Coastal Protection Structure

for

Glen Isla Protection Society

Engineering Design Report

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1.0 Introduction

The Glen Isla Protection Society ("**GIPS**") comprise a group of beachfront homeowners at 9, 11, 13, 15, 16, 14 and 12 Glen Isla Place, Waihi Beach. Their properties, located south of "Three Mile Creek", adjoin an approximately 200m stretch of unarmoured coastline, subject to coastal erosion. This land is vested in the Western Bay of Plenty District Council ("**WBOPDC**") as Esplanade Reserve.

Following a series of large erosion events, including Cyclone Gabrielle, the adjacent property owners are seeking to provide a buried rock revetment, supplemented with dune nourishment and replanting. The intent is to protect the backshore from further erosion, and future-proof their properties from the effects of sea-level rise.

The proposed structure will be landward of the foredune system and allow the beach to function as it has in recent decades. It will only be exposed during the largest storm events, when it will protect the backshore and limit the extent of the largest of the natural beach fluctuations.

GIPS have instructed Davis Coastal Consultants to prepare this report, setting out the design philosophy and assumptions and calculations undertaken as part of the structure and dune development. The retreat of the beach due to climate change driven sea level rise has been considered and is addressed through the design and future adaptation of the structure in future decades.

A further report by Davis Coastal Consultants (Assessment of Coastal Processes October 2024) addresses in detail the coastal processes at the site and the erosion management options considered for this project.

2.0 Description of Existing Environment

2.1 Location

Waihi Beach is a coastal township located on the east coast of the North Island at the northern extent of the Bay of Plenty Region. It is an open coast beach, situated approximately 10km east of the inland town of Waihi. The site is directly south of Three Mile Creek (Figure 2.1a) and extends south from the creek along approximately 200m of coastline, seaward of properties 9, 11, 13, 15, 16, 14 and 12 Glen Isla Place.

The wall extents are shown below (Figure 2.1b).

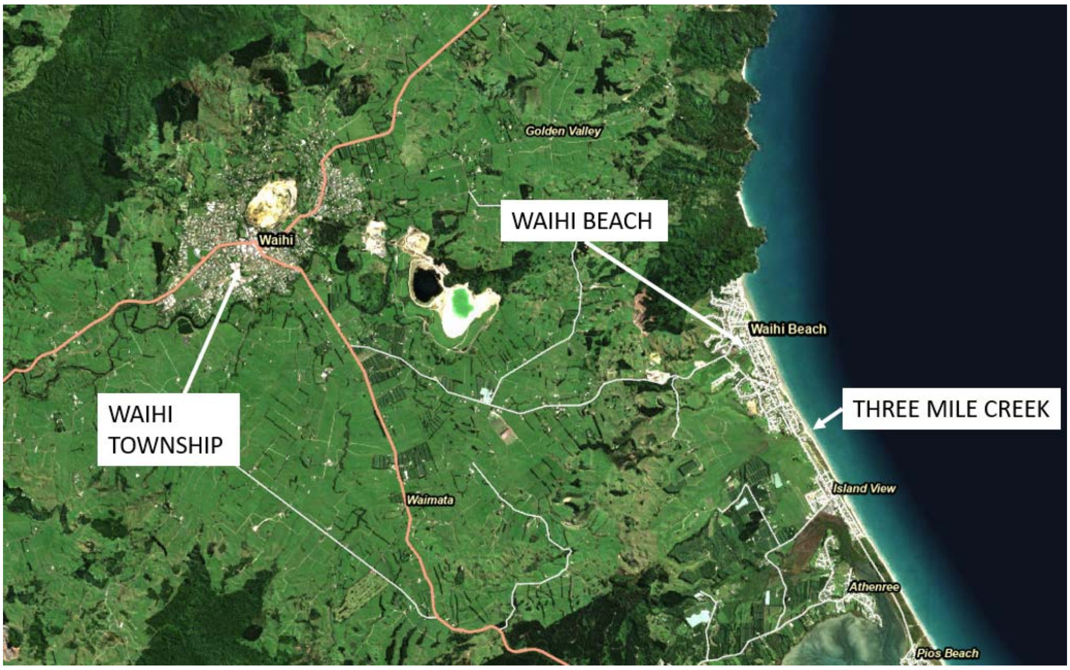


Figure 2.1a: Waihi Beach

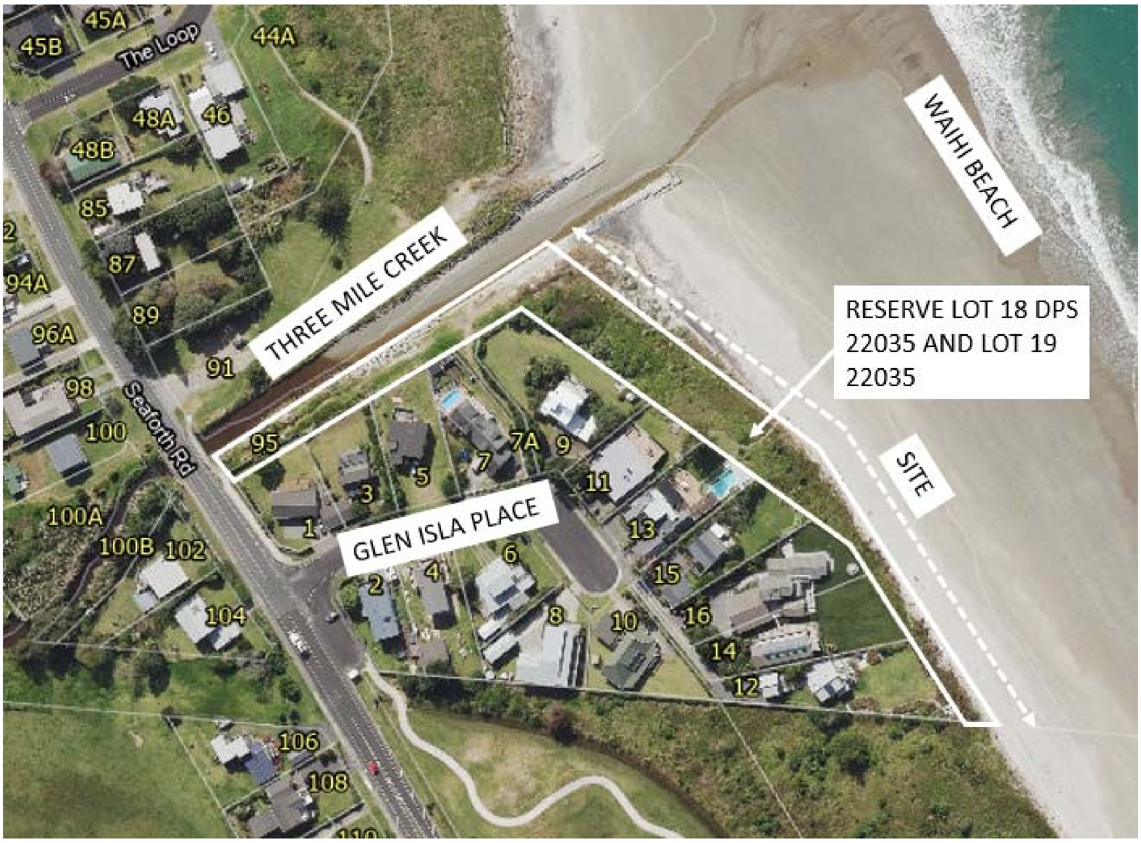


Figure 2.1b: Site adjacent to Three Mile Creek, Waihi Beach

2.2 Site Description

The site is located towards the centre of Waihi Beach, along a stretch of coastline characterised by a gently sloping intertidal area (approximately 1:17) which rises slowly to an unarmoured, narrow, vegetated back dune. The beach is part of the barrier system associated with Tauranga Harbour which all faces northeast and is exposed to the open ocean. The properties are located landward of the narrow frontal dune, at an elevation of approximately RL4-5.

The current vegetated extent of the back dune system at the site is approximately 20m wide closest to the channel outlet and narrows to approximately 4m wide in front of 12 and 14 Glen Isla Place. The dune is vegetated with a small number of isolated shrubs and trees at the landward section with more ground cover species at the seaward extent. At the northern end of the site, the dune slopes from RL4.5 at the bank crest to the foreshore at RL3 at approximately 1:16. At the southern extent of the site, the dune comprises of a recovering storm scarp and slopes more steeply at 1:4 from RL4.5 to RL3 towards a sandy beach which is approximately 100m wide.

Three Mile Creek comprises the northern extent of the site. The Creek discharges via twin geotextile training groynes across the upper intertidal area and foreshore. The banks of the creek are armoured up past the site for approximately 120m to the road bridge. The Creek discharge is naturally blocked by sand movement within the intertidal zone and is manually cleared by excavator approximately once a month with the sand deposited in front of the northern end of the site, near the groyne.

Since construction of the stream training groynes in 2011, the northern end of the site has been accreting. The dune has realigned closer to the general beach alignment rather than as part of the stormwater outlet “inlet” system. This has led to the wider partly-vegetated dune at the northern end.

Figure 2.2: Features of the site

3.0 Options Assessment

3.1 Beach Erosion Management Options

A number of options are available to address erosion of a dune beach where it threatens property, buildings and infrastructure. The preferred erosion management option is to ensure built development is kept sufficiently away from the coastline so as to avoid any issues. Many New Zealand beaches, including Waihi Beach, have undergone historic development that did not achieve this, rendering this management option irrelevant.

Similarly, where aggressive long-term shoreline retreat is occurring over short timeframes, options to remove built development from the area should be considered. Given the financial and emotional implications, this option is only considered where more modest intervention is not practicable or would have significant environmental effect.

Waihi Beach is already highly modified with a number of groynes and approximately 1.3km of seawall immediately north of the Glen Isla Place site. The coastal erosion risks can be addressed by conventional engineering approaches, similar to those elsewhere on Waihi Beach, so retreat options are not considered appropriate at this stage.

The options discussed for the site are:

- Do Nothing;
- Soft Engineering (Renourishment and Planting); and
- Seawalls / structures of various materials and configurations.

It is envisaged that dune reestablishment and a high-quality dune revegetation regime would be part of any coastal structure proposal.

3.2 Do Nothing

This is a preferred coastal management approach and allows the beach to act in an entirely natural manner. The dune line will fluctuate and may retreat with losses and gains accepted. This results in the highest quality natural system. For the proposed site, residential and urban infrastructure development has been undertaken too close to the beach front to allow this approach. Indeed, it is understood that residential housing development occurred while the Council had in place a seawall protecting this area and therefore provided a more seaward protected dune profile, promoting more seaward development.

The extensive dune front work of seawalls, groynes and dune replenishment along the residential properties that border the coastal margin along Waihi beach is ample evidence that the GIPS properties are at risk without coastal protection measures. The Do-Nothing Approach is therefore inappropriate at the site.

3.3 "Soft Engineering" Renourishment and Planting

Renourishment has been tried in front of Glen Isla Place in 2011. The cost of this is being financed by a targeted rate on the beachfront properties. There is no perceptible difference between the coastline that was nourished and the adjacent coastline to the south (Figure 3.3) suggesting that renourishment effects have now been lost. It is likely that the renourishment helped fill the dune adjacent to the Three Mile Creek groyne and more quickly attain a prograded position associated with the groyne.

The area immediately around the groyne is more seaward than it would be without the groyne and could be considered to be 'holding' additional sand. Dunes over time establish a varying position dictated by the differing forcing mechanisms. The dune south of the groyne is likely to have eventually obtained its natural range of profiles without renourishment although that may have taken more time.

Further, targeted nourishment is likely to have very short residence time, with sand redistributed to the adjacent beach. Renourishment placed on a beach system within the tidal zone is quickly redistributed by wave action during higher water levels. Very large volumes would be required for renourishment to hold an unnaturally seaward dune line. The minimal duration and ongoing cost of the previous renourishment preclude this being a practicable solution for the beach area adjacent to the GIPS properties.

However, reinstatement of areas of storm cut back dune can help to accelerate the natural dune recovery processes. When coupled with planting, this work helps reestablish a healthy dune system which will tend to sit further seaward on average than a degraded dune system. Dune reinstatement and replanting of this type provides minimal additional protection to the backshore against extreme storm events. However, it creates a more seaward average dune position which results in less risk from smaller, more regular events, and is effective in maintaining cover to a buried structure for longer periods.

Figure 3.3 – View south from 14 Glen Isla Place

3.4 Coastal Armouring Discussion

Design of seawalls on a sandy beach requires consideration of both the type of wall, and situation on the beach. There are reported issues with seawalls on sandy beaches and these are discussed below, before the options for wall type are analysed. This exercise was undertaken to inform the design of the structure for this proposal, thus ensuring a robust decision-making process has been undertaken.

3.4.1 Historic Seawall Performance on Sandy Beaches

Seawalls or coastal armouring solutions for mobile sand dune beaches have a history of generally poor performance in New Zealand and, indeed, in many places internationally.

Typically, the reasons for the poor performance and/or failure are a lack of understanding of the dynamic nature of the shoreline, and poor detailing. Designs commonly have not accounted for the particular issues of walls in the coastal environment - as opposed to retaining or river armouring structures. Often medium to long term shoreline changes or retreat have resulted in wall failure. These changes are often driven by storm events or a change in long term drivers of beach shape.

In New Zealand, many beach seawalls / structures have been constructed in an ad-hoc manner often by adjacent residents rather than qualified designers / engineers and this has added to the history of poor performance. Structure failure has often led to degradation of the visual and physical amenity of the beach through the loss of wall material (rock, concrete etc.) on to the beach.

Similarly, because of poor design and placement, walls have often resulted in an unnecessary loss of public access particularly at high tide.

These negative outcomes are considered to have been one of the drivers behind current coastal policy direction (eg Policy 27, NZCPS 2010) against coastal armouring. It is therefore important to ensure these sorts of outcomes are avoided in provision of a solution. If these issues are addressed, coastal armouring can have positive outcomes in terms of managing situations where property is threatened by coastal erosion while having minimal effect on the foredune and foreshore. Any effect will tend to be transient and comprise limiting the maximum extremes of the shoreline fluctuation.

3.4.2 Wall Location

Where shoreline armouring is identified as the appropriate solution, the key design consideration to minimise the effects and maximise its efficient function is the <u>location of the wall within the range of beach profiles</u> likely to occur at the beach.

A wall placed too far seaward will affect the nearshore processes over most of the tidal cycle. It is likely to cause beach lowering and more likely to create public access issues. It is also prone to failure from higher wave loading, the loss of sediment around or through the wall or overtopping. Significant end effects are likely to be associated with the location of the armouring in this position, due to a greater level of interaction with incoming wave energy.

The same specification for a wall placed more landward may however, be entirely adequate. In the extreme, if the wall is placed far enough landward it will never be impacted by the sea and, by definition, have no effect. Once an understanding of the range of profiles likely and the appropriate location for the wall is confirmed, then sound coastal engineering practices will ensure successful function and minimal negative impact on coastal processes.

3.4.3 Beach Profile

The profile of the beach varies due to many drivers on differing times scales. Diurnal tidal changes, seasonal and storm event changes as well as longer term changes from longer weather patterns. Changing weather patterns can be associated with El Nino/ La Nina cycles over many years and pan-decadal patterns have also been defined. In combination with these fluctuations, changes in the beach drivers such as a change in sediment supply, change in local control points (river outlets, headlands), sea level rise or man-made effects can also create changes in the profile.

Waihi Beach has reasonable monitoring data for 30 years, and a limited amount of older data, for consideration. This monitoring tends to show a generally stable beach with possibly some overall retreat, but this may be masked by human interventions. Neither of the existing monitoring locations are in the vicinity of the site, where the effect of the groynes and the Three Mile Creek outlet have led to accretion in the historic record, albeit with periods of extreme retreat such as that witnessed in Cyclone Gabrielle.

Given a 30-year envelope of beach position it is possible to locate any armouring at the rear of the defined envelope and expect it to be only rarely exposed. The provision of a structure markedly landward of the rear of the envelope will ensure that the structure will not be visible except as a result of extreme events. Between such events, sand on the profile will establish to historic equilibrium levels and the structure will become buried again.

Longer term changes due to such drivers as sea level rise or beach lowering associated with stormwater outlets and reflective walls would gradually tend to make the wall visible more often. If this were the case, it suggests that these long-term changes are threatening the properties and necessitate the armouring.

3.5 Wall Type Discussion

3.5.1 Stabilised Sediment Wall

Stabilised Sediment walls utilise beach sediment (sand) mixed with cement to create a mass structure to provide coastal protection. The result is a natural looking mass, similar to a sandstone, that has a similar texture and colour to the surrounding beach material. This ensures far less visual impact than say a rock revetment and has been mistaken for natural formation.

With suitable detailing, the wall configuration can incorporate informal or formal pedestrian high tide access (Photograph 3.5.1 below).

Photograph 3.5.1: Stabilised Sediment Wall forming a public path – Omaha 2012

The hardness of the wall depends on the quantity of binder (cement) added, with results ranging from uncemented (beach nourishment), improved soil (0.3-1MPa), to a fully Stabilised Sediment wall with an unconfined compressive strength of 3-15 MPa. This can be compared with concrete that has a strength of 10-60MPa.

The durability of a Stabilised Sediment solution is dependent on the consistency of mixing, amount of binder added and the exposure to wearing forces such as the continual lapping of tide and waves. Dune sand provides a readily mixed homogenous sediment which can provide a very strong hard stabilised wall. With sufficient well mixed cement there is no reason that the wall cannot have the hardness similar to concrete. However, in practical applications it is designed slightly less hard and allowance made for some loss of section over its lifetime. Unlike concrete it does not have the issue of steel reinforcement corroding and breaking up the structure and it is not a thin "sheet" structure like concrete. Stabilised sediment forms a much larger heavier mass than the discreet armour rocks of a revetment wall for instance.

Seawalls constructed of Stabilised Sediment are much less proven technology than rock revetments (see Section 3.5.4). Previous projects have been primarily on estuarine and harbour settings rather than open exposed beaches. Further, it requires specialist plant to achieve a high-quality outcome, and also tends to be more expensive and labour intensive than a rock revetment. Finally, the protection of Waihi Beach is dominated by existing rock revetment walls. Introduction of a differing wall material risks creating an ad hoc appearance to the beach front.

Due to the length of the structure, and the lack of local contracting experience with this style of structure, and lack of open coast examples there are risks with this solution that are avoided with the use of a Rock revetment.

3.5.2 Rip Rap/ Rock Revetment

This is a well-established coastal protection solution internationally and in New Zealand. Standardised design guidance is available defining a suitable rock size for a given wave climate and maintenance regime. A typical solution has been constructed north of Three Mile Creek and was previously considered for the Glen Isla Place properties in a report to Council (T&T February

2004). These existing walls performed well through the Gabrielle/Hale storm events and provide a ”site tested” example for a solution.

Rock revetment armouring is a proven and cost-effective solution, providing a relatively dissipative robust defence. Whilst the proposal is for a buried structure, storm events and/or shoreline retreat may expose the wall periodically. In this instance, there is value in having a single armouring system for a dune system and beachfront so that a haphazard ad hoc beach front appearance is avoided.

Rock revetments are wide structures occupying a large width of the foreshore and/or fore dune. They can limit access across the profile and can result in the discharge of small rock into the beach system, if poorly constructed. As protection tends to be required because there is limited dune between development and the ocean, the wide rock revetment structures tend to be exposed and cover the foredune and in cases the upper foreshore. This results in negative effects on natural character and public access.

Given the location of the Reserve between the coastal area and the properties, the Glen Isla location would allow a revetment solution to be placed further back in the beach profile to act as a backstop wall. Based on the historic beach location the wall would be buried for almost all states of the beach profile except in large storm events or prolonged periods of retreated beach. As the beach undergoes retreat due to sea level rise over the next 50-100 years it will become progressively more exposed.

A buried rock revetment solution is considered suitable in this location, and given the existing revetments on the subject coastline is the preferred option.

Photograph 3.5.2: Rock revetment at Matheson Bay, Auckland

3.5.3 Rock Masonry

A rock masonry wall can be a high amenity seawall solution. These solutions can be compact and will therefore occupy little space across the shore profile.

Masonry walls require a stiff foundation and would require piling to levels below where sand may shift. They are relatively expensive particularly where they become high and exposed to open coast wave load. They are not considered an appropriate solution at this location.

3.5.4 Concrete Walls

Similar to the rock masonry walls these structures need very stiff foundation to prevent settlement and associated cracking. However, these structures take up a reasonably small space on the profile and can be established as backstop walls in cases where there is little room. Concrete structures are relatively thin and require steel reinforcement for tension forces associated with flexure. If wall cracks become large enough or as concrete losses its alkalinity with age, steel reinforcement can begin to rust leading to failures of the wall. Accordingly, this wall type was considered no further.

3.5.5 Vertical Walls

Vertical walls of steel sheet piles or drilled concrete piles forming a palisade wall or timber vertical retaining are options used as seawalls in some locations. At an open ocean beach care would be needed with these structures so that they were well back in the beach profile and adequately designed. This type of wall has been subject to many failures in the coastal context. The situation is likely to require a wall too high for a timber wall leaving a steel or concrete wall. As referenced the Councils Coastal Erosion policy recommends against concrete walls. A steel sheet pile wall will be subject to surface rust and be unsightly.

Vertical walls provide a complete barrier to pedestrian access and if they become exposed could create a fall hazard. They are not considered appropriate at this location.

3.5.6 Preferred Option

Of the options considered, either the Stabilised Sediment or the Rock Revetment buried within the dune line were considered most appropriate. Alignments for both options were considered and an alignment 5m seaward of the private property boundary adopted.

By adopting a wall high up on the beach profile, that will be buried for much of its life, the structure achieves:

- Provides for construction outside the CMA
- Enables a soft engineering / replanting approach to be incorporated in the design
- Maintains public access during and post-construction

Further, a rock revetment solution has been identified as the preferred option because of the benefits of:

- Achieving a consistency of built form and style across the existing armouring on Waihi
- Widely, tested and accepted design formulation for coastal erosion
- Proven solution on the site, with northern walls managing recent large storm events
- Using locally sourced rock for the revetment
- Local contractor experience with rock revetments

4.0 Design Considerations

The proposed works include the construction of approximately 200m of buried revetment structure. The structure will primarily be located within the WBOPDC Reserve land at the northern extent, and the adjoining foredune area at the southern extent. Figure 4.0 shows the layout for the proposed structure.

The design details and design approach are detailed further below.

Figure 4.0: Layout Plan of rock wall

4.1 Design Philosophy

<u>Landward Structure and Dune</u>

The underlying design concept has been to provide a combination of hard and soft engineering options to achieve certainty of protection, while enhancing the natural dune's ability to adapt to short term shoreline fluctuation and thus minimise any effects on the natural character of the shoreline.]

The solution comprises a rock riprap wall constructed as landward as practicable, and a replenished, recontoured and replanted dune environment covering the structure as much as practicable. The intention is that this will provide a high-quality functioning dune environment and contribute to better environment outcomes, while also helping to mitigate the risk of coastal erosion impacts on the landward properties and the Council Reserve, as a result of large storm events and / or ongoing coastal retreat.

4.2 Design Features

Buried Rock Revetment

At current sea levels, based on 30 years of monitoring data, the landward siting of the structure will allow a full range of natural beach fluctuations to occur in front of the structure without exposing the structure. In the most extreme storm events, the upper structure of the southern end may become exposed, and it will limit the loss of the backshore area. The structure has been placed a minimal distance from the property boundary to allow future access along the top of the structure to allow future maintenance. In addition, in the fullness of time, if beach use becomes limited by sea level rise, it may provide high tide route for emergency use. The vegetated band between the structure and private property allows an area within which a native dune plant sequence can be established following the completion of works. Dune rehabilitation, discussed in detail in the *'Glen Isla Dune - Coastal Protection Project, Ecological Assessment'*, will include foredune planting with appropriate species but also room for the transition into back dune species typical of the less dynamic beach area.

Although it is intended the structure remains buried to the extent possible, the structure has been designed to be fully exposed to incident wave action as a rock rip rap wall. It is designed cognisant of the successful performance of similar structures immediately to the north during recent storm events.

Retreat of the shoreline, due to accelerated sea level rise associated with climate change, has been considered over a 100-year period. The structure has been designed to allow the expected beach retreat over the next 50 years. Over the 50–100-year period, the structure can be readily adapted to expected further retreat.

The structure has not been designed to address overtopping or inundation other than to ensure that overtopping would not cause the structure to fail.

Dune Replenishment and Planting

Dune re-establishment will occur in all areas of land disturbance, including the seaward area in front of 12, 14 and 16 Glen Isla Place, which has historically been vegetated dune. Recent extreme storm events have removed the foredune area and the works seek to reinstate this in a comprehensive manner. Large volumes of sand have already naturally returned to the foreshore and foredune areas since the storm events, supplemented by sand from the maintenance of Three Mile Creek. These areas are already being colonised by dune building species. The rebuilding will be enhanced by using excavated material to recontour the dunes and planting native dune species. (refer to the recommendation in the Ecology Report). Planting of native dune species, in particular spinifex, on the dune front will hold sand and promote dune growth. Backshore species, where provided, on the upper bank will help lift the wind flow and stabilise this less dynamic area. Pedestrian traffic across the dune will be controlled by wind fencing which will also protect young plants, encouraging vigorous early growth.

Large scale dune building and planting was undertaken in front of the Glen Isla properties in 2011 and largely lost to storm erosion the same year. This planting was immediately adjacent to, or partly within, the active upper intertidal area and part of the sand was provided by lowering the area in front of the planting. While there is a risk to any dune rehabilitation that large storms may significantly impact works, this risk is minimised by creating the planted area landward of the active intertidal area as far as practicable and letting the dune naturally grow seaward. It is anticipated, given the current dune is recovering from it's most landward over the last 30 or more years, that the new dune can be established in a position that is rarely subject to wave action. This should allow strong growth of vegetation before being subject to wave erosion.

4.3 Structure Siting

The 200m structure will run generally parallel to the landward site boundary (Figure 4.3). The southern approximate 70m runs parallel to the boundary and beach orientation with the rear of the structure approximately 5m from the property boundary. This enables construction access on the landward side of the structure and limits disturbance to the beach system to the extent possible. The northern approximate 130m of the structure is on two slightly different alignments. From 16 Glen Isla Place, the structure runs north approximately 57m to a point adjacent to a large established Norfolk Pine tree where it is located approximately 7.6m from the boundary of 13 Glen Isla Place. The structure continues to run north the final 73m to a point 6.5m off the boundary of 9 Glen Isla Place its northern end.

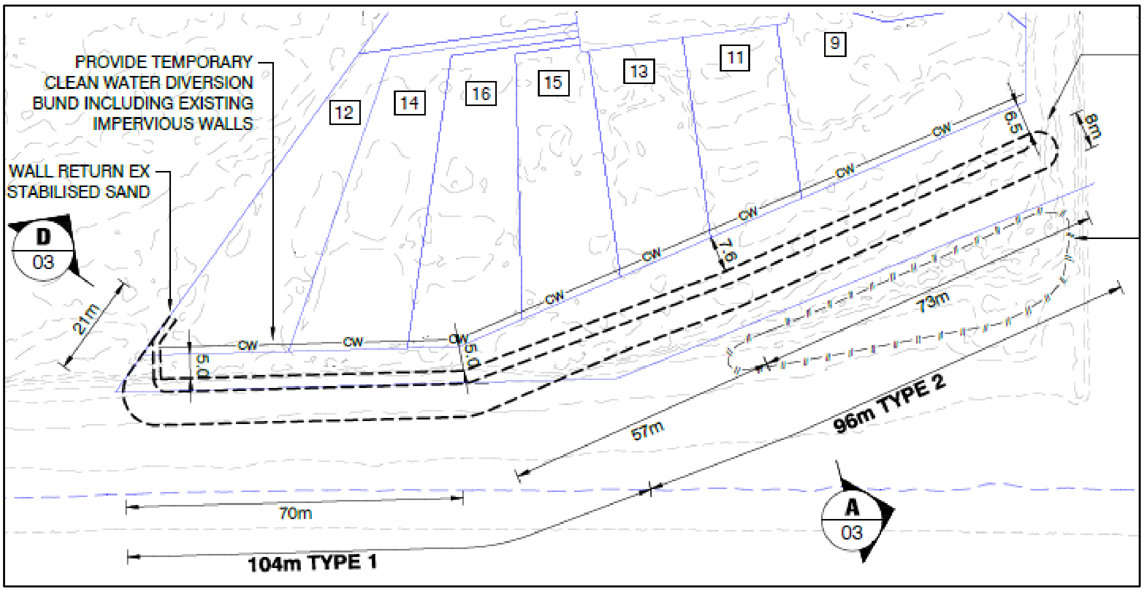


Figure 4.3: Structure Layout

4.4 Structure Ends

The northern end of the structure extends further landward and further from the seaward tip of the southern groyne at Three Mile Creek. Accordingly, the northern end of the proposed structure will be well protected from wave approach by greater width of vegetated dune.

The structure will be terminated with a sloped face similar to the front face. It will be separate from the groynes so that it does not affect them. The structure will be buried at this end, with vegetated dune re-instated to match the existing ground level.

At the southern end of the structure there is scope for a retreating beach to outflank the end of the structure. A return has therefore been designed on the structure, continuing approximately 20m landward. This could be extended in the future, if beach retreat threatens, but should be suitable for a minimum of 20 -30 years. Because there is limited room within the relevant property, and to avoid impacting the Island View Reserve land, a smaller footprint structure has been designed comprising Stabilised Sediment, instead of rock.

Stabilised Sediment is natural beach sand (sediment) mixed with cement. It forms a material similar to a natural sandstone and is also similar but not as strong as concrete. The material is used in thick mass structures rather than thin walls that might be used with concrete. The thick mass structured do not require reinforcement and allow for the variability in the material.

The return will be in the form of a large rectangular section with a steeper almost vertical face. Although this will be more reflective the more reflective face return will not be exposed to direct wave action and therefore not reflect energy.

4.5 Geotechnical Conditions and Foundation

The proposed wall is located near central to Waihi Beach, which is an open coast barrier beach comprised (per the Coastal Processes Assessment) of Holocene dune deposits of the Kariotahi Group. Sand has been reported (*Harray & Healy 2010*) to be fine to medium sand (0.15-0.3mm). Such dune deposits often have considerable depth. Observation of the excavations at the mouth of Three Mile Creek when these have been undertaken to clear sand built-up at the mouth indicated sand sediments extend to a depth far below expected scour level. The absence of any firm foundation layer upon which to support the wall structure is consistent with this observation and the wider morphology of the beach system.

Accordingly, the toe of the structure has been set at RL 0 which is at approximately mean sea level. This is well outside any scour depth at the current time. It is also near the practical limit of placing rock within a 4-5m high dune. Mean sea level is significantly offshore and below scour depth at the structure toe. As discussed below, as the beach retreats the toe is more at risk of exposure.

A structure of this nature is accommodating of settling movement of the foundation. When affected by rising sea levels (see Section 4.9), the riprap will tend to lower initially as the foundation becomes insufficiently supported. This would be a trigger to undertake remedial work rather than a failure criterion. Simple remedial work will be to place a scour blanket at the toe when required (Figure 4.5). The scour rock would be placed just below (3-400mm) sand level, and then tend to settle preferentially under wave action, to form a toe to the structure.

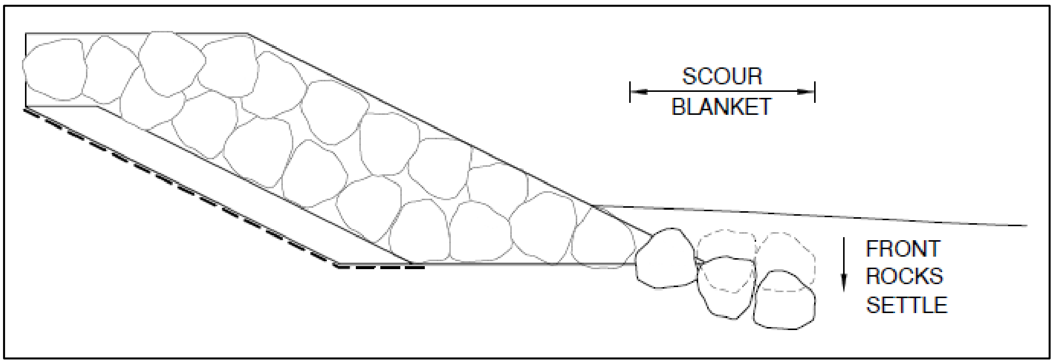


Figure 4.5: Structure Toe Scour Blanket

4.6 Structure Materials

The structure will be constructed out of Andesite sourced from a local quarry in Waihi. This material is understood to have been supplied for the majority of the work on the current Waihi Beach structures at Shaw Road and The Loop. The use of a local quarry will allow graded stockpiles to be developed so that the vast majority of rock can come to site in the correct size for immediate use. Minimal areas of the dune will need to be degraded by stockpiles and rock sorting.

4.7 Structure Gradient

The southern section of the structure will have a gradient of 1:2. The northern section is at a slight angle to the onshore wave and progressively further landward and less likely to be affected by waves. The structure has been detailed as a steeper 1:1.5 gradient.

4.8 Design Beach Profile

The beach profile in front of the site was surveyed for 6 years (2012-2018) on three profiles. The profile was averaged, and a composite average profile based on the average location of the 3, 2, 1, 0 and -0.5 contours and average slope was calculated (Figure 4.8). The slopes were sensitivity tested to ensure no significant errors arose out of the chosen profile. This slope was used to calculate depth limited waves for current wall position.

The slope of the same area of beach was taken from the 2011 LIDAR data, and used for transposing the beach landward to represent shoreline retreat due to sea level rise. The design profile was assumed to retreat uniformly back to the position indicated by the 2065 CEHZ line on the WBOPDC plan. The rate of sea level rise is increasing, suggesting the profile will retreat more slowly initially and increase in speed. The uniform retreat assumption is, therefore, conservative. Wave loads and rock sizing was checked at 10 yearly retreat intervals.

Clearly, the beach slope will be more variable and fluctuate more as it retreats. However, it is considered that this method portrays an average beach position and provides good guidance on the changing beach shape.

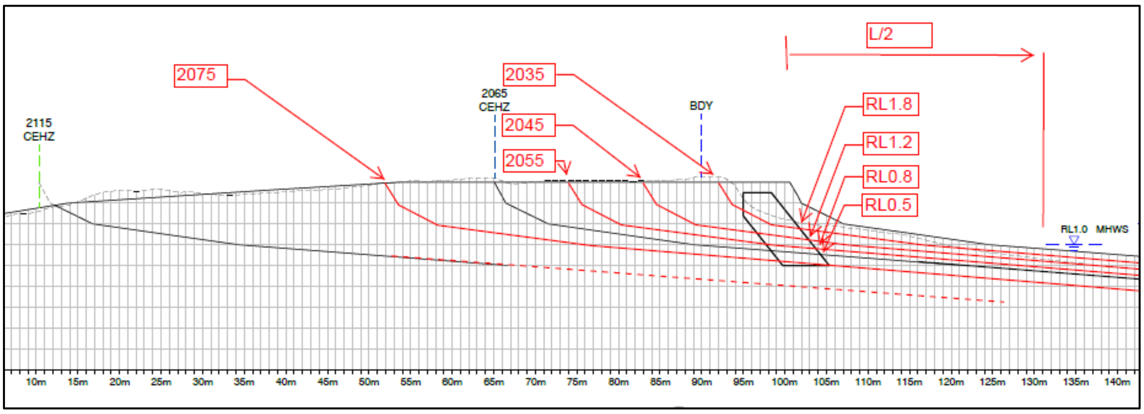


Figure 4.8: showing assumptions for foreshore lowering due to shoreline retreat

4.9 Future Sea-Level Rise Management

We have assessed the potential effects of sea-level rise over the next 100 years. Our full assessment is set out in the Coastal Processes Assessment.

The shore profile as defined by 2011 LIDAR was assumed to translate horizontally at a uniform rate between the current position and the adopted long term retreat positions. The position was plotted every 10 years and the actions on the wall derived for the resultant beach state. As the profile moves landward the beach in front of the structure will lower, leading to progressively larger wave heights and longer durations where the structure will be exposed to storm events.

A number of factors affecting Rock armour size will change over time.

Wave Height

Armour has been calculated based on incident depth limited waves. In assessing the result of the retreating beach lowering beach level and increasing water depth wave height will increase.

Currently the wall is not exposed to the design wave height and is unlikely to be in the next 10 – 20 years.

Storm Duration.

Rock size is a function of storm duration with a shorter storm requiring a smaller rock size and resulting in a slower development of damage as discussed below.

Wave height at site is depth limited and water depth at the wall has a large tidal component. Therefore, the peak wave load on the structure is short duration. More than 1.5 hours either side of high tide, water height is 0.5m lower than peak. This results in a reduced wave height such that only a 50-60% lighter rock is necessary. This short duration of peak storm waves limits overall damage propagation and allows repair if required.

Damage Development

Design of a Rock Armour wall which has varying sized rock around an "average" D_{n50} assumes a level of damage will occur in the design storm. This can be addressed by maintenance between storm events. Given design waves are associated with 1% AEP the amount of maintenance should be limited. When considering response of the wall over the next 50 – 100 years it is possible to consider higher but acceptable levels of damage in future decades. This avoids design for minimal damage in conditions the structure will not be subject to for a number of decades. This is particularly true when considered with the effects of overtopping set out below.

Overtopping Wave Load Reduction

With rising sea level greater over topping of the wall will occur. This can lead to a drop in required armour size on the wall of up to approximately 20%. This size difference equates to a drop in required armour rock mass of approximately 50% (CIRIA 5.2.2.4). Therefore, while wall damage will increase it is anticipated to stay within acceptable levels over the 30–40-year time frame.

Over the 40-50-year timeframe, the lowering foreshore level may threaten the foundation of the structure and remedial work may be required. This would comprise burying additional rock in front of the toe to act as a scour blanket during storm events (see Figure 4.5 above). It is not practicable or necessary to include this scour blanket now. It would be extremely difficult to place a rock blanket approximately 4-5 m below the beach face and it is not currently required. If required, it would not be for many decades. It is more sensible to place a scour blanket (or similar structure) in the future when the beach level will be lower and can be designed to suit the beach profile and wave climate.

If the beach progressively lowers as predicted between 50-100-years' time, the wall will need to have the foundation lowered, crest raised, and it will likely need a layer or layers of larger armour placed over the seawall. There is increasing uncertainty over the design actions extent of retreat and appropriate response with the increasing timeframe. It is anticipated that after 50 years of sea level change there will be a far greater understanding of the response of soft sediment coastlines to rapid sea level rise and of the likely future impacts of climate change.

The design of the structure allows it to be readily and progressively adapted to future requirements and this can be better addressed at that time.

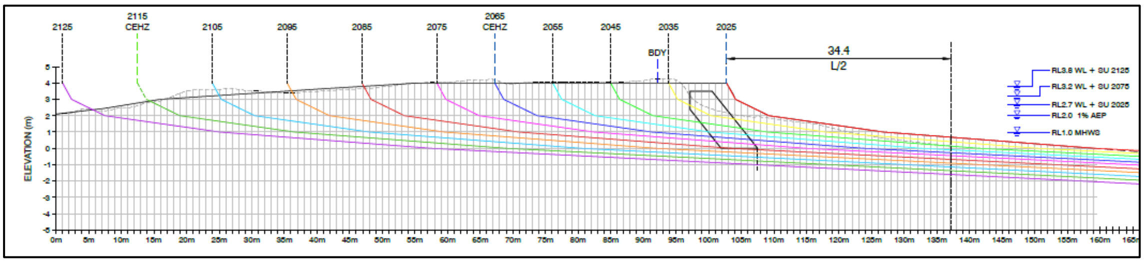


Figure 4.9: Beach retreat over 100 years

4.10 Dune Rehabilitation and Planting

Dune sand will be sourced from the footprint of the structure and used to cover the structure. The dune face will be placed at 1:3, on approximately the alignment of the dune face surveyed by LIDAR in 2011. This surveyed face was a storm cut dune and the reestablished dune should be slightly landward of a typical stable dune face. The dune face will be kept as landward as practicable.

Wind fencing will be established above and below the dune to limit windblown sand. During construction, the upper dune may be covered in coir matting or similar, to manage the windblown sand. Where used at the site, access through the fencing will be provided via an overlap, parallel to the shore so that there is no obvious path for wind to blow across the slope causing a blowout of the renourished dune.

Planting will be completed at the first spring or autumn after completion. Dune planting is a common activity in the Bay of Plenty and standard practices such as placing slow-release fertiliser and 0.7-1.0m spacing will be used. Vegetation will be native eco-sourced dune plants in accordance with a planting plan.

Over time, the dunes are likely to build seaward beyond the initial planted area. The seaward wind fence material will be removed following establishment of the foredune planting to allow this. The post and rope will be left in place to deter pedestrian traffic. This fence will only be removed within two years of construction, or earlier if it becomes storm damaged. Similarly, once the upper ground cover has established and the risk of windblown sand at the dune crest is managed, the landward fence can be removed and will be removed within two years of construction. Wind fencing may remain longer if management and ownership of the wind fencing was taken over by the WBOPDC as an aid to dune management. As discussed further below, limited retreat is anticipated in the 10-20 years.

As detailed further the Coastal Processes Assessment, large storms and beach retreat occurs, it is anticipated that the seaward toe of the vegetated dune will be progressively exposed to wave action and may be lost or significantly compromised. It is likely that in the 30 years plus timeframe, the dune will be lost. There is no proposal to manage this dune or reinstate it.

5.0 Design Parameters

The Coastal Processes Assessment provides a detailed description of the processes at the site. The adopted parameters for design of the wall are set out below (Table 5.0).

Parameter Type	Specific Parameter	Value
Astronomical Tide	Highest Astronomical Tide (HAT)	1.2 (MVD-53)
Storm Tide (without wave effects)	1% Annual Exceedance Probability	1.98 (MVD-53)
Offshore Wave Height	H_s	5.4m
Offshore Wave Period	T_p	11s
Wave set-up	η	0.7m
Sea-level Rise	30yrs	0.3m
Sea-level Rise	50yrs	0.6m
Sea-level Rise	100yrs	1.2m
Shoreline Retreat (per TnT, 2015)	Retreat to 2065	-18m
Shoreline Retreat (per TnT, 2015)	Retreat to 2115	-54m

Table 5.0: Summarised general parameters

5.1 Nearshore Wave Height

Wave height was calculated for a depth of one-half wavelength from the wall. Calculations were made using methods of van De Meer(1988b), Goda (2000) and Battjes and Stive (BaS) (1985) for comparison. The calculations of BaS were in close agreement with Goda, assuming a foreshore slope of m=0.02, and adopted (Table 5.1).

Summary Breaking Depth and Wave Height			
2024 Shoreline	**WL**	**d_b (T=11)**	**$Hb_{S\ Stive}$**
$WL_{current}$	2	1.1	0.9
WL_{50yr}	2.5	1.8	1.5
WL_{100yr}	3.1	2.6	2.1
$WL_{current}$ +Setup	2.7	2.1	1.7
WL_{50yr} +Set Up	3.2	2.7	2.2
WL_{100yr} + Set up	3.8	3.4	2.8
2065 Shoreline			
WL_{50yr}	2.5	3.3	2.7
WL_{100yr}	3.1	4	3.3
WL_{50yr} +Set Up	3.2	4.1	3.4
WL_{100yr} + Set up	3.8	4.8	4

Table 5.1: Breaking Wave Height example calculation

6.0 Engineering Design

6.1 Design of Armourstone

The wall has been designed and detailed in accordance with standard texts (CERC, CIRIA et al). The wall will comprise a toe at RL0.0m and a crest at RL3.5. The most exposed face to the wall will slope at 2 (horizontal) to 1 (vertical). The wall will have two layers of D_{n50}=1.1m (the average rock diameter) outer armour and two layers of D_{n50}=250mm filter layers on a geotextile (BIDIMA34 or similar). The crest will be comprised of at least three armour rocks.

Primary armour calculations were undertaken using the formulas of Hudson (CERC 1984), Van de Meer (1988b) and Van Gent et al (2004) (Figure 6.1). Permeability of the core had been set for an impermeable core for Van Gent. And P=0.1, the minimum permeability of Van de Meer. The Van de Meer and Van Gent formula have been developed specifically for shallow water conditions and as such do not require the consideration of Wave Setup (CIRIA 4.2.2.5).

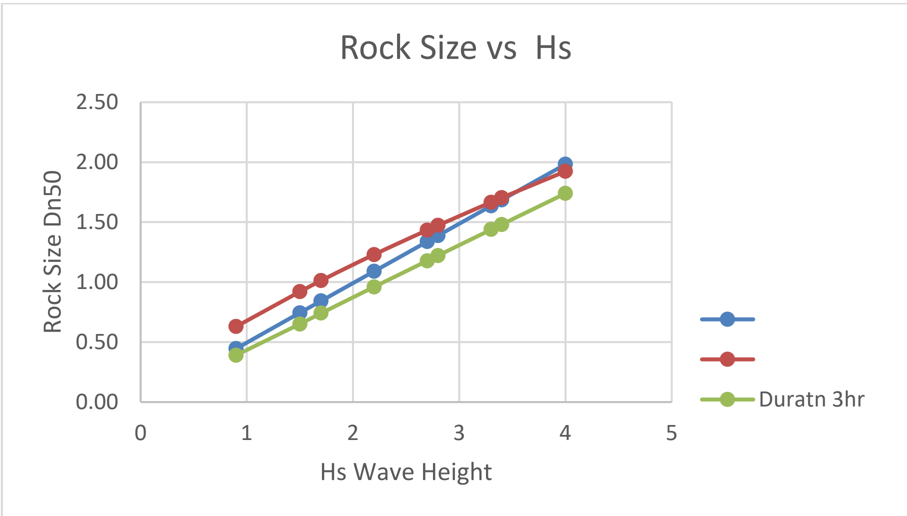


Figure 6.1: Comparison of Armour Rock Calculation

Relatively good correlation between the methods was obtained. Van Gent consistently produced the smallest rock size especially at lower wave heights. This appears to be because it takes account of the short storm duration. When a longer duration was included, it defined a rock size similar to Hudson (Table 6.1).

Hudson predictions are between the other formulas for the relevant range of rock size. Note the Hudson is not formulated for shallow water and needs consideration of wave set up.

Design was undertaken assuming wave set up and utilising Van Gent.

H_s	D_{n50}			
	Hudson	Van De Meer	Van Gent	*Van Gent*
			Duratn 3hr	*Duratn 6hr*
0.9	0.45	0.63	0.39	*0.43*
1.5	0.74	0.92	0.65	*0.72*
1.7	0.84	1.01	0.74	*0.84*
2.2	1.09	1.23	0.96	*1.06*
2.7	1.34	1.43	1.18	*1.30*
2.8	1.39	1.47	1.22	*1.35*
3.3	1.64	1.67	1.44	*1.59*
3.4	1.68	1.70	1.48	*1.64*
4	1.98	1.92	1.74	*1.92*

Table 6.1: Comparison of Rock Size and Storm Duration

The design water level used the 1%AEP plus wave set up was considered in rock sizing. Rock size was calculated by Van Gent to allow for the shorter storm duration and ability to progressively allow greater damage to the wall as sea level rises. The use of wave set-up with Van Gent is a conservative assumption but seemed to fit better in the context of the other two formulae.

6.2 Design of Cross Section

A design armour D_{n50} of 1.1m is adequate for 50 years with the potential for intermediate damage starting to occur near the end of that time, but this is driven by numerous factors including sea level rise, storm events, etc. With the proposed D_{n50} only the onset of intermediate damage (S_d = 3) occurs at the 50th year.

It is planned to use rock from a local quarry, and it is understood that 1.1- 1.2m rock is at the upper end of their manageable range. The design is for a D_{n50} 1.1m rock but the final specification may depend on rock availability.

The armour rock is not likely to be tested in the first 10-20 years, with only the upper wall exposed to swash run-up. With reference to Table 6.3a, the design armouring is adequate through to 40 years. Between 40 and 50 a slightly higher level of damage Level 3 would result from a 1%AEP wave event. Damage level 3 is between “start” and “minimal damage” and is considered a suitable level. After this period further work would need to be done to lower or protect foundation and additional larger armour rock may be needed.

The crest level is to be RL 3.5 to ensure the structure is buried beneath approximately 0.5m of sand, when recontoured to match the current dune profile, and the footing is to be to RL0.0 which is at mean sea level. The wall will generally have a face slope of 1:2 comprised of a double armour layer (D_{n50}=1100), with a three armour stone wide crest. Two layers of filter material will be provided at (D_{n50}Armour/4.5=) D_{n50}=250 over a BIDIM A44 Geotextile or similar (Figure 6.2).

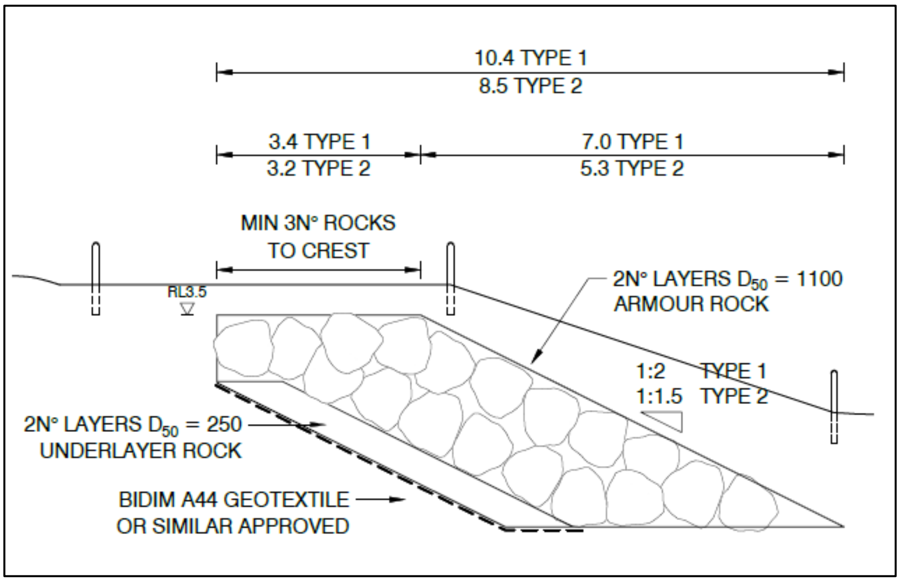


Figure 6.2: Typical section

The northern section of wall is at a slight angle to the onshore wave and progressively further landward and less likely to be affected by waves. The wall has been detailed as a steeper 1:1.5 gradient and as the 1.1m armour stone is only suitable for approximately a 300mm smaller wave. This lesser wave height is generally commensurate with the more landward position leading to a smaller wave height.

The location of the northern section of wall is close to important vegetation and limiting the structure footprint in this location helps manage any potential impacts on this vegetation.

Particular detailing care will need to be taken at the structure "elbow" (change in direction) and structure ends, particularly the southern end. At these points slightly larger armour may be required but will be dealt with in the detailed design exercise.

6.3 Allowance for Shoreline Retreat

Water depth was taken a half wavelength from the wall and is presented in the table below (Table 6.3a) for a constant wavelength. Half wave lengths varied from about 30-40m, but the shallow sloping foreshore means the changes of ground elevation and therefore water depth and depth limited wave are minimal. Damage parameters per CIRIA are included (Table 6.3b). No allowance has been made for the reduced load due to overtopping, suggesting the future performance (post 2085) are under predicted.

Year	Base of Structure GL	WL incl. setup	Bed Level @34m	Depth	Hb	S	Dn50
2025	4.00	2.7	0.68	2.02	1.7	2	0.73
2035	1.80	2.8	0.41	2.39	2.0	2	0.86
2045	1.22	2.9	0.13	2.77	2.3	2	1.00
2055	0.82	3.0	-0.1	3.09	2.6	2	1.11
2065	0.53	3.1	-0.27	3.37	2.8	2	1.12
2075	0.24	3.2	-0.45	3.65	3.0	3	1.09
2085	-0.08	3.3	-0.69	3.99	3.3	7	1.12
2095	-0.31	3.4	-0.9	4.33	3.6	8	1.18
2105	-0.54	3.5	-1.16	4.66	3.8	8	1.27
2115	-0.77	3.7	-1.4	5.07	4.2	8	1.38
2125	-0.80	3.8	-1.6	5.4	4.5	8	1.47

Wall at foundation level additional foundation required

Damage to structure excessive/ Wall failure - larger armour skin required

Table 6.3a Required armour size on retreating shoreline

Slope	Damage Level		
	Start of Damage	Intermediate Damage	Failure
1:2	2	4-6	8

Table 6.3b: Damage Level ex CIRIA Table 5.23

7.0 Exposure of the Seawall

The location of the southern end of the structure is such that over the period of monitoring records the upper layer (1-1.5m) would have only been exposed once over the last 30 years. This would have been during the recent Gabrielle and Hale set of storms. It is predicted at current sea level the wall is unlikely to be exposed more than once every 5-10 years. Such exposure would be the top layer of the wall which is quickly re buried by natural processes. Loads on the wall will be nominal and no damage will occur.

Beach retreat estimates, shown in Figure 4.9 above, provide some guidance as to the progressive exposure of the structure. Table 7.0 summarises the effect of anticipated beach retreat on the structure over the next 100 years.

Timeframe	Wall Exposure
0-10 years	Wall buried. 1-2 possible exposures of upper 1-1.5m during storm events greater than any that have occurred in the last 20 years. Structure quickly reburied by natural processes.
10-20 years	Top of 1m wall exposed for long periods. During storms upper 1.5-2.0m exposed. But all but upper 1m covers back up.
20-30 years	Upper 2-3m of face of wall exposed at all times. High tide may reach structure in large storm events.
30-40	MHWS at base of wall. Access in front may become limited for 1-2hr around high tide.
40-50	High tide at wall. Scouring during storm events may threaten toe and remedial works to toe may be required. Over topping may require raising wall crest.
50 -100	With beach retreat and sea level rise as predicted, significant upgrade may be required on the wall including increase in outer armour size which could be overlayed on the outer face. Foundation would need to be lowered or improved. Over topping may require raising wall crest.

Table 7.0 – Wall Exposure

Appendix A

Design Drawings – Buried Backstop Wall

COASTAL PROTECTION PROJECT

GLEN ISLA DUNE

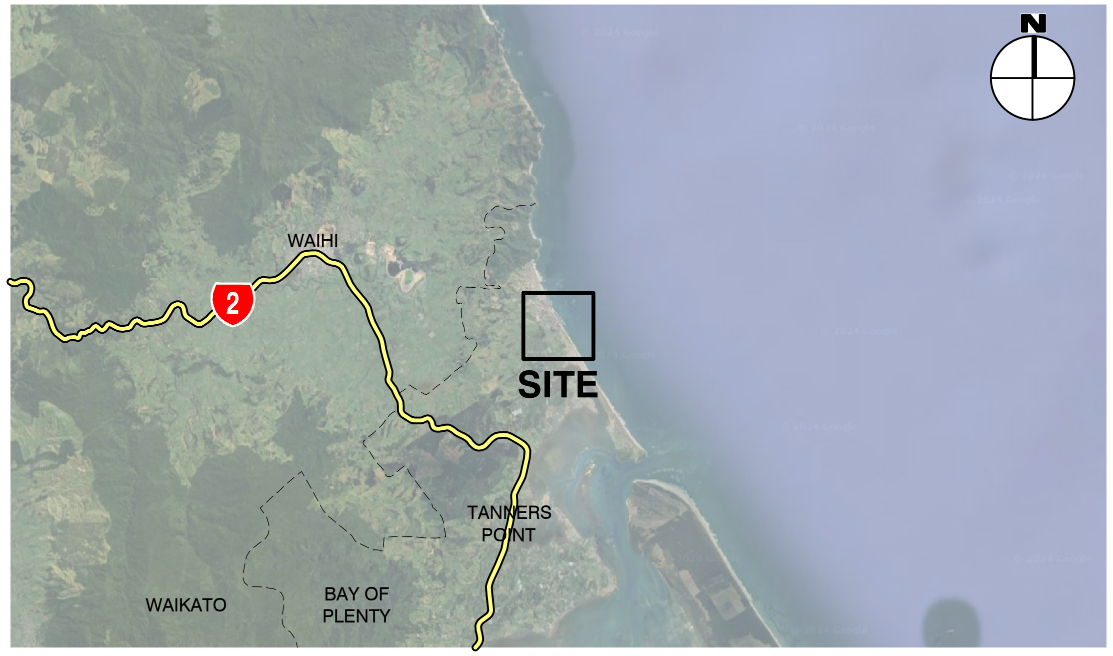
FOR

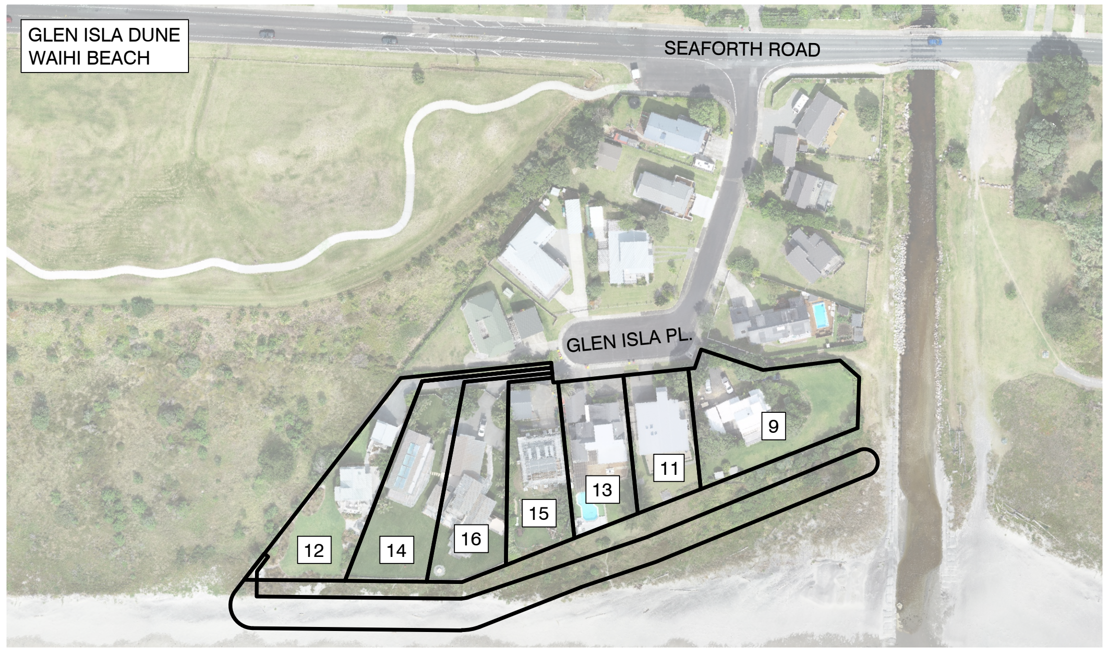
GLEN ISLA PROTECTION SOCIETY

PREPARED BY

DAVIS COASTAL CONSULTANTS

RESOURCE CONSENT





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NO	TITLE	REV	DATE
01	DRAWING SCHEDULE AND LOCATION PLAN	A	10.10.24
02	PROPOSED LAYOUT	-	23.08.24
03	PROPOSED SECTIONS	-	23.08.24

No.	REVISION DETAILS	DATE
A	RESOURCE CONSENT ISSUE	10.10.2024
-	RESOURCE CONSENT ISSUE	23.08.2024

DESIGN: DAVIS COASTAL CONSULTANTS
SURVEY: SEAM SPATIAL
DRAWN: JMA
CHECKED: -
DATE: OCTOBER 2024
SCALE NTS
CAD FILE: 23028-02 Glen Isla Place Waihi

NOT FOR CONSTRUCTION

JOB TITLE:

COASTAL PROTECTION PROJECT
GLEN ISLA DUNE
WAIHI BEACH

DAVIS COASTAL CONSULTANTS
COASTAL MANAGEMENT AND ENGINEERING
P.O. Box 185
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Email: coastal@daviscoastal.co.nz

DRAWING TITLE: DRAWING SCHEDULE AND LOCATION PLAN

FILE NO: 23028
SHT NO: 01
REV: A

SERIES: RESOURCE CONSENT

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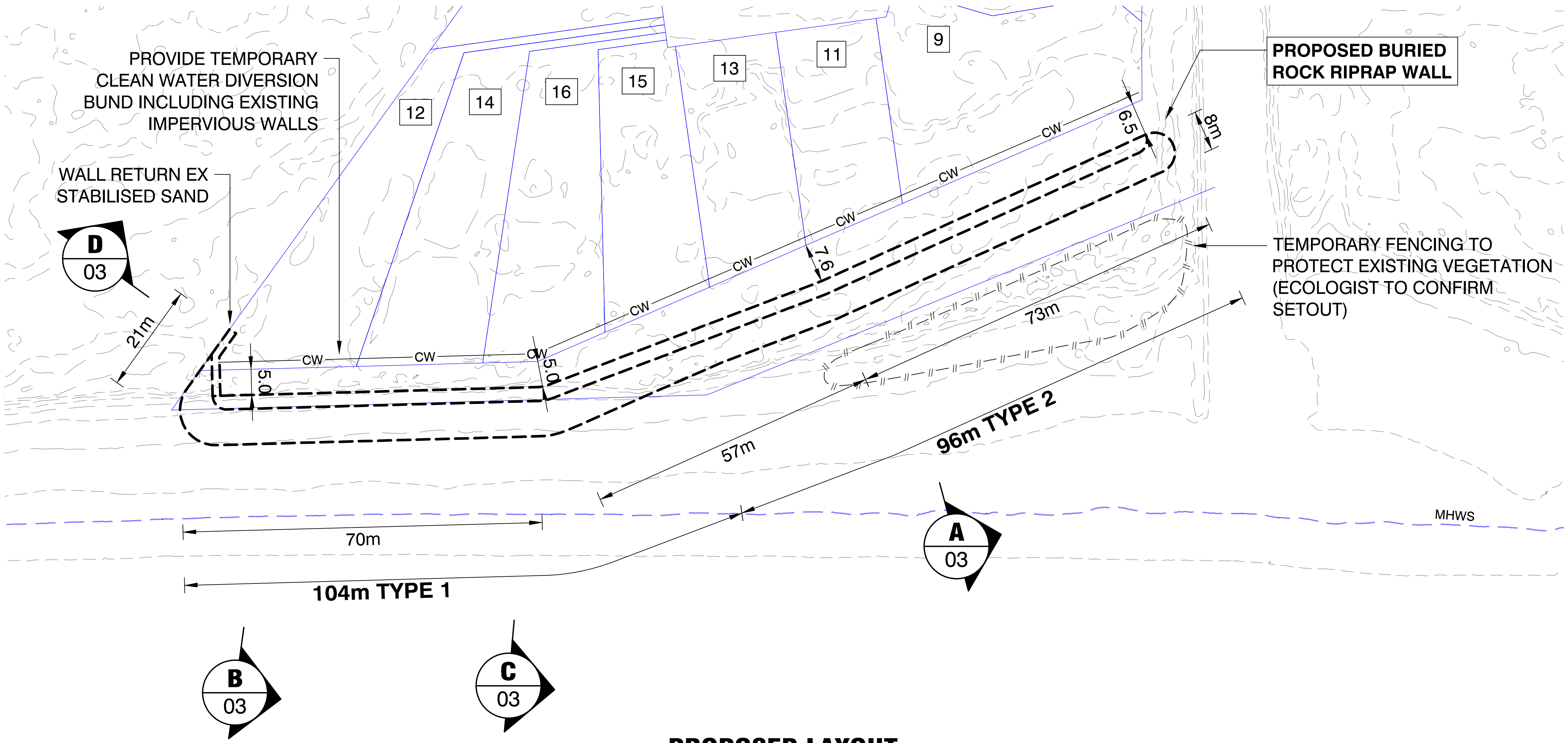

PROPOSED LAYOUT

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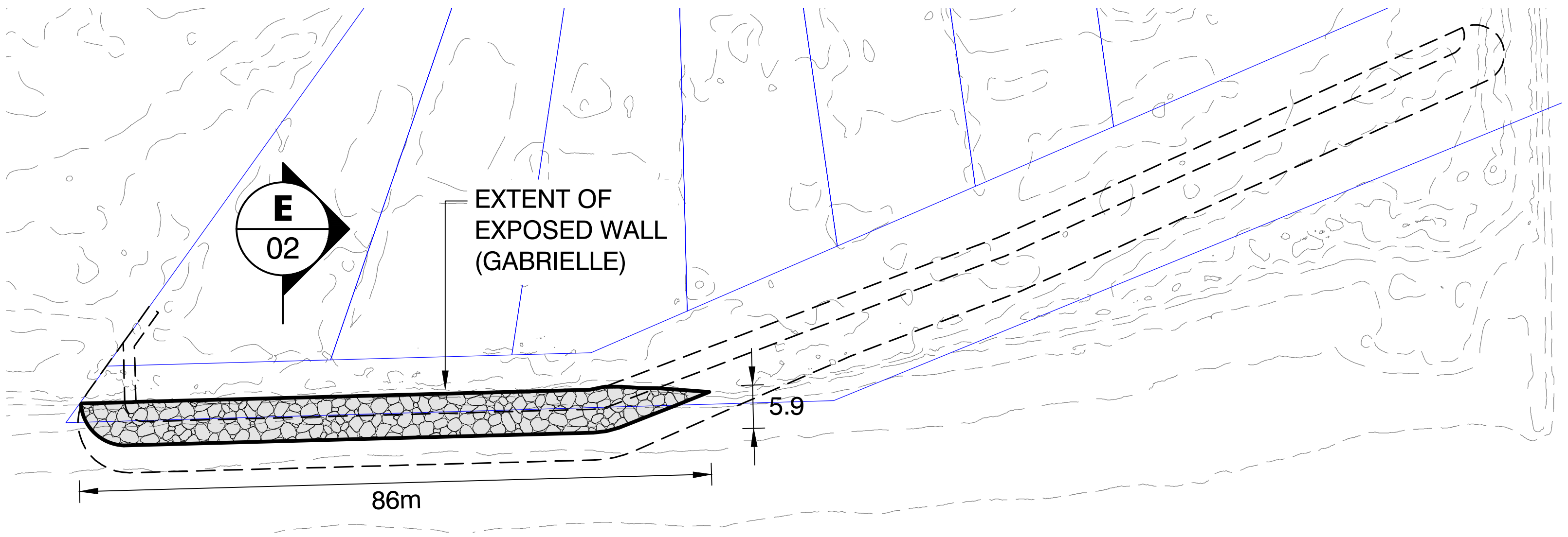
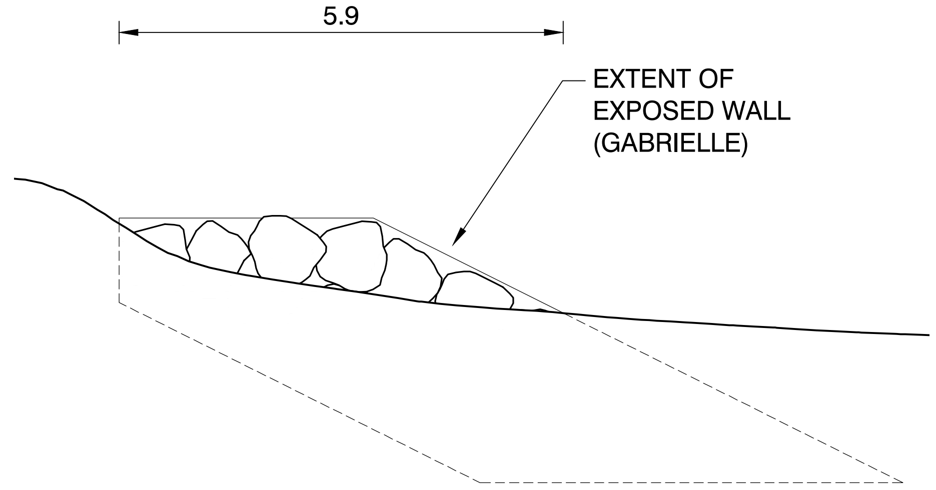


POTENTIAL EXPOSED WALL - 1% AEP STORM CONDITION

SCALE 1:1000



E 02 ELEVATION

SCALE 1:100

No.	REVISION DETAILS	DATE
-	RESOURCE CONSENT ISSUE	23.08.2024

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SURVEY: SEAM SPATIAL
DRAWN: JMA
CHECKED: -
DATE: AUGUST 2024
SCALE: VARIES @ A3
CAD FILE: 23028-02 Glen Isla Place Waihi

NOT FOR CONSTRUCTION

JOB TITLE:

COASTAL PROTECTION PROJECT
GLEN ISLA DUNE
WAIHI BEACH

DAVIS COASTAL CONSULTANTS
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DRAWING TITLE: PROPOSED LAYOUT
FILE NO: 23028
SHT NO: 02
REV: -
SERIES: RESOURCE CONSENT

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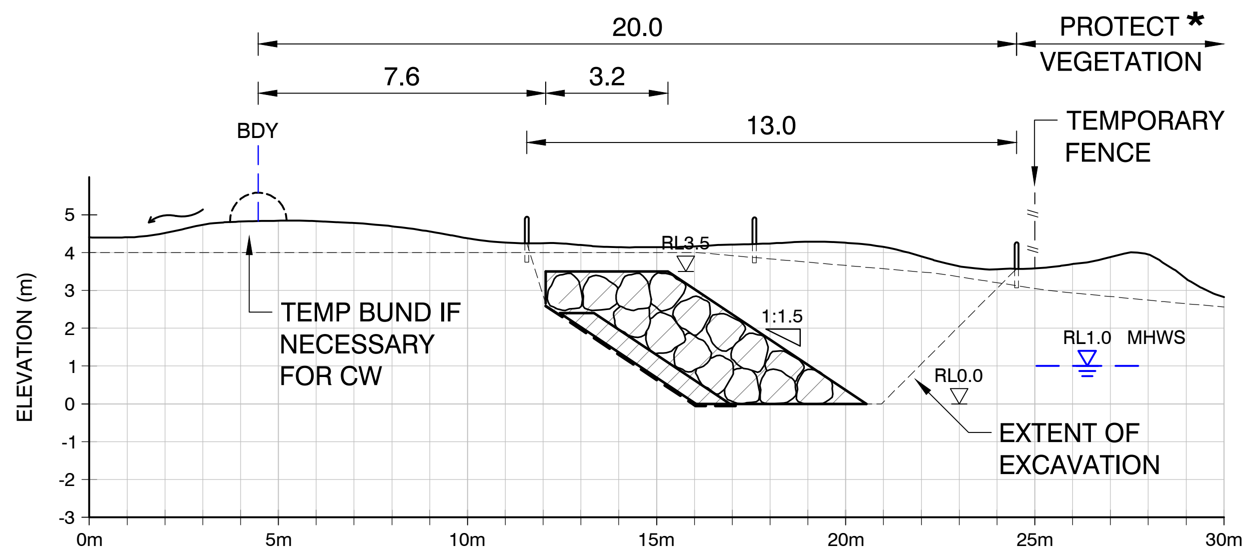

A 02 **No 11 GLEN ISLA - TYPE 2**
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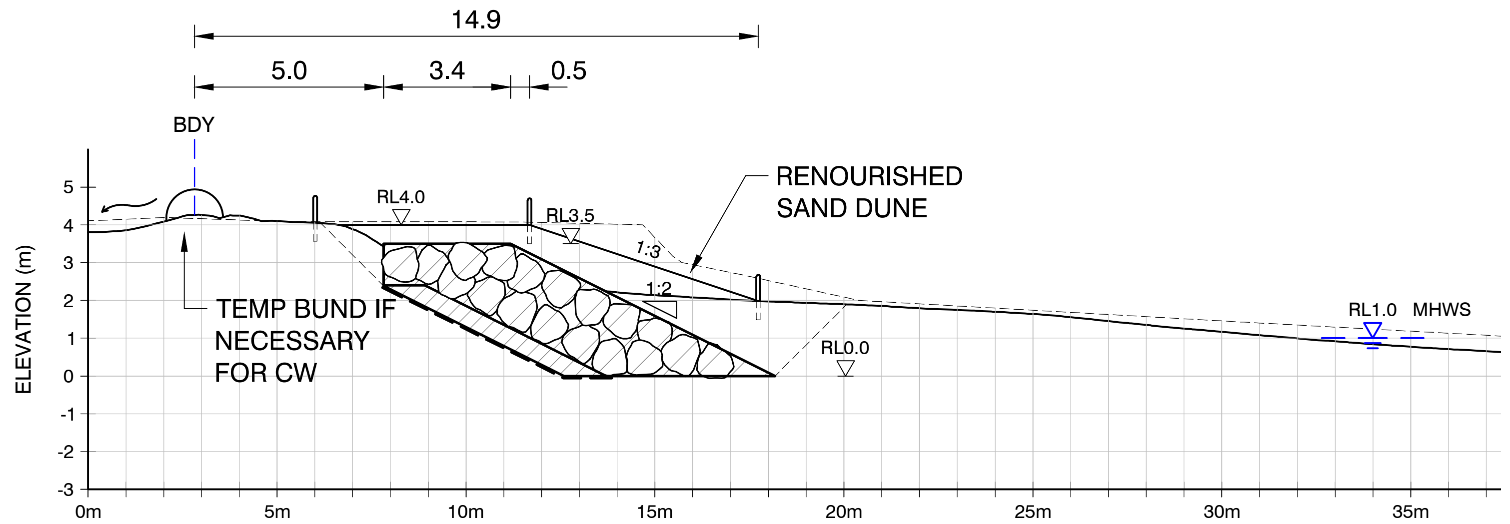

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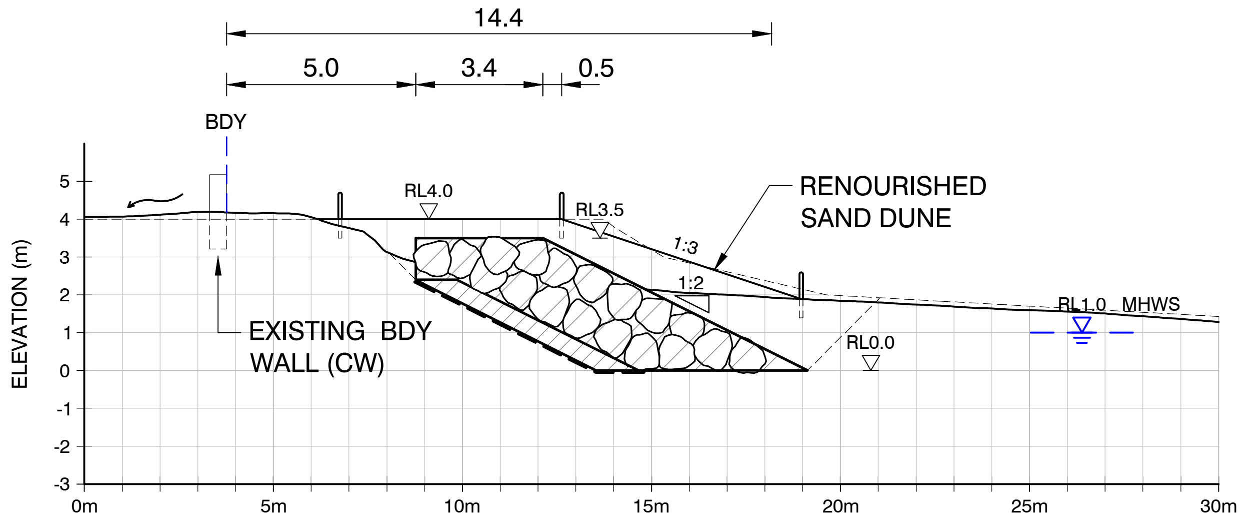

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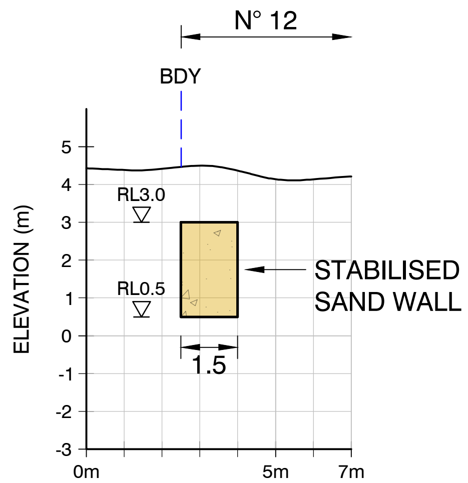

D 02 **SECTION**
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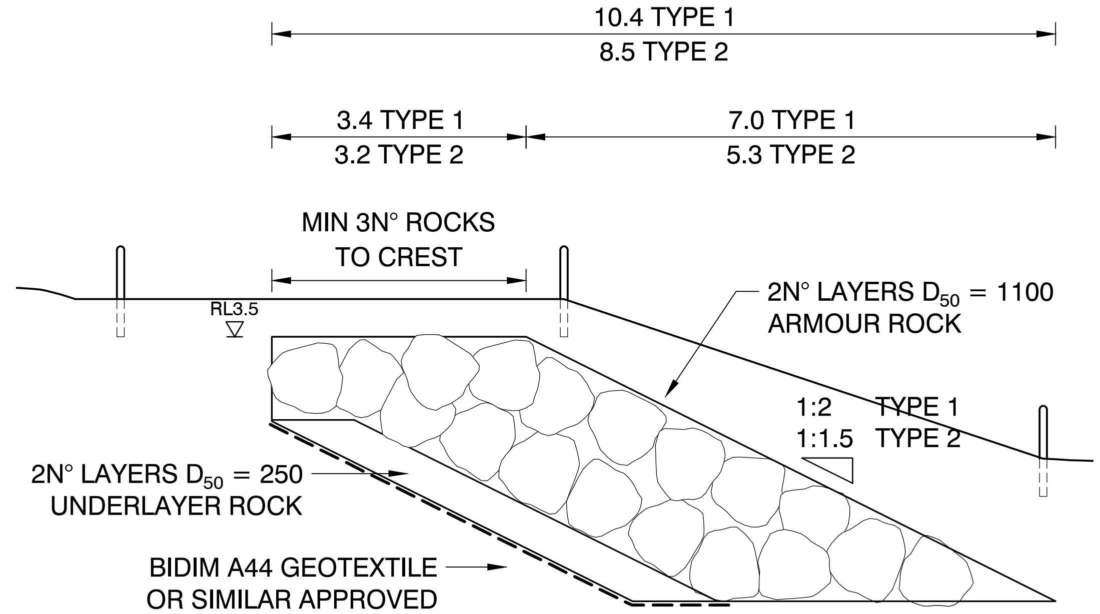

TYPICAL RIPRAP SECTION
SCALE 1:100

NOTES

* AREA OF EXISTING HIGH QUALITY DUNE VEGETATION TO BE DEFINED ON SITE BY ECOLOGIST AND PROTECTED BY TEMPORARY FENCE FOR DURATION OF WORKS

KEY

———	EXISTING SURFACE - 2024
- - - -	HISTORIC SURFACE - 2011

No.	REVISION DETAILS	DATE
-	RESOURCE CONSENT ISSUE	23.08.2024

DESIGN: DAVIS COASTAL CONSULTANTS
SURVEY: SEAM SPATIAL
DRAWN: JMA
CHECKED: -
DATE: AUGUST 2024
SCALE: VARIES @ A3
CAD FILE: 23028-02 Glen Isla Place Waihi

NOT FOR CONSTRUCTION

JOB TITLE:

COASTAL PROTECTION PROJECT
GLEN ISLA DUNE
WAIHI BEACH

DAVIS COASTAL CONSULTANTS

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DRAWING TITLE: PROPOSED SECTIONS

FILE NO: 23028
SHT NO: 03
REV: -
SERIES: RESOURCE CONSENT

Appendix B
Calculations

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ENGINEERING CALCULATIONS

BURIED RIPRAP WALL - Glen Isla Dune

		Job Ref:	23028
Revision History		Name	Date
Resource Consent	Rev -	Designed:Craig Davis	18/09/2024
		Approved: Craig Davis BE CPEng CMEngNZ	
		Date:	18/09/2024

Introduction

These Calculatiosn cover the design of a buried rock rip rap wall at Waihi Beach including consideration of a retreating coast line over 100 years

Deep water Wave Height

Ex Royal Haskonion Report

The tabular values for these offshore wave heights are presented below in **Table 2-1**. Peak wave period (T_P) was determined using the mean conditions associated with each wave height in the overall wave timeseries, binned by 1 m increments in the wave height.

Table 2-1: Joint-probability offshore significant wave height (H_{sig}) and associated peak wave period (T_p) at the east coast ERA5 output point for a 1% AEP water level

Direction	Joint-probability H_{sig} (m)	T_p (s)
OMNI	4.93	10.2
E	5.37	11.3
NE	4.28	9.9
N	4.56	9.7

Extreme Water Level

Ex NIWA 2019 Tauranga

	MVD-53	
Model Output Site	**1% AEP**	**2% AEP**
Site 95	2.12	1.91
Site 96 (adopted)	**1.98**	**1.78**
Moturiki Island (1997)	1.99	1.78

Sea Level Rise + Vertical Land Movement (30 – 100 years)			
Shared Socio-Economic Pathway	**30yrs (m)**	**50yrs (m)**	**100yrs (m)**
SSP5-8.5	0.34 (0.3)	0.55 (0.6)	1.21 (1.2)
SSP5-8.5 +VLM	0.29 (0.3)	0.49 (0.5)	1.11 (1.1)

Table 3.4: Sea-level rise and VLM to 2130 – Site 1712

Extreme Water level	$WL_{Present}$	2	RL
	WL_{50yr}	2.5	RL
	WL_{100yr}	3.1	RL

Use for Shallow Water Wave calculation for equations that already account for Wave set up (Van Gent etc)

Wave Set Up

Goda 2000 ex CIRIA 4.2.25

$\eta =$ 0.7

Extreme Water level +Set up

$WL_{Present}$	2.7 RL
WL_{50yr}	3.2 RL
WL_{100yr}	3.8 RL

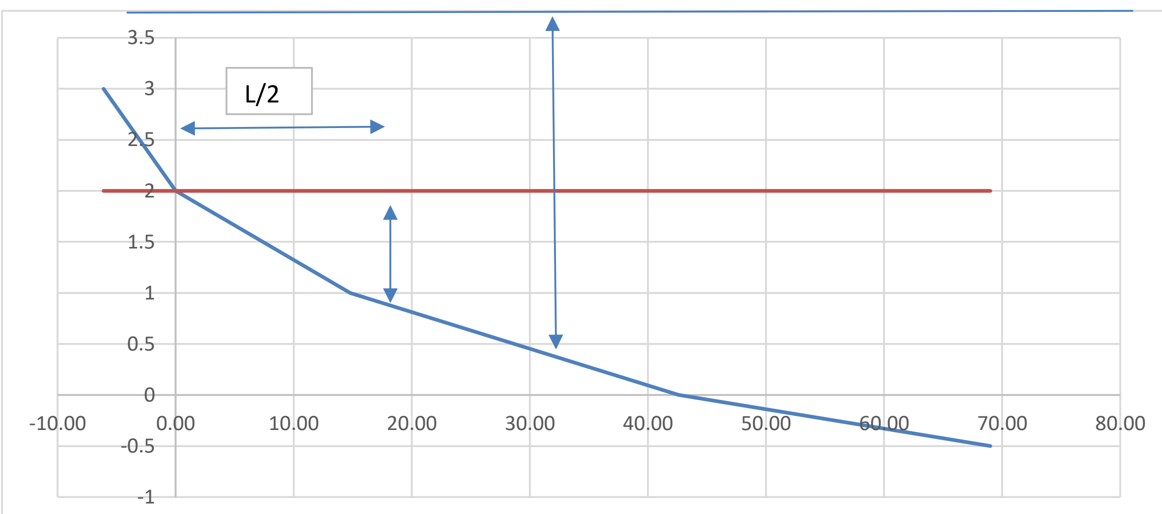
Use for Wave calculation for equations that do not already account for Wave set up (Hudson etc)

Assess Water Depth Shallow Water wave length

Present Day Beach Profile

Profile data S2 ex records

	x	RL	$x_{\text{base of bank}}$	WL
	8.40	**3**	-6.10	2
	14.50	2	0.00	2
	29.30	1	14.80	2
	57.10	0	42.60	2
26.40	83.50	-0.5	69.00	2



Waves will be depth limited check water depth at half wave length ex RL2
Shallow waves Period dependent iterate

$L = T(gd)^{0.5}$

$WL_{current} = \quad 2$

T	d_{trial}	$L_{shallow}$	$d_{S2\ at\ L/2}$
(s)	(m)	(m)	(m)
8	0.9	24	0.90
9	0.97	28	0.97
10	1.05	32	1.05
11	1.13	37	1.13

Van Gent

$Hs/\Delta D_{n50}$ = $1.75(\sqrt{\cot\alpha})(1+D_{n50core}/D_{n50})^{2/3}(S_d/\sqrt{N})^{0.2}$

H_s	=	2.2	m	
T_p	=	11	s	
$T_{m-1,0}$	=	10.0	s	
S_d	=	2		
Storm duration	=	3	hrs	Storm over high tide only
N	=	1080	Waves	
cotα	=	1.5		
$D_{n50core}$	=	0		
D_{n50}	=	0.9		
ρ_s		2.65	t/m^3	
ρw		1.028	t/m^3	
Δ=		1.622		
D_{n50}	=	1.11	m	

SUMMARY

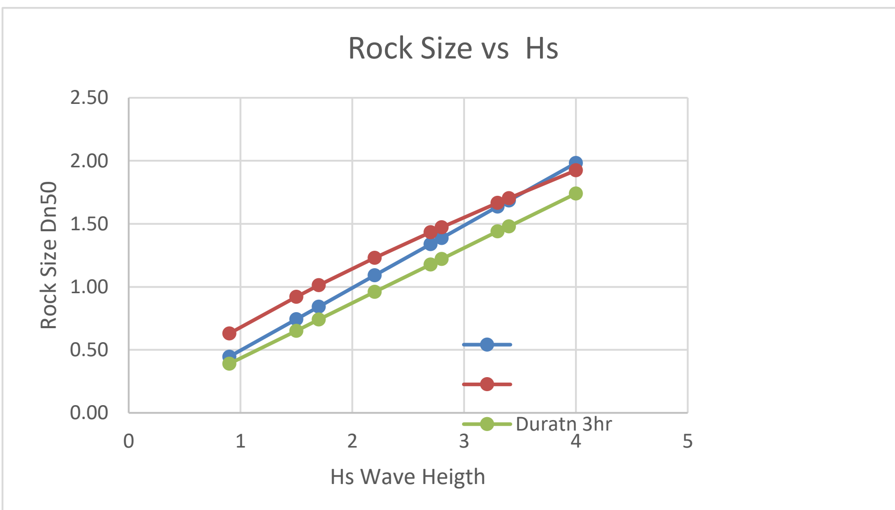
Wave Height	**D_{n50}**			
	Hudson	**deMeer**	**van Gent**	**Adopt**
2.2	1.2	1.36	1.06	1.1
2	1.09	1.27	0.97	1.1

Design Rock

D_{n50}	=	1.1	
M_{50}	=	3.5	t

2 No	Armour Layer	900-1200	mm		D_{n50} =	1100 mm
2 No	Underlayer	511	mm	Calculated as 1/10th of weight of armour layer		
		400 - 600	mm	SPM or CIRIA P =0.4		
2 No	Underlayer	200-300	mm	Calculated as Dn50 armour layer/4.5		
				CIRIA P=0.1		

H_s	D_{n50}			
	Hudson	Van De Meer	Van Gent	*Van Gent*
			Duratn 3hr	*Duratn 6hr*
0.9	0.45	0.63	0.39	*0.43*
1.5	0.74	0.92	0.65	*0.72*
1.7	0.84	1.01	0.74	*0.84*
2.2	1.09	1.23	0.96	*1.06*
2.7	1.34	1.43	1.18	*1.30*
2.8	1.39	1.47	1.22	*1.35*
3.3	1.64	1.67	1.44	*1.59*
3.4	1.68	1.70	1.48	*1.64*
4	1.98	1.92	1.74	*1.92*



WL_{50yr} 2.5

T	d	L	$d_{S2\ at\ L/2}$
(s)	(m)	(m)	(m)
8	1.53	31	1.53
9	1.62	36	1.62
10	1.71	41	1.71
11	1.81	47	1.81

WL_{100yr} 3.1

T	d	L	$d_{S2\ at\ L/2}$
(s)	(m)	(m)	(m)
8	2.25	38	2.25
9	2.35	44	2.35
10	2.46	50	2.46
11	2.57	56	2.57

$WL_{current}$ +Setup= 2.7

T	d_{trial}	$L_{shallow}$	$d_{S2\ at\ L/2}$
(s)	(m)	(m)	(m)
8	1.77	34	1.77
9	1.87	39	1.87
10	1.97	44	1.97
11	2.07	50	2.07

WL_{50yr} +Set Up 3.2

T	d	L	$d_{S2\ at\ L/2}$
(s)	(m)	(m)	(m)
8	2.37	39	2.37
9	2.47	45	2.47
10	2.58	51	2.58
11	2.69	57	2.69

WL_{100yr} + Set up 3.8

T	d	L	$d_{S2\ at\ L/2}$
(s)	(m)	(m)	(m)
8	3.06	44	3.06
9	3.18	51	3.18
10	3.3	57	3.30
11	3.42	64	3.42

Present Day Profile
T=11

WL	**$d_{S2\ at\ L/2}$**
2	1.13
2.5	1.81
2.7	2.07
3.1	2.57
3.2	2.69
3.8	3.42

Wave Set Up

Goda 2000CIRIA 4.2.25 pg329

Wave Height	H_0	5.4	m	1% AEP Storm ex RHDv
	K_r	1		
	K_d	1		
Wave Height	H'_0	5.4	m	
Wave Length	L_0	156	m	
	H'_0/L_0	0.035		
	m=tanα	0.04	1:25	
	η/H'0	0.12	ex Figure 4.13	

Wave Set Up	**η=**	**0.65**

CIRIA 4.2.2.5 Pg329

"If design formula to be used already includes this wave set up formulation, then this effect should not be added. This is the case with formulae that were derived for shallow water"

Glen Isla Armour Calculations

Job Ref 23028

Armour Calculation

H_s	=	2.2		

Hudson

H_s	=	2.2	m	
H_{10}=1.27Hs	=	2.794	m	
K_D	=	2		for breaking waves
cotα	=	2		
tanα	=	0.50		
ρ_s	=	2.65	t/m^3	
ρw	=	1.028	t/m^3	
Δ	=	1.622		
D_{n50}	=	1.09	m	
			1.00942	

Van De Meer

Hs/ΔDn50 = $c_{pl}P^{0.18}(S_d/\sqrt{N})^{0.2}(H_s/H_{2\%})(\xi_{s-1,0})^{-0.5}$ Plunging

= $c_sP^{-0.13}(S_d/\sqrt{N})^{0.2}(H_s/H_{2\%})\sqrt{\cot\alpha}(\xi_{s-1,0})^P$ Surging

H_s	=	2.2	m		
c_{pl}	=	8.4			
c_s	=	1.3			
T_p	=	11	s		
$T_{m-1,0}$	=	10.0	s		
P	=	0.1			
S_d	=	2			
Storm duration	=	3	hrs	Storm over high tide only	
N	=	1080	waves		
$H_{2\%}$/Hs	=	1.4			
$H_s/H_{2\%}$	=	0.7			
cotα	=	2			
tanα	=	0.50			
$L_{m-1,0}$	=	156	m		
$s_{m-1,0}$	=	0.014			
$\xi_{s-1,0}$	=	4.21		$\xi_{ms-1,0}$	$=\tan\alpha s_{m-1,0}{}^2$
$\xi_{s-1,0}$	=	4.21		$\xi_{ms-1,0}$	$=\tan\alpha/(2\pi H_s/(gT_{m-1,0}{}^2))^{0.5}$
ξ_{cr}	=	9.56			
ρ_s	=	2.65	t/m^3		
ρw	=	1.028	t/m^3		
Δ=	=	1.622			
	=				
D_{n50}	=	1.23	m	Plunging	
D_{n50}	=	0.62	m	Surging	

Van Gent

$Hs/\Delta D_{n50}$ = $1.75(\sqrt{\cot\alpha})(1+D_{n50core}/D_{n50})^{2/3}(S_d/\sqrt{N})^{0.2}$

H_s	=	2.2	m	
T_p	=	11	s	
$T_{m-1,0}$	=	10.0	s	
S_d	=	2		
Storm duration	=	3	hrs	Storm over high tide only
N	=	1080	Waves	
cotα	=	1.5		
$D_{n50core}$	=	0		
D_{n50}	=	0.9		
ρ_s		2.65	t/m^3	
ρw		1.028	t/m^3	
Δ=		1.622		
D_{n50}	=	1.11	m	

SUMMARY

Wave Height		**D_{n50}**		
	Hudson	**deMeer**	**van Gent**	**Adopt**
2.2	1.2	1.36	1.06	1.1
2	1.09	1.27	0.97	1.1

Design Rock

D_{n50}	=	1.1	
M_{50}	=	3.5	t

2 No	Armour Layer	900-1200	mm	D_{n50} =	1100	mm
2 No	Underlayer	511	mm	Calculated as 1/10th of weight of armour layer		
		400 - 600	mm	SPM or CIRIA P =0.4		
2 No	Underlayer	200-300	mm	Calculated as Dn50 armour layer/4.5		
				CIRIA P=0.1		

H_s	D_{n50}			
	Hudson	Van De Meer	Van Gent	*Van Gent*
			Duratn 3hr	*Duratn 6hr*
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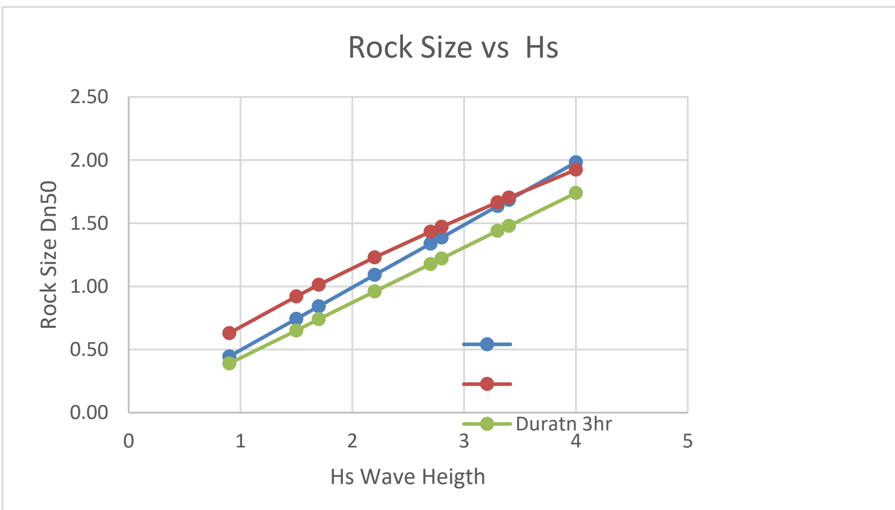
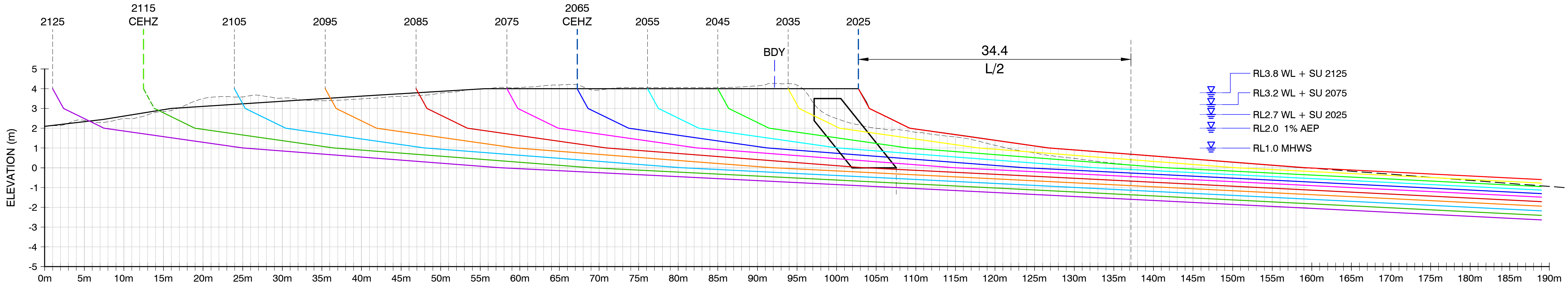
1.5	0.74	0.92	0.65	*0.72*
1.7	0.84	1.01	0.74	*0.84*
2.2	1.09	1.23	0.96	*1.06*
2.7	1.34	1.43	1.18	*1.30*
2.8	1.39	1.47	1.22	*1.35*
3.3	1.64	1.67	1.44	*1.59*
3.4	1.68	1.70	1.48	*1.64*
4	1.98	1.92	1.74	*1.92*





14 SECTION

SCALE (H) 1:500
SCALE (V) 1:200

KEY			
YEAR	LEVEL AT WALL	L/2	LEVEL AT L/2
2025	4.00		0.68
2035	1.80		0.41
2045	1.22		0.13
2055	0.82		-0.09
2065	0.53		-0.27
2075	0.24	34m	-0.45
2085	-0.08		-0.68
2095	-0.31		-0.91
2105	-0.54		-1.14
2115	-0.77		-1.37
2125	-1.00		-1.6

KEY	
- - - - -	EXISTING SURFACE - 2024
———	HISTORIC SURFACE - 2011
— — — (blue)	PROPOSED 2065 CEHZ
— — — (green)	PROPOSED 2115 CEHZ

Z:\Projects\23028 Glen Isla Place Waihi\Drawings\Dwg\23028-02 Resource Consent\23028-02 Glen Isla Place Waihi

DAVIS COASTAL CONSULTANTS

R02 100YR BEACH REGRESSION
23028-02 GLEN ISLA PLACE WAIHI
SCALE 1:500
DATE: 19.09.24
REV: -

Wave Height vs Armour Size over time

Year	Base of Wall GL	WL incl. setup	Bed Level	Depth	Hb	S	Dn50
2025	4.00	2.7	0.68	2.02	1.7	2	0.73
2035	1.80	2.8	0.41	2.39	2.0	2	0.86
2045	1.22	2.9	0.13	2.77	2.3	2	1.00
2055	0.82	3.0	-0.1	3.09	2.6	2	1.11
2065	0.53	3.1	-0.27	3.37	2.8	3	1.12
2075	0.24	3.2	-0.45	3.65	3.0	4	1.14
2085	-0.08	3.3	-0.69	3.99	3.3	7	1.12
2095	-0.31	3.4	-0.9	4.33	3.6	8	1.18
2105	-0.54	3.5	-1.16	4.66	3.8	8	1.27
2115	-0.77	3.7	-1.4	5.07	4.2	8	1.38
2125	-0.80	3.8	-1.6	5.4	4.5	8	1.47

GL at base < foundation level - additonal foundation required

Damage to wall excessive/ Wall failure larger armour skin required

Van Gent

$H_s/\Delta D_{n50}$ = $1.75(\sqrt{\cot\alpha})(1+D_{n50core}/D_{n50})^{2/3}(S_d/\sqrt{N})^{0.2}$

H_s	=	1.7	m
T_p	=	11	s
$T_{m-1,0}$	=	10.0	s
S_d	=	2	
Storm dura	=	3	hrs Storm over high tide only
N	=	1080	Waves
cotα	=	2	
$D_{n50core}$	=	0	
D_{n50}	=	0.8	
ρ_s		2.65	t/m^3
ρw		1.028	t/m^3
Δ=		1.622	
D_{n50}	**=**	**0.73**	**m**