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REPORT

TO

PTI GROUP

ON

GEOTECHNICAL AND HYDROGEOLOGICAL ASSESSMENT

FOR

PROPOSED RETAIL AND RESIDENTIAL DEVELOPMENT

AT

CORNER LINDFIELD AVENUE, KOCHIA LANE AND HAVILAH LANE, NSW

9 June 2010

Ref: 24013SPrpt

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1 INTRODUCTION

This report presents the results of a geotechnical desktop assessment on the likely geotechnical issues for a proposed retail and residential development at the corner of Lindfield Avenue, Kochia Lane and Havilah Lane, Lindfield, NSW. The assessment was commissioned by Mr Peter Israel of PTI Group in an email dated 2 June 2010, and was carried out in accordance with our proposal (Reference P32509SP dated 2 June 2010).

The purpose of our assessment was to review available subsurface information from nearby projects and to provide our opinion on likely geotechnical issues associated with the proposed development and how such issues can be addressed during the design and construction of the development. We have also provided preliminary comments and recommendations on geotechnical and hydrogeological aspects of the proposed development.

A preliminary contamination assessment of the site has been undertaken by Environmental Investigation Services (EIS), and reference should be made to the EIS report reference E24013Krpt for details of the contamination assessment.

2 PROPOSED DEVELOPMENT

As part of our assessment we have been provided with the following architectural drawings prepared by PTI Group, Reference P194.1;

- Drawing SK03 Basement Level Plan, Revision 6
- Drawing SK04 Lower Ground Level Plan, Revision 6
- Drawing SK05 Upper Ground Level Plan, Revision 6
- Drawing SK06 Level 1 Plan, Revision 6
- Drawing SK07 Level 2 Plan, Revision 6
- Drawing SK08 Level 3 Plan, Revision 5
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- Drawing SK10 Level 5 Plan, Revision 6
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- Drawing SK17 Sections Sheet 1, Revision 4
- Drawing SK18 Sections Sheet 2, Revision 3.

From these drawings we understand that it is proposed to construct one and two levels of retail development with towers of five and six levels of residential apartments above, over two basement levels (named Basement Level and Lower Ground Level). Excavation for the basement will extend to about 89.2m AHD to achieve a finished floor level of 89.5m AHD, which relates to depths of about 5m to 9m below the existing surface levels. The basement will extend to the site boundaries.

3 ASSESSMENT PROCEDURES

Our assessment of the subject site has included the following;

- A brief site visit by a geotechnical engineer to assess existing site conditions.
- Review of subsurface information from nearby sites as a basis for preliminary comments and recommendations on the likely geotechnical issues affecting the site and proposed development.

4 SITE OBSERVATIONS

The site is located on the eastern side of the ridgeline passing in a north-south direction through Lindfield, with the surface in the vicinity of the site generally sloping down to the north-east at about 3° to 4°.

The site is irregularly shaped and is bounded to the west, south and east by Lindfield Avenue, Kochia Lane and Havilah Lane, with the exception of the south-eastern corner of the site where there is another property at the Kochia Lane/Havilah Lane intersection. The Avenue and Lanes are asphaltic concrete surfaced with concrete kerbs and gutters and concrete footpaths.

The property to the south-east of the site contains a two storey brick building over a lower parking area which is near the level of Havilah Lane at its eastern end, and cut about 2.5m below the adjacent level at the western end of that site.

To the north of the eastern end of the site is an on-grade asphaltic concrete carpark. To the north of the western end of the site are two brick terrace buildings which have been converted into retail. These structures appear to be in fair condition, with minor cracking in the brickwork at roof level on the front walls of these structures. These structures extend to the common site boundary.

The existing structures on the subject site comprise two and three level brick masonry retail developments and two level concrete frame structures with brick infill panels. These structures appear to be in fair condition.

5 SUBSURFACE CONDITIONS

The Sydney 1:100 000 Geological Series Sheet indicates that the site is located within an area mapped as being underlain by Ashfield Shale, but is close to the Hawkesbury Sandstone region which is just to the east/downhill.

Based on nearby investigations, we expect the subsurface conditions would comprise the following;

- Surficial fill will be present over some areas of the site, with slightly deeper fill in some areas possibly up to a depth of about 1m;
- Residual silty clays of high plasticity and typically very stiff strength;
- Weathered shale bedrock at relatively shallow depths of about 1m to 2m or so. We note that the weathered shale is likely to be of very poor quality, generally no stronger than very low to low strength and is likely to contain significant thicknesses of extremely weathered shale and shaly clay, but with stronger sandstone, siltstone and ironstone seams and bands;
- Shale bedrock of medium and high strength from depths of possibly 3m to 4m below the existing surface levels;
- Only minor groundwater seepage will probably be encountered within the excavation.

6 COMMENTS AND RECOMMENDATIONS

6.1 Geotechnical Issues

In our opinion from a geotechnical perspective, the proposed development is suitable for the subject site and will involve relatively common construction techniques and methodologies carried out on many sites throughout Sydney and within this area. We consider that the primary geotechnical issues relating to this development will be as follows;

- There will be variable excavation conditions and the requirement for retention of at least the soil and shale parts of the profile for both temporary and permanent cases. The temporary support of the shoring may require rock anchors extending beyond the site boundaries.

- We expect excavation conditions to be quite variable, comprising residual clay in the shallower areas and poor quality weathered shale with thick zones of clay, while the lower metres of excavation are likely to be in medium and high strength sandstone probably requiring the use of rock breaker attachments to hydraulic excavators. Vibration effects (associated with general excavation but more critically sandstone excavation) on adjoining structures must be considered.
- The footings are likely to found within competent sandstone bedrock.
- Minor groundwater seepage could occur above bulk excavation level and therefore drainage behind the shoring and below the basement slabs is likely to be necessary.

These issues are discussed in more detail below.

6.2 Excavation Conditions

All excavation works will need to be carried out with reference to the Workcover NSW Code of Practice – Excavation Work (Cat 312).

Prior to any works commencing on site we consider that it would be wise to carry out dilapidation reports on adjoining buildings and roads. Dilapidation reports provide a benchmark for assessing any damage claims and it is recommended that the owners of the adjoining structures be asked to sign the reports to confirm that they present a fair assessment of existing conditions.

Excavation for the basement level will be to maximum depths of about 9.0m below the existing surface levels. We expect that the upper portion of the excavation will be through fill and residual soils and then into the weathered shale of up to very low to low strength, with medium and high strength sandstone expected in the deeper portions of the excavation. Excavation of soils and weathered rock up to very low strength will be able to be achieved with the buckets of larger hydraulic excavators.

If very low to low or low strength shale is encountered then this will probably require some light ripping with say a ripping tyne fitted to a large excavator. Shale or sandstone of medium or higher strength is likely to prove effectively unrippable and will require the use of hydraulic rock breakers fitted to excavators or a combination of sawing the sandstone into blocks with large diameter excavator mounted saws and ripping of the blocks with ripping tynes. We expect large hydraulic rock breakers will be used and therefore full-time quantitative vibration monitoring should be undertaken to confirm vibrations on adjoining structures are within tolerable limits.

6.3 Excavation Batters and Retention

As the excavation will extend to the site boundaries, temporary batters will not be feasible in this instance and permanent shoring will be required.

Where sandstone bedrock of medium or high strength is encountered within the proposed excavation depth, it will be difficult to pile through this material to found below the base of the proposed excavation and consideration could be given to founding above excavation level and providing additional anchors for lateral support. The shoring system will probably comprise soldier or contiguous pile walls probably with two levels of anchors for the temporary case, and possibly soil nail walls where there are no structures adjacent to the site. Further details of these options are provided below.

Anchored Soldier or Contiguous Pile Wall

If a pile wall is to be adopted, this could be soldier pile wall with reinforced shotcrete infill panels where it is adjacent to roadways and carparks. Where the pile wall will be adjacent to other buildings, a stiffer contiguous pile wall is preferable. Such walls should be socketed into the medium to high strength sandstone and, where this is above the excavation level, at least two rows of temporary anchors will be needed until permanent support can be provided by the floor slabs. We note that it would be necessary to obtain permission from neighbours where anchors are to extend beyond the site boundaries. Further specific advice on lateral earth pressures for design of an anchored soldier pile shoring system can be provided after specific subsurface investigations on the site are carried out. However as a guide a trapezoidal earth pressure of 6H kPa (where H is the depth of the excavation in metres) could be used for preliminary design purposes; this maximum pressure should be assumed to apply over the central 60% of the height of the shoring, tapering to zero at the crest and toe of the shoring. Where there are settlement sensitive structures or services within a distance equal to the depth of shoring from the excavation, the maximum earth pressure magnitude should be increased to 8H kPa. Appropriate surcharge loads and hydrostatic pressures are additional to the above. Depending on seepage levels and flows it may be necessary to pile using grout injected Continuous Flight Auger (CFA) techniques. Geotechnical inspections should be completed during the drilling of representative shoring piles to confirm the piles are extending to adequate depths and into appropriate strata.

It is likely that the sandstone bedrock of medium and high strength would be effectively self supporting, though this should be confirmed by investigation when access to the site is possible. Even if the sandstone is self supporting, geotechnical inspections will be necessary on each lift of excavation to observe for potentially unstable wedges of rock in the face which may require stabilisation.

Soil Nail Wall

An alternative to a soldier pile wall would be to use soil nailing with a reinforced shotcrete face. This system involves excavating to a depth of about 1.5m, installing a grid of rock bolts, tying mesh to the rock bolts and spraying the face with shotcrete. Following the application of the shotcrete, the next 1.5m excavation can be conducted, and the process repeated to the base of the excavation or until better quality bedrock is encountered. On this site we expect that the soil nailing and shotcrete will probably extend to the top of the medium and high strength sandstone. Such a system has been found to be quite cost effective on other similar sites such as this. The soil nail option would be subject to further specific geotechnical design, but the following could be used for preliminary costing purposes. Assume:

- The soil nails will be on a grid of 1.5m both vertically and horizontally;
- The upper soil nail must not be more than 0.5m from the surface;
- The lowest soil nail must not be more than 0.8m from the base of the shotcrete;
- The soil nail lengths would be approximately equal to the height of material to be retained (for a vertical face), slightly less if the face is laid back at about 4V in 1H , however soil nails would have an absolute minimum length of 4m;
- As the soil nail support will extend beyond the boundary, it will not be possible to use it as a permanent retention system and it will be necessary to brace separately constructed retaining walls from the slabs of the proposed structure.
- Shotcrete thickness would probably be 120mm to 150mm with centrally located SL82 mesh.

For this soil nail option permission would also need to be obtained from neighbours prior to installing soil nails into their property.

We would be pleased to carry out a soil nail wall design for you once the site specific subsurface investigations have been carried out.

6.4 Footing Design

Following bulk excavation, the exposed subsurface conditions in the base of the excavation are likely to expose sandstone bedrock of medium and high strength. Footings embedded into such rock should be suitable for an allowable bearing pressure of 3500kPa, though this is subject to confirmation following site specific investigation. Higher bearing pressures may also be proved feasible following investigation.

6.5 Hydrogeological Considerations

Minor seepage could be encountered above the bulk excavation level and may occur during or immediately following periods of extended wet weather. Therefore long term drainage of the basement should be allowed. This would comprise construction of subsoil drainage behind the shoring system and any retaining walls as well as subsoil drainage connected to a permanent failsafe sump and pump system placed below the car park floor slabs.

Seepage into such excavations is usually associated with seepage at the soil/bedrock interface, from perched water trapped in localised depressions in the bedrock and from general seepage flows from within fractures in the rock mass. Considering the expected low permeability of the site soils and weathered rock, we consider that removal of seepage is very unlikely to adversely affect nearby structures.

6.6 Car Park Floor Slabs

As discussed above we expect that bulk excavation will probably expose medium and high strength sandstone bedrock. Therefore the design of the car park floor slabs should incorporate a subbase layer of DGB20 or similar crushed rock, compacted to at least 98% of Standard Maximum Dry Density (SMDD). This will act as a separation/debonding layer from the weathered rock subgrade. Sand layers should not be used below trafficable slabs.

Joints in concrete pavements should be dowelled or keyed to resist shear forces but not bending moments.

6.7 Geotechnical Subsurface Investigations

The above comments and recommendations have been based on nearby investigations and experience. Prior to detailed design we recommend that a comprehensive geotechnical investigation be carried out on the subject site to assess the specific subsurface conditions and to provide an updated geotechnical report suitable for design purposes. We envisage at least 5 cored boreholes should be completed. In addition we recommend PVC piezometer standpipes be installed in two of the boreholes to check groundwater levels. At present access for a drill rig is quite limited and therefore at least some demolition would be required to enable access for drilling equipment.

7 GENERAL COMMENTS

The recommendations presented in this report are based on our nearby investigations. Therefore it is possible that the subsurface conditions on the subject site 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.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

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. 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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P Wright
Senior Associate

Reviewed by:

P Wright.
per.

P Stubbs
Principal
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

Jeffery and Katauskas Pty Ltd

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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.

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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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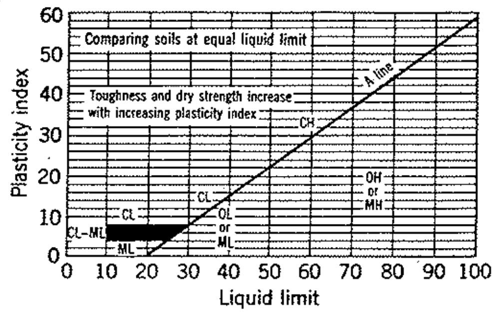
UNIFIED SOIL CLASSIFICATION TABLE

Major divisions			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 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*)	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	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ 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 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ 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	

Major divisions		Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	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 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*)
		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 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)	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	300 250	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	