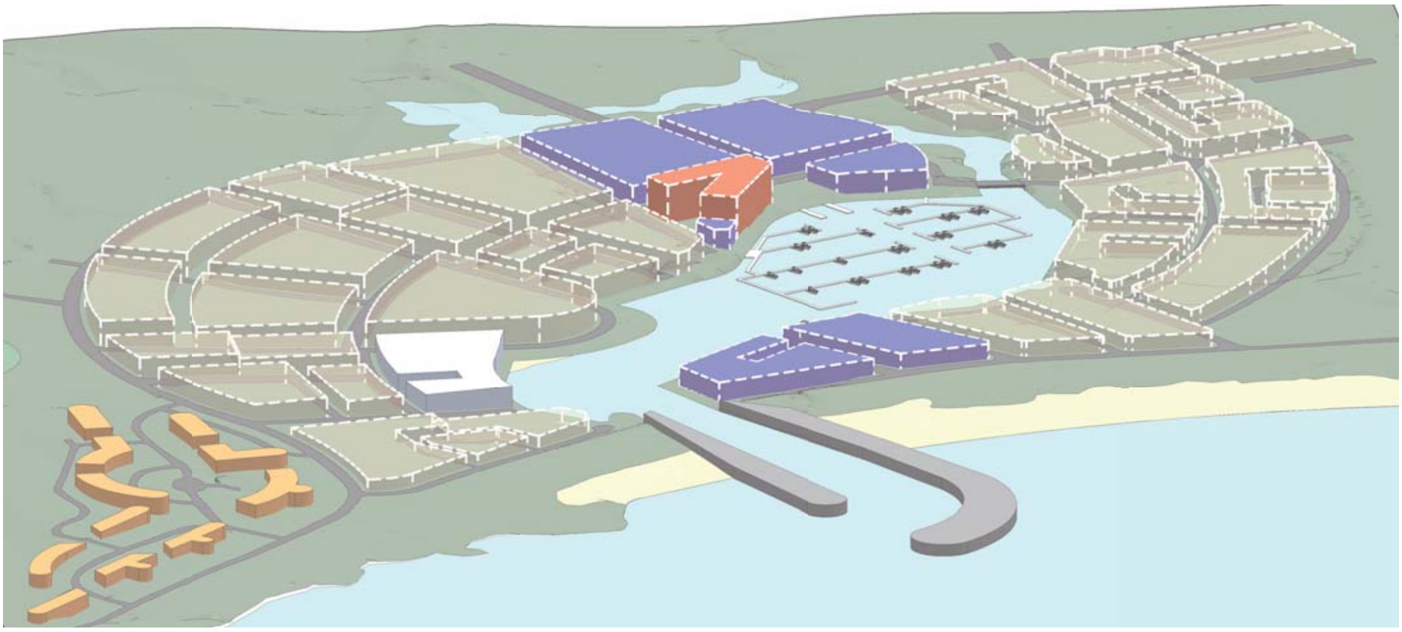


Shell Cove
Boat Harbour Precinct

Concept Plan Application
and Environmental Assessment
Appendix D - Geotechnical

prepared by LFA (Pacific) Pty Ltd

date February 2010

RESPONSE TO DIRECTOR GENERAL'S ENVIRONMENTAL ASSESSMENT REQUIREMENTS

Australand Holdings Pty Ltd
Shell Cove Boatharbour

GEOTUNAN02058AO-CH
18 September 2009

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

50 YEARS AN EXTRAORDINARY JOURNEY

18 September 2009

Australand Holdings Pty Ltd
PO Box A148
Shellharbour NSW 2529

Attention: Glenn Colquhoun

Dear Glenn,

RE: RESPONSE TO DIRECTOR GENERAL'S ENVIRONMENTAL ASSESSMENT REQUIREMENTS 5.4, 5.6 & 6.2 CONCEPT PLAN SUBMISSION UNDER PART 3A SHELL COVE BOAT HARBOUR

Coffey Geotechnics Pty Ltd is pleased to present the following report on our response to the Director General's Environmental Assessment Requirements 5.4, 5.6 and 6.2.

The attached document titled "Important Information about your Coffey Report" should be read in conjunction with this report.

Should you have any question in relation to this response please contact the undersigned.

For and on behalf of Coffey Geotechnics Pty Ltd

JON THOMPSON CPEng

Principal

Distribution: Original held by Coffey Geotechnics Pty Ltd

1 copy to Coffey Geotechnics Pty Ltd

3 bound copy to Australand Holdings Pty Ltd

1 electronic PDF copy to Australand Holdings Pty Ltd

Appendices

Appendix A: List of Figures with Reference to Previous Coffey Reports

1. Interpreted Extent of Estuarine (Unit 3) ASS Material – GEOTUNAN02058AM-AN – Figure 5
2. Interpreted Thickness of Estuarine Unit 3 (A & B) - GEOTUNAN02058AM-AN – Figure 6A
3. Aerial Photo of Site Showing All Previous and Current Test Locations and Terrain Regimes - GEOTUNAN02058AM-AN – Figure 1
4. Interpreted Thickness of Estuarine Unit 3B - GEOTUNAN02058AM-AN – Figure 6B
5. Reduced Level to Top of Rippable Material – GEOTUNAN02058AO-AC – Figure 1 - DRAFT
6. Groundwater Contours Based on the Calibrated Model Steady State – SC2058/2-BP – Figure I4
7. Groundwater Contours – Steady State Conditions for Excavated Harbour – SC2058/2-BP – Figure I5
8. Preloading areas – GEOTUNAN02058AM-AN Figure 7

1 INTRODUCTION

At the request of Australand Corporation (NSW) Pty Ltd, Coffey Geotechnics Pty Ltd (Coffey) has prepared this response to the Director General of Plannings Environment Assessment Requirements for the Shell Cove Boat Harbour Concept Plan Submission under Part 3A.

Coffey have been appointed to date by Australand in conjunction with the lead Boat Harbour design consultant, Worley Parsons, to provide geotechnical advice during the design phase and tender documentation phase of the Boat Harbour. This response references specific investigations undertaken as part of the Boat Harbour design and tender documentation consultant works.

Coffey has responded to the following Director General Requirements:-

Requirement 5.4 – Acid Sulfate Soils

> "Identify the presence and extent of acid sulfate soils on the site and where relevant appropriate mitigation measures in accordance with the Acid Sulfate Soil Manual (NSW Acid Sulfate Soil Management Advisory Committee, 1998)"

Requirement 5.6 – Geotechnical

> "Provide an assessment of any geotechnical limitations that may occur on this site and where necessary, appropriate design considerations that address these limitations."

Requirement 6.2 – Surface and Groundwater Hydrology and Quality

> "Assess the impacts of the proposed on surface and groundwater hydrology and quality."

Our response in relation to Requirement 5.4 includes reference to a number of investigations by Coffey and others. In relation to mitigation measures, the specification for the boat harbour construction works by Worley Parsons describes requirements and procedures for management of the acid sulfate soils during the Boat Harbour works.

Our response in relation to Section 5.6 includes reference to previous geotechnical investigations by Coffey, and appropriate design considerations outlined in the specification by Worley Parsons.

Our response in relation to Requirement 6.2 discusses the groundwater hydrology and quality. Surface water quality is not discussed in this report and is understood to be dealt with in other consultant reports.

This report presents a summary of the ASS, geotechnical and groundwater conditions and limitations over the site, based on earlier investigation reports by Coffey and others and discusses mitigation measures or design considerations, where appropriate.

2 GENERAL BACKGROUND INFORMATION

The subsurface conditions encountered in previous investigations have been subdivided into a number of stratigraphic units which are broadly consistent with the stratigraphic units previously described in past reports and in the Coffey Stage 1, 2 and 3 investigations at the site.

These units are summarised as follows:

Unit 1 **Fill**, which is divided into two subunits – Units 1A and 1B:

Unit 1A '**Clean' Fill** comprising gravelly clay and clay of medium to high plasticity with some cobbles. This material is generally in a moist and stiff condition.

Unit 1B **Refuse Fill** comprising mixtures of waste materials such as bricks, glass, car bodies, wire and general household refuse with a varying proportion of gravelly clay or clayey gravel.

Unit 2 **Littoral Sands** consisting typically of an upper layer of sand and a lower layer of silty sand/sandy silt. These materials are inferred to have been deposited in a combination of beach and dune environments. This unit is inferred to be generally in a loose to medium dense condition.

Unit 3 **Estuarine Sediments (Acid Sulfate Soils)** The estuarine sediments are generally dark grey to black in colour and have a high moisture content. The estuarine sediments are divided into two subunits – Units 3A and 3B as described below:

Unit 3A **Sand**: comprising silty sand and sand: this unit was generally loose to medium dense and encountered in the eastern parts of the site at the interface with the Unit 2 Littoral sands.

Unit 3B **Silt/Clay**: comprising clayey silt/silty clay and clay: This unit was generally very soft to firm. Some organic material and sandy lenses were encountered within this unit.

Unit 4 **Alluvium** consisting of silty clay, sandy clay and gravelly clay of medium to high plasticity. This unit was generally mottled brown, grey, dark brown and/or orange-brown and contained some fine subrounded gravel. This unit was generally stiff to very stiff with some firm zones.

Unit 5 **Residual Soil/Extremely Weathered Rock** which is gravelly clay/clayey gravel derived from insitu weathered latite. The consistency of this unit ranges from very stiff to hard. This unit tends to grade from residual soil to extremely weathered rock with increasing depth.

Unit 6 **Rock**, consisting of highly weathered to fresh Latite, divided into two sub units 6A and 6B. Both units were porphyritic and massive.

Unit 6A **Highly to Moderately Weathered Latite** which is brown and contains some clayey infilled joints and seams. The rock is generally medium to high strength with a defect spacing of less than 100mm. Whilst Unit 6A is highly fractured and contains many defects, some high to extremely high strength rock bands are present.

Unit 6B **Moderately Weathered to Fresh Latite** which is generally grey to dark grey and of high to extremely high strength. The occurrence of rock defects is less than in Unit 6A.

Groundwater inflows or standing water levels were encountered in all boreholes drilled as part of the Stage 1, 2 and 3 investigations at the site. No piezometers or groundwater monitoring wells were installed as part of the Stage 3 investigation works, hence standing water levels that were measured in the current series of boreholes as part of the Stage 3 investigation should be considered approximate only. Standing water levels recorded in piezometers and groundwater monitoring wells during the Stage 2 investigation were generally recorded as being between +0.5m and +2.0m AHD.

The site has also been split in to various ‘Terrain Units’. These Terrain Units provide a useful summary for the reader of the ground conditions in plan or ‘birds eye’ view of the site at the time of investigation. The units are outlined from ‘A’ to ‘F’ and are included in the Figure list as the second, fourth and eighth figure.

3 DIRECTOR GENERAL’S REQUIREMENT 5.4 – ACID SULFATE SOIL

3.1 Presence and Extent of Acid Sulfate Soil (ASS)

3.1.1 Previous Investigations and Assessments

Several Investigations have previously been carried out at the site of the proposed boat harbour which has addressed numerous issues including Geotechnical, Acid Sulfate Soil (ASS) and Groundwater. A number of reports were prepared by others dating back to 1983, including Public Works Department (December 1983, November 1984 and May 1985), Dames and Moore (December 1984), Golder Associates (April 1987 and May 1995), Walker Civil (1995), CSIRO – Ian White (March 1995) and Douglas Partners (May 2001 and November 2002). A summary of the involvement by Coffey in the major investigations carried out to date is presented below. The Coffey Stage 1 and 2 assessment reports and previous background information are referenced within this report. The test locations from these earlier reports and the three stages of Coffey reports are presented in Appendix A, Figure 1.

Stage 1 Report

Coffey carried out a Stage 1 assessment at the site in 2003, which included reviewing the relevant sections of the previous reports relating to geotechnical and ASS issues and some additional limited fieldwork to assess four issues, namely:

1. Potential for unconfined sea disposal of ASS;
2. Implications of a rotation of the boat harbour platform;
3. Potential for mechanical excavation of the ASS and other related geotechnical issues; and
4. Comparison of methods of testing for ASS.

This work was reported in Stage 1 report Ref: SC2058/1-AH, dated 19 September 2003, where as part of this report Coffey carried out additional boreholes, sampling of ASS, ASS lab testing and various other tasks.

Stage 2 Report

Coffey carried out a Stage 2 assessment at the site in 2004, which included additional fieldwork, laboratory testing and analysis to cover the following issues:

1. The extent of ASS;
2. Groundwater characteristics at the site including water levels, permeability of different soil units, dewatering issues during construction and modelling the effects of groundwater drawdown during and post harbour construction.

This work was reported in a Stage 2 report Ref: SC2058/2-BR, dated 27 October 2004.

3.1.2 Sub-Surface Conditions

The subsurface conditions at the site have been assessed through numerous intrusive sampling locations totalling over more than 110. The data from these locations was used to develop an interpreted geological model for the site which divided the subsurface into 6 main units. The Estuarine Sediment Unit (Unit 3) was assessed to be ASS and is described below:

Unit 3 **Estuarine Sediments (Acid Sulfate Soils)** The estuarine sediments are generally dark grey to black in colour and have a high moisture content. The estuarine sediments are divided into two subunits – Units 3A and 3B as described below:

Unit 3A **Sand**: comprising silty sand and sand: this unit was generally loose to medium dense and encountered in the eastern parts of the site at the interface with the Unit 2 Littoral sands.

Unit 3B **Silt/Clay**: comprising clayey silt/silty clay and clay: This unit was generally very soft to firm. Some organic material and sandy lenses were encountered within this unit.

Groundwater Inflow levels were encountered in the boreholes drilled between 0.6m (CGBH45) and 2.5m (CGBH43) below ground surface. Converted to metres AHD, groundwater inflow levels ranged between 0.8mAHD (CGBH44) and 1.42mAHD (CGBH45). Borehole CGBH47 was drilled within a partially inundated marsh area and no distinct groundwater zone was detected beneath the surface water. Standing water levels were not recorded. Boreholes were terminated on Unit 4 or Unit 5. Boreholes CGBH 46 and 47 were terminated on Unit 5 material due to practical refusal.

3.1.3 Lateral Extent of Acid Sulfate Soils (ASS)

Information on the extent of ASS was collected during investigations by Coffey from field mapping, logging of test pits, boreholes and vibrocores, ASS screening and SPOCAS laboratory testing was used to compliment previously existing data to better assess the extent of ASS at the site. ASS have been assessed as occurring in the Estuarine (Unit 3) soils and the extent of ASS has therefore been based on the extent of the Unit 3 soils.

Other information which was used in conjunction to the above included historical aerial photographs (dated 1948, 1966 and 2003), ASS Risk Map (Albion Park, 1:25,000) published by the former Department of Land and Water Conservation, 1:25,000 Albion Park Topographic Map, survey plan (Patterson Britton Drawing No. 4717-EFV1) and surface contour plan provided by BMD Consulting.

Field mapping of the potential extent of ASS which corresponds to Estuarine (Unit 3) soil was carried out by a Senior Environmental Engineer from Coffey in 2004. Field mapping of the potential extent of ASS was mainly based on observations of the local topography and surface conditions and complimented with information from aerial photographs, topographic maps, ASS risk maps, survey maps, previous mapping and available subsurface and laboratory data. Observations of soils exposed within the walls of existing ponds, creeks and drains was also used to assess the potential extent of the ASS.

A Global Positioning System (GPS) was used to obtain co-ordinates of about 80 site features and ground points approximating the potential extent of the ASS around the northern, western and southern perimeters of the site.

This extent of ASS soils is presented in Appendix A, Figure 5. This figure also shows previous test locations where estuarine soils (Unit 3) have been inferred to be identified (represented as a red dot), and the locations where this unit is not likely to be present due to the presence of deeper alluvial soils, residual soils or rock (represented as a blue dot).

3.1.4 Thickness of Acid Sulfate Soils (ASS)

Coffey transcribed the relevant previous subsurface data into gINT database, a contour plot of the estimated thickness of the Estuarine Unit 3 soils was produced from the information contained in Coffey gINT database. Figure 6A presents the total thickness of estuarine Unit 3 (A and B) soils. This figure was produced using Surfer® software package. The contours shown in the figure are based on the amount of subsurface data available and some extrapolation is carried out by the program. The approximate extent of ASS has been overlain on the figure to represent the boundary where ASS is likely to occur. Figure 6A is a schematic representation of potential thickness of ASS. As expected, the figure shows that the relatively thick ASS zones are located in the current wetland areas.

The delineation of AASS and PASS for the purpose of ASS management has been developed by Worley Parsons (WP) in conjunction with Coffey, with aid of data collected by Coffey. Extracts of the WP design specification are provided below that describe the delineation of ASS occurrence.

The Unit 3 estuarine sediments (ASS) can be separated into AASS and PASS depending on the degree of oxidation that has occurred. Generally, Unit 3 that has been permanently below the water table would be classified as PASS for which the pH is typically >4. The overlying Unit 3 is classified as AASS for which pH is <4. However, the condition for disposal of PASS as outlined in the Construction EMP and EPL is that the pH cannot be less than 5.5.

A number of geotechnical investigations have been undertaken in which it has been estimated that AASS does not generally occur greater than 2m below the top of Unit 3. For the purposes of preliminary disposal volume calculations, and as shown on the Drawings, it has been assumed that the top 2m depth of Unit 3 is AASS.

3.2 Acid Sulphate Soil – Mitigation Measures

The ASS mitigation measures for the construction of the Boat Harbour were mainly developed by Worley Parsons (WP) in association with Prof. Ian White of ANU, with aid of data collected by Coffey. Whilst the Boat Harbour construction and associated works to the land platform have separate approval and do not form part of this application, the mitigation measures developed for the construction of the Boat Harbour flow over into the land platform surrounding the Boat Harbour. Accordingly, any ASS management required after completion of the presently approved works is expected to comprise isolated and nominal treatment of excavated ASS. The treatment would comprise neutralisation with lime followed by either reuse at an appropriate location on site or offsite disposal (following appropriate waste classification).

Extracts of the WP design specification are provided below that describe the ASS mitigation measures for the approved Boat Harbour and associated land platform works.

ASS investigation measures from WP design specifications are outlined in Sections 3.2.1 to 3.2.3 below.

3.2.1 Excavated ASS

The preferred solution for the disposal of ASS from the Boat Harbour excavation and the localised 'chasing' within the land platform (i.e. for overland flow channels) has been selected with consideration of the material volume balance for the project and estimated costs. Factors have included AASS (Actual Acid Sulfate Soils) and PASS (Potential Acid Sulfate Soils) volumes, the volume and specifications of materials required for the capping and consolidation of insitu ASS in the Boat Harbour planform, and the earthworks staging and material availability for the project

In summary, the preferred ASS disposal strategy is as follows:

- over excavate within the Inner Harbour Boat Harbour planform, both by conventional ripping and PCF methods (if required) to create AASS disposal pits;
- excavate the AASS 'in-the-wet' by cutter suction dredger (CSD), pumping the resulting slurry into the AASS disposal pits within the Boat Harbour planform. Operation of the CSD may require perimeter bunds to be constructed around the excavation so as to provide a suitable water depth for efficient dredging. A 10cm thick sand capping layer is to subsequently be provided over the disposed AASS after > 90% self-consolidation; and
- excavate the PASS 'in-the-moist' by conventional earthworks machinery supported on firm underlying material, employing intermediate bunds and water sprays if required to prevent the PASS drying out. The PASS will then be transported to a nearby DECC licensed landfill using covered trucks. Three such landfills are available.

The area of the Boat Harbour planform to be over excavated has been estimated having regard to both the required storage capacity and the proposed construction staging.

To achieve reburial of AASS beneath the Boat Harbour planform requires initial excavation of the western side of the Inner Harbour which is in a non-ASS area. An insitu bund would be formed which would keep the ASS separated to the eastern side of the Inner Harbour.

The excavated AASS would ultimately be disposed beneath the design level of the Inner Harbour (-4.0mAHD) each side of the insitu bund by transferring a portion of AASS from the over-filled western side of the Inner Harbour to the eastern side of the Inner Harbour.

3.2.2 Insitu ASS Under the Land Platform

The insitu ASS located under the land platform adjacent to the Boat Harbour will be capped and consolidated. The site is being developed into residential and commercial precincts and a structural platform (i.e. fill layer) will be constructed over the ASS to support building and road loads. The structural platform shall act as a capping layer to the ASS.

‘Chasing’ of ASS in localised areas, where future finished levels are close to existing surface levels (e.g. overland flow paths) is required in order to construct the minimum structural platform requirements. These areas do not require capping and consolidation.

3.2.3 Excavation and Disposal of PASS

Excavation of the PASS underlying the AASS is to be undertaken using conventional equipment after dewatering the site. It is a requirement that the PASS shall be kept ‘moist’ to ensure it does not oxidise. Excavated PASS will be removed from the site and placed in a licensed DECC landfill.

The Contractor must comply with the specific requirements in the Environment Protection Licences for each DECC/Council landfill for the disposal of the PASS

4 DIRECTOR GENERAL’S REQUIREMENT 5.6 – GEOTECHNICAL

4.1 Geotechnical Limitations

Most of the site contains stratigraphic units typical of the local area including Alluvial Plains and Residual Soils. These soil types are able to support buildings and infrastructure with a range of commonplace structural solutions and consequently do not create any significant geotechnical limitations.

However, the following potential geotechnical limitations have been identified in the eastern sections of the site during the background studies carried out by Coffey Geotechnics:

- Soft soils subject to settlement,
- Risk of liquefaction

These matters are discussed in the following sections.

4.1.1 Soft Soils (Insitu Estuarine (Unit 3B) Soils) Settlement and Consolidation

4.1.1.1 Ground Treatment of Compressible Soils

Significant thicknesses of compressible Unit 3B clayey silt/silty clay type soils have been identified in multiple geotechnical investigations carried out at this site. The extent and thickness of these compressible soils is shown in Figure 6B from Coffey Report GEOTUNAN02058AM-AN titled ‘Interpreted Thickness of Estuarine Unit 3B’.

Much of the land apron surrounding the future Boat Harbour will require ground treatment so that future ground settlements beneath engineered structures (buildings, roads and services) can be controlled within tolerable limits.

Ground treatment of soft soils at this site will be undertaken as part of the approved Boat Harbour construction works. Conventional preloading methods have been selected as the preferred methodology for improving the engineering characteristics of the soft soils on the site. This methodology has been adopted due to the relative cost effectiveness of the method and general acceptance of the use of the method in the construction industry. Preloading both without wick drains and with wick drains is discussed in this report.

4.1.1.2 Lateral Extent of Preload Mounds

An assessment of the required lateral extent of future preload mounds has been carried out along with construction details of the Preload Mounds in relation to the edge of the Boat Harbour and recommendations as to the location of the edge of the preload mounds during construction of the Boat Harbour to control differential and total settlement to within tolerable limits.

Most of the potentially compressible ASS soils will be excavated around the edge of the Boat Harbour during construction, and surcharge mounds will extend to surcharge compressible soils nearby the edge. For the current construction methods and ground profiles, it is assessed that total settlement and differential settlement around the edges of the Boat Harbour will be within tolerable limits as outlined in the criteria for the surcharging strategy at this site.

We note that the success of surcharging of soft soils is highly dependent on the gathering of good quality monitoring data during and post construction. During boat harbour construction, settlement monitoring involving the installation of settlement plates, vibrating wire piezometers and hydrostatic profile gauges has been recommended at the site by Coffey. This type of instrumentation is necessary within this ‘edge treatment area’ as well as this area forms part of the surcharging strategy.

4.1.1.3 Time Rate of Settlement without Wick Drains

A significant variable in ground treatment employing preloading techniques is estimation of the time for completion of each preload mound at a site. In general, the completion of preloading can only be adequately assessed after the soft soil within a preload area has reached >90% primary consolidation. This is observed through regular monitoring of settlement monitoring equipment installed in the preload mound. Settlement monitoring usually occurs at fortnightly or monthly intervals and continues before, during and immediately after the completion of preloading. Greater than 90% primary consolidation is noted following ‘flattening off’ of a Log(time) vs settlement graph. This usually corresponds with dissipation of excess pore pressures within the soft clay unit that were generated by the construction of the preload mound.

The time rate of settlement of future preload mounds has been estimated based primarily on dissipation testing carried out on piezocones performed as part of the Stage 2 Coffey Report. This provides a reasonable initial assessment of c_v and the duration of preloading works can be estimated. However, no amount of initial geotechnical investigation work can fully replace data gained from full scale field preload trials carried out at the project site. Full scale field preload trials are the best method of assessing preload duration and other parameters such as creep.

The time rate of primary consolidation settlement for 2m, 3m, 4m and 5m thick Unit 3B soils has been predicted based on estimated c_v and the somewhat conservative assumption of one way drainage. It is assessed that two way drainage could potentially not exist at this site due to the presence of the low permeability clayey Unit 4 or Unit 5 soils being encountered beneath the Unit 3B soils. Conversely, if two way drainage could be proven by trial pad monitoring, the time rate of settlement would be about four times faster than the current prediction based on one way drainage.

The c_v of the Unit 3B soils at the site has previously been estimated as being somewhat variable depending on the thickness of the soft soil deposit:

- Thickness of Unit 3B ≤ 2m - c_v = 4m^2/year
- Thickness of Unit 3B >2m - c_v = 2.5m^2/year

For the purposes of this preloading strategy, an average c_v of 3m^2/year has been assumed for the entire thickness of the soft soil deposit.

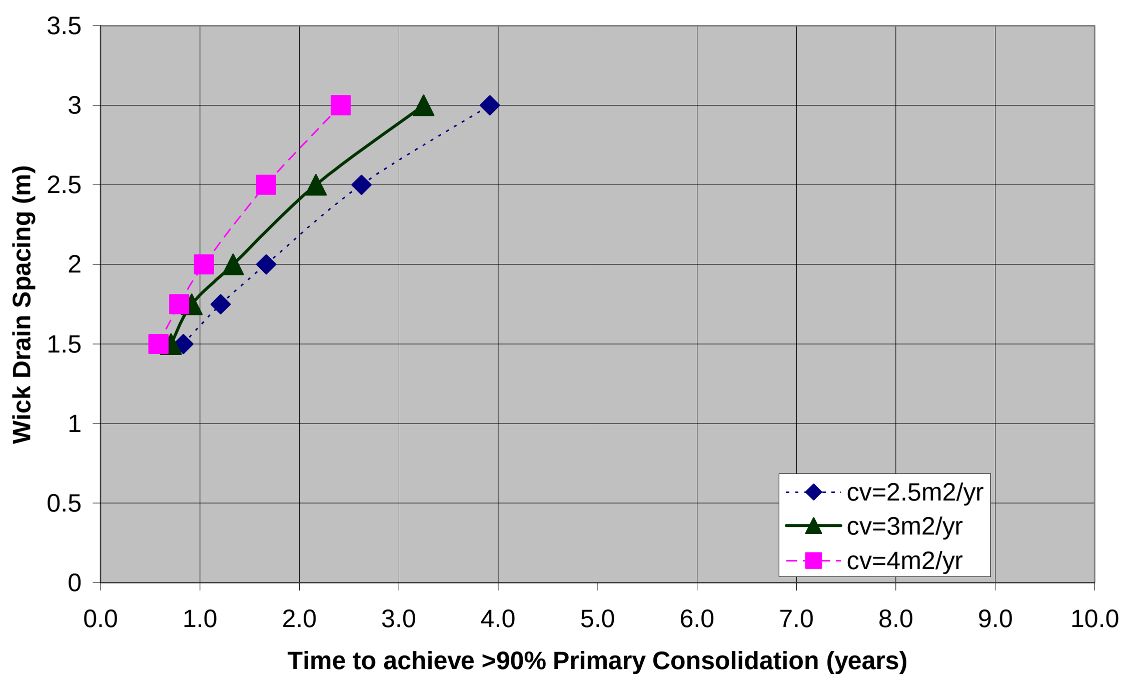
The time rate of settlement is estimated as follows:

TABLE 1 TIME RATE OF SETTLEMENT (WITHOUT WICK DRAINS)

Thickness of Unit 3B soils	**Approximate Time for 90% Primary Consolidation (years)**		
	c_v = 2.5m^2/year	**c_v = 3m^2/year**	**c_v = 4m^2/year**
2m	1.4	1.2	0.9
3m	3.2	2.7	2.0
4m	5.8	4.8	3.6
5m	9.0	7.5	5.6

With completion of field preload trials it would be possible to refine this prediction of the time rate of settlement of the soft soil under the preload mounds.

Due to the excessively long timeframe for consolidation, it is proposed that wick drains be installed to increase the rate of consolidation to about 1 year. Graph 2 from the Stage 3 Coffey Report shows the benefits obtained from using wick drains.



Graph 2 With Wick Drains: Preload Duration (time) plotted against Wick Drain Spacing for various assumed Coefficients of Vertical Consolidation

4.1.2 Risk of Liquefaction

Upon reference to Table 2.3 from AS1170.4-1993[1] the acceleration coefficient (*a*) for Wollongong is 0.08.

The site factor (*S*) is given in Table 2.4(a) and Table 2.4(b) for different soil profiles. For Domestic Structures[2], the *current* site factor from Table 2.4(b) is assessed as 2.0. For General Structures, the site factor from Table 2.4(a) is assessed as 1.4.

These site factors could be reassessed and reduced somewhat following ground improvement works (eg. preloading), removal of uncontrolled fill and placement of controlled fill. For Domestic Structures, the site factor *following ground improvement and placement of controlled fill* from Table 2.4(b) is assessed as 1.0. For General Structures, the site factor from Table 2.4(a) is assessed as 1.2.

Based on the design values given in AS1170.4, an assessment of Liquefaction Potential has been made with reference to methods outlined by Bolton Seed and others[3,4].

The results of the assessment indicate that out of all the soil units identified at the site, only the very loose to loose Estuarine Unit 3A (Silty Sand) soils are potentially susceptible to liquefaction. These soils are about 1m to 2m thick and were only encountered over small areas of the site near Boollwarroo Parade and between Boollwarroo Parade and the Pacific Ocean. For the Shell Cove Boat Harbour development, the affected areas lie beneath the breakwater and harbour entrance areas and therefore do not impact on the development proposal within the Part 3A Application.

For the majority of the site, Unit 3A soils are absent and it is assessed that these areas will not be affected by liquefaction under earthquake loads indicated in AS1170.4.

4.2 Design Considerations to Address Geotechnical Limitations

4.2.1 Treatment of Soft Soil Areas

The following approach has been adopted for treatment of soft soils within the land platform as part of the approved Boat Harbour works.

4.2.1.1 Preload (Surcharge) Mounds

To avoid having to support lightly loaded engineered structures on piled foundations, preloading will be undertaken as part of the approved Boat Harbour construction works to reduce post construction settlement and differential settlement to tolerable levels. It is assessed that after sufficient preloading, it should be feasible to construct articulated masonry veneer or more flexible types of structures provided they are not more than about 20m in size and are supported on stiffened raft foundations that are

[1] 'Minimum Design Loads on Structures, Part 4: Earthquake Loads' (1993) Homebush: Standards Australia

[2] In Section 2.2.2 of AS1170.4-1993, Domestic Structures are 'detached single dwellings, terrace houses, townhouses and the like'. The standard gives specific limitations for domestic structures such as the maximum height and width of structural elements.

[3] Bolton Seed, H and others, 'Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations' (1985) Journal of Geotechnical Engineering, Vol 111, No. 12, December 1985.

[4] Bolton Seed, H and others, 'Evaluation of Lieuqfaction Potential Using Field Performance Data' (1983) Journal of Geotechnical Engineering, Vol 109, No. 3, March 1983.

equivalent to, or stiffer than, those recommended in AS2870-1996 for **M** class sites. More substantial structures should be feasible with appropriate foundation and structural solutions.

The assessment of strategies for limiting long term settlement has been made based on the following design criteria:

- Design life of 60 years
- Long term (post preloading) total settlement less than 40mm
- Building loads of 25kPa (considered as a uniform load over the site)
- Wick Drains are installed as required to reduce the duration of preloading[5].
- Preloading will be carried out assuming a specific construction methodology. The construction criteria will involve placement of compacted fill up to the final design surface level followed by the placement of a *compacted* preload mound above the design surface.
- The total height of preload mound above final design surface level equals the calculated height of preload fill *plus* the estimated primary consolidation settlement over the duration of preload. Following satisfactory preloading, the compacted preload would be removed, and extra compacted fill placed (as required) to bring levels up to final design level.

It is difficult to quantify differential settlement as distinct from absolute settlement, but by designing post construction settlement to 40mm over 60 years by preloading, then differential settlement should be manageable. For structural design purposes, the differential settlement within one building could be taken as half of the total settlement nominally, i.e. 20mm.

Preload mounds will be required to overlap preload areas to minimise edge effects. The extent of preload mounds has been discussed in Section 4.1.1.2.

The final site classification following ground improvement works will need to be reviewed on the basis of settlement and pore water pressure monitoring results. Typically, the lots could be designed for Class “M” conditions as advised above. However, if the ground conditions are worse than what we assumed above in localised areas, some lots may need to have foundations designed in accordance with a more severe classification than ‘M’. Preload modelling provides an *estimate* of the time for preloading and settlement during preloading, but is an observational approach and the progress of preloading will need to be monitored throughout the duration of preload.

Initially the preload assessment has been carried out by subdividing the Boat Harbour land areas into eight individual areas that were modelled separately. Some commonality was found on completion of the assessment within each of these eight areas, and subsequently three overall preload zones have been recommended. (refer Appendix A – ‘GEOTUNAN02058AM-AN – Figure 7’ for preload zones)

Table 2 shows the results of the preload strategy assessment.

The Preload Thickness, ‘$H_{compacted}$’, and the resulting total fill height are sensitive to changes in the reduced level of the final land platform. If the levels of the final land platform increase by more than

[5] Wick Drains only reduce the duration of preloading, but have no effect on the height of preload for a given preload area.

0.1m nominally, it is probable that the preload strategy and Table 2 will require some updating, resulting in changes in preloading earthworks volumes.

It should be noted that ‘$H_{compacted}$’ does not decrease significantly with extension of preload duration. In fact, ‘$H_{compacted}$’ varies by less than a few percent if the preload duration is extended from 1 year to 4 years. This is due to the fact that 90% degree of consolidation was targeted in the baseline design. Any longer preloading period would simply gain a maximum 10% further consolidation on a log time scale.

Table 2 Results of Preloading Assessment

C1	*C2*	*C3*	*C4*	*C5*	*C6*	*C7*	*C8*	*C9*	*C10*
Zone	Terrain Unit	Current Ground Level, RL (m)	Design Land Platform Level, RL (m)	Design Fill Thickness (m)	Max Unit 3B Thickness (m)	Adopted Design Fill Thickness (m)	Preload Thickness, $H_{compacted}$ (1) (m)	Total Fill Height including Preload Thickness, to RL (m, AHD)	Settlement During Preloading, (m)
1	B North, D-1 West	2 ~ 3	4 ~ 4.5	1.5 ~ 2.5	3.5	2.5	3.7 - 5.2	+7.2	0.35 - 0.75
2a	B Central, D-1 East	1 ~ 3	5.5	2.5 ~ 4.5	5	3.5 - 4.5	6.6 - 7.7	+8.7	0.75 - 0.95
2b	D-2 South, D-3	4	5.5	1.5	5	1.5	4.74 (2)	+8.75	0.6 - 0.7
3	B South	2	3.5	1.5	4.5	1.5	4.05	+6.0	0.5 - 0.6

Notes and assumptions relating to Table 2:

(1) Compacted fill (=20kN/m^3) has been adopted for the preloading design. If loose preload fill is used, the equivalent preload thickness can be calculated based on the unit weight ratio and the height of the preload mound increased. For example, if the unit weight of the loose preload fill is 16kN/m^3, the new preload height, $H_{loose} = (20/16) * H_{compacted}$

(2) The existing Unit 1B fill in the existing landfill area is assumed to be replaced with controlled fill of unit weight 20kN/m^3.

(3) The major difference between Preload Zones 2a and 2b is the amount of primary consolidation settlement that is predicted during preloading. For preload design purposes, the reduced level to the top of preload mounds in Preload Zone 2 can be assumed as being +8.75mAHD.

4.2.1.2 Wick Drains

Where the Unit 3B soil is thicker than about 2m to 3m, then depending on the duration available for preloading, wick drains would be beneficial in reducing the time for primary consolidation under preloading. The use of wick drains has been discussed previously in this report.

The time rate of primary consolidation settlement at the site has been carried out assuming wick drain spacings of 1.5m, 1.75m, 2.0m, 2.5m and 3m and a triangular grid. This method still requires preloading to the full height, but significantly reduces the preload duration through dramatically increasing the ability of water to escape from the soft clay (Unit 3B) unit.

Note that the duration for completion of preloading for a given wick drain spacing is independent of soft clay thickness. This is because water travels horizontally to the wick drain and not vertically through the soil profile.

4.2.1.3 Geotechnical Monitoring

The survey and ongoing monitoring of settlement monitoring locations (settlement plates and hydrostatic profile gauges) and the monitoring of pore pressure dissipation within the soft soil unit is considered critical to the success of the surcharging technique during boat harbour construction works. These tasks have been planned for and will be implemented during the boat harbour construction works.

5 DIRECTOR GENERAL'S REQUIREMENT 6.2 – GROUNDWATER HYDROLOGY

5.1 General Groundwater Conditions

Groundwater levels would be expected to be shallow in the wetland, near the surface and vary up to several metres below the surface across the site. Groundwater is expected to flow in a general easterly direction.

The presence of shallow groundwater levels over part of the site indicates that groundwater interacts with surface water. The shallow water table conditions mean that the capacity of the groundwater system to accept rainfall recharge is limited. This is expected to result in surface runoff into the wetlands following extended periods of high rainfall.

Based on field investigation and monitoring of groundwater levels, a groundwater model was established and calibrated to model the existing groundwater conditions and groundwater conditions following construction of the Boat Harbour by Coffey Geotechnics (2004). Modelled groundwater contours are shown on Figure I4 and I5 which are provided in appendix A.

5.2 Previous Reports

A report by Dames and Moore (1984) broadly assumed that the final water level in the harbour would fluctuate around mean sea level. Groundwater levels measured in the boreholes carried out as part of this report ranged from –2.8m to +5.3m AHD.

A report by Golder (87650027, 1987) in relation to the landfill area to the east of the inner harbour included a number of piezometers. These were monitored over three days. Groundwater elevations ranged from 1.3m to 1.6m AHD and variations ranging from 60 to 150mm were considered to be a result of tidal effects closest to the higher water mark. A hydraulic gradient from west to east of 0.25m in 100m was also calculated based on the water levels in five piezometers.

A May 1995 Golder Report indicates that groundwater is typically at shallow depths in the wetland areas, generally between 0.5m and 1.0m AHD. In the landfill area local elevation of the groundwater level to as high as 1.7m AHD was noted.

No estimate of the final water table (post harbour development) was made in the Golder reports.

5.3 Groundwater Study by Coffey

The results of the groundwater study by Coffey (2004) are presented in a report Ref: SC2058/2-BP, dated 27 October 2004. Groundwater contours based on the Calibrated Model Steady State and for Steady State Conditions for Excavated Harbour are presented in Appendix A, Figures I4 and I5. In summary, the results of the groundwater study indicated the following:

- Modelled groundwater drawdown impacts to the west and south-west of the Boat Harbour range from approximately 2m to 2.5m at the Boat Harbour edge and reduce to about 1m at a distance of 200m from the proposed Boat Harbour shore;

- Modelled drawdown impacts to the north of the harbour affect groundwater levels within the estuarine aquifer, the alluvial aquifer and the underlying latite. Modelled drawdown in these aquifers ranges from approximately 2m at the harbour edge reducing to 1m at a distance of 300m from the harbour shore. Modelled groundwater flow direction rotates from roughly eastward to southerly and south-easterly.
- The creation of the Boat Harbour will act to interrupt natural easterly groundwater flow to the beach and ocean area. Modelled groundwater levels show the presence of a gentle mound between the Boat Harbour and ocean with groundwater levels varying between 0mAHD and 0.5m AHD.
- Modelled drawdown impacts to the south of the proposed inlet channel reduce from approximately 1.5m at the harbour shore to zero at a distance of 200m from the harbour shore.

5.4 Impact of the development proposed under the Part 3A submission on long term groundwater hydrology and quality

Coffey has assessed the groundwater level changes associated with several development options. The changes are mainly induced by boat harbour construction works. The following is assessed for the long term groundwater levels at this site:

1. The Unit 3B Acid Sulfate Soils are compressible and require ground treatment to allow a long term land platform to be constructed around the perimeter of the Boat Harbour. In the absence of surcharging, some settlement (estimated in the order of 0.1m to 0.2m) will occur within 12 months of the construction of the capping materials above the ASS up to bulk construction levels. The estimated settlement of the Unit 3B soils following surcharging is between 0.35m and 0.95m, with an average total estimated of about 0.5m to 0.6m. The current top surface of the Unit 3B soils will ultimately be lowered by these estimated amounts following surcharging.
2. The majority of the Acid Sulfate Soil materials around the perimeter of the Boat Harbour (within about 20m to 30m of the edge of the Boat Harbour) will be excavated and treated or disposed of in an approved manner in accordance with the Acid Sulfate Soil Management Plan. This is to occur in accordance with the treatment measures that are to be implemented around the edge of the Boat Harbour as discussed in the construction requirements for the edge treatment measures.
3. Service trenches are often installed using granular, permeable aggregate backfill materials. Service trenches can cause dewatering of surrounding soil materials if the granular backfill materials are installed below a groundwater inflow source and connected hydraulically by gravity to an outlet with a lower water level. Most service trenches for buried services such as sewer, water and other services will not extend to below about +0.5mAHD and therefore should not encounter groundwater. Where service trenches extend below the water level, consideration should be given in the trench design to the implications to the local groundwater regime. Coffey can assist with the groundwater issues and design aspects for trenching below the groundwater level, if required.
4. Long term groundwater levels will be governed by tidal levels within the Boat Harbour, sea level rise due to climate change and water recharge from rainfall or other man made sources such as buried service trenches. In the absence of continuous dewatering, groundwater levels will not fall below 0mAHD in the long term and would be more likely lie at about +0.5mAHD due to factors mentioned above.

5. The current top surface of the Unit 3 Acid Sulfate Soil materials lies at about +0.5m to +1.0m AHD. These soils will be surcharged and will be lowered by about 0.5m to 0.6m through the combined effects construction of the capping materials and surcharging.
6. The capping materials would act as a very low permeability cover over the Acid Sulfate Soil materials at this site. This very low permeability cover would act to restrict oxidation of the Acid Sulfate Soil.
7. It is assessed that the lowering of the top surface of the Acid Sulfate Soil materials, combined with the long term groundwater level within 20m of the Boat Harbour tending to equalise at about +0.5mAHD and the presence of the very low permeability capping over the ASS, would not allow Acid Sulfate Soil materials to oxidise to a greater extent than that allowed for during the construction phase for the Boat Harbour. Based on this assessment, it is assessed that groundwater quality should not be significantly affected over the long term post construction of buildings, service trenches and pavements at this site.
8. The top surface of PASS (Potential Acid Sulfate Soils) has been previously investigated and found to be consistently lower than the top surface of Actual Acid Sulfate Soil. Long term groundwater levels will be above the PASS level and will therefore restrict any further oxidation rates of the PASS material to levels at or lower than oxidation rates for PASS found in the current environment.

6 REFERENCES

References

1. Coffey Stage 1 Report – SC2058/1-AH
2. Coffey Stage 2 Report – SC2058/2-BR
3. Coffey Stage 3 Report – GEOTUNAN02058AM-AN
4. Coffey Draft Report Assessment of Rock Excavation – GEOTUNAN02058AO-AC
5. Worley Parsons Shell Cove Boat Harbour Technical Specification Issue No.2 November 2007
6. Coffey Report on Assessment of Settlement – Edge Treatment Areas – GEOTUNAN02058AO-BW dated 22 July 2008
7. Dames and Moore Report (1984) Shell Cove Boat Harbour
8. Golder Report (ref: 87650027, 1987) Shell Cove Boat Harbour
9. Golder Report (ref: ________, 1995) Shell Cove Boat Harbour

Appendix A

List of Figures with Reference to Previous Coffey Reports

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Important information about your **Coffey** Report

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment.
Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

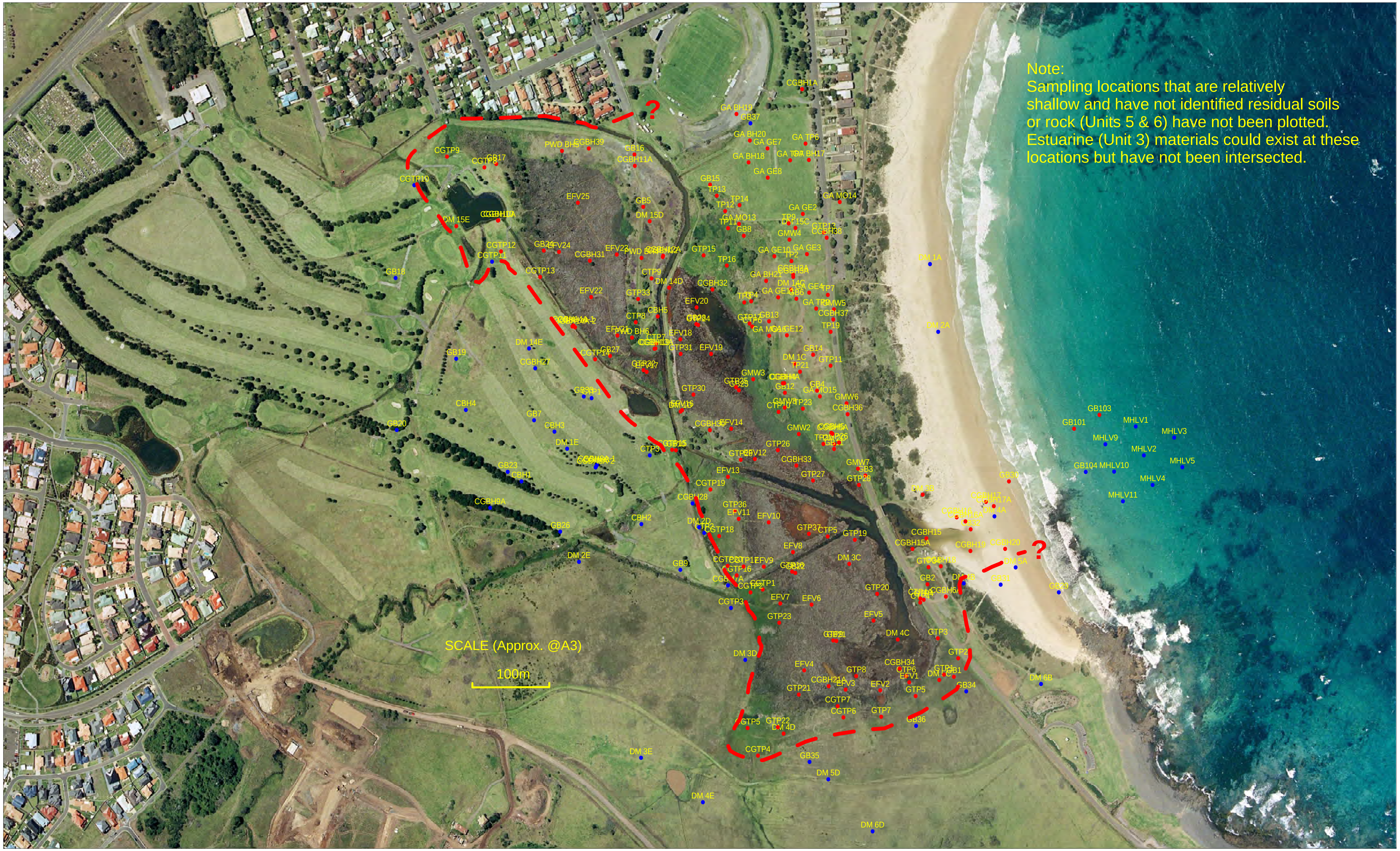
Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* 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.

Appendix A

List of Figures with Reference to Previous Coffey Reports

N



LEGEND:

Approximate Extent of ASS (based on available subsurface data, aerial photographs, survey maps, topographic maps, ASS Risk Map & site observations)

Test Location where Estuarine (Unit 3) Material has been Inferred to be Identified

Test Location where Estuarine (Unit 3) Material Not Likely to be Present

revision	description	drawn	approved	date

drawn	SD/CCQ
approved	SM
date	6/6/07
scale	AS SHOWN
original size	A3

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

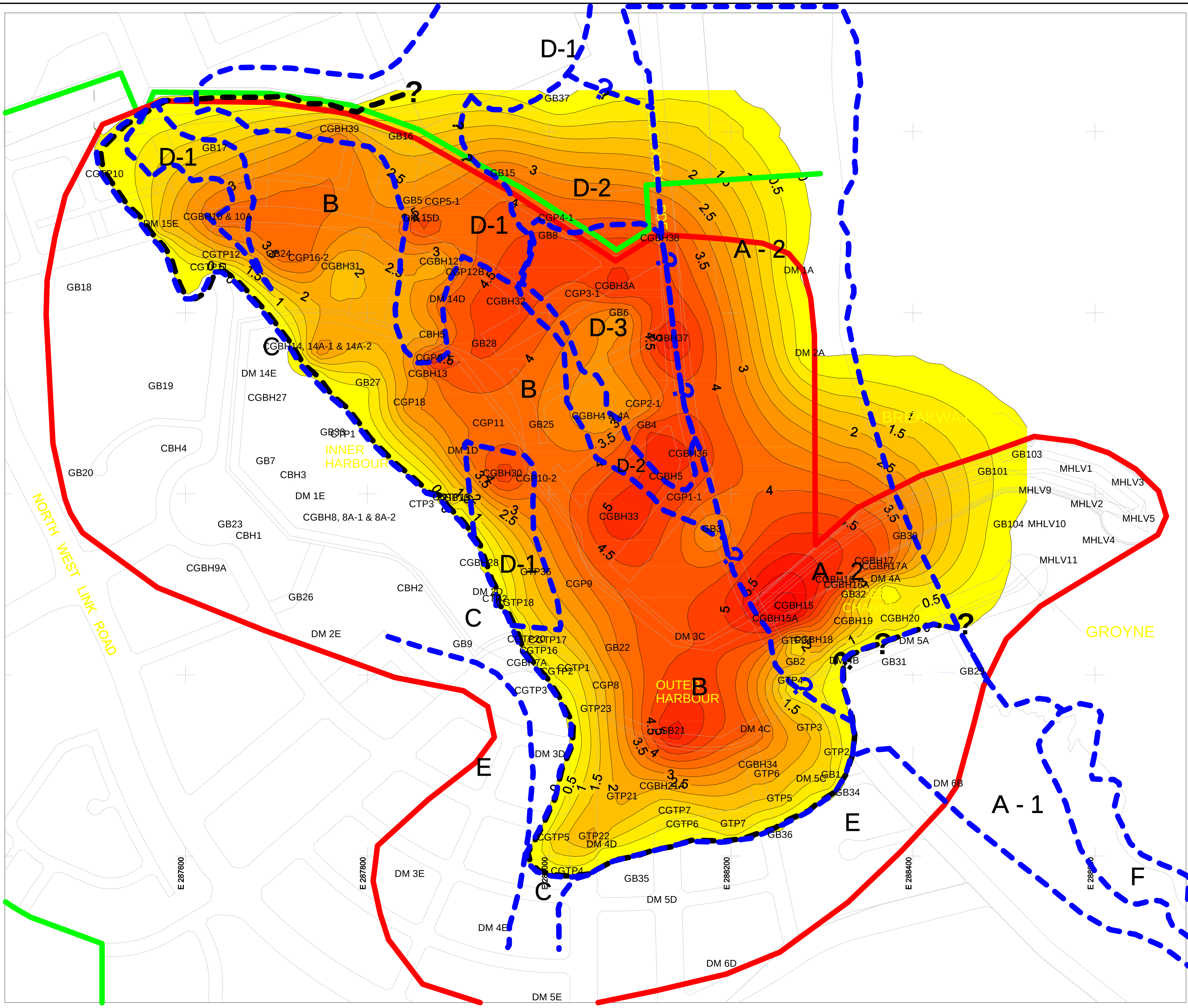
client: AUSTRALAND / PATTERSON BRITTON & PARTNERS

project: STAGE 3 GEOTECHNICAL INVESTIGATION WORKS SHELL COVE BOATHARBOUR, SHELLHARBOUR, NSW

title: INTERPRETED EXTENT OF ESTUARINE (UNIT 3) ASS MATERIAL

project no: GEOTUNAN02058AM-AN

figure no: 5



Approximate Thickness (m) of Estuarine (Unit 3B)

7, 6.5, 6, 5.5, 5, 4.5, 4, 3.5, 3, 2.5, 2, 1.5, 1, 0.5, 0

Terrain Units:

A-1 - LITTORAL SANDS - IMMEDIATELY UNDERLAIN BY UNIT 4 OR 5 SOILS
A-2 - LITTORAL SANDS - IMMEDIATELY UNDERLAIN BY UNIT 3 SOILS
B - ESTUARINE SEDIMENTS
C - ALLUVIUM
D-1 - "CLEAN" FILL
D-2 - REFUSE FILL: G.L <4m AHD
D-3 - REFUSE FILL: G.L >4m AHD
E - RESIDUAL SOIL / EXTREMELY WEATHERED ROCK
F - ROCK

LEGEND:

Approximate extent of Unit 3 (A&B)(ASS) (based on available subsurface data, aerial photographs, topographic map, ASS risk map and site observations)
Area of investigation works for Shell Cove Boatharbour
Shell Cove Site Boundary
CGP1 Test Location

Note: Contouring programme (Surfer) extrapolates contours beyond the approximate extent of ASS. This figure is a graphical representation only. The thicknesses shown have been inferred from current and previous borehole information.

revision	description	drawn	approved	date

0 100 200 300
Approximate Scale (metres)

drawn	SD
approved	SM
date	6/6/07
scale	1:5000 (Approx)
original size	A3

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

client: AUSTRALAND / PATTERSON BRITTON & PARTNERS
project: STAGE 3 GEOTECHNICAL INVESTIGATION WORKS SHELL COVE BOATHARBOUR, SHELLHARBOUR, NSW
title: INTERPRETED THICKNESS OF ESTUARINE UNIT 3 (A&B)
project no: GEOTUNAN02058AM-AN
figure no: 6A