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ABN 64 002 841 063

Job No: 11201/1
Our Ref: 11201/1-AA
18 September 2006

Beach Property Trust
c/- Bonus & Associates Architects
Level 1
597 Darling Street
ROZELLE NSW 2039

Attention: Mr T West

Dear Sir

re: **Proposed Mixed Development**
1184-1186 Pittwater Road, corner of Clarke Street, Narrabeen
Geotechnical Investigation

This report details the results of a geotechnical investigation for a proposed mixed development located at 1184 – 1186 Pittwater Road, Narrabeen. The investigation was commissioned by Mr T West in a letter dated 31 August 2006 and was conducted in general accordance with the Geotechnique fee proposal (Q3311-Revised) dated 25 August 2006 and subsequent email from Mr S Branch of Woolacotts Consulting Engineers.

A separate acid sulphate soils assessment report will be submitted after completion of laboratory testing.

PROPOSED DEVELOPMENT

It is understood that the proposed development will include demolition of the existing residence and construction of a two-three storey mixed-use development with a single basement car park to RL3.8. Excavation to 3m is expected.

REGIONAL GEOLOGY

The Sydney (1:100,000) Soil Landscape Map indicates that the landscape at the site belongs to the Newport Group, which is characterised by gently undulating plains, to rolling rises of Holocene sands, mantling other soil materials or bedrock, with local relief of less than 10m and ground surface slope less than 10% on lower slopes and up to 35% against obstacle facing prevailing winds. The sub-surface soils are usually shallow to moderately deep (0.5-1.5m), non-cohesive and susceptible to high erosion hazard.

Reference to the Sydney (1:100,000) Geological Map Sheet indicates that the site is underlain by Quaternary, medium to fine grained, marine sand deposits and quartz sand with minor shell and silt, belonging to a beach ridge system.

Geotechnique has completed a few projects in the local area, which included borehole drilling and insitu testing. Information from the investigations indicated marine sands to depths in excess of 10m.

SITE DESCRIPTION

The site is trapezoidal in plan dimensions, measuring about 30m along Pittwater Road by approximately 55m deep and is bound by Clarke Street (north), Pittwater Road (west), double storey units (south) and the ocean (east).

At the time of the investigation, the site contained a double storey residence in the western portion, with the remainder of the site being relatively flat and partially grass-covered.

GEOTECHNICAL INVESTIGATION
The following scope of work was completed:

- Due to the likely presence of underground services, an underground utility check was carried out as part of Geotechnique Pty Ltd OH&S Site Risk Assessment.
- A specialist services locator was hired to scan proposed testing locations for underground services.
- Drilling at three locations (BH1, 2 & 3), using a utility mounted drilling rig fully equipped for geotechnical investigations. Drilling was located in accessible areas and taken to depths of about 8.5m. The approximate borehole locations are shown on the attached Drawing 11201/1-1.
- The boreholes were located at approximately equal distances surrounding the proposed development, as requested in Woolacotts email dated 4 September 2006.
- Standard Penetration Tests (SPT) were carried out at regular depth intervals during drilling to estimate strength characteristics of the sub-surface soils.
- Open standpipe piezometers were installed at two locations (BH1 & BH2) to allow for future groundwater monitoring.
- Recovered sub-surface soils were sent for acid sulphate soils testing (Refer Report 11201/1-AB).

The field work was carried out on 6 September 2006 and supervised by an Engineering Geologist from this company, who was responsible for confirming borehole locations, conducting OH&S, supervising testing and recovering soils samples.

Results of Borehole Drilling
The detailed engineering logs are attached and revealed the following generalised profile:

Topsoil/Fill	Sand, fine grained, brown, with root and root fibres, to depths of 0.3m, underlain by
Beach sands	Sand, fine to medium grained, pale brown and yellow to termination depths

Results of SPT Testing
The detailed SPT test results are shown on the attached engineering logs (as N-value) and summarised below:

Depth (m)	BH1	BH2	BH3	Estimated Average Relative Density	Estimated Unit Weight (kN/m^3)	Estimated Friction Angle (degrees)
0.5	6	5	4	Medium Dense	18	32
2.0	8	7	9	Medium Dense	18	32
3.5	9	28*	6	Medium Dense	18	32
5.0	15	46*	25	Medium Dense	18	32
6.5	28	37*	25	Dense	18	36
8.0	32	31	31	Dense	18	36

* BH2 encountered a layer of very dense sand between about 3m and 7m

Groundwater Conditions
Groundwater was encountered in all boreholes at the following depths:

Groundwater Depth (m)	BH1	BH2	BH3
06.09.06	6.0	5.4	6.0
14.06.06	5.75	6.0	*

* No groundwater well installed

It should be noted that fluctuations in the level of groundwater and/seepage might occur due to variations in rainfall and/or other factors.

DISCUSSION & RECOMMENDATIONS

Excavation Conditions

The investigation encountered sandy material to termination depths. These materials are readily, excavated using conventional earthmoving equipment.

Groundwater is considered to be about 3m below proposed bulk excavation level of 3m from existing surface levels and so, we do not anticipate significant groundwater inflow into the excavation. Groundwater inflow, if any, would be adequately handled by a conventional sump and pump system.

Trafficability problems might arise locally during wet weather, or if water is allowed to pond on these materials. Over-excavation by about 300mm and replacement with compacted crushed granular material, such as crushed recycled concrete, is recommended.

Dilapidation Survey

Prior to demolition and excavation, the Structural Engineer should conduct a dilapidation survey on the adjoining double storey units on the southern boundary.

Batter Slopes

All temporary excavation faces should be battered for short-term stability. A maximum slope in the order of 1.6 Horizontal (H) to 1 Vertical (V) is recommended, which corresponds to the estimated angle of repose of about 32 degrees.

Surface protection of the cut slope can be provided by shotcreting. Alternatively, batters can be protected using plastic sheeting, providing an Engineering Geologist/Geotechnical Engineer inspects the batters.

Retaining Structures

If the recommended safe batters are not feasible due to space limitations, then engineered designed retaining structures should be used. An appropriate retaining structure for the proposed excavation could comprise a grout injected contiguous pile wall, designed as a cantilever wall or anchored wall. The piles should be drilled prior to excavation and taken to below bulk excavation level. Ground anchors might have to be used to reduce the load on the piles.

The pressure distribution on cantilever walls is assumed to triangular and estimated as follows:

$$p_h = \gamma kH + kq$$

Where,

p_h = Horizontal pressure (kN/m^2)
γ = Wet density of retained materials (estimated to be $18kN/m^3$)
k = Coefficient of earth pressure (k_a or k_o)
H = Retained height (m)
q = Surcharge load (kN/m^2)

Should the retaining structures be anchored or strutted, the earth pressure distribution is assumed to be rectangular and estimated as follows:

$$p_h = 0.65\gamma Hk + kq$$

Where,

p_h = Horizontal pressure (kN/m^2)
γ = Wet density of retained materials (estimated to be $18kN/m^3$)

For design of flexible retaining structures, where some lateral movement is acceptable, an active earth pressure coefficient is recommended. If it is critical to limit the horizontal deformation of a retaining structure, use of an earth pressure coefficient at rest should be considered. Recommended parameters for the design of retaining structures are as follows:

- Unit weight 1.8 t/m^3
- Coefficient of Active earth pressure (k_a) 0.32
- Coefficient of At Rest earth pressure (k_o) 0.47
- Coefficient of Passive earth pressure (k_p) 3.12

The above coefficients assume that ground level behind the retaining structures is horizontal and the retained material is effectively drained. If retained soil is subject to groundwater pressure, additional earth pressure resulting from groundwater should be allowed for in the design.

The design of any retaining structure should be checked for bearing capacity, overturning, sliding and overall stability.

Rock Anchors
It is likely that the retaining walls of the basement excavation could require anchorage or tie-back in order to resist lateral pressure. The allowable bond stress for use in the anchorage design may be taken as 25kPa. Permission should be sought from adjacent property owners if anchors extend beneath these properties.

Floor Slabs
Floor slabs may be designed for an estimated Modulus of Subgrade Reaction of 25kPa/mm. The design of slabs does not need to allow for uplift pressure from the groundwater, unless required by the Hydraulic Engineer.

Footings & Estimated Settlements
Loading conditions for the proposed development are not known at this stage. However, we anticipate that shallow footings (pad and strip) founded in the exposed medium dense sands would be appropriate.

Shallow footings (from bulk excavation levels) can be used in areas where the depth to founding stratum is less than about 1.5m from the excavated basement level. In areas where the founding depth is in excess of 1.5m, excavation of shallow footings would be difficult and therefore, deep footings would be appropriate.

Should footings be founded above and within the 1H to 1V plane projected from the base of excavation, the allowable bearing pressures are half of the above recommended value.

The recommended allowable bearing pressures are given in the following table:

	Assumed 1m rigid square footing*	Assumed 500mm wide strip footing*
Allowable bearing pressure (kPa)	200	200
Anticipated settlement (mm)	5 - 15	3 - 10

* Assumed footing depth of 500mm

Alternatively, grout injected piles may be used to depths above the anticipated groundwater level of 6m. A typical 500mm diameter pile taken to 6m depth would have an overall capacity of about 50 tonnes, with estimated settlement in the range of 5mm to 10mm.

Limitations

The sub-surface profile presented in this report is based on information obtained from three boreholes drilled at accessible locations. Actual sub-surface conditions across the site and between boreholes might differ or vary from those expected (interpreted).

The recommendations presented in this report are based on the expected (interpreted) sub-surface profile. The scope of this investigation did not include assessment of footing configurations and founding conditions of neighbouring buildings.

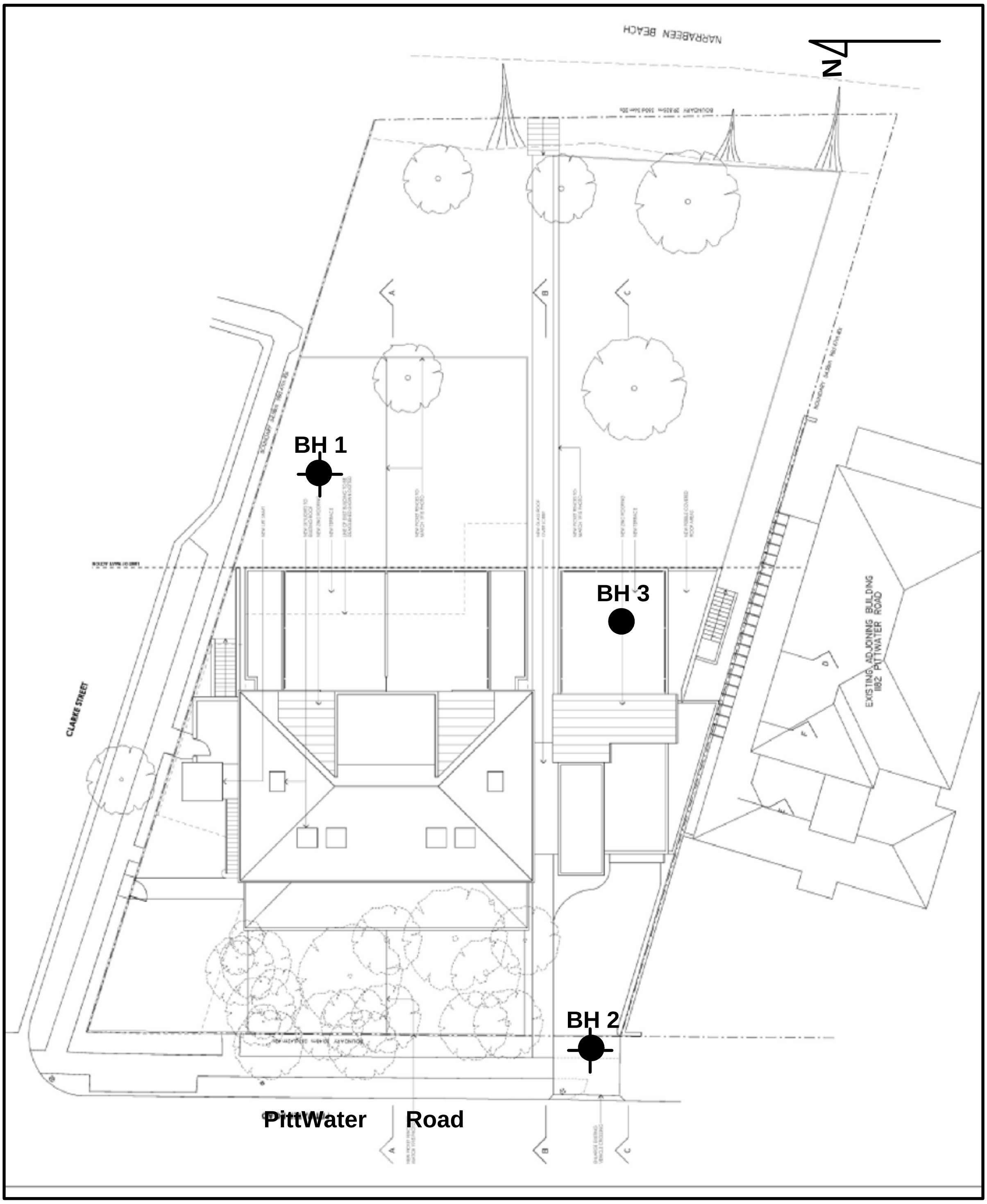
The report contains geotechnical parameters to be used as input for the structural design of footings, retaining walls, etc.

Should you have any questions relating to this report, please do not hesitate to contact the undersigned.

Yours faithfully
GEOTECHNIQUE PTY LTD

EMGED RIZKALLA
Principal Geotechnical Engineer

Encl Location Plan – Drawing No 11201/1-1
Engineering Borehole Logs
Explanatory Notes



LEGEND

- Boreholes With Groundwater Monitoring Well
- Borehole

PREPARED BY:

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Bonus & Associates Architects
Proposed Mixed Development
1184-1186 Pittwater Road,
Narrabeen

Borehole & Monitoring Well Location Plan

Drawing No: 11201/1-1
Job No: 11201/1
Drawn By: ZM
Scale: 1: 350
Date: 08/09/2006
Checked By: ZM

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engineering log - borehole

Client : Bonus & Associates Architects
Project : Proposed Mixed Development
Location : 1184-1186 Pittwater Road, Narrabeen

Job No. : 11201/1
Borehole No. : 1
Date : 6 September 2006
Logged/Typed/Checked by: ZM/ac/

drill model and mounting : XP60 **slope :** 90° **deg.** **R.L. surface :** -
hole diameter : 125 **mm** **bearing :** - **deg.** **datum :** -

method	groundwater	samples	field test	depth or R.L. in meters	graphic log	classification symbol	MATERIAL DESCRIPTION soil type, plasticity or particle characteristic, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa	Remarks and additional observations
TC-Bit				0			FILL: Sand, fine grained, brown, with glass and ceramic fragments	D			Fill appears loosely compacted
		DS	N=6 2,2,4			SP	SAND: Fine to medium grained, pale brown	D	L		
				1		SP	SAND: Fine to medium grained, pale yellow, brown	D	L		
		DS	N=8 2.3,5	2							
				3		SW	SAND: Fine to coarse grained, pale yellow, brown, some shell fragments	M	MD		
								W	D		
		DS	N=9 3,4,5	4							
		DS	N=15 3,5,10	5							
	▼			6							More shell fragments from 6.0m Stand pipe installed to 6.2m
		DS	N=28 3,7,21	7							
		DS	N=32 5,8,23	8			Borehole No 1 terminated at 8.0m				
				9							

form no. 002 version 02 - 11/04

engineering log - borehole

Client : Bonus & Associates Architects
Project : Proposed Mixed Development
Location : 1184-1186 Pittwater Road, Narrabeen

Job No. : 11201/1
Borehole No. : 2
Date : 6 September 2006
Logged/Typed/Checked by: ZM/ac/

drill model and mounting : XP60 **slope :** 90° **deg.** **R.L. surface :** -
hole diameter : 125 **mm** **bearing :** - **deg.** **datum :** -

method	groundwater	samples	field test	depth or R.L. in meters	graphic log	classification symbol	MATERIAL DESCRIPTION soil type, plasticity or particle characteristic, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa	Remarks and additional observations
TC-Bit				0			TOPSOIL: Sand, fine grained, brown, roots and root fibres	D			
						SW	SAND: Fine to medium grained, pale yellow, brown	D	L		
		DS	N=5 2,2,3								
				1							
				2							
		DS	N=7 2,3,4								
				3							
						SW	SAND: Fine to medium grained, pale yellow, with shell fragments	M	MD		
		DS	N=28 9,15,13					W	D		
				4							
				5							
	▼	DS	N=46 13,16,30								
				6							Stand pipe installed to 6.4m
		DS	N=37 9,15,22								
				7							
				8							
							Borehole No 2 terminated at 8.45m				
		DS	N=31 10,15,16								
				9							

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engineering log - borehole

Client : Bonus & Associates Architects
Project : Proposed Mixed Development
Location : 1184-1186 Pittwater Road, Narrabeen

Job No. : 11201/1
Borehole No. : 3
Date : 6 September 2006
Logged/Typed/Checked by: ZM/ac/

drill model and mounting : XP60 **slope :** 90° **deg.** **R.L. surface :** -
hole diameter : 125 **mm** **bearing :** - **deg.** **datum :** -

method	groundwater	samples	field test	depth or R.L. in meters	graphic log	classification symbol	MATERIAL DESCRIPTION soil type, plasticity or particle characteristic, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa	Remarks and additional observations
TC-Bit				0			TOPSOIL: Sand, fine grained, brown				
						SP	SAND: Fine to medium grained, pale brown	D	L		
		DS	N=4 2,1,3								
						SP	SAND: Fine to medium grained, pale yellow, brown	D	L		
				1							
				2							
		DS	N=9 3,4,5								
				3		SW	SAND: Fine to coarse grained, pale yellow, brown, with some shell fragments	M	MD		
								W	D		
		DS	N=6 1,2,4								
				4							
				5							
		DS	N=25 5,11,14								
	▼			6							
		DS	N=25 4,10,15								
				7							
				8							
		DS	N=31 6,15,16								
							Borehole No 3 terminated at 8.45m				
				9							

KEY TO SYMBOLS

Symbol Description

Strata symbols

Fill

Sand

Topsoil

Misc. Symbols

Groundwater encountered during drilling

Notes:

1. Exploratory borings were drilled on 6 September 2006 using a 125mm diameter continuous flight power auger.

2. Boring locations were taped from existing features and elevations extrapolated from the final design schematic plan.

3. These logs are subject to the limitations, conclusions, and recommendations in this report.

4. Results of tests conducted on samples recovered are reported on the logs.

EXPLANATORY NOTES

Introduction
These notes have been provided to simplify the geotechnical report with regard to investigation procedures, classification methods and certain matters relating to the Discussion and Comments section. Not all notes are necessarily relevant to all reports.

Geotechnical reports are based on information gained from finite sub-surface probing, excavation, boring, sampling or other means of investigation, supplemented by experience and knowledge of local geology. For this reason they must be regarded as interpretative rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods
The methods of description and classification of soils and rocks used in this report are based on AS1726 - 1993 "Geotechnical Site Investigations". In general, descriptions cover the following properties; strength or density, colour, structure, soil or rock type, and inclusions. Identification and classification of soil and rock involves, to a large extent, judgement within the acceptable level commonly adopted by current geotechnical practices.

Soil types are described according to the predominating particle size, qualified by the grading or other particles present (e.g. sandy clay) on the following basis:

Soil Classification	Particle Size
Clay	Less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2.00mm
Gravel	2.00mm to 60.00mm

Cohesive soils are classified on the basis of strength, either by laboratory testing or engineering examination. The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very Soft	Less than 12
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT), as below:

Relative Density	SPT 'N' Value (blows/300mm)	CPT Cone Value (q_c-MPQ)
Very Loose	Less than 5	Less than 2
Loose	5 – 10	2 – 5
Medium Dense	10 – 30	5 – 15
Dense	30 – 50	15 – 25
Very Dense	>50	>25

Rock types are classified by their geological names, together with descriptive terms on degrees of weathering, strength, defects and other minor components. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling
Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, type, moisture content, inclusions and depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin walled sample tube (normally known as U_{50}) into the soil and withdrawing a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils. Details of the type and method of sampling are given in the report.

Field Investigation Methods
The following is a brief summary of investigation methods currently carried out by this Company and comments on their use and application.

Hand Auger Drilling
The borehole is advanced by manually operated equipment. The diameter of the borehole ranges from 50mm to 100mm. Penetration depth of hand augered boreholes may be limited by premature refusal on a variety of materials, such as hard clay, gravels or ironstone.

Test Pits
These are excavated with a tractor-mounted backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to about 3.0m for a backhoe and up to 6.0m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Care must be taken if construction is to be carried out near, or within the test pit locations, to either adequately recompact the backfill during construction, or to design the structure to accommodate the poorly compacted backfill.

Large Diameter Auger (e.g. Pengo)
The hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed, but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers and is usually supplemented by occasional undisturbed tube sampling.

Continuous Spiral Flight Augers
The hole is advanced by using 90mm-115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be highly mixed with soil of other stratum.

Information from the drilling (as distinct from specific sampling by SPT or undisturbed samples) is of relatively lower reliability due to remoulding, mixing or softening of samples by groundwater, resulting in uncertainties of the original sample depth.

The spiral augers are usually advanced by using a V-bit through the soil profile to refusal, followed by Tungsten Carbide (TC) bit, to penetrate into bedrock. The quality and continuity of the bedrock may be assessed by examination of recovered rock fragments and through observation of the drilling penetration resistance.

Non-core Rotary Drilling (Wash Boring)
The hole is advanced by a rotary bit, with water being pumped down the drill rod and returned up the annulus carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the feel and rate of penetration.

Rotary Mud Stabilised Drilling
This is similar to rotary drilling, but uses drilling mud as a circulating fluid, which may consist of a range of products from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (e.g. SPT and U_{50}) samples).

Continuous Core Drilling
A continuous core sample is obtained using a diamond tipped core barrel. Providing full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush.

Portable Proline Drilling
This is manually operated equipment and is only used in sites which require bedrock core sampling and there is restricted site access to truck mounted drill rigs. The boreholes are usually advanced initially using a tricone roller bit and water circulation to penetrate the upper soil profile. In some instances, a hand auger may be used to penetrate the soil profile. Subsequent drilling into bedrock involves the use of NMLC triple tube equipment, using water as a lubricant.

Standard Penetration Tests
Standard penetration tests are used mainly in non-cohesive soils, but occasionally also in cohesive soils, as a means of determining density or strength and of obtaining a relatively undisturbed sample. The test procedure is described in AS1289 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 769mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In a case where full penetration is obtained with successive blow counts for each 150mm of, say 4, 6 and 7 blows as;

N = 13
4,6,7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm as;

15, 30/40mm

The results of the tests can be related empirically to the engineering properties of the soil. Occasionally the test method is used to obtain samples in 50mm diameter thin walled sample tubes in clays. In these circumstances, the test results are shown on the bore logs in brackets.

Cone Penetrometer Testing and Interpretation
Cone penetrometer testing (sometimes referred to as Dutch Cone-CPT) described in this report, has been carried out using an electrical friction cone penetrometer and the test is described in AS1289 6.5.1.

In the test, a 35mm diameter rod with cone tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig, which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output on continuous chart recorders. The plotted results given in this report have been traced from the original records. The information provided on the charts comprises:

- Cone resistance - the actual end bearing force divided by the cross sectional area of the cone, expressed in MPa *

- Sleeve friction - the frictional force on the sleeve divided by the surface area, expressed in kPa

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and very soft clays, rising to 4% to 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:

$$q_c = (12 \text{ to } 18) C_u$$

Interpretation of CPT values can also be made to allow estimate of modulus or compressibility values, to allow calculation of foundation settlements. Inferred stratification, as shown on the attached report, is assessed from the cone and friction traces, from experience and information from nearby boreholes etc.

This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties and where precise information or soil classification is required, direct drilling and sampling may be preferable.

Portable Dynamic Cone Penetrometer (DCP)
Portable Dynamic Cone Penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows per successive 100mm increment of penetration.

There are two similar tests, Cone Penetrometer (commonly known as Scala Penetrometer) AS1289 6.3.2 and the Perth Sand Penetrometer AS1289 6.3.3. Scala Penetrometer is commonly adopted by this company and consists of a 16mm rod with a 20mm diameter cone end, driven with a 9kg hammer, dropping 510mm (AS1289 Test P3.2).

Laboratory Testing
Laboratory testing is carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedures are given on the individual report forms.

Engineering Logs
The engineering logs presented herein are an engineering and/or geological interpretation of the sub-surface conditions and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, however, this is not always practicable or possible to justify economically. As it is, the boreholes represent only a small sample of the total sub-surface profile. Interpretation of the information and its application to design and construction should take into account the spacing of boreholes, frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Groundwater
Where groundwater levels are measured in boreholes, there are several potential problems:

- in low permeability soils groundwater, although present, may enter the hole slowly or perhaps not at all during the investigation period

- a localised perched water table may lead to an erroneous indication of the true water table

- water table levels will vary from time to time due to the seasons or recent weather changes. They may not be the same at the time of construction as indicated in the report

- the use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole if water observations are to be made

More reliable measurements can be achieved by installing standpipes that are read at intervals over several days, or weeks for low permeability soils. Piezometers sealed in a particular stratum may be advisable in low permeability soils, or where there may be interference from a perched water table or surface water.

Engineering Reports
Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, perhaps a three-storey building, the information and interpretation may not be relevant if the design proposal is changed, say to a twenty-storey building. If this occurs, the Company will be pleased to review the report and sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of sub-surface conditions, discussions of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on bore spacing and sampling frequency.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of contractors responding to commercial pressures.

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies
In the event that conditions encountered on-site during construction appear to vary from those that were expected from the information contained in the report, the Company requests immediate notification. Most problems are much more easily resolved when conditions are exposed rather than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes
Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institute of Engineers Australia. Where information obtained from this Investigation is provided for tendering purposes; it is recommended that all information, including the written report and discussion, be made available.

In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or make additional copies of the report available for contract purposes, at a nominal charge.

Site Inspection
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that the conditions exposed are as expected, to full time engineering presence on site.

Review of Design
Where major civil or structural developments are proposed, or where only a limited investigation has been completed, or where the geotechnical conditions are complex, it is prudent to have the design reviewed by a Senior Geotechnical Engineer.