# REPORT

TO

# EGC CUSTODIAN SERVICES PTY LTD

ON

# GEOTECHNICAL INVESTIGATION

FOR

# PROPOSED RESIDENTIAL DEVELOPMENT

AT

# WHITESIDE STREET SITE
# DAVID AVENUE & EPPING ROAD
# NORTH RYDE, NSW

11 April 2008

Ref: 21873Zrpt2

**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 'Whiteside Street 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 shale and sandstone bedrock at relatively shallow depth. Groundwater seepage was encountered at about 4m depth.

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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## 1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at the property known as the 'Whiteside Street Site' between David Avenue and Epping Road, North Ryde, NSW. The investigation was commissioned by Mr Michael Zelas of EG Funds, by email dated 30 January 2008. The commission was on the basis of our proposal (Ref: P15114Zemail) dated 8 January 2008.

Details of the proposed development were not available at the time of preparing this report. We understand, however, 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 moderate range will apply.

The purpose of the investigation was to obtain 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). The geotechnical report must be read in conjunction with the contamination report (Ref: E21873FJrpt).

## 2 INVESTIGATION PROCEDURE

The fieldwork for the investigation comprised the auger drilling of seven boreholes (BH1 to BH7) to depths between 5.4m and 7.5m using our truck mounted JK550 and track mounted JK300 drilling rigs. The borehole locations, as indicated on attached Figure 1, were set out using taped measurements from inferred site boundaries. A survey plan was not available at the time of completion of this report, and therefore the surface reduced levels (RLs) at the borehole locations, have not been determined.

The nature and composition of the subsurface soils and rocks were assessed by logging the materials recovered during drilling. The strength of the subsoils was assessed from the Standard Penetration Test (SPT) 'N' values augmented by hand penetrometer readings on clay samples recovered in the SPT split tube sampler. The strength of the underlying bedrock was assessed by observation of the drilling resistance when using a tungsten carbide (TC) bit, examination of the recovered rock chip samples, and subsequent correlation with laboratory moisture content testing. 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 engineering geologist (Matthew Green) was full time on site during the investigation, and set out the borehole locations, nominated sampling and testing, and logged the subsurface profile. The borehole logs are presented with this report, together with a glossary of logging terms and symbols used.

Selected soil and rock chip samples were recovered from site and 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 has an irregular plan shape, is bounded by Epping Road to the north, Whiteside Street to the west, and David Avenue to the east, and is located in topography which slopes gently down towards the west at around 10°.

At the time of the fieldwork, the site was occupied by two residential properties fronting on David Avenue, and a larger sized semi-rural property fronting on Epping Road with several buildings and paddocks. The buildings and structures on site were assessed to be in fair external condition based on a cursory inspection. Several trees were located over the southern portion of the site.

To the south and east of the site were numerous one and two storey brick and weatherboard houses which appeared in good condition when viewed from within the subject site.

## 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 and fill over residual silty clay, with shale and sandstone bedrock at moderate depth. Reference should be made to the attached borehole logs for detailed subsurface conditions at specific locations. A graphical borehole summary is presented in Figure 2, and a summary of the encountered subsurface conditions is presented below:

- Topsoil comprising silty clayey sand approximately 0.3m thick was encountered at the surface of BH3.
- Roadbase 0.3m thick was encountered at the surface of BH2.

- Fill comprising clayey silty sand and silty sandy clay was encountered at the surface of BH1 and BH4, and extended to depths of 0.7m and 0.4m respectively. The fill was assessed to be both poorly and moderately compacted.
- Natural silty clay was encountered beneath the fill/topsoil/roadbase in BH1 to BH4, and from surface in BH5 to BH7. The silty clay was generally of high plasticity, decreasing to medium plasticity with depth, and of very stiff or hard strength. A layer of sandy clay of low to medium plasticity and hard strength was encountered beneath the silty clay at a depth of 3.2m in BH7.
- Weathered shale bedrock was encountered beneath the silty clay in BH1, BH2, BH3 and BH6 at depths between 1.8m (BH1) and 4.7m (BH3). The shale bedrock was generally of low to medium strength, except in BH6, where very low to low strength shale was encountered to 7.5m depth.
- Weathered sandstone bedrock was encountered below the shale in BH2 and BH3 at depths of 5.4m and 5.3m respectively and extended to the borehole termination depths. Weathered sandstone bedrock was encountered below the silty clay in BH4, BH5 and BH7 at depths between 2.8m (BH4) and 4m (BH5), and extended to the borehole termination depths. The weathered sandstone was generally of low or medium strength.
- Groundwater was generally encountered at depths between 4m and 4.8m below ground levels shortly following completion of drilling each individual borehole. 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 moisture content and Atterberg Limit test results generally confirmed our field assessed soil classification. The moisture content of the rock chip samples tested correlated reasonably well with our field assessed rock strengths.

## 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 basement excavations will encounter the silty clay profile over the upper 2m to 3m and then extend into the underlying shale or sandstone bedrock.

The soil cover should be readily excavatable using conventional earthworks equipment (eg. hydraulic excavators). Some of the underlying weathered shale or sandstone 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 or sandstone which would be encountered for the deeper excavation 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. However, 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 of the neighbouring buildings and structures 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 during rock excavation should be made once the final excavation details are known. However, we 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 rock 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 or sandstone 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 rock 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 lateral earth pressure distribution, with an 'active' earth pressure coefficient, $K_a$, of 0.3, for the soil profile and extremely weathered 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 bedrock, assuming a horizontal retained surface.

- A bulk unit weight of $20kN/m^3$ should be adopted for the soil profile and extremely weathered 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 kPa for the soil profile and the extremely weathered 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 kPa for the soil profile and extremely weathered 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 rock. This value assumes excavation is not carried out within the zone of influence of the toe, 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 rock 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, we recommend that a site classification of Class 'H' be adopted, in accordance with AS2870. We note, however, that 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 or hard residual silty clays. The footings may be designed for a maximum allowable bearing pressure of 150kPa.

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

We suggest geotechnical engineering inspections be carried out during footing excavation to confirm the allowable bearing pressures.

### 4.5.3 Pile Footings

Bored piles founded at least 0.3m into the underlying 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 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.

We recommend that geotechnical engineering inspections be carried out during pile drilling to confirm the allowable bearing pressures.

### 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 separated 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 circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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

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SOIL TEST SERVICES

ABN 43 002 145 173



## 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 % |
| 1 | 5.50-6.00 | 11.2 | | | | |
| 2 | 5.00-5.20 | 6.6 | | | | |
| 2 | 5.50-6.00 | 6.9 | | | | |
| 3 | 6.60-7.00 | 8.8 | | | | |
| 4 | 5.40-5.60 | 7.7 | | | | |
| 5 | 0.50-0.95 | 20.6 | 60 | 23 | 37 | 14.5 |
| 5 | 5.00-5.20 | 7.8 | | | | |
| 6 | 5.80-6.00 | 10.5 | | | | |
| 7 | 3.90-4.50 | 8.7 | | | | |
| 7 | 5.00-5.20 | 7.5 | | | | |

**Notes:**
- The test sample for liquid and plastic limit was oven-dried(50$^{o}$C) & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

# BOREHOLE LOG

**Borehole No.** 1

1/1

**Client:** EG FUNDS

**Project:** PROPOSED DEVELOPMENT

**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z

**Date:** 14-2-08

**Method:** SPIRAL AUGER JK550

**R.L. Surface:** N/A

**Datum:**

**Logged/Checked by:** M.G./

| Groundwater Record | SAMPLES (ES, U50, DB, DS) | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | | | FILL: Clayey silty sand, fine to medium grained, brown, with fine to medium grained igneous gravel. | M | (L) | | GRASS COVER |
| | | N = 13 2,6,7 | | | | FILL: Silty sandy clay, high plasticity, dark brown, with fine to medium grained igneous gravel. | MC>PL | (St) | | APPEARS MODERATELY COMPACTED |
| | | | 1 | | CH | SILTY CLAY: high plasticity, dark orange red, with fine to medium grained gravel. | MC>PL | VSt | 350 300 | |
| | | N > 25 20,25/ 110mm | | | CL | SILTY CLAY: medium plasticity, grey, with ironstone bands. | MC<PL | H | | |
| | | | 2 | | - | SHALE: dark grey, with iron indurated bands. | DW | L | - | VERY LOW TO LOW 'TC' BIT RESISTANCE |
| | | | 3 | | | | | | | |
| AFTER 2 HRS | | N > 25 13,25/ 120mm | 4 | | | | XW | VL | | |
| | | | 5 | | | | DW | L | | LOW RESISTANCE |
| ON COMPLETION | | | 6 | | | | SW | L-M | | LOW TO MODERATE RESISTANCE |
| | | | 7 | | | END OF BOREHOLE AT 7.0m | | | | |

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# BOREHOLE LOG

Borehole No.
**2**
1/1

**Client:** EG FUNDS
**Project:** PROPOSED DEVELOPMENT
**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z
**Date:** 14-2-08
**Method:** SPIRAL AUGER JK550
**R.L. Surface:** N/A
**Datum:**
**Logged/Checked by:** M.G./

| Groundwater Record | SAMPLES (ES, U50, DB, DS) | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | | | FILL: Roadbase gravel. | M | MD | | APPEARS WELL COMPACTED |
| | | | | | CH | SILTY CLAY: high plasticity, orange red. | MC>PL | VSt | - | |
| | DS | N = 13<br>5,4,9 | | | | | | | 450<br>400<br>490 | |
| | | | 1 | | CL | SILTY CLAY: medium plasticity, grey mottled red, with ironstone bands. | MC<PL | H | | |
| | DS | N = 31<br>8,13,18 | | | | | | | >600<br>>600<br>>600 | |
| | | | 2 | | | | | | | |
| | DS | SPT<br>25/100mm | 3 | | | | | | | |
| | | | 4 | | | | | | | |
| | DS | SPT<br>25/50mm | | | - | SHALE: dark grey, with iron indurated bands. | DW | L | | LOW 'TC' BIT RESISTANCE |
| AFTER 1 HR | DS | | 5 | | | | SW | M | | MODERATE TO HIGH RESISTANCE |
| | DS | | | | - | SANDSTONE: fine to medium grained, light red brown. | DW | | | MODERATE TO LOW RESISTANCE |
| | | | 6 | | | END OF BOREHOLE AT 6.0m | | | | |
| | | | 7 | | | | | | | |

# Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

# BOREHOLE LOG

Borehole No. **3**
1/1

**Client:** EG FUNDS
**Project:** PROPOSED DEVELOPMENT
**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z
**Date:** 14-2-08
**Method:** SPIRAL AUGER JK550
**R.L. Surface:** N/A
**Datum:**
**Logged/Checked by:** M.G./

| Groundwater Record | Samples (ES, U50, DB, DS) | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | | | TOPSOIL: Silty clayey sand, fine to medium grained, light brown, with root fibres and igneous gravel. | M | (L) | | |
| | | | | | CH | SILTY CLAY: high plasticity, dark orange red, with a trace of root fibres. | MC>PL | VSt | - | |
| | DS | N = 10<br>2,4,6 | | | | | | | 480<br>490<br>420 | |
| | | | 1 | | | | | | | |
| | | | | | CL | SILTY CLAY: medium plasticity, grey red mottled, with ironstone bands. | MC<PL | H | | |
| | DS | N = 24<br>4,10,14 | | | | | | | >600<br>>600<br>>600 | |
| | | | 2 | | | | | | | |
| | DS | N > 25<br>15,25/<br>50mm | 3 | | | | | | | |
| | | | 4 | | | | | | | |
| ▶ | | | | | | | | | | |
| | DS | | | | - | SHALE: dark grey, with iron indurated bands. | DW | L | | LOW 'TC' BIT RESISTANCE |
| | | | 5 | | | | SW | M | | MODERATE RESISTANCE |
| | DS | | | | - | SANDSTONE: fine to medium grained, dark red and light grey banded. | DW | L | | LOW RESISTANCE |
| | | | 6 | | | SANDSTONE: fine to medium grained, light grey. | SW | M | | |
| | | | | | | | | | | MODERATE TO HIGH RESISTANCE |
| | DS | | 7 | | | END OF BOREHOLE AT 7.0m | | | | |

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# BOREHOLE LOG

Borehole No. **4**

1/1

**Client:** EG FUNDS

**Project:** PROPOSED DEVELOPMENT

**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A

**Date:** 4-3-08 **Datum:**

**Logged/Checked by:** M.G./

| Groundwater Record | Field Tests | Depth (m) | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|
| | | 0 | | FILL: Silty clayey sand, fine to medium grained, dark brown, with root fibres and fine to coarse grained igneous gravel. | M | | | MULCH COVER<br>APPEARS POORLY COMPACTED |
| | N = 10<br>4,4,6 | | CH | SILTY CLAY: high plasticity, dark brown, with fine to medium grained sandstone gravel and root fibres. | MC>PL | VSt | - | |
| | | 1 | | SILTY CLAY: high plasticity, dark orange, with ironstone bands. | | | | |
| AFTER 4 HRS | N = 27<br>7,12,15 | 2 | | SILTY CLAY: high plasticity, light grey mottled red, with ironstone bands. | | H | >600<br>>600<br>>600 | |
| | SPT<br>25/50mm<br>REFUSAL | 3 | - | SANDSTONE: fine to medium grained, light grey. | DW | L | - | LOW 'TC' BIT RESISTANCE<br>MODERATE RESISTANCE |
| ON COMPLETION | | 4<br>5 | | SANDSTONE: fine to medium grained, grey, with orange staining and shale bands. | SW | L-M<br>L | | LOW RESISTANCE |
| | | | | SANDSTONE: fine to medium grained, light grey and red. | | | | |
| | | 6 | | END OF BOREHOLE AT 6.0m | | | | |
| | | 7 | | | | | | |

# Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

# BOREHOLE LOG

Borehole No. **5**
1/2

**Client:** EG FUNDS
**Project:** PROPOSED DEVELOPMENT
**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A
**Date:** 4-3-08 **Datum:**
**Logged/Checked by:** M.G./

| Groundwater Record | SAMPLES (ES, U50, DB, DS) | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | | CH | SILTY CLAY: high plasticity, brown, with a trace of root fibres. | MC≈PL | VSt | | GRASS COVER |
| | | N = 12 3,6,6 | | | | SILTY CLAY: high plasticity, dark orange, with ironstone bands and a trace of root fibres. | | H | 560 >600 >600 | |
| | | | 1 | | | | | | | |
| | | N = 29 8,12,17 | | | CL | SILTY CLAY: medium plasticity, light grey, with ironstone bands. | MC<PL | | >600 >600 >600 | |
| | | | 2 | | | | | | | |
| | | N > 25 19,25/ 120mm REFUSAL | 3 | | | | | | >600 >600 >600 | |
| | | | 4 | | - | SANDSTONE: fine to medium grained, light grey and grey. | DW | L | | LOW TO VERY LOW 'TC' BIT RESISTANCE |
| ON COMPLETION | | | 5 | | | | | | | |
| | | | 6 | | | | | L-M | | LOW TO MODERATE RESISTANCE |
| | | | 7 | | | | | | | |

# Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **5**



# BOREHOLE LOG

**Client:** EG FUNDS

**Project:** PROPOSED DEVELOPMENT

**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A

**Date:** 4-3-08 **Datum:**

**Logged/Checked by:** M.G./

| Groundwater Record | SAMPLES ES | U50 | DB | DS | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | SANDSTONE: fine to medium grained, light grey and grey. | DW | L-M | | |
| | | | | | | | | | END OF BOREHOLE AT 7.5m | | | | |
| | | | | | | 8 | | | | | | | |
| | | | | | | 9 | | | | | | | |
| | | | | | | 10 | | | | | | | |
| | | | | | | 11 | | | | | | | |
| | | | | | | 12 | | | | | | | |
| | | | | | | 13 | | | | | | | |
| | | | | | | 14 | | | | | | | |

# Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

# BOREHOLE LOG

Borehole No. **6**


**Client:** EG FUNDS
**Project:** PROPOSED DEVELOPMENT
**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A
**Date:** 4-3-08 **Datum:**
**Logged/Checked by:** M.G./

| Groundwater Record | Samples (ES, U50, DB, DS) | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| DRY ON COMPLETION | ES | | 0 | | CH | SILTY CLAY: high plasticity, dark brown, with ironstone gravel and a trace of root fibres. | MC>PL | H | | GRASS COVER |
| | DS | N = 15<br>3,6,9 | | | | SILTY CLAY: high plasticity, dark orange with ironstone bands. | | | >600<br>>600 | |
| | | | 1 | | CL | SILTY CLAY: medium plasticity, light grey, with ironstone bands. | MC<PL | | | |
| | DS | N > 41<br>13,16<br>25/50mm<br>REFUSAL | | | | | | | >600<br>>600<br>>600 | |
| | | | 2 | | - | SHALE: dark grey, with iron indurated bands. | DW | VL-L | - | VERY LOW TO LOW 'TC' BIT RESISTANCE |
| | | | 3 | | | | | | | |
| | | | 4 | | | | | | | |
| AFTER 1 HR | | | | | | | | | | |
| | | | 5 | | | | SW | L-M | | LOW TO MODERATE RESISTANCE |
| | DS | | 6 | | | | | L | | LOW RESISTANCE |
| | DS | | 7 | | | | | L-M | | MODERATE RESISTANCE |

# Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

# BOREHOLE LOG

Borehole No. **6**

2/2

**Client:** EG FUNDS

**Project:** PROPOSED DEVELOPMENT

**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z

**Date:** 4-3-08

**Method:** SPIRAL AUGER JK300

**R.L. Surface:** N/A

**Datum:**

**Logged/Checked by:** M.G./

| Groundwater Record | SAMPLES ES | U50 | DB | DS | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | SHALE: dark grey, with iron indurated bands. | SW | L-M | | |
| | | | | | | | | | END OF BOREHOLE AT 7.5m | | | | |
| | | | | | | 8 | | | | | | | |
| | | | | | | 9 | | | | | | | |
| | | | | | | 10 | | | | | | | |
| | | | | | | 11 | | | | | | | |
| | | | | | | 12 | | | | | | | |
| | | | | | | 13 | | | | | | | |
| | | | | | | 14 | | | | | | | |

# Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 7

1/1

# BOREHOLE LOG

**Client:** EG FUNDS

**Project:** PROPOSED DEVELOPMENT

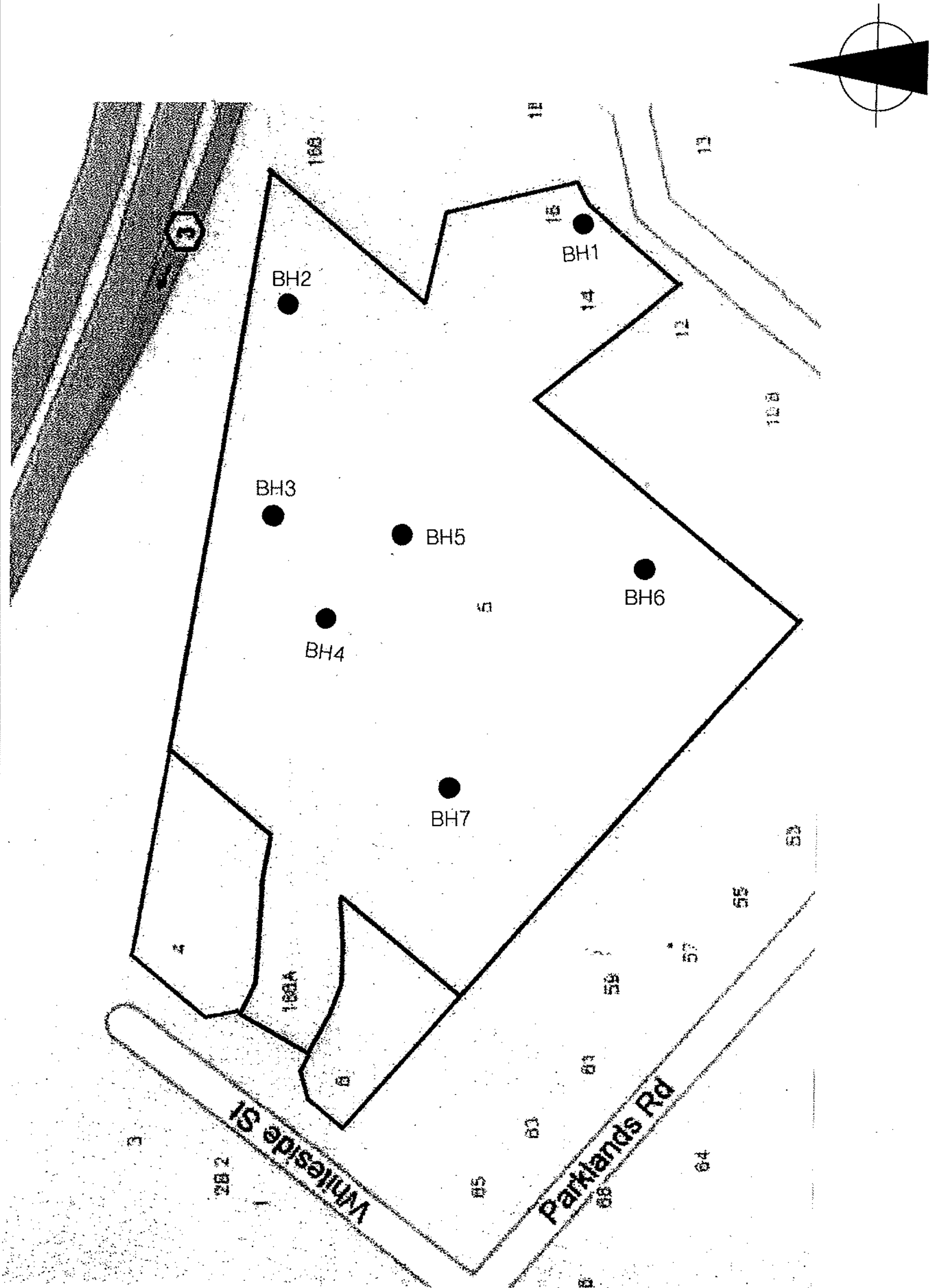
**Location:** WHITESIDE STREET AND DAVID AVENUE, NORTH RYDE, NSW

**Job No.** 21873Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A

**Date:** 4-3-08 **Datum:**

**Logged/Checked by:** M.G./

| Groundwater Record | Samples (ES / U50 / DB / DS) | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | | CL | SILTY CLAY: medium plasticity, dark brown, with fine to medium grained sandtone gravel and root fibres. | MC>PL | S | | GRASS COVER |
| | | N = 12<br>3,5,7 | | | CH | SILTY CLAY: high plasticity, grey mottled orange red. | | VSt | 210<br>290<br>350 | |
| | | | 1 | | | | | | | |
| | | N = 28<br>7,10,18 | | | CL-CH | SILTY CLAY: medium to high plasticity, light grey mottled red. | | H | >600<br>>600<br>>600 | |
| | | | 2 | | CL | SANDY CLAY: low to medium plasticity, light grey, with fine to medium grained sand. | MC<PL | | | |
| | | N > 25<br>15,25/<br>50mm<br>REFUSAL | 3 | | - | SANDSTONE: fine to medium grained, light grey. | DW | L | - | LOW 'TC' BIT RESISTANCE |
| | | | 4 | | | | | | | VERY LOW TO LOW RESISTANCE |
| ON COMPLETION | | | | | | | | | | MODERATE RESISTANCE |
| | | | 5 | | | as above, but with iron indurated bands. | SW | L-M | | |
| | | | | | | END OF BOREHOLE AT 5.4m | | | | 'TC' BIT REFUSAL |
| | | | 6 | | | | | | | |
| | | | 7 | | | | | | | |




BOREHOLE LOCATION PLAN

SCALE (m)
0 50

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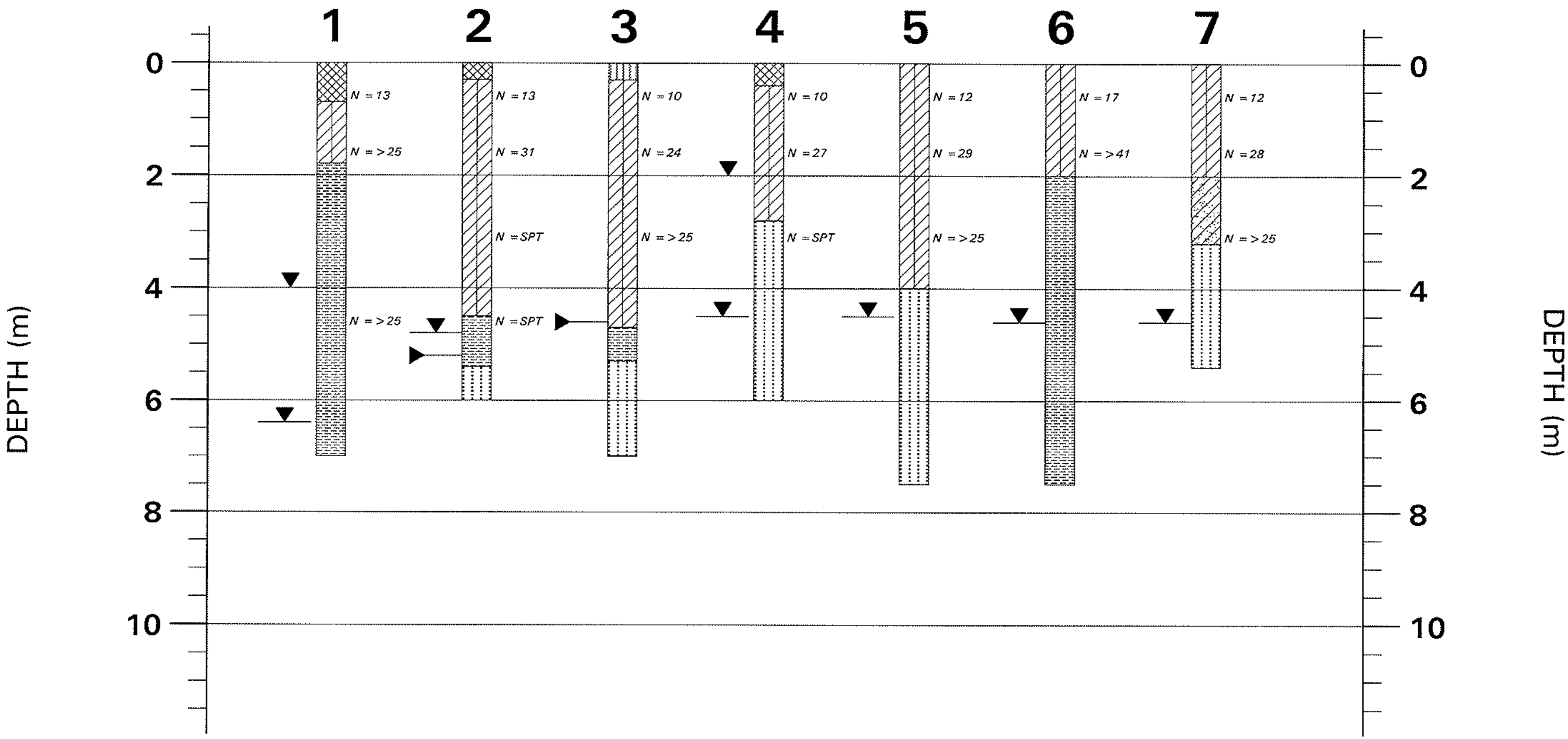
**Report No. 21873Z** **Figure No. 1**

# GRAPHICAL BOREHOLE SUMMARY




Fill
Silty Clay
Shale
Sandstone
Topsoil
Sandy Clay
Observed water level
Groundwater seepage level
N SPT "N" VALUE
Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 21873Z

Figure No.: 2

# Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
ABN 17 003 550 801

# 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, 30/40mm

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 sub-surface 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 <u>or</u> where only a limited investigation has been completed <u>or</u> 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

- FILL
- TOPSOIL
- CLAY (CL, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CH)
- SILTY CLAY (CL, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML)
- PEAT AND ORGANIC SOILS

## ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

## DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

## OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

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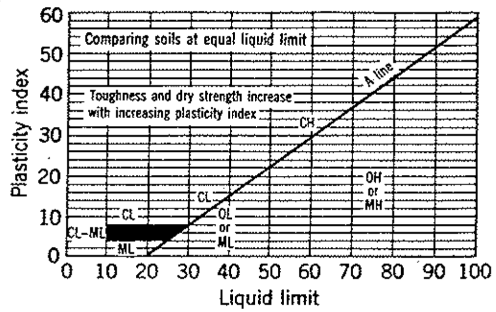
# UNIFIED SOIL CLASSIFICATION TABLE

| Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights) | | | | Group Symbols[a] | 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[b] (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<br>For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics<br>Example:<br>*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 4<br>$C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3 |
| | | | Predominantly one size or a range of sizes with some intermediate sizes missing | *GP* | Poorly graded gravels, gravel-sand mixtures, little or no fines | | Not meeting all gradation requirements for *GW* |
| | | Gravels with fines (appreciable amount of fines) | Nonplastic fines (for identification procedures see *ML* below) | *GM* | Silty gravels, poorly graded gravel-sand-silt mixtures | | Atterberg limits below "A" line, or *PI* less than 4 / Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols |
| | | | Plastic fines (for identification procedures, see *CL* below) | *GC* | Clayey gravels, poorly graded gravel-sand-clay mixtures | | Atterberg limits above "A" line, with *PI* greater than 7 |
| | 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 | | $C_U = \frac{D_{60}}{D_{10}}$ Greater than 6<br>$C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3 |
| | | | Predominantly one size or a range of sizes with some intermediate sizes missing | *SP* | Poorly graded sands, gravelly sands, little or no fines | | Not meeting all gradation requirements for *SW* |
| | | Sands with fines (appreciable amount of fines) | Nonplastic fines (for identification procedures, see *ML* below) | *SM* | Silty sands, poorly graded sand-silt mixtures | | Atterberg limits below "A" line or *PI* less than 5 / Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols |
| | | | Plastic fines (for identification procedures, see *CL* below) | *SC* | Clayey sands, poorly graded sand-clay mixtures | | Atterberg limits below "A" line with *PI* greater than 7 |

Laboratory classification criteria (coarse-grained soils): Use grain size curve in identifying the fractions as given under field identification. 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*; 5% to 12% *Borderline* cases requiring use of dual symbols

| Fine-grained soils More than half of material is *smaller* than 75 μm sieve size | Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing character-istics) | Dilatancy (reaction to shaking) | Toughness (consistency near plastic limit) | Group Symbols | Typical Names | Information Required for Describing Soils |
|---|---|---|---|---|---|---|
| Silts and clays liquid limit less than 50 | None to slight | Quick to slow | None | *ML* | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity | 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<br>For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions<br>Example:<br>*Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*) |
| | Medium to high | None to very slow | Medium | *CL* | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays | |
| | Slight to medium | Slow | Slight | *OL* | Organic silts and organic silt-clays of low plasticity | |
| Silts and clays liquid limit greater than 50 | Slight to medium | Slow to none | Slight to medium | *MH* | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts | |
| | High to very high | None | High | *CH* | Inorganic clays of high plasticity, fat clays | |
| | Medium to high | None to very slow | Slight to medium | *OH* | Organic clays of medium to high plasticity | |
| Highly Organic Soils | Readily identified by colour, odour, spongy feel and frequently by fibrous texture | | | *Pt* | Peat and other highly organic soils | |




Plasticity chart
for laboratory classification of fine grained soils

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.

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## 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<br>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<br>7<br>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) | 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. |
| (Cohesionless Soils) | 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) | | **Density Index ($I_D$) Range (%)** — **SPT 'N' Value Range (Blows/300mm)** |
| | VL | 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<br>250 | Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise. |
| 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. |

Ref: Standard Sheets/Log Symbols
November 2007

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# 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 | | Easily remoulded by hand to a material with soil properties. |
| ------------------------ | ------------- | 0.03 | |
| Very Low: | VL | | May be crumbled in the hand. Sandstone is "sugary" and friable. |
| ------------------------ | ------------- | 0.1 | |
| Low: | L | | 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. |
| ------------------------ | ------------- | 0.3 | |
| Medium Strength: | M | | A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife. |
| ------------------------ | ------------- | 1 | |
| High: | H | | 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. |
| ------------------------ | ------------- | 3 | |
| Very High: | VH | | 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. |
| ------------------------ | ------------- | 10 | |
| 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 | |

Ref: Standard Sheets/Log Symbols
November 2007