Tarban Creek Seawall

Report MHL670

November 1993
TARBAN CREEK SEAWALL

NSW Public Works
Manly Hydraulics Laboratory

Report MHL670, November 1993
PW:J Report No. 93091. ISBN 0 7310 2730 2
Foreword

The work described in this report was carried out by Manly Hydraulics Laboratory for NSW Public Works, Metropolitan Northwest Branch, acting on behalf of the NSW Health Department. The work was carried out and report prepared by R A Cook. The assistance of officers of Survey and Land Information Services who carried out the site survey and staff of Coffey Partners International Pty Ltd who carried out the geotechnical investigation is greatly appreciated. The efforts of officers of Coast and Estuaries Branch who carried a preliminary appraisal of the existing seawall, administered the investigation on behalf of Metropolitan Northwest Branch and contributed suggestions during the project is also appreciated.
Summary

The grounds of Gladesville Hospital contain an area of reclaimed land with a frontage to Tarban Creek. The reclaimed land is retained by an old seawall, approximately 580m long, constructed from dry stacked sandstone blocks. The seawall has deteriorated over time and is now in poor condition. Attempts have been made to repair some failed sections of the seawall but these repairs have not been successful. The owner of the site, the NSW Health Department, wishes to restore the seawall to a satisfactory state or to replace it with a suitable alternative as part of the redevelopment of the site.

Site inspections to determine the condition of the existing seawall and investigations to provide design parameters for the reconstruction of the seawall or construction of a replacement structure were carried out. Design parameters included water levels, wave conditions, a strip survey along the wall and a geotechnical investigation.

It was concluded that the present seawall is collapsing because of erosion of fill from directly behind the wall and that the seawall is beyond economic repair. A number of options for replacement structures were evaluated and indicative cost estimates prepared. It was found that the cost of constructing a sandstone seawall to replace the present wall would be prohibitive due to the high cost of buying sandstone blocks to bring the dimensions of the seawall up to current rock wall practice.

The option recommended as most suitable was a composite scheme comprising 250m of seawall in the area most exposed to wave attack, a sandy beach approximately 50m long to replace the eastern end of the present seawall and lining the creek section of the bank with riprap. The seawall would be of dry rock construction similar to the present seawall and utilise the sandstone blocks recovered from the existing wall supplemented by cast concrete blocks in the rear of the seawall where they would not be visible to increase the thickness of the seawall. The estimated cost of this option is $309,000.
Table of Contents

1. Introduction 6

2. Investigations 7
   2.1 Condition of Existing Seawall
   2.2 Site Survey
   2.3 Geotechnical Investigations
   2.4 Water Levels
   2.5 Wave Climate
   2.6 Currents
   2.7 Construction Constraints

3. Options for Restoration or Replacement of Seawall 14
   3.2 Option 1 - Rock Seawall
   3.3 Option 2 - Gabion Seawall
   3.4 Option 3 - Crib Wall
   3.5 Option 4 - Stable Beach
   3.6 Option 5 - Riprap Armoured Slope
   3.7 Option 6 - Reno Mattress Armoured Slope

4. Preferred Option 19

5. Conclusions and Recommendations 22

6. References 23
List of Tables

2 1  Seawall Inspection
2 2  Still Water Levels for Sydney Harbour
2 3  Design Wave Conditions
2 4  Boat Wave Measurements
3 1  Unit Costs
4 1  Indicative Costs of Seawall Options
4 2  Riprap Rock Grading

List of Plates

1  Existing Seawall - Chainage 81m and 190m
2  Existing Seawall - Chainage 286m

List of Figures

1 1  Site Locality Plan
2 1  Key to Site Inspection And Plates
2 2  Conceptual Failure Mechanism
3 1  Option 1 - Typical Rock Seawall
3 2  Option 2 - Typical Gabion Seawall
3 3  Option 3 - Typical Crib Wall
3 4  Option 4 - Stable Beach
3 5  Option 5 - Riprap Slope
3 6  Option 6 - Reno Mattress Armoured Slope
4 1  Plan of Preferred Option
4 2  Preferred Option - Beach Cross-Section
4 3  Preferred Option - Rock Seawall
4 4  Preferred Option - Riprap in Creek Section

Appendices

A  Survey Results
B  Geotechnical Investigation
1. Introduction

Tarban Creek is a tributary of the Parramatta River, joining the river just downstream of the Gladesville Bridge. It is relatively narrow for much of its length before broadening out into an estuary approximately 100m wide and 800m long. A footbridge over the creek effectively marks the transition from the narrow creek section to the wider estuary section. The seawall which is the subject of this report lies along the southern side of Tarban Creek within the grounds of Gladesville Hospital. The location of the seawall is shown in Figure 1.

The seawall, constructed from a single row of dry stacked sandstone blocks, is approximately 580m long overall with approximately half of its length bordering the wider part of Tarban Creek and half bordering the narrower creek section. The upper limit of the seawall is near to the tidal limit of the creek. The maximum normal tidal range along the wall section is 1.8m.

The primary purpose of the seawall is to retain fill which has been placed on estuarine mudflats to form a level area presently used as playing fields. The section of seawall bordering the creek also protects the bank from flood scour while the section bordering the estuarine section of Tarban Creek protects the bank from wave action.

The seawall has deteriorated over time and is now in poor condition. The owner of the site, the NSW Health Department, wishes to restore the seawall to a satisfactory state or to construct other suitable protection as part of its redevelopment of the Gladesville Hospital site. Following restoration of the bank the Health Department wishes to transfer the care and maintenance of the foreshore area to Hunters Hill Municipal Council.

This study was undertaken to determine whether it would be feasible to restore the seawall to a satisfactory state and to investigate options for replacement of the wall. All levels in this report have been reduced to a datum 100.00m below Australian Height Datum (AHD).
2. Investigations

2.1 Condition of Existing Seawall

The condition of the existing seawall was assessed following a site inspection carried out by Manly Hydraulics Laboratory (MHL) and from a site inspection and geotechnical investigations carried out by Coffey Partners International.

The existing seawall is constructed from dry stacked blocks of Hawkesbury Sandstone with sizes ranging between 600 x 400 x 200 and 1300 x 500 x 400. The surface of the playing fields behind the wall is uneven but is at approximately RL 101.5m. The crest of the seawall appears to have originally been at about RL 101.5m but due to the deterioration of the wall is now generally lower. It appears that the original height of the wall varied between about 1.6m and 2.3m.

The overall length of the existing wall is approximately 580m. A plan view of the site showing changes along the seawall is shown as Figure 2.1. This figure provides a key to the changes referred to in the site inspection and to the location of photos. Changes along the bank have been measured from the return in the seawall at its eastern end. The 10m length of the return should be added to the change to get the overall wall length.

A total of about 440m of the wall has collapsed or partially collapsed with material eroded away from behind the wall for distances of up to 3m. The concave (in plan) section of the seawall from Change 0 to 140m appears to have sustained less damage than the convex section between Change 140m and 270m, probably due to the horizontal arching action of the wall in the concave section and the less severe wave climate in this area.

The collapsed sections of seawall could only be satisfactorily repaired by reconstruction because a wall cross-section with a wider base would be required to comply with current rock wall practice. It would also be necessary to backfill behind the wall with coarse graded material and to provide a geotextile filter layer between the retained fill and the backfill.

Notes made during the site inspection carried out by MHL staff are presented in Table 2.1.

<table>
<thead>
<tr>
<th>Change</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 19</td>
<td>Reasonable condition, needs attention to crest. Four courses of rock. Two bottom courses approx 1300x500x400 high, third course 600x500x200 high, top course 1000x500x300 high. Bottom course straight on gravel/mud.</td>
</tr>
<tr>
<td>19 - 25</td>
<td>Collapsed outwards.</td>
</tr>
<tr>
<td>25 - 34</td>
<td>Reasonable but bulging outwards.</td>
</tr>
<tr>
<td>34 - 42</td>
<td>Displaced out, erosion behind.</td>
</tr>
<tr>
<td>42 - 46</td>
<td>Collapsed outwards.</td>
</tr>
</tbody>
</table>

continued/
### Table 2.1 Seawall Inspection (continued)

<table>
<thead>
<tr>
<th>Chainage</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>46 - 66</td>
<td>Displaced outwards, erosion behind</td>
</tr>
<tr>
<td>66 - 84</td>
<td>Displaced outwards, top stones collapsed landward, level of crest low, extensive erosion behind wall</td>
</tr>
<tr>
<td>84 - 91</td>
<td>Bulging outwards in middle, top fallen landward</td>
</tr>
<tr>
<td>91 - 99</td>
<td>Bulging outwards, top rocks fallen landward, some rocks in water, extensive erosion behind wall, crest low</td>
</tr>
<tr>
<td>99 - 111</td>
<td>Full height, reasonable condition, top course needs some attention</td>
</tr>
<tr>
<td>111 - 120</td>
<td>Badly collapsed outwards, rocks in water, bad erosion behind wall</td>
</tr>
<tr>
<td>120 - 126</td>
<td>Reasonable, top needs rebuilding</td>
</tr>
<tr>
<td>126 - 131</td>
<td>Badly collapsed outwards, extensive erosion behind wall</td>
</tr>
<tr>
<td>131 - 137</td>
<td>Reasonable but standing vertical</td>
</tr>
<tr>
<td>137 - 143</td>
<td>Badly collapsed outwards, extensive erosion behind wall</td>
</tr>
<tr>
<td>143 - 145</td>
<td>Reasonable but standing vertical</td>
</tr>
<tr>
<td>145 - 147</td>
<td>Top half of wall collapsed outwards</td>
</tr>
<tr>
<td>147 - 150</td>
<td>Reasonable condition</td>
</tr>
<tr>
<td>150 - 152</td>
<td>Wall bulging, crest low</td>
</tr>
<tr>
<td>152 - 154</td>
<td>Badly collapsed outwards, erosion behind wall</td>
</tr>
<tr>
<td>154 - 161</td>
<td>Wall bulging outwards very badly, crest very low</td>
</tr>
<tr>
<td>161 - 164</td>
<td>Collapsed landward, extensive erosion behind wall</td>
</tr>
<tr>
<td>164 - 171</td>
<td>Wall bulging badly, crest falling landward</td>
</tr>
<tr>
<td>171 - 184</td>
<td>Wall bulging, extensive landward collapsing of crest, crest low, extensive erosion  Six courses Bottom and second courses 800x500x300 high, third course 600x500x300 high, fourth course 600x500x300 high, fifth and sixth courses 900x500x300 high Variable irregular shape No footing</td>
</tr>
<tr>
<td>184 - 196</td>
<td>Complete collapse, landward and outwards</td>
</tr>
<tr>
<td>196 - 232</td>
<td>Crest rocks fallen landward, gives bulging appearance, some crest rocks in mud 2m from wall</td>
</tr>
<tr>
<td>232 - 256</td>
<td>Wall reasonable, pronounced batter, crest a bit low, some crest rocks in mud</td>
</tr>
<tr>
<td>256 - 266</td>
<td>Extensively collapsed, extensive erosion, rocks in mud but some rocks must have been removed from site</td>
</tr>
<tr>
<td>266 - 272</td>
<td>Reasonable, big voids between rocks</td>
</tr>
<tr>
<td>272 - 278</td>
<td>Reasonable but 20% of crest rocks removed and in mud, top low Seven courses, 600x400x200 All courses similar size Similar condition to Ch 353m</td>
</tr>
<tr>
<td>278 - 335</td>
<td>Top half of wall removed and in mud, extensive erosion Appears to be flood scour, wall must have been too low in this area Bridge at 303m</td>
</tr>
<tr>
<td>335 - 353</td>
<td>Wall reasonable, large batter, top fallen landward, erosion/scour above wall</td>
</tr>
<tr>
<td>353 - 358</td>
<td>Wall completely collapsed, bank scoured 3m back Smaller rocks, 600x400x200 high</td>
</tr>
<tr>
<td>358 - 364</td>
<td>Wall half collapsed</td>
</tr>
<tr>
<td>364 - 386</td>
<td>Wall in reasonable condition, crest low, scour above wall</td>
</tr>
<tr>
<td>386 - 417</td>
<td>Wall collapsed, rocks in creek, bank scoured above remains of wall</td>
</tr>
<tr>
<td>417 - 443</td>
<td>Bottom part of wall reasonable, some rocks removed</td>
</tr>
<tr>
<td>443 - 445</td>
<td>Culvert at Ch 4.4 Wall collapsed around culvert</td>
</tr>
<tr>
<td>445 - 463</td>
<td>Wall reasonable, overgrown with vegetation</td>
</tr>
<tr>
<td>468 - 472</td>
<td>Wall collapsed</td>
</tr>
<tr>
<td>472 - 490</td>
<td>Wall intact but has fallen landward, scour above wall</td>
</tr>
<tr>
<td>490 - 508</td>
<td>Wall collapse, mangroves</td>
</tr>
<tr>
<td>508 - 570</td>
<td>Orange painted nail in post at Ch 508 Wall low but reasonable condition Overgrown Approximate end of wall at Ch 570</td>
</tr>
</tbody>
</table>

The failure mechanism of the seawall appears to be that collapse has been brought about by the removal by erosion of fill material from directly behind the wall. Geotextile fabric was not used behind the wall when it was originally constructed and it is possible for fill material to escape through the gaps between the rocks of the seawall. Visible remnants of geofabric in
some sections of the seawall where repair work has been carried out indicate that geofabric has been used in repairs. The repairs have however been unsuccessful. Erosion is likely to have been caused by rainfall runoff, wave attack and seepage flow through the retained fill. Because of its sandy nature, the fill will have a high permeability and seepage out of the fill will occur during and immediately after rainfall events as well as during low tide conditions. During periods of high wave activity, waves will agitate the sandy material behind the seawall increasing the rate of erosion. At elevated harbour water levels, waves will be able to overtop the seawall causing direct erosion of the fill as well as increasing seepage flow back through the structure.

As fill is removed, the upper sandstone blocks progressively tilt backwards applying rotational force or 'outward thrust' on the lower blocks. This mechanism is shown in Figure 2.2 and can be seen in Plate 1. The presence of rocks in the water in front of the wall as far away as 3m may be due to wave action or to vandalism with rocks having been removed from the wall and thrown into the water.

Following identification of the failure mechanism, more critical inspection showed that in most cases even those sections of the wall that appear to be in reasonable condition are showing early symptoms of failure with slight landward displacement of top rocks and a bulging appearance of the wall.

While the existing seawall appears largely to have performed adequately from a structural viewpoint, the present wall proportions are not adequate for current rock wall practice. It is therefore not practical to repair the present wall other than possibly by dumping rock on the outside of the seawall. Repairing the wall by this method would require the placement of a geotextile layer to prevent further removal of fill from behind the existing wall. Such a repair would also probably encroach on the site boundary.

2.2 Site Survey
A strip survey along the seawall to Chamage 500m (Figure 2.1) was carried out by Public Works Survey and Land Information Services (SLIS). The wall actually extends to approximately Ch 570m; the last 70m being badly overgrown. A plan of the seawall showing the site boundary and the alignment of the crest and toe of the existing seawall along with a number of cross-sections of the wall are shown in Appendix A of this report. SLIS holds large scale drawings of the survey which also show the location of survey marks that would allow the set out of new construction. The survey has been catalogued by SLIS as Plan Cat No 9711.

Levels in the SLIS survey and all levels in this report have been reduced to a datum 100.00m below Australian Height Datum (AHD). The survey shows that the level of the playing fields behind the seawall is approximately RL 101.5m, the level of the crest of the relatively undamaged sections of the wall varies between RL 101.0m and RL 101.55m and the level of the toe of the seawall varies between RL 99.2m and RL 99.9m. The seawall lies within the site boundary with a minimum clearance between the seawall and the boundary of 0.4m at Ch 25 and a maximum clearance of about 3m at Ch 165m.
2.3 Geotechnical Investigations

Geotechnical investigations were carried out by Coffey Partners International Pty Ltd to determine the condition of the existing wall and footings, to determine the soil profile behind the wall, to comment on the stability of the wall and to provide foundation design parameters for wall reconstruction. The full report describing the investigations carried out is presented as Appendix B.

Geotechnical investigations showed that the fill material behind the wall is fine sand which has been placed directly over recent alluvial sediments overlying Hawkesbury Sandstone to form the reclaimed area. The alluvium is at least 2.2 m thick. Dynamic penetrometer soundings were advanced up to 2.5 m below the seawall footings without encountering bedrock. The seawall appears to be founded on a variety of materials including alluvial sediments, fine grained fill and coarse blocky fill. Existing bearing pressures are probably less than 50 kPa. Field observations have not highlighted any obvious problems with the bearing capacity of the foundations, so wall construction along similar lines to the existing wall should not require any special foundation preparation.

2.4 Water Levels

Design still water levels for Sydney Harbour have been determined for other projects by MHL based on Fort Denison tide records for the period 1914 to 1990 (Australian Water and Coastal Studies, 1991). The design still water levels are based on the expected peak levels that are likely to occur over a designated return period under the combined influence of an astronomical tide and storm surge. This level does not include wave set-up and wave run up.

No allowance has been made for Greenhouse Effect. The still water levels are shown in Table 2.2.

<table>
<thead>
<tr>
<th>Return Period (Yrs)</th>
<th>Still Water Level Rel. ISLW (m)</th>
<th>Still Water Level Rel. Survey Datum* (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>2.35</td>
<td>101.43</td>
</tr>
<tr>
<td>50</td>
<td>2.39</td>
<td>101.47</td>
</tr>
<tr>
<td>100</td>
<td>2.42</td>
<td>101.50</td>
</tr>
</tbody>
</table>

* Survey Datum 100.00 m below Australian Height Datum

The maximum normal tide level for Sydney Harbour (High High Water Solstice Springs) is approximately 1.89 m (ISLW) or 100.97 m (Survey Datum). As noted earlier, the general level of the playing fields is at RL 0.51 m which corresponds to the 100 year Return Period still water level. There appears to be little point in raising the crest level of the wall above this level as it will affect surface runoff from the playing fields. However, the wall would be overtopped by wave action during extreme conditions and measures need to be put in place to avoid damage to the bank protection as well as to the fill material.

Floods will have no measurable effect on the water levels of the estuarine water body of Tarban Creek beyond the entrance to the actual creek due to the width of the water body. During site inspections, no indication of past flood flow over the playing fields was observed.
It was therefore concluded that bank protection to a level of RL 1015 m would provide adequate protection from flood scour.

2.5 Wave Climate

The seawall is in a sheltered location but could be subjected to both locally generated wind waves and boat generated waves over some of its length. Wave fetches are short apart from a narrow fetch of 1200 m to the east. Waves from this direction can only approach the seawall at an oblique angle with the section of wall from channel 80 m to channel 220 m (Figure 21) the most exposed. The alignment of the seawall at these channels is approximately parallel to the fetch direction. The greatest angle of attack, at about channel 130 m, is approximately 40°. The bed level at the toe of the seawall in this section is at approximately mean sea level so the wall can only be subjected to waves at water levels above mid tide. The toe of the wall could however be subject to erosion at low water levels.

Design conditions for locally generated wind waves were determined using a computer model employing the methods presented in CERC 1984. The method predicts significant wave height (Hsig) and peak spectral wave period (Tp) given the design wind velocity and fetch conditions for the site. Wave conditions were determined for 20, 50 and 100 year average return interval winds based on Melbourne, W H. It was decided to use wind velocities for SE winds, which are approximately 25% greater than E winds, because local topography will tend to funnel SE winds along the Tarban Creek fetch. Design wave conditions are summarised in Table 2.3.

Table 2.3 Design Wave Conditions

<table>
<thead>
<tr>
<th>RI (yr)</th>
<th>V (m/s)</th>
<th>Hsig (m)</th>
<th>Tp (s)</th>
<th>Dur. (min)</th>
<th>H1% (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>20.0</td>
<td>0.49</td>
<td>2.0</td>
<td>10.5</td>
<td>0.74</td>
</tr>
<tr>
<td>50</td>
<td>21.5</td>
<td>0.53</td>
<td>2.0</td>
<td>10.0</td>
<td>0.80</td>
</tr>
<tr>
<td>100</td>
<td>23.0</td>
<td>0.57</td>
<td>2.1</td>
<td>9.5</td>
<td>0.87</td>
</tr>
</tbody>
</table>

The 50 year average recurrence interval significant wave height of 0.53 m compares well with a wave height of 0.54 m determined for Tarban Creek in a study carried out for the Maritime Services Board to identify areas suitable for fore-and-aft moorings (Maritime Services Board of New South Wales, 1987).

Published field measurements of boat generated waves are shown in Table 2.4.

Table 2.4 Boat Wave Measurements

<table>
<thead>
<tr>
<th>Boat Type</th>
<th>Maximum Wave Height (m)</th>
<th>Wave Period (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15 m Motor Cruiser</td>
<td>0.80</td>
<td>3.0</td>
</tr>
<tr>
<td>(Thackray, 1984)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13 m Tug</td>
<td>0.76</td>
<td>1.4</td>
</tr>
<tr>
<td>(Sorenson, 1967)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Speedboat</td>
<td>0.40</td>
<td>2.0</td>
</tr>
<tr>
<td>(Lesleigher, 1964)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

MHL670 - 11
In addition, MHL staff have made a large number of observations of boat generated waves on the Parramatta River. Large vessels were observed to generate waves with periods between 2.0 and 4.0 seconds with a largest measured height of 0.5m and speedboats were observed to generate waves with periods of less than 2.0 seconds with a largest measured height of 0.3m. The number of waves in a wave train was observed to generally number less than 10.

Boat speeds within Tarban Creek will be limited by the 4 knot speed limit within the moorings along both sides of the waterway, by the narrow congested nature of the waterway and by the Creek being a dead end for navigation.

From the above information it is estimated that boat generated waves at this site will be smaller than the largest wind waves and will have similar wave periods to large wind waves. Boat waves will, however, be able to approach the shore from a range of directions, including normal to the shoreline.

It is proposed that overtopping of the seawall by waves be allowed for and that a concrete path 1500mm wide be constructed along the top of the seawall to prevent scour immediately behind the crest of the wall.

2.6 Currents
Possible mechanisms for generating currents within Tarban Creek are tidal circulation, wind and flood flow from the creek. Of these the only mechanism able to generate significant currents would be flood flow and then only in the area near the mouth of the creek. Inspection of the bed near the creek entrance did not show any significant scour of the bed which was of similar composition to the bed near the toe of the seawall. It was concluded that scour would not pose a problem.

2.7 Construction Constraints
Access to the site of the seawall is through the grounds of the northern campus of Gladesville Hospital. Some minor widening of the access track from the upper level of the site to the playing fields will be required to allow large trucks to reach the site. The area adjacent to the seawall provides a large level site well suited to construction of a site compound and the stockpiling of materials during construction without major encroachment on the playing fields.

Enquiries were made with the authorities listed below regarding the presence of services in proximity to the seawall:

- Water Board
- MSB Waterways
- Hunters Hill Council
- Telecom
- NSW Health Department
- Sydney Electricity
- Natural Gas Company

Replies were received from all authorities. No services were known to exist in the immediate area of the seawall. The closest service is the Water Board sewage pumping station on the
northern bank of the creek near the footbridge. The location of the pumping station is shown in Figure 21.

Because the seawall lies within the intertidal zone some components of construction will be affected by the tide. Construction using land based plant will be possible when the water level is above the toe but preparation of the base and operations using hand labour will not be able to be carried out at high water levels.

The seawall is located in open space on the southern side of the creek but is between 30m and 170m from houses on the northern bank of the creek. Removal of the existing seawall and construction of a new seawall should not generate excessive noise levels. No particular advantage is seen in any of the possible types of seawall in terms of construction noise.
3. Options for Restoration or Replacement of Seawall

3.1 Introduction

The primary purpose of the seawall is to retain the fill underlying the playing fields. It also protects the bank from damage by wave action in the open area of Tarban Creek and from flood scour in the creek section.

The design of the seawall will be affected by the relatively poor foundation conditions and the requirement to allow drainage of groundwater without the removal of fill material. It will also be necessary to protect the footings of the footbridge from scour by ensuring that the bank is protected from scour in this area. The footings of the footbridge are approximately 4m from the toe of the bank.

There are a number of seawall construction options. These may be broken down into the basic classifications of sloping and vertical (including steeply battered) structures and porous or non-porous structures. The site is suited to a porous structure because such structures generally offer good drainage characteristics and their inherent flexibility is suited to sites with relatively poor bearing capacity. A suitable structure could be sloping or vertical (battered) although a sloping structure will encroach on the level area behind the existing seawall if its toe follows the alignment of the present seawall.

Because of the variation in wave climate, with the bank only subject to substantial wave attack between Ch 80m and 230m (Figure 2.1), it is obvious that the best solution may be a composite scheme where the bank protection is varied along its length.

Possible vertical structures include:

(i) gravity rock seawall,
(ii) gabion wall,
(iii) crib wall

Possible sloping structures would include:

(i) stable beach,
(ii) rip rap armoured slope,
(iii) gabion mattress armoured slope

Details of these possible options are listed below along with their advantages and disadvantages along with indicative costing to allow a comparison between the various options.

The indicative costs were developed assuming bank protection 2m high and 580m long overall. It was estimated from the surveyed seawall cross-sections and field measurements of
the seawall thickness that 380 m$^3$ of sandstone rock could be recovered from the existing seawall, for re-use in some options and for disposal in others.

Unit costs used in developing indicative costing are shown below. Material costs shown include cartage to site.

### Table 3.1 Unit Costs

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Unit Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill, granular</td>
<td>$28/tonne, $50/m$^3$</td>
</tr>
<tr>
<td>Backfill, plant and labour costs</td>
<td>$3/m$^3</td>
</tr>
<tr>
<td>Concrete</td>
<td>$120/m$^3</td>
</tr>
<tr>
<td>Concrete path</td>
<td>$50/linear m</td>
</tr>
<tr>
<td>Disposal of site material</td>
<td>$14/m$^3 (including excavation)</td>
</tr>
<tr>
<td>Excavation</td>
<td>$2/m$^3</td>
</tr>
<tr>
<td>Fill, for gabions and reno mattresses</td>
<td>$33/tonne, $60/m$^3</td>
</tr>
<tr>
<td>Gabions</td>
<td>$47/m$^3</td>
</tr>
<tr>
<td>Gabion and reno mattresses, plant and labour costs</td>
<td>$60/m$^3</td>
</tr>
<tr>
<td>Geotextile fabric for gabions and reno mattresses, supplied and placed</td>
<td>$5/m^2$</td>
</tr>
<tr>
<td>Geotextile fabric for riprap, supplied and placed</td>
<td>$7/m^2$</td>
</tr>
<tr>
<td>Reno mattresses</td>
<td>$52/m^3$</td>
</tr>
<tr>
<td>Sand, $19/tonne</td>
<td>$30/m^3$</td>
</tr>
<tr>
<td>Sand, plant and labour costs</td>
<td>$3/m^3</td>
</tr>
<tr>
<td>Sandstone blocks</td>
<td>$1150/m^3$</td>
</tr>
<tr>
<td>Riprap, $33/tonne</td>
<td>$60/m^3$</td>
</tr>
<tr>
<td>Riprap, plant and labour costs</td>
<td>$5/m^3$</td>
</tr>
</tbody>
</table>

It can be seen from the unit rates that two key items that will have a large bearing on the costs of particular options are the cost of sandstone blocks at $1150/m^3$ and the cost of disposal of site materials at $14/m^3$.

In addition to the cost of the bank protection a sum of $10,000 has been allowed for site establishment and later clean up, $30,000 has been allowed for construction of a concrete path and $10,000 has been allowed for supervision, survey, tender documentation and contingencies, a total of $80,000 for the total length of protection. This amount has been converted to a per metre price and included in all costing.

### 3.2 Option 1 - Rock Seawall

A seawall of similar construction to the existing seawall could be constructed. However, the proportions of the present seawall are not adequate for current rock wall practice, and a replacement wall would have to be wider at the base. Two alternative cross-sections for a dry rock seawall are shown in Figure 3.1, one having a vertical front face and the other a face battered at a slope of 1 (horizontal) to 2 (vertical). Both walls have the same cross-sectional area of 2m$^2$ and hence require the same quantity of rock. It is estimated that the present wall contains about 380m$^3$ of rock. The total quantity required to reconstruct 580m of wall is estimated at 1160m$^3$.

**Advantages**

- Good drainage
- Inherent flexibility
Similar appearance to present seawall and other seawalls in area
Makes use of rock from existing wall
Does not take up extra space
Minimum excavation
Minimum disposal of site materials

Disadvantages
Will require additional (expensive) rock for adequate wall section
Requires skilled labour

The unit cost of a sandstone dry rock seawall will vary with the length of wall to be constructed. For a length up to 190m it should be possible to construct the wall from rock recovered from the existing wall. Beyond 190m it will be necessary to either buy in sandstone blocks or to use concrete blocks behind the face course of sandstone blocks. Beyond about 250m it would be necessary to buy sandstone blocks to provide enough sandstone face blocks. The cost of any particular length of sandstone seawall will therefore have to be calculated for that particular case.

Indicative costs for a full sandstone rock seawall are $495/m for a 190m wall and $2010/m for a 580m wall. Costs for a sandstone rock faced seawall with supplementary concrete blocks are $585/m for a 250m wall and $1380/m for a 580m wall.

3.3 Option 2 - Gabion Seawall
A typical gabion seawall cross-section is shown in Figure 3.2

Advantages
Good drainage
Inherent flexibility
Could make use of rock from existing wall broken down on site
Maintains existing level area

Disadvantages
Different appearance to other walls in area
Larger excavation than rock seawall
Prone to vandalism (e.g., cutting of wires)
Possible lower life expectancy due to corrosion

The indicative cost of a gabion seawall is $840/m.

3.4 Option 3 - Crib Wall
A typical crib wall cross-section is shown in Figure 3.3. A special crib wall system for marine use is available at extra cost. This system has a closed face to prevent wash out of fill and corrosion protection for reinforcement. A crib wall could be faced with sandstone blocks at large extra cost.

Advantages
Good drainage
Inherent flexibility
Could backfill with rock from existing wall broken down on site
Disadvantages
Requires footing
Different appearance to other seawalls near site
Will require closed face or large rock fill

Indicative costs of a crib wall are $629/m

3.5 Option 4 - Stable Beach
Two possible cross-sections for a stable beach are shown in Figure 3.4

Advantages
Good drainage
Environmentally attractive

Disadvantages
Beach subject to littoral movement due to wave attack at angle to beach - convex plan shape susceptible to wave attack
Requires large area, would encroach on playing fields
Large amount of excavation - may encounter rocks in fill
May be colonised by mangroves
May be covered in silt from creek flow
Will require maintenance
Not suitable for creek section
Requires removal of existing wall
Requires disposal of material

Two options are available - the first a wide beach with no back wall and the second a narrower beach with a low back wall constructed from stone from the existing seawall. The indicative costs are $935/m for a wide beach and $675/m for a narrow beach

3.6 Option 5 - Riprap Armoured Slope
A typical cross-section for a riprap slope is shown in Figure 3.5

Advantages
Good drainage
Inherent flexibility
Could use rock from existing wall broken down on site

Disadvantages
Large rocks required in area subject to waves
Requires more space than vertical wall
Requires large excavation
Appearance dissimilar to seawalls in area
Will collect debris
Prone to vandalism

Two sizes of riprap would be required, a smaller size with a D50 of 200mm in the creek area and a larger size with a D50 of 400mm in the area exposed to waves requiring a greater
thickness of protection in the wave area. Indicative cost for the creek section is $450/m ($167,000 for 370m) and for the wave area $770/m ($185,000 for 240m)

3.7 Option 6 - Reno Mattress Armoured Slope
A typical cross-section for a reno mattress slope is shown in Figure 3 6

Advantages
Good drainage
Inherent flexibility
Can use rock from existing wall broken down on site

Disadvantages
Requires more space than vertical wall
Requires large volume of excavation
Disimilar appearance to other seawalls in area
Prone to vandalism
Possible lower life expectancy due to corrosion
Requires disposal of materials

The indicative cost of a reno mattress slope is $442/m
4. Preferred Option

The indicative costs of the options considered in the last section are summarised in Table 4.1

<table>
<thead>
<tr>
<th>Option</th>
<th>Length (m)</th>
<th>Cost/m ($)</th>
<th>Indicative Cost ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full Rock Seawall</td>
<td>190</td>
<td>495</td>
<td>94,000</td>
</tr>
<tr>
<td></td>
<td>580</td>
<td>2010</td>
<td>1,166,000</td>
</tr>
<tr>
<td>Rock Face Seawall</td>
<td>250</td>
<td>585</td>
<td>146,000</td>
</tr>
<tr>
<td></td>
<td>580</td>
<td>1,380</td>
<td>800,000</td>
</tr>
<tr>
<td>Gabion Seawall</td>
<td>580</td>
<td>840</td>
<td>487,000</td>
</tr>
<tr>
<td>Crib Wall</td>
<td>580</td>
<td>629</td>
<td>365,000</td>
</tr>
<tr>
<td>Wide Beach</td>
<td>300</td>
<td>935</td>
<td>281,000</td>
</tr>
<tr>
<td>Narrow Beach</td>
<td>300</td>
<td>675</td>
<td>203,000</td>
</tr>
<tr>
<td>Riprap (creek) (wave area)</td>
<td>330</td>
<td>450</td>
<td>149,000</td>
</tr>
<tr>
<td></td>
<td>250</td>
<td>770</td>
<td>193,000</td>
</tr>
<tr>
<td></td>
<td>580</td>
<td>-</td>
<td>342,000</td>
</tr>
<tr>
<td>Reno Mattress</td>
<td>580</td>
<td>442</td>
<td>256,000</td>
</tr>
</tbody>
</table>

The cheapest forms of bank protection are reno mattresses or riprap in the creek section at basically the same cost of $450/m.

The next cheapest option per metre, for a length of wall of 190m, is the full rock seawall constructed from rock recovered from the existing seawall, at a price of $495/m. Above this length the price per metre increases as it becomes necessary to either buy rock for an all rock wall or to use concrete blocks to supplement a face row of rocks.

The preferred option is a composite scheme comprising three types of bank protection - natural beach, rock seawall and riprap. A plan view of the proposal is shown in Figure 4.1. The toe of the protection would follow the alignment of the toe of the existing seawall. The estimated total cost of protection is $309,000.

The details of the proposed protection are outlined below.

Ch 0 - 50m

A natural beach would replace the eastern end of the existing seawall from Ch 0 to Ch 50m. In this area the level area behind the wall is relatively narrow (approximately 10 - 20m) and is not used as part of the playing fields. This part of the foreshore is also relatively sheltered from wind waves and adjoins a section of shoreline to the east containing pockets of sandy beach among areas of rocks. It is envisaged that the beach would form a continuation in appearance of this shoreline. The beach to the east backs onto large natural rocks rising steeply from the sloping beach. The continuation of the line of rocks can be seen at the back.
of the level area and it should be possible to construct the beach without a wall or pathway at the rear. It will however be necessary to construct a drain along the back of the beach to replace an existing open drain. The estimated total cost of the beach area is $41,000 including the drain at the back of the beach.

A typical cross-section of the beach is shown in Figure 4.2. The beach should be constructed with a maximum slope of approximately 1:10, similar to areas of beach to the east. The drain at the rear of the beach could be constructed from reno mattresses as shown in Figure 4.2 with rubble filling in the space between the mattresses and the natural bank. The grading of sand used in the beach should be chosen so that it is not finer than sand in the areas of beach to the east.

**Ch 50m - 300m**

A sandstone seawall similar in appearance to the existing seawall is proposed between Ch 50 and the footbridge at Ch 300. By limiting the length of wall to less than 250m it should be possible to construct it from sandstone blocks recovered from the existing seawall with concrete blocks placed in the rear course of the wall to make up sufficient blocks. This section of seawall is in the area subjected to the most adverse wave climate and is the most visible section of the seawall. It is also the area closest to the marked soccer fields and a vertical seawall will maintain the present distance from the soccer field to the water’s edge. The footbridge will provide a point of visual demarcation between the creek and estuarine sections of the bank protection. The estimated cost of the seawall section is $146,000.

The rock wall should be constructed in the manner shown in Figure 4.3. Rocks should be placed in such a manner that they are stable and interlocking and laid roughly coursed and bedded on the broadest base. Where rocks cannot be bedded in a stable manner because of their shape they may be laid in a bed of mortar. However, the use of mortar should be limited in order to maintain the free draining properties and flexibility of the structure. Rock is to be of sound durable sandstone or other approved material with a minimum dimension of 0.5m. Backfill is to be granular, free draining and compacted. Weepholes are to be provided by spacing blocks with 20-50mm gaps. A geofabric combining high tensile strength and stiffness with good filtration capacity, such as Propex 3220 (Amoco Chemicals) should be used under and behind the wall and between the coarse free draining backfill and the sandy fill of the reclaimed area.

**Ch 300m - Ch 570m**

A riprap slope is proposed for the creek section from Ch 300 to Ch 570. This section is not exposed to substantial wave attack and its main purpose is to protect the bank from flood scour. The estimated cost of the riprap protection is $122,000.

Based on the recommendations of Rural Water Commission of Victoria, 1991, the D50 size of riprap is 200 mm. The recommended grading of rock for the riprap protection is shown in Table 4.2.
Table 4.2 Riprap Rock Grading

<table>
<thead>
<tr>
<th>Equivalent Sphere Diameter (mm)</th>
<th>Rock Mass (Kg)*</th>
<th>% Smaller by Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>77</td>
<td>100</td>
</tr>
<tr>
<td>200</td>
<td>10</td>
<td>50</td>
</tr>
<tr>
<td>60</td>
<td>0.8</td>
<td>10</td>
</tr>
</tbody>
</table>

* Based on specific gravity of 2.3

There should be no significant voids in the rock riprap through which underlying material could be washed out and no individual rock should be free to move by itself.

Rock used in the riprap shall meet the following requirements:

1. All rock shall be hard, durable and clean rock from a quarry (or quarries) approved in writing as a supply source by the works superintendent and be strictly in accordance with any conditions of that approval. The rock shall be resistant to abrasion and free from cracks, cleavage planes, seams and other defects including unstable minerals which could result in its breakdown under a marine environment.
2. All rock shall have a Los Angeles Value of not more than 25%.
3. All rock shall have a sodium sulphate test result not exceeding 12% loss.
4. All rock shall have a minimum specific gravity of 2.3.
5. All rock shall have a saturated unconfined compressive strength greater than 50MPa.
6. All rock shall have a minimum wet/dry strength ratio greater than 50%.
7. All rock shall be of such a shape that the maximum dimension does not exceed 2.5 times the minimum dimension.

A geotextile underlayer shall be provided between the embankment and the riprap. This geotextile is to have a unit mass greater than 350g/m² (e.g. Bidim A64 or Terrafix 369R or similar) and should be moved, joined and placed in accordance with the manufacturers' specifications. Care must be taken to ensure maximum resistance between the riprap and the geotextile. To ensure this it is recommended that the bank not be prepared to a smooth, even batter before placing the geotextile and that the geotextile is not stretched tightly over the bank. Scour protection should be provided at the toe by placing extra rock as a 'self launching toe' should the wall be undercut. The geotextile fabric should extend underneath all rock placed at the toe such that the as the rock is 'launched' into a new position it will continue to be underlain by the geotextile.

The alignment of the toe of the riprap is to remain within the alignment of the toe of the existing seawall. The crest alignment should be a smooth curve.
5. Conclusions and Recommendations

The existing seawall has reached the end of its useful life and is beyond repair. To rebuild the wall to modern design standards over its present length would require the purchase of additional rock to raise the crest level and to thicken the base of the wall. Because of the high cost of sandstone blocks alternative structures were investigated.

Following assessment of alternative structures a composite design comprising a beach, a sandstone seawall similar in appearance to the existing seawall and riprap protection is proposed. This scheme will minimize the cost of protection while maintaining a similar appearance in the area most exposed to public view.

The crest of the protection has been set at RL 101.5, the level of the playing fields and the 100 year average recurrence interval still water level for Sydney Harbour. No provision has been made for sea level rises associated with the Greenhouse Effect. A concrete path, 1500mm wide should be constructed along the crest to provide protection from overtopping that could occur due to wave action combined with high water levels in the area of the seawall and from the possibility of flood scour above the riprap protection in the creek area.

As the cost of the sandstone rock seawall section of the proposed bank protection is highly dependent upon recovering sufficient suitable sandstone blocks from the existing seawall to construct the visible front face of the seawall it is recommended that confirmation of the quantity of blocks be made a condition of tendering.

It is likely that the design of bank protection along Tarban Creek will be affected by the overall drainage design for the redevelopment area. Drainage outlets will have to be accommodated within the bank protection and protection from scour provided at the toe of the structure. From the point of view of the bank protection the best locations for drainage outlets are in the creek upstream of the riprap protection and on the rocky beach area to the east of the proposed beach. It is recommended that the overall site drainage be designed so that discharges are diverted away from the bank protection.
6. References


Coastal Engineering Research Centre (CERC), 1984, Shore Protection Manual, Fourth Edition, Department of the Army, Waterways Experiment Station, Corps of Engineers

Lesleighter E J, 1964, Hawkesbury River - The Effect of Speedboat Activities on Bank Erosion, Manly Hydraulics Laboratory, Report No 276

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Melbourne W H, Extreme Wind Gusts for Sydney, Report to Ove Atup and Partners

Rural Water Commission of Victoria, 1991, Guidelines For Stabilising Waterways, Standing Committee on Rivers and Catchments, Victoria

Sorenson R M, 1967, Investigation of Ship Generated Waves

Looking west. Trundle Wheel at Chainage 81m

Looking east. Trundle Wheel at Chainage 190m
Looking west. Trundle Wheel at Chainage 286m
SITE LOCALITY PLAN

STATE PROJECTS
Manly Hydraulics Laboratory
Report No. 670

Figure 1.1
1. Minor erosion of fill
   Backtilting block
   Fill carried away by groundwater flow
   FILL
   SEDIMENT

2. Progressively back tilting blocks
   Foot block thrust out
   FILL
   SEDIMENT

3. Eroded fill
   Blocks
   FILL
   SEDIMENT

CONCEPTUAL FAILURE MECHANISM
OPTION 1 - ALTERNATIVE TYPICAL ROCK SEAWALL
CROSS-SECTION VERTICAL FRONT FACE
AND BATTERED FRONT FACE
OPTION 2 - TYPICAL GABION SEAWALL CROSS-SECTION
TYPICAL CRIB WALL CROSS-SECTION

Concrete path
Filter cloth
Granular backfill
Concrete footing

Figure 33
WIDE BEACH

NARROW BEACH WITH WALL
Figure 3.6

OPTION 6 - RENO MATTRESS ARMOURSED SLOPE
Maximum slope

Natural bed

Imported sand

Min 500

Reno mattress drain

Natural bank

1

10

2000

State Projects
Manly Hydraulics Laboratory
Report No 670

PREFERRED OPTION - TYPICAL BEACH CROSS-SECTION

Figure 42
Geotextile underlayer

Concrete path

Riprap minimum thickness 400m
Appendix A

Site Survey
Figure A2

CROSS SECTION - CHAINAGE 00
Figure A4

CROSS SECTION - CHAINAGE 100
CROSS SECTION - CHAINAGE 120

State Projects
Manly Hydraulics Laboratory
Report No 670
Figure A6

State Projects
Manly Hydraulics Laboratory
Report No 670

CROSS SECTION - CHAINAGE 150
CROSS SECTION - CHAINAGE 200
Figure A8

CROSS SECTION - CHAINAGE 250
Figure A12

CROSS SECTION - CHAINAGE 400
FELAVATION

CHAINAGE 500

Figure CROSSECTION - CHAINAGE 500
Appendix B

Geotechnical Investigation
Public Works - State Projects

MANLY HYDRAULICS LABORATORY
GLADESVILLE HOSPITAL SEA WALL

GEOTEchnICAL INVESTIGATION

S9986/1-AB  September, 1993
The Manager
Public Works - State Projects
Manly Hydraulics Laboratory
110B King Street
MANLY VALE, NSW 2093

ATTENTION: MR BOB COOK

Dear Sir,

RE: GLADESVILLE HOSPITAL SEA WALL

Attached is our report on geotechnical investigations for the Gladesville Hospital sea wall on the southern bank of Tarban Creek.

Please don't hesitate to contact me if you need to discuss any aspects of this work.

For and on behalf of
COFFEY PARTNERS INTERNATIONAL PTY LTD

[Signature]

N S MATTES
CONTENTS

1.0 INTRODUCTION 3

2.0 FIELD WORK 3

3.0 SITE CONDITIONS 4
   3.1 Surface Conditions 4
   3.2 Subsurface Conditions 4

4.0 DISCUSSION AND RECOMMENDATIONS 5
   4.1 Site Model 5
   4.2 Failure Mechanism 6
   4.3 Footing Requirements 6
   4.4 Sea Wall Reconstruction 6

Important Information About Your Geotechnical Engineering Report

Drawing S9986/1-1 Details of Investigation

Photographs 1 and 2

Appendix A - Geological Logs of Drill Holes
1.0 INTRODUCTION

This report describes geotechnical studies carried out for the NSW Department of Health on the proposed reconstruction of a sea wall along Tarban Creek at Gladesville Hospital, Sydney.

This work was commissioned by Mr Bob Cook of Manly Hydraulics Laboratory, Public Works Department - State Projects on 31 August, 1993.

The purpose of the studies was to investigate the condition of the existing wall and footings, determine the soil profile behind the wall, comment on stability of the wall, and provide foundation design parameters for wall reconstruction.

Preliminary information consisted of a scope of work statement and survey plan of the sea wall provided by Mr John Murtagh of Public Works.

2.0 FIELD WORK

Field work consisted of:

- logging of 4 power auger boreholes together with dynamic penetrometer testing, to maximum depths of up to 2.7m for augered boreholes and 3.9m for penetrometer tests
- a detailed walkover inspection by one of our Principal Engineers

This work was carried out between the 2nd and 13th of September, 1993. The log are presented in Appendix A, together with explanation sheets defining the terms and symbols used in their preparation.

Location of boreholes are shown in Drawing No. S9986/1-1. They were obtained by paced measurements from existing surface features shown on the supplied plan. Ground surface levels were not measured.

Water level reading have been made in auger hole before backfilling, and levels are shown on the logs. It must be noted that fluctuations in groundwater level will occur due to variation in tides, rainfall, temperature and other factors.
3.0 SITE CONDITIONS

3.1 Surface Conditions

The sea wall is approximately 1 to 2m high, with face batter ranging from vertical to about 2V 1H, and runs for about 600m along the southern bank of Tarban Creek. An uneven, but approximately horizontal grassed playing field lies between the sea wall and the flanking slope leading up to Victoria Road.

The wall is constructed of dry stacked Hawkesbury Sandstone blocks, with dimensions of about 0.7m x 0.5m x 0.3m. Several parts of the wall have collapsed or partially collapsed, and material has been eroded away from behind the wall for distances of up to 3m.

3.2 Subsurface Conditions

The site is situated on recent alluvial sediments overlying Hawkesbury Sandstone and typically has a subsurface profile consisting of:

- **TOPSOIL**
  - 0.15 to 0.2m thick
    - above -

- **FILL**
  - 1.2m to 1.85m
    - above -

- **ALLUVIUM**
  - greater than 2.2m thick
    - **CLAYFY SAND** fine grained, dark grey, trace of shell and wood fragments.
    - Minor amounts of **SANDY CLAY** medium plasticity, dark grey, shell fragments and root fibres.
    - Generally very loose consistency, with a medium dense layer at the FILL/ALLUVIUM interface.
The profile is based on power auger holes drilled on the playing field approximately 4m behind the sea wall. Observation of eroded faces immediately behind the wall suggests that coarse fill, including sandstone cobbles, has been used immediately behind the wall but it is possible that this is material which has been placed to repair areas subject to erosion from surface run-off, and hence may not exist for the full length of the wall.

Groundwater was encountered at 1 to 1.2m below ground level, which is approximately 0.2 to 0.5m above the sea wall footings. The tide was rising during the period of field work, water levels in Tarban Creek were estimated to be between low and midwater. During a subsequent inspection at low tide after a rain period, significant seepage flows were observed to be coming from the toe of the wall at numerous locations. There is also a concentrated subsurface drainage outlet in an eroded area about 50m west of the footbridge, which was flowing freely after the rain period.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 Site Model

Fill material consisting mostly of fine sand has been placed directly over alluvial sediments to create a reclaimed area presently utilized as a playing field. The sea wall was built around the fill to protect it from erosion and probably to initially retain the fill material.

Alluvium is at least 2.2m thick. Dynamic penetrometer soundings were advanced up to 2.5m below the sea wall footings without encountering bedrock.

The sea wall appears to be founded on a variety of materials, including:

- alluvial sediments (e.g., TC4)
- fine grained fill (e.g., TC1 & TC3)
- coarse blocky fill (e.g., TC2)

The sea wall is constructed of a single row of sandstone blocks, stacked 1m to 2m high. Existing bearing pressures are probably less than 50 kPa.
4.2 Failure Mechanism

The sea wall appears to be collapsing because of removal by erosion of fill material from directly behind the wall. As fill material is removed, the upper sandstone blocks progressively tilt backwards applying rotational force or 'outwards thrust' on the lower blocks. This mechanism is sketched on Drawing S9986/1-1. The erosion is probably caused by both wave effects from Tarban Creek, and seepage flows out of the retained sandy fill, the latter would occur particularly after extended periods of rainfall, when an elevated groundwater table can be expected to develop in the fill, which because of its sandy nature will have a relatively high infiltration capacity. Photographs 1 and 2 show typical examples of early and late stages of wall deterioration.

Failure may also occur because of undermining of the footing blocks by erosion of foundation materials, but this mechanism cannot be confirmed or discounted without inspection of footing blocks exposed at low tide. Nevertheless, there is no obvious evidence of any problems with the footings, either from erosion or bearing capacity viewpoints.

It would also appear that the wall has suffered significantly from vandalism over the years, with rocks being removed from the top of the wall and thrown into the creek, (to either splash or splat, depending on the state of the tide).

4.3 Footing Requirements

The existing wall bearing pressures are probably about 50 kPa. Field observations have not highlighted any obvious problems with bearing capacity of the foundations, so wall reconstruction along similar lines to the existing wall should not require any special foundation preparation.

4.4 Sea Wall Reconstruction

In reconstruction of the seawall, the following points should be noted:

- While the existing wall appears largely to have performed adequately from a structural viewpoint, the present wall proportions are not adequate for current rock wall practice. For a 2m high wall, a base width of at least 1m, and preferably 1.5m, should be adopted, while for a 1m high wall, a base width of at least 0.75m is required. A top width of 0.5m will be satisfactory. For walls of these
proportions the front face can be either vertical, or battered no flatter than 2V 1H

- geofabric should be used to prevent the erosion of material from the base of the wall, to assist in bridging over localised soft spots in the foundation, and to prevent the erosion of the retained sandy fill. A geofabric combining high tensile strength and stiffness with good filtration capacity, such as Propex 3220 (Amoco Chemicals) should be used at the base of the wall in particular. Any backfill between the wall and the geofabric-protected sandy fill should be coarse, free-draining granular material.

- it would be possible to use some other form of retaining wall to provide the physical constraint, with the sandstone blocks being used as a facade. A wall system with inherent flexibility, such as a crib wall, would be preferable, given the relatively poor foundation conditions for a reinforced concrete structure. Alternatively, the heights are such that a simple cantilevered sheet pile wall could be feasible, but in this case care would need to be taken to ensure that the retained fill could drain, to avoid building up an elevated water table.

For and on behalf of

COFFEY PARTNERS INTERNATIONAL PTY LTD

[Signature]

DR N S MATTS
As the client of a consulting geotechnical engineer, you should know that site subsurface conditions cause more construction problems than any other factor. ASFE/The Association of Engineering Firms Practicing in the Geosciences offers the following suggestions and observations to help you manage your risks.

**A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS**

Your geotechnical engineering report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. These factors typically include: the general nature of the structure involved; its size and configuration; the location of the structure on the site; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask your geotechnical engineer to evaluate how factors that change subsequent to the date of the report may affect the report's recommendations.

Unless your geotechnical engineer indicates otherwise, do not use your geotechnical engineering report:

- when the nature of the proposed structure is changed. For example, if an office building will be erected instead of a parking garage or a refrigerated warehouse will be built instead of an unrefrigerated one.
- when the size, elevation, or configuration of the proposed structure is altered.
- when the location or orientation of the proposed structure is modified.
- when there is a change of ownership or for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems that may occur if they are not consulted after factors considered in their report’s development have changed.

**SUBSURFACE CONDITIONS CAN CHANGE**

A geotechnical engineering report is based on conditions that existed at the time of subsurface exploration. Do not base construction decisions on a geotechnical engineering report whose adequacy may have been affected by time. Speak with your geotechnical consultant to learn if additional tests are advisable before construction starts. Note too that additional tests may be required when subsurface conditions are affected by construction operations at or adjacent to the site or by natural events such as floods, earthquakes, or ground water fluctuations. Keep your geotechnical consultant apprised of any such events.

**MOST GEOTECHNICAL FINDINGS ARE PROFESSIONAL JUDGMENTS**

Site exploration identifies actual subsurface conditions only at those points where samples are taken. The data were extrapolated by your geotechnical engineer who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your geotechnical engineer can work together to help minimize their impact. Retaining your geotechnical engineer to observe construction can be particularly beneficial in this respect.

**A REPORT’S RECOMMENDATIONS CAN ONLY BE PRELIMINARY**

The construction recommendations included in your geotechnical engineer’s report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Because actual subsurface conditions can be discerned only during earthwork, you should retain your geotechnical engineer to observe actual conditions and to finalize recommendations. Only the geotechnical engineer who prepared the report can be fully familiar with the background information needed to determine whether or not the report’s recommendations are valid and whether or not the contractor is abiding by applicable recommendations. The geotechnical engineer who developed your report cannot assume responsibility or liability for the adequacy of the report’s recommendations if another party is retained to observe construction.

**GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS**

Consulting geotechnical engineers prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your geotechnical engineer prepared your report expressly for you and expressly for purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the geotechnical engineer. No party should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.

**GEOENVIRONMENTAL CONCERNS ARE NOT AT ISSUE**

Your geotechnical engineering report is not likely to relate any findings, conclusions, or recommendations.
about the potential for hazardous materials existing at the site. The equipment techniques and personnel used to perform a geotechnical exploration differ substantially from those applied in geotechnical engineering. Contamination can create major risks. If you have no information about the potential for your site being contaminated, you are advised to speak with your geotechnical consultant for information relating to geoenvironmental issues.

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION
Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid misinterpretations, retain your geotechnical engineer to work with other project design professionals who are affected by the geotechnical report. Have your geotechnical engineer explain report implications to design professionals affected by them and then review those design professionals’ plans and specifications to see how they have incorporated geotechnical factors. Although certain other design professionals may be familiar with geotechnical concerns, none knows as much about them as a competent geotechnical engineer.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE REPORT *
Geotechnical engineers develop final boring logs based upon their interpretation of the field logs (assembled by site personnel) and laboratory evaluation of field samples. Geotechnical engineers customarily include only final boring logs in their reports. Final boring logs should not under any circumstances be redrafted for inclusion in architectural or other design drawings because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, disputes and unanticipated costs are the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, give contractors ready access to the complete, geotechnical engineering report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report’s limitations assuming that a contractor was not one of the specific persons for whom the report was prepared and that developing construction cost estimates was not one of the specific purposes for which it was prepared. In other words, while a contractor may gain important knowledge from a report prepared for another party, the contractor would be well-advised to discuss the report with your geotechnical engineer and to perform the additional or alternative work that the contractor believes may be needed to obtain the data specifically appropriate for construction cost estimating purposes. Some clients believe that it is unwise or unnecessary to give contractors access to their geotechnical engineering reports because they hold the mistaken impression that simple disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems. It also helps reduce the adversarial attitudes that can aggravate problems to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY
Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical engineers. To help prevent this problem, geotechnical engineers have developed a number of clauses for use in their contracts, reports, and other documents. Responsibility clauses are not exculpatory clauses designed to transfer geotechnical engineers’ liabilities to other parties. Instead, they are definitive clauses that identify where geotechnical engineers’ responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report. Read them closely. Your geotechnical engineer will be pleased to give full and frank answers to any questions.

RELY ON THE GEOTECHNICAL ENGINEER FOR ADDITIONAL ASSISTANCE
Most ASFE-member consulting geotechnical engineering firms are familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a construction project. Contact a member of ASFE for a complimentary directory of ASFE publications.

* For further information on this aspect reference should be made to “Guidelines for the Provision of Geotechnical Information in Construction Contracts” published by the Institution of Engineers Australia, National Headquarters, Canberra, 1987.

THE ASSOCIATION OF ENGINEERING FIRMS PRACTICING IN THE GEOSCIENCES
8811 COLESVILLE ROAD/SUITE G106/SILVER SPRING MD 20910
TELEPHONE 301/565-2733 FACSIMILE 301/589-2017

Reprinted with permission by Coffey Partners International Pty Ltd 1993.
View towards eastern end of wall, showing erosion of fill behind wall, back-toppling of blocks in foreground.

Advanced deterioration, with wall fully collapsed due to loss of support from retained fill.
FAILURE DETAILS

1. MINOR EROSION OF FILL
   BACKFILLED BLOCK
   PUSHED OUT CENTRAL BLOCK
   FOOT BLOCK IN PLACE
   FILL
   SEDIMENTS

2. PROGRESSIVELY BACK TILTING BLOCKS
   FOOT BLOCK THRUST OUT
   FILL
   SEDIMENTS

3. ERODED FILL
   BLOCKS
   APPROX. MID WATER
   FILL
   SEDIMENTS

PUBLIC WORKS DEPARTMENT
GLADESVILLE HOSPITAL; SEA WALL
DETAILS OF INVESTIGATION

Coffey Partners International Pty Ltd Consulting Engineers in the geotechnical sciences A.C.N. 003 692 019
APPENDIX A
results of field investigation
SOIL DESCRIPTIONS

Classification of Material based on Unified Classification System (refer SAA Site Investigation Code AS1726–1975 and No 1 Table D1)

Moisture Condition based on appearance of soil
- dry: Looks and feels dry; the soils usually hard, powdery or friable; granular soils run freely through hands
- moist: Soil feels cool; darkened in colour; cohesive soils usually weakened by moisture; granular soils tend to cohere but one gets no free water on hands on remoulding
- wet: Soil feels cool; darkened in colour; cohesive soils weakened; granular soils tend to cohere; free water collects on hands when remoulding

Consistency based on unconfined compressive strength (Qu) (generally estimated or measured by hand penetrometer)

\[
\begin{array}{c|c|c|c|c|c}
\text{term} & \text{very soft} & \text{soft} & \text{firm} & \text{stiff} & \text{very stiff} & \text{hard} \\
\text{Qu kPa} & 0 & 50 & 100 & 200 & 400 \\
\end{array}
\]

If soil crumbles on test without meaningful result it is described as friable

Density Index (generally estimated or based on penetrometer results)

\[
\begin{array}{c|c|c|c|c|c}
\text{term} & \text{very loose} & \text{loose} & \text{medium dense} & \text{dense} & \text{very dense} \\
\text{density index %} & 15 & 35 & 65 & 85 \\
\end{array}
\]

ROCK DESCRIPTIONS

Weathering based on visual assessment

- Fresh: Rock substance unaffected by weathering
- Slightly Weathered: Rock substance affected by weathering to the extent that partial staining or partial discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable; strength properties are essentially those of the fresh rock substance
- Moderately Weathered: Rock substance affected by weathering to the extent that staining extends throughout whole of the rock substance and the original colour of the fresh rock is no longer recognisable
- Highly Weathered: Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and signs of chemical or physical decomposition of individual minerals are usually evident. Porosity and strength may be increased or decreased when compared to the fresh rock substance, usually as a result of the leaching or deposition of iron. The colour and strength of the original fresh rock substance is no longer recognisable
- Extremely Weathered: Rock substance affected by weathering to the extent that the rock exhibits soil properties i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident

Strength based on point load strength index corrected to 50 mm diameter 15(50) (refer IS R M., Commission on Standardisation of Laboratory and Field Tests Suggested Methods for Determining the Uniaxial Compressive Strength of Rock Materials and the Point Load Strength Index Committee on Laboratory Tests Document No 1) (Generally estimated \( x \) indicates test result)

\[
\begin{array}{c|c|c|c|c|c|c|c}
\text{classification} & \text{extremely low} & \text{very low} & \text{low} & \text{medium} & \text{high} & \text{very high} & \text{extremely high} \\
\text{Is (50) MPa} & 0.03 & 0.1 & 0.3 & 1 & 3 & 10 \\
\end{array}
\]

The unconfined compressive strength is typically about 20 x Is(50) but the multiplier may range, for different rock types from as low as 4 to as high as 30

Defect Spacing

\[
\begin{array}{c|c|c|c|c|c|c|c}
\text{classification} & \text{extremely close} & \text{very close} & \text{close} & \text{medium} & \text{wide} & \text{very wide} & \text{extremely wide} \\
\text{spacing m} & 0.03 & 0.1 & 0.3 & 1 & 3 & 10 \\
\end{array}
\]

Defect description uses terms contained on AS1726 Table D2 to describe nature of defect (fault, joint, crushed zone, clay seam etc.) and character (roughness, extent, coating etc.).
graphic symbols
soil and rock

SOIL

- Asphal tic Concrete or Hotmix
- Concrete
- Topsoil
- Fill
- Peat, Organic Clays and Silts (Pt, OL, OH)
- Clay (CL, CH)
- Silt (ML, MH)
- Sandy Clay (CL, CH)
- Silty Clay (CL, CH)
- Gravelly Clay (CL, CH)
- Sandy Silt (ML)
- Clayey Sand (SC)
- Silty Sand (SM)
- Sand (SP, SW)
- Clayey Gravel (GC)
- Silty Gravel (GM)
- Gravel (GP, GW)

ROCK

- Claystone (massive)
- Siltstone (massive)
- Shale (laminated)
- Sandstone (undifferentiated)
- Sandstone, fine grained
- Sandstone, coarse grained
- Conglomerate
- Limestone
- Coal
- Dolerite, Basalt
- Tuff
- Porphyry
- Granite
- Schist
- Gneiss
- Quartzite
- Tuff
- Talus
- Alluvium
- Pegmatite

SEAMS

- Seam >0.1 m thick
  (on a scale 1:50)
- Seam 0.01 m to 0.1 m thick
  (on a scale 1:50)

INCLUSIONS (Special purposes only)

- Rock Fragments
- Swamp
- Ironstone Gravel, Laterite
- Shale Breccia in Sandstone

Water Level

Surfaces ——— Known Boundary ——— Probable Boundary ——— Possible Boundary
**Engineering Log**

**Hand Auger & Dynamic Penetrometer**

**Client**: Public Works Department  
**Project**: Tarban Creek Seawall  
**Location**: 4m inside seawall, upstream end  
**Field Work by**: BMCD  
**Date**: 10/9/93  
**Sounding No**: TCI  
**Sheet**: 1 of 1  
**Office & Job No**: S9986/1

<table>
<thead>
<tr>
<th>Hole Diameter</th>
<th>Hammer Mass</th>
<th>Hammer Drop</th>
<th>RL Surface</th>
<th>Blow Count</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>100mm</td>
<td>9kg</td>
<td>510mm</td>
<td>not measured</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Material and Structure</th>
<th>Classification</th>
<th>Consistency Index</th>
<th>Blows/150mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW 10</td>
<td>Grained, dark grey, silt, root fibres top</td>
<td>M D</td>
<td>1 1 1</td>
<td></td>
</tr>
<tr>
<td>SW 20</td>
<td>Clayey sand: fine grained, dark grey, high plasticity clay, trace of shell &amp; wood fragments, strong rotting smell, Alluvium</td>
<td>MD</td>
<td>3 2 2</td>
<td></td>
</tr>
<tr>
<td>2.7m</td>
<td>Stopped hole because auger screwing into ground could not control.</td>
<td>VI</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

**Penetrometer**

- **Soil Type**: Plasticity or particle characteristics, colour, secondary and minor components, fissures, roots, residual structure.
- **Consistency Index**: Classification symbols based on unconfined classification system.
- **Moisture**: Dry, M, Moist, W, Wet.
**Public Works Department**

**Tarban Creek Seawall**

downstream of pumping station

---

**Table:**

<table>
<thead>
<tr>
<th>Hole Diameter</th>
<th>Hammer Masts</th>
<th>Hammer Drop</th>
<th>N.L. Surface</th>
<th>Datum</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Notes:**

3 attempts made to drill in this area - all refused at 0.8-0.9m

Interpret as Sandstone boulder fill.

---

**Penetrometer:**

- **Support:** Eastern States 915kg 300mm drop
- **Water:**
  - 10 Apr 93 water level on date shown
  - **water table**
  - **water outflow**

---

**Classification:**

- **Classification Symbols and Soil Description:**
  - Based on unified classification system

**Notes:**

- **Samples and Pens:** Undisturbed sample 50mm diameter
- **Controlled Sample:**
  - **D:** disturbed sample

**Consistency/Density Index:**

- **V0:** very soft
- **S:** soft
- **F:** firm
- **M:** medium
- **H:** hard
- **V:** very loose
- **L:** loose
- **D:** dense
- **V0:** very dense
**Public Works Department**

**Tarban Creek Seawall**

4m inside seawall, NE Facing section

**hole diameter** | **hammer mass** | **hammer drop** | **R.L surface** | **datum** | **blow count**
--- | --- | --- | --- | --- | ---

| 10 | SW | SAND, medium grained, dark | M | 1 1 1 | -- |
| 10 | SP | fibre, TOPSOIL | M | 2 2 3 | -- |
| 20 | SAND: fine, trace silt, brown, grey colour. | MD | 1 1 1 | -- |
| 20 | FILL | VL | 1 | -- |

Approximate base of seawall

| 30 | CLAYEY SAND: fine, dark grey, trace of shells & vegetation, clay high plasticity. ALLUVIUM | MD | 2 2 1 | -- |
| 30 | Auger pulling down strongly, could not control. | MD | 2 2 1 | -- |

40

**penetrometer**

**support**

- **casing penetration**
- **305mm**

**notes**

- **USO**
- **samples and tests**
- **undisturbed sample 50mm diameter**
- **displaced sample**

**classification symbols and soil description**

- **classification system**

- **consistency/compaction index**

- **V* very soft**
- **S* soft**
- **F* firm**
- **V** very firm
- **H** hard
## Tarban Creek Seawall

4m inside seawall, downstream end

<table>
<thead>
<tr>
<th>Hole diameter</th>
<th>Hammer mass</th>
<th>Hammer drop</th>
<th>R.L. surface</th>
<th>Datum</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5m</td>
<td></td>
<td>500mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5m</td>
<td>260kg</td>
<td>500mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10m</td>
<td>400kg</td>
<td>500mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15m</td>
<td>400kg</td>
<td>500mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20m</td>
<td>520kg</td>
<td>500mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25m</td>
<td>520kg</td>
<td>500mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30m</td>
<td>692kg</td>
<td>500mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>40m</td>
<td>692kg</td>
<td>500mm</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Material and Structure

- **SW:** SAND: fine grained, some silt, root fibres, topsoil
  - SAND: fine to coarse, brown grey, some fine gravel. Sandstone bedding

### Notes

- **FILL:**
  - CLAYEY SAND: fine grained, shale & sandstone gravel
  - SANDY CLAY: medium
  - CLAYEY SAND: fine grained, dark grey, some shells & vegetation, ALLUVIUM

- **Limit of Auger Control Putting down into Soil**
# Engineering Log

## Hand Auger & Dynamic Penetrometer

### Public Works Department

**Name:** Tarban Creek Seawall

**Location:** 4m inside seawall, down stream end

**Date:** 10-9-93

**Log Work by:** RMCD

<table>
<thead>
<tr>
<th>Hoie Diameter</th>
<th>Hammer Mass</th>
<th>Hammer Gong</th>
<th>RL Surface</th>
<th>Datum</th>
</tr>
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<tr>
<td>10</td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>20</td>
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</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Material and Structure**

- **SW:** SAND, fine grained, some
- **CL:** SANDY CLAY, medium
- **SC:** CLAYEY SAND, fine grained, shale & sandstone gravel
- **FILL:**

**Soil Type:**
- Plasticity or particle characteristics
- Colour secondary and minor components
- Textures, roots, residual structure

**Additional Notes:**
- Limit of auger control putting down into soil
- Putting down into soil

**CoRfey Partners International Pty Ltd**

ACN: 005 657 014

**Sounding No.** TC4

**Sheet:** 1 of 1

**Office and Project No.** S9966/1