DOLPHIN BLUE DEVELOPMENT
PHASE 1 GEOTECHNICAL ASSESSMENT

Jones Lang Lasalle Pty Ltd

GEOTCOFH01613AA-AF
18 October 2006

Coffey Geotechnics Pty Ltd ABN 93 056 929 483
1/18 Hurley Drive Coffs Harbour NSW 2450 Australia

18 October 2006

Jones Lang Lasalle Pty Ltd
Level 18, 400 George Street
Sydney NSW 2000

Attention: Vince Mulholland

Dear Sir,

RE: DOLPHIN BLUE DEVELOPMENT

PHASE 1 GEOTECHNICAL ASSESSMENT

Coffey Geotechnics Pty Ltd is pleased to present our report on the Phase 1 geotechnical assessment for the above site.

We draw your attention to the attached sheet entitled "Important Information About Your Coffey Report" which should be read in conjunction with this report.

We trust that this report meets with your requirements. If you require further information please contact the undersigned in our Coffs Harbour office.

For and on behalf of Coffey Geotechnics Pty Ltd

David Barker

Senior Geotechnical Engineer

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1 INTRODUCTION

Coffey Geotechnics Pty Ltd has conducted a Phase 1 geotechnical assessment for the proposed Dolphin Blue development to be located at the existing Blue Dolphin Caravan Park located on Yamba Road Yamba. The aims of the work, which was commissioned by Mr Jonathon Moss of Rider Hunt Terotech, were to provide an assessment and recommendations in relation to the geotechnical issues for the site. Specifically this report addresses the following issues:

- Foundation design parameters, including alternative footing types and founding levels and recommendations as to bearing pressures and data to assess settlements;
- Excavation support (shoring and batter slopes), including recommendations on retaining wall options and design parameters for the basement car park retaining walls;
- Design parameters for foundation tie down structures and/or uplift piles to resist buoyancy forces for structures under flood conditions;
- Discussion of issues relating to liners of water ponds;
- Groundwater aggressivity to buried structural elements (piles, anchors, basement and ramp structures);
- Site preparation and excavation conditions;
- Suitability of site soils for reuse as engineered fill on-site;

A dewatering management strategy for the site is presented in the Coffey report Ref: GEOTCOFH01613AA-AG dated 18 October 2006.

Coffey conducted the work in general accordance with proposal no. CH1613/1-AB. This report presents the results of the site investigation.

2 PROPOSED DEVELOPMENT

We understand that the proposed development will include construction of residential and resort style accommodation. We understand that the current design includes buildings up to about four stories in height with basements for car parking beneath. The proposed basement levels are between RL0.5mAHD and RL-0.5mAHD, or approximately 1m to 2m below the existing ground surface. The design of the underground car park parking areas and lower floor levels are to be designed as water tight basements areas to withstand a design peak flood level of 2.4mAHD. We understand that the access ramp areas for vehicles into basements will need to incorporate a form of “levee bank” to retain water from the underground basements.

A limited number of man made water features/ponds will also be constructed as part of the development. We understand that the depths of the water features are relatively shallow and that they will be lined with an essentially impermeable material. The water features also need to be designed to manage fluctuations in the ground water levels so that the liners of the water features are not affected by groundwater uplift pressures.

3 SCOPE OF WORK

3.1 Fieldwork

Fieldwork for the Phase 1 geotechnical assessment was carried out on 9 May 2006, and between 29 May 2006 and 1 June 2006. The field work involved the drilling of eight boreholes (BH1 to BH8). Five of the shallow boreholes were drilled using a small 4WD truck mounted drilling rig fitted with solid flight augers. Boreholes were drilled to depths of 6m with SPT testing undertaken at about 1.5m intervals in each borehole.

Three deeper boreholes were drilled by a 6 tonne truck mounted drill rig to depths of between 25.45m and 44.95m. The boreholes were initially drilled with continuous spiral flights augers and a V-bit then progressed further by wash boring techniques. Standard Penetration Tests (SPTs) were carried out in the borehole at nominal 1.5m intervals and more widely spaced within the deeper section of the boreholes. Environmental and geotechnical samples were taken from the boreholes for subsequent laboratory testing.

Fieldwork was conducted by a Scientist from Coffey who located the boreholes, took samples and recorded results of in-situ testing, and produced engineering logs of the interpreted subsurface conditions. Figure 1 shows a locality plan for the site, and Figure 2 shows the investigation locations. Engineering logs of the boreholes are presented in Appendix A, along with explanation sheets defining the terms and symbols used in their preparation.

3.2 Laboratory Testing

Laboratory testing was conducted on samples recovered during fieldwork. Laboratory testing comprised the following:

- 5 Particle Size Distribution tests to assess materials classifications; and
- 2 Groundwater Aggressivity tests.

Atterbergs limits tests were proposed as part of the scope of work but were not carried out due to the granular nature of the soils within the subsurface profile.

The results of the laboratory testing are summarised in Table 1 and Table 2. Laboratory test reports are presented in Appendix B.

Table 1 - Results of Particle Size Distribution Testing

Borehole	Sample Depth (m)	Percentage Passing Sieve Sizes (mm)			
		2.36	**0.60**	**0.15**	**0.075**
BH1	1.5-1.95	100	98	5	1
BH2	2.9-3.35	100	98	3	0
BH3	1.4-1.95	100	100	3	0
BH4	1.4-1.85	100	100	4	1
BH5	2.9-3.35	96	93	5	2

Table 2 - Results of Aggressivity Testing

Borehole	Sample Depth (m)	Soluble Sulphate (mg/kg)	Soil Resistivity (ohm/mm)	Chloride (mg/kg)	pH
BH6	5	0.002	153,846	0.003	7.2
BH8	11.5-11.65	0.002	322,581	0.001	6.34

4 SITE CONDITIONS

4.1 Existing Surface Conditions

The site of about 5.2Ha is situated at lot 1 and 2 in DP706628 Yamba Road, Yamba. The site is situated on the north-eastern side of Yamba Road and is bounded by low rise detached dwellings to the south-west, the Clarence River to the north-east, the 'Moby Dick' motel to the south-east and the Clarence Valley Nature Reserve to the north-west.

Regionally the site is situated within a flat alluvial floodplain area of the Clarence River. Locally, the ground surface is generally flat. Some low rise, man made landscaping soil mounds are located on the site. Existing developments at the existing caravan park comprise numerous demountable cabins, brick amenities buildings and several in ground swimming pools. Vegetation on the site consists mainly of mowed lawns and scattered trees.

4.2 Local Geology

The Maclean 1:250,000 Geological Series Sheet produced by the Geological Survey of NSW indicates that the site is underlain by Quaternary aged sediments.

This area is located near the river mouth of the Clarence River which comprises deep sedimentary deposits of sands, silts and clays. Based on our previous experience in the low lying areas of the township of Yamba, a shallow band of indurated (or weakly cemented) sand has been encountered at a number of the sites investigated. These indurated sands are commonly referred to as 'coffee rock'.

A search of DLWC groundwater bore data in the area shows that the area comprises mainly sand with some clay lenses, from previous experience to the west of the site along Yamba Road, shallow deposits of soft clays have been encountered.

4.3 Subsurface Conditions

The subsurface conditions interpreted from the boreholes (BH1 to BH8) are summarised in Table 3. Based on limited number of locations investigated, the subsurface conditions across the site appear to be relatively uniform. The depths indicated have been measured from the ground surface levels during the investigation.

Table 3 - General Summary of Subsurface Profile at Locations Investigated

Unit	Material Description	Depth to Base of Unit (m)							
		BH1	BH2	BH3	BH4	BH5	BH6	BH7	BH8
1	**Topsoil/Fill:** Silty Sand, fine to medium grained, dark brown.	0.05	0.05	0.05	0.05	0.05	0.5	0.05	0.05
2a	**Marine Soil**: Sand and Silty Sand, fine to medium grained, loose to medium dense, grey.	2.5	4.0	2.5	4.0	4.0	1.5*	2.6	4
2b	**Marine Soil (Interbedded Dense to Very Dense Sands and Indurated Sands):** Sand and Silty Sand, medium dense to dense, fine to medium grained, grey and dark brown, low plasticity fines.	≥6.0	≥6.0	≥6.0	≥6.0	≥6.0	21.5	25.45	≥25.45
2c	**Marine Soil (Medium Dense to Very Dense Sands):** Sand and Silty Sand, fine to medium grained, grey and brown, low plasticity fines.	-	-	-	-	-	≥25.5	38.5	-
3a	**Marine Soil (Firm to Loose Sandy Clay and Clayey Sand):** Sandy Clay/Clayey, Sand, fine to medium grained sand, medium plasticity clay, grey, loose sand, estimated firm clays.	-	-	-	-	-	-	41.0	-
3b	**Marine Soil (Stiff to Very Stiff Clays):** Silty Clay, firm to very stiff, grey, medium to high plasticity fines, shell fragments.	-	-	-	-	-	-	≥44.95	-

*Note: Possibly deeper than indicated due to gravel/cobble affecting in-situ testing description.

Further description of the subsurface materials intersected by the boreholes is given on the engineering logs presented in Appendix A. Locations of the boreholes BH1 to BH8 are presented on Figure 2.

4.4 Groundwater Depths

A summary of the depths of standpipe piezometers installed at the site is presented in Table 4. Table indicates the depth of the boreholes, the depth of the piezometers and the standing groundwater levels as measured on 14 June 2006.

Table 4 - Borehole and Piezometer Summary

Borehole Number	Depth Drilled (m)	Ground Level (m, AHD)	Depth Of Piezometer (m, below ground level)	Groundwater Depth (m, below ground level) 14/6/06	Approximate Groundwater Level (m, AHD)
BH1	6	1.29	2.2	1.0	0.29
BH2	6	1.56	2.35	1.05	0.51
BH3	6	1.47	2.3	0.85	0.62
BH4	6	1.82	2.45	1.37	0.45
BH5	6	1.8	2.4	1.4	0.4
BH6	25.45	1.75	6.0	1.47	0.28
BH7	44.95	1.56	6.0	0.95	0.61
BH8	25.45	1.61	6.0	1.09	0.52

The levels of the groundwater indicated in Table 4 indicate the groundwater flows in a north-east direction towards the Clarence River.

5 CONSIDERATIONS FOR DESIGN & CONSTRUCTION

5.1 Geotechnical Model

Based on the results of the subsurface investigations, the following preliminary geotechnical model has been developed in Table 5 below.

Table 5 - Preliminary Geotechnical Model for Foundations

Unit	Material Description	Depth To Base Of Unit (m)	Level Of Base Of Unit (m, AHD)
1	**Topsoil:** Silty Sand, fine to medium grained, dark brown.	0.05	-
2a	**Marine Soil:** Sand and Silty Sand, fine grained, medium dense to loose, grey.	2.5 to 4.0	-1.0 to -2.4
2b	**Marine Soil (Interbedded Dense to Very Dense Sands and Indurated Sands):** Sand and Silty Sand, medium dense to dense, fine grained, grey and dark brown, low plasticity fines.	21.5 to 25.5	-19.75 to -23.9
2c	**Marine Soil (Medium Dense to Dense Sands):** Sand and Silty Sand, fine to medium grained, grey and brown, low plasticity fines.	38.5	-36.9
3a	**Marine Soil (Loose to Firm Sandy Clay and Clayey Sand):** Sandy Clay/Clayey, Sand, fine to medium grained sand, medium plasticity clay, grey, loose sand, estimated firm clays.	41.0	-39.5
3b	**Marine Soil (Firm to Stiff Clays):** Silty Clay, firm to very stiff, grey, medium to high plasticity fines, shell fragments	≥44.95	≥-43.4

5.2 Founding Conditions & Parameters

Depending on the final design of the development, options considered appropriate for the foundations for the proposed structures forming the development at the site include:

- Shallow footings founded within the Unit 2a loose to medium dense Sand and Silty Sand materials which are encountered up to about 4m depth;

- Pile foundations within the Unit 2a, Unit 2b and/or Unit 2c sands;
- A combination of shallow footings and pile foundations (e.g. piled raft).

The decision as to the adopted foundation system will depend on the applied loads and the settlement that can be tolerated by each structure. It is possible that both options indicated above may be adopted at different locations and for different structures at the site. Preliminary design parameters for the relevant geotechnical units are provided in the following sections.

5.3 Shallow Footings

For shallow footings founded at the base of the excavations, the allowable bearing pressure may be assessed using the equation below, depending on the minimum dimension of the footing:

$$ABP = 50B + 80kPa \leq 250kPa$$

where ABP = allowable bearing pressure

B = minimum footing dimension

The values of Young's Modulus indicated in Table 6 below should be adopted in the assessment of settlements under shallow footings.

Table 6 - Preliminary Young's Modulus Values for Shallow Footings

Unit	Young's Modulus, E (MPa)
2a	20
2b	50
2c	40

5.4 Pile Foundations

Where footing loads are higher than can be supported by shallow footings, or settlements under shallow footings are excessive, pile foundations could be adopted.

A number of pile types could be utilised for the proposed structures depending on constraints such as allowable vibration levels during construction and Council approvals. Each of these piles may be designed to found in the alluvial sand units. Piling types considered suitable for the site are shown in Table 7 together with their advantages and disadvantages.

Table 7: Advantages and Disadvantages of Alternate Pile Types

Pile Type	**Advantages**	**Disadvantages**
Atlas Screw / Omega type piles. *Displacement type*	Low vibration and noise on installation. High level of performance confidence because of torque measurements on installation. Relatively high load carrying capacity due to displacement of materials.	May be more expensive than other options. Unlikely that a local supplier can conduct the installation.
Driven Pre-Cast Concrete Piles. *Displacement type*	High level of performance confidence. Relatively high load carrying capacity due to displacement type. No spoil removal issues	Relatively high noise and vibration during installation may affect nearby developments Would require dilapidation studies for adjacent structures and vibration monitoring during construction. Splice joints will be required due to depth if end-bearing, increasing cost. Unlikely that a local supplier can conduct the installation. Driving refusal may occur prior to design depth, affecting uplift capacity, which may require heavier driving.
Grout Injected/Continuous Flight Auger (CFA) Piles *Non-displacement type*	Low vibration and noise on installation.	Unlikely that a local supplier can conduct the installation. Additional cost of spoil removal. Typically lower load carrying capacity compared to displacement piles, due to mode of installation and lower confidence in pile performance. Some uncertainty about integrity due to installation process.

It is considered that open bored piles would not be practical at the site as pile hole support and dewatering of the base of the piles would be needed prior to pouring concrete.

Design parameters for the above pile types founded at least 4 pile diameters below the excavation level are shown in Table 8. The design should include assessment of both strength and serviceability limit states. At this stage a geotechnical reduction factor of Φ_g = 0.5 should be applied to the ultimate values indicated in Table 8 when considering the ultimate strength limit state. It is likely that once further investigations are carried out, provided those investigations indicate uniformity of geotechnical conditions at the site, a higher reduction factor may apply. Where driven piles are adopted, should a suitable proportion be dynamically tested, a higher geotechnical strength reduction factor of $\Phi_g = 0.65$ may be adopted.

The geotechnical parameters for pile design are given below.

Table 8 - Preliminary Geotechnical Strength Parameters for Pile Design

Founding Material	**Pile Type**	**Ultimate End Bearing Capacity (MPa)**	**Ultimate Shaft Adhesion (kPa)**
Unit 2a – Medium Dense to Loose Sand and Silty Sand	Displacement	1.8	40
	Non-displacement	0.9	25
Unit 2b – Dense to Very Dense Sand and Silty Sand	Displacement	10.0	150
	Non-displacement	5.0	75
Unit 2c – Medium Dense to Dense Sand and Silty Sand	Displacement	6.0	100
	Non-displacement	4.0	50

For piles designed as tension piles (e.g. to withstand basement uplift forces) the above shaft adhesion parameters should be reduced by 50% for the assessment of ultimate tension capacity. When designing piles for tension, reference must also be made to the discussion relating to other failure methods requiring consideration indicated in Section 5.4.

Table 9 - Preliminary Young's Modulus Parameters For Pile Serviceability Assessment

Founding Material & Expected Depth Range*	**Pile Type**	**Young' Modulus, E (MPa)**
Unit 2a – Medium Dense to Loose Sand and Silty Sand	Displacement	35
	Non-displacement	20
Unit 2b – Dense to Very Dense Sand and Silty Sand	Displacement	85
	Non-displacement	70
Unit 2c – Medium Dense to Dense Sand and Silty Sand	Displacement	60
	Non-displacement	45

5.5 Resistance of Basements to Uplift Pressures

Basement carparks beneath the existing structures will need to be designed to withstand hydrostatic pressures on the base of the concrete slabs during times of high groundwater and/or flood conditions. It is understood that the basement carparking areas will be designed as fully tanked structures, with a tanked access ramp that extends over a local levee system providing access. The levee system will be higher than the design flood level for the site.

The basement structures will need to be effectively 'tied down' such that uplift cannot occur. This could be achieved by using the dead weight of the structure and/or pile foundations capable of carrying tension loads. Ground anchors may be another possibility to provide tie down, but due to their smaller diameter would likely need to be much deeper that pile foundations to carry the same tensile load.

The design of tension piles will require consideration of two potential failure mechanisms. These include a failure by slip along the shaft of the pile ('shaft adhesion failure') and a 'cone pullout failure' mechanism. The 'cone pullout failure' mechanism is essentially the weight of the soil within conical shape extending up from the toe of the pile that would have to be moved in the event of the pile failing with no slippage along the pile shaft. The adopted geotechnical pile tension capacity would be the lower of the capacity assessed using these methodologies.

The first mechanism would be assessed assuming the preliminary design parameters indicated for tension piles in Section 5.4. The assessment of the potential failure second mechanism would depend on size and spacing of the piles, and the angle of the 'cone' up from the base of the pile. We recommend that the assessment of the 'cone pullout failure' mechanism be carried out in accordance with the information indicated below:

- An angle of the cone relative to the axis of the pile of 30°;
- Groundwater at the surface for the assessment of the effective unit weight of the soil;

- Where adjacent or nearby cones from different piles overlap, only the ‘non-overlapping’ volume only of soil may be considered;

Our preliminary calculations indicate that depending on the spacing and depth of the piles, the shaft adhesion calculation may be the critical condition when considering the tensile capacity of the piles. However, as this may change depending on the factors involved, both mechanisms should be checked.

5.6 Aggressivity to Buried Structural Elements

The results of the laboratory testing of samples taken during the investigation for aggressivity to buried structural elements are presented in Table 2, Section 3.2.

From results of the testing, the classification for concrete piles is assessed as ‘Mild’ in accordance with Table 6.1 in AS2159-1995 Piling - Design & Installation. For steel piles, the exposure classification is assessed as “Non Aggressive” in accordance with Table 6.3 in AS2159-1995 Piling - Design & Installation.

However based on the subsurface conditions at the site, and the close proximity of the site to the Clarence River it would be expected that the marine sand units could be affected by saline water. On this basis we suggest that the concrete and steel piles be designed for a minimum of a “moderate” exposure classification. Details of the design for cover to reinforcement and corrosion allowances for the various exposure classifications are presented in Table 10 below.

Table 10 - Summary for Design of Piles for Aggressivity

Exposure Classification	**Concrete Minimum Cover to Reinforcement (mm)**		**Steel Uniform Corrosion Allowance (mm/year)**
	Pre Cast	**Cast In Place**	
Non Aggressive	20	40	<0.01
Mild	20	50	0.01-0.02
Moderate	25*	55*	0.02-0.04*

Note: Values with (*) indicate the recommended exposure classifications to be designed for concrete and steel piles.

5.7 Excavation Conditions

Excavations to depths of up to 2m to 3m are proposed for the basement car parking facilities. It is expected that soil profiles encountered at the site could be readily excavated by conventional earthmoving equipment such as backhoes and excavators.

As noted above, groundwater was encountered between 0.8m and 1.0m depth below the ground surface. As the site soils within the depth of the proposed excavation and to a depth of about 45m below the ground surface comprised mainly sand and silty sand, groundwater inflows into excavations could be significant if adequate measures are not taken to address the issue.

A copy of the dewatering management statement discussing dewatering options is presented in Coffey report GEOTCOFH01613AA-AG dated 18 October 2006.

5.8 Site and Excavation Retention

As indicated in the dewatering management statement, the current philosophy for dewatering of the site does not rely on use of groundwater cut-off measures such as sheet pile walls installed to particular depths, but rather involves the use of local groundwater drawdown measures, such as spear points or the like. With this in mind, the consideration of excavation support could involve two general philosophies, namely the construction of batter slopes, or the use of excavation support in the form of vertical retaining systems around the perimeter of the investigations.

Where temporary batter slopes area adopted, a maximum slope angle of 2H:1V should apply. Batter slopes may apply where there is space around the excavation.

Where there is insufficient space for temporary batter slopes, or where vertical cuts are required, retaining walls will need to be constructed. In those instances, retaining walls will need to be constructed prior to excavation and could comprise:

- Drive sheet piles; or
- Contiguous pile or secant pile walls;

For an excavation depth of the order indicated for the development, it is considered that the wall options indicated above could be designed as cantilevered walls, or incorporate ground anchors for restraint (where necessary) and where space permits. The depth of embedment for retaining walls will require analysis and design, but may be of the order of 3m to 4m below the base of the excavation.

The depth of embedment for sheet piles will limited due to the presence of indurated sand at the site. Based on our experience, we consider that sheet piles may be able to be installed about 2m into the Unit 2b dense to very dense Sands (i.e. a depth of 6m from the existing ground surface) assuming that a large hammer is used to drive the sheet piles.

Pile design for retaining structures will require an assessment of the induced bending moments and shear forces within the retaining wall to enable design of reinforcement for the piles. Where walls are designed to be low permeability cut off walls that will be cast internally into the tanked basement structure, they should be designed to withstand the full hydrostatic pressure under high water level conditions.

Cantilevered walls or walls with one row of anchors may be designed on the basis of a triangular stress distribution using the design parameters shown in Table 11. Preliminary ‘At rest’, active and passive earth pressure coefficients for each of the geotechnical units are also provided as requested.

Table 11 - Preliminary Retention Design Parameters

Geotechnical Unit	**Unit 2a – Medium Dense To Loose Sand**	**Unit 2b – Dense To Very Dense Sand**
Assessed Depth Range (m)	0.4-4	4-22
Unit Weight, γ (kN/m^3)	17	18
Effective Friction Angle, Ø'	30	37
Effective Cohesion, c' (kPa)	0	0
'At rest' Earth Pressure Coefficient (K_0)	0.5	0.4
Active Earth Pressure Coefficient (K_a)	0.33	0.25
Passive Earth Pressure Coefficient (K_p)	3.0	4.0

If walls with more than one row of anchors are to be adopted a different stress distribution would apply. Coffey can provide this information and details for anchor design if necessary.

The values of earth pressure coefficient above assume that the area at the crest of the retaining wall is level. Design of the walls must take into account any surcharge from sloping ground or other loadings (such as from proposed or existing buildings) behind the wall.

With respect to excavation induced movements, the magnitude and extent of settlement will be dependant on the lateral deflection of the retaining structures and achieving little or no groundwater drawdown. Depending on the excavation sequence and active pressures, lateral displacements may be of the order of 0.3% to 0.7% of the retained wall height. The effect of this movement would generally extend to a distance represented by a line projected at about 45^{O} to 60^{O} up from the toe of the excavation. Where existing buildings or other movement in tolerant structures are located near the crest of the excavation, "at rest" earth pressure parameters should be used.

5.9 Preliminary Design of Anchor Support

Temporary ground anchors should be specified based on their required performance. Capacities for non pressure grouted anchors are likely to be relatively low in the marine soil. Significantly higher capacities should be able to achieve if pressure grouting is carried out. For the preliminary design of anchors, the bond stress for a non pressure grouted anchor can be estimated from the following equation:

$$\text{Ultimate Bond Stress} = \sigma'_0 \times \tan\phi \text{ (kPa)}$$

where:

- σ'_0 = the effective overburden pressure at a depth (D) from ground surface at the top of the retaining wall to the mid point of the bond length of the anchor. If the groundwater table is at about 1m depth σ'_0 will be equal to 10+10D where D is in metres.
- ϕ = effective friction angle for the geotechnical unit within which the anchor bond length is embedded and can be obtained from Table 11.

For pressure grouted, anchors the bond stress can be estimated from the following equation:

$$\text{Ultimate Bond Stress} = 1/3\ P_i \times \tan\phi \text{ (kPa)}$$

where:

- P_i = the grout injection pressure in kPa

The bond stresses calculated using the above equation is applicable to anchors with a bond length within the range 3m to 7m. Bond lengths should not lie within a zone bounded by line drawn at a 45^o angle upwards from the based of the excavation. Temporary anchors should be tested to 1.3 times their working load.

For the design and installation of ground anchors specialist contractors should be engaged to design and install anchors based on a performance specification.

5.10 Discussion of Issues Relating to Liners for Ponds

The development will incorporate a limited number of man made ponds and water features. Due to the relatively high permeability of the underlying soils, these ponds will need to be lined to enable water retention. However, in making the ponds watertight, unless mitigation measures are adopted in the design, the underside of the liner will be subjected to hydrostatic water (uplift) pressures in the event of groundwater level rise. This could result in the pond liners lifting, and potentially being damaged, depending on the materials used in their construction.

Options for the liners could include reinforced concrete, clay, or a form of synthetic low permeability barrier such as a HDPE liner.

Options for reducing the risk of hydrostatic uplift and potential damage to the liner include structural solutions such as tie down, or other options such as the incorporation of one-way valves that allow water to flow to the top of the liner as groundwater rises (therefore equilibrating the pressure top and bottom), but not to flow back once the groundwater level falls. The structural solutions are only applicable when the liner itself has a structural strength (e.g. reinforced concrete), and would likely be relatively expensive compared to other options. The one-way valve option could reasonably be adopted with most liner systems, and is therefore likely to be a reasonable option for the site. Consideration will need to be given to the flow capacity of the valves such that water can flow through at a fast enough rate to ensure no excess water pressure build-up on the underside of the liner and the connection of the valves to the liner such that the valve / liner connection is suitable for the life of the development.

5.11 Reuse of Geotechnical Materials

Based on the subsurface materials encountered during the investigation is considered that the materials won from the excavations may be suitable for reuse as engineered fill. However materials won from excavations should be observed by a suitable qualified geotechnical practitioner prior to use to give approval to the use of those materials. Furthermore, all materials won from excavations on site should be subject to testing/contamination in relation to acid sulphate soils in accordance with the acid sulphate soil management plan for the site.

5.12 Site Preparation

5.12.1 Base of Excavation

Based on the assumption that the excavations will extend to a depth of about 2m to 3m, the materials at the base of the excavation will comprise wet loose to medium dense sands which are expected to provide poor trafficability. To improve trafficability, a granular working platform at least 300mm thick (and possibly greater depending on the conditions evident) may be required at the base of the excavation. The need for such a platform will be dependant on a number of factors including the dewatering level, the nature of the exposed materials and the type of plant used in the excavation base.

5.12.2 General

The following general comments and recommendations are provided for site preparation beneath structures:

- Following excavation to design level, the exposed natural in-situ materials should be proof rolled to identify any wet, excessively deflecting or other deleterious material. Any such areas should be over-excavated and backfilled with a clean select material. All topsoil should be stripped and stockpiled for re-use as landscaping materials only.
- Approved fill beneath roads should be placed in layers not exceeding 300mm loose thickness and be compacted to a minimum density index of 65%.
- The top 300mm of natural subgrade or subgrade fill below pavements should be compacted to a minimum density index of 85%.
- Approved fill beneath structures should be placed in layers not exceeding 300mm loose thickness and be compacted to a minimum density index of 70%. All filling beneath structures should be carried out under Level 1 construction monitoring and testing as defined in AS3798-1996.
- Earthworks should be carried out in accordance with the recommendations outlined in AS3798-1996, '*Guidelines for Earthworks for Commercial and Residential Developments*'.

6 PHASE 1 ACID SULPHATE SOILS ASSESSMENT

6.1 Formation of Acid Sulfate Soils

Acid Sulphate Soils (ASS) are soils which contain significant concentrations of pyrite which, when exposed to oxygen, in the presence of sufficient moisture, oxidises, resulting in the generation of sulphuric acid. Unoxidised pyritic soils are referred to as potential ASS (PASS). When the soils are exposed, the oxidation of pyrite occurs and sulphuric acids are generated, the soils are said to be actual ASS (AASS).

Pyritic soils typically form in waterlogged, saline sediments rich in iron and sulphate. Typical environments for the formation of these soils include tidal flats, salt marshes and mangrove swamps below about RL 5m AHD. They can also form as bottom sediments in coastal rives and creeks.

Pyritic soils of concern on low lying NSW and coastal lands have mostly formed in the Holocene period, (i.e. 10,000 years ago to present day) predominantly in the 7,000 years since the last rise in sea level. It is generally considered that pyritic soils which formed prior to the Holocene period would already have oxidised and leached during periods of low sea level which occurred during ice ages, exposing pyritic coastal sediments to oxygen.

Disturbance or poorly managed development and use of acid sulphate soils can generate significant amounts of sulphuric acid, which can lower soil and water pH to extreme levels (generally <4) and produce acid and salts, resulting in high salinity.

The low pH, high salinity soils can reduce or altogether preclude vegetation growth and can produce aggressive soil conditions which may be detrimental to concrete and steel components of structures, foundations, pipelines and other engineering works.

Generation of the acid conditions often releases aluminium, iron and other naturally occurring elements from the otherwise stable soil matrices. High concentrations of such elements, coupled with low pH and alterations to salinity can be detrimental to aquatic life. In severe cases, affected waters flowing off-site can have detrimental effect on aquatic ecosystems.

6.2 Acid Sulphate Soils Risk Map

The Department of Land and Water Conservation 1:25,000 Acid Sulphate Soil Risk Map of Yamba shows that the site is located in an area of disturbed terrain and indicates that investigations for acid sulphate soils are required in these areas.

6.3 Laboratory Testing

Samples collected during fieldwork were placed in tightly sealed plastic bags and stored in chilled insulated containers during transit to cold storage at Coffey's Coffs Harbour laboratory.

Samples obtained for the acid sulphate assessment were sent to an external NATA registered laboratory and screened for the presence of potential ASS using laboratory methods 21Af and 21Bf of Ahern CR, Blunden B and Stone Y (eds) (1998), Acid Sulphate Soil Laboratory Methods Guidelines, ASSMAC. The results of the acid sulphate soil screening tests are summarised in Table 12.

Table 12 - Summary of Acid Sulphate Soil Screening Tests

Borehole & Depth (m)	**pH in water**	**pH in H_2O_2**	**Borehole & Depth (m)**	**pH in water**	**pH in H_2O_2**
BH6 0.5	5.38	3.52	BH7 4.0	8.72	6.14
BH6 1.0	5.24	4.14	BH7 5.0	6.78	5.22
BH6 1.5	6.11	4.09	BH7 5.5-5.7	6.57	4.74
BH6 2.5	8.59	6.14	BH7 7.0-7.3	6.52	4.26
BH6 3.5	8.90	6.02	BH7 8.5-8.8	6.62	4.04
BH6 4.0	8.7	6.33	BH8 0.5	7.5	6.17
BH6 4.5	8.37	6.10	BH8 1.0	7.16	2.96
BH6 5.5-5.8	7.44	4.75	BH8 1.5	8.27	5.9
BH6 7.0-7.15	6.42	4.5	BH8 2.5	8.48	6.49
BH6 10.0-10.15	6.09	3.13	BH8 3.5	8.2	6.21
BH7 1.0	6.99	4.84	BH8 4.0	8.53	6.52
BH7 1.5	8.05	6.42	BH8 4.5	8.62	6.27
BH7 2.0	7.73	2.96	BH8 5.5-5.7	6.17	4.51
BH7 3.0	8.6	6.64	BH8 7.0 -7.3	6.11	4.34
BH7 3.5	9.04	6.48	BH8 11.5-11.65	6.55	3.79

The following points are noted from Table 12:

- Soil in water produced pH<4 for none of the samples tested. Soil:water pH<4 in this test is an indication of actual acid sulphate soil,
- Oxidation with hydrogen peroxide produced pH<3 in five of the 30 samples tested. Soil:peroxide pH<3 in this test is an indication of potential acid sulphate soil.

On the basis of the screening results, 14 samples were selected for additional testing to assess the potential for acid generation. The results of this testing are presented in Appendix B and are summarised in Table 13.

Table 13 - Summary of CRS Testing

Borehole & Depth (m)	**Texture**	**Reduced Inorganic Sulfur (%Scr)**	**Action Criteria For %Scr**	**Total Potential Acidity (TPA) moleH^{+}/tonne**	**Action Criteria For TPA**
BH6 1.0	Coarse	0.005	0.03	-	18
BH6 1.5	Coarse	<0.005	0.03	-	18
BH6 3.5	Coarse	**0.03**	**0.03**	0	18
BH6 4.5	Coarse	**0.031**	**0.03**	0	18
BH6 5.5-5.8	Coarse	0.011	0.03	-	18
BH6 10.0-10.15	Coarse	0.03	0.03	**45**	**18**
BH7 1.0	Coarse	<0.005	0.03	-	18
BH7 1.5	Coarse	**0.056**	**0.03**	0	18
BH7 2.0	Coarse	**0.052**	**0.03**	5	18
BH7 5.5-5.7	Coarse	0.005	0.03	-	18
BH7 8.5-8.8	Coarse	<0.005	0.03	-	18
BH8 0.5	Coarse	0.05	0.03	-	18
BH8 1.0	Coarse	**0.248**	**0.03**	**20**	**18**
BH8 11.5-11.65	Coarse	<0.005	0.03	-	18

Note: Values in bold and underlined exceed action criteria;
Action criteria adopted are based on disturbance of more than 1000 tonnes of acid sulphate soils.

6.4 Discussion and Recommendations

Based on the results of the laboratory testing, samples from the Unit 2a and 2b marine soils exceeded the action criteria for both $\%S_{CR}$ and TPA. In accordance with the ASSMAC (1998) Acid Sulphate Soil Manual, an acid sulphate soils management plan is required for the development where soils exceed the action criteria. A preliminary acid sulphate soil management plan is presented in Appendix F.

Good quality fine agricultural lime should be used to treat excavated PASS. It should be noted that liming is only one of a number of techniques to lower the risk posed by PASS. Other options include avoidance of disturbing PASS and placing the soil below the water table level immediately following excavation to prevent oxidation from occurring. The final option chosen could be a combination of techniques based on the likely construction scenario and the volumes of ASS requiring management.

In calculating the liming ratios, a factor of safety of 1.5 has been allowed (as recommended in the ASSMAC guidelines) above the theoretical requirement to take into account the rate of lime reactivity and the possibility of inhomogeneous mixing.

Results of the CRS and TPA testing show that there are zones of potential acid sulphated soils at the site and that the TPA analysis shows that there is a significant buffering affect within the soil which reduces the acid generation during oxidation. Most of the samples of the Unit 2a and 2b were assessed to not require liming to neutralise their acid generating potential. Several of the samples were assessed to require liming rates between 4kg per cubic metre and 6kg per cubic metre. Based on the results of the laboratory testing, we consider that a preliminary treatment rate of about 4kg of lime per cubic metre would be appropriate.

Additional investigation and assessment of acid sulphate soils is recommended prior to development. Additional investigation might include sampling of soils at 0.5m intervals to at least 1m below the base of any disturbance or groundwater drawdown level in about seven to eight locations, laboratory testing and analysis of results.

7 CONCLUSION & RECOMMENDATIONS FOR FURTHER WORKS

Based on the data presented in this report, we consider that the site is suitable for the proposed development with respect to the geotechnical conditions.

Further investigation works will be required for the development during the design and construction phase of the development. Further work will likely include:

- A detailed geotechnical investigation of the subsurface conditions across the site including shallow and deep boreholes to further assess the geotechnical model.
- Assessment of design parameters for road pavements.
- Level 1, monitoring and testing during site earthworks.
- Detailed acid sulphate soils investigation and assessment.
- Further work for the dewatering/groundwater control issues (these are provided in Coffey report GEOTCOFH01613AA-AG).

We consider that the further investigation works recommended above could be carried out and the matters appropriately resolved during detailed design, pursuant to a Part 4A Construction Certificate for the development.

For and on behalf of Coffey Geotechnics Pty Ltd

David Barker

Senior Geotechnical Engineer

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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.

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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

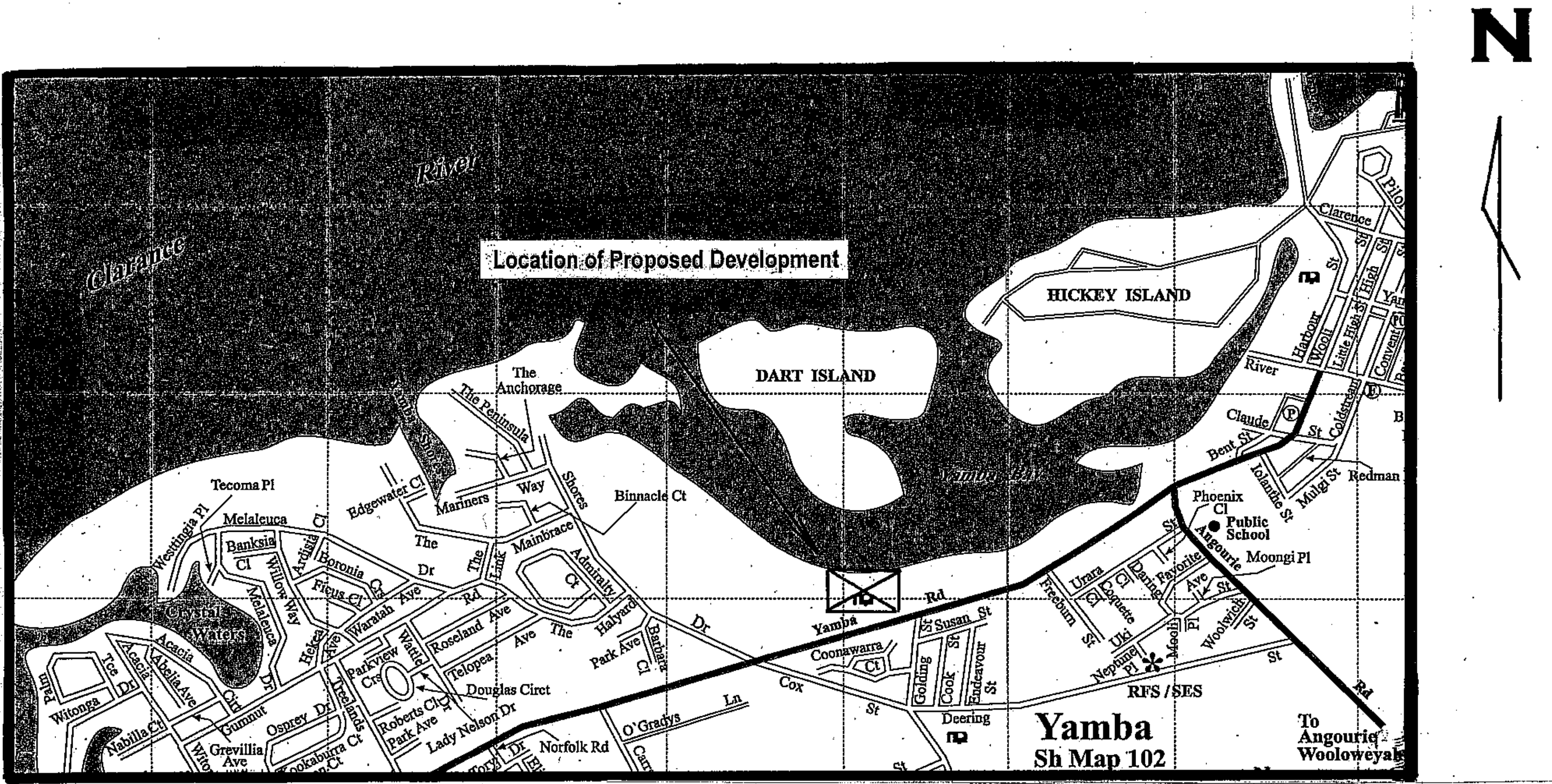
Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As 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

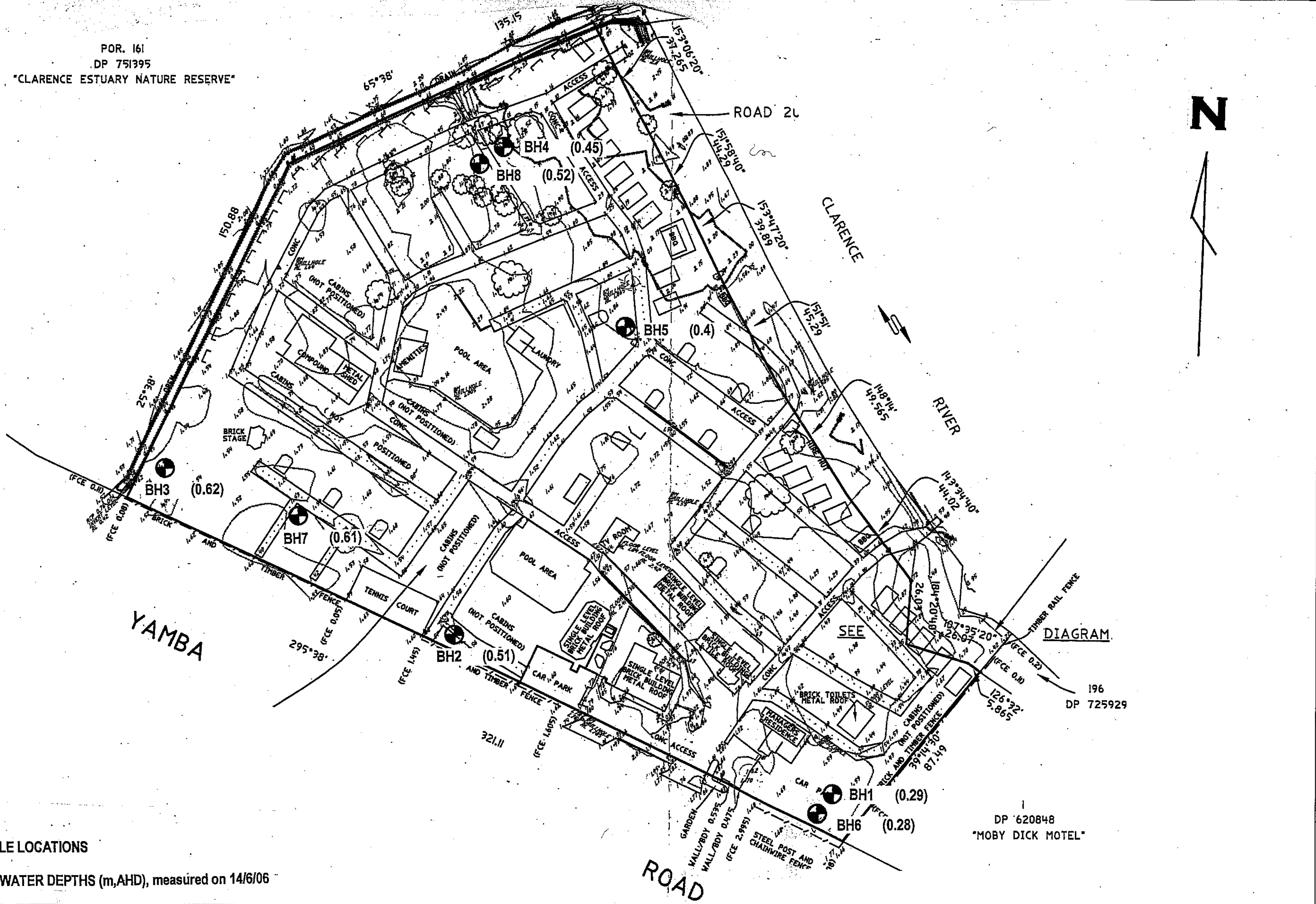
Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, 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.

Figures



drawn:	DJB	coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	JONES LANG LASALLE PTY LTD
approved:			project:	DOLPHIN BLUE DEVELOPMENT YAMBA ROAD, YAMBE
date:	18/10/06			
scale:	NTS		title:	SITE LOCATION PLAN
original size:	A4		project no.: GEOTCOFH01613AA-AG	figure no.: FIGURE 1



LEGEND

BH1 BOREHOLE LOCATIONS

(0.29) GROUNDWATER DEPTHS (m,AHD), measured on 14/6/06

revision	description	drawn	approved	date

0 15 30 60 90

Scale (metres)

drawn	DJB
approved	
date	18/10/06
scale	1:1,500
original size	A3

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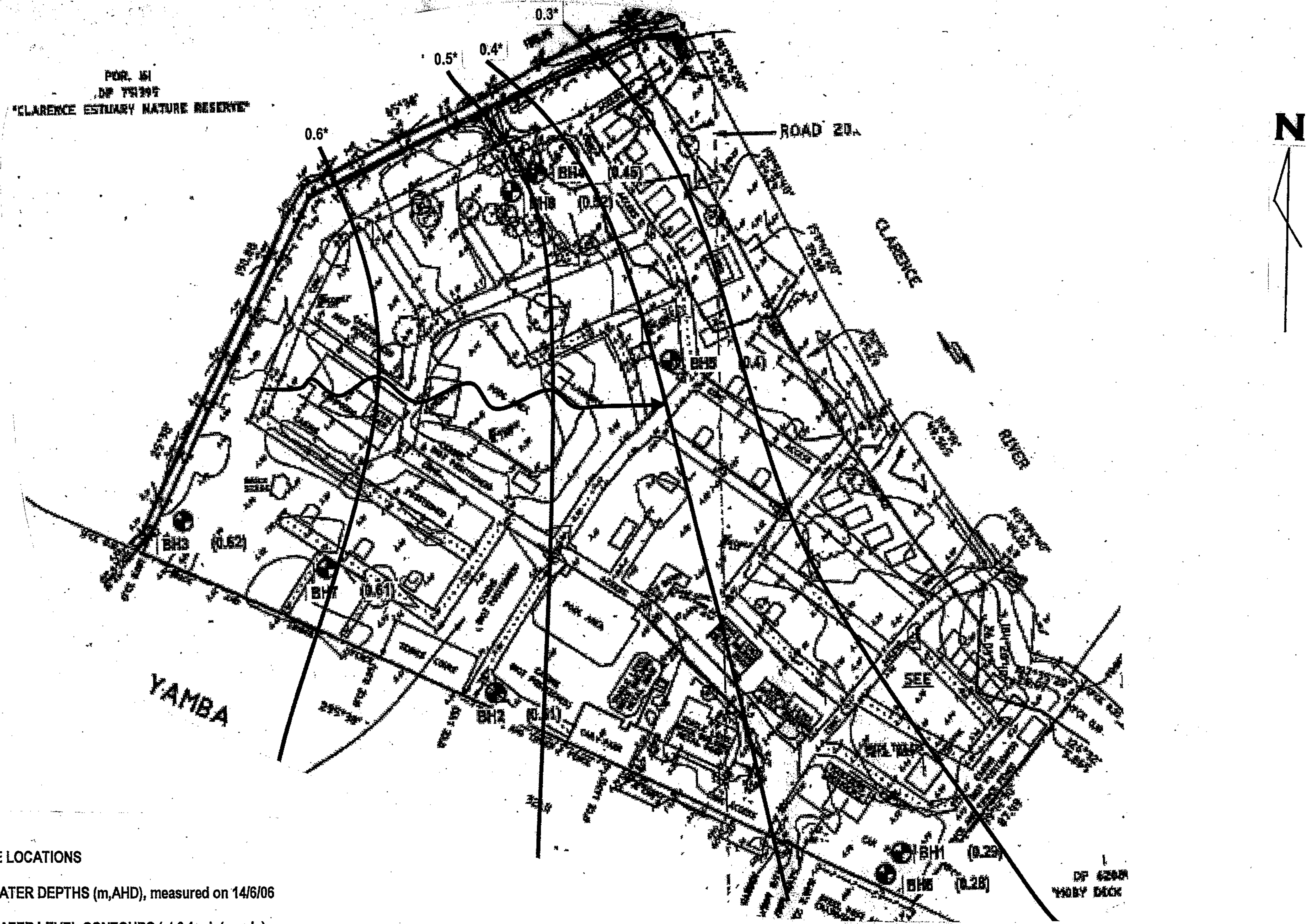
SPECIALISTS MANAGING THE EARTH

client:	JONES LANG LASALLE PTY LTD
project:	DOLPHIN BLUE DEVELOPMENT YAMBA ROAD, YAMBA
title:	EXISTING SITE PLAN
project no: GEOTCOFH01613AA-AG	figure no: FIGURE 2



LEGEND

BH1 BOREHOLE LOCATIONS

(0.29) GROUNDWATER DEPTHS (m,AHD), measured on 14/6/06

0.1* GROUNDWATER LEVEL CONTOURS (at 0.1m intervals)

revision	description	drawn	approved	date

0 15 30 60 90

Scale (metres)

drawn	DJB
approved	
date	18/10/06
scale	1:1,500
original size	A3

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SPECIALISTS MANAGING THE EARTH

client: JONES LANG LASALLE PTY LTD

project: DOLPHIN BLUE DEVELOPMENT
YAMBA ROAD, YAMBA

title: GROUNDWATER LEVEL CONTOUR PLAN

project no: GEOTCOFH01613AA-AG | figure no: FIGURE 3

Appendix A

Engineering Logs

Soil Description Explanation Sheet (1 of 2)

DEFINITION:
In engineering terms soil includes every type of uncemented or partially cemented inorganic or organic material found in the ground. In practice, if the material can be remoulded or disintegrated by hand in its field condition or in water it is described as a soil. Other materials are described using rock description terms.

CLASSIFICATION SYMBOL & SOIL NAME
Soils are described in accordance with the Unified Soil Classification (UCS) as shown in the table on Sheet 2.

PARTICLE SIZE DESCRIPTIVE TERMS

NAME	SUBDIVISION	SIZE
Boulders		>200 mm
Cobbles		63 mm to 200 mm
Gravel	coarse	20 mm to 63 mm
	medium	6 mm to 20 mm
	fine	2.36 mm to 6 mm
Sand	coarse	600 µm to 2.36 mm
	medium	200 µm to 600 µm
	fine	75 µm to 200 µm

MOISTURE CONDITION
Dry Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.

Moist Soil feels cool and darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.

Wet As for moist but with free water forming on hands when handled.

CONSISTENCY OF COHESIVE SOILS

TERM	UNDRAINED STRENGTH s_u (kPa)	FIELD GUIDE
Very Soft	<12	A finger can be pushed well into the soil with little effort.
Soft	12 - 25	A finger can be pushed into the soil to about 25mm depth.
Firm	25 - 50	The soil can be indented about 5mm with the thumb, but not penetrated.
Stiff	50 - 100	The surface of the soil can be indented with the thumb, but not penetrated.
Very Stiff	100 - 200	The surface of the soil can be marked, but not indented with thumb pressure.
Hard	>200	The surface of the soil can be marked only with the thumbnail.
Friable	–	Crumbles or powders when scraped by thumbnail.

DENSITY OF GRANULAR SOILS

TERM	DENSITY INDEX (%)
Very loose	Less than 15
Loose	15 - 35
Medium Dense	35 - 65
Dense	65 - 85
Very Dense	Greater than 85

MINOR COMPONENTS

TERM	ASSESSMENT GUIDE	PROPORTION OF MINOR COMPONENT IN:
Trace of	Presence just detectable by feel or eye, but soil properties little or no different to general properties of primary component.	Coarse grained soils: <5% Fine grained soils: <15%
With some	Presence easily detected by feel or eye, soil properties little different to general properties of primary component.	Coarse grained soils: 5 - 12% Fine grained soils: 15 - 30%

SOIL STRUCTURE

ZONING		CEMENTING	
Layers	Continuous across exposure or sample.	Weakly cemented	Easily broken up by hand in air or water.
Lenses	Discontinuous layers of lenticular shape.	Moderately cemented	Effort is required to break up the soil by hand in air or water.
Pockets	Irregular inclusions of different material.		

GEOLOGICAL ORIGIN
WEATHERED IN PLACE SOILS

Extremely weathered material	Structure and fabric of parent rock visible.
Residual soil	Structure and fabric of parent rock not visible.

TRANSPORTED SOILS

Aeolian soil	Deposited by wind.
Alluvial soil	Deposited by streams and rivers.
Colluvial soil	Deposited on slopes (transported downslope by gravity).
Fill	Man made deposit. Fill may be significantly more variable between tested locations than naturally occurring soils.
Lacustrine soil	Deposited by lakes.
Marine soil	Deposited in ocean basins, bays, beaches and estuaries.

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Soil Description Explanation Sheet (2 of 2)

SOIL CLASSIFICATION INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 60 mm and basing fractions on estimated mass)				USC	PRIMARY NAME
COARSE GRAINED SOILS More than 50% of materials less than 63 mm is larger than 0.075 mm (A 0.075 mm particle is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than 2.0 mm	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.	GW	GRAVEL
			Predominantly one size or a range of sizes with more intermediate sizes missing.	GP	GRAVEL
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	SILTY GRAVEL
			Plastic fines (for identification procedures see CL below)	GC	CLAYEY GRAVEL
	SANDS More than half of coarse fraction is smaller than 2.0 mm	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate sizes missing	SW	SAND
			Predominantly one size or a range of sizes with some intermediate sizes missing.	SP	SAND
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).	SM	SILTY SAND
			Plastic fines (for identification procedures see CL below).	SC	CLAYEY SAND

FINE GRAINED SOILS More than 50% of material less than 63 mm is smaller than 0.075 mm		IDENTIFICATION PROCEDURES ON FRACTIONS <0.2 mm.				
		DRY STRENGTH	**DILATANCY**	**TOUGHNESS**	**USC**	**PRIMARY NAME**
	SILTS & CLAYS Liquid limit less than 50	None to Low	Quick to slow	None	ML	SILT
		Medium to High	None	Medium	CL	CLAY
		Low to medium	Slow to very slow	Low	OL	ORGANIC SILT
	SILTS & CLAYS Liquid limit greater than 50	Low to medium	Slow to very slow	Low to medium	MH	SILT
		High	None	High	CH	CLAY
		Medium to High	None	Low to medium	OH	ORGANIC CLAY
HIGHLY ORGANIC SOILS		Readily identified by colour, odour, spongy feel and frequently by fibrous texture.			Pt	PEAT

• Low plasticity – Liquid Limit W_L less than 35%. • Modium plasticity – W_L between 35% and 50%.

COMMON DEFECTS IN SOIL

TERM	DEFINITION	DIAGRAM
PARTING	A surface or crack across which the soil has little or no tensile strength. Parallel or sub parallel to layering (eg bedding). May be open or closed.	
JOINT	A surface or crack across which the soil has little or no tensile strength but which is not parallel or sub parallel to layering. May be open or closed. The term 'fissure' may be used for irregular joints <0.2 m in length.	
SHEARED ZONE	Zone in clayey soil with roughly parallel near planar, curved or undulating boundaries containing closely spaced, smooth or slickensided, curved intersecting joints which divide the mass into lenticular or wedge shaped blocks.	
SHEARED SURFACE	A near planar curved or undulating, smooth, polished or slickensided surface in clayey soil. The polished or slickensided surface indicates that movement (in many cases very little) has occurred along the defect.	

TERM	DEFINITION	DIAGRAM
SOFTENED ZONE	A zone in clayey soil, usually adjacent to a defect in which the soil has a higher moisture content than elsewhere.	
TUBE	Tubular cavity. May occur singly or as one of a large number of separate or inter-connected tubes. Walls often coated with clay or strengthened by denser packing of grains. May contain organic matter	
TUBE CAST	Roughly cylindrical elongated body of soil different from the soil mass in which it occurs. In some cases the soil which makes up the tube cast is cemented.	
INFILLED SEAM	Sheet or wall like body of soil substance or mass with roughly planar to irregular near parallel boundaries which cuts through a soil mass. Formed by infilling of open joints.	

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Rock Description Explanation Sheet (1 of 2)

The descriptive terms used by Coffey are given below. They are broadly consistent with Australian Standard AS1726-1993.

DEFINITIONS: Rock substance, defect and mass are defined as follows:

Rock Substance	In engineering terms roch substance is any naturally occurring aggregate of minerals and organic material which cannot be disintegrated or remoulded by hand in air or water. Other material is described using soil descriptive terms. Effectively homogenous material, may be isotropic or anisotropic.
Defect	Discontinuity or break in the continuity of a substance or substances.
Mass	Any body of material which is not effectively homogeneous. It can consist of two or more substances without defects, or one or more substances with one or more defects.

SUBSTANCE DESCRIPTIVE TERMS:

ROCK NAME Simple rock names are used rather than precise geological classification.

PARTICLE SIZE Grain size terms for sandstone are:

Coarse grained	Mainly 0.6mm to 2mm
Medium grained	Mainly 0.2mm to 0.6mm
Fine grained	Mainly 0.06mm (just visible) to 0.2mm

FABRIC Terms for layering of penetrative fabric (eg. bedding, cleavage etc.) are:

Massive	No layering or penetrative fabric.
Indistinct	Layering or fabric just visible. Little effect on properties.
Distinct	Layering or fabric is easily visible. Rock breaks more easily parallel to layering of fabric.

CLASSIFICATION OF WEATHERING PRODUCTS

Term	Abbreviation	Definition
Residual Soil	**RS**	Soil derived from the weathering of rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely Weathered Material	**XW**	Material is weathered to such an extent that it has soil properties, ie, it either disintegrates or can be remoulded in water. Original rock fabric still visible.
Highly Weathered Rock	**HW**	Rock strength is changed by weathering. The whole of the rock substance is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Some minerals are decomposed to clay minerals. Porosity may be increased by leaching or may be decreased due to the deposition of minerals in pores.
Moderately Weathered Rock	**MW**	The whole of the rock substance is discoloured, usually by iron staining or bleaching , to the extent that the colour of the fresh rock is no longer recognisable.
Slightly Weathered Rock	**SW**	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.
Fresh Rock	**FR**	Rock substance unaffected by weathering.

Notes on Weathering:

1. AS1726 suggests the term "Distinctly Weathered" (DW) to cover the range of substance weathering conditions between XW and SW. For projects where it is not practical to delineate between HW and MW or it is judged that there is no advantage in making such a distinction. DW may be used with the definition given in AS1726.
2. Where physical and chemical changes were caused by hot gasses and liquids associated with igneous rocks, the term "altered" may be substituted for "weathering" to give the abbreviations XA, HA, MA, SA and DA.

ROCK SUBSTANCE STRENGTH TERMS

Term	Abbreviation	Point Load Index, I_S50 (MPa)	Field Guide
Very Low	**VL**	Less than 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; pieces up to 30mm thick can be broken by finger pressure.
Low	**L**	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show with firm bows of a pick point; has a dull sound under hammer. Pieces of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium	**M**	0.3 to 1.0	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High	**H**	1 to 3	A piece of core 150mm long by 50mm can not be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High	**VH**	3 to 10	Hand specimen breaks after more than one blow of a pick; rock rings under hammer.
Extremely High	**EH**	More than 10	Specimen requires many blows with geological pick to break; rock rings under hammer.

Notes on Rock Substance Strength:

1. In anisotropic rocks the field guide to strength applies to the strength perpendicular to the anisotropy. High strength anisotropic rocks may break readily parallel to the planar anisotropy.
2. The term "extremely low" is not used as a rock substance strength term. While the term is used in AS1726-1993, the field guide therein makes it clear that materials in that strength range are soils in engineering terms.
3. The unconfined compressive strength for isotropic rocks (and anisotropic rocks which fall across the planar anisotropy) is typically 10 to 25 times the point load index (Is50). The ratio may vary for different rock types. Lower strength rocks often have lower ratios than higher strength rocks.

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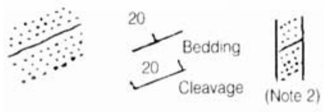
Rock Description Explanation Sheet (2 of 2)

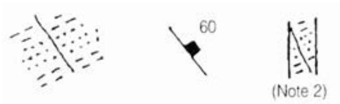
COMMON DEFECTS IN ROCK MASSES

Term | Definition | Diagram | Map Symbol | Graphic Log (Note 1)

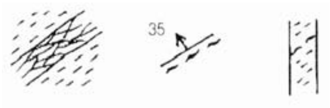
Parting — A surface or crack across which the rock has little or no tensile strength. Parallel or sub parallel to layering (eg bedding) or a planar anisotropy in the rock substance (eg, cleavage). May be open or closed.



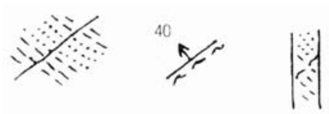
Joint — A surface or crack across which the rock has little or no tensile strength. but which is not parallel or sub parallel to layering or planar anisotropy in the rock substance. May be open or closed.



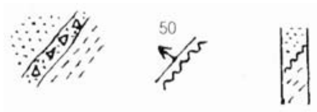
Sheared Zone (Note 3) — Zone of rock substance with roughly parallel near planar, curved or undulating boundaries cut by closely spaced joints, sheared surfaces or other defects. Some of the defects are usually curved and intersect to divide the mass into lenticular or wedge shaped blocks.



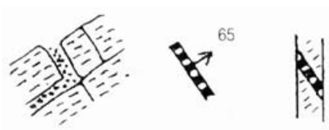
Sheared Surface (Note 3) — A near planar, curved or undulating surface which is usually smooth, polished or slickensided.



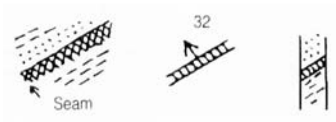
Crushed Seam (Note 3) — Seam with roughly parallel almost planar boundaries, composed of disoriented, usually angular fragments of the host rock substance which may be more weathered than the host rock. The seam has soil properties.



Infilled Seam — Seam of soil substance usually with distinct roughly parallel boundaries formed by the migration of soil into an open cavity or joint, infilled seams less than 1mm thick may be described as veneer or coating on joint surface.



Extremely Weathered Seam — Seam of soil substance, often with gradational boundaries. Formad by weathering of the rock substance in place.



DEFECT SHAPE — TERMS

Planar	The defect does not vary in orientation
Curved	The defect has a gradual change in orientation
Undulating	The defect has a wavy surface
Stepped	The defect has one or more well defined steps
Irregular	The defect has many sharp changes of orientation

Note: The assessment of defect shape is partly influenced by the scale of the observation.

ROUGHNESS TERMS

Slickensided	Grooved or striated surface, usually polished
Polished	Shiny smooth surface
Smooth	Smooth to touch. Few or no surface irregularities
Rough	Many small surface irregularities (amplitude generally less than 1mm). Feels like fine to coarse sand paper.
Very Rough	Many large surface irregularities (amplitude generally more than 1mm). Feels like, or coarser than very coarse sand paper.

COATING TERMS

Clean	No visible coating
Stained	No visible coating but surfaces are discoloured
Veneer	A visible coating of soil or mineral, too thin to measure; may be patchy
Coating	A visible coating up to 1mm thick. Thicker soil material is usually described using appropriate defect terms (eg, infilled seam). Thicker rock strength material is usually described as a vein.

BLOCK SHAPE TERMS

Blocky	Approximately equidimensional
Tabular	Thickness much less than length or width
Columnar	Height much greate than cross section

Notes on Defects:

1. Usually borehole logs show the true dip of defects and face sketches and sections the apparent dip.
2. Partings and joints are not usually shown on the graphic log unless considered significant.
3. Sheared zones, sheared surfaces and crushed seams are faults in geological terms.

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Engineering Log - Piezometer

Borehole No.	**BH1**
Sheet	1 of 1
Project No:	**CH1613/1**

Client:	**RIDER HUNT TEROTECH**	Date started:	**9.5.2006**
Principal:		Date completed:	**9.5.2006**
Project:	**PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT**	Logged by:	**ELC**
Borehole Location:	**REFER TO FIGURE 1**	Checked by:	

drill model & mounting: MD200 TRUCK	Easting: 533265.923	slope: -90°	R.L. Surface: 1.59
hole diameter:	Northing: 6744036.446	bearing:	datum: AHD

drilling information / **material substance**

method	penetration 1 2 3	support	water	notes samples, tests, etc	well details	RL	depth metres	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	structure and additional observations
ADT		N					0		SP	SAND: fine to medium grained, dark brown	D/M	MD	ALLUVIAL SOIL Roots to 0.05m.
						1					M		
							1			Colour change to pale brown/brown.			
			11.30 11/5/06							Colour change to pale grey.		MD/D	
				SPT 2,4,7 N*=11		0					W		
							2						
						-1						D	
				SPT 3,9,10 N*=19			3			Indurated layer at 3.25m to 3.4m, dark brown. shell fragments observed. Colour change to grey.			
						-2	4					VD	
				SPT R N=R		-3				Indurated layer at 4.4m-4.8m.			SPT carried out with solid cone. 25 blows for 50mm penetration.
							5			Thin bands of indurated sand between 4.8m and 6m			
						-4	6						
										End of BH1 due to limit of required investigation. Borehole terminated at 6m			
						-5	7						
						-6	8						

method
AS auger screwing*
AD auger drilling*
RR roller/tricone
W washbore
CT cable tool
DT diatube
B blank bit
V V bit
T TC bit
TBX Tubex
*bit shown by suffix
e.g. ADT

support
C casing N nil

penetration
1 2 3 4
no resistance ranging to refusal

water
10/1/98 water level on date shown
water inflow
water outflow

notes, samples, tests
U_{50} undisturbed sample 50mm diameter
D disturbed sample
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
P pressure meter
Bs bulk sample
R refusal
E environmental sample
PID PID measurement
WS water sample
PZ piezometer
ALT air lift test

classification symbols and soil description
based on unified classification system

moisture
D dry
M moist
W wet
Wp plastic limit
W_L liquid limit

consistency/density index
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

PIEZOMETER CH1613-1 DEEP LOGS.GPJ COFFEY.GDT 19.10.06

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Engineering Log - Piezometer

Borehole No.	**BH2**
Sheet	1 of 1
Project No:	**CH1613/1**
Date started:	**9.5.2006**
Date completed:	**9.5.2006**
Logged by:	**ELC**
Checked by:	

Client: **RIDER HUNT TEROTECH**

Principal:

Project: **PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT**

Borehole Location: **REFER TO FIGURE 1**

drill model & mounting: MD200 TRUCK	Easting: 533105.125	slope:	-90°	R.L. Surface:	1.56
hole diameter:	Northing: 6744107.829	bearing:		datum:	AHD

drilling information | **material substance**

method	penetration 1 2 3	support	water	notes samples, tests, etc	well details	RL	depth metres	graphic log	classification symbol	material: soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/density index	structure and additional observations
ADT		N					0			SAND: fine to medium grained, dark brown	D/M	L	ALLUVIAL SOIL Roots to 0.05m.
						1				Colour change to pale brown/brown.			
			1.50PM 9/5/06				1					L/MD	
										Colour change to pale brown.	W		
				SPT 2,2,2 N*=4		0				Colour change to grey.			
							2					D	
						-1							
				SPT 3,6,10 N*=16			3						
						-2				Possible indurated sand layer at 3.3m			
							4					VD	
				SPT 25,R N*=R		-3				Possible indurated sand layer at 4.45m Shell fragments observed			SPT carried out with solid cone. 25 blows for 140mm penetration.
							5						
						-4				Indurated dark brown sand.			
							6			END of BH2 at 6m due to limit of required investigation. Borehole terminated at 6m			
						-5	7						
						-6	8						

method

AS	auger screwing*
AD	auger drilling*
RR	roller/tricone
W	washbore
CT	cable tool
DT	diatube
B	blank bit
V	V bit
T	TC bit
TBX	Tubex

*bit shown by suffix
e.g. ADT

support
C casing N nil

penetration
1 2 3 4
no resistance ranging to refusal

water
10/1/98 water level on date shown
water inflow
water outflow

notes, samples, tests

U_{50}	undisturbed sample 50mm diameter
D	disturbed sample
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
P	pressure meter
Bs	bulk sample
R	refusal
E	environmental sample
PID	PID measurement
WS	water sample
PZ	piezometer
ALT	air lift test

classification symbols and soil description
based on unified classification system

moisture

D	dry
M	moist
W	wet
Wp	plastic limit
W_L	liquid limit

consistency/density index

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

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Engineering Log - Piezometer

Borehole No.	**BH3**
Sheet	1 of 1
Project No:	**CH1613/1**
Date started:	**9.5.2006**
Date completed:	**9.5.2006**
Logged by:	**ELC**
Checked by:	

Client: **RIDER HUNT TEROTECH**

Principal:

Project: **PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT**

Borehole Location: **REFER TO FIGURE 1**

drill model & mounting: MD200 TRUCK | Easting: 533004.455 | slope: -90° | R.L. Surface: 1.47

hole diameter: | Northing: 6744176.818 | bearing: | datum: AHD

drilling information | **material substance**

method	penetration 1 2 3	support	water	notes samples, tests, etc	well details	RL	depth metres	graphic log	classification symbol	material soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	structure and additional observations
ADT		N								SAND: fine to medium grained, dark brown.	D	MD	ALLUVIAL SOIL Roots to 0.05m.
						1					M		
										Colour change to pale brown and brown at 0.7m			
			3pm 9/5/06				1			Colour change to pale grey at 1.0m.			
				SPT 2,2,2 N*=4		0					W		
							2			Colour change to grey at 2.0m.			
						-1						D	
				SPT 3,6,10 N*=16			3						
						-2				Indurated layer from 3.25 to 3.4m, dark brown. Shell fragments observed Colour change to grey at 3.4m.			
							4					D/VD	
				SPT 25,-,- N*=R		-3				Indurated layer from 4.4m to 4.8m.			SPT carried out with solid cone. 25 blows for 140mm penetration
							5						
						-4				Possibly becomes gravelly sand			
							6			BH3 terminated at 6m due to limit of required investigation. Borehole terminated at 6m			
						-5							
							7						
						-6							
							8						

method

AS	auger screwing*
AD	auger drilling*
RR	roller/tricone
W	washbore
CT	cable tool
DT	diatube
B	blank bit
V	V bit
T	TC bit
TBX	Tubex
*bit shown by suffix	
e.g.	ADT

support

C casing N nil

penetration

1 2 3 4 — no resistance ranging to refusal

water

10/1/98 water level on date shown

water inflow

water outflow

notes, samples, tests

U_{50}	undisturbed sample 50mm diameter
D	disturbed sample
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
P	pressure meter
Bs	bulk sample
R	refusal
E	environmental sample
PID	PID measurement
WS	water sample
PZ	piezometer
ALT	air lift test

classification symbols and soil description

based on unified classification system

moisture

D	dry
M	moist
W	wet
Wp	plastic limit
W_L	liquid limit

consistency/density index

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

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Engineering Log - Piezometer

Borehole No. **BH4**
Sheet 1 of 1
Project No: **CH1613/1**

Client:	*RIDER HUNT TEROTECH*	Date started:	*9.5.2006*
Principal:		Date completed:	*9.5.2006*
Project:	*PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT*	Logged by:	*ELC*
Borehole Location:	*REFER TO FIGURE 1*	Checked by:	

drill model & mounting: MD200 TRUCK	Easting: 533147.421	slope: -90°	R.L. Surface: 1.82
hole diameter:	Northing: 6744309.382	bearing:	datum: AHD

drilling information | **material substance**

method	penetration 1 2 3	support	water	notes samples, tests, etc	well details	RL	depth metres	graphic log	classification symbol	material: soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	structure and additional observations
ADT		N					0			SAND: fine to medium grained, brown to dark brown.	D	MD	ALLUVIAL SOIL
							0.2				M		
						1	1			Colour change to pale grey at 0.8m.		D	
							1.2				W		
			11:20am 11/5/06	SPT 2,5,9 N*=14		0	2						
							2.2					MD	
				SPT 1,2,6 N*=8		-1	3			Shell fragments observed			
						-2	4					D/VD	
				SPT 8,12,17 N*=29		-3	5						
						-4	6			BH4 terminated at 6m due to limit of required investigation. Borehole terminated at 6m			
						-5	7						
						-6	8						

method
AS auger screwing*
AD auger drilling*
RR roller/tricone
W washbore
CT cable tool
DT diatube
B blank bit
V V bit
T TC bit
TBX Tubex
*bit shown by suffix
e.g. ADT

support
C casing N nil

penetration
1 2 3 4
no resistance ranging to refusal

water
10/1/98 water level on date shown
water inflow
water outflow

notes, samples, tests
U_{50} undisturbed sample 50mm diameter
D disturbed sample
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
P pressure meter
Bs bulk sample
R refusal
E environmental sample
PID PID measurement
WS water sample
PZ piezometer
ALT air lift test

classification symbols and soil description
based on unified classification system

moisture
D dry
M moist
W wet
Wp plastic limit
W_L liquid limit

consistency/density index
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

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Engineering Log - Piezometer

Borehole No.	**BH5**
Sheet	1 of 1
Project No:	**CH1613/1**

Client:	*RIDER HUNT TEROTECH*	Date started:	*9.5.2006*
Principal:		Date completed:	*9.5.2006*
Project:	*PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT*	Logged by:	*ELC*
Borehole Location:	*REFER TO FIGURE 1*	Checked by:	

drill model & mounting: MD200 TRUCK	Easting: 533192.849	slope: -90°	R.L. Surface: 1.80
hole diameter:	Northing: 6744254.937	bearing:	datum: AHD

drilling information / **material substance**

method	penetration 1 2 3	support	water	notes samples, tests, etc	well details	RL	depth metres	graphic log	classification symbol	material: soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	structure and additional observations
ADT		N					0			SAND: Fine to medium grained, brown.	M	L	ALLUVIAL SOIL
										Colour change to pale brown at 0.4m.			
						1	1			Colour change to grey at 1.2m.	W		
			12;10 11/5/06	SPT 1,0,2 N*=2		0	2						
												MD	
				SPT 2,2,5 N*=7		-1	3			Numerous shell fragments throughout profile.			
						-2	4					D/VD	
				SPT 6,13,17 N*=30		-3	5						
						-4	6			BH5 terminated at 6m due to limit of required investigation. Borehole terminated at 6m			
						-5	7						
						-6	8						

method

AS	auger screwing*
AD	auger drilling*
RR	roller/tricone
W	washbore
CT	cable tool
DT	diatube
B	blank bit
V	V bit
T	TC bit
TBX	Tubex
*bit shown by suffix	
e.g.	ADT

support

C casing N nil

penetration

1 2 3 4 no resistance ranging to refusal

water

10/1/98 water level on date shown

water inflow

water outflow

notes, samples, tests

U_{50}	undisturbed sample 50mm diameter
D	disturbed sample
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
P	pressure meter
Bs	bulk sample
R	refusal
E	environmental sample
PID	PID measurement
WS	water sample
PZ	piezometer
ALT	air lift test

classification symbols and soil description

based on unified classification system

moisture

D	dry
M	moist
W	wet
Wp	plastic limit
W_L	liquid limit

consistency/density index

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

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Engineering Log - Piezometer

Borehole No. **BH6**

Sheet 1 of 4

Project No: **CH1613/1**

Client:	*RIDER HUNT TEROTECH*	Date started:	*31.5.2006*
Principal:		Date completed:	*31.5.2006*
Project:	*PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT*	Logged by:	*ELC*
Borehole Location:	*REFER TO FIGURE 1*	Checked by:	

drill model & mounting: P120 TRUCK	Easting: 533262.034	slope: -90°	R.L. Surface: 1.75
hole diameter:	Northing: 6744033.133	bearing:	datum: AHD

drilling information / material substance

method	support	water	notes samples, tests, etc	RL	depth metres	material (soil type: plasticity or particle characteristics, colour, secondary and minor components)	moisture condition	consistency/density index	structure and additional observations
ADV	N				0	**FILL:** Sand, fine to medium grained, dark brown, trace of coarse grained gravel (sandstone origin)	M	L?	FILL
			D	1		**SAND:** fine to medium grained, brown			ALLUVIAL SOIL
			D		1				
		►—	D			Colour change to grey.	W		
			SPT 5,10,11 N*=21	0				D	No sample returned (rock in tip of SPT sampler)
					2				
			D	-1					
			D		3				
			SPT 4,7,10 N*=17						
				-2					
			D		4	Colour change to dark brown at approximately 4m.		D/VD	
WB	M		D						
			SPT 7,12,15 N*=27	-3					
					5			VD	
			SPT 14,25,R N*=R	-4		Becomes indurated sand.			SPT 25 blows for 150mm penetration.
					6				
				-5					
					7				
			SPT 31,R,- N*=R						SPT 31 blows for 150mm penetration.
				-6					
					8				

method
AS auger screwing*
AD auger drilling*
RR roller/tricone
W washbore
CT cable tool
DT diatube
B blank bit
V V bit
T TC bit
TBX Tubex
*bit shown by suffix
e.g. ADT

support
C casing N nil

penetration
1 2 3 4
no resistance ranging to refusal

water
10/1/98 water level on date shown
water inflow
water outflow

notes, samples, tests
U_{50} undisturbed sample 50mm diameter
D disturbed sample
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
P pressure meter
Bs bulk sample
R refusal
E environmental sample
PID PID measurement
WS water sample
PZ piezometer
ALT air lift test

classification symbols and soil description
based on unified classification system

moisture
D dry
M moist
W wet
Wp plastic limit
W_L liquid limit

consistency/density index
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense