CLONTARF SEDIMENTARY PROCESSES AND FORESHORE STABILITY STUDY

FORESHORE STABILITY REPORT

Report Prepared for
Manly Council

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1. INTRODUCTION

Cardno Lawson Treloar was engaged by Manly Council and the Department of Environment and Climate Change (DECC) to undertake an investigation of the sedimentary processes within the Clontarf/Bantry Bay region and to assess the stability of shoreline structures along Council land within the study area. These two studies have been undertaken concurrently due to the inherent linkages between any shoreline change within the study area and the stability of shoreline structures.

This report addresses the stability of foreshore structures located on public land in the study area. Figure 1.1 is a locality plan of the study area.

A separate report (Cardno Lawson Treloar, 2009) addresses the sedimentary processes.

This foreshore stability report utilised a range of data sources to investigate the stability of seawalls within the study area including:

- Site inspections (July 2008 and February 2009) and discussions with Council and DECC personnel (Appendix A),
- Collection of detailed topographic survey data (July 2008, Appendix B)
- Review of sedimentary processes in the region (as documented in Cardno Lawson Treloar, 2009)
- Acquisition of site specific geotechnical information (Appendix D) and interpretation of this information for stability issues.

The outcome of the assessment provides a detailed understanding of the foreshore stability within the Clontarf/Bantry Bay region and identifies corresponding management actions.
2. STUDY AREA AND CONTEXT

Manly Council is responsible for the management of the portion of the Sydney Harbour shoreline which is located within its Local Government Area (LGA). Manly Council has embarked on the planning and management of coastal and estuarine areas within its LGA by following the NSW Coastline Management Manual (NSW Government, 1990) and Estuary Management Manual (NSW Government, 1992) respectively. The Clontarf/Bantry Bay Estuary Management Plan (referred to in this report as ‘the Management Plan’) (Manly Council, 2008) was completed in May 2008.

The Management Plan identifies:

- the need to understand estuarine sediment transport patterns near Clontarf, and
- to manage the impacts of shoreline erosion and accretion on adjoining structures.

The two purposes of this foreshore stability investigation are generally understood under the strategic category of “Addressing Hazards and Risks Including Climate Change” within the management plan. The strategic management options for sediment transport and beach erosion are addressed under Section 4.4 of the management plan. The intent of understanding and managing the stability of and long-term maintenance of foreshore structures within the study area is described in Section 4.2/AH5.3 and Section 4.5/HR1.2 of the management plan.

The study area lies within Sydney Harbour and covers the shoreline and estuarine environment between Castle Rock Beach and the Spit Bridge. The study area has a wide variety of different users and is subject to the common urban impacts on estuarine ecosystems. The study area includes a swimming enclosure, marina, swing boat moorings, public reserve, private property and urban infrastructure within the immediate vicinity of the shoreline.

Figure 2.1 presents an aerial view of the study area.
3. STUDY APPROACH

The general study approach adopted, summarised in the following sections, included:

- Site Inspections and Discussions with Council/DECC;
- Undertake detail/topographical survey of the study shoreline including foreshore structures;
- Define coastal hazard and design criteria;
- Investigation of shoreline processes;
- Geotechnical investigation surrounding shoreline structures;
- Stability assessment; and
- Identify Management Options for the shoreline structures and Clontarf Swimming Enclosure.

3.1 Site Inspection and Discussions with Council

An inspection of the shoreline within the study area was undertaken by Cardno Lawson Treloar, Manly Council and Department of Environment and Climate Change (DECC) on 22 July 2008 between 2:30 pm and 3:30 pm. A walkover of the study area was undertaken and key features and issues were identified by Council and DECC representatives. Digital photographs (tagged with GPS locations) were taken during the inspection. A photo log is provided in Appendix A. The tidal conditions during the inspection were approaching a low tide of -0.4mAHD (at 4:36pm).

An additional inspection was undertaken by a Cardno Senior Structural Engineer on 12 February 2009 between 8:00 am and 10:00 am in order to review the foreshore structures with respect to the information provided in the geotechnical report contained in Appendix D. A photolog is provided in Appendix A. The tidal conditions during the inspection were approaching a considerably high tide of +0.85mAHD (at 10:10am).

3.2 Site Survey

A detail/topographical survey of the site was undertaken in July 2008. Appendix B presents the documentation of the survey.

3.3 Physical Processes and Coastal Hazard Investigation

Section 4 presents a summary of the key physical processes which influence the shoreline in the study area. A detailed investigation of the shoreline processes in the study area is presented in Cardno Lawson Treloar (2009). The study investigated shoreline processes at Clontarf using data analysis and modelling techniques. Photogrammetric survey data between 1961 and 2007 was analysed together with wave, current and sediment transport modelling to investigate shoreline changes at the site and the processes which have driven this change.

The shoreline structures subject to the physical processes and coastal hazards are described in Section 5.

3.4 Geotechnical Investigations

A geotechnical investigation was undertaken in October/November 2008 generally to examine the subsurface conditions behind and in front of the three noteworthy shoreline structures in the study area and to describe the sand composition at the Clontarf Beach area. An assessment of the stability of the three selected structures has also been undertaken.
With the concurrence of Manly Council, the geotechnical investigation for this report was limited to a high-level investigation to aid in the determination of strategic management options for the subject structures. No significant test pits were excavated behind the walls. Recommendations for more local investigations and/or monitoring are an outcome of this investigation in the discussion of appropriate management options.

The outcomes of the geotechnical engineering assessment are presented in Section 6 and Appendix D.

The structural engineering assessment has been presented in the context of the results of the geotechnical investigation.

3.5 Climate Change Impacts

An assessment of potential climate change impacts up to a 50-year planning period has been undertaken for selected structures. The climate change impact investigations includes changes to the mean sea-level due to sea-level rise (SLR) and also increase wave heights due to higher storm wind speeds and increases in the mean water depth in front of the shoreline structures.

Climate change related impacts are presented in Section 7.

3.6 Structural Engineering Assessment

A general structural engineering assessment has been undertaken for the selected structures based on a visual inspection of their present conditions by a structural engineer, the findings of the geotechnical investigation, the assessed function of the selected structures, the risk associated with failure of the selected structures, and the potential climate change impacts on the selected structures.

The structural engineering assessment is presented in Section 8.

3.7 Management Options for the Shoreline Structures

Based on the outcomes of the investigations described in Sections 3.1 to 3.6, management options, including further structural investigation, monitoring, and/or remediation options, where appropriate, have been presented for the selected shoreline structures.

Management options are discussed in Section 9.
4. PHYSICAL PROCESSES

4.1 Water Levels

4.1.1 Tidal Variations and Anomalies

Water levels in Sydney Harbour are dominated by semi-diurnal astronomical tides with a tidal range of approximately 2m (between Highest Astronomical Tide and Lowest Astronomical Tide). Table 4.1 presents the tidal planes for Fort Denison which is the nearest Standard Port to the Lowest Astronomical Tide (LAT) datum. The LAT datum is at approximately -0.925 mAHHD.

Table 4.1: Tidal Planes for Fort Denison (m LAT)

<table>
<thead>
<tr>
<th>Tide Levels</th>
<th>mLAT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highest Astronomical Tide (HAT)</td>
<td>2</td>
</tr>
<tr>
<td>Mean High Water Springs (MHWS)</td>
<td>1.6</td>
</tr>
<tr>
<td>Mean High Water Neaps (MHWN)</td>
<td>1.3</td>
</tr>
<tr>
<td>Mean Sea Level (MSL)</td>
<td>1</td>
</tr>
<tr>
<td>Mean Low Water Neaps (MLWN)</td>
<td>0.6</td>
</tr>
<tr>
<td>Mean Low Water Springs (MLWS)</td>
<td>0.3</td>
</tr>
<tr>
<td>Lowest Astronomical Tide (LAT)</td>
<td>0</td>
</tr>
</tbody>
</table>

In addition to the astronomical tides, water levels are also influenced by daily, seasonal and inter-annual oceanographic processes. These processes can cause variations to the predicted tide (astronomical) of up to +/- 0.2m.

Storm events can influence water levels through a number of processes, including:

- Inverse barometer effect (water level increase with falling atmospheric pressure),
- Wind induced set-up along the coastline,
- Wave set-up, and
- Meteorological pressure-system induced coastal trapped waves, propagating northward along the continental shelf.

The central regions of the NSW coastline are subject to storm surge during intense storm systems; the most severe are referred to commonly as 'East Coast Lows' (ECL). These storms can form from strong frontal systems passing through the southern Tasman Sea or from remnant tropical weather systems.

Historically, tsunamis generated from distant sub-sea earthquakes, or locally by landslides on the continental shelf, have affected the central to northern NSW coast. For this study site, such events are likely to be very rare with return periods well in excess of 100-years ARI. Geoscience Australia is currently undertaking a study to quantify the tsunami risk for the whole Australian coastline. As yet, there are no coastal planning design guidelines for tsunami on the NSW coast. The highest recorded (Fort Denison) tsunami (0.8m trough to peak) in the NSW region occurred in 1960 and was caused by an earthquake in Chile. This event caused significant damage to the shoreline near Clontarf Point Park which is documented in Manly Council (2007).
4.1.2 Sea Level Rise

General scientific consensus predicts that, under enhanced climate change conditions, sea levels will rise in response to isothermic expansion and melting of polar ice shelves. Predictions of global sea level rise due to the climate change vary considerably and are linked to greenhouse gas emission scenarios. It is impossible to state conclusively by how much the sea may rise, and no policy yet exists regarding the appropriate provision that should be made in the investigation of coastal processes under those conditions, other than to examine a range of cases.

In recent times, the sea level along the NSW has been generally rising. Since 1917, the increase in mean sea level at Sydney (Fort Denison) has averaged approximately 0.9mm per year. Since 1991, the net sea level rise recorded for the next major port to the south of Sydney, Port Kembla has been approximately 1.5mm per year (BoM, 2007). Over a period of 100 years this rate of increase would equate to 9cm – 15cm.

The reported range for IPCC (2007) is for a sea level rise of between 0.18m and 0.59m by 2100. These estimates exclude the potential sea level rise increase that might be caused by a continuation of ice sheet melting in polar regions. The additional sea level rise, if this were to occur, is estimated to be between 0.1 and 0.2m by 2100.

DECC (2007) recommend for year-2100 timeframes that sea level rise scenarios between 0.18m and 0.91m should be considered (see Table 4.2). In February 2009, the NSW Government released a draft sea-level rise policy statement which projects sea-level rise up to 0.4m by 2050, and 0.9m by 2100 compared to the 1990 mean sea level (NSW Government, 2009). The policy projections are based on the latest scientific data which is presented in DECC (2007).

<table>
<thead>
<tr>
<th>Impact Range</th>
<th>Sea-Level Rise (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low Level</td>
<td>0.18</td>
</tr>
<tr>
<td>Mid Level</td>
<td>0.55</td>
</tr>
<tr>
<td>High-Level</td>
<td>0.91</td>
</tr>
</tbody>
</table>

4.2 Wave Climate

4.2.1 Offshore Wave Conditions

The offshore wave climate in the Sydney region is dominated by south to south-east sector swell conditions, though other offshore wave directions can occur. Two general types of weather systems produce the most frequent and largest storm waves along the central and northern NSW coastline. Intense low pressure systems located in the southern Tasman Sea that generally form from systems which have moved north from the southern ocean are the most common coastal storm system. These systems occur two to three times (as severe storms) per year on average, and due to the southerly location of the storm centre, generate onshore propagating waves from the south-east to south sector at the study site. As these waves propagate across the continental shelf into the nearshore region the process of refraction reduces the wave energy (height) that reaches the shoreline.

A second weather system is referred to as an East Coast Low (ECL) storm. These complex weather systems often originate from a tropical low pressure region and generally move southwards down the NSW coast; but have been known to move northward on occasion. They can be particularly damaging to the central-NSW coast because they can form relatively close to the coastline and often generate powerful onshore propagating
waves from the east-north-east to south-east sector. As a result these waves experience less refraction compared to more southerly weather systems and larger waves can impinge on the coastline. They also generate a range of onshore propagating wave directions as they move along the coastline. ECL events can also generate a storm surge, which can further increase the impact on shoreline areas. It has been observed that ECL can occur frequently when conditions are favourable; that is, they tend to be episodic. This was observed in 1974 when two ECL storms damaged the central-NSW coast a few weeks apart. More recently, the June 2007 period featured several intense ECL events, including the storm that caused extensive damage in the Newcastle region.

There is no current consensus on the impact of climate change on coastal storms in the central-NSW coastal region. Recent studies, for example CSIRO (2007) and McInnes et al (2007), present climate change predictions which indicate increased and decreased wind speeds along the NSW coast, depending on the model and/or climate change scenario applied. Of more importance for the NSW coast is the potential change in ECL event frequency or intensity due to climate change. Current understanding on ECL events is limited, although it is widely believed that the ENSO cycle has a significant influence on the frequency of ECL events. Climate change models to date have not been able to investigate changes to wind conditions generated by small scale systems such as ECL events. CSIRO (2007) concludes that for ECL events "model studies do not as yet indicate how the occurrence of east coast low pressure systems may change". The design coastal storm condition for the central NSW coastal region for AR1s greater than 10-years are ECL events. Due to the lack of consensus related to climate change impacts on the frequency and/or intensity of these events it is appropriate to adopt coastal storm conditions based on the current climatology and historical record.

Large offshore waves from the north-east to east sector occur occasionally on the central-NSW coast. These waves often have long periods (Tp = 12 to 16s) and are normally generated from tropical cyclones. These events tend to affect the NSW coastline only a few times each decade.

4.2.2 Nearshore Wave Conditions

Swell wave conditions near the shoreline of the Clontarf region are influenced by the large degree of refraction and diffraction which influence waves as they propagate from offshore, across the nearshore shelf (refraction) and into Sydney Harbour (diffraction). Once inside the heads, waves undergo further refraction and diffraction prior to reaching the shoreline near Clontarf. Swell wave directions are governed by the diffraction and refraction processes therefore there is very little variation in wave direction in the Clontarf region compared to the variation in offshore wave directions. Swell wave breaking occurs very close to the shoreline along Clontarf because of the small wave heights and steep beach slopes.

4.3 Sediment Transport and Storm Erosion

Two broad mechanisms of sediment transport exist for beaches. They are:

- Longshore transport, and
- Cross-shore transport.

Longshore transport refers to the movement of sand along the coastline caused by a current parallel to the beach. These currents are generally caused by waves breaking at some angle to the beach alignment, thereby generating a current acting parallel to the shoreline. It should be noted that, in detail, the plan alignment of a beach is generally not regular. These waves also increase seabed shear stress and cause sediment re-suspension.
Cross-shore sediment transport refers to transport normal to the beach face. During storm conditions cross-shore transport generally removes sand from the upper beach profile and deposits this material in deeper water, sometimes forming a bar that acts to reduce the rate of further beach face erosion. During more moderate wave conditions, swell waves can promote the transport of sand from deeper water onto the beach face – this process occurs over periods of some months.

Long-shore transport is the dominant transport process considered in this study because there is a constant northerly drift of sand in the study area due to swell waves. The large (relative) gradient in swell wave energy from south to north results in the potential for areas of erosion near the southern end of the study area and deposition in more northern areas such as Sandy Bay.
5. **SHORELINE STRUCTURES DESCRIPTION**

5.1 **Physical Environment**

The general study area is detailed in Figure 5.1. The study has considered sediment transport processes within the whole coastal compartment between Grotto Point and the Spit Bridge, but the focus area has been the shoreline between Clontarf Point and north to Sandy Bay (see Figure 2.1).

The study area is situated in the lower reaches of Middle Harbour within Sydney Harbour. The study area has significant environmental, heritage, social and economic value. The catchment is primarily comprised of residential private properties but also features significant areas of road surface and public open space. A full description of the catchment surrounding the Clontarf region is presented in Manly Council (2007).

The estuary itself is a drowned river valley which is generally dominated by oceanic water. During periods of wet weather, the estuary can become stratified with a thin layer of freshwater being present in the Clontarf region (Manly Council, 2007).

Tidal currents driven by the semi-diurnal tide offshore of Sydney provides the dominant estuary mixing process. Tidal currents may also be important in sediment transport processes at particular locations within the general study area; however, tidal currents on their own appear to have little influence on shoreline processes within the study area.

Wave forcing is generated by both local sea and swell waves. Swell waves undergo significant refraction and diffraction as they propagate from offshore to the shorelines within the study area. Swell waves have the most significant effect on shoreline processes between Clontarf Point and the Clontarf Swimming enclosure (see Figure 5.1). Swell waves along the shoreline of the study area have a near constant direction which results in the potential for significant net sediment transport in a north-west direction. Local sea waves, on the other hand, can originate from a range of directions, but have a reduced potential for net sediment transport.

Tsunamis (see Section 4.1) have impacted on the study area in the past, most noticeably was the May 1960 tsunami generated from a large undersea earthquake off the coast of Chile. This event caused significant damage to the shoreline near Clontarf Point Park which is documented in Manly Council (2007).

5.2 **Identified Shoreline Structures on Public Land**

**Figure 5.2** presents a plan view of the public shoreline areas which have structures at the coastal interface. A range of structures are identified in **Figure 5.2** including:

- Conventional vertical and steep-sloped seawalls;
- Headwall and reticulation outlet structures (e.g. stormwater outlets);
- Low concrete retaining walls;
- Low timber retaining walls.

The site inspection indentified seven key shoreline structures on public land which are described in detail in the following sections.
5.2.1 Structure 1 – Seawall between Sandy Bay and Clontarf Marina

This structure is a formal seawall approximately 200m in length which runs along the shoreline between Sandy Bay and Clontarf Marina. Photographs of particular sections of the structure are presented in Appendix A (Photographs 81 to 118). The structure is generally a steep-sloped dimensioned sandstone seawall. The wall is generally vertical to chainage 69 (from the north) at the stormwater outlet with the slope of the wall becoming sub-vertical from chainage 69 to chainage 200. The slope of the wall becomes flatter along the alignment south of chainage 126. For the entire wall, the sandstone units are generally coursed in a stretcher-type bond pattern and grouted into position. Referencing the photogrammetric data dating back to 1961 and unreconciled aerial photography dating back to the early 1950’s, the wall appears to have been extant in all photographic records, indicating its initial construction was prior to the early 1950’s, making it at least of the order of 50+ years old. At an unknown point after its original construction, a concrete coping was added to the wall (see Figure 5.3).

Sandy Bay Road has been constructed behind the wall (Figure 5.4 and Appendix A, Photographs 81 to 118). Sandy Bay Road or one of its predecessors also appears on all of the photographic data dating back to the early 1950’s.

Table 5.1 presents a general summary of the wall condition based on the inspections undertaken by GHD (Appendix D) and a visual inspection by a Cardno Senior Structural Engineer.

Table 5.1: Summary of Structure 1 Condition

<table>
<thead>
<tr>
<th>Chainage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Description and Condition</td>
<td>Sandstone block faced retaining wall with concrete coping, varying from 0 to 2 m in height. Lower wall sections are vertical to near vertical, varying to a face angle of about 45 degree at the higher sections. Erosion of the sandstone blocks and mortar bed is apparent throughout the wall and consistent with that typical to such structures in a marine environment. Ch. 0 to 69: Near vertical wall to the culvert. Ch. 69 to 200: Sub-vertical wall.</td>
</tr>
<tr>
<td>Ch. 25</td>
<td>Mortar bed below top course of sandstone blocks eroded over a length of approximately 10m. Wall height approximately 1 m. (Plate 1 of Appendix D). There is a small pothole behind the wall at this location. Sandy Bay Road is 5+m behind back of coping.</td>
</tr>
<tr>
<td>Ch. 32</td>
<td>Geotechnical Probe able to be pushed 0.5 m behind the face of the wall at a weep hole / mortar gap location. Cobbles observed to rear of facing blocks. Possible indication of voids in the backfill material. Wall height approximately 1.2 m. Sandy Bay Road is 5+m behind back of coping.</td>
</tr>
<tr>
<td>Ch. 35</td>
<td>Cobblesized sandstone backfill visible behind wall. Wall height approximately 1.2 m. Sandy Bay Road is 5+m behind back of coping.</td>
</tr>
<tr>
<td>Ch. 69</td>
<td>Culvert in wall, 1.06 m diameter. Wall height approximately 1.6 m (Plate 2 of Appendix D). Sandy Bay Road is approximately 3m behind back of coping at approximately chainage 70 to the east of the culvert.</td>
</tr>
<tr>
<td>Ch. 85</td>
<td>Indication of possible wall settlement based on block arrangement. 10mm wide crack in concrete crest block, crack extends to base of wall through mortar bed and sandstone blocks. Wall height approximately 2.0 m (Plate 3</td>
</tr>
<tr>
<td>Chainage</td>
<td>Description</td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
</tr>
</tbody>
</table>
| Ch. 90   | Sandy Bay Road is approximately 5m behind back of coping.  
|          | State survey mark at wall coping. |
| Ch. 95   | Eroded mortar bed.  
|          | Sandy Bay Road is approximately 2.5m behind back of coping.  
| Ch. 98   | Near roundabout. Wall approximately 1.8m high.  
|          | Sandy Bay Road is approximately 2m behind back of coping.  
| Ch. 105  | Rotation of penultimate course with respect to top course. Wall is approximately 1.8m high.  
|          | Sandy Bay Road is approximately 3m behind back of coping.  
| Ch. 120  | Rotation of wall crest capping block (concrete). Crest block not in line with other crest blocks. Possible past settlement and rotation. A survey mark is present on the adjacent crest block (Plate 4 of Appendix D).  
|          | Sandy Bay Road is approximately 4m behind back of coping.  
| Ch. 126  | Culvert located at wall toe (see Figure 5.3).  
|          | Sandy Bay Road is approximately 3m behind back of coping.  
| Ch. 175  | Telegraph Pole to the west of a parking pay station. Pinch point of wall with respect to road. Wall face angle approaching 45 degrees. Wall approximately 1.8m high.  
|          | Sandy Bay Road is approximately 1.5m behind back of coping.  

For its age, the wall appears to be generally in reasonable condition with areas of local dilapidation noted in Table 5.1 and discussed in Section 9. From inspection, it appears that remediation work may have been carried out at isolated locations in the past.

5.2.2 Structure 2 – Timber Landscaping Wall (South of Clontarf Marina)

This structure is essentially a landscaping feature to retain soil behind the beach. The structure would provide no shoreline protection during moderate to severe storm events and would likely be severely damaged during such an event. This type of structure has minimal impacts on the coastal processes of the area.

5.2.3 Structure 3 – Wall at Stormwater Outlet

This structure appears to be the most recent shoreline structure on public land in the Clontarf area. The wall has been constructed at the outlet to a stormwater pipe. The structure is a vertical sandstone block faced wall configured in a headwall/wingwall position relative to the stormwater asset, and was likely constructed to minimise stormwater erosion along the shoreline near the pipe outlet. A condition assessment recorded during the geotechnical investigations is presented in Table 5.2.
Table 5.2: Summary of Structure 3 Condition

<table>
<thead>
<tr>
<th>Chainage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Description and Condition</td>
<td>Structure 3 is a sandstone block faced retaining wall with a concrete coping. The wall varies in height from about 0.5 m up to 0.9m. A culvert discharges through the central portion of the wall, with the wall effectively forming a headwall and wing wall to this structure. Timber &quot;landscape&quot; walls extend north and south beyond the ends of Wall 2. Wall 2 appears to be in generally in reasonable condition with some minor weathering of the sandstone blocks and localised loss of mortar (Plate 5 of Appendix D).</td>
</tr>
</tbody>
</table>

5.2.4 Structure 4 – Timber Landscaping Wall (Clontarf Reserve)

A low timber retaining structure is present along much of the shoreline surrounding Clontarf Reserve. The structure is generally well behind the beach area which is exposed to tide and wave action, and as with Structure 2 (Section 5.2.2), this structure is essentially a landscaping feature to retain material behind the beach. Photograph 31 (Appendix A) presents a typical section of the shoreline which has a timber structure at the interface between the sandy beach and Clontarf Reserve. Photograph 119 (Appendix A) presents the most notably dilapidated section of the wall located just to the north of the concrete wall at the swimming enclosure.

5.2.5 Structure 5 – Concrete Landscaping Wall (Clontarf Reserve)

Immediately landward of the Clontarf swimming enclosure, a concrete retaining wall has been constructed at some point in time in the past. No information was available to Cardno to indicate the age of the structure. This structure does not appear to be a seawall structure or a headwall structure, unlike Structures 1 and 3, but it appears to have been constructed to retain soil along the edge of the reserve. Given the tendency for sand and sediment transport and the accretion around the swimming enclosure, this wall may have retained more soil in the past, although the stormwater drainage line most likely delineates the ultimate base of the wall. Photographs 48, 51 and 120-122 (Appendix A) present typical views on the concrete landscaping wall. A general condition assessment is presented in Table 5.3.

Table 5.3: Summary of Structure 5 Condition

<table>
<thead>
<tr>
<th>Chainage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Description and Condition</td>
<td>Structure 5 is a vertical concrete retaining structure, typically of approximately 0.7 m height. A stormwater drainage line discharges through the face of the wall. The wall appears in reasonable condition, though leaning slightly forward in some areas. Heavily corroded reinforcing steel is exposed local areas. (Plate 6 of Appendix D).</td>
</tr>
</tbody>
</table>

In its current configuration, Structure 5 is essentially a landscaping feature to retain soil behind the beach. The structure would provide no shoreline protection during moderate to severe storm events and would likely be severely damaged during such an event. This type of structure has minimal impacts on the coastal processes of the area.

5.2.6 Structure 6 – Seawall at Public Assess Location

Approximately 150m south of Clontarf Reserve, a narrow stretch of shoreline provides public access to the beach from Monash Crescent (the locality of Monash Crescent is shown in Figure 5.4). The public reserve is approximately 5m in width. A sloping rock seawall has been constructed at this location. Photographs 8, 9, 123, and 124 in Appendix A show the seawall. The seawall has a face angle approaching 45 degrees. This section of shoreline is predominantly composed of vertical or near-vertical seawalls on private property. The seawall is sub-vertical and is similar in style to Structure 1 (Section...
5.2.1. This seawall, and the seawalls along surrounding private property are in the active coastal zone and the high water level comes up to the structure. It is likely that these structures have an impact on coastal processes in some form and may have altered the local littoral processes.

5.2.7 Structure 7 – Timber Landscaping Wall (Clontarf Point)

Timber landscaping structures have been installed along the public shoreline at the southern end of the study area. Photos 125 to 127 in Appendix A present some photographs of these structures. As with all timber structures in the study area, these are landscaping features and provide little to no shoreline protection during storm conditions. However, these timber landscaping walls appear to have been installed as part of a vegetation initiative along the public access at the littoral interface.
6. GEOTECHNICAL INVESTIGATIONS

6.1 Overview

Appendix D presents the detailed geotechnical investigation report prepared by GHD Geotechnics. As part of this study, investigations, including hand excavations, were undertaken at the three shoreline structures of significance (see Figure 6.1). A summary of the geotechnical investigations is presented in the following sections.

6.2 Wall Profiles

6.2.1 Wall 1 (Structure 1)

Three locations along this wall were identified for detailed investigation (chainages 20, 40, and 66, see Figure 6.1). As a result of the excavations and Dynamic Cone Penetrometer (DCP) testing at the site, typical indicative cross sections have been developed at these three locations. Figures 6.2 and 6.3 present the indicative cross-sections developed from the site investigations.

6.2.2 Wall 2 (Structure 3)

A single location on this wall was identified for detailed investigation (see Figure 6.1). As a result of the excavations and DCP testing at the site, a typical cross section has been developed at this site. Figure 6.4 presents the indicative cross-section developed for this structure.

6.2.3 Wall 3 (Structure 5)

A single location on this wall was identified for detailed investigation (see Figure 6.1). As a result of the excavations and DCP testing at the site, a typical cross section has been developed at this site. Figure 6.4 presents the indicative cross-section developed for this structure.

6.3 Acid Sulfate Soils

A total of five sediment samples were tested for Potential Acid Sulfate Soils (PASS) and Actual Acid Sulfate Soil (AASS) (see Appendix D). None of these samples returned readings which are considered PASS or AASS (Appendix D).

6.4 General Stability Assessment

A general stability assessment of the investigated seawalls was undertaken. Appendix D presents the full report. The stability assessment is presented in the following sections.

While visual observations indicate that the walls have generally performed adequately to date, Appendix D notes that the walls appeared to be generally founded at shallow depth within a sand profile. Seawall 3 has somewhat deeper embedment, though appears to rely on cantilever action (i.e. toe embedment) for stability. No toe protection works (such as armour rock) were observed in the geotechnical investigation, and Appendix D notes that the removal of the sand at the toes of the walls, by scour action or general beach depletion, would compromise the stability of the walls and could result in failure.

Weepholes and eroded mortar were observed within the sandstone blockwork of walls 1 and 2 (particularly wall 1). The weepholes were generally noted to be above the apparent general tidal range. A rise in sea levels could vary the relative location of the weepholes.
and areas of eroded mortar with respect to the sea level, increasing the potential for loss of backfill material due to the “pumping” action of water lapping in and out of the weepholes on a more frequent basis. An extrapolation of this process is that erosion or passing through of the backfill material could result in slumping of the wall face back into the settlement depression.

6.4.1 Wall 1 (Structure 1)

Appendix D considers the wall to be in moderate condition, with isolated signs of possible settlement visible. The sandstone facing blocks have been weathered and eroded and there are some gaps in the mortar bed between the blocks at some locations. Evidence of mortar bed repair is apparent in some areas.

There are two features of relevance: the cracking at Ch. 85 m and the displacement of the concrete capping block at Ch. 120 m. The crack in the wall at chainage Ch. 85 m was noted because it appears to descend through the concrete crest cap, the mortar and the sandstone blocks. The crack is tight and if settlement has occurred at this location it is likely to have been small.

Displacement and rotation of the capping block may also be an indication of wall settlement. There were no signs within the wall face of movement and it is noted that the topsoil layer extended up to the wall cap. The apparent displacement could simply be the result of poor construction and the block may have been cast in-situ in this state.

An existing survey station adjacent to the wall section around Ch. 120 m was found during the geotechnical investigation (Appendix D). This may be from some previous monitoring of the wall cap.

6.4.2 Wall 2 (Structure 3)

The geotechnical report (Appendix D) indicates wall 2 appears to be in generally in reasonable condition, with some minor weathering of the sandstone blocks and localised loss of mortar.

6.4.3 Seawall 3 (Structure 5)

The wall appears in reasonable condition according to the geotechnical investigation (Appendix D). However, a section of wall has undergone minor rotation. There is no historical monitoring or performance data to assess the rate of rotation. The rotation could have been caused by a specific unidentified past abnormal loading (such as construction activity, or past scouring at the toe of the wall), or it may be associated with initial movement during the construction and backfilling process. However, in the absence of any historical information, it is prudent to assume that this section of wall has moved due to excessive soil and water pressures. As such, it may be expected that the degree of rotation will increase with time, gradually decreasing the wall stability.

Heavily corroded reinforcing steel is visible in local areas of the wall.
7. CLIMATE CHANGE IMPACTS

Climate change may potentially impact on the coastal processes of the study area. The key climate change impacts may be:-

- Increased mean water levels due to SLR,
- Increased local sea and swell wave heights due to increase storm wind speeds, and
- Increase storm runoff rates leading to increase erosion near stormwater outlets.

Of these three impacts, the most significant impact is likely to occur because of increased mean sea-levels. An increase in mean-sea-level will likely increase the swell wave energy at each structure and also increase the period of time that the water level is on the seawall structures. This may result in a number of impacts including increasing scour depths in front of the seawalls.

Key structural stability issues related to climate change are described in Section 8.

In recent years, the NSW Government and CSIRO have undertaken specific research into the potential climate change related to coastal winds, waves and storm surges due to climate change. A number of publications have been released including McInnes et al (2007) and Ranasinghe et al (2007).

At present, there is a reasonable degree of confidence in the estimates of sea-level rise over the next 50 to 100-years. With regard to changes to wind and wave conditions there is considerably more uncertainty in the expected climate change impacts. There is no current consensus on the impact of climate change on coastal storms in the region. Recent studies, for example CSIRO (2007) and McInnes et al (2007), present climate change predictions which indicate increased and decreased wind speeds along the NSW coast, depending on the model and/or climate change scenario applied. Of more importance for the NSW central coast is the potential change in ECl event frequency or intensity due to climate change. Current understanding on ECL events is limited, although it is widely believed that the ENSO cycle has a significant influence on the frequency of ECL events. Climate change models to date have not been able to investigate changes to wind conditions generated by small scale systems such as ECL events. CSIRO (2007) concludes that for ECL events "model studies do not as yet indicate how the occurrence of east coast low pressure systems may change".

7.1 Climate Change Scenarios

Coastal wind scenarios for the NSW coast presented in McInnes et al (2007) indicated that coastal and ocean storm wind speeds may increase by 5 to 10% by 2070. For this study, a climate change scenario for a 50-year planning period (2059) has been developed which includes a 10% increase in the design storm wind speed. Based on long-term measured wind data from Mascot (Sydney Airport, 1939-1999), the 100-year ARI, 10-minute average wind speed from directions between SW and NW is 26.6m/s. This has been converted to a 1-hour storm wind condition based on the techniques presented in USACE (2003).

Swell wave conditions for the 2059 climate condition have been adjusted to account for a 10% increase in the ‘over-water’ wind speed in the Tasman sea. Long-term wave data from the Botany Bay Wave Rider Buoy (WRB) indicates that the 100-year ARI offshore wave height is approximately 10.4m (Hs). The effect on the design 100-year ARI swell wave height from increasing the storm winds by 10% has been estimated using the Jonswip wave growth formula presented in USACE (2003). For the 2059 climate scenario, a 0.3m SLR has been adopted. Table 7.1 presents a summary of the existing and post-2059 100-year ARI coastal storm parameters adopted in this study.
Table 7.1: 100-Year ARI Design Storm Conditions - Climate Scenarios Investigated For Seawall Stability

<table>
<thead>
<tr>
<th>Climate Condition</th>
<th>SLR Rise</th>
<th>Storm Tide</th>
<th>Wind Speed (m/s, SW to NW)</th>
<th>Offshore Wave Height (Hs, m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing</td>
<td>-</td>
<td>1.5m AHD</td>
<td>25.3</td>
<td>10.4</td>
</tr>
<tr>
<td>2059</td>
<td>+0.3m</td>
<td>1.8m AHD</td>
<td>27.8</td>
<td>11.5</td>
</tr>
</tbody>
</table>

The SWAN model presented in Cardno Lawson Treloar (2009) has been used to estimate the present and 2059 100-year ARI storm wave conditions for the seawall structures in the study. Figure 7.1 presents a plan view comparison of the 100-year ARI design wave conditions near the key seawalls for the present and 2059 climate condition.

The shoreline change model SBEACH has been used to estimate the scour depth in front the seawalls following the 100-year ARI storm event. The 100-year ARI storm has been simulated in SBEACH using the approach recommended by Carley and Cox (2003). The SBEACH model was applied to the three profiles presented in Figure 7.1. Based on this simulation, the SBEACH model indicated that the level of scour in front of the seawalls at all three profiles was approximately 0.15m for the present climate, and 0.2m for the 2059 climate scenario. SBEACH does not include the additional scour which may develop due to wave reflection from the hard structures. The seawalls in the study area would reflect nearly all the incident wave energy because of their near-vertical and impermeable character. USACE (2003) provides design recommendations for scour in front of seawalls due to incident and reflected waves. In general, in local sea dominated areas, the scour depth in front of a seawall can be equal to the maximum incident significant wave height. Based on the SBEACH model, all the study seawalls have a maximum incident wave height on the wall of approximately 0.45m for the present climate condition. For the 2059 climate condition, the maximum wave height on the seawalls is approximately 0.55m (Hs).

Along the study area, storm wave conditions are dominated by local sea waves rather than ocean swell waves. As a result, the design wave heights are generally fetch and depth limited. At the three seawall locations investigated in this study, the storm wave heights acting on the seawall structures have little variation along the study area. Table 7.2 presents the 100-year ARI design scour levels due to wave action for the present and 2059 climate scenarios.

Table 7.2: 100-Year ARI Design Scour Conditions – Present and Future Climate Scenarios

<table>
<thead>
<tr>
<th>Climate Condition</th>
<th>Scour Depth (m, below surface)</th>
<th>Approximate Scour Level (July 2008 Survey)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Wall 1 (Ch66m) Wall 2 Wall 3</td>
</tr>
<tr>
<td>Existing</td>
<td>0.5m</td>
<td>-0.55m AHD +0.1m AHD +0.6m AHD</td>
</tr>
<tr>
<td>2059</td>
<td>0.6m</td>
<td>-0.65m AHD +0.0m AHD +0.5m AHD</td>
</tr>
</tbody>
</table>

Comparing the scour levels presented in Table 7.2 with the profiles in Figures 6.2 to 6.4 indicates that the seawall between Sandy Bay and Clontarf Marina is most at risk from structural damage due to undermining of the seawall toe. Based on the geotechnical study (Appendix D), the toe level at the excavation site near Chainage 66m was estimated at approximately -0.4m AHD. This indicates that the seawall is presently at risk of undermining during severe storms and this risk will increase in the future with rising sea-levels and potentially increased storm wave heights.

7.2 Sea Level Rise Impacts

The foreshore around Clontarf is a vulnerable area for impacts associated with sea level rise. Cardno (2008) undertook a comprehensive climate change assessment of the Manly LGA and Clontarf was one of the areas that was identified to be most at risk from impacts associated with ocean inundation under sea level rise scenarios. A relatively large area of
the Clontarf area is below 3m AHD. In particular, the area surrounding Clontarf Reserve has significant portions of land at less than 2m AHD.

At present, the 100-years ARI water level for Sydney Harbour is approximately +1.5m AHD. **Table 7.3** presents a summary of the typical crest elevations of the structures along the Clontarf foreshore identified in this study.

**Table 7.3: Typical Crest Elevations of Shoreline Structures in the Study Area**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Typical Crest Elevation (mAHD)</th>
<th>~100 year ARI WL (2009) (mAHD)</th>
<th>~100 year ARI WL (+50 year planning period) (mAHD)</th>
<th>Potential Overtopping Issue?</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.9</td>
<td>1.5</td>
<td>1.9</td>
<td>Possible Minor</td>
</tr>
<tr>
<td>2</td>
<td>1.3</td>
<td>1.5</td>
<td>1.9</td>
<td>Yes</td>
</tr>
<tr>
<td>3</td>
<td>1.5</td>
<td>1.5</td>
<td>1.9</td>
<td>Yes</td>
</tr>
<tr>
<td>4</td>
<td>1.3 to 1.7</td>
<td>1.5</td>
<td>1.9</td>
<td>Yes</td>
</tr>
<tr>
<td>5</td>
<td>1.6</td>
<td>1.5</td>
<td>1.9</td>
<td>Yes</td>
</tr>
<tr>
<td>6</td>
<td>2.4</td>
<td>1.5</td>
<td>1.9</td>
<td>No</td>
</tr>
<tr>
<td>7</td>
<td>2.0</td>
<td>1.5</td>
<td>1.9</td>
<td>Possible Minor</td>
</tr>
</tbody>
</table>

Under the current sea level conditions, Structures 2, 3, 4 and 5 are at risk of significant overtopping and inundation under the 100-years ARI water level condition. During an event such as this, the structures would be at risk of failure due to scouring near the crest of the structures, and also due to an increase in the hydraulic pressure behind the wall as the sea level begins to decrease.

Considering a 50-years planning period, and a potential sea-level rise of 0.3m to 0.4m, all structures, particularly Structures 1 to 5, would be at risk of damage due to overtopping during a major ocean storm event. The potential impact of inundation under sea level rise conditions is discussed further in Cardno (2008).
8. STRUCTURAL ENGINEERING ASSESSMENT

The stated context of the assessment of the shoreline structures is related to the impacts associated with coastal hazards and risks including those associated with climate change. The general hazards of interest as enumerated in the management plan are:

- Beach erosion
- Shoreline recession
- Storms
- Coastal inundation
- Slope and cliff instability
- Longer term hazards from climate change and tsunami.

8.1 Scope of Structural Assessment

The structural assessment is limited to Structures 1, 3, 5, and 6. Refer to Figure 6.1 for the location of these structures.

Structures 2, 4, and 7, although noted in this report, are considered to be landscaping elements. The assessment and maintenance of such minor structures is considered to be encompassed in an assessment of supra-littoral landscapes at the littoral interface outside the scope of this structural investigation. These structures should be understood and assessed in the context of beach erosion and sedimentary transport; failure of these structures poses little or no risk in the context of the strategic management of the area.

The stability of structures of the nature identified in the study area can be generally assessed based on functionality and/or structural adequacy. Functionality encompasses assessment based on the practical operation of the structure – is it in good working order and serving its intended purpose? Structural adequacy encompasses assessment based on theoretical and mathematical modelling of the structure to determine its load-carrying capacity. Assessment for structural adequacy requires an intimate understanding of the construction of the element(s) under scrutiny. As the geotechnical investigation was limited with the concurrence of Manly Council, the limited available data precludes a full analysis for structural adequacy, and the structural investigation is limited to an assessment of the functionality of the subject structures.

This assessment is based on a visual assessment of the structures conducted by Cardno in February 2009 and the finding of the geotechnical investigation by GHD (Appendix D).

This structural assessment is further contextualized with respect to risk identification and management – the process of understanding risk as a function of the likelihood and consequence of a specific occurrence.

8.2 Wall Assessments

8.2.1 Structure 1

Structure 1 is in the active coastal zone. The wall appears generally to have performed its intended function to date with only localised dilapidations and indications of previous repair. Reference should be made to the description of the wall in Section 5. The wall appears to be in a moderate condition not uncommon of seawalls of its age and type.
The wall is vulnerable to general marine hazards and risks associated with those processes and mechanisms that have a tendency to:

1. Erode the in situ material at the toe of the structure, as the wall appears to be founded at shallow depth.
2. Inflict wave actions with significant enough force to inflict mechanical damage on the face of the wall.
3. Erode the wall backfill via water transport of backfill materials through weepholes and eroded mortar joints in the wall face causing the wall to slump into the backfill material (washout/washthrough).
4. Erode the backfill through water overtopping the wall and forming potholes or depressions from mechanically scouring and or inundating the backfill.
5. Erode the wall base through scouring at the outlet of pipes. Of particular concern is the erosion at the outlets at chainage 69 and chainage 126.
6. Erode the backfill via uncontrolled stormwater runoff from behind the wall.

The most likely marine processes to lead to such negative impacts on the walls are discussed in Section 7.

For Structure 1, it is considered possible that negative impacts associated with its relationship to Sandy Bay Road may occur in addition to those from marine hazards. Table 5.1 presents the relationship of the back of the coping strip at the top of the wall with Sandy Bay Road. In the areas where the road is near or within the distance of the wall height to the back of the wall, a theoretical "zone of influence," the wall is at risk of negative impacts from road loadings. The adequacy of the wall and the backfill with respect to the conditions behind the wall can not be assessed without further invasive site investigation. It should be noted that the dilapidations and possible failures associated with road failure are likely to be preceded by indicative symptoms that could be perceived by an appropriately qualified person during regular maintenance inspections.

The likelihood of any dilapidation has a consequence analogue in determining risk. As the consequence of any road failure could jeopardize access for the residents served by the Sandy Bay Road, certainly the risk profile is heightened and precautionary and/or preemptive measures may be in order depending on the risk management expectation of Manly Council.

Structure 1 should also be considered in the context of the nearby slopes above the road. No obvious severe problem with the nearby slope was noted during our brief site inspections; a full investigation of the slopes is outside the scope of this investigation. However, the management of the upper slope should correspond with the management of the seawall structure.

8.2.2 Structure 3

As noted in Sections 5 and 7, Structure 3 at the stormwater outlet appears to be a headwall/wingwall structure that is subject to some tide action but is generally sheltered from any significant wave action.

The wall appears to be functional without any significant dilapidation.

The wall should be generally assessed and managed as a stormwater asset.

From the information provided by the geotechnical investigation (Appendix D) and a visual inspection, the likelihood of wall failure appears to be low and the strategic consequence of failure is likely relatively low (wall height varies from 0.5m to 0.9m).
8.2.3 Structure 5

Noting the sedimentation associated with the swimming enclosure, Structure 5 may have been retaining a larger height in the past, and sediment movements over time may have deposited sand in front of the wall. As noted in Section 5, the wall near the swimming enclosure currently is acting essentially as a landscaping wall outside the active coastal zone.

The wall appears to be functional with the dilapidations noted in Sections 5 and 6 and the Appendix D.

In its current configuration, the wall should be generally assessed and managed as a local landscaping asset, noting that excavation in front of the wall would likely cause instability. The management of the wall should be coordinated with that for the swimming enclosure.

From the information provided by the geotechnical investigation and a visual inspection, the likelihood of wall failure in the short term appears to be low and the strategic consequence of failure is relatively low (wall height approximately 0.7m).

8.2.4 Structure 6

Structure 6 is in the active coastal zone. The wall appears to be functional with no obviously apparent significant dilapidation.

The wall is vulnerable to general marine hazards and risks associated with those processes and mechanisms that have a tendency to:

1. Erode the in situ material at the toe of the structure as the wall appears to be founded at shallow depth.
2. Inflict wave actions with significant enough force to inflict mechanical damage on the face of the wall.
3. Erode the wall backfill via water transport of backfill materials through weepholes and eroded mortar joints in the wall face causing the wall to slump in to the back fill material.
4. Erode the backfill through water overtopping the wall and forming potholes or depressions from mechanically scouring and or inundating the backfill.
5. Erode the wall base through scouring at the outlet of pipes. Of particular concern at the outlet at chainage 69.
6. Erode the backfill via uncontrolled stormwater runoff.

The most likely marine processes to lead to such negative impacts on the walls are discussed in Section 7.

From visual inspection, the likelihood of wall failure in the short term appears to be low and the strategic consequence of failure is relatively low (sub-vertical wall). However, failure would restrict public access until the wall was remediated. Significant dilapidations preceding failure will likely be preceded by indicative symptoms that could be perceived during regular maintenance inspections by an appropriately qualified person.
9. MANAGEMENT OPTIONS

As indicated in the conclusions to Section 8, Structures 1 and 6 should be considered assets of interest with regard to actions for the management plan.

These assets should continue to be regularly monitored at intervals consistent with their importance and risk significance. Given the need to ensure continuous of access for local residents, it would be prudent for Manly Council to contemplate pre-emptive and/or precautionary actions to ensure the continued functionality of Sandy Bay Road.

The recommendations described below are consistent with those objectives addressed in Sections AH5, SE2, HR1, and EU1 of the management plan.

9.1.1 Structure 1

It is recommended that the wall be included as an item of interest as part of ongoing management in line with the objectives of the management plan, and it is prudent to undertake at least continued, regular monitoring of the structure (e.g. on an annual basis or after an ocean storm event).

Given the circumstances of access for the local residents, it would be prudent for Manly Council to contemplate pre-emptive and/or precautionary actions to ensure the continued functionality of Sandy Bay Road.

9.1.2 Structure 3

It is recommended that the headwall structure be monitored and maintained as part of the management of Council’s stormwater assets.

9.1.3 Structure 5

It is noted that the wall is essentially a landscaping element in its current configuration. It is recommended that the wall be monitored and maintained as part of the management of landscaping assets for Clontarf Beach. The management of the structure should be coordinated with that for the swimming enclosure. Cardno Lawson Treloar (2009) presents detailed management options for the swimming enclosure.

9.1.4 Structure 6

The wall should be monitored and maintained with the same recommendations for Structure 1 in Section 8.3.1. We note that the likelihood and consequence of wall failure is not as significant as that for Structure 1.

9.1.5 Sand Berm Shoreline and Structures 2, 4 and 7

Shoreline sections that are predominantly natural sand berms or only have landscaping structures (Structures 2, 4 and 7) will also have to be monitored into the future. Cardno Lawson Treloar (2009) presented an assessment of the sedimentary and shoreline processes in the study area which indicated that the natural shoreline areas of Clontarf are dynamic in nature. Within Sandy Bay, the volume of sub-aerial sand has varied considerably since 1960, and the spit near Clontarf Reserve has undergone a sustained accretion process. The natural shoreline areas around Clontarf are most likely to suffer from erosion and damage during ocean storm events. Council should consider undertaking periodic visual assessment of the shoreline and to also undertake such assessments following significant storm events.
Into the future, sea-level rise will impact on the natural shoreline areas of Clontarf. Cardno Lawson Treloar (2009) presents some projections of shoreline change, particularly between Clontarf Point and the swimming enclosure over a 50-year planning period. It is expected that for sea level rises up to 0.3m, the accretion process near Clontarf spit will continue. Over time, sea-level rises of 0.5m to 0.9m will significantly impact on the infrastructure and land use in the Clontarf region generally. As part of Council’s management of the impact of climate change on the Manly LGA into the future, the Clontarf region will need to be a key focus area.
10. CONCLUSIONS

As part of an investigation of the sedimentary processes within the Clontarf/Bantry Bay region a condition and stability study of shoreline structures along Council land within the study area has been undertaken. As part of this study, a detailed shoreline survey and geotechnical investigation was undertaken.

The physical processes of the study area, including shoreline changes, are described in detailed in Cardno Lawson Treloar (2009) and formed a key component of this study. The objectives of the Estuary Management Plan (Manly Council, 2008) for the Clontarf/Bantry Bay area were also considered in this study.

A variety of shoreline structures are present along the study shoreline including:-

- Steeped sloped seawalls,
- Culvert headwalls,
- Timber landscaping walls, and
- Concrete walls.

This study has focused on the seawalls and headwalls which are designed to provide a degree of shoreline protection to various infrastructure. The timber landscaping walls have been identified and noted but are not relevant to the scope of this structural investigation. Failure of these structures poses little or no risk in the context of the strategic management of the area.

The four structures identified for structural investigation have been constructed over a range of time periods. Overall their present condition is adequate and the primary functions are being achieved. The oldest structures, Structures 1 and 6, while they are unlikely to be designed to present engineering standard and are at risk of damage during to toe erosion in the event of a severe storm, at present these walls appear to be in a moderate condition not uncommon of seawalls of their age and type. The focus of the future management of these structures is regular monitoring of the structures, and also in the case of Structure 1, monitoring the adjoining roadway.

Climate change is likely to have an adverse impact on the stability of all structures within the study area. Increased wave action on the structures, and more significantly potentially increased scour depths at the toe of the wall will reduce the stability of the walls. Under sea level rise conditions, inundation during high water levels will impact on all the within the study area and also on the land use in the foreshore area. However, the observable impacts of climate change on the stability of the shoreline structures is several decades into the future which is likely to be greater than the expected design life of any shoreline structures in the study area.
11. REFERENCES


CSIRO (1998) The Impact of Climate Change on Coastal NSW.


Engineers Australia (2004) Guidelines for Responding to the Effects of Climate Change in Coastal and Ocean Engineering. Prepared by the National Committee on Coastal and Ocean Engineering for Engineers Australia for Engineers Australia, Canberra, Australia.


FIGURES
Figure 1.1

LOCALITY PLAN

Study Area
Manly Council LGA
Figure 2.1

Overall Study Area

Limit of Tidal Delta

Castle Rock Beach

Grotto Point

Overall Study Area

Limit of Council Aerial Photography

Clontarf Point

Clontarf Marina

Swimming Enclosure

Sandy Bay

Fisher Bay

Detailed Study Area

Clontarf Sedimentary Processes and Foreshore Stability Study - Shoreline Structures Report

STUDY AREA

Figure 2.1
Concrete Cap Wall

Cap Wall Added to Sea Wall 1

Ch 126 - Culvert at bottom of wall

Figure 5.3
CHAINAGE 66m

Concrete capping block

Sandstone Block Wall

+1.5m AHD*

0.25m

DCP Ch 66B at crest of wall, deflecting rods at 0.3-0.4m (boulders/cobbles?)

0.45m

1.55m

0.16m Outstand

DCP 66A at toe of wall, refusal at 2.4m depth (possible bedrock at 2.4m)

-0.05m AHD*

Excavation filling with water

Shovel penetrates beneath wall at 0.32m below ground level (approx). Foundation level possibly 0.32m (TBC)

* Approximate Levels from Site Survey

SAND (topsoil)

BACKFILL?

SAND?

NTS

Concrete capping block

Sandstone Block Wall

+1.5m AHD*

0.25m

DCP Ch 66B at crest of wall, deflecting rods at 0.3-0.4m (boulders/cobbles?)

0.45m

1.55m

0.16m Outstand

DCP 66A at toe of wall, refusal at 2.4m depth (possible bedrock at 2.4m)

-0.05m AHD*

Excavation filling with water

Shovel penetrates beneath wall at 0.32m below ground level (approx). Foundation level possibly 0.32m (TBC)

* Approximate Levels from Site Survey
Chainage 40m

- Concrete capping block
- Sandstone Block Wall

DCP Ch 66B at crest of wall, deflecting rods at 0.3-0.4m depth (cobbles/boulders?)

- 0.45m
- 0.28m
- 1.28m

Clayey SAND (topsoil)

- Foundation material not determined

Excavation filling with water

Shovel penetrates beneath wall at 0.44m below ground level (approx.). Foundation level possibly 0.44m (TBC)

Chainage 20m

- Concrete capping block
- Sandstone Block Wall

DCP 20B at crest of wall, deflecting rods at 0.3-0.4m depth (cobbles/boulders?)

- 0.45m
- 0.25m
- 1.07m

Clayey SAND (topsoil)

- Foundation material not determined

Excavation filling with water

- Probing indicates underside of footing at 0.4m below ground level (approx.)

NTS

Clontarf Sedimentary Processes and Foreshore Stability Study - Shoreline Structures Report

WALL 1 - Profile at Chainages 20 and 40
Prepared by GHD Geotechnics

Figure 6.3
SEA WALL 2

Sandstone Block Wall

DCP Ch 66B at crest, deflecting rods at 0.3-0.4m (cobbles/boulders?)

0.15m

+1.45m AHD*

DCP 1A at toe of wall, penetrates to 4.0m depth (bedrock level >4.0m)

0.83m

+0.60m AHD*

0.37m

Excavation filling with water

Base of footing and footing type (concrete/sandstone block? not determined)

0.68m

NTS

SAND?

BACKFILL?

* Approximate Levels from Site Survey
SEA WALL 3

Concrete Wall

DCP 2A at toe of wall, refusal at 2.2m depth (possible bedrock at 2.2m)

DCP 2B at crest. Penetrates to 2.2m depth

FILL. Sand with trace clay

Hand excavation refusal at 0.4m on concrete block

Excavation filling with water

Base of wall not determined

NTS

* Approximate Levels from Site Survey
Wave Output and Scour Analysis Profiles

- Structure 1 - Steep Sloped Seawall (Wall 1)
- Structure 2 - Timber Landscaping Wall
- Structure 3 - Vertical Wall (Wall 2)
- Structure 4 - Timber Landscaping Wall
- Structure 5 - Concrete Wall (Wall 3)
- Structure 6 - Steep Sloped Seawall
- Structure 7 - Timber Landscaping Wall

Clontarf Sedimentary Processes and Foreshore Stability Study - Shoreline Structures Report

PRESENT AND 2059-CLIMATE CONDITION 100-YEAR ARI WAVE CONDITIONS
AND SCOUR ANALYSIS SITES

Figure 7.1
Figure A.1: Location Map Showing Points at which the Following Photographs were Taken
Photos taken on 22 July 2008
(See Figure A.1)
<table>
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<th>Photos Taken on 12 February 2009</th>
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<tr>
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<td><img src="103" alt="Image 4" /></td>
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<td><img src="106" alt="Image 7" /></td>
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<td><img src="109" alt="Image 10" /></td>
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<tr>
<td><img src="112" alt="Image 13" /></td>
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</table>
APPENDIX B

Topographic Survey July 2008
APPENDIX C

Photogrammetric Survey – Aerial Photographs 1961 - 2006
Geotechnical Report
16 March 2009

Cardno Lawson Treloar
Level 2, Cardno Building, 910 Pacific Highway,
Gordon NSW 2072

Attn: Emma Maratea

Dear Emma,

Clontarf Seawall
Geotechnical Investigation

This report presents the results of GHD Geotechnics' geotechnical investigation of three seawalls at Sandy Bay, Clontarf. We understand that the investigation is to be used as an input to the planning process associated with a possible sea level rise due to global warming, with further assessment being conducted by, or managed by, Cardo Lawson Treloar.

The report contains the factual results of the field investigations and laboratory testing, together with comments on the encountered subsurface conditions. Some preliminary comments on seawall stability have also been provided.

We trust this report is sufficient for your current requirements. Please contact either of the undersigned should you have any questions in regard to this report or require further assistance with this project.

Yours faithfully
GHD Geotechnics

Prepared by

Cillian Mc Colgan
Geotechnical Engineer

Reviewed by

Tony Colenbrander
Group Manager – Geotechnics and Dams
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Document History

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<td>17 December 2008</td>
<td>Initial issue to client for comment.</td>
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<td>1</td>
<td>27 February 2009</td>
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Issued by: [Signature]
Date: 27/2/09
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Standard Sheets

Appendices
Appendix A: Site and Test Location Plan
Appendix B: Excavation Photographs
Appendix C: Walkover Photographs
Appendix D: DCP Test Results
Appendix E: Laboratory Test Results
1. Introduction.

This report presents the results of the geotechnical investigation undertaken by GHD Geotechnics (GHD) at Sandy Bay, Clontarf. The site comprises three seawalls of different length, height and construction, and the immediately adjacent sandy beach.

The investigation was undertaken in accordance with the scope of work advised to GHD by David Taylor of Cardno Lawson Treloar (CLT) in a telephone conversation with Bob Batchelder on 31 July 2008. Cardno Lawson Treloar also provided GHD Geotechnics with a detailed scope of works and coordinates of the required investigation sites in digital form on 20 August 2008.

This report has been prepared for the use of CLT in relation to planning purposes and has been based on the provided information and the test locations nominated by CLT.

We have provided information based on conditions as present at the time of our investigation. The potential future affects of climatic variation are outside the current GHD scope of work. Climatic variation affects may include, amongst others, sea level variation, increased flooding, increased storm intensity and increased temperatures, and more indirect effects such as altered wash-off regimes. We understand that CLT is to conduct, or manage, further assessment with regard to potential effects of climate change effects, including sea level rise.

This report should be read in conjunction with the attached General Notes.
2. Scope of Work

The objective of the fieldwork was to provide information on subsurface conditions behind and in front of the seawalls and to obtain sediment samples for laboratory testing.

Specifically, the objectives of the geotechnical investigation were to:

- Obtain near surface sediment samples along the foreshore.
- Obtain sediment samples near the low tide mark.
- Undertake laboratory testing on selected samples to obtain Particle Size Distribution (PSD) and Acid Sulphate Soil (ASS) results.
- Record seawall geometry and the encountered subsurface materials at the toe and crest, at selected locations.
- Undertake excavation at the seawall toe to obtain footing details (if encountered within the depth of excavation).
- Provide preliminary comments on seawall stability, for selected locations.
- Report the results of the investigation.

The sediment sampling locations and the number of PSD tests were nominated by CLT.
3. Site Description

Sandy Bay, Clontarf, is located on Middle Harbour, Sydney, and is surrounded by residential development. The site comprises three seawalls and a beach area as shown on Figure 1, Test Location Plan, Appendix A.

The main seawall (Seawall No. 1) is positioned at the northern end of the site between Sandy Bay and Sandy Bay Road to the east, and extends south to the Clontarf Marina. This wall is about 200 m in length.

The second wall (Seawall No. 2) is located south of Clontarf Marina and adjacent to Sandy Bay Road. This wall is approximately 15 m in length and appears to be the newest of the seawalls investigated. The third wall (Seawall No. 3) is located along the shore in front of a small park about 150 m south of the marina. This wall is approximately 50 m in length. There is a fenced-off public ocean swimming area directly in front of this wall.

A sandstone cliff, extending over a length of about 75 m, is present along the shoreline near the northern end of Seawall No 1. A section of the cliff has been excavated to provide access for Sandy Bay Road.

3.1 Geology

Reference to the 1:100,000 scale Geological Series Sheet for Sydney (9130) indicates Hawkesbury Sandstone underlies the site. This unit comprises medium grained quartz sandstone with minor shale and laminite lenses. The geological map indicates the presence of an area of quaternary deposits comprising medium to fine “marine” sands and coarse quartz sand with varying amounts of shells south of the site.

3.2 Acid Sulphate Soils

Reference to the 1:100,000 Acid Sulphate Soil (ASS) Risk map for Sydney Heads indicates for the site a “low risk” of ASS material between 1.0 m and 3.0 m below ground level or “no known occurrence”. The area represented by “no known occurrence” is located along the coastline and is likely associated with the intertidal range for the bay.
4. Methodology

4.1 General
The fieldwork was performed by a GHD Geotechnical Engineer, who performed in-situ testing, collected the samples, and logged and photographed the seawalls and excavations.

The fieldwork comprised:
- Shallow hand excavation at the toe of the seawall to expose the footing (if encountered).
- Recording of the seawall geometry at selected locations.
- Dynamic Cone Penetration Testing (DCP) to probe for bedrock at the toe of the seawall, and to assist in the assessment of soil consistency at the toe and the crest of the seawalls.
- Hand augered boreholes at the crest of the seawall to enable assessment of soil types.
- Shallow sediment sampling in the beach area.

4.2 Preliminary Activities
Prior to the investigation, a Dial Before You Dig (DBYD) services search was conducted, and underground service plans were obtained and reviewed. The test locations were set out using hand-held GPS, in areas well away from the indicated locations of underground services. A hand-held GPS unit is typically accurate to ±5m.

In accordance with standard GHD procedure, a Job Safety and Environment Analysis (JSEA) was prepared for site work. A project safety briefing of all site personnel was completed prior to the commencement of the field investigations.

The DBYD search indicated that there were no underground services in the vicinity of the works.

4.3 Hand Augered Boreholes
Two hand augered boreholes were drilled to assess the backfill material to the seawalls. Refusal was encountered at 0.3 m to 0.4m depth on cobble fill before the backfill profile could be fully established. Hand excavations were subsequently undertaken at these locations (refer Section 4.4).

4.4 Seawall Excavations
A total of ten excavations were undertaken at locations shown on Figure 1. A summary table of the excavation locations is presented in Table 1 below. All the excavations were carried out either at the crest of the wall in the backfill material, or at the base of the wall to expose the footing. The excavations were dug by hand using a shovel.
Table 1  Excavation Location Summary

<table>
<thead>
<tr>
<th>Excavation</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Seawall No. 1</strong></td>
<td></td>
</tr>
<tr>
<td>Excavation 1</td>
<td>Ch. 20 Crest (backfill)</td>
</tr>
<tr>
<td>Excavation 2</td>
<td>Ch. 20 Toe (foundation)</td>
</tr>
<tr>
<td>Excavation 3</td>
<td>Ch. 40 Crest (backfill)</td>
</tr>
<tr>
<td>Excavation 4</td>
<td>Ch. 40 Toe (foundation)</td>
</tr>
<tr>
<td>Excavation 5</td>
<td>Ch. 66 Crest (backfill)</td>
</tr>
<tr>
<td>Excavation 6</td>
<td>Ch. 66 Toe (foundation)</td>
</tr>
<tr>
<td><strong>Seawall No. 2</strong></td>
<td></td>
</tr>
<tr>
<td>Excavation 7</td>
<td>Crest (backfill)</td>
</tr>
<tr>
<td>Excavation 8</td>
<td>Toe (foundation)</td>
</tr>
<tr>
<td><strong>Seawall No. 3</strong></td>
<td></td>
</tr>
<tr>
<td>Excavation 9</td>
<td>Crest (backfill)</td>
</tr>
<tr>
<td>Excavation 10</td>
<td>Toe (foundation)</td>
</tr>
</tbody>
</table>

With the exception of Excavation No. 9, all the excavations carried out in the backfill material at the seawall crest reached refusal at depths between 0.3 to 0.4m depth in gravely sand with cobbles and trace clay. Excavation No. 9 terminated at 0.4 m on a concrete block.

The footing excavations were carried out until the foundation materials supporting the footings could be observed or the excavation began to fill in with water or running sand. Footing excavations were carried out to depths ranging between about 0.4 m to 0.8 m. A footing excavation was attempted at Seawall No. 1 Ch. 90, but was abandoned due to rapid water ingress causing the excavation to collapse.

All the excavations were backfilled with the excavated spoil.

4.5  DCP Testing

Dynamic Cone Penetrometer (DCP) tests were conducted at a total of 16 locations, including all the excavation locations noted in Table 1 above. Additional DCP tests were undertaken at Seawall No.1 Ch90 m 120 m and 130 m (toe and crest). The DCP tests were undertaken to provide soil strength information and to probe for rock.

DCP readings were generally recorded in blows per 100 mm. However, some probes were conducted within the water at the toe of the seawall structures. Due to fluctuating water levels, a stable reference point was not available and the accuracy of the readings was less precise. In these instances penetration was recorded in blows per 200 mm.
The tests were conducted to a maximum depth of nominally 4m, or terminated at 'practical refusal', that is, where more than 20 hammer blows are required to penetrate an increment of 100 mm or the DCP rod deflected excessively. Where the rods deflected excessively (eg on inferred cobbles etc) the test was repeated nearby in an attempt to increase the test depth. Where repeated attempts resulted in deflecting rods, the testing was terminated at that location and a representative test was reported. The DCP tests were advanced to depths ranging between 0.3 m to 4.0 m. The DCP results are presented in Appendix D.

4.6 Seawall Walkover Survey

A Geotechnical Engineer of this firm carried out a limited visual survey of the current state of the seawalls on the site. The observations are presented in the fieldwork section of this report with some representative photographs included in Appendix B.

4.7 Sediment Sampling

Sediment samples were recovered at 20 locations nominated by CLT, using a hand operated silt pump (i.e. a "yabby pump"). The sampling locations (SS1 to SS20) are shown on Figure 1, Test Location Plan. Particle Size Distribution (PSD) testing was carried out on all of the recovered sediment samples.

In addition, five samples were recovered at test locations nominated by GHD for Acid Sulphate Soil (ASS) testing. These samples were recovered at low tide next to the seawater using the hand operated yabby pump. Potential Acid Sulphate Soil (PASS) and Actual Acid Sulphate (AASS) indicator tests were scheduled on each of the samples. The locations of the ASS samples are shown on Figure 1.

All laboratory testing was carried out in our NATA accredited laboratory. The results are presented in Appendix E.
5. Results

The following sections contain a summary of the investigation results. The appendices should be consulted for additional information.

5.1 Excavation to Expose Seawall Footing

Seawall No. 1

Three excavations were carried out at the toe of Seawall No.1, at Ch. 20 m (wall height 1.07m), Ch. 40 m (1.28m high) and Ch. 66 m (1.55m high). A fourth excavation was attempted at Ch. 90 m (1.9m high) but was abandoned due to rapid ingress of water into the excavation and collapsing excavation. No further excavations were carried out beyond Ch. 90 as the water level at low tide was just below or at ground surface thus making excavation by hand not practicable.

The excavations revealed the wall to be founded at between 0.22 m and 0.44 m depth below ground level at the test locations between Ch. 20 m and Ch. 66 m. A layer of black sand with trace organics was encountered at the base of the footings at all of the excavation locations. The material appears to contain black decomposed fragments of wood. This layer was about 0.1 m thick with clean sand at depth.

The base of the retaining wall footing has been estimated based on the level at which a shovel could be pushed horizontally beneath the exposed footing.

At Ch 66m, the wall footing extended out from the face of the seawall by up to 0.16 m. No extension was observed at the other excavation locations. The seawall geometry at selected locations is presented as Appendix B.

Three more excavations were carried out in the seawall backfill at the same chainages. The backfill to the retaining wall was observed as 0.1 m of topsoil overlying sand with trace clay and sandstone cobbles, glass fragments and concrete rubble. Hand auger excavations and manual excavations both reached refusal at between 0.3 m and 0.4m below ground level in the cobbles. The nature of the retaining wall backfill at greater depth could not be determined using the adopted hand excavation methods.

Seawall No. 2

Two excavations, one at the toe and one at the crest, were carried out for Seawall No. 2. The toe excavation revealed a footing founded at 0.37 m below ground level with the wall footing extending beyond the face of the seawall by 0.68 m (wall height 0.83 m). Rapid water ingress and running sand conditions, possibly associated with the adjacent culvert, prevented the excavation revealing the base of the footing. Clean sand was encountered in the excavation to a depth of 0.4 m.

The excavation carried out in the backfill material behind the retaining wall revealed 0.1 m of topsoil overlying sand with trace clay and gravel sized fragments of sandstone. Manual excavation reached refusal at 0.4 m depth on coarse material.
Seawall No. 3

Two excavations, one at the toe and one at the crest, were carried out at Seawall No. 3, a short section of retaining wall located behind the public swimming pool. The toe excavation was advanced to 0.8 m, with no footing encountered. The face of the seawall was still present at this depth with ingress of water terminating excavation. Clean sand was encountered in the excavation to this depth.

The excavation carried out in the backfill material behind the retaining wall revealed 0.1 m of topsoil overlying sand with trace clay and gravel sized sandstone fragments. Manual excavation terminated at 0.4 m depth on a concrete block contained in the fill.

5.2 DCP Testing

The results of the DCP tests carried out at the site are included in Appendix B.

Seawall No. 1

All of the DCP tests carried out in the backfill material to Seawall No. 1 deflected excessively due to the presence of coarse material in the backfill. Probing to a depth of 0.3 m to 0.9 m was achieved. Generally the backfill material was assessed as loose to medium dense and dense sand over the range tested.

DCP tests carried out in front of the wall indicated variable consistency material. The material consistency has been assessed as generally ranged from medium dense to dense, except at the location of DCP Ch 90 m where apparently very loose material was encountered from 0.5 m to 1.0 m below ground level.

Refusal was encountered in DCP Ch. 40A, DCP Ch. 66A and DCP Ch. 90A at 1.8 m, 2.4 m and 2.4 m below ground level respectively. Whilst DCP refusal could potentially be associated with bedrock, it may also be associated with buried structures or debris, boulders or cemented sand etc.

Seawall No. 2

DCP tests carried out in the backfill material caused the rods to deflect considerably (probably on cobble sized fill material) at shallow depth and the maximum penetration achieved was 0.4 m.

The test at the toe indicated loose material to about 0.5 m, overlying very dense (or possibly cemented) material to 1.5 m depth, and generally medium dense and dense soils between 1.5 m and 3.9 m depth. The test was terminated at 3.9 m depth without reaching refusal.

Seawall No. 3

The DCP test carried out in the backfill material resulted in deflecting rods at shallow depth, with the maximum penetration depth achieved being 0.7 m.

The test carried out at the toe generally indicated dense soils becoming very dense at about 0.5 m. Refusal of the DCP occurred at 2.2 m depth where blow counts were gradually increasing.
5.3 Sediment Sampling

5.3.1 Particle Size Distribution Testing

The results of the Particle Size Distribution testing are presented in Appendix E. The test results indicate that the samples recovered were generally a fine to medium grained sand.

5.3.2 Acid Sulphate Soil Testing

Five samples recovered during the sediment-sampling phase of the investigations were tested for Potentially Acid Sulphate Soils (PASS) and Actual Acid Sulphate Spoil (AASS) indicator tests. The results of the tests are included in Appendix E. Reaction levels for the tested samples ranged from 1 (No Reaction) to 4 (Strong Reaction). pH\textsubscript{W} values (Distilled Water) ranged from 7.85 to 8.62 and pH\textsubscript{FOX} (Hydrogen Peroxide) ranged from 5.85 to 6.74.

Based on these results these samples are not considered PASS or AASS soils.

5.4 Walkover Survey

The results of the walkover are summarised in Table 2 below. Photographs are presented in Appendix C.

Table 2 Results of Wall Walkover

<table>
<thead>
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<th>Chainage</th>
<th>Comments</th>
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<tbody>
<tr>
<td><strong>Seawall No. 1</strong></td>
<td></td>
</tr>
<tr>
<td>General Description</td>
<td>Sandstone block faced retaining wall with concrete coping, varying from 0 to 2 m in height. Lower wall sections have at vertical, varying to a face angle of about 45 degree at the higher sections.</td>
</tr>
<tr>
<td>Ch. 25 m</td>
<td>Mortar bed below top course of sandstone blocks eroded over a length of approximately 10m. Wall height approximately 1 m. (Plate 1)</td>
</tr>
<tr>
<td>Ch. 32 m</td>
<td>Probe able to be pushed 0.5 m behind the face of the wall at a weep hole / mortar gap location. Cobbles observed to rear of facing blocks. Possible indication of voids in the backfill material. Wall height approximately 1.2 m.</td>
</tr>
<tr>
<td>Ch 35 m</td>
<td>Cobble sized sandstone backfill visible behind wall. Wall height approximately 1.2 m.</td>
</tr>
<tr>
<td>Ch. 69 m</td>
<td>Culvert in wall, 1.06 m diameter. Wall height approximately 1.6 m (Plate 2)</td>
</tr>
<tr>
<td>Ch. 85 m</td>
<td>Indication of possible wall settlement based on block arrangement. 10 mm wide crack in concrete crest block, crack extends to base of wall through mortar bed and sandstone blocks. Wall height approximately 2.0 m (Plate 3)</td>
</tr>
<tr>
<td>Ch. 95 m</td>
<td>Eroded mortar bed.</td>
</tr>
</tbody>
</table>
### Seawall No. 2

**General Description**

Wall No. 2 is a sandstone block faced retaining wall with a concrete capping unit. The wall varies in height from about 0.5 m up to 0.9 m. A culvert discharges through the central portion of the wall, with the wall effectively forming a headwall and wing wall to this structure.

Timber "landscape" walls extend are beyond the ends of Wall 2.

Wall 2 appears to be in generally in reasonable condition, with some minor weathering of the sandstone blocks and localised loss of mortar (Plate 5).

### Seawall No. 3

**General Description**

Wall No. 3 is a vertical concrete retaining structure, typically of approximately 0.7 m height.

A stormwater line discharges through the face of the wall.

The wall appears in reasonable condition, though leaning slightly forward in some areas.

Heavily corroded reinforcing steel is exposed in a local area at the crest of the wall. (Plate 6)

In summary, signs of observed distress (cracking, erosion) were contained in Seawall No. 1. Seawalls No. 2 and 3 appear to be in reasonable condition.
6. Discussion - Seawall Condition

6.1 General

While visual observations indicate that the walls have generally performed adequately to date, it is noted that the walls appeared to be generally founded at shallow depth within a sand profile. Wall 3 has somewhat deeper embedment, though appears to rely on cantilever action (ie toe embedment) for stability. No toe protection works (such as armour rock) were observed in our investigation. While we have not conducted any assessment of the mobility of the sands in this area under current or future conditions, we note that the removal of this material, by scour action or general beach depletion, would compromise the stability of the walls and could result in failure.

Weepholes and missing mortar were observed within the sandstone blockwork walls (particularly wall 1). The weepholes were generally above the apparent general tidal range. While we have not assessed the impacts of varying sea levels, we note that a rise in sea levels could vary this condition, increasing the potential for loss of backfill material due to the “pumping” action of water lapping in and out of the weepholes on a more frequent basis. This could result in slumping of the wall face back into the settlement depression.

6.2 Sea Wall No. 1

Generally the wall appears to be in moderate condition, though there are isolated signs of possible settlement visible. The sandstone facing blocks have been weathered and eroded and there are some gaps in the mortar bed between the blocks at some locations. Evidence of mortar bed repair is apparent in some areas.

Two features of note are the cracking at Ch. 85 m and the displacement of the concrete capping block at Ch. 120 m.

The crack in the wall at chainage Ch. 85 m was noted because it appears to descend through the concrete crest cap, the mortar and the sandstone blocks. The crack is tight and if settlement has occurred at this location it is likely to have been small.

The displacement and rotation of the capping block may also be an indication of wall settlement. There were no signs within the wall face of movement and it is noted that the topsoil layer extended up to the wall cap. The apparent displacement could simply be the result of poor construction and the block may have been cast in-situ in this state.

An existing survey station adjacent to the wall section around Ch. 120 m was found. This may be from some previous monitoring of the wall cap.

6.3 Sea Wall No. 2

Sea Wall No. 2 appears to be in generally in reasonable condition, with some minor weathering of the sandstone blocks and localised loss of mortar.
6.4 Sea Wall No. 3

The wall appears in reasonable condition. However a section of wall has undergone minor rotation.

We have no historical monitoring or performance data to assess the rate of rotation. The rotation could have been caused by a specific unidentified past abnormal loading (such as construction activity, or past scouring at the toe of the wall), or it may be associated with initial movement during the construction and backfilling process. However, in the absence of any historical information it is considered prudent to assume that this section of wall has moved due to excessive soil and water pressures. As such, it may be expected that the degree of rotation will increase with time, gradually decreasing the wall stability.

Heavily corroded reinforcing steel is exposed a local area at the crest of the wall.
7. Conclusions and Recommendations

7.1 Seawall No. 1

Based on the results of the subsurface investigation and site walkover survey it is not considered that the sea wall presents a high likelihood of failure in the short term. However, as the recorded depth of embedment of the wall / foundation level is relatively shallow (between about 0.3 m to 0.4 m), the stability of the wall would be compromised should significant erosion occur at the toe.

The main areas of concern highlighted by our investigations was the possible movement of one of the concrete capping blocks on the wall at approximately Ch. 120 m, and cracking near Ch 85 m. It is recommended that a program of periodic survey monitoring of the wall capping block near Ch 120 m and the cracking near Ch 85 m be undertaken by Council. A suitably qualified surveyor could typically take position and level readings at 3-month intervals for 1 year. We would be in a position to select appropriate monitoring points and to review the data once available. We would also be available to advise on action to be taken if it is determined that the wall is moving.

As a general note it appears that the condition of the face of the retaining wall is degraded / weathered and particularly the mortar beds are in need of some repair.

7.2 Seawall No. 2

Based on the results of the subsurface investigation and site walkover survey it is not considered that the retaining wall presents a high likelihood of failure in the short term. The wall appears to have been constructed on a footing that protrudes about 0.7 m out from the face of the wall with the top of the footing at a depth of about 0.4 m, and with a wall height above ground level of about 0.8 m. Any scour below the footing would compromise the stability of the wall.

7.3 Seawall No. 3

Based on the results of the subsurface investigation and site walkover survey it is not considered that the sea wall presents a high likelihood of failure in the short term. The wall appears to extend at least the visible height below ground level. The depth of embedment was not determined by the investigation due to seawater entering the excavation.

Any scour below the footing (if any) or base of the wall would compromise the stability of the wall, which appears to largely rely on cantilever action for stability.

The local rotation of the wall may increase with time, also decreasing stability. If the level of stability and future performance is to be further assessed, a survey monitoring programme could be implemented.
Standard Sheets

General Notes
DCP Testing
Soil Description
Laboratory Testing
The report contains the results of a geotechnical investigation conducted for a specific purpose and client. The results should not be used by other parties, or for other purposes, as they may contain neither adequate nor appropriate information. In particular, the investigation does not cover contamination issues unless specifically required to do so by the client.

**TEST HOLE LOGGING**

The information on the test hole logs (boreholes, test pits, exposures etc.) is based on a visual and tactile assessment, except at the discrete locations where test information is available (field and/or laboratory results). The test hole logs include both factual data and inferred information. Moreover, the location of test holes should be considered approximate, unless noted otherwise (refer report). Reference should also be made to the relevant standard sheets for the explanation of logging procedures (Soil and Rock Descriptions, Core Log Sheet Notes etc.).

**GROUNDWATER**

Unless otherwise indicated, the water levels presented on the test hole logs are the levels of free water or seepage in the test hole recorded at the given time of measuring. The actual groundwater level may differ from this recorded level depending on material permeabilities (i.e. depending on response time of the measuring instrument). Further, variations of this level could occur with time due to such effects as seasonal, environmental and tidal fluctuations or construction activities. Confirmation of groundwater levels, phreatic surfaces or piezometric pressures can only be made by appropriate instrumentation techniques and monitoring programmes.

**INTERPRETATION OF RESULTS**

The discussion or recommendations contained within this report normally are based on a site evaluation from discrete test hole data, often with only approximate locations (e.g. GPS). Generalised, idealised or inferred subsurface conditions (including any geotechnical cross-sections) have been assumed or prepared by interpolation and/or extrapolation of these data. As such these conditions are an interpretation and must be considered as a guide only.

**CHANGE IN CONDITIONS**

Local variations or anomalies in the generalised ground conditions do occur in the natural environment, particularly between discrete test hole locations. Additionally, certain design or construction procedures may have been assumed in assessing the soil-structure interaction behaviour of the site. Furthermore, conditions may change at the site from those encountered at the time of the geotechnical investigation through construction activities and constantly changing natural forces.

Any change in design, in construction methods, or in ground conditions as noted during construction, from those assumed or reported should be referred to this firm for appropriate assessment and comment.

**GEOTECHNICAL VERIFICATION**

Verification of the geotechnical assumptions and/or model is an integral part of the design process - investigation, construction verification, and performance monitoring. Variability is a feature of the natural environment and, in many instances, verification of soil or rock quality, or foundation levels, is required. There may be a requirement to extend foundation depths, to modify a foundation system and/or to conduct monitoring as a result of this natural variability. Allowance for verification by appropriate geotechnical personnel must be recognised and programmed for construction.

**FOUNDATIONS**

Where referred to in the report, the soil or rock quality, or the recommended depth of any foundation (piles, caissons, footings etc.) is an engineering estimate. The estimate is influenced, and perhaps limited, by the fieldwork method and testing carried out in connection with the site investigation, and other pertinent information as has been made available. The material quality and/or foundation depth remains, however, an estimate and therefore liable to variation. Foundation drawings, designs and specifications should provide for variations in the final depth, depending upon the ground conditions at each point of support, and allow for geotechnical verification.

**REPRODUCTION OF REPORTS**

Where it is desired to reproduce the information contained in our geotechnical report, or other technical information, for the inclusion in contract documents or engineering specification of the subject development, such reproductions must include at least all of the relevant test hole and test data, together with the appropriate Standard Description sheets and remarks made in the written report of a factual or descriptive nature.

Reports are the subject of copyright and shall not be reproduced either totally or in part without the express permission of GHD.
This procedure involves the description of a soil in terms of its visual and tactile properties, and relates to both laboratory samples and field exposures as applicable. A detailed soil profile description, in association with local geology and experience, will facilitate the initial (and often complete) site assessment for engineering purposes.

The method involves an evaluation of each of the items listed below and is in general agreement with both Australian Standard AS 1726 (the Site Investigation Code) and ASTM D2487 and D2488.

**MOISTURE**

The moisture condition of the soil is most applicable for cohesive soils as a precursor to the assessment of consistency and workability. The moisture condition is described as:-

- **Dry** (dusty, dry to the touch) **Slightly Moist** 
- **Moist** (damp, no visible water) **Very Moist** or **Wet** (visible free water, saturated condition)

In addition, the presence of any seepage or free water is noted on the testhole logs.

**COLOUR**

Colour is important for correlation of data between testholes and during subsequent excavation operations. The prominent colour is noted, followed by (spotted, mottled, streaked etc.) then secondary colours as applicable. Colour is usually described at as-received moisture condition, though both wet and dry colours may also be appropriate.

**CONSISTENCY / DENSITY INDEX**

This assessment is based on the effort required to penetrate and/or mould the soil, and is an indicator of shear strength. Granular soils are generally described in terms of density index as listed in AS 1726. These soils are inherently difficult to assess and normally a penetration test procedure (SPT, DCP or CPT) is used in conjunction with published correlations. Alternatively, in-situ density tests can be conducted in association with minimum and maximum densities performed in the laboratory.

<table>
<thead>
<tr>
<th>Term</th>
<th>Symbol</th>
<th>Tactile Properties</th>
<th>Undrained Strength $S_u$ (kPa)</th>
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</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>VL</td>
<td>Extrudes between fingers when squeezed in hand</td>
<td>&lt;12</td>
</tr>
<tr>
<td>Loose</td>
<td>L</td>
<td>Easily penetrated by thumb about 30-40 mm. Pick head can be pushed in up to shaft.</td>
<td>12-25</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>MD</td>
<td>Penetrated by thumb 20-30(mm) with moderate effort. Sharp end of pick pushed in 30-40mm.</td>
<td>25-50</td>
</tr>
<tr>
<td>Dense</td>
<td>D</td>
<td>Indented by thumb about 5mm with moderate effort. Pick pushed in up to 10mm.</td>
<td>50-100</td>
</tr>
<tr>
<td>Very Dense</td>
<td>VD</td>
<td>Readily indented by thumb nail. Slight indentation produced by pushing pick into soil.</td>
<td>100-200</td>
</tr>
<tr>
<td>Hard</td>
<td>H</td>
<td>Difficult to indent with thumb nail. Requires power tools for excavation.</td>
<td>&gt;200</td>
</tr>
</tbody>
</table>

**STRUCTURE/OTHER FEATURES**

The soil structure is generally applicable to cohesive soils and mainly refers to the presence or absence of joints and layering. Typical terms use are intact (no joints), fissured (closed joints), shattered (open joints), slickensided (polished joints indicative of movement), and stratified/laminated. In addition, the presence of other features (ferricrete nodules, timber inclusions) should also be noted as applicable.

For granular soils, an assessment of grading (well, uniform or poor), particle size (fine, medium etc.) and angularity and shape may also be given.

**SOIL TYPE**

The soil is described in terms of its estimated grain size composition and the tactile behaviour (plasticity of any fines (less than *0.06 mm))). This system does not differentiate on grading below 0.06 mm, in accordance with the Unified Soil Classification (USC) procedure. However, in some situations a soil can exhibit different characteristics between the undisturbed and disturbed/remolded condition (eg. 'sand' sized particles which break down a clay). The Soil Type generally relates to the latter state but the former condition should be noted where applicable.

Furthermore, as most natural soils frequently are combinations of various constituents, the primary soil is described and modified by minor components. In brief, the system is as follows:-

<table>
<thead>
<tr>
<th>Coarse Grained Soils</th>
<th>Fine Grained Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Fines</td>
<td>Modifier</td>
</tr>
<tr>
<td>&lt;5</td>
<td>omit, or use &quot;trace&quot;</td>
</tr>
<tr>
<td>5-12</td>
<td>describe as &quot;with clay/silt&quot; as applicable</td>
</tr>
<tr>
<td>&gt;12</td>
<td>prefix soil as &quot;silty/clayey&quot; as applicable</td>
</tr>
</tbody>
</table>

(*The 2009 sieve (0.075 mm) is commonly used in practice to differentiate between fine and coarse grained soils.*

Note: For soils containing both sand and gravel the minor coarse fraction is omitted if less than 15%, or described as "with sand/gravel" as applicable when greater than 15%.

The applicable USC symbol may also be given after the soil type description in accordance with ASTM D2487 and D2488.

**ORIGIN**

An attempt is made, where possible, to assess origin (transported, residual, pedogenic, or fill etc.) since this assists in the judgement of probable engineering behaviour. This assessment is generally restricted to field logging activities. An interpretation of landform is a useful guide to the origin of transported soils (e.g. colluvium, talus, slide debris, slope wash, alluvium, lacustrine, estuarine, aeolian and littoral deposits) while local geology and remnant fabric will assist identification of residual soils.

GHD GEOTECHNICS

Specialists in Geotechnical Engineering, Geology, Field Laboratory Testing and Hydrogeology

DYNAMIC CONE PENETROMETER (DCP) TESTING

SCOPE

The Dynamic Cone Penetrometer (DCP) test comprises the measurement of the soil resistance to a steel rod driven into the ground by a dropped weight.

The DCP test is a simple manual test used in both sandy and clayey soils. The test is a measure of the shear strength of the soil at relatively shallow depth.

EQUIPMENT AND METHOD

A general description of the dynamic penetrometer apparatus used by our firm is presented in Australian Standard AS 1289.6.3.2. The equipment utilises a 9kg sliding weight with a drop height of 510mm. It is fitted with a conical tip. The equipment can be adjusted for a fall of 600mm and use of a blunt tip in accordance with AS 1289.6.3.3.

The test data are generally recorded as the number of blows (n) per 50mm of penetration. The test data are processed by our in-house computer software. For specific applications (such as pavement investigations), the data may be collected in the reverse form, i.e. as mm per blow. The results are presented either in tabular or graphic form for reporting purposes.

INTERPRETATION

The interpretation of the DCP results is generally based on the assumption that the measured resistance is a function of soil strength. A profile of soil strength (cohesive soils) or density index (cohesionless soils) can thus be established. The test often can be used to qualitatively indicate the presence of soft or loose zones within a soil profile.

The energy of the system per unit area is similar to that of an SPT approach. Thus, the common relationships of SPT and other parameters (say Dutch cone) can be utilised as a means of estimating soil properties, after appropriate site specific correlation. The interpretations from the test are approximate only, and this is particularly pertinent to sand profiles where the magnitude of confinement stress is important in the assessment of the results.

Interpretation of the DCP penetration rate at depth (up to 5m) must be conducted with due regard to side friction effects. In particular, care must be exercised with soft clay profiles where shaft resistance may have a significant unconservative impact upon the results.

In-situ California Bearing Ratio (CBR) values of clay soil subgrades are sometimes interpreted directly from DCP test results for use in road pavement design. In this case, the correlation between DCP and CBR based on that published in AUSTROADS Pavement Design Manual (1992) may be applied. This correlation should be verified by site specific laboratory testing, where appropriate. In addition, the effects of moisture content variations (in-situ verses design conditions) must be considered, as clearly the DCP test only reflects the shear strength of the soil at the time of testing.
GENERAL

Samples extracted during the fieldwork stage of a site investigation may be "disturbed" or "undisturbed" (as generally indicated on the trial hole logs) depending upon the nature and purpose of the sample as well as the method of extraction, transportation, extrusion and testing. This aspect should be taken into account when assessing test results, which must of necessity reflect the effects of such disturbance.

All soil properties (as measured by laboratory testing) exhibit inherent variability and thus a certain statistical number of tests is required in order to predict an average property with any degree of confidence. The site variability of soil strata, future changes in moisture and other conditions and the discrete sampling positions must also be considered when assessing the representative nature of the laboratory programme.

Certain laboratory test results provide interpreted soil properties as derived by conventional mathematical procedures. The applicability of such properties to engineering design must be assessed with due regard to the site, sample condition, procedure and project in hand.

TESTING

Laboratory testing is normally carried out in accordance with Australian Standard AS 1289 as amended, or RTA Standards when specified. The routine Australian Standard tests are as follows:-

- Moisture Content: AS1289 2.1.1
- Liquid Limit: AS1289 3.1.1
- Plastic Limit: AS1289 3.2.1 (collectively known as Atterberg Limits)
- Plasticity Index: AS1289 3.3.1
- Linear Shrinkage: AS1289 3.4.1
- Particle Density: AS1289 3.5.1
- Particle Size Distribution: AS1289 3.6.1, 3.6.2 and 3.6.3
- Emerson Class Number: AS1289 3.8.1
- Percent Dispersion: AS1289 3.8.2 (collectively, Dispersive Classification)
- Pinhole Dispersion Classification: AS1289 3.8.3
- Hole Erosion (HE): GHD Method
- No Erosion Filter (NEF): GHD Method
- Organic Matter: AS1289 4.1.1
- Sulphate Content: AS1289 4.2.1
- pH Value: AS1289 4.3.1
- Resistivity: AS1289 4.4.1
- Standard Compaction: AS1289 5.1.1
- Modified Compaction: AS1289 5.2.1
- Dry Density Ratio: AS1289 5.4.1
- Minimum Density: AS1289 5.5.1
- Density Index: AS1289 5.6.1
- California Bearing Ratio: AS1289 6.1.1 and 6.1.2
- Shear Box: AS1289 6.2.2
- Undrained Triaxial Shear: AS1289 6.4.1 and 6.4.2
- One Dimensional Consolidation: AS1289 6.6.1
- Permeability Testing: AS1289 6.7.1, 6.7.2 and 6.7.3

Where tests are used which are not covered by appropriate standard procedures, details are given in the report.

LABORATORY

Our laboratory is NATA accredited to AS ISO / IEC17025 for the listed tests.

The oedometer, triaxial and shear box equipment are fully automated for continuous operation using computer controlled data acquisition, processing and plotting systems.
Appendix A

Figure 1 Test Location Plan
Figure 2 Seawall No.1 Ch20m
Figure 3 Seawall No.1 Ch40m and 66m
Figure 4 Seawall No.2 and 3
LEGEND

- SEDIMENT SAMPLE LOCATION
- TEST PIT DUG AT TOE OF SEA WALL
- ACID SULPHATE SOIL SAMPLING LOCATIONS
- WALL

GHD GEOTECHNICS

Figure 1

GHD GEOTECHNICS
Cardno Lawson Treacy
Sandy Bay - Clontarf Sea Wall
Geotechnical Investigation
Test Location Plan

job no. 21-17981
rev no. A

scale 1:5000 for A4

date November 2008

Plot Date: 16 December, 2008 - 13:20 AM
Card File No: 21-17981-FIG001.jpg
Figure 2

Sea Wall 1 at Ch 20m

Cardno Lawson Treloar
Clontarf Sea Wall

SAND

Clayey SAND (topsoil)

SCS S

Probing indicates underside of footing at 0.4m below ground level (approx)

SAND?

BACKFILL?

NTS

DCP Ch 20B at crest of wall deflecting rods at 0.3-0.4m depth (cobbles/boulders?)

DCP 20A at toe of wall penetrates to 3.9m depth (indicating bedrock depth >3.9m)

Concrete capping block

Sandstone Block Wall

CHAINAGE 20m

0.45m

0.25m

1.07m

0.4m
CHAINAGE 40m

DCP 40A at toe of wall, refusal at 1.8m depth (possible bedrock at 1.8m?)

Sandstone Block Wall

Concrete capping block

SAND? (topsoil)

Shovel penetrates beneath wall at 0.44m below ground level (approx). Foundation level possibly 0.44m (TBC)

Excavation filling with water

CHAINAGE 66m

DCP Ch 66B at crest of wall, deflecting rods at 0.3-0.4m (cobbles/boulders?)

Sandstone Block Wall

Concrete capping block

SAND? (topsoil)

Shovel penetrates beneath wall at 0.32m below ground level (approx). Foundation level possibly 0.32m (TBC)

Excavation filling with water

Sea Wall 1 at Ch 40m and Ch 66m

Cardno Lawson Treloar
Clontarf Sea Wall

Scale | nts | Date | 26 November 2008

Figure 3
Sea Wall 2

- DCP 1A at toe of wall, penetrates to 4.0m depth (bedrock level >4.0m)
- Sandstone Block Wall
- DCP Ch 66B at crest, deflecting rods at 0.3-0.4m (cobbles/boulders?)
- Excavation filling with water
- Base of footing and footing type (concrete/sandstone block? not determined)
- NTS

Sea Wall 3

- DCP 2A at toe of wall, refusal at 2.2m depth (possible bedrock at 2.2m)
- Sand?
- Hand excavation refusal at 0.4m on concrete block
- Concrete Wall
- DCP 2B at crest, penetrates to 2.2m depth
- Excavation filling with water
- Base of wall not determined
- NTS
Appendix B

Excavation Photographs
Excavation 1 Ch. 20 Backfill

Excavation 2 Ch. 20 Toe
Appendix C

Photographs
Appendix D

DCP Test Results
**DYNAMIC CONE PENETROMETER LOG SHEET**

**Client:** CLT  
**Project:** Clontarf Seawall  
**Location:** Clontarf, NSW

**PROBE No.** Wall 2A  
**Position:** Seawall 2  
**Elevation:** Toe  
**Chainage:**  
**Offset:**  
**Date:** 24/10/2008  
**Operator:** CMC

### NUMBER OF BLOWS TO PENETRATE 100 mm

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>0.0</th>
<th>0.5</th>
<th>1.0</th>
<th>1.5</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
<th>3.5</th>
<th>4.0</th>
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**Comments:**
Test carried out at toe of wall. DCP terminated, limit of investigation.
Client: CLT
Project: Clontarf Seawall
Location: Clontarf, NSW

PROBE No.  Wall 2B
AS1289 6.3.2 (Cone tip)  510 mm drop height.

Position: Seawall 2
Elevation: Crest

Date: 24/10/2008
Operator: CMC

NUMBER OF BLOWS TO PENETRATE 100 mm

Comments:
carried out behind second seawall. DCP deflected, possibly due to presence of cobbles.

GHD GEOTECHNICS

Template: I:\G21\GeoLAB\DCP Results\GHD_ATN_DCP2007.XLT
File:N:\Aam\Arham\Projects\Geo_Projects\21\2117981 Clontarf Sea wall\Fieldwork\DCP\DCP1B Wall 2.xls
Comments:
Carried out in front of third retaining wall. Terminated at 2.2m, high blow counts
**Client:** CLT  
**Project:** Clontarf Seawall  
**Location:** Clontarf, NSW  

**PROBE No.** Wall 3B  
**Chainage:** AS1289 6.3.2 (Cone tip)  
**Date:** 24/10/2008  
**Operator:** CMC

**Elevation:** Crest  
**Position:** Seawall 3  
**Offset:**

---

### NUMBER OF BLOWS TO PENETRATE 100 mm

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>0.0</th>
<th>0.5</th>
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</tbody>
</table>

**Comments:**

Carried out behind third sea wall

---

Refusal at 0.7 m
Comments:
Carried out in front of wall. Terminated at 3.9m limit of investigation
Comments:
Carried out behind retaining wall. DCP deflecting possibly due to cobbles
Number of blows to penetrate 100 mm

Comments:
Carried out in front of wall. Refusal at 1.78m DCP weight bouncing
Comments:
Carried out behind retaining wall. DCP deflecting possibly due to cobbles.
**Client:** CLT  
**Project:** Clontarf Seawall  
**Location:** Clontarf, NSW  
**Position:** Seawall 1  
**Chainage:** Ch 66 m  
**Date:** 23/10/2008  
**PROBE No.:** DCP Ch. 66A  
**AS1289 6.3.2 (Cone tip)**  
**510 mm drop height.**

**NUMBER OF BLOWS TO PENETRATE 100 mm**

**Comments:**  
Carried out in front of toe of retaining wall. End of probe at 2.4m
Comments:
Carried out behind retaining wall. Three DCP's carried out all refusing between 0.3 - 0.4m
Client: CLT
Project: Clontarf Seawall
Location: Clontarf, NSW
Position: Seawall 1
Elevation: Toe

PROBE No. DCP Ch. 90A
AS1289 6.3.2 (Cone tip) 510 mm drop height.

Offset: 0
Chainage: Ch 90m
Date: 24/10/2008
Operator: CMC

NUMBER OF BLOWS TO PENETRATE 100 mm

Rods fell 0.2 m under static weight of hammer

Comments:
DCP located 1.1m out from toe of wall.

Refusal at 3.8 m
Client: CLT
Project: Clontarf Seawall
Location: Clontarf, NSW

Position: Seawall 1
Elevation: Crest
Offset: Operator: CMC

PROBE No. DCP Ch. 90B
AS1289 6.3.2 (Cone tip) 510 mm drop height.

Date: 24/10/2008

NUMBER OF BLOWS TO PENETRATE 100 mm

Comments:
Carried out behind wall. DCP deflecting, possibly due to cobbles
Comments:
DCP Terminated at 3.8m scheduled depth. Increment increased to 200mm as DCP carried out below water and readings difficult.
**PROBE No. DCP Ch. 120B**

AS1289 6.3.2 (Cone tip)  510 mm drop height.

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<th>Position</th>
<th>Chainage</th>
<th>Date</th>
<th>Operator</th>
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<td>Seawall 1</td>
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**NUMBER OF BLOWS TO PENETRATE 100 mm**

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<tr>
<th>Depth (m)</th>
<th>0.0</th>
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<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
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**Comments:**

Carried out behind wall, DCP bent
Comments:
Carried out in front of wall. 200mm increments as sea water making readings difficult

Refusal at 3.8 m
Client: GHD  
Project: Clontarf Seawall  
Location: Clontarf, NSW  
PROBE No. DCP Ch. 130B  
Chainage:  
Offset:  
Date: 24/10/2008  
Operator: CMC  

NUMBER OF BLOWS TO PENETRATE 100 mm

Depth (m)  
0.0  
0.5  
1.0  
1.5  
2.0  
2.5  
3.0  
3.5  
4.0  

Refusal at 0.3 m  

Comments:  
Carried out behind wall, DCP bent
Appendix E

Laboratory Test Results
Client: Cardno Lawson Treloar  
Project: Clontarf Seawall  
Location: Clontarf, NSW

**PASS Indicator Tests - Hydrogen Peroxide**

<table>
<thead>
<tr>
<th>Test Hole No.</th>
<th>Sample Depth (m)</th>
<th>Soil Description</th>
<th>Reaction (1 to 5) Note 1</th>
<th>Approximate Depth to Water Table (m)</th>
<th>Soil pH (5:1) Mixture</th>
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**Note 1, Reaction Levels:**

1. No Reaction – No effervescence or heat generation;
2. Mild Reaction – Slight effervescence, cold or just perceptible heat generation;
3. Moderate Reaction – Distinct effervescence (bubbling, foaming), cylinder becomes warm;
4. Strong Reaction – Strong effervescence (strong bubbling), steam, cylinder becomes very warm / hot;
5. Extreme reaction – Violent effervescence (very strong bubbling, overflowing top of cylinder), steaming, high heat generation.
Material Test Report

Client: Cardno Lawson Treloar Pty Ltd
Level 9, 10 Pacific Hwy
Gordon NSW 2072

Project: 2117981 Clontarf Seawall

Material Details

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Sample Details

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Particle Size Distribution

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<td>Washed</td>
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Other Test Results

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Comments

Form No: 18980.V1.00
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# Material Test Report

**Client:** Cardno Lawson Treloar Pty Ltd  
Level 9, 910 Pacific Hwy  
Gordon NSW 2072

**Project:** 2117981 Clontarf Seawall

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## Other Test Results

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## Comments
Material Test Report

Client: Cardno Lawson Treloar Pty Ltd
Level 910 Pacific Hwy
Gordon NSW 2072

Project: 2117981 Clontarf Seawall

Material Details

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Particle Size Distribution

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<tr>
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Other Test Results

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Comments

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Material Test Report

Client: Cardno Lawson Treloar Pty Ltd
Level . 910 Pacific Hwy
Gordon NSW 2072

Project: 2117981 Clontarf Seawall

Material Details
Source: N/A
Description: N/A
Specification: N/A
Sample From: N/A
Location: N/A
Sample Method: N/A

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Sample ID: SYD08-2865
Field Sample ID: SS19
Date Sampled: 24/10/2008
Progressive Quantity (tL):

Particle Size Distribution
Method: Sieve Size % Passing Limits
53.0mm
37.5mm
26.5mm
19.0mm
13.2mm
9.5mm
Drying by:
N/A
4.75mm 100
2.36mm 100 100
1.18mm 100 98
Sample Washed
600μm 99 88
425μm 96 60
300μm 66 16
150μm 1 1
75μm 1 0

Other Test Results
Description Method Results Limits

Comments
GHD Pty Ltd  ABN 39 008 488 373
57-63 Herbert St, Artamon NSW 2064
T: 61 2 9462 4700  F: 61 2 9465 4701  E: atnmail@ghd.com.au
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Clontarf Seawall
Geotechnical Investigation

GHD GEOTECHNICS
21/17811/AY953-Rev1
16 March 2009 / Rev 1