June 2014 | Scale: 1:1000 @ A3

No. | Date | Revision Details | By | Ver. | App.

TOPOGRAPHICAL SURVEY OF EXISTING REVETMENT STRUCTURE BUFFALO BEACH ROAD, WHITIANGA

Drawn CRS | Designed | Verified | Approved

RMS SURVEYORS

Land & Engineering Surveyors • Property Development Consultants • Resource Management Consultants

Project No. F7116

Drawing No. Rev. T1

Appendix B: Selected site photos

Photo 1: existing structure in Macrocarpa Reserve – administered by Thames-Coromandel District Council.

Photo 2: Typical profile for northern end of existing coastal protection structure

Photo 3: Existing public access structure

Photo 4: Typical section through central area.

Photo 5: Typical section at southern end of the existing structure

Photo 6: Southern extent of existing structure – showing landward retreat for unprotected area

Appendix C: Existing Seawall

/ — BRETT MASON
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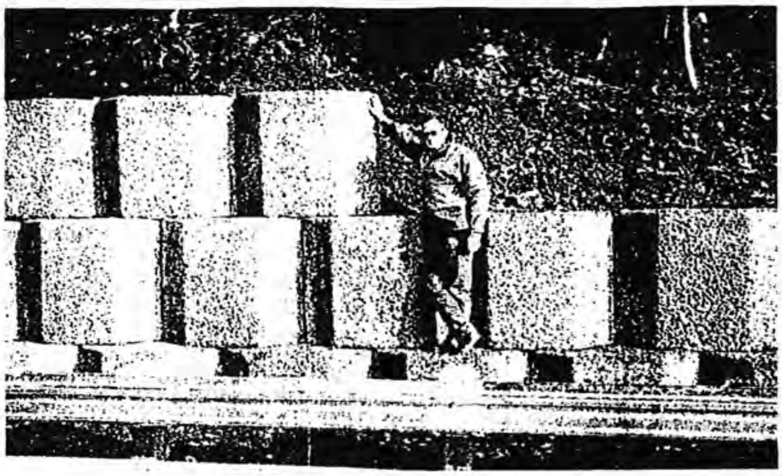

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- *suited to coastal protection works, dissipating wave energy,*
- *suitable as a controlled fill for mass fill operations on land or totally immersed*
- can be cast with inserts to accommodate safety/handrails, posts, Reidbar systems etc.

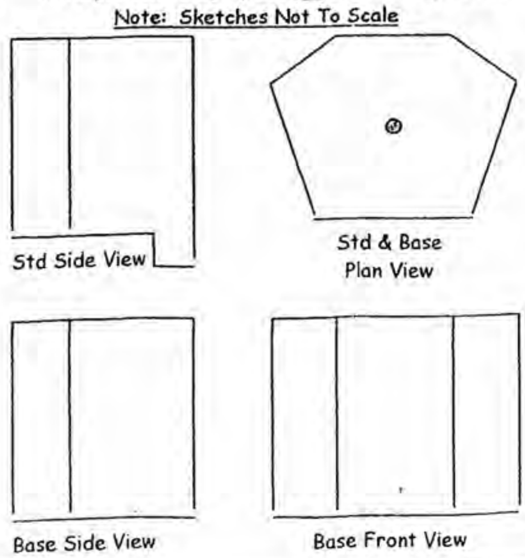
Standard Unit Dimensions: (Also available in a range of heights)

Face width	1185mm
Vertical height of front face	1000mm
Vertical height of rear face	1150mm
Depth of unit	880mm
Concrete strength	10MPa
Normal weight	1.8T
Normal density	$2.01 Tm^{-3}$

Base Unit Dimensions: (Also available in a range of heights)

Face width	1185mm
Vertical height of front face	1000mm
Vertical height of rear face	1000mm
Depth of unit	880mm
Concrete strength	10MPa
Normal weight	1.7T
Normal density	$2.01 Tm^{-3}$

Note: Sketches Not To Scale



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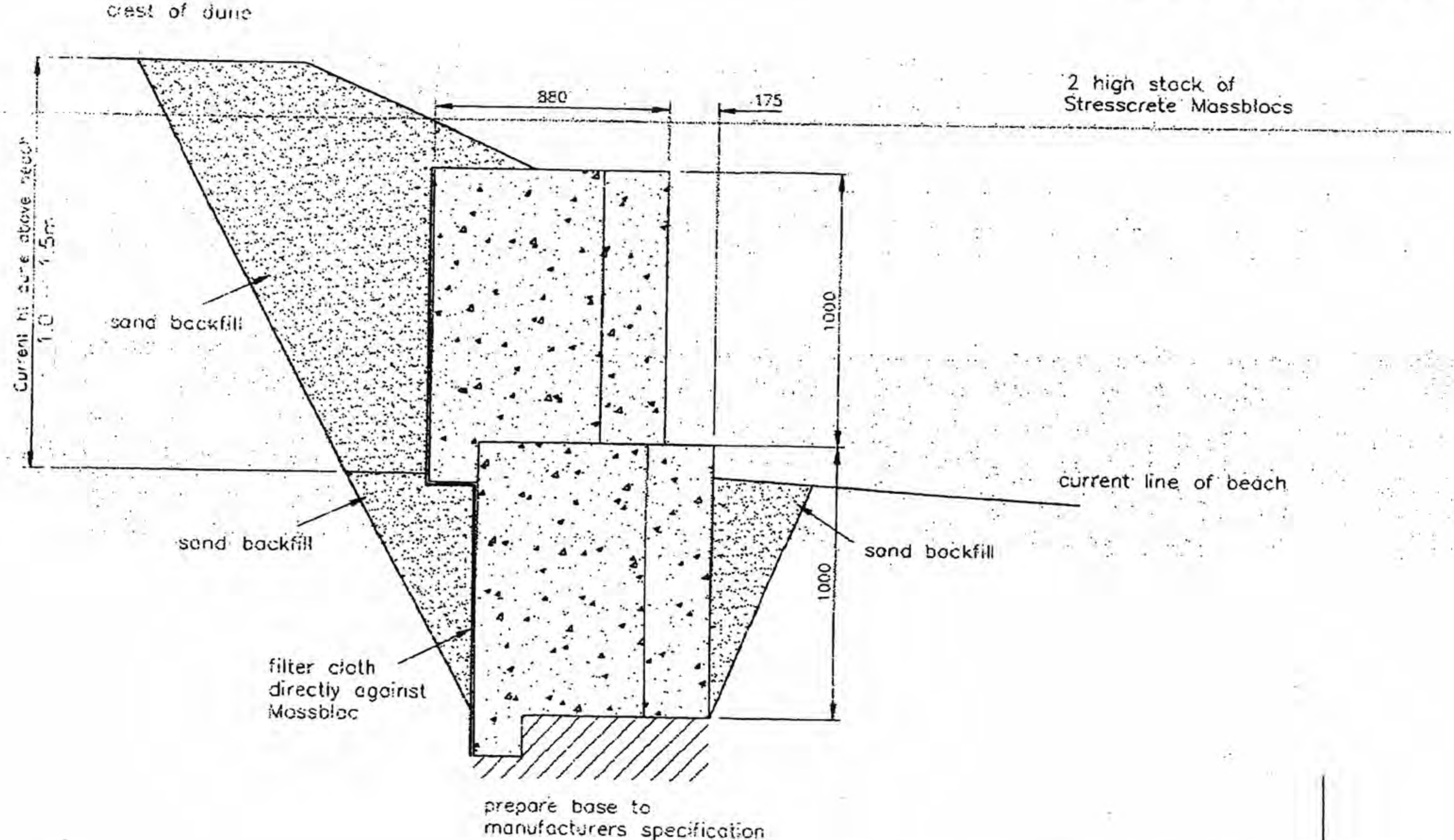
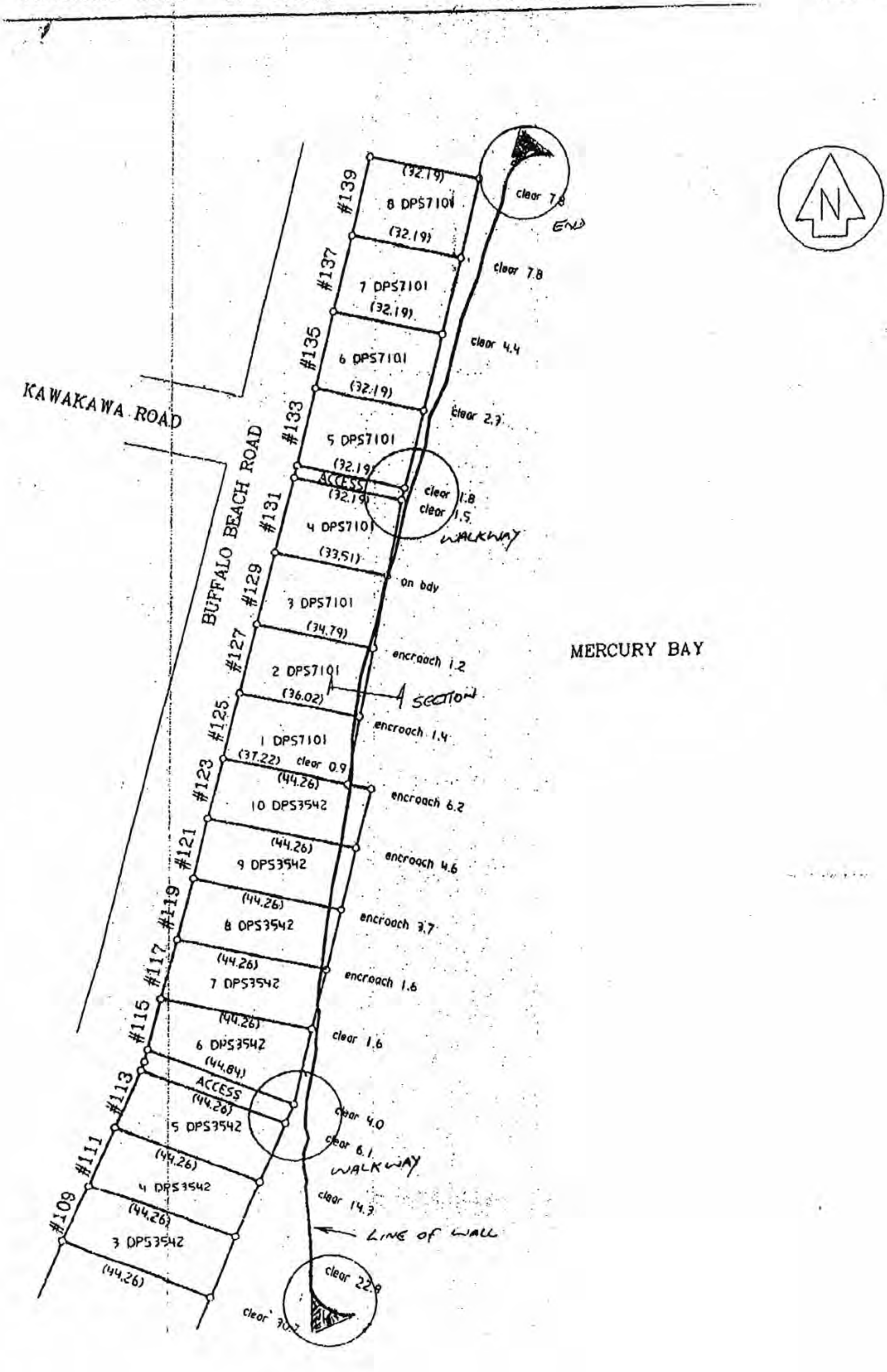


Designed & drawn A4 SIZE at 1:20 scale by B.F.Boult

Typical section through sea-wall — TITLE

Area D, Buffalo Beach — JOB NAME

2k46(section) — JOB NUMBER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clear 2.7
ACCESS
clear 1.8
clear 1.5
WALKWAY
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#127
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(36.02)
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#121
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encroach 4.6
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#117
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#111
4 DPS3542
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3 DPS3542
LINE OF WALL
clear 22.8


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Appendix D: Coastal Parameters & Buffalo Beach Monitoring Profile CCS25

1 Coastal Processes

Sea water levels

Sea water level at any location varies across a range of timescales and environmental conditions. Key components that determine water level are:

- Astronomical tides
- Barometric and wind effects, generally referred to as storm surge
- Medium term sea level fluctuations, including the effects of ENSO and IPO
- Long-term changes in sea level
- Wave breaking can also contribute to water level through wave setup and runup.

1.1 Datums

Levels have been provided according to Auckland Vertical Datum 1946 (AVD46) and the relationship to Moturiki Vertical Datum and Whitianga Vertical Datum is shown in the table below.

Land Datum	Offset	Source
Moturiki Vertical Datum 1953 (MVD-53	+1.3 m	Goring (2003) provides an offset from Whitianga RL Gaugeboard Zero (Chart Datum) to Mean Sea level of 1.3 m
Auckland Vertical Datum 1946 (AVD-46)	+1.29 m	Goodhue (2012) define difference between AVD-46 and MVD-53 as -0.0094m
Whitianga Vertical Datum 1994 (WVD-94)	0 m	Assumed Chart Datum based on Earthtech (2014)

1.2 Astronomic tide

Tidal levels derived by NIWA (2012) are based on sea-level records from the Whitianga sea-level gauge from 1999 to 2012. A tidal harmonic analysis was undertaken to predict high-tides for a 100 year period for Whitianga. These values for Whitianga, representative for Buffalo Beach, are presented in Table 1-1. The tidal levels are presented in terms of Auckland Vertical Datum 1946 (RL) as well as Moturiki Vertical Datum and Whitianga Vertical Datum 1994 (WVD94).

Table 1-1 Tidal levels for Whitianga (Goodhue, 2012)

Tide state	Moturiki Vertical Datum (m)	Auckland Vertical Datum 1946 (m)	Whitianga Vertical Datum 1994 (m)
Highest Astronomical tide (HAT)	1.02	1.03	2.32
Mean High Water Springs exceeded by 10% of occurrences (MHWS-10)	0.84	0.85	2.14
Mean High Water Neaps (MHWN)	0.59	0.60	1.89
Mean Level of the Sea[1] (MSL)	0.11	0.12	1.41

[1]Based on annual average mean sea level (1999 – 2014)

1.3 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 elevate the water level above the predicted tide (Figure 1-1). The combined elevation of the predicted tide and storm surge is known as the storm tide. Storm tide estimates for Whitianga were derived by performing a Monte Carlo simulation using recorded annual maximum sea levels provided by NIWA (Figure 1-2). These storm tide values including astronomical tide, storm surge and fluctuations in mean sea level are shown in Table 1-2 with respect to mean sea level at Whitianga and AVD-46. Consideration of a 1% AEP event for assessment of coastal flooding is considered industry best practice (Ramsey et al., 2013). There is a 39.3% chance of such an event being exceeded over the next 50 years and 63.2% chance over the next 100 years.

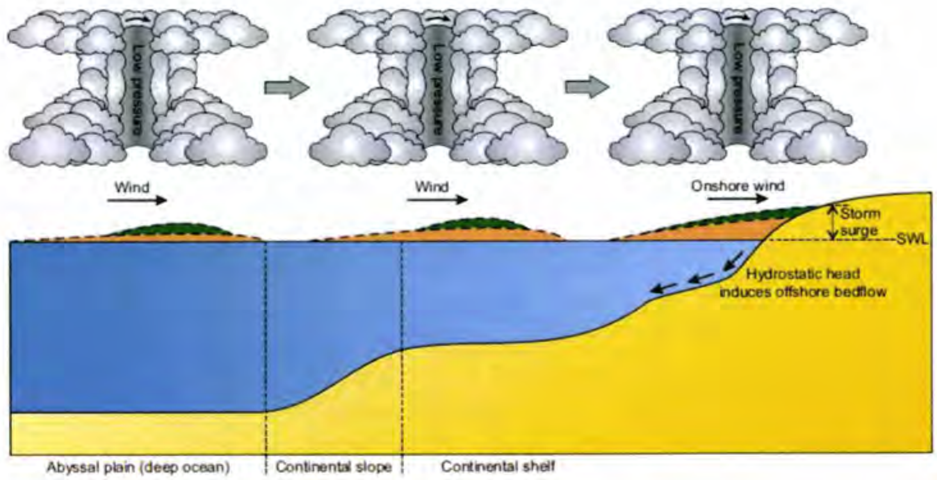


Figure 1-1 Processes causing storm surge (source: Shand, 2010)

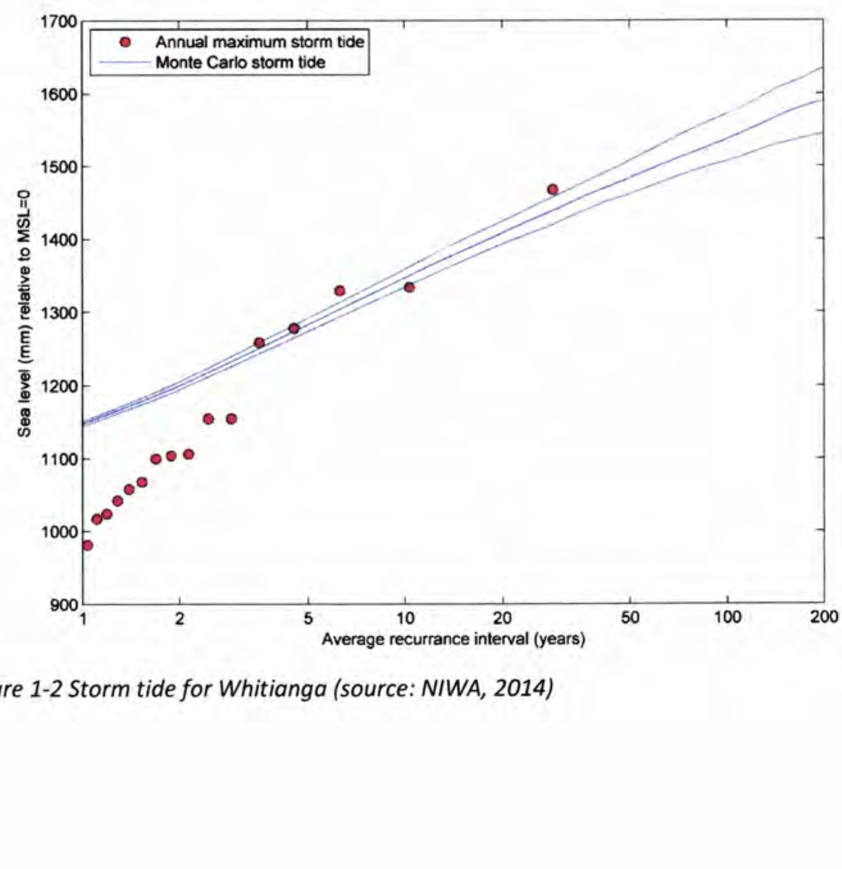


Figure 1-2 Storm tide for Whitianga (source: NIWA, 2014)

Table 1-2 Storm-tide elevations offshore of Whitianga (source: NIWA, 2014)

AEP (Annual Exceedance Probability)	Average Recurrence Interval (ARI)	MSL (m)	AVD46 (m)
50%	2 year	1.2	1.3
20%	5 year	1.29	1.39
10%	10 year	1.35	1.45
5%	20 year	1.41	1.51
2%	50 year	1.49	1.59
1%	100 year	1.54	1.64

1.4 Wave set-up

Waves can both super-elevate the mean water level during the breaking process (termed wave set up) and cause impulsive damage due to wave runup.

Numerical wave transformation modelling has been undertaken to determine likely wave conditions offshore of Whitianga Beach that may elevate water levels due to wave setup. SWAN (Simulating WAves Nearshore) is a third-generation wave model that computes random, short-crested wind-generated waves in coastal regions and inland waters. SWAN was developed at Delft University of Technology in the Netherlands and is widely used by government authorities, research institutes and consultants worldwide. Further details of SWAN can be found in Booij *et al.* (1999).

The model domain was constructed using bathymetry sourced from the LINZ Nautical Charts. Bathymetry maps of the central eastern part of the Coromandel peninsula (regional grid) and Mercury Bay, including Buffalo Beach (nested grid), have been generated. A 1% AEP significant wave height of 7.1 m and peak period of 11 seconds has been adopted as model boundary conditions based on wave data analysis by NIWA offshore of Mercury Bay. Winds are based on the yearly 1 hour wind speed from AS/NZS 1170.2:2011. Winds and waves from the north to southeast were tested with conditions from the easterly direction found to result in largest waves conditions at Buffalo Beach.

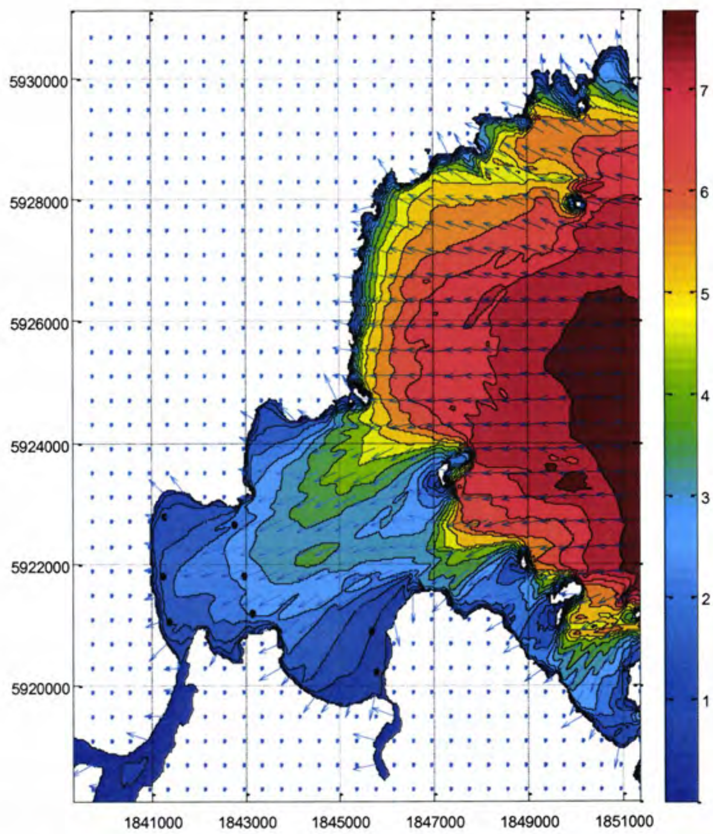
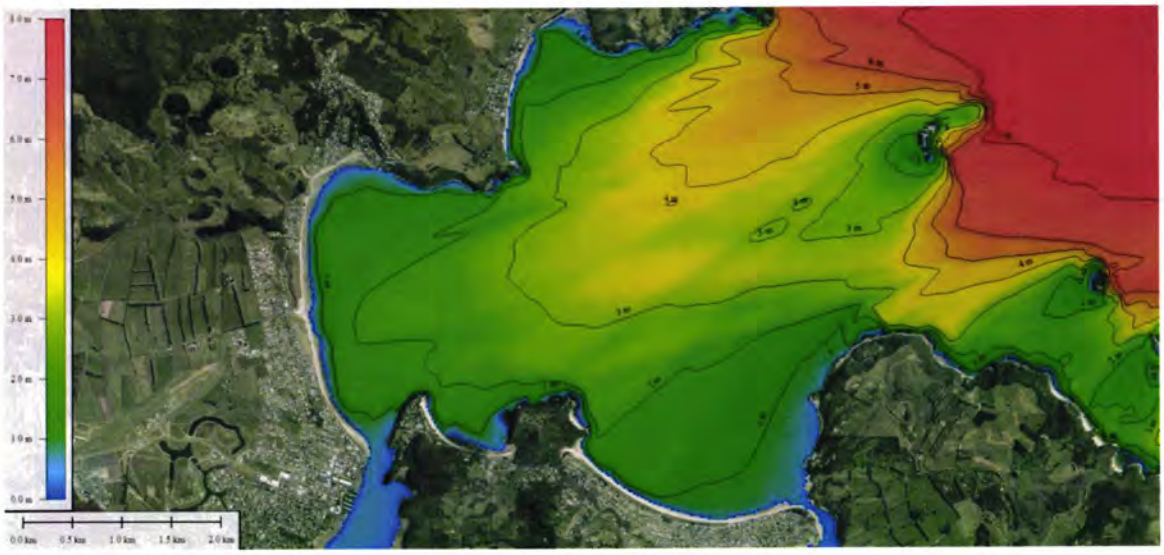


Figure 1-3 SWAN output for Mercury Bay for a 100 year ARI event from the East.



The SWAN model predicts a nearshore wave height of 2.15 m at the 5 m depth contour offshore of Buffalo Beach for a 1% AEP easterly event. Wave set up has been assessed based on the methods described within the Coastal Engineering Manual (CEM, 2006), with the resulting estimated wave set-up being 0.6m.

1.5 Sea level rise

Long-term changes in mean sea level should be considered. Historic sea level rise in New Zealand has averaged 1.7 ± 0.1 mm/year (Hannah and Bell, 2012). However, 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) guideline recommends a base value sea level rise of 0.25 m by 2050 (relative to the 1980-1999 average) with consideration of the consequences of sea level rise of at least 0.38 m by (refer to Table 1-3 below).

Table 1-3 Baseline SLR recommendations for different future timeframes (MfE)

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	

1.6 Design water level

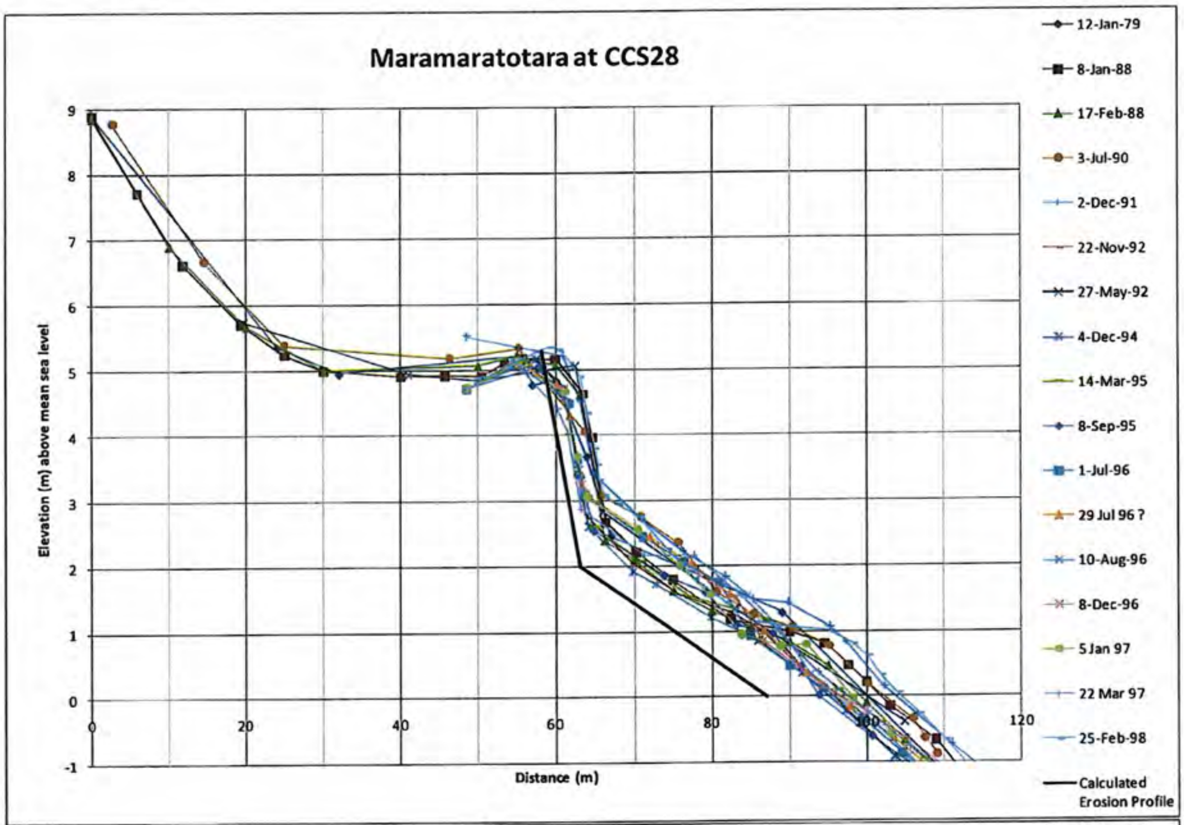
- Allowing for the above coastal processes, two design water levels have been determined for the seawall design, as follows:
- Present day 1% AEP storm tide level of RL=1.64m, plus 0.6m wave set-up, for a present day design still water level (SWL) of RL=2.2m AVD-46,
- Present day 1% AEP storm tide level of RL=1.64m, plus 0.6m wave set-up, plus allowance for predicted sea level rise of 0.38m, for an estimated 2050-2059 design SWL of RL = 2.6m AVD-46.

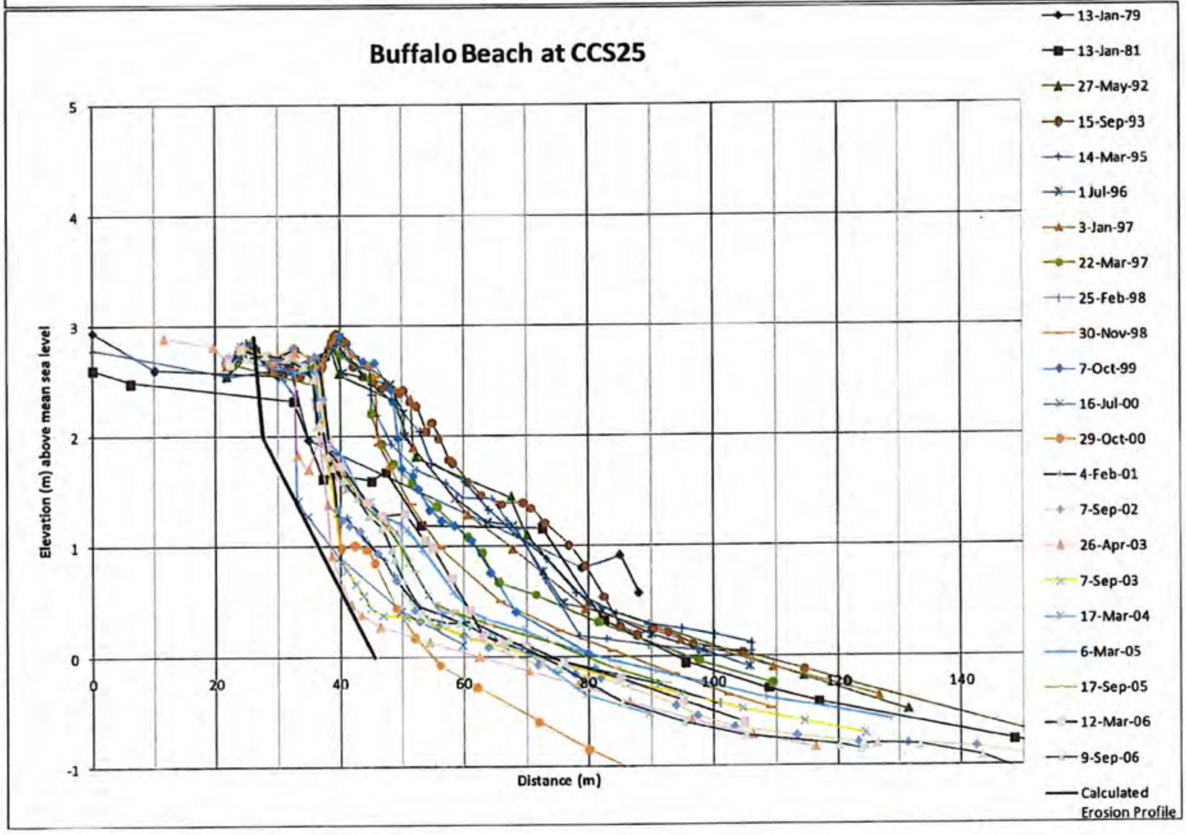
1.7 Beach profile monitoring

Beach cross-section profiles have been surveyed and monitored at Buffalo Beach since 1979, and monitoring cross-section CCS25 (see below), is located to the north of the site, in Macrocarpa Reserve. This monitoring location is considered close enough to be typical of beach fluctuations at the site, and the results of the monitoring were presented in the Focus Resource Management Group report "Coromandel Beaches-Coastal Hazards: Review of Primary Development Setback at Selected Beaches" (2009).

1.8 Design wave climate

The design wave climate is assumed to be depth limited, and has been determined at the toe of the beach, assuming the beach has been scoured to historic levels of approximately -0.5m AVD-46 (as shown in the beach cross-section monitoring profile CCS25 results). For design of the rock armour size wave set-up is ignored as it will not be fully developed at the seawall, and a design significant wave height (H_s) of 1.5m has been assumed. As the wave climate is depth limited, a variety of wave periods are possible and can combine with the significant wave height. The critical wave period for the rock armour design has been determined to be 7 seconds.


Maramaratotara at CCS28
Elevation (m) above mean sea level
Distance (m)
12-Jan-79
8-Jan-88
17-Feb-88
3-Jul-90
2-Dec-91
22-Nov-92
27-May-92
4-Dec-94
14-Mar-95
8-Sep-95
1-Jul-96
29 Jul 96 ?
10-Aug-96
8-Dec-96
5 Jan 97
22 Mar 97
25-Feb-98
Calculated Erosion Profile


Buffalo Beach at CCS25
Elevation (m) above mean sea level
Distance (m)
13-Jan-79
13-Jan-81
27-May-92
15-Sep-93
14-Mar-95
1 Jul-96
3-Jan-97
22-Mar-97
25-Feb-98
30-Nov-98
7-Oct-99
16-Jul-00
29-Oct-00
4-Feb-01
7-Sep-02
26-Apr-03
7-Sep-03
17-Mar-04
6-Mar-05
17-Sep-05
12-Mar-06
9-Sep-06
Calculated Erosion Profile

Appendix E: Typical section

Tonkin+Taylor	Project: Whitianga HOMEOWNERS SEAWALL	Computed: [signature] 19/8/2016	Office: AK
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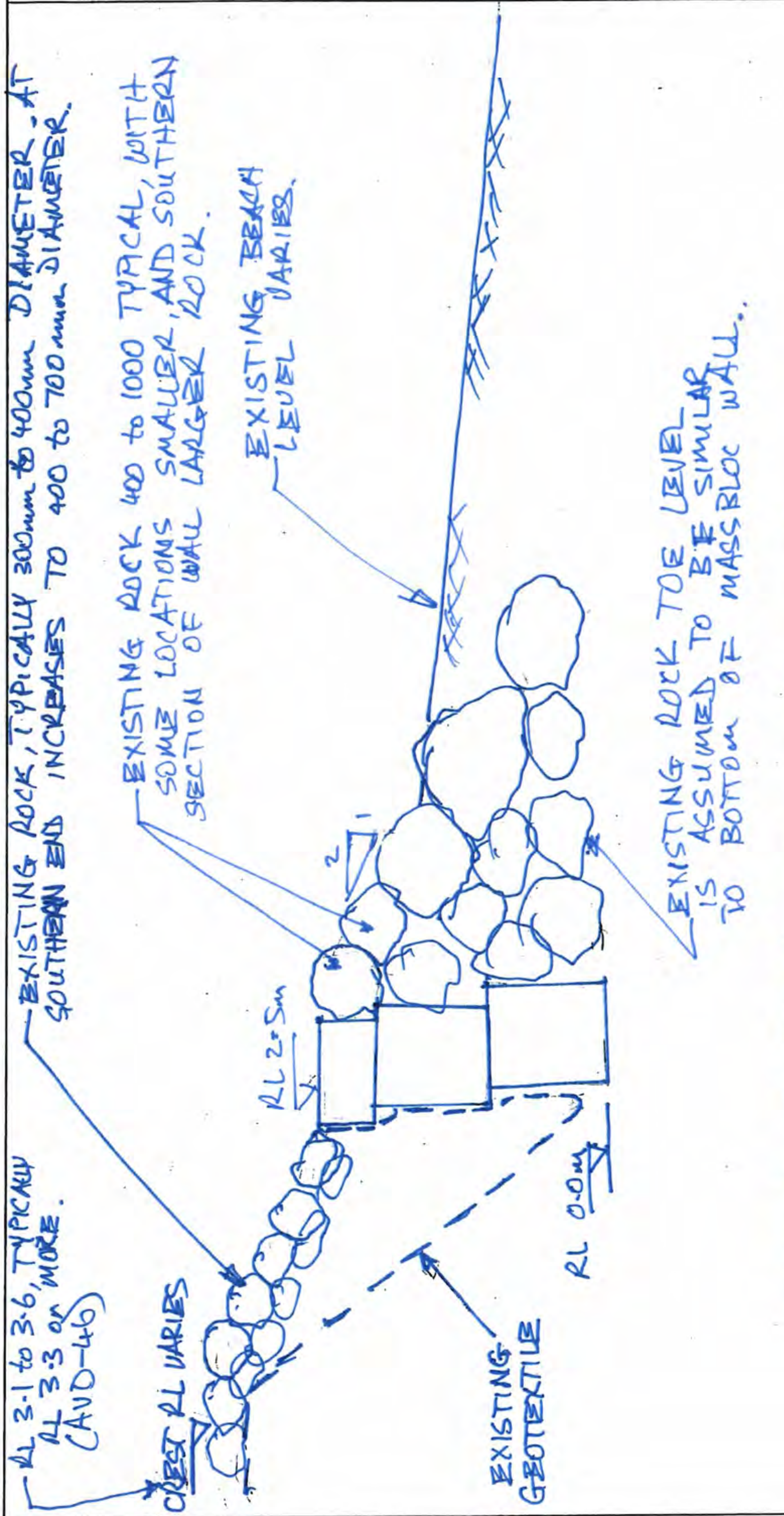
Description:		Revised: 20	File:
		Checked: 20	Sheet No. SK01



TYPICAL EXISTING WALL CONSTRUCTION

Tonkin+Taylor

Description:

Project: Whitianga HOMEOWNERS SEAWALL

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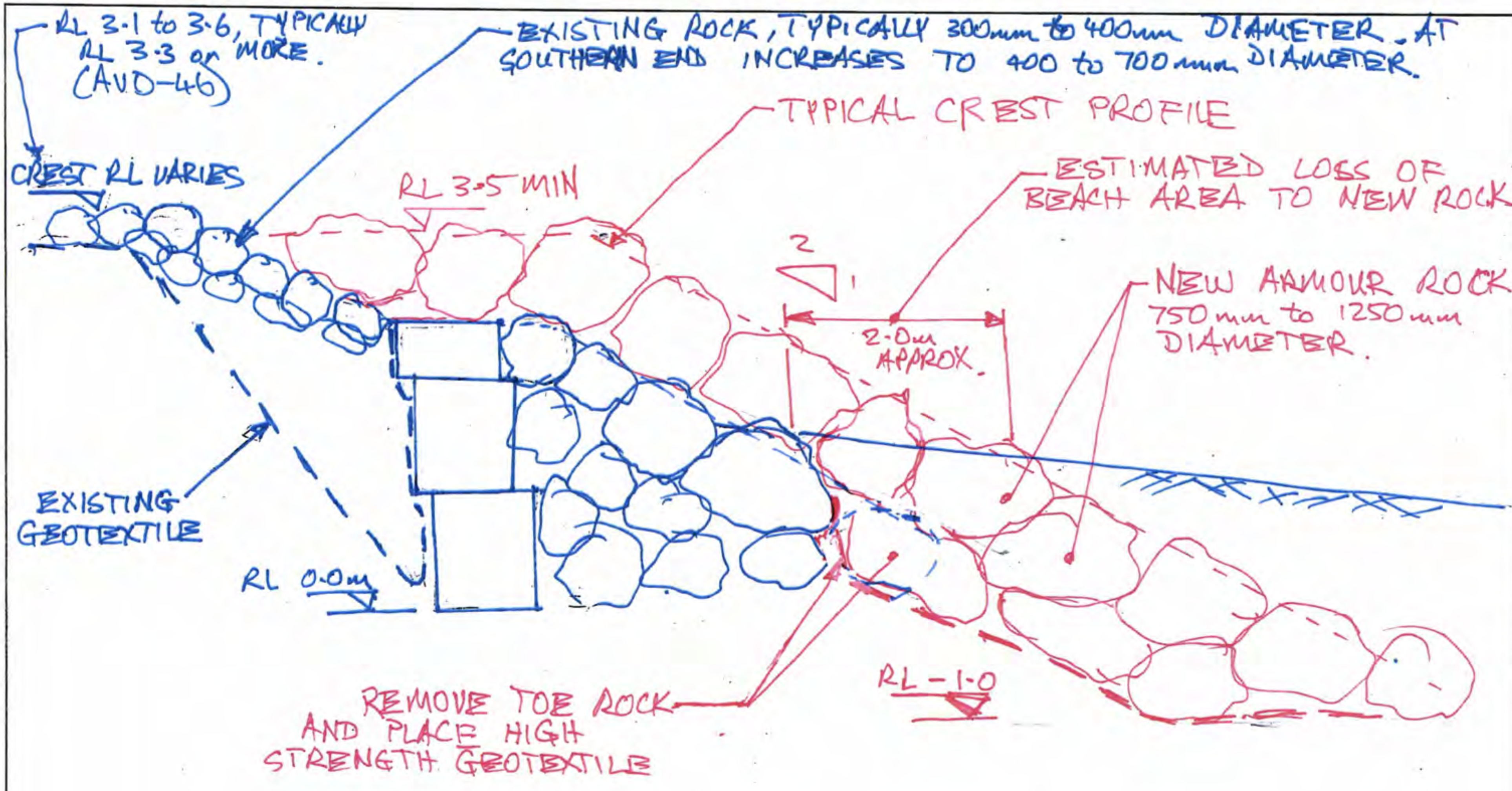
Checked:	20	Job No: 851872	
Revised:	20	File:	
Checked:	20	Sheet No. SK 02	



PROPOSED SEAWALL UPGRADE WORKS

OPTION A: EXISTING WALL REMAINS.

Tonkin+Taylor

Description:	Project: Whitianga HOMEOWNERS SEAWALL	Computed: GWP 19/8 2016	Office: AK
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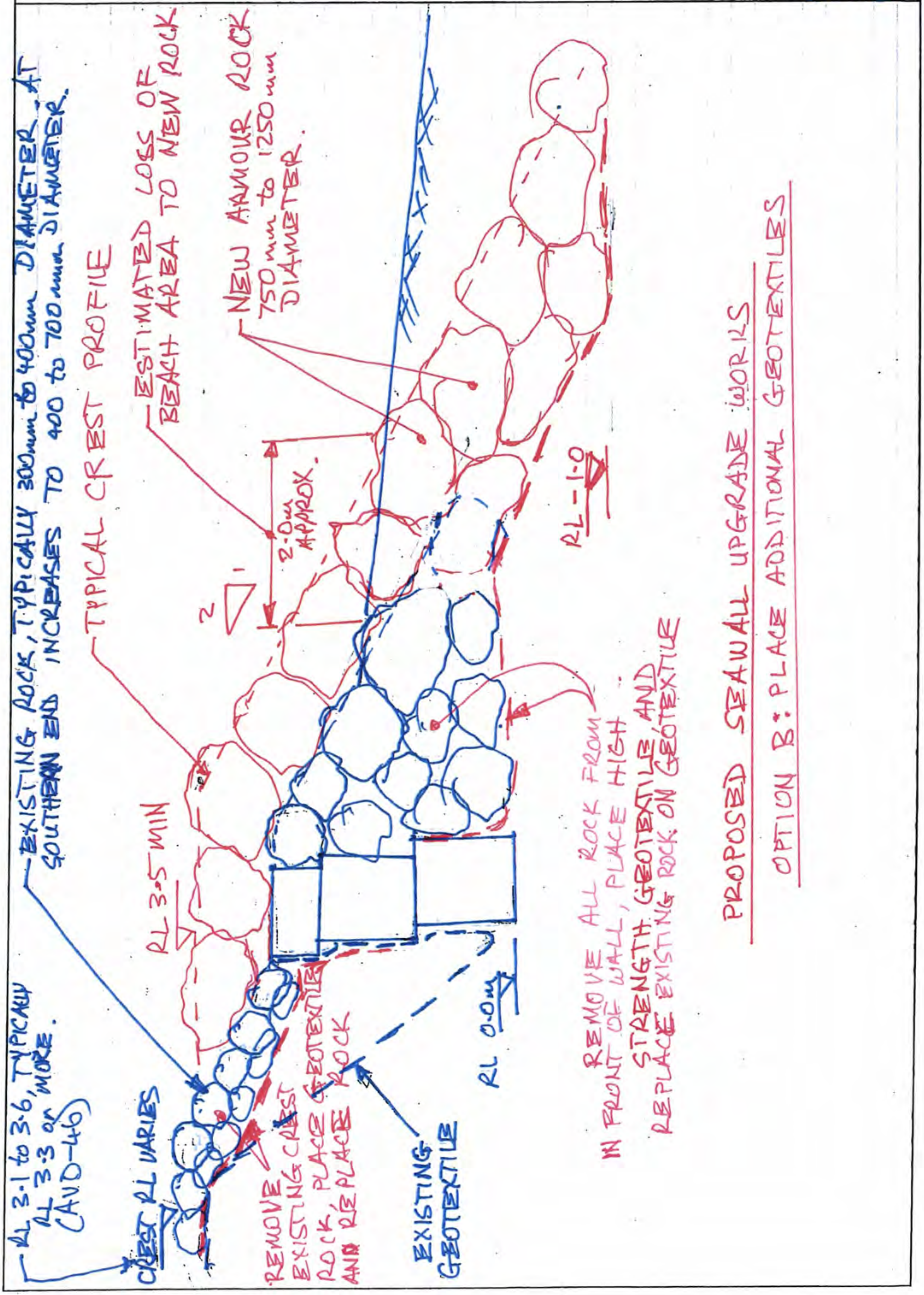
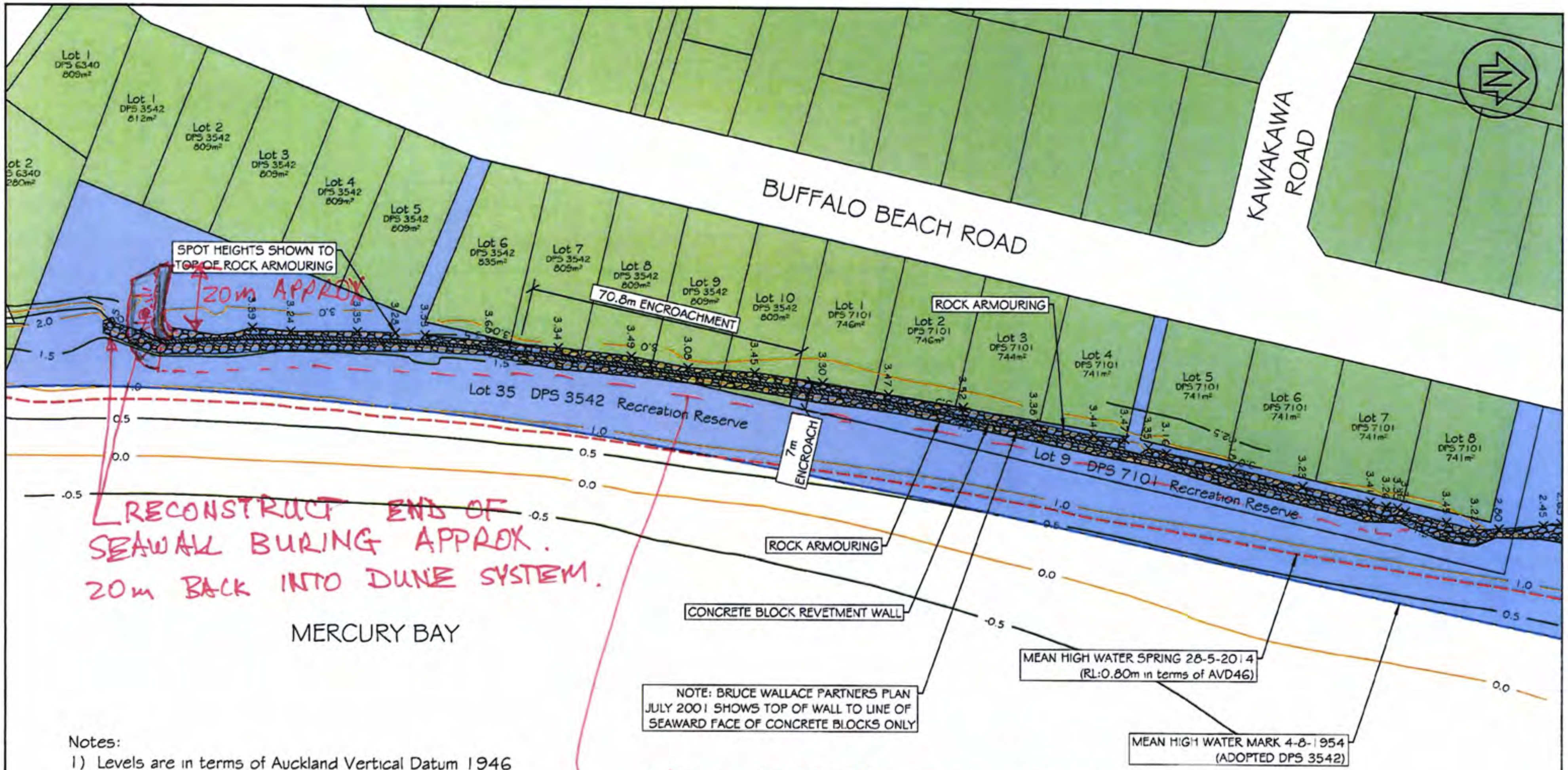
		Revised: 20	File:
		Checked: 20	Sheet No. SK 03





Notes:

1) Levels are in terms of Auckland Vertical Datum 1946
2) Origin of levels OBLK I SO 51500 RL: 1.99m
3) Source: TCDC Benchmark Booklet 2008
4) Contour interval: 0.5m
5) Mean High Water Springs = RL: 0.80m
(sourced from Whitanga Waterways level datum chart ref: 3808)

Date: 24 June 2014	Scale: 1:1000 @ A3	TOPOGRAPHICAL SURVEY OF EXISTING REVETMENT STRUCTURE BUFFALO BEACH ROAD, WHITIANGA	Drawn CRS / Designed / Verified / Approved	RMS SURVEYORS Stewart Group Limited	Project No. F7116
No. Date Revision Details	By Ver. App.				Drawing No. Rev. T1

SKETCH SK04.