



REPORT

TO

EGC CUSTODIAN SERVICES PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

**THE ALLENGROVE CRESCENT SITE
CNR LANE COVE ROAD & EPPING ROAD,
NORTH RYDE, NSW**

11 April 2008

Ref: 21874Zrpt2

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

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EXECUTIVE SUMMARY

A geotechnical investigation was carried out at the site known as 'The Allengrove Crescent Site'. It is proposed to rezone the site to a denser residential zoning to allow a two to five storey development with basement parking levels.

The site was found to be underlain by surficial topsoil over residual silty clay with inferred shale bedrock at relatively shallow depth. Groundwater was not encountered within the depths investigated.

Although the investigation was to relatively shallow depth due to access considerations restricting the type of equipment used, no adverse geotechnical issues which mitigate against the proposed rezoning were encountered.

Based on the investigation results, the site is considered suitable for the proposed development and there were no significant constraints to the completion of the associated excavations, earthworks, retaining walls, footings, on grade floor slabs, etc, using conventional construction techniques. Detailed design and construction recommendations are presented in the report.



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TABLE A: SUMMARY OF LABORATORY TEST RESULTS
BOREHOLE LOGS 1 TO 7 INCLUSIVE
DYNAMIC CONE PENETRATION TEST RESULTS (1 TO 7)
FIGURE 1: INVESTIGATION LOCATION PLAN
REPORT EXPLANATION NOTES



1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for the proposed residential development at the property known as 'The Allengrove Crescent Site' on the corner of Lane Cove Road and Epping Road, North Ryde, NSW. The investigation was commissioned by Mr Mark Syke of EGC Custodian Services Pty Ltd by email dated 30 January 2008. The commission was on the basis of our proposal (Ref: P15115Zemail) dated 8 January 2008.

Details of the proposed development had not been completed at the time of this investigation. However, we understand that it is intended to construct a medium density residential development probably comprising two to five storeys over one or two basement levels. We have assumed that structural loads in the medium range will apply.

The purpose of the investigation was to obtain generalised geotechnical information on subsurface conditions as a basis for preliminary comments and recommendations on excavation conditions, excavation support, retaining walls, footings and on-grade floor slabs.

We note that a contamination investigation was carried out concurrently with the geotechnical investigation by our environmental division, Environmental Investigation Services (EIS). This geotechnical report must be read in conjunction with the contamination report (Ref: E21874FJrpt).



2 INVESTIGATION PROCEDURE

The fieldwork for the investigation comprised the drilling of seven boreholes (BH1 to BH7) to depths between 1.25m and 1.5m, using a hand auger. In addition, seven Dynamic Cone Penetration (DCP) tests (DCP1 to DCP7) were carried out adjacent to the boreholes to refusal depths between 1.25m and 2.72m. The test results, as indicated on attached Figure 1, were set out using taped measurements from apparent site boundaries. A survey plan was not available at the time of completion of this report, and therefore the reduced levels (RLs) at the test locations, have not been determined.

The nature and composition of the subsoils were assessed by logging the materials recovered during drilling. The strength of the subsoils was assessed by interpretation of the DCP test results augmented by hand penetrometer readings carried out on clay lump samples recovered by the hand auger. The DCP refusal depth can also provide an indicative depth to rock, though we note that refusal can also occur on obstructions in fill, 'floaters', other hard layers, and not necessarily on bedrock. Groundwater observations were made during and on completion of drilling individual boreholes. Long term groundwater monitoring was not carried out. For further details on the investigation procedure adopted, reference should be made to the attached Report Explanation Notes.

Our geotechnical engineer carried out the fieldwork and also set out the test locations, nominated sampling and testing, and logged the subsurface profile. The borehole logs and DCP test results are presented with this report together with a glossary of logging terms and symbols used.

Several representative soil samples were submitted to a NATA registered laboratory (Soil Test Services Pty Ltd) for moisture content, Atterberg Limit and linear shrinkage testing. The test results are summarised in attached Table A.



3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in topography which slopes gently down towards the north-west, and is bounded by Epping Road along the north-east, Lane Cove Road along the north-west, and Allengrove Crescent along the south-west. The site has an approximately square plan shape covering about 12,000m².

At the time of the investigation, the site comprised 16 residential lots, each occupied by single storey brick, timber or fibro houses with associated driveways and gardens. The northern-most lot (No 285) was used as a recreation area whilst the lot over the west (No 255) was not within the designated site area. Based on a cursory inspection, the existing houses on the lots which were investigated appeared in good external condition, except for stepped cracking within the dwarf wall over the north-eastern elevation of the house on No 118, and patched cracks within the brickwork over the south-western elevation of the house on No 259. The concrete driveways were generally in poor condition.

Similar residential lots bounded the site along the south-east, and were located across Allengrove Crescent to the south-west.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by Ashfield Shales close to the contact with the underlying Hawkesbury Sandstone to the east. The investigation has revealed a generalised subsurface profile comprising surficial topsoil over residual silty clay, with inferred shale bedrock at relatively shallow depth. Reference should be made to the attached borehole logs and DCP test results for detailed subsurface conditions at specific locations. A summary of the encountered subsurface conditions is presented below:



- Topsoil (or fill) comprising silty clay of high plasticity was encountered in all boreholes to depths between 0.2m and 0.4m. The topsoil/fill contained brick and concrete fragments, ironstone, sandstone and igneous gravel, and roots.
- Residual silty clay was encountered below the topsoil/fill in all boreholes and extended to the borehole termination depths between 1.25m and 1.5m. The residual silty clay was generally of high plasticity and very stiff and hard strength.
- Based on the DCP results, the residual silty clay extends to a maximum depth of about 2m. Blowcounts of greater than about 10 per 100mm penetration are interpreted to indicate extremely weathered shale or clay with numerous shale and ironstone bands. The DCP refusal depth has been interpreted to indicate the surface of the weathered shale bedrock.
- Groundwater seepage was encountered at a depth of 1.35m and 0.4m whilst drilling BH1 and BH2 respectively. All boreholes were 'dry' on completion of drilling. We note that the groundwater levels may not have stabilised during the limited observation period. Long term groundwater monitoring was not carried out.

3.3 Laboratory Test Results

The laboratory test results have confirmed our field classification of the site soils. The liquid limit and linear shrinkage test results confirm the sampled residual silty clays to have a high plasticity with an assessed high shrink-swell reactivity.



4 COMMENTS AND RECOMMENDATIONS

The comments and recommendations which follow are of a preliminary and generalised nature, as the development details have not been finalised. Once the proposed development details are known, the comments and recommendations should be reviewed and possibly revised to address specific requirements of the proposed development.

4.1 Excavation Conditions

The proposed basement excavations will encounter the clay profile over the upper 2m or so and then extend into the underlying weathered shale bedrock.

The soil cover should be readily excavatable using conventional earthworks equipment (eg. hydraulic excavators). Some of the underlying weathered shale of extremely or very low strength, if encountered, may also be excavated by a large bucket excavator, possibly with some ripping. However, we expect excavation of low to medium and higher strength shale which would probably be encountered for deeper basements would be most effectively excavated using hydraulic impact rock hammers. This equipment would also be required for breaking up of boulders or blocks, for trimming rock excavation side slopes, and for detailed rock excavations, such as for footings or buried services.

We would expect some groundwater seepage flows will occur through gravel bands, joints and bedding planes within the completed cut faces, particularly after periods of heavy rain. Seepage, if any, during excavation is expected to be satisfactorily controlled by conventional sump pumping or gravity drainage systems. The groundwater seepage into the bulk excavation should be monitored so that unexpected conditions can be timeously addressed.



4.2 Excavation Techniques

We recommend that considerable caution be taken during rock excavation on this site, as there will likely be direct transmission of ground vibrations to adjoining buildings and structures. Depending on the setback of the proposed excavation from the neighbouring buildings and structures, dilapidation reports should be compiled and the owners asked to confirm that the reports present a fair record of existing conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage resulting from the works. The dilapidation reports and the excavation procedures should be carefully reviewed prior to excavation commencing, so that appropriate equipment is used.

An assessment on the need for vibration monitoring should be made once the final excavation details are known. However, we would expect that a moderately sized excavator fitted with a relatively low energy hydraulic hammer would be suitable for rock excavations in close proximity to neighbouring buildings and structures. Where ground vibrations need to be reduced, a vertical saw cut slot should be provided along the perimeter of the excavation and the base of the slot maintained at a lower level than the adjoining rock excavation at all times. Alternative excavation techniques which will further reduce vibrations include grid sawing or the use of smaller rock hammers. When using a rock saw, the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations if rock hammers are used:

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Operate one hammer at a time and in short bursts only to reduce amplification of vibrations.



- Use excavation contractors with experience in confined work, with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

4.3 Excavation Support

Where space permits, excavations through the soil and extremely weathered shale profile may be temporarily battered to a side slope no steeper than 1 Vertical (V) in 1 Horizontal (H). We note that possible seepage at the soil-rock interface may cause localised instability at the toe of soil batters, and allowance should be made for sandbagging. A retaining wall may then be constructed at the toe of the batter, and subsequently backfilled. Where temporary batters cannot be accommodated or where they are not preferred, a retention system will be required, possibly installed prior to excavation commencing. Suitable retention systems given the subsurface conditions encountered, include soldier pile walls with reinforced shotcrete infill panels.

We expect that good quality shale of low or higher strength may be cut vertically. However, on site stabilisation measures may be necessary if adverse defects, such as inclined joints or bedding, are found. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc. Clay or extremely weathered seams occurring in permanently exposed shale slopes may also require 'dental' treatment. We therefore recommend that the rock face be progressively inspected by a geotechnical engineer/engineering geologist as excavation proceeds (at no more than 1.5m depth intervals), to identify adverse defects and to propose appropriate stabilisation measures. Provision should be made in the contract documents (budget and program) for the above inspections and stabilisation measures.



4.4 Retaining Walls

Retaining walls, if required, should be designed using the following characteristic earth pressure coefficients and subsoil parameters:

- Free-standing cantilever walls supporting areas where movement is of little concern (ie. where only garden or open areas are being retained), should be designed using a triangular later earth pressure distribution, with an 'active' earth pressure coefficient, K_a , of 0.3, for the soil profile and extremely weathered shale bedrock, assuming a horizontal retained surface.
- Cantilever walls, the tops of which will be restrained by the ground floor slab of the main structure, or which are supporting movement sensitive structural elements, should be designed using a triangular lateral earth pressure distribution, with an 'at rest' earth pressure coefficient, K_o , of 0.5, for the soil profile and extremely weathered shale bedrock, assuming a horizontal retained surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile and extremely weathered shale bedrock.
- Anchored or internally propped walls supporting areas where only minor movements can be tolerated, should be designed using a trapezoidal earth pressure distribution of $6H\text{ kPa}$ for the soil profile and the extremely shale bedrock, where 'H' is the retained height in metres. These pressures should be assumed to be uniform over the central 60% of the support system.
- Anchored or internally propped walls which are supporting areas which are highly sensitive to movement, should be designed using a trapezoidal earth pressure distribution of $8H\text{ kPa}$ for the soil profile and weathered shale bedrock, where 'H' is the retained height in metres. These pressures should be assumed to be uniform over the central 60% of the support system.



- Any surcharge affecting the walls (eg. traffic, construction loads, nearby high level footings, etc) should be allowed in the design using the appropriate earth pressure coefficient from above. If inclined retained surfaces are proposed, then the earth pressure coefficient would have to be appropriately increased, or the inclined surface treated as a surcharge.
- The retaining walls should be designed to withstand full hydrostatic pressures unless measures are taken to induce complete and permanent drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile fabric (eg. Bidim A34) to act as a filter against subsoil erosion.
- Lateral toe restraint may be achieved by keying (socketing) the wall footing into bedrock below bulk excavation level. An allowable lateral toe resistance of 200kPa may be tentatively adopted for low or higher strength shale. This value assumes excavation is not carried out within the zone of influence of the toe wall, and the rock does not contain unfavourable defects, etc. The upper 0.3m depth of socket below bulk excavation level should be ignored in the analysis to allow for disturbance effects during excavations.
- Rock anchors should be bonded at least 3m into low or higher strength shale and tentatively designed using an allowable bond stress of 100kPa. All anchors should be proof-tested to 1.3 times the working loads under the direction of an experienced engineer independent of the anchor contractor. The testing may allow an upgrading of 100kPa bond stress. Where rock anchors extend beyond the site boundaries, then the permission of the neighbours should be obtained before installation.



4.5 Footings

4.5.1 Site Classification

Based on the investigation results, the site classifies as Class 'H' in accordance with AS2870. We note, however, that the influence of the existing development will have the effect of abnormal moisture conditions, therefore resulting in a Class 'P' site classification. Further, AS2870 refers to residential slabs and footings for typical buildings up to 30m long, and may not be applicable for the subject development.

4.5.2 High Level Footings

Relatively lightly loaded buildings may be supported using a high level footing option consisting of strip and/or pad footings or a stiffened raft slab, founded in the very stiff residual silty clays. The footings may be designed for a maximum allowable bearing pressure of 150kPa. All strip or pad footings should be designed on the basis that the site classifies as Class 'H', in accordance with AS2870.

Where a basement is provided or the site has been cut, strip and pad footings may be founded in the weathered shale bedrock. The footings may be designed for a maximum allowable bearing pressure of 700kPa.

We suggest geotechnical engineering inspections be carried out to confirm the allowable bearing pressures and the presence of any abnormal moisture conditions.



4.5.3 Pile Footings

Bored piles founded at least 0.3m into the underlying shale bedrock may be designed for an allowable end bearing pressure of 700kPa. In addition, an allowable shaft adhesion of 70kPa may be applied to that length of rock socket in excess of 0.3m into shale bedrock of very low or better strength. The safe shaft adhesion should be halved in the case where tension or uplift is to be resisted.

4.5.4 Footing Construction

We recommend that pad and strip footings be excavated, cleaned, inspected and poured with minimal delay to avoid deterioration. If delays in pouring concrete are anticipated, we recommend that the base of the footings be protected with a blinding layer of concrete.

Water should be prevented from ponding in the base of footings, as this will tend to soften the foundation material, resulting in further excavation and cleaning being required.

Groundwater inflow would be expected into bored pile excavations, particularly after heavy rains, and we expect that this inflow will be controllable by conventional pumping methods. The bored piles should be drilled, cleaned, inspected and poured with minimal delay (ie. on the same day).



4.6 On-Grade Floor Slabs

Slab-on-grade construction is considered feasible, provided any clay subgrade which is exposed is adequately prepared.

Following demolition of buildings and pavements, the removal of trees and shrubs, and the stripping of root affected soils, any obvious deleterious or contaminated existing fill should be removed. These stripped materials should be separately stockpiled as they are not suitable for reuse as engineered fill. The site should then be excavated to suit the design subgrade levels for the development.

The exposed clay subgrade should be proof-rolled. Proof-rolling should be carried out under the direction of an experienced earthworks superintendent or geotechnical engineer to assist in the detection of unstable areas which were not disclosed by this investigation. Any unstable areas identified during proof-rolling should be locally excavated down to a competent base and replaced with engineered fill.

Engineered fill may comprise the existing silty clays, provided suitable ('over-wet' and 'over-size') material and any building rubble is excluded. The fill for backfilling earthworks platforms should be compacted in layers of not greater than 200mm loose thickness to a density strictly between 98% and 102% of Standard Maximum Dry Density (SMDD) and within 2% of Standard Optimum Moisture Content (SOMC). A well graded granular material (ripped or crushed sandstone) free of deleterious substances and having a maximum particle size of 75mm may also be used as engineered fill, and compacted as above to a minimum density of 98% SMDD.

Density testing should be carried out on engineered fill, to confirm that the above specification has been adhered to. At least Level 2 testing of earthworks should be carried out to a frequency detailed in AS3798. Preferably the Geotechnical Testing Authority should be engaged directly on behalf of the client and not as part of the earthworks contract.



Underfloor drainage should be provided in all cut areas. The underfloor drainage should comprise a strong, durable, single sized washed aggregate, such as 'blue metal' gravel. The underfloor drainage should connect with the wall drains and lead groundwater seepage to a sump for pumped or gravity disposal to the stormwater system.

The on-grade floor slab should be separately from all walls, footings, columns, etc, to permit relative movement. Joints in the concrete on-grade floor slab should be designed to accommodate shear forces but not bending moments, by using dowelled or keyed joints.

4.7 Pavements

The design of carpark pavements and driveways will depend on subgrade preparation, subgrade drainage, the nature and composition of new fill imported to site, as well as vehicular loadings and use. We recommend that flexible pavements for parking and driveway areas which overlie a silty clay subgrade be based on a CBR value of 3% for the compacted clay subgrade, which is prepared as outlined for on-grade floor slabs above. For concrete or rigid pavement design, an equivalent modulus of subgrade reaction of 25kPa/mm (75mm plate) may be adopted.

Concrete pavements should be supported on a subbase layer of RTA 3051 Specification unbound or equivalent good quality crushed rock, compacted to a density of at least 100% SMDD. The subbase material would provide more uniform slab support and would reduce 'pumping' of subgrade 'fines' at joints. Concrete pavements should be provided with effective shear connection at joints by using dowels or keys.



Subsoil drains should be provided on the uphill side of the development and along the perimeter of all pavements, with inverts not less than 0.2m below clay subgrade level. The drainage trench should be excavated with a longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding.

4.8 Additional Geotechnical Investigations

Once the development details are known, it may be necessary to carry out additional geotechnical investigations to confirm the bedrock levels and quality. The extent of further investigations should be tailored to suit the proposed development details.

4.9 Further Geotechnical Input

The following summarises the further geotechnical input which will be required and which has been detailed in the preceding sections of this report:

- Review of this report once the proposed development details become available.
- Additional geotechnical investigations, if appropriate.
- Dilapidation surveys of surrounding buildings and structures, depending on excavation location and depth with respect to the neighbouring buildings.
- Vibration monitoring during rock excavation, if appropriate.
- Geotechnical inspections of cut rock faces, if appropriate.
- Monitoring of groundwater seepage into bulk excavation.
- Geotechnical footing inspections.
- Density testing of engineered fill.
- Witnessing of proof-rolling.



5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and Jeffery and Katauskas Pty Ltd accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed boreholes and DCP tests may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar



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

A ZENON
Senior Associate
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

Ref No:21874Z

Table A: Page 1 of 1

TABLE A
SUMMARY OF LABORATORY TEST RESULTS

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
2	0.70-0.80	22.6	64	21	43	16.0
4	1.00-1.10	19.9	63	22	41	15.5

Notes:

- The test sample for liquid and plastic limit was oven-dried(50°C) & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions



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Authorised Signature
(A.Tatikonda)



Date: 22/2/08



Borehole No.

1

1/1

BOREHOLE LOG

Client: EG FUNDS

Project: PROPOSED REDEVELOPMENT

Location: LANE COVE ROAD AND ALLENGROVE CRESCENT, NORTH RYDE, NSW

Job No. 21874Z

Method: HAND AUGER

R.L. Surface: N/A

Date: 13-2-08

Datum:

Logged/Checked by: M.P./*MP*

Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	US	DB									
				REFER TO DCP TEST RESULTS	0			TOPSOIL: Silty clay, high plasticity, brown, with a trace of fine to medium grained brick and ironstone gravel, and a trace of root fibres.	MC > PL			GRASS COVER
					0.5		CH	SILTY CLAY: high plasticity, red brown, with a trace of fine to medium grained ironstone gravel, and a trace of ash and root fibres.	MC > PL	St-VSt	120 180 220	HP TESTING CARRIED OUT ON REMOULDED SAMPLES
					1			as above, but red brown mottled light grey brown.	MC > PL	VSt	230 280 380	
					1.5			SILTY CLAY: high plasticity, light grey mottled red brown.			180 220 240	
					1.5			END OF BOREHOLE AT 1.5m				
					2							
					2.5							
					3							
					3.5							


Borehole No.

2

1/1

BOREHOLE LOG

Client: EG FUNDS Project: PROPOSED REDEVELOPMENT Location: LANE COVE ROAD AND ALLENGROVE CRESCENT, NORTH RYDE, NSW										
Job No. 21874Z Date: 13-2-08			Method: HAND AUGER Logged/Checked by: M.P./ <i>MP</i>				R.L. Surface: N/A Datum:			

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Ref. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	US	DB	DS									
					REFER TO DCP TEST RESULTS	0			TOPSOIL: Silty clay, high plasticity, dark brown, with roots and root fibres.	MC > PL			GRASS COVER
						0.5		CH	SILTY CLAY: high plasticity, red brown, with a trace of fine to medium grained ironstone gravel, and root fibres.	MC > PL	VSt	250 260 310	HP TESTING CARRIED OUT ON REMOULDED SAMPLES
						1			as above, but red brown mottled light grey.			350 360 360	
									as above, but light grey mottled red brown.				
						1.5			SILTY CLAY: high plasticity, light grey, with XW shale gravel. END OF BOREHOLE AT 1.25m				HAND AUGER REFUSAL ON INFLECTED SHALE
						2							
						2.5							
						3							
						3.5							



Borehole No.

3

1/1

BOREHOLE LOG

Client: EG FUNDS

Project: PROPOSED REDEVELOPMENT

Location: LANE COVE ROAD AND ALLENGROVE CRESCENT, NORTH RYDE, NSW

Job No. 21874Z

Method: HAND AUGER

R.L. Surface: N/A

Date: 13-2-08

Datum:

Logged/Checked by: M.P./*MP*

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			TOPSOIL: Silty clay, high plasticity, dark brown and grey, with organic matter.	MC > PL			LEAF COVER
						0.5		CH	SILTY CLAY: high plasticity, orange brown, with a trace of ash and fine grained ironstone gravel.	-	VSt	-	HP TESTING CARRIED OUT ON REMOULDED SAMPLES
						1			SILTY CLAY: high plasticity, light grey mottled red brown, with a trace of fine grained ironstone gravel.	MC < PL	H	450 420 410	
						1.5			END OF BOREHOLE AT 1.5m			> 600 540 580	
						2							
						2.5							
						3							
						3.5							



Borehole No.

4

1/1

BOREHOLE LOG

Client: EG FUNDS Project: PROPOSED REDEVELOPMENT Location: LANE COVE ROAD AND ALLENGROVE CRESCENT, NORTH RYDE, NSW												
Job No. 21874Z Date: 13-2-08			Method: HAND AUGER Logged/Checked by: M.P./ <i>RL</i>				R.L. Surface: N/A Datum:					
Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB									
DRY ON COMPLETION	█	█	█	REFER TO DCP TEST RESULTS	0			TOPSOIL: Silty sandy clay, high plasticity, dark brown, with a trace of fine grained ironstone and sandstone gravel, root fibres and ash.	MC≈PL			GRASS COVER
					0.5	CH	SILTY CLAY: high plasticity, red brown mottled red brown, with a trace of ash, root fibres and fine grained ironstone gravel.	MC < PL	VSt -H	410 310 310	HP TESTING CARRIED OUT ON REMOULDED SAMPLES	
					1							as above, but light grey mottled red brown, and without ironstone gravel.
					1.5			END OF BOREHOLE AT 1.5m			410 410 450	
					2							
					2.5							
					3							
					3.5							



Borehole No.
5
1/1

BOREHOLE LOG

Client: EG FUNDS
Project: PROPOSED REDEVELOPMENT
Location: LANE COVE ROAD AND ALLENGROVE CRESCENT, NORTH RYDE, NSW

Job No. 21874Z **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 13-2-08 **Datum:**
Logged/Checked by: M.P./ *RP*

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	USO	DB	DS									
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			TOPSOIL: Silty clay, high plasticity, dark brown, with a trace of igneous gravel.				GRASS COVER
						0.5		CH	SILTY CLAY: high plasticity, red brown mottled brown red, with a trace of ironstone gravel.	MC > PL	VSt	260 280 340	HP TESTING CARRIED OUT ON REMOULDED CLAY
						1			as above, but red brown mottled light grey.			320 300 240	
									as above, but light grey and red brown.	MC ≈ PL	H	510 501 410	
						1.5			as above, but with fine to medium grained ironstone gravel.				
									END OF BOREHOLE AT 1.5m				
						2							
						2.5							
						3							
						3.5							



Borehole No.
6
1/1

BOREHOLE LOG

Client: EG FUNDS
Project: PROPOSED REDEVELOPMENT
Location: LANE COVE ROAD AND ALLENGROVE CRESCENT, NORTH RYDE, NSW

Job No. 21874Z **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 13-2-08 **Datum:**
Logged/Checked by: M.P./ *MR*

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	D8	D5									
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			TOPSOIL: Silty sandy clay, high plasticity, red brown, with roots and root fibres, and a trace of fine to medium grained brick and concrete fragments.	MC > PL			WOOD CHIPS COVER
						0.5		CH	SILTY CLAY: high plasticity, red brown, with a trace of root fibres and ash and fine to medium grained ironstone gravel.	MC > PL	VSt	320 400 380	HP TESTING CARRIED OUT ON REMOULDED SAMPLES
						1			as above, but red brown mottled grey brown.				
									as above, but light grey mottled red brown, without root fibres.		VSt-H H	430 380 390	
						1.5			END OF BOREHOLE AT 1.5m			500 470 470	
						2							
						2.5							
						3							
						3.5							



Borehole No.
7
1/1

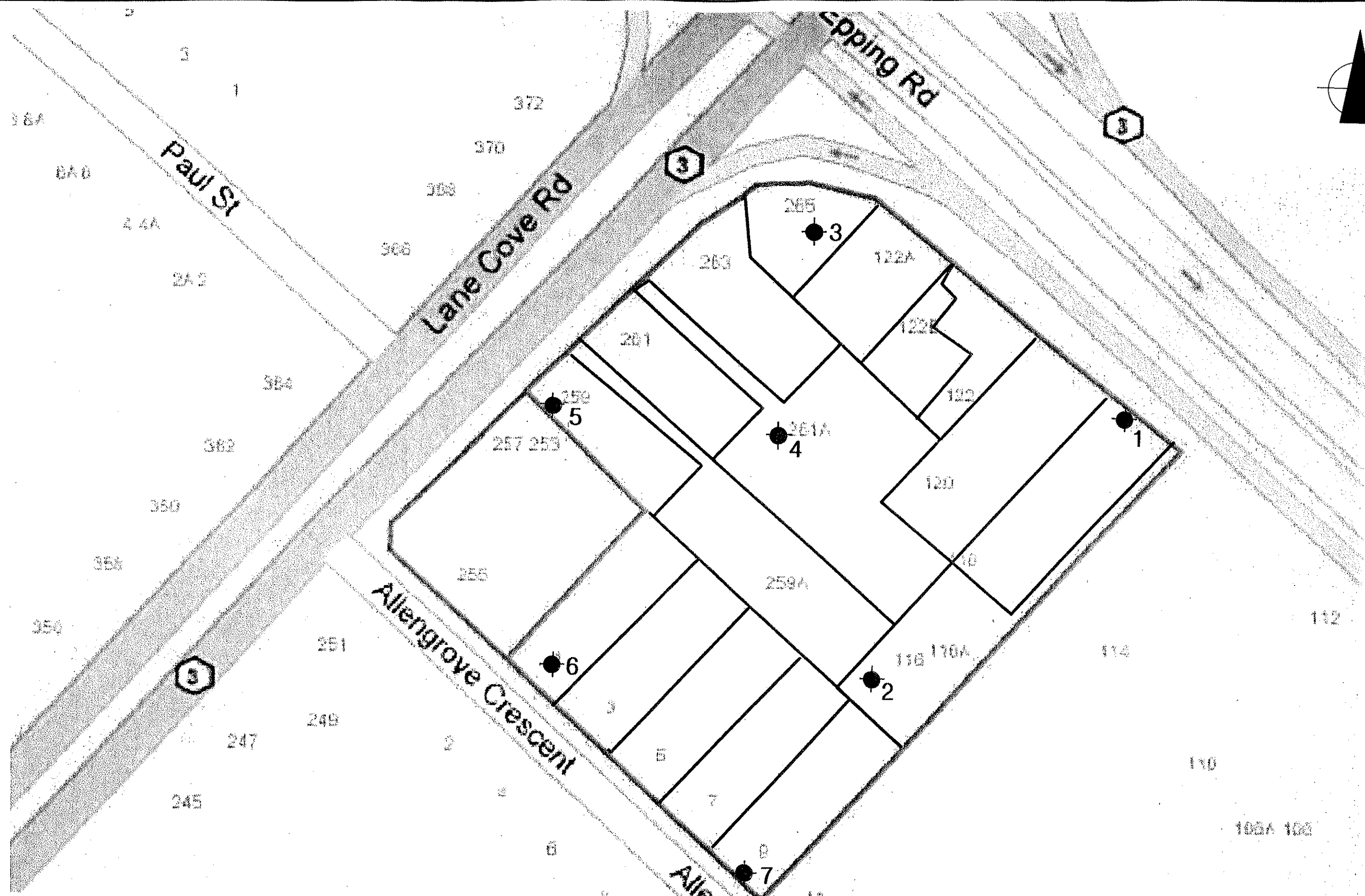
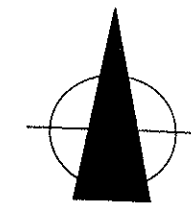
BOREHOLE LOG

Client: EG FUNDS Project: PROPOSED REDEVELOPMENT Location: LANE COVE ROAD AND ALLENGROVE CRESCENT, NORTH RYDE, NSW												
Job No. 21874Z Date: 13-2-08			Method: HAND AUGER Logged/Checked by: M.P./ <i>[Signature]</i>			R.L. Surface: N/A Datum:						
Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	US	DB									
DRY ON COMPLETION				REFER TO DCP TEST RESULTS	0			TOPSOIL: Silty clay, high plasticity, dark brown, with a trace of fine to medium grained iron indurated shale, sandstone and roots and root fibres.	MC > PL			GRASS COVER
					0.5		CH	SILTY CLAY: high plasticity, red brown mottled dark grey, with a trace of ironstone gravel.	MC > PL	VSt	280	HP TESTING CARRIED OUT ON REMOULDED SAMPLES
					1			as above, but red brown mottled light grey, with a trace of ash.			280	
								as above, but light grey mottled red brown.			260	
									MC ≈ PL		340	INSUFFICIENT CLAY RECOVERED FOR HP TESTING HAND AUGER REFUSAL ON INFILLED SHALE
										350		
										400		
					1.5			END OF BOREHOLE AT 1.35m				
					2							
					2.5							
					3							
					3.5							



DYNAMIC CONE PENETRATION TEST RESULTS

Client:	EG FUNDS						
Project:	PROPOSED REDEVELOPMENT						
Location:	LANE COVE ROAD AND ALLENGROVE CRESCENT, NORTH RYDE, NSW						
Job No.	21874Z			Hammer Weight & Drop: 9kg/510mm			
Date:	13-2-08			Rod Diameter: 16mm			
Tested By:	M.P.			Point Diameter: 20mm			
Number of Blows per 100mm Penetration							
Test Location							
Depth (mm)	1	2	3	4	5	6	7
0 - 100	2	2	↓	5	2	1	1
100 - 200	1	2	2	4	2	1	2
200 - 300	3	1	3	4	3	3	2
300 - 400	2	3	2	3	2	3	4
400 - 500	3	2	2	4	5	3	4
500 - 600	3	3	2	4	4	3	5
600 - 700	3	2	2	5	3	2	5
700 - 800	2	4	4	5	4	3	6
800 - 900	3	6	6	6	5	3	4
900 - 1000	3	6	4	6	4	4	3
1000 - 1100	6	6	6	5	3	4	5
1100 - 1200	4	3	6	5	5	3	5
1200 - 1300	4	9	9	5	8	5	17/50mm
1300 - 1400	6	25/50mm	7	5	7	5	REFUSAL
1400 - 1500	14	REFUSAL	7	4	8	4	
1500 - 1600	REFUSAL		9	5	12	13	
1600 - 1700			9	4	8	10	
1700 - 1800			9	4	12	14	
1800 - 1900			12	6	10	23	
1900 - 2000			20	12	11	15/50mm	
2000 - 2100			20	16	13	REFUSAL	
2100 - 2200			3/20mm	21	23		
2200 - 2300			REFUSAL	10/60mm	24		
2300 - 2400				REFUSAL	23		
2400 - 2500					21		
2500 - 2600					26		
2600 - 2700					28		
2700 - 2800					5/20mm		
2800 - 2900					REFUSAL		
2900 - 3000							
Remarks:	1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2. 2. Usually 8 blows per 20mm is taken as refusal						





REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, the SAA Site Investigation Code. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the grading of other particles present (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'Shale' is used to describe thinly bedded to laminated siltstone.

SAMPLING

Sampling is carried out during drilling or from other excavations 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, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained 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 used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.



Test Pits: These are normally excavated with a 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 3m for a backhoe and up to 6m for an excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and 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 by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock fragments. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods 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 "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided 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. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. 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 the 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
$$N > 30$$
$$15, 30/40\text{mm}$$

The results of the test can be related empirically to the engineering properties of the soil.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as "N_c" on the borehole logs, together with the number of blows per 150mm penetration.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by 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 as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- 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.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

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 occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The attached explanatory notes define the terms and symbols used in preparation of the logs.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than “straight line” variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- 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 with seasons or recent weather changes and may not be the same at the time of construction.
- 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 or ‘reverted’ chemically if water observations are to be made.



More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to perhaps 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 perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg bricks, steel etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

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

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 (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg to a twenty storey building). If this happens, the company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion 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 be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that 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 Institution 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 to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

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/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer.


































SITE INSPECTION

The company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

- i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
- ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or
- iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL		ROCK		DEFECTS AND INCLUSIONS	
	FILL		CONGLOMERATE		CLAY SEAM
	TOPSOIL		SANDSTONE		SHEARED OR CRUSHED SEAM
	CLAY (CL, CH)		SHALE		BRECCIATED OR SHATTERED SEAM/ZONE
	SILT (ML, MH)		SILTSTONE, MUDSTONE, CLAYSTONE		IRONSTONE GRAVEL
	SAND (SP, SW)		LIMESTONE		ORGANIC MATERIAL
	GRAVEL (GP, GW)		PHYLLITE, SCHIST		
	SANDY CLAY (CL, CH)		TUFF		
	SILTY CLAY (CL, CH)		GRANITE, GABBRO		
	CLAYEY SAND (SC)		DOLERITE, DIORITE		
	SILTY SAND (SM)		BASALT, ANDESITE		
	GRAVELLY CLAY (CL, CH)		QUARTZITE		
	CLAYEY GRAVEL (GC)				
	SANDY SILT (ML)				
	PEAT AND ORGANIC SOILS				
				OTHER MATERIALS	
					CONCRETE
					BITUMINOUS CONCRETE, COAL
					COLLUVIUM



UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 µm and basing fractions on estimated weights)				Group Symbols	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria			
Coarse-grained soils More than half of material is larger than 75 µm sieve size (The 75 µm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{D_{10}^2}{D_{30} \times D_{60}}$ Between 1 and 3 Not meeting all gradation requirements for GW			
			Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines					
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see ML below)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures					
			Plastic fines (for identification procedures, see CL below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures					
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well graded sands, gravelly sands, little or no fines	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: Silty sand, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{D_{10}^2}{D_{30} \times D_{60}}$ Between 1 and 3 Not meeting all gradation requirements for SW			
			Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands, little or no fines					
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see ML below)	SM	Silty sands, poorly graded sand-silt mixtures					
			Plastic fines (for identification procedures, see CL below)	SC	Clayey sands, poorly graded sand-clay mixtures					
			Identification Procedures on Fraction Smaller than 380 µm Sieve Size							
			Silt and clays liquid limit less than 50	Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)			Toughness (consistency near plastic limit)		Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
None to slight	Quick to slow	None		ML						
Medium to high	None to very slow	Medium		CL						
Slight to medium	Slow	Slight		OL						
Silt and clays liquid limit greater than 50	Slight to medium	Slow to none		Slight to medium	MH	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{D_{10}^2}{D_{30} \times D_{60}}$ Between 1 and 3 Not meeting all gradation requirements for SW			
	High to very high	None		High	CH					
	Medium to high	None to very slow		Slight to medium	OH					
	Readily identified by colour, odour, spongy feel and frequently by fibrous texture				Pt			Peat and other highly organic soils		

Determine percentages of gravel and sand from grain size curve
Depending on percentage of fines (fraction smaller than 75 µm sieve size) coarse grained soils are classified as follows:
Less than 5% GW, GP, SW, SP
More than 5% GM, GC, SM, SC
Borderline cases requiring use of dual symbols

Use grain size curve in identifying the fractions as given under field identification

Plasticity index

Comparing soils at equal liquid limit

Toughness and dry strength increase with increasing plasticity index

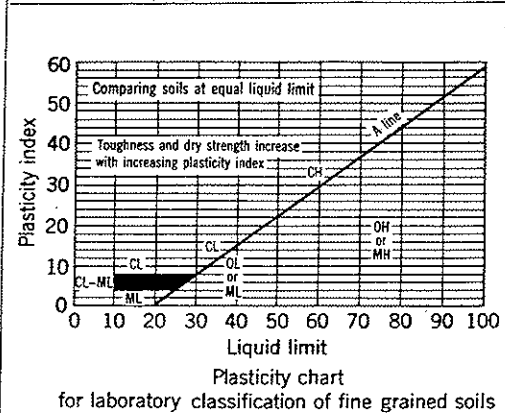
A line

CL, OL, CH, OH, MH, ML

Liquid limit

Plasticity chart for laboratory classification of fine grained soils

Determine percentages of gravel and sand from grain size curve
Depending on percentage of fines (fraction smaller than 75 µm sieve size) coarse grained soils are classified as follows:
Less than 5% GW, GP, SW, SP
More than 12% GM, GC, SM, SC
Borderline cases requiring use of dual symbols



NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. GW-GC, well graded gravel-sand mixture with clay fines).

2) Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.



LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
		Extent of borehole collapse shortly after drilling.
		Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N _c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils) (Cohesionless Soils)	MC > PL	Moisture content estimated to be greater than plastic limit.
	MC ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	MC < PL	Moisture content estimated to be less than plastic limit.
	D	DRY - runs freely through fingers.
	M	MOIST - does not run freely but no free water visible on soil surface.
	W	WET - free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT - Unconfined compressive strength less than 25kPa
	S	SOFT - Unconfined compressive strength 25-50kPa
	F	FIRM - Unconfined compressive strength 50-100kPa
	St	STIFF - Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF - Unconfined compressive strength 200-400kPa
	H	HARD - Unconfined compressive strength greater than 400kPa
	()	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)	VL	Density Index (I _d) Range (%) SPT 'N' Value Range (Blows/300mm) Very Loose < 15 0-4
	L	Loose 15-35 4-10
	MD	Medium Dense 35-65 10-30
	D	Dense 65-85 30-50
	VD	Very Dense > 85 > 50
	()	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
	250	
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T ₆₀	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.



LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered 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 rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL	0.03	Easily remoulded by hand to a material with soil properties.
Very Low:	VL	0.1	May be crumbled in the hand. Sandstone is "sugary" and friable.
Low:	L	0.3	A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
Medium Strength:	M	1	A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
High:	H	3	A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
Very High:	VH	10	A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	