Report

Paekakariki Coastal Edge - Geotechnical Interpretive Report

Prepared for Kapiti Coast District Council
Prepared by Beca Ltd (Beca)

23 December 2014
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<tr>
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Executive Summary

Beca Ltd (Beca) has been commissioned by Kapiti Coast District Council to assist in the development of potential options to replace the existing seawall, located along the beach adjacent to The Parade, Paekakariki. The scope of services undertaken by Beca is presented within the letter titled 'Paekakariki Coastal Protection – Ground Investigation Scope and Cost Estimate', dated 4 August 2014.

The purpose of this geotechnical interpretive report is set out below.

- Present a summary of the ground conditions, and geotechnical considerations for the site.
- Identify the geotechnical issues associated with the two options that have been taken forward from the stakeholder meeting.
- Provide a set of soil parameters and geotechnical effects to inform the preliminary design of both options.

The proposed replacement options considered are set out below. Both of these options were at concept design stage during preparation of this report.

- Option 1 – Timber Wall and Revetment
- Option 2 – Stepped Concrete Wall and Revetment.

The site contains a seawall approximately 960m long, extending parallel to The Parade, Paekakariki, from Sand Track in the south to 124 The Parade in the north. The Parade provides access to the beach, local amenities and coastal properties.

The soil profile confirmed through the geotechnical investigation typically comprises a variable thickness of fill and loose sands, overlying a thick unit of medium dense sands. Dense to very dense sands were proven, although they lie below the anticipated founding depth of the retaining wall piles.

There are no geotechnical considerations that clearly differentiate Option 1 and Option 2.

The key design considerations include scour, slope stability and seismic effects. It is suggested that an Importance level of IL1 is adopted for the seismic design, which corresponds to the Importance Level normally adopted for structures presenting a low degree of hazard to life and other property. We have assumed a design life of 50 years.

The wall is expected to exhibit adequate global stability under static long term and scour conditions, although it will rely on support from the deadman anchor (Option 1) or two rows of piles (Option 2).

Based on the geotechnical data available, it is likely that the medium dense sands will liquefy in events exceeding a 1 in 100 year return period, however the lateral extent of this liquefaction is not certain. Option 1 and 2 may not achieve adequate global stability under a 1 in 100 year return period earthquake (Ultimate Limit State) if extensive liquefaction occurs, resulting in potentially considerable damage to the wall and path behind. A supplementary investigation and analysis is required to better define the likely lateral extent of liquefaction.

Timber piles are expected to be the most economic foundation solution for both options, when compared to other alternatives such as steel driven or bored cast in situ piles. The piles should be founded within the medium dense sands, nominally at a depth of around 8m below the level of the beach. There is an opportunity to refine the pile length during the preliminary design stage.

The key considerations for construction include temporary works to provide a stable and safe working environment, earthworks testing on the engineered fill and a limited number of construction stage proof bores to confirm the soil profile and founding conditions to the piles.
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1 Introduction

Beca Ltd (Beca) has been commissioned by Kapiti Coast District Council to assist in the development of potential options to replace the existing seawall, located along the beach adjacent to The Parade, Paekakariki. The scope of services undertaken by Beca is presented within the letter titled ‘Paekakariki Coastal Protection – Ground Investigation Scope and Cost Estimate’, dated 4 August 2014.

The existing seawall was constructed circa 1980 and is typically in poor condition. Beca initially developed five options, which were presented in the report titled ‘Options for Community Consultation’, (Beca, January 2014). Beca undertook a site walkover and then prepared a ‘Geotechnical Desktop Study Report’; dated 5 May 2014. Beca and Council then held a consultation workshop with a variety of stakeholders from the local community. The outcome from this workshop was two preferred options, namely Option 1 – Timber Wall and Revetment’ and Option 2 – Stepped Concrete Wall and Revetment’, herein referred to as Option 1 and Option 2. It is likely that both options will be developed for detailed design, with a view that options will be selected to match the surrounding environment and site conditions.

The purpose of this geotechnical interpretive report is as follows:

- Present a summary of the ground conditions, and geotechnical considerations for the site.
- Identify the geotechnical issues associated with the two options that have been taken forward from the stakeholder meeting.
- Provide a set of soil parameters and geotechnical effects to inform the preliminary design of both options.

This report should be read in conjunction with the ‘KCDC Paekakariki Coastal Edge Geotechnical Factual Report’, dated 7 November 2014 (Final).

2 Proposed Development

The proposed development comprises demolition of the existing seawall and construction of an entirely new seawall. The new seawall will be approximately 960m long and extend along the alignment of the existing wall, adjacent to The Parade, Paekakariki. Preliminary sketches illustrating the general arrangement of each option are presented in Appendix A.

A brief description of each option is presented below. Both of these options were at concept design stage during preparation of this report.

- Option 1 – a timber wall comprising timber piles and lagging, supported by a deadman anchor system, with a 2.5m wide footpath immediately behind the wall and a rock revetment then extending up to The Parade.
- Option 2 – reinforced concrete steps supported on two rows of timber piles, with a 2.5m wide footpath immediately behind the wall and a rock revetment then extending up to The Parade.
3 Site Description

The site contains a seawall approximately 960m long, extending parallel to The Parade, Paekakariki, from Sand Track in the south to 124 The Parade in the north. The carriageway of The Parade road is approximately 1.5m-4.0m higher than the beach below. The Parade provides access to the beach, local amenities and coastal properties. Figure 1 below provides an illustration of the beach environment, the existing wall and The Parade and dwellings behind.

![Figure 1 – Typical Photo of the Site](image1)

A major storm event occurred the late 1970’s which resulted in considerable damage immediately below The Parade. The timber retaining wall was constructed circa 1980 to provide protection to The Parade. In later years further protection was provided, in the form of rock revetment, along sections of the wall. Figure 2 below presents historic photos of the timber wall during construction.

![Figure 2 – The Timber Wall During Construction](image2)
4 Published Geology

The published geological map (Begg and Johnston, 2000) indicates that the site is underlain by geologically young (<15,000 years) Holocene Aeolian sand dunes, comprising very loose to medium dense fine to medium sand. This unit is underlain by a deep deposit of older (>15,000 years) Pleistocene soils, comprising gravels and minor sands and silts. The depth of bedrock at the site is not clearly defined in the published geology.

The published geological map (Begg and Johnston, 2000) indicates the nearest active fault is the Ohariu Fault, which lies in the order of 1.5km to the east of the site. There is a fault mapped as 'approximate' splaying from the Ohariu Fault, which passes through the site, however, this fault is not mapped as active.

The risk of fault rupture, and the abrupt surficial movement at the site, is considered to be very low.

5 Geotechnical Investigation

The geotechnical investigations were undertaken by Griffiths Drilling (NZ) Ltd, on 22 September 2014 to 25 September 2014. All material was logged by a Beca engineering geologist. Geotechnical investigations comprised the following:

- Three boreholes reaching depths of 21.50m, 12.23m and 18.35m. The boreholes were carried out from The Parade.
- Two test pits reaching depths of 2.8m and 3.1m. The test pits were carried out from immediately behind the existing wall.

Refer to the site plan in Appendix B for investigation locations.

Laboratory grading tests were undertaken on four samples obtained from the near surface beach deposits. The testing was undertaken in order to inform the assessment of scour during a storm event. The results of the grading tests are summarised in Section 6.3 below.

A complete summary of the geotechnical investigations, including the logs, photographs and laboratory test data are presented within the Geotechnical Factual Report (Beca 2014).

6 Ground Conditions

6.1 Soil Profile

The published geology indicates the site is underlain by Holocene Aeolian sand dunes. The exploratory holes generally confirmed the published geology, encountering sand over the full depth investigated. Moderately thick to thick (400 to 700mm thick) layers of organics (wood) and organic/shelly sand were also encountered typically at depths of 2m to 4m depth below The Parade. The soil profile is summarised in Table 1 below.
A geological cross section extending along The Parade is presented in Appendix C. Table 1 – Soil Profile

<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>Description</th>
<th>Depth to top of layer (m bgl)</th>
<th>Elevation of top of layer (m RL)</th>
<th>Thickness (m)</th>
<th>SPT ‘N’ (blows/300mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retaining Wall Backfill</td>
<td>Asphalt and fine to coarse GRAVEL, some medium to coarse sand.</td>
<td>0.0</td>
<td>4 - 5</td>
<td>0.35</td>
<td>N/A</td>
</tr>
<tr>
<td>Loose sand</td>
<td>Fine to medium SAND, trace fine gravel.</td>
<td>0.35</td>
<td>3.65 – 4.65</td>
<td>1.5 – 4.0</td>
<td>2 – 6</td>
</tr>
<tr>
<td>Medium dense sand (A)</td>
<td>Fine to medium SAND, minor shell fragments, trace fine gravel</td>
<td>1.5 – 3.6</td>
<td>2.5 – 0.4</td>
<td>2.5 – 4.5</td>
<td>7 – 16</td>
</tr>
<tr>
<td>Medium dense sand (B)</td>
<td>Fine to medium SAND, trace shell fragments, trace fine gravel</td>
<td>7.4 – 8.9</td>
<td>-3.4 – -4.9</td>
<td>6.0 – 8.5</td>
<td>22 – 29</td>
</tr>
<tr>
<td>Dense to very dense sand</td>
<td>Fine to medium SAND</td>
<td>11.9 – 18.1</td>
<td>-7.9 – -14.1</td>
<td>1.5+</td>
<td>40 – 50+</td>
</tr>
</tbody>
</table>

Note 1 – bgl, below ground level

Note 2 – Reduced level has been estimated from Google Earth™, adopting the World Geodetic System 84 (WGS84) datum, and are approximate only.

6.2 Groundwater

Water levels were measured the day after the boreholes were completed, within a collapsed hole (no casing). Only BH03 contained groundwater, which was 2.9 m bgl.

Water levels were measured within the test pits as seepages during the excavation. Seepage within the test pits was slow and the groundwater level was recorded at 2.7 m bgl and 3.1 m bgl within TP1 and TP2 respectively.

For design purposes, it is recommended that ground water is modelled at beach level, corresponding to a high tide. For the seismic case the groundwater level should be taken at 1 m below beach level.

6.3 Laboratory Testing

Disturbed samples were collected from machine boreholes and bulk samples were obtained from the test pits and at the beach front. Geotest Ltd carried out testing of these samples. The tests undertaken, and the testing specifications, were as follows:

- Natural Moisture Content: NZS4402, 1986; test 2.1
- Wash Grading: NZS4402, 1986; test 2.8.1 (wet sieve).

Although the moisture contents were reported, they are not considered to be representative of the insitu soil, which will have highly variable moisture contents depending on the tidal range.
The results of the grading tests are summarised below:

- Beach sample 01 - 94% passing 2mm sieve, 0% passing the 63µm sieve
- Beach sample 02 – 82% passing the 2mm sieve, 0% passing the 63µm sieve
- BH01 4.85m-5.4m - 98% passing the 2mm sieve, 7% passing the 63µm sieve
- BH02 4.79m -5.74m - 98% passing the 2mm sieve, 15% passing the 63µm sieve.

These results confirm that the beach sands near the ground surface comprise non-plastic uniform sands with low fines content.

7 Soil Parameters

A set of soil parameters were derived from published correlations, the laboratory testing data and previous experience with similar materials encountered within the Kapiti Region. The soil parameters are summarised in Table 2 below.

<table>
<thead>
<tr>
<th>Geological unit</th>
<th>Unit Weight (KN/m³)</th>
<th>Friction Angle, Ø (°)</th>
<th>Cohesion, c¹ (kPa)</th>
<th>Youngs Modulus (MPa)</th>
<th>K²</th>
<th>Kp²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose Sands</td>
<td>17</td>
<td>28</td>
<td>0</td>
<td>10</td>
<td>0.37</td>
<td>2.8</td>
</tr>
<tr>
<td>Medium Dense Sands (Unit A)</td>
<td>18</td>
<td>32</td>
<td>0</td>
<td>10 +2.5z¹ (to a maximum of 40)</td>
<td>0.31</td>
<td>3.3</td>
</tr>
<tr>
<td>Medium Dense Sands (Unit B)</td>
<td>18</td>
<td>34</td>
<td>0</td>
<td>40</td>
<td>0.28</td>
<td>3.6</td>
</tr>
<tr>
<td>Existing Retaining Wall Backfill</td>
<td>18</td>
<td>36</td>
<td>0</td>
<td>30</td>
<td>0.25</td>
<td>3.8</td>
</tr>
<tr>
<td>Proposed Engineered Fill</td>
<td>18</td>
<td>36</td>
<td>0</td>
<td>40</td>
<td>0.25</td>
<td>3.8</td>
</tr>
<tr>
<td>Dense to Very Dense Sands</td>
<td>19</td>
<td>38</td>
<td>0</td>
<td>100</td>
<td>0.24</td>
<td>4.2</td>
</tr>
<tr>
<td>Proposed Rock Rip-Rap</td>
<td>20</td>
<td>40</td>
<td>0</td>
<td>100</td>
<td>0.21</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Note 1 – z = depth below top of layer.

Note 2 – These values assume an interface friction angle of zero appropriate for a king post wall (refer to Section 9.2).
8 Design Considerations

The geotechnical methodology for assessing some of the key design considerations is set out below. Further commentary is provided in Section 9.

8.1 Scour

The retaining wall is positioned within a very active coastal environment. Through our discussions with Council and local stakeholders there is anecdotal and pictorial evidence of a number of storm events in recent decades. In particular, the storm event in the late 1970’s caused widespread damage along The Parade, and lead to the construction of the existing timber pole retaining wall. Remedial works in the form of the rock revetment were implemented following the storm events.

A qualitative assessment of scour was undertaken during the options selection process. The slope stability assessments have allowed for a 2.5m deep zone of scour across the full length of the retaining wall. Refer to Section 8.2 and Section 9 below for further commentary.

The design team will undertake a quantitative assessment of scour during the preliminary design of Option 1 and Option 2. This will include an assessment of regional scour along the full length of the wall, and any localised effects at the location of discrete piles. The effects of scour are not expected to be a differentiator between Option 1 and Option 2.

Four laboratory grading tests were undertaken immediately after the ground investigation. The results from this data will inform the design team’s assessment of scour.

8.2 Slope Stability

The site is typically gently sloping, with localised steep and vertical slopes associated with the existing retaining wall. The overall height difference of 3m between The Parade and the beach will not be altered.

A series of slope stability assessments have been undertaken using GEO-SLOPE ‘Slope/W’ software, adopting the limit equilibrium method of Morgenstern and Price (1965) and also Bishop (1955).

In slope stability studies, the Factor of Safety (FoS) is the ratio of resisting forces (shear strength of soil) to the disturbing forces (weight of soil and surcharge load). A Factor of Safety of unity (FoS=1.0) indicates that disturbing forces are equal to resisting forces and that the soil mass is just on the point of moving (i.e. becoming unstable). Seismic forces are considered pseudo-statically i.e. as a continuous force as a function of the slip body mass.

The probabilities of failure associated with various factors of safety are summarised in Table 3 below:
### Table 3 FoS vs Annual Probability of Slope Failure (after Silva, et al, 2008)

<table>
<thead>
<tr>
<th>Factor of Safety (FoS)</th>
<th>Annual Probability of Slope Failure*</th>
<th>Category II**</th>
<th>Category III***</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1/5</td>
<td>1/2</td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>1/20</td>
<td>1/4</td>
<td></td>
</tr>
<tr>
<td>1.3</td>
<td>1/200</td>
<td>1/20</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>1/10,000</td>
<td>1/100</td>
<td></td>
</tr>
</tbody>
</table>

*Good NZ practice typically lies somewhere between Categories II and III in this table
**Category II Facilities designed and built to good engineering practice
***Category III Facilities designed and built without site specific design and good control over construction

A FoS of 1.5 is normally used for permanent civil engineering works in New Zealand. With a high level of design and control during construction, a lower FoS may be used, provided the consequences of failure are minor. A lesser FoS of 1.2 to 1.3 may be used for temporary conditions (i.e. a few weeks to a few months), while a FoS of 1.0 to 1.1 is only considered appropriate for earthquake loading.

### 8.3 Seismic Effects

A summary of the seismic effects for consideration in the design of each option are set out below.

#### 8.3.1 Seismic Design Criteria

The seismic design criteria have been developed in accordance with AS/NZS1170.0: 2002 and NZS1170.5: 2004.

The consequences of failure in terms of life safety for the proposed seawall are low. Therefore, it is suggested that an Importance level of IL1 is adopted for the design, which corresponds to the Importance Level normally adopted for structures presenting a low degree of hazard to life and other property. We have assumed a design life of 50 years.

The Ultimate Limit State (ULS) defines the level of performance expected under an extreme event that has a small chance of occurring within the design life of the structure. The Serviceability Limit State (SLS) defines the level of performance expected under an event that could be reasonably expected to occur during the design life of the structure. The Standard does not apply SLS criteria to IL1 structures.

A summary of the derivation of the peak ground accelerations is set out below.

- A hazard factor, $Z$, of 0.4;
- A near-fault factor, $N(D,T)$ of 1.0;
- A site subsoil classification of $D$ (deep soil), and a resultant spectral shape factor, $C_h(T)$ of 1.12;
- Table 4 provides a summary of the derived PGA's.
Table 4 - Summary of Derived PGA's

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Design Life/Importance Level</th>
<th>APE</th>
<th>Rs or Ru</th>
<th>PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLS</td>
<td>50 years/IL1</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>ULS</td>
<td>50 years/IL1</td>
<td>1/100</td>
<td>0.5</td>
<td>0.23</td>
</tr>
</tbody>
</table>

8.3.2 Liquefaction

Liquefaction is a phenomenon where saturated granular soils temporarily lose strength due to high pore pressure development during earthquake shaking. It predominantly occurs in loose silts and sands below the water table.

We have carried out a preliminary liquefaction assessment in accordance with the methodology outlined by Idriss and Boulanger (2008) and using the results from Standard Penetration Tests. The assessments indicate liquefaction is first triggered at peak ground accelerations (PGA’s) ranging from 0.12g to 0.15g (BH1 data set). This range is less than the design PGA of 0.23g for the ULS event.

The analyses indicate that the ULS event (0.23g) could result in liquefaction within layers of the beach sands. Liquefied soils will have considerably lower strength, when compared to the non-liquefied soils, and may result in soil settlement in the order of 100mm to 300mm (BH1 data set). These effects are further discussed in Section 9 below.

A series of cone penetration tests (CPT’s) could be undertaken to permit refinement of the lateral extent and effects of liquefaction. Our ‘Ground Investigation Scope and Cost’ letter dated 4 August 2014, included a cost estimate of $7,800 (excl. GST) for this activity. This covers contractor’s costs and Beca fees.

9 Geotechnical Considerations

9.1 Foundations

Shallow foundations are not considered appropriate due to the potential effects of scour and liquefaction.

There are a number of piled foundation alternatives available, which include bored concrete piles, pre-cast concrete driven piles, or steel driven piles. These alternatives would typically be adopted for much more heavily loaded structures, and therefore are not likely to be economic for this project. Timber piles founded within the medium dense sands are expected to be appropriate for this project. The timber piles could be driven or jetted to their design depth, noting the comments below.

A geotechnical assessment of the pile capacity has been undertaken following the methods of Meyerhof (1976). For axial loads, the timber piles will act in predominantly in end bearing, adopting a geotechnical ultimate bearing capacity of 8MPa. This assumes the following:

- The piles are founded in the medium dense sands with an corrected SPT(N) value in excess of 20, at a depth below -4m RL (nominally at a depth of around 8m below the level of the beach).
The piles are jetted to a depth in the order of 1m above the founding level, and are then driven to the required set.

A geotechnical strength reduction factor of 0.5 should be applied to the geotechnical ultimate capacity, for comparison with code factored structural loads.

The geotechnical capacity is expected to be mobilised at settlements in the order of 10% of the pile diameter, for example 30mm for a 300mm diameter timber pile. Settlements at working loads are expected to be less than 10mm.

9.2 Retaining Walls

Both retaining wall options should be designed as king post walls as the proposed spacing of the piles is greater than three times the pile diameter. The walls are expected to be sufficiently flexible such that they do not generate at-rest earth pressures. A set of active and passive earth pressure coefficients for preliminary design are provided in Table 2 above.

Under seismic conditions, based on the recommendations of researchers Wood and Elms in RRU Bulletin 84, it is anticipated that the walls will act as ‘flexible walls’. This is subject to confirmation of the form of the deadman anchors during preliminary design, which may result in the overall wall system behaving as a ‘stiff wall’. The combined static and seismic pressures should be derived in accordance with Section 3.5.3 of Wood and Elms (1990).

A traffic surcharge of 12KPa should be adopted for long term load conditions, and ignored for the scour and seismic load cases.

Refer to Section 9.3.2 below for commentary on local stability.

Option 1 and Option 2 comprise 8m deep timber piles. On the basis of our initial assessment, there exists an opportunity to refine the pile depth during preliminary design. This will require confirmation of KCDC’s expectation in terms of performance under seismic load case (for example, potential damage to the wall and path behind), coupled with an assessment of the tolerance of the structural units to total and differential settlements.

9.3 Slope Stability

9.3.1 Global Stability

Stability assessments were undertaken for Option 1 and Option 2. It was found that Option 1 returned slightly lower FoS for all load cases. Therefore the findings from the analyses, set out in Table 5 below, are based on Option 1.

<table>
<thead>
<tr>
<th>Load case</th>
<th>Description</th>
<th>Minimum Restoring Force to Achieve Target FoS(kN/m)</th>
<th>Outcome FoS</th>
<th>Target FoS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>Every day loads, with traffic surcharge</td>
<td>0</td>
<td>1.8</td>
<td>1.5</td>
</tr>
<tr>
<td>Scour</td>
<td>Allowance for 2.5m depth scour</td>
<td>10</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>ULS Seismic</td>
<td>Non liquefied soil conditions</td>
<td>0</td>
<td>1.3</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Liquefied | See Note 1 below | 0.4 | 1.0  
Post Earthquake Liquefied | 50 | 1.0 | 1.0  

Note 1 – Under ULS seismic load conditions, where the soil profile is partially liquefied, the analyses indicate that flow failure may occur, resulting in considerable lateral displacements. This assumes that liquefaction occurs in continuous layers. It is possible that the liquefaction occurs in more discrete zones, however this will require further investigations, such as CPT’s and assessment to confirm the liquefaction susceptibility of the beach sands.

The conclusions from these assessments are set out below.

- The wall is expected to exhibit adequate global stability under static long term and scour conditions, although it will rely on support from the deadman anchor (Option 1) or two rows of piles (Option 2). Refer to Table 5 above for the minimum restoring forces.
- On the basis of the geotechnical data available, Option 1 and 2 may not achieve adequate global stability under ULS seismic conditions if liquefaction occurs, resulting in potentially considerable damage to the wall and path behind. A supplementary investigation is required to better define the likely lateral extent of liquefaction.

9.3.2 Local Stability

The local stability of the wall will be assessed during the preliminary design. This could be undertaken using a limit equilibrium analysis package, such as Wallap™. The local stability should address the considerations set out below.

- The contribution of the proposed dead man anchor for Option 1.
- The performance of the twin rows of piles in Option 2, in particular whether they behave as a pile couple and the magnitude of the axial and lateral loads.
- The traffic surcharge, ground water profiles, lateral earth pressures and modulus values set out in this report.
- The tolerance of the path, concrete steps and underground utilities to horizontal and vertical movements.
- Check the stability of the wall, under static long term load conditions, for an over-dig of 0.5m depth immediately in front of the wall.

10 Options Assessment

The geotechnical considerations for Option 1 and Option 2 are very similar and there are currently no clear differentiators between each option.

Some minor differentiators are set out below.

- Option 2 - will require to carefully consider the spacing between the two rows of piles such that installation of the second row of piles does not affect the performance of the first row of piles. The piles should be installed at a centre to centre spacing in excess of three pile diameters.
- Option 1 – the deadman anchor could take a number of forms, such as a row of soldier piles or alternatively cast in situ or pre-cast panel wall. The deadman anchor system is expected to be a considerable proportion of the overall cost of the scheme. It may also temporarily limit the use of The Parade by pedestrians and vehicles.
Option 2 – there is a risk that Option 2 will require additional support, for example in the form of a deadman anchor or ground anchors, to ensure that it exhibits adequate local stability.

Both options will require a considerable amount of rock rip rap, it may be costly to locally source rock of appropriate quality. The quantity of rip-rap for Option 2 is expected to be slightly less when compared to Option 1, with the resultant saving in cost.

11 Construction Considerations

A summary of the construction considerations are set out below.

11.1 Constructability

- It is envisaged that a large excavator could install the timber piles. This will require design and construction of a safe working platform.
- Borehole logs indicate it is unlikely that large cobbles or boulder obstructions will be encountered within the beach sands. However, the presence of boulders or cobbles is possible within the fill behind the existing wall.
- Identification and protection of underground and overhead utilities.
- Temporary works to support the open excavation to permit the construction of the walls. Temporary excavations will need to be battered at an angle to suit the saturated granular nature of the soils. Temporary retention may be required where space is limited.
- Provision within the construction programme for tidal restrictions.
- Providing a safe working area within the tidal range and during inclement weather.
- Particular methodologies or plant to allow installation of piles (such as pile casing or heavy driving).
- Dealing with obstructions, such as a series of relic seawalls and boulders or cobbles within the fill behind the existing wall.

11.2 Verification and Testing

- Construction stage verification testing of the engineered fill.
- A limited number of construction stage proof bores to confirm the soil profile and founding conditions to the piles.
12 Conclusion and Recommendations

A summary of the key geotechnical conclusions and recommendations are set out below.

- The soil profile confirmed through the geotechnical investigation typically comprises a variable thickness of fill and loose sands, overlying a thick unit of medium dense sands. Dense to very dense sands were proven, although they lie below the anticipated founding depth of the retaining wall piles. Insitu rock was not encountered in the machine boreholes, and is expected to lie at considerable depth.
- Soil parameters have been derived to inform the preliminary design, and subsequent stages.
- A qualitative assessment of scour was undertaken during the options selection process. The slope stability assessments have allowed for a 2.5m deep zone of scour across the full length of the retaining wall.
- The seismic assessments have adopted an importance level of IL1 and a design life of 50 years. The analyses indicate there is the potential for layers of liquefiable material during the ULS seismic event. There is insufficient data to determine whether these liquefiable zones are discrete or continuous.
- The wall is expected to exhibit adequate global stability under static long term and scour conditions, although it will rely on support from the retaining wall (refer to Table 5 for the minimum restoring forces).
- On the basis of the geotechnical data available, Option 1 and 2 may not achieve adequate global stability under ULS seismic conditions if liquefaction occurs, resulting in potentially considerable damage to the wall and path behind. A supplementary investigation is required to better define the likely lateral extent of liquefaction. Our ‘Ground Investigation Scope and Cost’ letter dated 4 August 2014, included a cost estimate of $7,800 (excl. GST) for this activity.
- Timber piles are expected to be the most economic foundation solution for both options. The piles should be founded within the medium dense sands, nominally at a depth of around 8m below the level of the beach. There is an opportunity to refine the pile length during the preliminary design stage.
- There are no geotechnical considerations that clearly differentiate Option 1 and Option 2.
- The key considerations for construction include temporary works to provide a stable and safe working environment, earthworks testing on the engineered fill and a limited number of construction stage proof bores to confirm the soil profile and founding conditions to the piles.
13 Applicability

This report has been prepared by Beca on the specific instructions of our Client. It is solely for our Client’s use for the purpose for which it is intended in accordance with the agreed scope of work. Any use or reliance by any person contrary to the above, to which Beca has not given its prior written consent, is at that person’s own risk.

Should you be in any doubt as to the applicability of this report and/or its recommendations for the proposed development as described herein, and/or encounter materials on site that differ from those described herein, it is essential that you discuss these issues with the authors before proceeding with any work based on this document.

14 References

- AS/NZS 1170.0:2002 (Incorporating Amendment No 1,2 & 4) Structural Design Actions Part 0: General Principles
- Beca Ltd, Geotechnical Desktop Study Report, 5 May 2014
- Beca Ltd, KCDC Paekakariki Geotechnical Factual Report, 7 November 2014
- Geology of the Wellington Area’, Begg and Johnston, 2000, including 1:250 000 geological map 10.
Appendix A

Option Sketches
**Calculation Sheet**

**Job Number** 47601160  
**Date** 4/8/14

**Beca**

**Job Name** PAREKARIKI SEA WALL  
**Date** 4/8/14

**Subject** OPTION 1 - TIMBER WALL AND ROCK

**By** C. BROWN  
**Page No** 1 of 2

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**Option 1 - Timber Wall and Rock Revetment**

**Notes:**

1. Piles to be founded in medium dense sand at RL - 3.0m
2. All bolts to be grade 310 S/S
3. Footway slabs to be concrete with mesh and joints at 3000 cts
4. Rock revetment to be:
   - 2 layers: 0.30m MP = 40kPa, 0.50m layer reinforcement

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**Diagram:**

- Heavy duty geotextile
- RC anchor blocks 2m x 1.5m x 0.5m
- Vertical 100 x 100 timber lagging H4 treated
- 200 x 100 timber walers H4 treated
- Beach + 2.0m
- Excavation profile

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**Notations:**

- Heavy duty geotextile
- RC anchor blocks 2m x 1.5m x 0.5m
- Vertical 100 x 100 timber lagging H4 treated
- 200 x 100 timber walers H4 treated
- Beach + 2.0m
- Excavation profile
NOTES:

1. PILES TO BE FOUND AT MEDIUM DENSE SAND IN MEDIUM DENSE SAND AT 2L-GOM.

2. PROVIDE STEP UNITS WITH 300X300 STEPS 2000 WIDE WHERE REQUIRED AT ACCESS POINTS.

3. FOOTWAY SLABS TO BE CONCRETE WITH MESH AND JOINTS AT 3000 CTRS.

4. ROCK REVESTMENT TO BE AS OPTION 1.

5. RC STEP UNITS TO HAVE 200 KG/M³ REINFORCEMENT, GRADE 50 CONCRETE.
Appendix B

Exploratory Hole Location Plan
The proposed Seawall is 960m long. The wall starts at the Sand Track and terminates at 124 The Parade (past Tangahoe Road).
Appendix C

Geological Section
FIGURE 1

GEOLOGICAL SECTION The Parade, Paekakariki

Legend
- Asphalt
- Gravel (FILL)
- Sand
- Organics/wood
- Shelly sand
- No recovery
- Water level
- Existing ground level

N = Standard Penetration Test Blows (/300mm)
Where the SPT’N’ is bracketed, the test has been assessed as unreliable.