This report has been prepared for Pittwater Council in accordance with the terms and conditions of appointment for McCarrs Creek Road, Church Point – Seawall Construction/Realignment dated 18th April 2012. Hyder Consulting Pty Ltd (ABN 76 104 485 289) cannot accept any responsibility for any use of or reliance on the contents of this report by any third party.
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## APPENDICES

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INTRODUCTION

1.1 Project Description

Pittwater Council (the Council) is currently developing a concept to provide additional long term and commuter parking and small vessel tie up facilities on McCarrs Creek Road to service residents of Scotland Island as well as freeing up existing parking for visitors to the area.

The concept developed to date runs from outside the Pasedena Restaurant to Rosstrevor Reserve and includes the upgrading of the existing commuter and cargo wharfs. The existing road is relatively narrow and runs between a sea wall and a rock escarpment.

The road is proposed to be on reclaimed ground with the provision of a new seawall. Pedestrians will be catered for by a new boardwalk which would form part of the upgraded commuter wharf facility.

The area between the realigned road and the existing rock face is proposed to be developed as a commuter carpark, at grade initially with the flexibility to have an additional level constructed above.

1.2 Scope of Works

Hyder has been engaged to identify and review seawall options at the seaward edge of the proposed realignment of McCarrs Creek Road. This report addresses the following:

- A review of background information including preliminary geotechnical investigation;
- Establishment functional design criteria;
- A Review and comparison of identified seawall structure options;
- A review of seawall layout options;
- An understanding of high level risks and opportunities; and
- Recommendations on additional work to progress to the next phase.

1.3 Objectives

The aim of this report is to achieve the following objectives:

- To assess preliminary geotechnical investigation findings and identify key constraints associated with the site;
- To identify functional design criteria for the new seawall;
- To identify seawall structure options
- To review seawall layout options and provide recommendation;
- To identify high level risks and opportunities associated with the alignment of the seawall; and
- To provide recommendations to progress to the next phase.
2 BACKGROUND INFORMATION

Hyder has reviewed and considered the following documents prior to developing seawall structure and layout options:

1. *Church Point Plan of Management* by Pittwater Council, dated November 2009
3. Report on Geotechnical Investigation for Proposed New Commuter Wharf at McCarrs Creek Road, Church Point, NSW by Crozier – Geotechnical Consultants, dated September 2012.
4. ‘Plan Showing Proposed Commuter Pontoon at Church Point” by Souter & Associates (refer Appendix B).
5. Hyder drawing no SKC007-AA003240-01 ‘Proposed Carpark Ground Floor Design Contours’ (refer Appendix A).
6. Council Proposed Carpark 60 degree parking layout (refer Appendix A).

2.1 Church Point Plan of Management

The Plan of Management has been prepared based upon the 2004 Master Plan and further developed in response to input from representatives of on-shore and off-shore communities. The objective of the Plan of Management is to ‘ensure that Church Point retains its environmental, recreational, scenic, cultural and social values, while key issues relating to the management of the study areas are addressed’.

Some of the key issues surrounding the existing McCarrs Creek Road are:

- Precinct 1 (McCarrs Creek Road) is described as ‘quite an eyesore’;
- The existing seawall is in a state of disrepair with sections beginning to fail and fall into the estuary;
- The commuter ‘dinghy’ wharf suffers from insufficient mooring space leading to chaotic and dangerous dinghy tie ups;
- McCarrs Creek Road is particularly dangerous as it is narrow and cars tend to travel at excessive speeds;
- Lack of parking; and
- Walkway adjacent to the road is too narrow.

A management strategy has been developed, key points related to McCarrs Creek Road are:

- Options to increase parking in designated car park area;
- Provide 2.4m cycleway/pedestrian path along foreshore;
- Realignment of McCarrs Creek Road including new seawall that ensures habitat for aquatic organisms;
- Develop detail masterplan for dinghy/pontoon facility; and
- Geotechnical assessment of cliff line to ensure long term safety of carpark area.
2.2 Fisheries Guidelines for Fish-Friendly Structures

DPI&F NSW currently only has fish friendly guidelines for waterway crossings. Whilst the ‘Fisheries Guidelines for Fish-Friendly Structures’ document was developed for fisheries in Queensland, Hyder recommends the use of this guideline in order to achieve Council’s requirements for a fish-friendly structure.

The purpose of this guideline is ‘to encourage consideration of, and provide guidance for, the planning, design, construction and operation of aquatic infrastructure so that it is ‘fish-friendly’.

According to the guideline, fish-friendly structures:

- Cause minimal disturbance to the existing environment; and
- Incorporate design features that provide an enhanced habitat in which fish can live.

The guideline also provides general design features for various infrastructure examples. Rather than repeating the design guidelines’ content, below are an example of key points relating to design of seawalls:

- Recommended materials such as rubble toe/ riprap revetment provide more habitats for biota than homogeneous structures such as smooth concrete;
- Various rock sizes within a structure create greater habitat diversity through larger and more varied spaces; and
- Vertical structures provide less fish habitat than sloping structures. If vertical walls are necessary, these should contain weep holes covered with a geotextile membrane to prevent water build-up behind the structure.

2.3 Report on Geotechnical Investigation

A geotechnical site investigation was carried out by Crozier – Geotechnical Consultants. The scope of this work includes:

- Three on-land boreholes with coring of bedrock; and
- A series of probing holes in the vicinity of the proposed seawall alignment carried out at low tide.

The key geotechnical constraints identified by this geotechnical investigation are summarised below:

- A potential submarine bedrock cliff line has been identified and is located approximately 10 to 13m west of the existing seawall. This may cause potential instability of the proposed seawall should a gravity structure is considered. If a piled wall is considered the toe of the wall could be potentially unstable within the cliff area unless the toe of a pile is beyond the base of cliff line. This is likely to result in excessive pile total length in these areas.
- The alignment of the submarine bedrock cliff line is not clearly defined due to limited site investigation carried out. It has been assumed that part of the new seawall may be located above the crest of the cliff and the remainder would be below. This may cause further instability risks, particularly at areas where the wall intersects at the edge of the submarine cliff. Therefore, it is highly recommended to carry out detailed offshore
geotechnical investigation in order to further assess the risks and impacts associated with the location of the proposed new seawall.

- The investigation identified that the site is underlain by highly weathered sandstone and shale bedrock. The nature of the rock escalates the potential instability of the seawall, particularly at the bedrock cliff line.
- A layer of soft mud has also been identified during the probing in the vicinity of the seawall alignment. This layer should be removed for any gravity type of structure for stability and settlement control. Similarly this removal of soft mud will be required for reclamation behind the seawall to limit the long term settlement.
- Potential unstable rock wedges have also been identified on the existing rock cut of the existing road. This may pose potential risks for the proposed future car park. As such stabilisation or treatment measures should be taken into account in the planning of the development.
- The new pontoon is anchored in place by concrete and steel pylons which are drilled into the seafloor. As-built piling records suggest that the pylons are drilled into the seabed between 4.6 to 5.8m. It is understood that screw piles were installed through sandy sediments until refusal, which is assumed to have occurred on bedrock of at least low strength. For piled wall, the pile length will depend on the position of the wall in relation to the submarine bedrock cliff alignment, i.e. above or below cliff line.
3 FUNCTIONAL DESIGN CRITERIA

Reference Standards and Guidelines
The following lists Australian and international guidelines used for design:

- AS 3600 – 2001: Concrete Structures
- AS 1720 – 2010: Timber Structures – Design Methods
- AS 4678 – 2002: Earth Retaining Structures
- AS 3962 – 2001: Guidelines for Design of Marinas
- PIANC Guidelines

Design Life
In accordance to AS4997-2005, a minimum design life of 50 years has been adopted.

Surface Levels
Based on discussions with the Council, the proposed realigned McCarrs Creek Road shall maintain a surface level of +1.80AHD as per the existing road level.

Bathymetry
Hyder has been provided with the following survey file (also attached in Appendix B) to be utilised for design:

- ‘Plan Showing Proposed Commuter Pontoon at Church Point” by Souter & Associates. Levels are given in Australian Height Datum (AHD), which is +1.542mCD

By observation, from the existing seawall the seabed level is on a 1V:5H gradient past the proposed pontoon location.

Tidal Planes
Due to lack to tide level information, the Fort Denison tide gauge predictions have been adopted as approximately representative of the site:

- Lowest Astronomical Tide = -0.925m AHD
- Mean Sea Level = -0.035m AHD
- Highest Astronomical Tide = +1.175m AHD

The survey file included in Appendix B noted that the zero tide (average lowest tide) is approximately -0.9m AHD.
Design Wave Heights

The design wave heights have been calculated using the USACE Coastal Engineering Manual. wind wave hindcasting approach

The parameters assumed were:

Table 3-1  Design Wave Heights Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 year average recurrence interval wave event for seawall structure design</td>
<td>AS4997 Table 5.4</td>
</tr>
<tr>
<td>Importance Level 2 – Ordinary Structure</td>
<td>AS1170.0 Table 3.1</td>
</tr>
<tr>
<td>45m/s wind speed</td>
<td>AS1170.2 Table 3.1</td>
</tr>
<tr>
<td>Wave run-up and overtopping for a 1 in 25 year design wave condition</td>
<td>AS1170.0 Table 3.3</td>
</tr>
<tr>
<td>~1.5km narrow fetch length to the North</td>
<td>Figure 3-1</td>
</tr>
</tbody>
</table>

Based on the parameters described above, the wave height characteristics are as follows:

- Significant wave height, $H_s = 1.1m$;
- Significant wave period, $T_s = 2.5s$; and
- Design wave height, $H_1 = 1.5 \times H_s = 1.65m$ (1.5 for short narrow fetch)

The effects of vessel wake and current have been considered insignificant and are assumed not governing.

Figure 3-1  Church Point Approximate Fetch Length
**Design Vessel**

It is understood that the pontoon shall cater for a vessel length up to 5m. Based on AS3962, the following maximum design vessel dimensions have been adopted:

- Length: 5.0m
- Draft: 1.0
- Beam: 2.0m

**Seawall Length**

Based on the available survey file, the proposed seawall is approximately 150m long.

**Fairway Width**

In accordance to AS3962 Clause 3.1.2, a minimum fairway width of 1.5L is required, where L is the length of design vessel. This equates a minimum fairway width of $1.5 \times 5m = 7.5m$.

Figure 3-2 below provides a diagrammatic explanation of the above.

![Fairway width requirements](image)

**Water Depth**

Within the fairway width, a minimum clear water depth of 1.5m is required to prevent vessel damage due to vessel movements at any tide stage. The minimum water depth of 1.5m is achieved based on the following factors:

- Design vessel draft = 1.0m; and
- Half significant wave height = 0.5m
Figure 3-2 shows a 1.5m minimum clearance from lowest astronomical tide (-0.9m AHD). Seawall construction tolerances are to be below this safe vessel access envelope.

**Design Levels**

It is understood that the design surface level of the proposed McCarrs Creek Road is to be maintained at its existing level of +1.8m AHD. This level has been assumed the same for the top of the seawall.

**Walkway**

A 2.4m wide walkway has been assumed in accordance to the Plan of Management.

**Structural Criteria**

- AS4997 Table 6.1: minimum 50 year design life for a normal maritime structure;
- AS4997 Table 5.4: a normal maritime structure is to be designed for a 500 year average recurrence interval wave event;
- AS4997 Table 5.1: the structure (Class 5) designed to allow for 5kPa uniformly distributed load (pedestrian crowd load) and 20kN traffic surcharge at the new McCarrs Creek road located behind the walkway;
- Design load combinations to be in accordance with AS1170 and AS4997;
- Timber structures to be designed in accordance with the requirements of AS1720.1 and AS4997; and
- Total and differential settlement limits in accordance with the requirements of Pittwater Council and/or relevant authorities. Indicative limits for edge structures are:
  - ~100mm total vertical settlement and horizontal movement over 20 years; and
  - ~1/250 to 500 differential over 20 years.

**Geotextile Membranes**

Geotextile membrane is required for filtration purposes where fill and armour layers are not filter graded. Geotextiles shall have the following minimum properties:

<table>
<thead>
<tr>
<th>Properties</th>
<th>Units</th>
<th>Type 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>g/m²</td>
<td>500</td>
</tr>
<tr>
<td>Width of fabric</td>
<td>m</td>
<td>6</td>
</tr>
<tr>
<td>Geotextile Strength Class*</td>
<td></td>
<td>E</td>
</tr>
<tr>
<td>Elongation</td>
<td>%</td>
<td>≥ 30</td>
</tr>
<tr>
<td>Grab strength</td>
<td>N</td>
<td>1600</td>
</tr>
<tr>
<td>Tear</td>
<td>N</td>
<td>650</td>
</tr>
<tr>
<td>G Rating</td>
<td></td>
<td>4500</td>
</tr>
<tr>
<td>Filtration Class*</td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>EOS (0₉₅)</td>
<td>μm</td>
<td>≤ 200</td>
</tr>
</tbody>
</table>
### Properties

<table>
<thead>
<tr>
<th>Properties</th>
<th>Units</th>
<th>Type 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Rate (Q)</td>
<td>L/m³/sec</td>
<td>≥ 50</td>
</tr>
<tr>
<td>Permittivity (ψ)</td>
<td>s⁻¹</td>
<td>≥ 0.5</td>
</tr>
</tbody>
</table>

*supply, testing, hold points and placement specifications will be provided at detailed design stage.

### Drainage

Provision for drainage behind the seawall has been assumed as:

- Additional drainage at the seawall (e.g. weepholes) may be required to ensure effective drainage behind the wall if an effectively impermeable solution is adopted;
- Gross pollutant interception, surface water drainage outfalls and other water quality requirements to be assessed in subsequent design stages.

### Reusable Materials

Following discussions from meetings with the Council representatives, it has been suggested to re-use the stone / blocks from the existing seawall post demolition. Recycling of these blocks will provide protection to the seawall structure by minimising erosion under the wall, promote a fish-friendly habitat and reduce overall cost of material sourcing. Note that re-using of these stone / blocks will be subject to satisfaction of design specifications requirements.

Figure 3-3 below shows an example of recycled stones placed in front of a seawall with the exception of the stones being underwater at McCarrs Creek Road.

### Geometric Constraints

The realignment of the seawall is subjected to the following constraints:

- The new pontoon location with small vessels moored at both sides; and
- A minimum 8m wide road for the realigned McCarrs Creek Road;
4 SEAWALL STRUCTURE OPTIONS

Hyder has previously carried out seawall options evaluation and has identified a number of seawall structure including L-shaped retaining wall, block wall and continuous piled wall (Hyder Draft Interim Report issued 1st June 2012). It has been considered that the option of precast L-shaped reinforced concrete retaining wall. However, this option is considered costly and impractical from the immense and complex task of lifting the walls and placing them in a curved alignment.

Two options have been considered for the proposed new seawall:

- Option 1: Block wall;
- Option 2: Continuous piled wall

These options have been designed based on a number of assumptions as described in Section 3. A brief description on the types of materials / method of construction, and advantages and disadvantages for each option is included in the following sections. Note that for both of these structures, a timber boardwalk for pedestrians shall be provided as specified by the Council.

From Section 2.3, it is noted that parts of the seawall may be located on top of the submarine bedrock cliff and the remainder may be located beyond the cliff line. Given this scenario, there is a possibility that the new seawall may be built as a ‘combination’ wall, i.e. block wall at the shallow sections and continuous piled wall at the deeper sections (refer Figure 4-4). However, the feasibility of a ‘combination’ wall depends on the following key factors:

- Results from further geotechnical investigations and assessment of issues such as stability risks particularly where wall alignment intersects with submarine cliff line; and
- Distribution of loads between varying walls and how these structures will tie-in with each other.

Figure 4-4 ‘Combination’ wall example

4.1 Option 1: Block Wall

Block wall, similar to the existing McCarrs Creek Road seawall, is one of the most commonly used gravity-wall type structures in areas for coastal protection. These structures consist of blocks, either natural stone or proprietary concrete elements such as Durahold, placed one on top of the other in a masonry wall pattern on a prepared bed and backfilled with suitable material. To ensure overall stability of the wall, the wall needs to sit on a sound foundation and may require some form of anchorage (e.g. geogrid).
This foundation is achieved where the wall can be laid directly on a levelled rubble base surfaced with crushed stone or an in situ concrete blinding.

Based on the outcome of preliminary geotechnical investigations, sediments above bedrock will need to be removed and levelled to some extent to build the foundation with a minimum thickness of 600mm. A rubble base consisting of crushed stone is preferred for the foundation as it is less costly compared to concrete and less complex compared to concrete blinding under water.

Table 4-3 Block Wall Option Advantages & Disadvantages

<table>
<thead>
<tr>
<th>Option</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block Wall</td>
<td>• Good durability performance with minimum maintenance if built well.</td>
<td>• Less suitable near to or beyond bedrock cliff compared to piled wall with tieback / anchor.</td>
</tr>
<tr>
<td></td>
<td>• Easy to construct in a dry environment or low tide.</td>
<td>• Generally, stripping of unsuitable material at the top of seabed level is required before levelling of rock or firm ground to provide for a solid foundation.</td>
</tr>
<tr>
<td></td>
<td>• The use of natural blocks is aesthetically appealing</td>
<td>• Requires effective drainage behind the wall to prevent tidal build up behind the wall. Drainage measures such as the use of perforated drains and weepholes will need to be cleaned regularly to prevent blockage.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Depending on ground conditions, provisions for tie back may be required for wall height greater than 1.5m to 2m high.</td>
</tr>
</tbody>
</table>
4.2 Option 2: Continuous Piled Wall

Continuous piled construction involves driving/boring/screwing a row of piles along the seawall face. The piled wall needs to be designed to resist lateral forces due to backfill, water, traffic surcharge etc. Some of the horizontal forces are absorbed by passive earth pressure in front of the piles through their penetration into the seabed, and tied anchor rods to a deadman or anchor structure behind the pile wall will transmit majority of the horizontal forces.

Pile materials are generally either reinforced concrete or steel for structures requiring a 50 year design life or greater (i.e. timber excluded). In this report, it is suggested to adopt reinforced concrete as the preferred material. This is due to the fact that steel materials is aesthetically unpleasant and requires extensive maintenance measures through its design life, such as repainting and/or the use of cathodic protection. On the other hand, a reinforced concrete pile wall can have less to no maintenance required if constructed properly and design measures are taken to protect the reinforcement (eg specifying appropriate concrete admixtures, adopting stainless steel or glass fibre reinforcement).

A reinforced concrete piled wall can be in various forms, such as secant piled wall and contiguous piled wall (refer Figure 4-6) depending on groundwater/tidal conditions and backfill materials. Upon completion of pile driving, a reinforced concrete capping beam may be poured at the top of the piles for the entire length of the retaining wall. This beam will assists in distributing loads, and is also more aesthetically appealing for pedestrians walking along the seawall.

Figure 4-6 Reinforced Concrete Piled Wall images
### Table 4-4 Continuous Piled Wall Option Advantages & Disadvantages

<table>
<thead>
<tr>
<th>Option</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuous piled</td>
<td>• Pile driving is a relatively simple exercise requiring standard construction plant</td>
<td>• Requires the construction of bund from landside to receive piling plant and to drive piles through</td>
</tr>
<tr>
<td>wall</td>
<td>• Increased construction alignment flexibility</td>
<td>• Contiguous pile wall is not watertight. Measures may be required to prevent fines wash-out between piles</td>
</tr>
<tr>
<td></td>
<td>• Secant piled wall reduces water seepage and fines wash-out</td>
<td>• Local remediation may be required where piles intersect with the submarine cliff due to high risk of instability</td>
</tr>
</tbody>
</table>

### 4.3 Timber Boardwalk

It is understood there are provisions for a timber boardwalk to be constructed for pedestrian access and is located adjacent to the new seawall. This structure shall be designed to tie in with the seawall.

The timber piles shall be designed at minimum 3m centre to centre spacing and the pile embedment length will depend on the insitu soil material properties. A 1m high marine grade aluminium handrail shall be provided along the seaward edge of the boardwalk for pedestrian safety. A concrete barrier shall also be provided between the boardwalk and McCarrs Creek Road.

As described in Section 3, there is possibility to recycle blocks from the existing seawall and place them in front of the new seawall. This means that the blocks will ultimately be located under the timber boardwalk. It is recommended that the timber boardwalk should be installed prior to placement of these blocks as piling through blocks will be difficult otherwise.
5 SEAWALL LAYOUT OPTIONS

Following the structure options, this section compares two different layout options:

- Case 1: Proposed Ultimate layout
- Case 2: Alternative layout

These layouts have been provided by the Council and are included in Appendix B.

The following sections provide further description of each layout, discussions on their constructability, preliminary cost comparison and any risks related issues that are of concern.

5.1 Case 1: Ultimate Layout

Description

The ultimate layout (refer Figure 5-7) entails the development of the following elements:

- Realignment of the existing McCarrs Creek Road to provide for a multi-storey carpark located at the existing road and adjacent to the cliff.
- The maximum distance between the existing seawall and the new seawall is approximately 20m.

![Proposed Ultimate Layout](image)

Constructability

Based on the review preliminary geotechnical investigation report as described in Section 2.3, it has been interpreted that parts of the new seawall appears to be located at the edge of a submarine bedrock cliff approximately 2m high.

Due to this rock platform, a stone / block seawall is not feasible due to stability issues from sitting at the edge of the platform. If a continuous piled wall is considered the toe of the wall could be potentially unstable within the cliff area unless the toe of a pile is beyond the base of cliff line.

As discussed in Section 4, there is a possibility to construct the new seawall as a ‘combination’ wall, i.e. block wall at shallow sections and continuous piled wall at deeper sections. Feasibility of this type of wall will depend on further offshore geotechnical site investigations to gain a deeper understanding of the submarine cliff profile and its underlying material properties. It is envisage that the block wall shall be constructed at a certain distance away from the edge of the cliff to reduce stability risks. Furthermore,
extensive localised treatment may be required for stability at areas where piles intersect at the edge of the cliff.

5.2 Case 2: Alternative Layout

Description

The alternative layout (refer Figure 5-8) is a ‘reduced’ version compared to the ultimate layout, with the following elements:

- Replace multi-storey carpark with on-ground carpark with up to 57 parking bays; and
- New seawall is positioned further landward than the ultimate layout, where the maximum distance between the existing seawall and the new seawall is approximately 10m.

Figure 5-8 Proposed Alternative Layout

Constructability

In comparison to the ultimate layout, the alternative seawall is positioned further landward. It is assumed that majority of the wall may be located above and away from the crest of the submarine bedrock cliff. Feasibility of a block seawall will depend on further geotechnical investigation for details of the bedrock profile. Despite being further landward, part of the seawall is still located below zero tide. At these areas, a working platform will need to be constructed from land to allow access for construction equipment. Constructability issues related to continuous piled wall are as described in Section 5.1.
5.3 Cost Estimates Comparison

Preliminary cost estimates for block wall and continuous piled wall under Case 1, 2, and 3 have been carried out. The following table shows order of magnitude per meter length of the seawall. Note that the cost will likely vary along the length of the wall due to varying retained height and foundation conditions. These estimates represent an average over the full wall length.

<table>
<thead>
<tr>
<th>Structure Option</th>
<th>Case 1 Ultimate Layout</th>
<th>Case 2 Alternative Layout</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block Wall</td>
<td>$22,000/m</td>
<td>$15,500/m</td>
</tr>
<tr>
<td>Continuous Piled Wall</td>
<td>$30,000/m</td>
<td>$22,000/m</td>
</tr>
</tbody>
</table>

The cost estimates in Table 5-5 were determined based on the following assumption:

- Timber boardwalk cost has been excluded from the cost estimates.
- 15% allowance has been assumed for construction in a wet environment depending on the location of seawall for Case 1 and 2.
- Allowance has been assumed for preliminaries (15%), contingency (10%) and profit (10%).
- It is assumed that the existing seawall stones / blocks will be recycled and the amount is sufficient through the length of the new seawall. Additional rock material has not been considered.
6 OPPORTUNITIES & CONSTRAINTS

Based on the review of seawall structure and layout options as described in Section 4 and Section 5, the following summarises key constraints and opportunities identified earlier.

Opportunities

- The Ultimate Layout provides maximum at grade carparking spaces with the potential for additional second storey spaces.
- There is a possibility to build a ‘combination’ wall to reduce overall cost of construction, i.e. block wall at shallow sections and continuous piled wall at deeper sections. Feasibility of this type of wall will depend on further geotechnical investigations and assessment.
- There is an opportunity to align the seawall further landward to a certain point where construction of seawall will be in a ‘dry’ environment and the entire seawall may be located above the submarine bedrock cliff. This will reduce construction risks and its overall cost.
- Additional carparking may be provided on a suspended deck structure adjacent to the new seawall.
- Availability to recycle existing seawall blocks to as seawall protection reduces cost of material supply.

Constraints

- The alignment and nature of the submarine bedrock cliff is unknown due to the limited site investigations. Further information on the foundation conditions is required to confirm the use of the proposed wall types and the feasibility of a combination wall.
- Local remediation may be required at areas where piles intersect with the submarine cliff due to high risk of instability.
- Block wall will need to be set back at some distance from the submarine cliff line as there is a higher risk of stability issues.
- The Alternative Layout eliminates the opportunity for a multi-storey carpark and has a reduced the number of on-ground carparking spaces compared to the Ultimate Layout.
CONCLUSION & RECOMMENDATIONS

In summary, the table below summarises key findings of the report:

<table>
<thead>
<tr>
<th>Case</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1: Ultimate Layout</td>
<td>• Able to provide for multi-storey carpark</td>
<td>• Seawall may be located above or beyond the assumed submarine bedrock cliff line which poses risks of instability issues</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Highest cost due to increased wall height and backfill quantities</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Local remediation may be required at areas where piles intersect with the submarine cliff due to high risk of instability</td>
</tr>
<tr>
<td>Case 2: Alternative Layout</td>
<td>• Less backfilling than ultimate layout</td>
<td>• Seawall may be located above or beyond the assumed submarine bedrock cliff line which poses risks of instability issues</td>
</tr>
<tr>
<td></td>
<td>• Portion of wall located beyond the submarine bedrock cliff is likely to be less than Case 1.</td>
<td>• Local remediation may be required at areas where piles intersect with the submarine cliff due to high risk of instability</td>
</tr>
<tr>
<td></td>
<td>• Less costly than ultimate layout</td>
<td>• Limited to on-ground carpark due to insufficient area for multi-storey carpark</td>
</tr>
</tbody>
</table>

In order to further progress to detailed design stage, Hyder recommends the following:

- There is great uncertainty surrounding the nature and alignment of the submarine bedrock cliff. Further detailed geotechnical investigation is important to assess the submarine bedrock cliff profile, underlying soil layers and properties prior to detailed design phase;
- Council to consider cost benefits between each option (i.e. on-ground carpark vs. multi-storey carpark);
- Detailed design of the roadway alignment, seawall, footpath, drainage provisions and utility relocations;
- Contractor consultation to confirm construction sequence and methods to refine the budget estimate; and
- Consultation with community and Department of Fisheries for feedback on seawall layout and structure options.
APPENDIX A

LAYOUTS AND OPTION SKETCHES
CASE I: ULTIMATE LAYOUT
CASE 2: ALTERNATIVE LAYOUT
APPENDIX B

SURVEY FILE
NOTES:
- Levels are on the Australian Height Datum (AHD).
- Origin of levels PM 27879 - R.L. 1.542 AHD.
- Top of stabilizing piles to be RL 2.7 AHD.
- Zero tide approx. -0.9 AHD.
- Boundaries not marked.
- No investigation of underground services has been made. All relevant authorities should be contacted prior to any excavation on or near the site.
- Building Envelope for pontoon shown in orange. Please note set out is related to one centre point only.

CLIENT: PITTWATER COUNCIL
I.J. SOUTER
REGISTERED SURVEYOR

PONTOON AT CHURCH POINT
PLAN SHOWING PROPOSED COMMUTER

SOUTER & ASSOCIATES
SUITE 6, HERON COVE MARINA, QUEENS PDE (WEST), NEWPORT 2106
CONSULTING SURVEYORS AND PLANNERS

LGA PITTWATER
PHONE: 02 9979 5709   FAX: 02 9979 9489

03/03/2012
REF. No. 27-94
SCALE 1: 600
DATE
APPENDIX C

CROZIER GEOTECHNICAL REPORT
REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED NEW COMMUTER WHARF

at

McCARRS CREEK ROAD, CHURCH POINT, NSW

Prepared for

Pittwater Council

Project: 2012-195
September, 2012
ABSTRACT: This report details the results of a geotechnical investigation carried out for a proposed new commuter wharf adjacent to McCarrs Creek Road, Church Point, NSW. The investigation was undertaken for Pittwater Council. The existing sandstone boulder and block sea wall supporting McCarrs Creek road is showing signs of deterioration whilst the existing boat mooring facility is proposed to have increased capacity, public access and car parking.

The results of the investigation suggest that the proposed new sea wall will be located within an area containing sandy sediments over the highly weathered bedrock which is formed with an interpreted submerged and buried foreshore rock platform, of up to 2.0m height along its western edge. The bedrock consists predominantly of sandstone with shale horizons related to the Narrabeen Group of rocks which appear deeply weathered and of generally low strength. Zones of closely spaced near vertical joint defects and isolated moderately dipping joint defects were identified in the existing McCarrs Creek Road rock cutting and the bore holes.

Geotechnical issues which were identified and need to be considered in the design of the new sea wall include the edge of the foreshore rock platform which is very close to the alignment of the proposed wall, the deeply weathered nature of the bedrock and the moderately sloping joint defects and jointed zones which could impact the stability and weathering of the bedrock.

1. INTRODUCTION:

It is proposed to construct a new sea wall on the western side of Church Point extending south from the existing mini-market and public ferry wharf. This will allow the re-alignment of McCarrs Creek Road over to the edge of the sea wall and the construction of a new car parking facility at the base of the existing sandstone road cutting. A boardwalk with floating pontoon system for boat mooring is proposed offshore.
1. INTRODUCTION: (continued)

The investigation was undertaken at the request of Mr. Mark Eriksson of Pittwater Council to assess sub-surface conditions and provide geotechnical advice for design of the sea wall by others. The investigation was undertaken as per our tender (No. P12-306, Approved: 27th August 2012).

The investigation comprised:

a) A detailed geological inspection and mapping of the site and adjacent land by a Senior Engineering Geologist and Geotechnical Engineer.
b) Review of Ortho Photomaps and Aerial Photography of the site.
c) Drilling of three boreholes to determine subsurface geology.
d) In situ testing of the soils and laboratory testing of rock core samples.
e) DCP and pushed probe tests to assess the depth of sediment overlying bedrock

Details of the fieldwork are given in the report, together with comments relating to the geotechnical assessment and recommended design and construction. The following reports, plans and diagrams were supplied by Pittwater Council for this work:

- Pile installation record by International Marina Consultants, Job No. 4240, Email: 28th June 2012.

2. LOCATION AND GEOLOGY:

2.1. Site Description:

The site is located on the western side of Church Point where McCarrs Creek Road makes a sharp turn to the south adjacent to the end of the point. There is currently a low sandstone block sea wall around the northern end of Church Point which supports a reclaimed/filled area containing access to the existing Public Wharf and a mini-market. To the south-west of this location a low sandstone block and boulder sea wall has been formed to support the McCarrs Creek Road pavement and pedestrian pathway. In the centre of this section of shoreline the sea wall strikes north-west several metres to provide access out to a floating pontoon and boat mooring facility. Further to the south the sea wall again strikes several metres west to create a parking area and then a cargo wharf before Rosstrevor Reserve continues to the south of the site.

The proposed sea wall and associated works will extend south from the existing sea wall supporting the community mini-market to the car parking area and cargo wharf.
2.2. Geology:
Reference to the Sydney 1:100,000 Geological Series sheet (9130) indicates that the site is underlain by
Newport Formation (Upper Narrabeen Group) rock (Rnn) which is of middle Triassic Age. The Newport
Formation typically comprises interbedded laminites, shale and quartz to lithic quartz sandstones and pink clay
pellet sandstones. The rock unit was identified and mapped on the site along with recent marine sand deposits
along the foreshore.

3. FIELD WORK:

3.1 Field Work Methods:
The investigation comprised detailed inspections of the site and adjacent land by Senior Engineering Geologists
and a Geotechnical Engineer. It included the drilling of three augered and diamond cored boreholes (BH1 to
BH3) adjacent to the existing sea wall to identify sub-surface geology on the 11\textsuperscript{th} and 12\textsuperscript{th} September 2012. This
portion of the investigation included the use of Standard Penetration Tests (SPT) conducted at regular intervals
through the surficial soils during auger drilling to estimate soil strengths and collect samples for identification
purposes. Detailed geotechnical logging of the rock core was undertaken with selected samples tested via Point
Load tests in line with the relevant Australian Standards (AS4133).

A total of seventeen probe and DCP tests were also undertaken on the 18\textsuperscript{th} September 2012 along the shoreline
at low tide to determine the depth to bedrock below the sea bed where access for a drilling rig was not possible.

A line of chainages were measured along the western edge of McCarrs Creek road, with CH 0 marked at the
northern end, adjacent to the south-east corner of the existing mini-market. Borehole and probe test locations
along with geotechnical mapping information are included on Figure: 1. A series of sections with geological
interpretation are included in Figure: 2. Detailed bore log sheets are included in Appendix: 1 along with core
photography and selected photos of the site and adjacent land.

3.2 Field Observations:
McCarrs Creek Road is a two lane road with bitumen pavement passing north-south through the eastern side of
the site. The eastern edge of the road is formed at the base of a rock and soil cutting into the natural hill slope
that increases in height to the south matching the rise of the ridgeline. A bitumen paved pedestrian walkway
extends along the western side of McCarrs Creek road, delineated from the road by a row of timber bollards.
The outer western edge of this walkway is marked by a sandstone block and sandstone boulder sea wall that
supports fill soils used in the construction of McCarrs Creek Road.

The existing seawall from Chainage 0 to Ch. 26 is formed as a mortared and dry stacked sandstone block
retaining wall of significant age that shows signs of salt deterioration/erosion. Sandy sediments have been
deposited at the shoreline with the exposed portion of the wall up to 1.3m in height.
3.2 Field Observations: (continued)

From Ch. 26 the remainder of the seawall is formed of dry stacked irregular shaped sandstone boulders. This wall is up to 1.7m in height with a gently sloping rocky shoreline extending approximately 8m to the west from the base of the wall, see Photo: 1.

Between Ch. 118 and Ch. 122 the sea wall extends west approximately 10m to create a path out to the access ramp for the existing floating commuter wharf and boat mooring facility. The base of the seawall from Ch.105 to Ch. 125 is covered in sandy soils which also extend west into deeper water around the outside edge of the rocky shoreline. Portions of the boulder wall have collapsed or are missing between Ch. 130 and Ch. 150 with fill soils exposed at a very steep slope angle down to the rocky shoreline and showing signs of minor erosion.

From Ch. 160 the boulder wall again extends west up to 15m with backfilling to create a level car parking area and then the cargo wharf at similar level to McCarrs Creek Road. Inspection of the outer edge of the car parking area revealed a moderately sloping boulder wall extending up from the sandy sea floor. This wall ends approximately 1.0m below the car parking level with a narrow level terrace and the near vertical face of compacted sandy and gravely soil extending up to the car park. The exposed soil face shows signs of previous erosion. It appears, but could not be confirmed, that the boulder wall has settled into the sea floor resulting in the exposed soil face, see Photo: 2.

The existing commuter wharf with floating pontoon and boat mooring facility is a recent replacement to a previous similar structure. The new pontoon is located between 24m and 34m west from the existing sea wall and is anchored in place by a total of 6 concrete and steel pylons which were drilled into the sea floor. A review of the pile records suggests that the pylons are formed with steel screw piles drilled into the seabed between 4.6 and 5.8m with concrete top piles spliced as per an engineered design. It is understood that the screw piles were installed through sandy sediments until refusal, which is expected to have occurred on bedrock of at least low strength. The sea floor was measured adjacent to each pile location as part of our site investigation to provide an assessment of sediment thickness and bedrock depth utilising the pile installation records, see Figure: 2.
3.2 Field Observations: (continued)
The eastern side of the site is marked by an outcrop of sandstone bedrock which shows signs of previous excavation to form a very steep to near vertical rock face. The crest of this excavation is formed with a natural to over steepened soil slope which is covered in extensive vegetation. The sandstone bedrock cutting and soil slope increase steadily in height from north to south matching the ridge slope and are bound along the eastern upslope side by an existing common access driveway for the properties upslope.

Between Ch. 0 and Ch. 21 the slope above the cutting is extremely steep and predominantly formed with exposed fill and natural soil with outcrops of low to medium strength sandstone bedrock at the base. This portion of the slope shows signs of minor erosion. From Ch. 21 to Ch. 38 the rock cutting is generally 1.3m in height and vertical with an extremely steep (>56°) slope above, exposing weathered sandstone and siltstone bedrock with residual clays and colluvium near surface. This portion of the slope shows signs of erosion, slumping and over-steepening.

Between Ch. 38 and Ch. 134 the rock cutting is vertical and generally between 2.5 to 3.0m in height. The bedrock is predominantly sandstone of medium strength. Above the bedrock cutting the slope is extremely steep (50°) however it is very densely covered in vegetation and no obvious signs of erosion or instability were noted. From Ch. 134 the rock cutting is formed at a steep batter, due to steeply west dipping joint defects (-55°/315°mN), with the soil slope above at approximately 35° and densely vegetated, see Photo: 3.

Mapping along the rock face identified the bedrock to consist of near horizontally bedded, closely bedded and massive, sandstone units with several zones of very closely (<150mm) spaced near vertical joint defects, see Photo: 4. The bedrock was predominantly moderately weathered and of low to medium strength. The orientation of the closely spaced defect zones were measured in an attempt to provide further detail about the bedrock and sea floor through the western side of the site. The jointed zones were generally between 0.5m and 1.0m in width however one large zone, potentially consisting of several overlapping or converging zones, extends from Ch. 28 to Ch. 39. The location and orientation of these defects are detailed in Figure: 1, along with an estimated trace out through the location of the proposed sea wall.
3.3 Test Results:

Borehole 1 was drilled in the centre of the site (Ch. 118), close to the existing commuter wharf pathway and ramp. This bore intersected clay fill with sandstone gravel and boulders from surface to 2.40m depth. Low strength, highly weathered, fine grained, grey sandstone interpreted as bedrock was intersected to 2.70m depth where core drilling was started. This sandstone unit extended to 4.10m depth and was medium strength from 2.70 to 2.95m depth and then predominantly of low to very low strength. Several very steeply (70-85°) dipping joint defects were identified along with extremely weathered, extremely low strength seams. From 4.10m to 6.95m depth an interbedded sequence of shale and sandstone was intersected which was dominantly of very low strength with occasional thin extremely low strength clay seams and low strength rock horizons. This unit contained iron and clay coated bedding plane defects along with several moderately (40°) dipping joint defects. Below 6.95m fine grained, thinly bedded sandstone of medium strength was identified before the borehole was discontinued at 7.50m depth within this unit.

Borehole 2 was drilled at the southern end of the site (Ch. 171), within the car parking area near the cargo wharf. This bore intersected clayey sand fill with gravel and sandstone boulders to 1.40m then loose sandy fill with some clay, gravel and boulders. At 4.36m depth auger/wash bore refusal occurred and core drilling was started. The core drilling identified low strength, extremely to highly weathered, fine grained, sandstone bedrock with a thin zone of extremely low strength rock from 4.45m. This unit also contained closely spaced bedding plane defects and several steeply dipping joint defects. At 6.50m depth interbedded sandstone and shale bedrock was intersected that was of very low to low strength and contained numerous moderately (30-65°) dipping joint defects and gently dipping bedding plane defects. Below 7.90m depth a low to medium strength horizon of sandstone bedrock was intersected before the borehole was discontinued at 8.35m depth in this unit.

Borehole 3 was drilled through the pedestrian walkway at the northern end of the site (Ch. 39). This bore intersected sandy clay fill with boulders from surface to 2.50m depth where extremely low to very low strength, fine grained sandstone bedrock was encountered. At 4.00m depth core drilling was started in low strength sandstone bedrock. From 4.40m this unit was extremely low to very low strength before grey very low strength interbedded siltstone and shale was intersected at 5.00m depth. A thin low strength sandstone horizon was intersected below 5.90m depth before interbedded sandstone and shale of very low to low strength was intersected from 6.15m. This horizon contained moderately (20-50°) dipping joint defects along with numerous gently dipping bedding plane defects to 7.10m depth. Light grey, fine grained, sandstone of low strength was then intersected. This unit contained numerous sub-horizontal bedding plane defects and moderately dipping joint defects along with a thin shale horizon from 8.10m depth. The borehole was discontinued at 9.0m depth within this sandstone horizon.

Laboratory testing of the rock core samples confirmed the strengths assessed in field and the results of this testing are included on the geotechnical bore logs.
3.3 Test Results: (continued)
The probe tests were conducted at four cross-sections along the proposed sea wall alignment. Section Line ‘A’ at Ch. 40, Line ‘B’ at Ch. 70, Line ‘C’ at Ch. 115 and Line ‘D’ at Ch. 155. These tests in conjunction with a rough tape survey of the seafloor were used in an attempt to determine the depth of sediment overlying the bedrock surface. Whilst not definitive these tests suggest the bedrock surface is gently sloping and then drops away suddenly between 10m and 13m to the west of the existing seawall. The drop off was not identified through the centre of the site as the depth of the water did not allow testing. The drop off is interpreted to represent a small submarine cliff along the edge of a foreshore rock platform, likely formed within a sandstone unit, as seen around adjacent shore lines in this area. The rocky and sandy sediments overlying the bedrock form a relatively gently north-west sloping sea floor with slight slope increases noted in several locations.

4. COMMENTS:

4.1. Geotechnical Assessment:
The investigation identified that the site is underlain by deeply weathered sandstone and shale bedrock with a rocky foreshore and then overlying sandy sediments extending to the north-west. The sediments show a generally gentle north-west dip with a slight steepening beyond the low tide level. The bedrock appears gently dipping out to a buried, very steeply sloping, 2.0m high sub-marine cliff, located approximately 10 to 13m west of the existing sea wall. This cliff is interpreted to represent the outer edge of a foreshore bedrock platform, formed by a sandstone unit identified within the boreholes and noted around adjacent foreshore locations. Below this platform edge the bedrock surface is poorly defined as a result of limited investigation due to water depth. However it is interpreted from an understanding of local geology, borehole logs and the pile installation records to be gently west dipping as a series of thin ledges with low cliffs formed by more resilient sandstone units. The northern portion of the proposed new sea wall appears to be located on the foreshore platform however the southern three-quarters will be located to the west and therefore founded below the break in slope.

The bedrock is relatively deeply weathered where testing occurred adjacent to the existing sea wall and it is expected to be similar to more deeply weathered in the location of the proposed wall. The bedrock of sandstone and interbedded sandstone and shale horizons is near horizontally bedded and contains both very steeply (70-85°) dipping to sub-vertical joint defects along with moderately (30-40°) dipping joint defects. The bedding plane and very steeply dipping joint defects are not expected to result in any significant stability issues however the moderately dipping joints could result in small scale wedge or block slide detachment of the bedrock along the identified submerged platform edge. A moderately sloping defect in the bedrock has resulted in previous instability, similar to that expected, in the road cutting at the southern end of the site.

The jointed zones identified during the mapping of the rock cutting to the east of the site show increased weathering and fracturing in the rock mass. Therefore where these zones are intersected in the sea wall alignment, additional excavation depth for construction of a footing or pile drilling depth may be required.
4.1. Geotechnical Assessment: (continued)
The actual increase in weathered depth is difficult to determine without further targeted testing however for the larger ‘swarm’ noted near the northern end of the site the increased weathering may be expected to extend up to 3.0m depth below the surrounding bedrock surface. It is expected that the bedrock will show these jointed zones as shallow gullies with increased sediment thickness. The approximate alignment of these jointed zones has been traced from the road cutting across to the sea wall alignment, see Figure: 1.

The proposed design will place the road pavement adjacent to the sea wall therefore compacted fill and significant lateral loads will be placed on the supporting geology and sea wall. To ensure the long term stability of this wall and the road, without significant ongoing remedial works, will require a robust retaining wall system. It is not considered likely that a sandstone rock wall, as utilised around the Public Wharf and mini-market to the north of the site or the car park at the southern end, will be suitable to provide this level of long term stability.

It appears that the boulder wall supporting the existing car park at the southern end of the site may have settled into the sea floor sediments, exposing the compacted fill to ongoing erosion. A similar problem is expected to occur should a similar method be utilised for the new sea wall, however the impact to a road pavement located adjacent is expected to be more significant. Excavation of a footing down to bedrock may be suitable to reduce the likelihood of this occurrence, however it would not guarantee it.

4.2. Design and Construction Recommendations:
The proposed sea wall will pass along a location which appears to contain a buried submarine cliff formed in the bedrock along the outer edge of a foreshore rock platform. This platform edge closely matches the local topography and strikes south-west through the site approximately 10 to 13m to the west of the existing sea wall. This places the new wall partly above the crest of the cliff and partly below. The site geological conditions with respect to the proposed sea wall are outlined in Figure: 2.

Considering the deeply weathered nature of the bedrock and the joint defects identified within it, it is considered likely that the bedrock close to the edge of the foreshore platform could become unstable through wedge or block slide style instability. This instability could occur as a natural process, very slowly and at any time in the future. At the northern end of the site, the sea wall is proposed upon the platform and close to the crest of the cliff, therefore the location and founding depth of the wall is critical to ensure its long term stability.

It is considered that this northern portion of the wall should either be moved west to be founded below the bedrock cliff or its footings extended to ensure suitable socket into the sea bed to provide lateral resistance below the base of the platform edge. Additional loading at the crest of the cliff could increase the potential for detachment along the joint defects.
4.2. Design and Construction Recommendations: (continued)

It is considered likely that a sea wall retaining system utilising piles will be the most practical given the depth of water and sediment on the site. Whilst the bedrock is deeply weathered it can be seen from the boreholes that the surface of the bedrock may contain a capping of medium to high strength iron cemented bands. These could create significant problems for a driven style of pile/retaining wall (i.e. sheet piling). It is considered likely that a boulder style sea wall will not provide suitable long term stability.

In designing the sea wall retaining system the following parameters are suggested for the soils/weathered rock on this site, without surcharge loading:

<table>
<thead>
<tr>
<th>Soil/Rock Unit</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$\phi$</th>
<th>$E$ (MPa)</th>
<th>$K_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose Sand</td>
<td>16</td>
<td>30</td>
<td>15</td>
<td>0.33</td>
</tr>
<tr>
<td>Low Strength bedrock</td>
<td>24</td>
<td></td>
<td>100</td>
<td>0.15</td>
</tr>
<tr>
<td>Backfill (drainage – medium dense)</td>
<td>20</td>
<td>35</td>
<td>25</td>
<td>0.24</td>
</tr>
<tr>
<td>Backfill (dense compacted granular)</td>
<td>22</td>
<td>38</td>
<td>50</td>
<td>0.27</td>
</tr>
</tbody>
</table>

In suggesting these parameters it is assumed that the sea wall will be fully drained and it is envisaged that suitable subsoil drains would be provided at the rear of the wall footings to allow for tidal fluctuations. If this is not done, then the wall should be designed to support full hydrostatic pressures in addition to pressures due to the soil backfill.

Where pile footings are used in construction of the sea wall, the depth of piles will be governed by both the location of the pile with respect to the identified geology, the depth of sandy sediments and the weathering within the bedrock. The jointed zones identified in the bedrock cutting for McCarrs Creek Road are expected to increase the depth to the bedrock surface and also the depth of weathering. As such it is recommended that geotechnical supervision of pile drilling/installation occur to ensure that suitable depth of embedment is achieved. Based on the site investigation results the sea floor in the location of the sea wall is generally expected to be located at R.L. -2.0m with the bedrock surface at R.L. -5.0m. The jointing is considered to potentially increase the depth to low strength bedrock by up to 3.0m.

The interbedded sandstone and shale bedrock of low strength is considered suitable for an allowable bearing pressure of 800kPa whilst the medium strength sandstone identified below 7.0 to 8.0m depth in borehole 1 and 2 and interpreted at <10.0m in borehole 3 is considered suitable for an allowable bearing pressure of 1500kPa. It should be noted that the foundation design parameters/criteria outlined in Pells et. al 1978 should not be utilised for this site due to the geological differences.

The sandy foreshore soils located through the area of the proposed sea wall are assessed as being of generally medium density at 0.5m depth below the sea floor surface and therefore are considered suitable for an allowable bearing pressure of 150kPa.
4.2. Design and Construction Recommendations: (continued)

The use of ‘dead-man’ anchors to provide lateral stability to the crest of the retaining wall would assist to reduce the depth of embedment into weathered bedrock required for piles. In the use of cantilever design of piles a passive earth pressure of 400kPa is considered suitable at >2.0m depth below the surface of the bedrock, which is assessed as dominantly of low strength. Due to the variability expected in the sea bed onsite geotechnical supervision of pile drilling will be required to ensure a suitable socket is achieved on all piles.

As there is the potential for scouring of the sandy sediments following the changes to site conditions and sea bed topography it is not recommended to rely on any passive resistance from the sediments.

5. CONCLUSIONS:

The site investigation identified sandy sediments overlying deeply weathered bedrock which contains a buried cliff line close to the alignment of the proposed new sea wall. Whilst the majority of the sea wall appears located directly to the west and down slope from this break in slope the northern portion will be located above and adjacent to the crest. This could create stability issues for this portion of the wall depending on the style of wall utilised.

The bedrock below the site consists of generally highly weathered, low strength, sandstone and interbedded shale units with several localised joint swarms which are expected to result in zones of increased weathering and depth to bedrock. The bedrock is generally interpreted to be overlain by up to 4.0m of sandy sediments whilst the bedrock surface is likely to contain an iron rich capping of medium to high strength. It is therefore considered that bored piles will be the most suitable method to provide long term stability in this location.

The site investigation was limited on the western side of the site by the local topography and sea level however several methods were used to provide an interpretation of geological conditions including depth to bedrock through this part of the site. Should a more detailed assessment of bedrock depth and strength be required directly along the wall alignment then it is considered that an offshore geophysical survey coupled with additional boreholes drilled offshore would be the most suitable given the identified site conditions. The cost of offshore boreholes is expected to be relatively high.

Troy Crozier
Senior Engineering Geologist
MEng. Sc, BSc (Geol)

Reviewed by:
Peter Crozier
Principal

CROZIER – Geotechnical Consultants
LEGEND

CROZIER - GEOTECHNICAL CONSULTANTS

SCALD 1:200 # AD
PREPARED FOR: Pittwater Council.

PROPOSED SEA WALL & WATERFRONT DEVELOPMENT
CHURCH POINT COMMUNITY WHARF
PROJECT #: 1015-195 DATE: 03/09/2012

FIGURE 2
Appendix 1
### Description of Strata

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sampling</th>
<th>In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>Asphalt</td>
<td>SPT 0.50</td>
</tr>
<tr>
<td></td>
<td>FILL: Orange-brown, low plasticity, moist, clay fill with sandstone gravel and boulders. (Sandstone fill)</td>
<td>SPT</td>
</tr>
<tr>
<td>1.00</td>
<td>1.10</td>
<td>* Large sandstone boulders</td>
</tr>
<tr>
<td>2.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.70</td>
<td>SANDSTONE: Low strength, highly weathered, grey with purple iron oxide staining, sandstone bedrock.</td>
<td>2.70</td>
</tr>
<tr>
<td>3.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Sampling and In Situ Testing

**RIG:** Tracess - Track Mounted Rig  
**DRILLER:** Scott  
**LOGGED:** JB  

**METHOD:** Auger (0.0 - 2.50m and Rock Roller 2.50 - 2.70m)  
**GROUND WATER OBSERVATIONS:**  
**REMARKS:**  
**CHECKED:** TMC  

---

CROZIER - Geotechnical Consultants, January 2012
### Test Bore - Diamond Core Bore Logs

**Client:** Pittwater Council  
**Date:** 11/09/2012  
**Borehole:** 1  
**Project:** Church Point Commuter Wharf  
**Project No.:** 2012-195  
**Dip:** 90°  
**Azimuth:**  
**Location:** McCarrs Creek Road, Church Point  
**Surface Level:** R.L. 1.80  
**Sheet:** 2 of 3

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Core</th>
<th>Degree of Discontinuities</th>
<th>Rock Strength</th>
<th>Fracture Spacing</th>
<th>Sampling and In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.70</td>
<td>BEGIN CORES BOREHOLE at 2.70m depth</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>GNEISS: orange, distinctly bedded, fine grained</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.50</td>
<td>NO CORE 3.54m - 3.60m</td>
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<tr>
<td>3.65</td>
<td>Rounded gravels, conglomerate zone 3.50m to 3.60m</td>
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</tr>
<tr>
<td>4.00</td>
<td>GNEISS: orange, distinctly bedded, fine grained</td>
<td></td>
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</tr>
<tr>
<td>4.10</td>
<td>SHALE: dark grey, fine grained, laminated</td>
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<tr>
<td>5.00</td>
<td>SHALE: dark grey, fine grained, laminated</td>
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<tr>
<td>5.10</td>
<td>SHALE: dark grey, fine grained, laminated</td>
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<tr>
<td>6.00</td>
<td>SHALE: dark grey, fine grained, laminated</td>
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<tr>
<td>6.30</td>
<td>SHALE: dark grey, fine grained, laminated</td>
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<tr>
<td>7.00</td>
<td>SHALE: dark grey, fine grained, laminated</td>
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</table>

**Rig:** Traccass - track mounted rig  
**Driller:** Scott  
**Logged By:** JB  
**Type of Boring:** NMLC  
**Casing:**  
**Water Observations:**  
**Comments:**  

---

**Water Observations:**

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**Discontinuities:**

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**Rock Strength:**

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<td>7.00</td>
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**Fracture Spacing:**

<table>
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**Sampling and In Situ Testing:**

<table>
<thead>
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<th>Depth (m)</th>
<th>Test Results and Comments</th>
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<td>7.00</td>
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Test Bore - Diamond Core Bore Logs

Client: Pittwater Council  Date: 11/09/2012  Borehole: 1

Project: Church Point Commuter Wharf  Project No.: 2012-195  Dip: 90°  Azimuth:

Location: McCarrs Creek Road, Church Point  Surface Level: R.L. 1.80  Sheet: 3 of 3

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Core</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.00</td>
<td>ENDS BORE - as above</td>
</tr>
<tr>
<td>7.50</td>
<td>END CORED BOREHOLE at 7.50m</td>
</tr>
<tr>
<td>8.00</td>
<td></td>
</tr>
<tr>
<td>9.00</td>
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<tr>
<td>10.00</td>
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<td>11.00</td>
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Discontinuities

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<tr>
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<th>J</th>
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Degree of Weathering

<table>
<thead>
<tr>
<th>Rock Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fracture Spacing</td>
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<td></td>
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Sampling and In Situ Testing

<table>
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<tr>
<th>Sample Type</th>
<th>Core Rec %</th>
<th>RPD %</th>
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</table>

Test Results and Comments

Rig: Traccess - track mounted rig.  Driller: Scott  Logged By: JB

Type of Boring: NMLC  Casing:

Water Observations: NIL

Comments:
### TEST BORE REPORT

**CLIENT:** Pittwater Council  
**DATE:** 11/09/2012  
**BORE No.:** 2  
**PROJECT:** Church Point Commuter Wharf  
**PROJECT No.:** 2012-195  
**LOCATION:** McCarrs Creek Road, Church Point  
**SURFACE LEVEL:** R.L. 1.70  
**CHAINAGE:** 171m

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Sampling</th>
<th>In Situ Testing</th>
</tr>
</thead>
</table>
| 0.00     | Asphalt  
FILL: Orange-brown, fine to medium grained, moist, clayey sand and sandstone gravel fill with sandstone boulders. | SPT      | 0.50            | SPT 3 / 2 / 1 N* = 3 |
| 1.00     |                                                                                      |          |                 |
| 1.40     | FILL: Loose, brown, fine grained, wet sand with some clay and sandstone gravel and boulders. | SPT      | 1.50            | SPT 3 / 3 / 3 N* = 6 |
| 2.00     |                                                                                      |          |                 |
| 2.20     | *sandstone boulder fill.                                                             |          |                 |
| 3.00     |                                                                                      | SPT      | 3.00            | SPT 6 / 3 / 4 N* = 7 |
| 4.00     |                                                                                      |          |                 |
| 4.10     | AUGER REFUSAL at 4.10m depth, begin cored borehole.                                   |          |                 |

**RIG:** Tracess - Track Mounted Rig  
**DRILLER:** Scott  
**LOGGED:** JB  
**METHOD:** Auger (0.0 - 4.10m) and Rock Roller (4.10m - 4.36m)  
**GROUND WATER OBSERVATIONS:** Soil is wet at 1.40m depth.  
**REMARKS:**

---

*CROZIER - Geotechnical Consultants, January 2012*
**Test Bore - Diamond Core Bore Logs**

**Client:** Pittwater Council  
**Date:** 11/09/2012  
**Borehole:** 2  
**Project:** Church Point Commuter Wharf  
**Project No.:** 2012-195  
**Dip:** 90°  
**Azimuth:**  
**Location:** McCarrs Creek Road, Church Point  
**Surface Level:** R.L. 1.70  
**Sheet:** 2 of 2

### Depth (m) | Description of Core | Degree of Weathering | Discontinuities | Rock Strength | Fracture Spacing | Sampling and In Situ Testing | Test Results and Comments
--- | --- | --- | --- | --- | --- | --- | ---
4.00 | | | | | | |
4.36 | **BEGIN CORED BOREHOLE at 4.36m depth**  
SANDSTONE: Light grey, massive, fine grained. | | | | | |
4.60 | Purple and dark red iron staining. | | | | | |
5.00 | | | | | | |
6.00 | | | | | | |
6.25 | Rounded gravel - conglomerate zone  
Iron rich sandstone band | | | | | Water loss below 5.30m, approx. 15% water return
6.50 | INTERBEDDED SANDSTONE/SLATE: Sandstone is grey, massive to indistinct, the grained, shale bands, and dark grey and black, with some dark red and purple iron cinder staining. | | | | | |
7.00 | | | | | | |
7.90 | SANDSTONE: Grey, fine grained | | | | | |
8.00 | | | | | | |
8.35 | **END OF BOREHOLE at 8.35m depth.** | | | | | |
9.00 | | | | | | |

**Rig:** Tracess - track mounted rig.  
**Driller:** Scott  
**Logged By:** JB

**Type of Boring:** NMCL  
**Casing:**  
**Water Observations:** NIL  
**Comments:**
### Test Bore Report

**Client:** Pittwater Council  
**Date:** 12/09/2012  
**Bore No.:** 3  
**Project:** Church Point Commuter Wharf  
**Project No.:** 2012-195  
**Sheet:** 1 of 2  
**Location:** McCarrs Creek Road, Church Point  
**Surface Level:** R.L. 1.90  
**Chaining:** 39m

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Sampling</th>
<th>In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>Asphalt</td>
<td>SPT 0.50</td>
<td>SPT 3 / 3 / 4</td>
</tr>
<tr>
<td></td>
<td>FILL: Brown, red, and orange-brown, low plasticity, moist sandy clay fill with gravels, sandstone boulders, crushed sandstone fill.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>*more boulders from 1.0m depth, grey, orange-brown, brown and light grey, very low strength.</td>
<td>SPT 1.50</td>
<td>SPT 2 / 9 / 8</td>
</tr>
<tr>
<td>2.00</td>
<td><strong>SANDSTONE:</strong> Extremely low strength to very low strength, highly weathered, light grey, fine grained sandstone bedrock?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.50</td>
<td><strong>SANDSTONE:</strong> Extremely low strength to very low strength, highly weathered, light grey, fine grained sandstone bedrock?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>SPT 3.00</td>
<td>SPT 14 / 21 / 17 (17/100mm)</td>
<td>Refusal Bouncing</td>
</tr>
<tr>
<td>3.80</td>
<td>*auger grinding on low strength sandstone</td>
<td></td>
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<tr>
<td>4.00</td>
<td>AUGER REFUSAL at 4.00m, begin cored borehole.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Rig:** Tracess - Track Mounted Rig  
**Driller:** Scott  
**Logged:** JB  
**Method:** Auger (0.00 - 4.00m) and Rock Roller (4.00 - 4.20m)  
**Groundwater Observations:**  
**Remarks:**  
**Checked:** TMC

---

CROZIER - Geotechnical Consultants, January 2012
### Test Bore - Diamond Core Bore Logs

**Client:** Pittwater Council  
**Date:** 12/09/2012  
**Borehole:** 3  
**Project:** Church Point Commuter Wharf  
**Project No.:** 2012-195  
**Dip:** 90°  
**Azimuth:**  
**Location:** McCarrs Creek Road, Church Point  
**Surface Level:** R.L. 1.90  
**Sheet:** 2 of 2

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Core</th>
<th>Degree of Weathering</th>
<th>Discontinuities</th>
<th>Rock Strength</th>
<th>Fracture Spacing</th>
<th>Sampling and In Situ Testing</th>
<th>Test Results and Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.00</td>
<td>SEGUN CORED BOREHOLE at 4.00m depth. Sandstone - light grey with some dark red and dark orange iron oxide staining, massive, fine grained.</td>
<td></td>
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<tr>
<td>4.20</td>
<td>SEGUN CORED BOREHOLE at 4.20m depth. Sandstone - light grey with some dark red and dark orange iron oxide staining, massive, fine grained.</td>
<td></td>
<td></td>
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<td></td>
<td>70 to 80% water loss below this bottomly weathered zone from 4.00m depth.</td>
</tr>
<tr>
<td>5.00</td>
<td>INTERBEDDED SANDSTONE/SANDSTONE - sandstone is grey, massive, fine grained, shale is dark grey, some dark red and dark orange staining of the core.</td>
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<tr>
<td>5.90</td>
<td>Sandstone band 100mm</td>
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<tr>
<td>6.15</td>
<td>INTERBEDDED SANDSTONE/SANDSTONE - sandstone is grey, massive, fine grained, shale is dark grey, some dark red and dark orange staining of the core.</td>
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<tr>
<td>7.00</td>
<td>SANDSTONE - Light grey with dark orange and dark red iron oxide staining, massive, fine grained.</td>
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<td>7.10</td>
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<tr>
<td>8.00</td>
<td>Shale band 8.10 - 8.40m</td>
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<tr>
<td>9.00</td>
<td>END CORED BOREHOLE at 9.00m depth.</td>
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</table>

**Rig:** Tracess - track mounted rig.  
**Driller:** Scott  
**Logged By:** JB  
**Type of Boring:** NMRC  
**Casing:**  
**Water Observations:** NIL

**Comments:**
NOTES RELATING TO THIS REPORT

Introduction
These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the discussion and comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods
The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties – strength or density, colour, structure, soil or rock and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Particle Size</th>
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<td>Clay</td>
<td>Less than 0.002 mm</td>
</tr>
<tr>
<td>Silt</td>
<td>0.002 to 0.06 mm</td>
</tr>
<tr>
<td>Sand</td>
<td>0.06 to 2.00 mm</td>
</tr>
<tr>
<td>Gravel</td>
<td>2.00 to 60.00 mm</td>
</tr>
</tbody>
</table>

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Under drained Shear Strength kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
<td>Less the 12</td>
</tr>
<tr>
<td>Soft</td>
<td>12-25</td>
</tr>
<tr>
<td>Firm</td>
<td>25-50</td>
</tr>
<tr>
<td>Stiff</td>
<td>50-100</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>100-200</td>
</tr>
<tr>
<td>Hard</td>
<td>Greater than 200</td>
</tr>
</tbody>
</table>

None-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>SPT &quot;N&quot; Value (blows/300mm)</th>
<th>CPT Cone Value (qc-MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>Less than 5</td>
<td>Less than 2</td>
</tr>
<tr>
<td>Loose</td>
<td>5-10</td>
<td>2-5</td>
</tr>
<tr>
<td>Medium dense</td>
<td>10-30</td>
<td>5-15</td>
</tr>
<tr>
<td>Dense</td>
<td>30-50</td>
<td>15-25</td>
</tr>
<tr>
<td>Very Dense</td>
<td>greater than 50</td>
<td>greater than 25</td>
</tr>
</tbody>
</table>

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling
Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods
The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a back hole and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate of short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more that 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable that with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, ect is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90-115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling of in-situ testing. This is a relatively economical means of drilling in Clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed
The test results are reported in the following form.

Non-core Rotary Drilling – the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from ‘feel’ and rate of penetration.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in 50mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

In the tests, a 35mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0-5 MPa) is used in very soft soils in the graphs as a dotted line. The main scale (0-50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10 % in stiff clays.

In clays, the relationship between cone resistance and SPT value is commonly in the range:

\[ q_c \ (\text{MPa}) = (0.4 \text{ to } 0.6) \ N \ \text{(blow per 300mm)} \]

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:

\[ q_c = (12 \text{ to } 18) \ \text{cu} \]

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension roads.

Two relatively similar tests are used.

- Perth sand penetrometer – a 16mm diameter flat-ended rod is driven with a 9kg hammer, dropping 600mm ( AS 1289, test 6.3.3). This test was developed for testing the desity of sands (originating in Perth) and is mainly used in granular soils and filling.

Cone and Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.
• Cone penetrometer (sometimes known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ration have been published by various road Authorities.

Laboratory Testing
Laboratory testing is carried out in accordance with Australian Standard 1289 “Methods of Testing Soil for Engineering Purposes”. Details of the test procedure used are given on the individual report forms.

Bore Logs
The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than straight line variations between the boreholes.

Ground Water
Where ground water levels are measured in boreholes, there are several potential problems.
• In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
• A localized perched water table may lead to an erroneous indication of the true water table.
• Water table levels will vary from time to time with seasons of recent weather changes. They may not be the same at the time of construction as are indicated in the report.
• The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, Sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Engineering Reports
Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg to a twenty storey building). If this happens, the company will be pleased to review the report and the sufficiency of the investigation work. Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the company cannot always anticipate or assume responsibility for
• Unexpected variations in ground conditions – the potential for this will depend partly on bore spacing and sampling frequency.
• Changes in policy or interpretation of policy by statutory authorities.
• The action of contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advise to resolve the matter.

Site Anomalies
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes
Attention is drawn to the document “Guideline for the Provision of Geotechnical Information in Tender Documents”. Published by the institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection
The company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.
APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY

Example of Mapping Symbols