DOLPHIN BLUE DEVELOPMENT DEWATERING MANAGEMENT STATEMENT

Jones Lang Lasalle Pty Ltd

GEOTCOFH01613AA-AG
18 October 2006

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

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

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

DEWATERING MANAGEMENT STATEMENT

Coffey Geotechnics Pty Ltd is pleased to present our Dewatering Management Statement 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

This report presents a dewatering management statement developed by Coffey Geotechnics Pty Ltd (Coffey) for the proposed Dolphin Blue development to be located at an 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 the following:

- A preliminary assessment of the presence of acid sulphate soils (ASS);
- Groundwater assessment of:
  - Inflow rates into basement excavations (underground car parking);
  - Treatment of groundwater from excavations prior to discharge;
  - Potential for groundwater drawdown from dewatering of excavations and the possibility of settlement to nearby structures;
  - The possible need for the reinjection of groundwater to control groundwater drawdown effects; and

Coffey conducted the work in general accordance with proposal no. CH1613/1-AB. This report presents the results of the site investigation. A separate Coffey report address geotechnical conditions at the site (Ref: GEOTCOFH01613AA-AF). A preliminary site contamination assessment was carried out by Coffey Geosciences Pty Ltd (Ref: CH1613/2-AC).

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 AND METHODOLOGY

The scope of this report was to compile a Dewatering Management Statement (DMS) for the site, for issue to regulatory authorities, which addresses the information requirements of Department of Natural Resources (DNR) regarding construction dewatering. The DNR information requirements for the DMS can be summarised as follows:

- **Development Plan**: General information including the geometry of the excavated area occurring beneath the water table, the method of dewatering, and maps showing proposed dewatering infrastructure.

- **Impact Assessment**: Assessment of dewatering impacts including predicted drop in water table, estimates of total groundwater volume to be extracted, predicted impacts on local groundwater users or groundwater dependent ecosystems, and off-site impacts of a geotechnical nature (such as the potential for groundwater-induced settlement).
- **Groundwater Quality and Disposal**: Assessment of the local groundwater quality, the method of disposal of pumped water, and the compatibility of extracted groundwater to in-situ groundwater if re-injection is proposed (for stability purposes).

The completed basements will require tanking wherever they fall below the water table, therefore long-term impacts would generally only include any potential rise in groundwater levels following installation of the barrier caused by tanking. Organic contaminants are known to occur in groundwater elsewhere in the area, and treatment may be required prior to disposal depending on disposal methods.

The methodology employed to carry out the scope of work consisted of the following:

- Desktop review of available data and request of DNR database records on surrounding groundwater users.
- A preliminary field assessment to assess local groundwater levels, groundwater quality, depth of aquifer and soil characteristics. This involved the drilling of eight boreholes, five of the boreholes (BH1 to BH5) were drilled to depths of 6m and had shallow groundwater monitoring piezometers installed to depths of about 2.5m. Three addition holes (BH6 to BH8) were also drilled to depths of between 25.45m and 44.95m (to assess the depths of the sand aquifer) at the site. Standpipe piezometers were installed in or adjacent to each of the deep boreholes to depths of approximately 6m. Field activities comprised:
  - a preliminary visit to the site to assess locations of underground services and mark preliminary borehole locations;
  - drilling of eight boreholes, and installation of groundwater monitoring piezometers;
  - collection of soil data (Standard Penetrometer Testing for geotechnical parameters of unconsolidated sediments, lithology logging);
  - groundwater level measurement with data loggers and sampling; and
  - hydraulic testing at each piezometer.
- Review of field data, and assessment of:
  - depth of aquifer and the requirement for partial cut-off walls or other means of groundwater control;
  - dewatering volumes and requirements;
  - dewatering statement and any requirements for recharge;
  - groundwater quality and potential treatment requirements for disposal or for use in a recharge system (if required);
  - groundwater monitoring requirements;

  - potential for groundwater-induced settlement off-site;
  - long-term changes in groundwater levels due to installation of a tanked basement; and
  - potential impacts on local groundwater users (if any).
- Compilation of this Dewatering Management Statement as a report.

4 FIELD WORK

4.1 Drilling

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.

4.2 Installation of Standpipe Peizometers

Groundwater monitoring wells were installed in boreholes BH1 to BH5 to depths of about 2.5m. Standpipe piezometers were installed adjacent to the three deeper boreholes (BH6 to BH8) to depths of about 6m. The piezometers were installed to allow sampling to assess groundwater quality and to assess groundwater levels and the permeability parameters of the near surface sand deposits.

The five shallow piezometers and the lower 3m section of each of the deeper piezometers were screened with 50mm machine slotted PVC pipe. Fine gravel was placed within the well annulus to a level of about 0.4m below the ground surface, followed by an annular seal of granular bentonite pellets to about 0.2m below ground surface and then backfilled with drilling cuttings.

4.3 Groundwater Level Monitoring

Automatic water level recorders (data loggers) were installed in the five shallow piezometers (BH1 to BH5) on 24 May 2006 to collect information on the response of the groundwater table over time, particularly with respect to rainfall and tidal fluctuations. Data recorded by the loggers was downloaded on 14 June 2006, at which time the water depth was assessed by hand measurement. Two of the loggers failed to record data over the period, one due to tampering and the other due to water entering the logger. The results of the three groundwater loggers that recorded data over the period are presented graphically in Appendix B, along with daily rainfall data. Groundwater levels are discussed in later sections of this report.

4.4 Falling Head Testing

Falling head tests were carried out in seven of the eight piezometers installed. The test involved introduction of water into the piezometer and recording of the timing of recovery of water level within the piezometer to the pre-test level. The results of the falling head testing are presented and discussed in section 6.2.

4.5 Groundwater Sampling

Groundwater sampling was conducted on 29 May 2006. A total of five samples were collected (one from each piezometer of the shallow piezometers). Bores were purged of a minimum of five well bore volumes prior to sampling. Groundwater samples were placed in sample bottles prepared by the laboratory and dispatched under Chain of Custody conditions (within the holding times nominated by the laboratory) to a NATA-registered laboratory for analysis.

5 SITE CONDITIONS

5.1 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.

5.2 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.

5.3 Subsurface Conditions

The subsurface conditions interpreted from the boreholes (BH1 to BH8) are summarised in Table 1. 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 1 - 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.

5.4 Groundwater Depths

A summary of the depths of standpipe piezometers installed at the site is presented in Table 2. Table indicates the depth of the boreholes, the depth of the piezometers and the standing groundwater levels as measured on 14 June 2006.

Table 2 - 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 2 indicate the groundwater flows in a north-east direction towards the Clarence River.

6 FIELD AND LABORATORY TESTING AND DISCUSSION OF RESULTS

6.1 Groundwater Level Monitoring

Table 2 shown above indicates the depth of the boreholes, the depth of the piezometers and the standing groundwater levels as measured on 14 June 2006.

Groundwater levels measured on 14 June 2006 have been reduced using local features where levels reduced to Australian Height Datum were available. Figure 3 shows the measured groundwater levels at each monitoring location and presents an interpretation of groundwater head contours and the direction of groundwater flow. Groundwater is interpreted to flow northward towards the Clarence

River. The water table gradient is approximately 0.0007 or a 0.07m fall in water table over a distance of 100m.

Automatic water level recorders (data loggers) were installed in the five shallow piezometers (BH1 to BH5) on 24 May 2006 to collect information on the response of the groundwater table over time, particularly with respect to rainfall and tidal fluctuations. Two of the loggers failed to record data over the period, one due to tampering and the other due to water entering the logger. The results of the three groundwater loggers that recorded data over the period are presented graphically in Appendix B, along with daily rainfall data. The groundwater fluctuations, based on the recorded data, are summarised in Table 3 below. Small increases (typically 0.1m) were noted in mid June in the days following rainfall (23.4mm of rainfall was recorded on 10 June 2006). Minor fluctuations of up to about 70mm were noted, which may relate to tidal effects.

Table 3 - Summary of Groundwater Fluctuations

Borehole	**Highest Water Level (m bgs*)**	**Lowest Water Level (m bgs*)**
BH3	0.88	1.04
BH4	1.39	1.67
BH5	1.42	1.51

* m bgs = metres below ground surface

6.2 Falling Head Testing

Falling head tests were carried out in seven of the eight piezometers installed. The test involves introduction of water into the piezometer and recording of the timing of recovery of water level within the piezometer to the pre-test level. In the shallow boreholes (BH1 to BH5), the groundwater infiltration was too rapid to allow reliable measurement of the initial rate of inflow with groundwater level recovering to within a few centimetres of pre-test level within 30 seconds of filling the piezometers. Several of the falling head tests undertaken in the deeper piezometers had inflow rates slow enough to measure over time. However these results may have been affected by the drilling polymer used when drilling the holes. Given the rapid response rate of the falling head tests in the shallow piezometers, manual measurement of groundwater recovery is not considered responsive enough to allow assessment of permeability. Based on the rapid recovery rates of the sand units in BH1 to BH5, permeability is likely to exceed 10m/d at each of the boreholes tested.

The permeability results for two of the three deep piezometers were calculated using Hvorslev's method (1951) and are presented in Table 4. The data calculation sheets for BH6 and BH7 are detailed in Appendix C.

Table 4 - Summary of Permeability Testing

Borehole	Unit Screened (m)	Calculated Permeability (m/day) (Hvorslev’s Method 1951)
BH1	Sand	Rapid recovery - 10m/d or more
BH2	Sand	Rapid recovery - 10m/d or more
BH3	Sand	Rapid recovery - 10m/d or more
BH4	Sand	Rapid recovery - 10m/d or more
BH5	Sand	Rapid recovery - 10m/d or more
BH6	Sand	0.84
BH7	Sand	0.38

Based on the above results, it is the considered that the permeability results of the upper sand materials are indicative of sands and silty sand soils.

Based on the above test results and our experience with similar soil materials we have adopted a soil permeability (k) value of 10m/day for the upper sand units for the assessment of groundwater inflows for basement design.

The above permeability values are based on the assessment of the geotechnical unit as a whole. Zones of higher and lower permeability could occur in the sand units. Permeability values for discrete layers may be of the order of one to two orders of magnitude higher than that given above.

6.3 Groundwater Quality

A summary of the results of laboratory testing on groundwater samples taken on 29 May 2006 are summarised in Table 5. Laboratory report sheets are presented in Appendix D. Laboratory quality assurance testing comprising testing of a method blank and spike recovery testing did not produce significant anomalies and the results are considered to be usable.

Table 5 - Groundwater Quality Test Results (Sampled 29 May 2006)

Parameter	**BH1**	**BH2**	**BH3**	**BH4**	**BH5**
Cations					
Calcium (mg/L)	80	82	260	110	82
Magnesium (mg/L)	13	18	130	62	11
Potassium (mg/L)	1.7	2.1	30	23	2.4
Sodium (mg/L)	33	67	1100	320	34
Chloride (mg/L)	61	110	2000	530	60
Sulfate (a S mg/L)	5.9	9.1	110	57	1.8
Ammonia (as N mg/L)	0.18	0.16	0.63	0.28	0.29
Nitrate (as N mg/L)	<0.02	<0.02	0.02	<0.02	<0.02
Alkalinity (mg $CaCO_3$/L)					
Bicarbonate Alkalinity	210	250	260	260	210
Carbonate Alkalinity	<0.5	<0.5	<0.5	<0.5	<0.5
Total Alkalinity	210	250	260	260	210
Total Recoverable Hydrocarbons (TPH)	Not detected	Not detected	Not tested	Not tested	Not tested
Monocyclic Aromatic Hydrocarbons (BTEX)	Not detected	Not detected	Not tested	Not tested	Not tested
Polycyclic Aromatic Hydrocarbons (PAH)	Not detected	Not detected	Not tested	Not tested	Not tested

A service station is present near the Yamba Road side of the site. Testing was carried out for petroleum hydrocarbons and monocyclic aromatic hydrocarbons and polycyclic aromatic hydrocarbons, as these contaminants are commonly associated with petrol stations. Concentrations of Total Petroleum Hydrocarbons (TPH), Benzene Toluene, Ethyl-benzene, Xylene (BTEX) and Poly-Aromatic Hydrocarbons (PAH) are below the laboratory detection limits, indicating that groundwater at the sampling locations (i.e. BH1 to BH5) is not affected by the petrol station operations.

A separate report addressing soil and groundwater contamination associated with the service station was prepared by Coffey (CH1613/2-AC dated 5 June 2006). This report identified the presence of petroleum hydrocarbon contamination in soil and groundwater in the vicinity of underground fuel storage tanks. It is possible that groundwater down gradient (to the north east) of the petrol station is also affected by hydrocarbon contamination.

6.4 Acid Sulphate Soils

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.

On the basis of the screening results, 14 samples were selected for additional testing to assess the potential for acid generation. The results of the acid sulphate testing to assess acid sulphate soil conditions are presented in Appendix E.

7 DISCUSSION AND RECOMMENDATIONS

7.1 Assessment of Dewatering Requirements

In the following discussions, construction dewatering requirements have been considered in the absence of groundwater drawdown mitigation measures in assessing impacts on the groundwater system and local groundwater users. This provides a basis for appraisal of the potential significance of the project in relation to groundwater, and serves to confirm that mitigation measures would be required as part of groundwater control during the construction process. The analysis also provides a basis for assessment of the level of control required to manage groundwater impacts during construction.

7.2 Construction Requirements

The development will involve installation of basements with surface elevations of approximately 0.5m AHD (approximately 1m below the existing ground level) in general. One area of basement about 70m long by 30m wide and located near the middle of the site will have a surface elevation of -0.5m AHD. During construction, it is anticipated that drawdown to a level of -1.5m AHD to -0.5m AHD will be required to achieve workable conditions within the construction excavation site and to allow excavation to sufficient depth for the bulk of construction activity. Localised dewatering will also be required to about 1m to 1.5m below the basement levels for lift pit construction.

Measured groundwater levels during the field program varied from 0.3mAHD to 0.6mAHD, indicating that a groundwater drawdown of approximately 1.5m to 2m will be required to support construction activities. Data from the NSW Department of Public Works (1964) on tidal gradient within the Yamba area indicates spring tides up to about RL1.5AHD can be expected at the site. Monitoring records have shown groundwater fluctuation which may be related to tidal effects. These fluctuations were small at the monitored locations but tidal influence on groundwater level can be expected to increase closer to the Clarence River.

Taking the above into consideration the current groundwater level would need to be drawn down about 1.5m to 2m to allow construction, requiring extraction of significant volumes of water and resulting in drawdown of groundwater levels in the vicinity of the site. Groundwater drawdown due to dewatering can lead to settlement of houses and other structures near the site during construction. The volume and quality of groundwater extracted needs to be considered to assess disposal options for the water.

The extent and sequencing of construction works is a relevant consideration in the assessment of dewatering impacts. For work carried out progressively with individual dewatering operations for each structure the drawdown impacts would be significantly less than if the entire site were to be dewatered during the complete construction program. Coffey recommend dewatering be carried for individual construction areas as required and the assessment of groundwater inflows and drawdown impacts is carried out on this basis.

7.3 Acid Sulphate Soils

7.3.1 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.

7.3.2 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.

7.3.3 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 6.

Table 6 - 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 6:

- 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 E and are summarised in Table 7.

Table 7 - 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.

7.3.4 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.

7.4 DNR Record Search

A search of registered groundwater bores in close proximity of the site was undertaken. Records were obtained for registered bores within a 0.5km radius of the site. A total of nine bores were identified, and all being used for domestic purposes.

Appendix G lists the records extracted by DNR for the bores within 0.5km of the site, and a location plan of the bores. A summary of the bored is presented in Table 8.

Table 8 - Summary of Registered Groundwater Users

Bore Number	Bore Depth (m)	Use	Standing Water Depth Below Ground Level (m)	Water Bearing Zone Depth (m)	Distance And Direction From Site
GW305621	10	Domestic	Not recorded	Not recorded	200m east-southeast
GW305519	4	Domestic	Not recorded	Not recorded	120m south
GW305417	4.5	Domestic	1.5	1.4 to 4.5	65m south
GW305102	4	Domestic	1.5	1.5 to 4	60m south
GW305448	4	Domestic	3.0	3 to 4	40m south
GW303817	4	Domestic	2.0	Not recorded	200m west-northwest
GW305494	4.5	Domestic	1.5	1.5 to 4.5	200m west-northwest
GW304896	6	Domestic	2.0	2 to 6	300m northwest
GW304901	6	Domestic	1.8	1.8 to 6	300m northwest

These bores are either up-gradient from the site or at distances too great to be influenced by water quality on the site. It is assessed that these locations would not be affected by hydrocarbon contamination identified in groundwater at the service station location.

7.5 Aquifer Parameters

The results of current investigations provide information on the depth and hydraulic properties of the sand aquifer in the vicinity of the site. Analysis and calculations for the dewatering are based on aquifer parameter values set out in Table 9.

Table 9 - Adopted Aquifer Parameters for Assessment of Groundwater Inflow

Parameter	**Adopted Value**	**Comment**
Hydraulic Conductivity (m/d)	10	Value consistent with literature and limited testing on site.
Specific yield (dimensionless)	0.15	Value considered reasonable for the sandy material recorded in borehole logs based on experience.
Saturated aquifer thickness (m)	50	Based on typical depth of strata and groundwater depth recorded on site. The greatest borehole depth of 38m did not encounter rock. For the purposes of this assessment an aquifer thickness of 50m was adopted.

Variations in aquifer thickness and properties occur and for this reason the parameter estimates for the site should be considered indicative. Actual properties encountered during dewatering may be significantly different from the estimates made from site investigation results.

7.6 Estimated Groundwater Inflows and Dewatering Volumes

Based on the assumed aquifer parameters, analysis was carried out using a numerical modelling tool LAMBS developed by Coffey for to assess dewatering measures to achieve construction dewatering and to assess potential impacts due to dewatering. The following assumptions have been made in the analysis:

- The Clarence River acts a boundary where groundwater levels are not affected by dewatering operations; and
- The aquifer is extensive with a uniform thickness.

Based on the number of proposed structures at the site it is anticipated that the various structures will be built progressively across the site. As such the size of excavations at the site will also vary depending on the size of the buildings. For the assessment of groundwater inflows, we have assumed that each building will be built separately and that dewatering will be required for each separate building.

For the purposes of assessment we have considered dewatering of an area of 65m by 30m at various locations on the site. This area is considered to be typical of the site of construction areas which would be active.

The rate of groundwater extraction is higher for dewatering sites closer to the Clarence River due to the increase in influence of river recharge. Table 10 sets out the modelled rates of groundwater extraction for two cases to address construction sites at distances of 40m and 200m from the Clarence River.

Further to this we have assumed that basement areas beneath buildings will be either (Case 1) 40m from the river or (Case 2) 200m. Table 10 lists the modelled dewatering flow rates for various times. An average groundwater inflow in the vicinity of 4000m^3/day (46 L/s) is assessed as being required in the first month to maintain the target drawdown for excavations 40m from the river. This reduces to 1900m^3/d (22L/s) for excavations 200m from the river. In each case the dewatering rate is expected to be relatively stable with time.

Table 10 - Assessed Inflow Rates For Adopted Aquifer Parameters

Time From Start Of Pumping (Days)	**Inflow - Construction Site 40m From River**	**Inflow - Construction Site 200m From River**
14	4000 m^3/d (46L/s)	1900 m^3/d (22 L/s)
30	4000 m^3/d (46L/s)	1900 m^3/d (22L/s)
90	4000 m^3/d (46L/s)	1800 m^3/d (20L/s)
180	4000 m^3/d (46L/s)	1800 m^3/d (20L/s)

These assessments depend strongly upon the aquifer parameter estimates, and variations in calculated inflows by a factor of two in either direction are considered possible.

The average rate of groundwater inflow required to achieve the required drawdown of approximately 1.5m range from about 4000m^3/d for excavations close (at about 40m distance) to the Clarence River to 1900m^3/d for excavations 200m from the Clarence River. Dewatering rates are expected to be reasonably stable with time. Inflows into the deeper basement and lift pit excavations will be higher than, but roughly of the same order of, those inflows indicated above.

The dewatering rate and volume extracted will depend upon the conditions present at the site. Rates would increase roughly in proportion to the permeability of the sand at the site should this exceed the assessed permeability value of 10m/d.

7.7 Dewatering Methods

The following dewatering methods are considered to be technically feasible to achieve 1.5m to 2m drawdown over the site during construction:

- Use of large diameter production bores;
- Use of spear point batteries – it is anticipated that one ring of spear points would need to be installed to achieve the required 1.5m to 2m of drawdown at individual construction sites.

Of these methods use of spear point systems is considered likely to be most effective given the modest dewatering requirements. Lines of spears at 1 to 2m centres around the perimeter of each excavation are assessed as sufficient to achieve drawdown of 1.5m to 2m. If large diameter wells were to be used it is expected that yields of 10L/s could be achieved. On this basis, it is considered that a total of six bores would be sufficient to provide construction dewatering for individual excavations.

The use of perimeter cut-off walls to prevent groundwater inflow during construction is considered impracticable on the site as such walls would need to penetrate to underlying low permeability material identified in drilling at a depth of 38m below ground level.

7.8 Water Quality and Method of Disposal of Pumped Water

Based on experience in a variety of council areas, we recommend that the policy adopted for the site should be that any excess water intended to be disposed to stormwater drains must satisfy the ANZECC (2000) guidelines for 95% protection of aquatic ecosystems. The general groundwater quality as indicated by laboratory analysis may be suitable for disposal to stormwater. This would require approval by Council.

It is noted that hydrocarbon contamination was identified in groundwater at the service station or the site possibly associated with leakage from buried fuel storage tanks. Groundwater affected by hydrocarbon contamination would require treatment to achieve ANZECC guideline water quality for protection of marine aquatic ecosystems prior to disposal to the stormwater system.

Options for disposal of excavation dewatering flows include:

- Treatment to achieve suitable water quality before discharge to the stormwater system;
- Disposal to sewer; or
- Re-injection into bores adjacent to the dewatering site.

If re-injection were adopted as the method for disposal, Council and Department of Natural Resources approval would be required. In addition, testing would need to be carried out to assess chemical suitability for injection. Groundwater dewatering rates would increase substantially if re-injection were undertaken. The increase in dewatering rate would depend upon the proximity of the re-injection system but flow rates would be expected to increase by at least a factor of two compared with the dewatering rate required in the absence of re-injection.

7.9 Impacts of Dewatering

7.9.1 Estimated Groundwater Drawdowns

Groundwater drawdown of approximately 1.5m to 2m will be required to facilitate construction of the proposed structures. In the absence of mitigating measures this would lead to localised lowering of the water table surrounding the site. Because of the close proximity of the Clarence River groundwater drawdown in the vicinity of dewatering operations is expected to stabilise within about one month of commencement of pumping.

Figure 4 shows modelled drawdown contours for dewatering of a 65m by 30m excavation centred 40m from the bank of the Clarence River. The drawdown at the excavation is assumed to be 1.5m. Drawdown impacts are contained largely within the site with off-site impacts limited to a drawdown of about 0.7m. Impacts on groundwater levels at the locations of registered bores are expected to minimal (less than 300mm).

Figure 5 shows modelled drawdown contours for dewatering of a 65m by 30m excavation centred 200m from the Clarence River. Drawdown impacts of up to 1m occur at the location of residences fronting Yamba Road. Impacts on registered bores in the area are again expected to be minimal (less than 500mm). Drawdowns of up to 1m are assessed likely to occur beneath dwellings on the southern side of Yamba Road near the site. At distances of 100m from the site boundary groundwater drawdown due to dewatering operations is not expected to exceed 0.7m.

With respect to the deeper basement excavation to -0.5m AHD, off-site groundwater drawdown is expected to be of the order of 25% higher than that indicated above.

7.9.2 Estimated Settlement Impacts Associated with Groundwater Drawdown

The assessment of potential ground surface settlements due to dewatering has been made using one-dimensional elastic theory. The analysis requires the assessment of the potential increase in effective stress within the compressible units, and an assessment of their stiffness (Young's modulus) due to applied loading.

Assuming a groundwater drawdown of 1.5m the presence of 5m of loose sand with a Young's Modulus of 10MPa settlement of the order of 10mm could be anticipated. Actual settlement will depend upon the nature of the soil profile from place to place.

This assessment assumes that the subsurface materials across the site to at least 20m depth comprises sand materials with no soft or firm clay materials which are subject to both load and time dependant settlement. Should soft clays be present it is possible that significantly larger settlement could occur.

Damage to surface structures including buildings can occur as a result of ground settlement.

While the assessed settlement values are considered unlikely to result in significant impact it is recommended that building condition surveys be carried out for properties within 100m of the site boundary and that a program of settlement and groundwater level monitoring be established to monitor settlement and drawdown in the vicinity of the site during dewatering operations. Should monitoring indicate development of excessive settlement mitigation measures in the form of groundwater injection in a trench or a line of wells along the Yamba Road site boundary would reduce the magnitude of off site drawdown and consequent settlement to the south of Yamba Road.

7.9.3 Potential for Building Damage due to Ground Movement (Settlement)

Damage to surface structures including buildings can occur as a result of ground settlement. In severe cases settlement can result in structural damage and prevent continued use of a building.

In the assessment of potential for damage to buildings and other structures arising from ground movement it is important to recognise that angular distortion and out of plane movement is more important than the magnitude of total settlement. Structures subjected to a uniform settlement are generally unaffected. Damage to buildings results from distortion caused by uneven ground movement. Recognising this fact, measures of acceptable settlement are generally framed in terms of angular distortion and horizontal strain or curvature (hogging or sagging).

Based on the relatively uniform subsurface conditions at the site (encountered in the boreholes) we consider that differential movements across the site that may affect the use of the structure are likely to be low. However, saying this, the most critical area for the potential of settlement for dewatering would be to the existing Moby Dick Motel to the east of the site. Based on the proposed site plans the proposed buildings will be about 10m to 15m from the existing Moby Dick Motel.

Table 11 sets out limits on angular distortion and horizontal strain nominated by a range of authors. These assessments generally identify angular distortion of 1/1000 as a value where ground movement is unlikely to result in damage to structures and numerous instances are noted in the literature where angular distortion greater than this value has not resulted in damage.

The Australian Standard AS2870-1996 (Residential Slabs and Footings – Construction), in the classification of damage to foundation movements, notes that local deviation of slope of more than 1/100 will normally be clearly visible and deviations in excess of 1/150 are undesirable. The Code calls for foundation designs to limit differential footing movement to 1/300 for clad frames and 1/2000 for full masonry buildings.

Assessments relating to horizontal stain generally relate to movements adjacent to deep excavations where horizontal tensile strain can develop or to horizontal strains developed as a result of mine subsidence where settlements of the order of one metre or more are not uncommon.

A range of structures surrounding the site are present including brick houses, blocks of flats of brick construction and commercial buildings of reinforced concentre construction. The most sensitive of these structures are considered to be clay brick buildings. Based on the literature review of settlement impact it is considered reasonable to assume that damage to surface structures would be nil to negligible for angular distortion less than 1/2000. This equates to a differential settlement of 10mm over a distance of 20m.

Table 11: Settlement Criteria

Source	Angular Distortion (δ/L)^{A}	Horizontal Strain (%)	Comment
Bjerrum, 1963	1/750 1/500 1/300		Sensitive industrial equipment Buildings - cracking not permissible First cracking of panel walls expected
Boscardin and Cording, 1989	1/1000^{b} 1/300 1/150		Negligible Slight Moderate
Polshin and Tokar, 1957		0.05	Brick buildings with length/height >3
Bozozuk, 1962		0.1	Clay brick with cement lime mortar
Littlejohn, 1974		0.02	Full scale field testing on brick walls
Polshin and Tokar, 1957	1/333 1/3333 1/2000 1/100		Slope of crane way Brick buildings (L/H^{b} ratios < 3) Brick buildings (L/H^{b} ratios > 5 Single storey mills
Skempton and MacDonald, 1956	1/150 1/500 1/1000		Limit to avoid severe damage Design limit to avoid impacts$^{(c)}$ Design limit for critical cases$^{(c)}$
Woodburn, 1976	1/2000 1/800 1/500 1/200		Solid masonry Articulated Masonry Brick Veneer Timber or pre-fabricated structures

Notes:
(a) δ = Differential settlement, L = length of building
(b) angular distortion in the absence of tensile strain
(c) L building length, H=building height
(d) Design values nominated by Skempton and MacDonald incorporate a safety factor to account for uncertainties in settlement assessment

Based upon the above discussion, the assessed values of settlement would be unlikely to result in significant damage for the assessed drawdown profile provided conditions are consistent with those assessed from site borehole records.

It is assessed, based on the above criteria, that settlement of between 10mm and 20mm (as predicted in nearby areas – see Section 7.6) would not be likely to result in unacceptable impacts and it is therefore considered that dewatering in the absence of mitigation measures would not be acceptable.

7.9.4 Estimated Impacts to Local Groundwater Users and the Environment

It is noted that groundwater dependent ecosystems such as the nature reserve to the west will be situated within the area of influence of the dewatering.

The nearest private registered groundwater bore is GW305448 located approximately 40m south of the site. Groundwater drawdowns of less than 500mm are assessed to result from construction dewatering at this location.

Given the depth of GW305448, no loss of bore performance is expected to occur.

7.10 Dewatering Risk Management Measures

The assessment of dewatering requirements for the site has identified that, in the absence of mitigating measures:

- Site dewatering would require substantial volumes of groundwater extraction (average inflow rates of 54L/s are assessed for excavations close to the Clarence River). The magnitude of dewatering flows together with possible water quality constraints may give rise to difficulties in disposal of dewatering flows.
- It is considered that there would be a risk of damage to buildings in the vicinity of the site as a result of construction induced drawdown.

It is therefore clear that mitigation measures would be required to control impacts on the surrounding area associated with construction dewatering. Mitigation measures to address the above issues are summarised in Table 12 below and discussed in the following paragraphs.

A geotechnical assessment of structures potentially affected by dewatering induced drawdowns could be conducted. The assessment would identify structures at higher risk from the proposed dewatering (i.e. the existing Moby Dick Motel directly to the east of the site. The assessment could potentially be used as a basis for remediation measures in the event of claims arising from the proposed dewatering. The cost of remediation to damaged structures is likely to be significant.

Construction of a partial cut-off wall could potentially reduce groundwater inflows, and if used in combination with groundwater re-injection could reduce the risk of damage to structures in the vicinity of the site. Given the significant saturated thickness at the site, further analysis would be required to provide a basis for design of a system of this kind.

Groundwater re-injection is a means of controlling groundwater drawdown by re-introducing dewatering flow back into the groundwater system. Based on the results of the site investigation, re-injection wells could be placed just inside the site boundary.

Groundwater monitoring should be conducted to assess the impacts of dewatering as construction of the basement progresses.

Table 12: Discussion of Mitigation Measures

Mitigation Measure	**Cost**	**Benefits**	**Comment**
None – dewater as required	Lowest cost	Low cost – technically straight forward	Disposal of groundwater discharge may be problematic. Significant risk of damage to structures in the vicinity of the site would be present.
Geotechnical risk assessment prior to dewatering	Low to moderate (depends on area and number of structures assessed) Would include review of historic groundwater levels.	Information on condition of structures prior to dewatering could be used to assist in assessing claims for dewatering settlement-induced damage.	Reactive measure. Cost of damage remediation potentially extremely high.
Curtain Walls in Combination with Reinjection	High (for construction of walls). Very high recharge system costs	Moderate risk to surrounding structures	A high level of control required for recharge system. Technical feasibility of reinjection has not been established. Reinjection wells would need to be located offsite Groundwater treatment prior to reinjection would be required.
Re-injection	High	Low to Moderate	A high level of control required for recharge system. Technical feasibility of re-injection has not been established. Re-injection wells would need to be located offsite. Groundwater treatment prior to re-injection may be required. Considered impractical for given site conditions.

Coffey is aware that the rainforest located to the north-west of the site will be sensitive to groundwater drawdown. To reduce the potential impacts of groundwater drawdown, mitigation measures including groundwater monitoring and re-injection along the north-western site boundary will likely be required. On the basis of our understanding of the conditions evident at the site, it is considered that a scheme of re-injection possibly involving absorption trenches and/or wells can be developed to effectively reduce the effects of groundwater drawdown outside the site boundaries. The effects of the scheme would be to increase the groundwater turnover through the dewatering system, which would have to be taken into account in system design. The scheme would also ideally involve dewatering and potentially trialling recharge systems at locations on the site sufficiently away from the rainforest area, thus allowing consideration of the effects prior to dewatering within the more sensitive areas. Monitoring would need to provide real-time measurement of drawdown during dewatering, thus allowing changes to the process to be adopted during the works.

8 CONCLUSIONS AND RECOMMENDATIONS

A program of groundwater investigation has been carried out comprising:

- Drilling of eight boreholes to various levels;
- Limited aquifer testing using falling head testing;
- Construction of on-site monitoring boreholes;
- Initial appraisal of groundwater inflow rates and potential drawdown impacts.

Drilling results indicate that the site is underlain by sand. Bedrock was not observed in any of the boreholes. No soft to firm clay layers were observed during drilling which, if present, can dramatically increase settlements.

Key assessments from the groundwater investigation include:

- Significant flows would be required for dewatering of the proposed excavation. The rate of groundwater inflow to achieve the proposed drawdown of 1.5m during construction is estimated to be approximately 4000m^3/d for excavations close to the Clarence River reducing to about 1900m^3/d for excavations near Yamba Road at the western end of the site. Inflow assessment is influenced by the choice of permeability values and could vary by a factor of two in either direction from the assessed values.
- A groundwater level reduction of 1.5m in the excavation would lead to lowering of the water table outside the site in areas to the east of the site. A drawdown of about 1m is assessed at the site boundary, falling to 0.5m at a distance of 250m from the excavation location.
- Based on the results of site investigations, use of a spear point battery or six deep dewatering bores of at least 250mm diameter could be used for dewatering of each excavation.
- In the absence of mitigation measures, no significant groundwater drawdown is assessed for private water bore GW305448 (or other bores which are further from the site). A groundwater dependent ecosystem is identified to the west within the vicinity of the site. Groundwater levels at the bank of the Clarence River will be maintained by recharge from the river and no significant changes in groundwater level are expected.

- In the absence of mitigation measures, settlement outside the site boundary to the east would result from construction dewatering. The magnitude of settlement is assessed as potentially exceeding 10mm. Significantly larger settlement could occur if unfavourable ground conditions are present beneath existing structures.

In the absence of mitigation measures, drawdown and settlement would potentially impact an area within 200m of the site boundary. It is assessed that groundwater impacts in the absence of mitigation measures may be unacceptable and therefore it is recommended that mitigation measures be established as a contingency to control groundwater level reduction outside the site boundary.

The information in this report provides a basis for design of groundwater control measures to be employed during construction of the proposed two-level basement. Groundwater management and control measures may include all or some of the following:

- Geotechnical assessment of structures potentially affected by settlement prior to dewatering;
- Groundwater injection along the site boundary;
- Curtain walls around excavation with re-injection of groundwater outside of the curtain wall;
- Settlement monitoring; and
- Groundwater monitoring.

Design of groundwater control measures would need to be carried out in conjunction with the design of basement excavation shoring. This is likely to involve some form of low permeability wall restrained with ground anchorages. Groundwater dewatering would be achieved through the use of large diameter bores or spear point batteries. Recharge of the groundwater system is likely to be required though the level and nature of recharge will be linked to the shoring design.

Results of groundwater testing reported in an investigation of the site service station revealed the presence of petroleum hydrocarbons. Petroleum hydrocarbons in groundwater pumped from the excavation at the site will require treatment prior to discharge to stormwater or to injection systems. Alternatively extracted groundwater affected by hydrocarbon contamination could be collected and disposed to a licensed waste water facility.

It may be possible to discharge contaminated groundwater to sewer. This would be subject to approval by the responsible authority. A re-injection system would need to include groundwater treatment to reduce the potential for aquifer clogging and such treatment could include reduction of hydrocarbons.

It is recommended that:

- Design of dewatering systems integrated with basement / foundation excavation and construction be undertaken as part of the detailed design;
- Continuous monitoring of groundwater levels be undertaken in the on-site piezometers up to the beginning of construction works (collected data from the data loggers should be analysed to assess the response of the groundwater table over time, particularly with respect to rainfall); and
- A groundwater monitoring plan be developed for the construction phase this would include:
    - monitoring of onsite piezometers;
    - installation and monitoring of additional off-site piezometers to track dewatering impacts;

- monitoring of dewatering volumes and water quality; and
- establishment and regular monitoring of settlement at the site boundary and at various points beyond the site boundary to provide a basis of assessment of settlement impacts and allow timely intervention if needed.

Control of the groundwater dewatering operation will be essential. It is possible for drawdown well in excess of construction requirements to occur if monitoring and control is inadequate. This could have the result of causing off-site drawdown impacts well in excess of those assessed in this report. For this reason it is recommended that involvement of a groundwater professional occur during the construction works and that regular review of monitoring results be carried out by a groundwater professional.

We consider that the further investigation works recommended in this report 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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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.

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.

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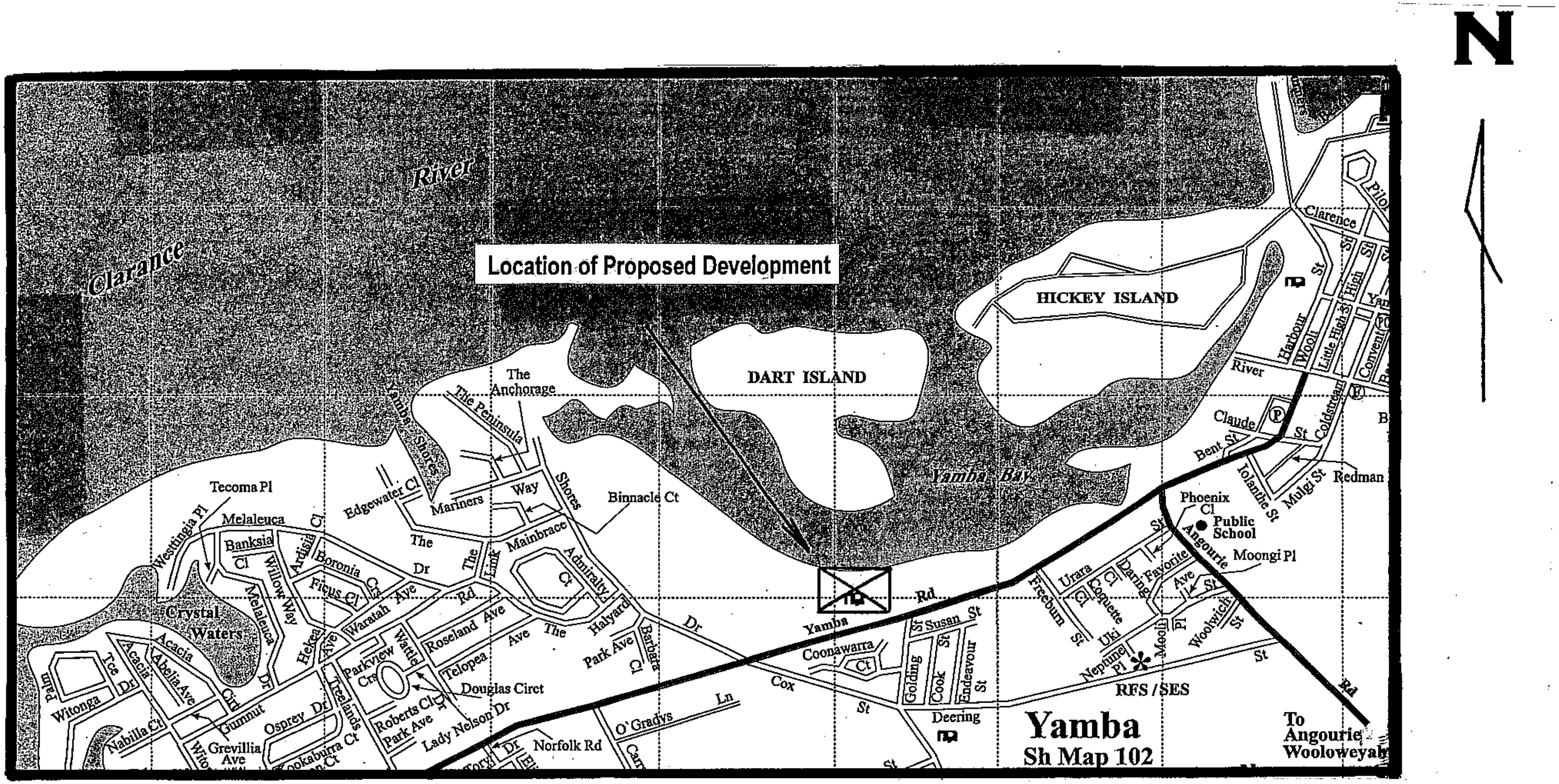
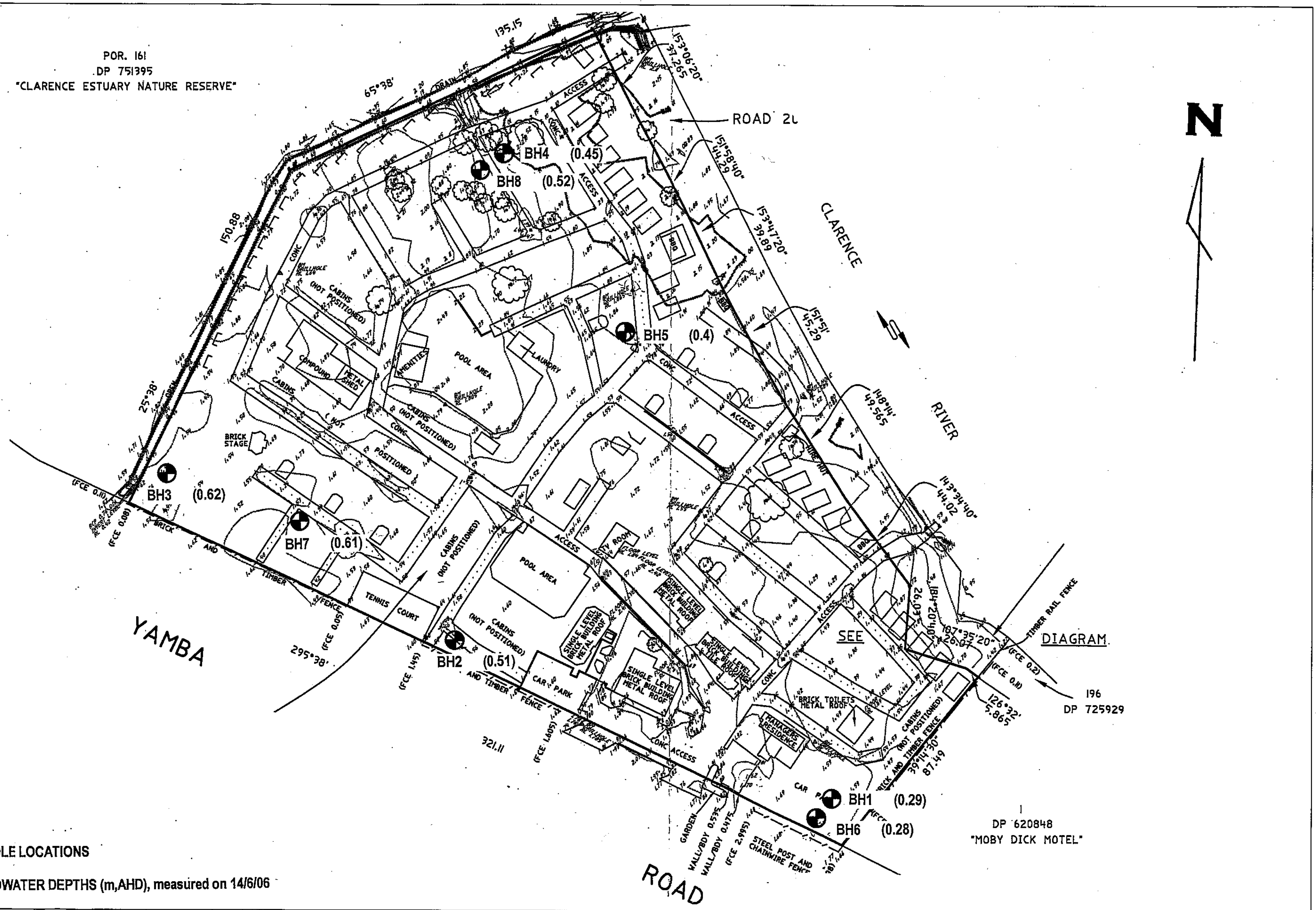
* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.

Figures



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

coffey
geotechnics
SPECIALISTS MANAGING
THE EARTH



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

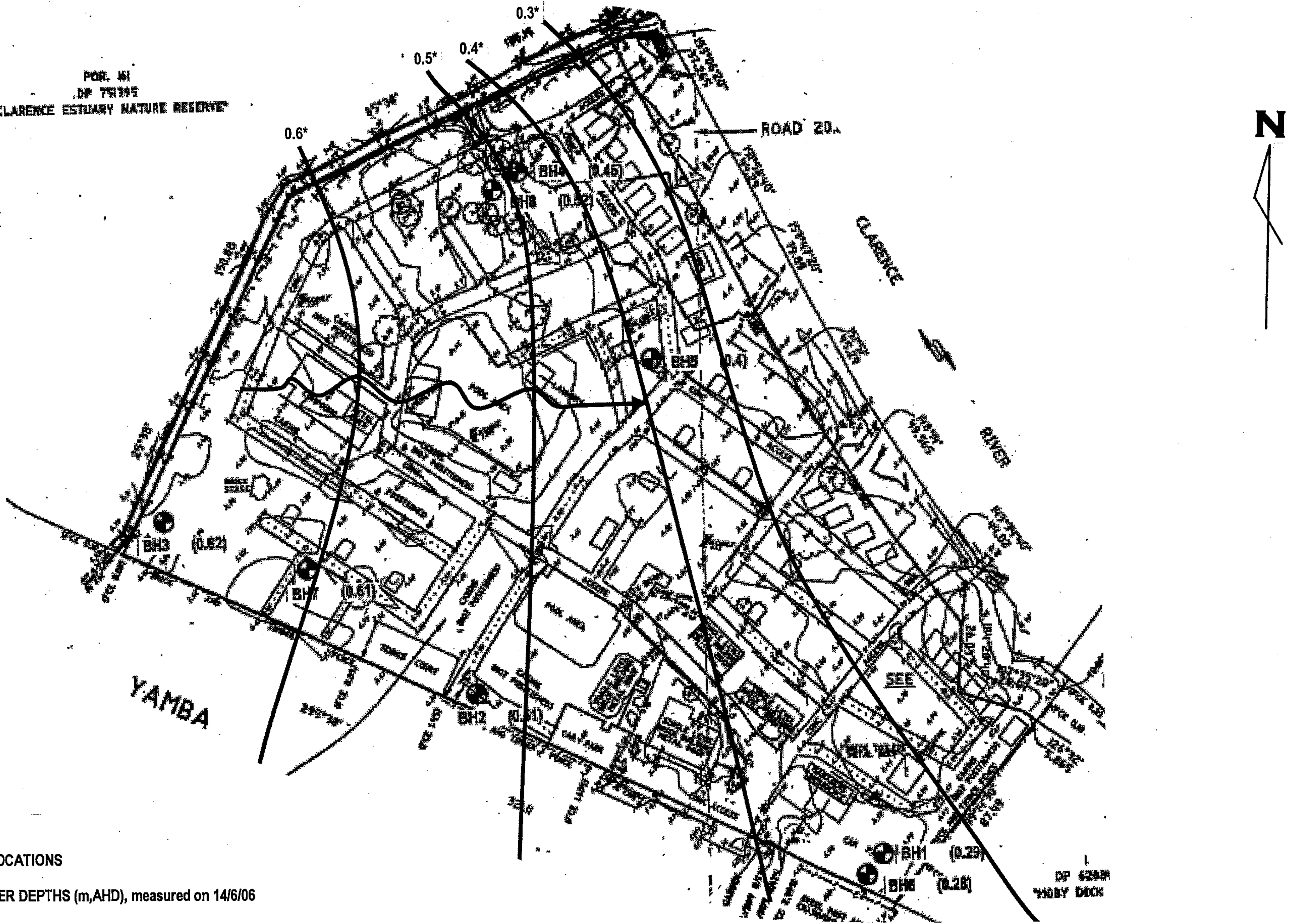
client: JONES LANG LASALLE PTY LTD

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

coffey geotechnics
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

LEGEND

◆	Registered Groundwater Bore Location
⁄	Modelled Drawdown Contour (0.5m contours)

drawn:	DJB	client:	JONES LANG LASALLE PTY LTD
approved:	DJB	project:	DOLPHIN BLUE DEVELOPMENT YAMBA ROAD, YAMBE
date:	18/10/06		
scale:	NTS	title:	ASSESSED DRAWDOWN CONTOURS – CASE 1
original size:	A4	project no.:	GEOTCOFH01613AA
		figure no.:	FIGURE 4

coffey geotechnics
SPECIALISTS MANAGING
THE EARTH

LEGEND

◆ Registered Groundwater Bore Location

— Modelled Drawdown Contour (0.5m contours)

drawn:	DJB	coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	JONES LANG LASALLE PTY LTD
approved:	DJB		project:	DOLPHIN BLUE DEVELOPMENT YAMBA ROAD, YAMBE
date:	18/10/06			
scale:	NTS		title:	ASSESSED DRAWDOWN CONTOURS – CASE 2
original size:	A4		project no.:	GEOTCOFH01613AA
			figure no.:	FIGURE 5

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.

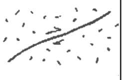
Soil Description Explanation Sheet (2 of 2)

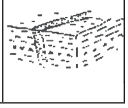
SOIL CLASSIFICATION INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 60 mm and basing fractions on estimated mass)							USC	PRIMARY NAME
COARSE GRAIINED 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**		
		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.	

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

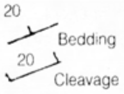
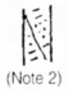
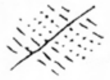
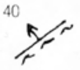
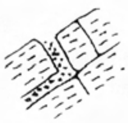
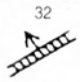
Term	Abbrev-iation	Point Load Index, I_S50 (MPa)	Field Guide
Very Low	**VL**	Less than 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; pieces up to 30mm thick can be broken by finger pressure.
Low	**L**	0.1 to 0.3	Easily scored with a knife; indentations 1mm 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.

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.

coffey geotechnics

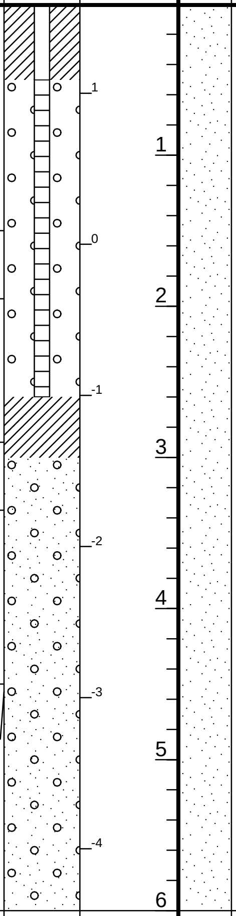
Engineering Log - Piezometer

Borehole No.	**BH1**
Sheet	1 of 1
Project No:	**CH1613/1**
Client:	**RIDER HUNT TEROTECH**
Principal:	
Project:	**PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT**
Borehole Location:	**REFER TO FIGURE 1**
Date started:	**9.5.2006**
Date completed:	**9.5.2006**
Logged by:	**ELC**
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	depth metres	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.
					0.1			M		
					0.7		Colour change to pale brown/brown.			
					1.0		Colour change to pale grey.		MD/D	
			11.30 11/5/06		1.2					
				SPT 2,4,7 N*=11	1.5			W		
					2.5				D	
				SPT 3,9,10 N*=19	3.0					
					3.25		Indurated layer at 3.25m to 3.4m, dark brown. shell fragments observed. Colour change to grey.			
					4.0				VD	
				SPT R N=R	4.4		Indurated layer at 4.4m-4.8m.			SPT carried out with solid cone. 25 blows for 50mm penetration.
					5.0		Thin bands of indurated sand between 4.8m and 6m			
					6.0		End of BH1 due to limit of required investigation. Borehole terminated at 6m			

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

Engineering Log - Piezometer

Borehole No.	**BH2**
Sheet	1 of 1
Project No:	***CH1613/1***
Client:	***RIDER HUNT TEROTECH***
Principal:	
Project:	***PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT***
Borehole Location:	***REFER TO FIGURE 1***
Date started:	***9.5.2006***
Date completed:	***9.5.2006***
Logged by:	***ELC***
Checked by:	

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								SAND: fine to medium grained, dark brown	D/M	L	ALLUVIAL SOIL Roots to 0.05m.
						1				Colour change to pale brown/brown.			
							1						
												L/MD	
			▼ 1.50PM 9/5/06							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							
							6			Indurated dark brown sand.			
										END of BH2 at 6m due to limit of required investigation. Borehole terminated at 6m			
						-5							
							7						
						-6							
							8						

method

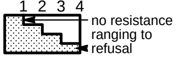
AS	auger screwing*
AD	auger drilling*
RR	roller/tricone
W	washbore
CT	cable tool
DT	diatube
B	blank bit
V	V bit
T	TC bit
TBX	Tubex

*bit shown by suffix
e.g. ADT

support

C casing N nil

penetration



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

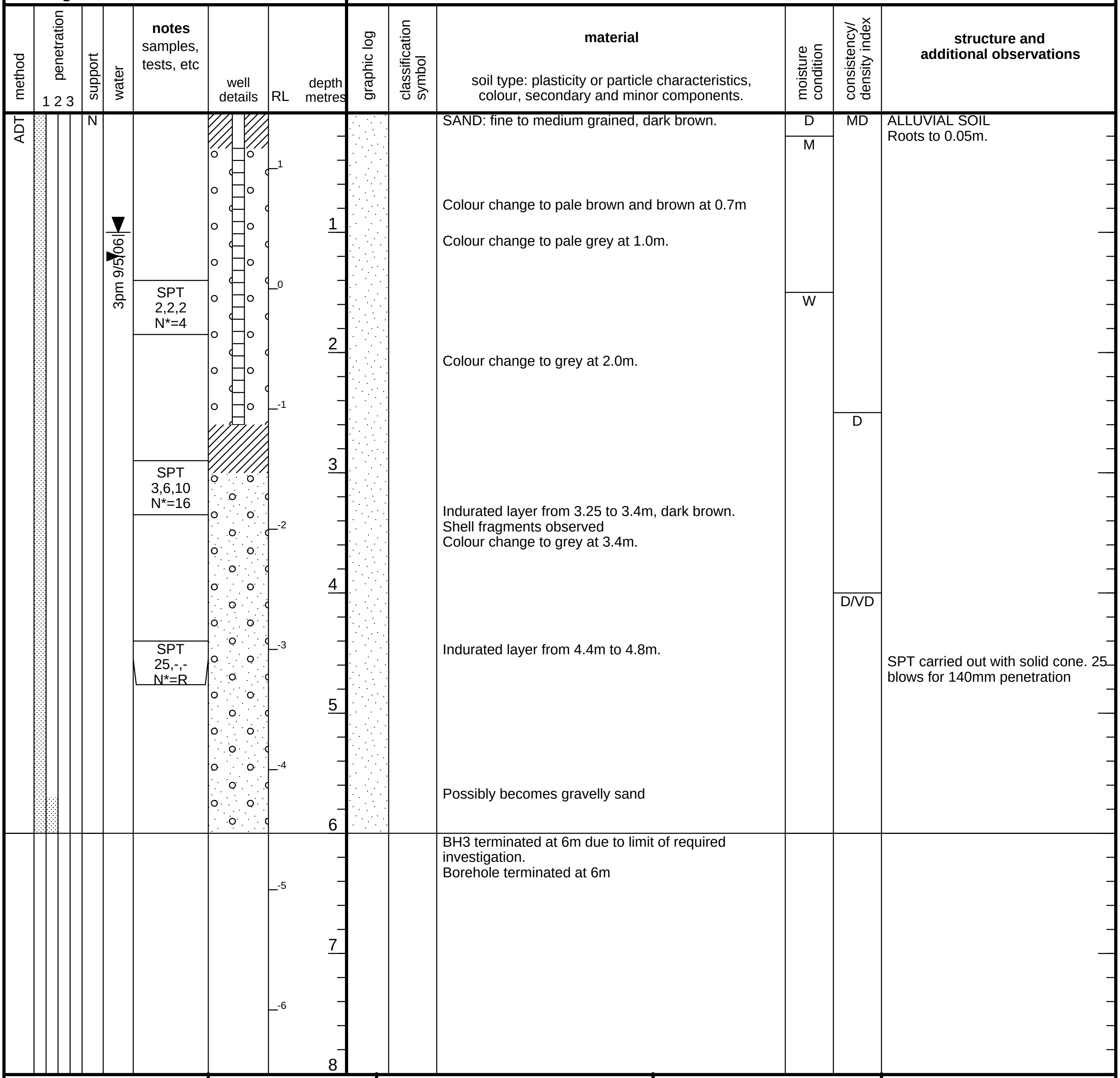
VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

Engineering Log - Piezometer

Borehole No.	***BH3***
Sheet	1 of 1
Project No:	***CH1613/1***
Client:	***RIDER HUNT TEROTECH***
Principal:	
Project:	***PROPOSED DEVELOPMENT - BLUE DOLPHIN RESORT***
Borehole Location:	***REFER TO FIGURE 1***
Date started:	***9.5.2006***
Date completed:	***9.5.2006***
Logged by:	***ELC***
Checked by:	

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**



SAND: fine to medium grained, dark brown.

Colour change to pale brown and brown at 0.7m

Colour change to pale grey at 1.0m.

Colour change to grey at 2.0m.

Indurated layer from 3.25 to 3.4m, dark brown.
Shell fragments observed
Colour change to grey at 3.4m.

Indurated layer from 4.4m to 4.8m.

Possibly becomes gravelly sand

BH3 terminated at 6m due to limit of required investigation.
Borehole terminated at 6m

ALLUVIAL SOIL
Roots to 0.05m.

SPT carried out with solid cone. 25 blows for 140mm penetration

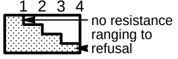
method

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



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

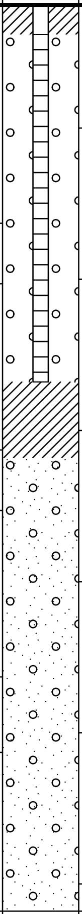
coffey geotechnics

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	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, brown to dark brown.	D	MD	ALLUVIAL SOIL
										M		
					1	1			Colour change to pale grey at 0.8m.		D	
			11:20am 11/5/06							W		
				SPT 2,5,9 N*=14								
					0	2					MD	
					-1	3						
				SPT 1,2,6 N*=8					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

coffey geotechnics

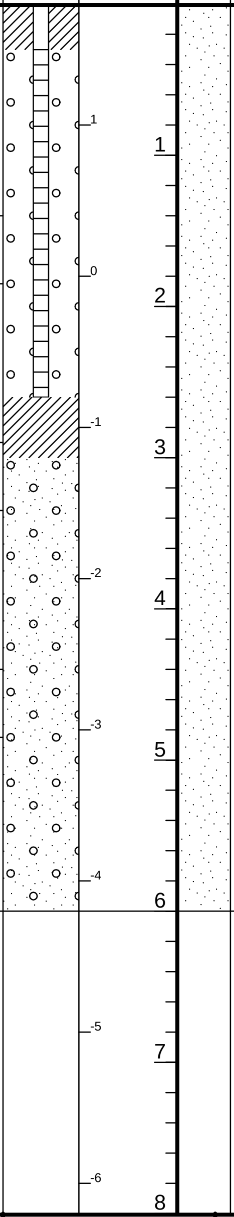
Engineering Log - Piezometer

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

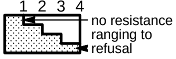
drilling information | **material substance**

method	penetration 1 2 3	support	water	notes samples, tests, etc	depth metres	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
					0.4	Colour change to pale brown at 0.4m.			
					1.2	Colour change to grey at 1.2m.	W		
			12;10 11/5/06	SPT 1,0,2 N*=2	1.5–1.95				
					2.2			MD	
				SPT 2,2,5 N*=7	3.0–3.45	Numerous shell fragments throughout profile.			
					4.0			D/VD	
				SPT 6,13,17 N*=30	4.5–4.95				
					6	BH5 terminated at 6m due to limit of required investigation. Borehole terminated at 6m			



method		support		notes, samples, tests		classification symbols and soil description	consistency/density index	
AS	auger screwing*	C casing	N nil	U_{50}	undisturbed sample 50mm diameter	based on unified classification system	VS	very soft
AD	auger drilling*			D	disturbed sample		S	soft
RR	roller/tricone			N	standard penetration test (SPT)		F	firm
W	washbore			N*	SPT - sample recovered		St	stiff
CT	cable tool			Nc	SPT with solid cone		VSt	very stiff
DT	diatube			P	pressure meter		H	hard
B	blank bit			Bs	bulk sample		Fb	friable
V	V bit			R	refusal		VL	very loose
T	TC bit			E	environmental sample		L	loose
TBX	Tubex			PID	PID measurement		MD	medium dense
*bit shown by suffix				WS	water sample		D	dense
e.g.	ADT			PZ	piezometer		VD	very dense
				ALT	air lift test			

penetration



water

▼ 10/1/98 water level on date shown

► water inflow

◄ water outflow

moisture

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

coffey geotechnics

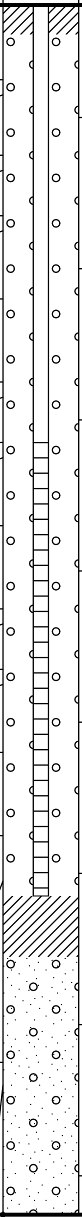
Engineering Log - Piezometer

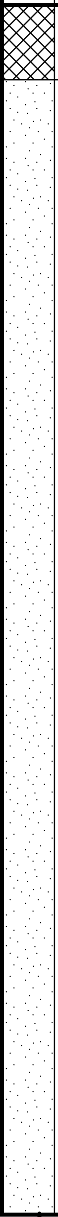
Borehole No. **BH6**
Sheet 1 of 4
Project No: **CH1613/1**

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

coffey geotechnics

Borehole No.	**BH6**
Sheet	2 of 4
Project No:	**CH1613/1**

Engineering Log - Piezometer

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	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
WB		M								**SAND:** fine to medium grained, brown *(continued)*	W	VD	
				SPT R,-,- N*=R		-7							SPT 31 blows for 125mm penetration.
							9						
						-8							
							10						
				SPT R,-,- N*=R									SPT 32 blows for 150mm penetration.
										Colour change to brown at approximately 10.5m.			
						-9							
							11						
				SPT 26,R,- N*=R		-10							SPT 26 blows for 150mm penetration.
							12						
						-11							
							13						
				SPT 25,R,- N*=R									SPT 25 blows for 85mm penetration.
						-12							
							14						
				SPT 28,R,- N*=R		-13							SPT 28 blows for 150mm penetration.
							15						
						-14							
							16						

method

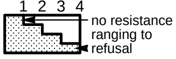
AS	auger screwing*
AD	auger drilling*
RR	roller/tricone
W	washbore
CT	cable tool
DT	diatube
B	blank bit
V	V bit
T	TC bit
TBX	Tubex

*bit shown by suffix
e.g. ADT

support

C	casing	N	nil

penetration



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