REPORT

TO

WATERBROOK AT GREENWICH PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED PRIVATE HOSPITAL

AT

1-8 NIELD AVENUE, GREENWICH, NSW

9 May 2008
Ref: 22027VTrpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of a proposed private hospital at 1-8 Nield Avenue, Greenwich, NSW. The investigation was commissioned by Mr Ben MacGibbon of Murlan Consulting Pty Ltd on behalf of Waterbrook At Greenwich Pty Ltd by email dated 18 March 2008, in response to our proposal Ref: P15374VTFax.

The proposed development will involve demolition of existing houses and improvements followed by the construction of a private hospital. We understand that the hospital is to comprise six building levels over an in-ground basement with a finished floor level at RL 82.5m. The proposed recreation centre in the south-east corner will be at RL 82.0m. The base of the pool will be at about RL 80.5m. The development will include construction within residential lots and will encompass the western end of Nield Avenue. Construction will require graded bulk excavation to about 12m (maximum) depth; locally deeper excavation would presumably be required for, footings, service trenches, and lift wells. Structural loads have not been supplied and therefore, light to moderate loads have been assumed for this type of development.

The scope of the investigation was limited to obtaining information on subsurface conditions at seven locations, nominated by Murlan Consulting Pty Ltd and as shown on Figure 1, as a basis for comments and geotechnical recommendations to assist the structural engineers and builders with the design and construction of the proposed development, including excavation, retention, groundwater issues, footing and floor slab design.

A summary of the principal geotechnical issues for the proposed development is provided in Section 4.1.

2 INVESTIGATION PROCEDURE

The investigation comprised the drilling of seven boreholes using small crawler mounted drill rigs (JK250 and JK300). The borehole locations, as shown on Figure 1, which is based on the supplied survey plan of the site, were set out by taped measurements from the inferred site boundaries and surface features. The locations of the boreholes were partly dictated by access constraints imposed by existing, vegetations and trees. BH8 could not be drilled due to the presence of parked cars, buried services and overhead obstructions. Prior to drilling, all test locations were checked by a specialist sub-contractor for buried services using electronic detection equipment, after referring to Dial Before You Dig services drawings.

The boreholes were auger drilled to depths ranging from 7.25m to 9.0m below existing levels. BHs 1, 4, 6 and 10 were extended by rotary diamond coring techniques, using an NMLC triple tube core barrel with water flush, to termination at depths between 10.87m and 16.0m.

The apparent compaction of the fill and strength of the subsurface soils was assessed from Standard Penetration Test (SPT) 'N' values supplemented by hand penetrometer readings on recovered split tube clayey samples. The strength of the weathered rock was assessed from observations of the auger penetration resistance using a tungsten carbide (TC) bit, together with examination of the recovered rock cuttings and subsequent laboratory moisture content tests. The strength of the cored bedrock was assessed by examination of the recovered rock core and subsequent correlation with the results of rock strength testing.

Monitoring for groundwater was carried out in the boreholes during and on completion of individual boreholes. No longer term monitoring of groundwater levels has been carried out.

Our geotechnical engineer, Mr Joseph Chaghouri, and our geotechnician, Mr William Wijaya, set out the borehole locations, nominated the sampling and testing, and prepared the borehole logs. The surface levels, as shown on the borehole logs, were interpolated from the spot levels shown on the supplied survey plan (Reference No. 70200) prepared by Rygate & Company Pty Ltd, and as such, should be considered as approximate. The datum is Australian Height Datum (AHD). The borehole logs are included with this report, together with a Standard Set of Notes, which describes the methods and procedures employed in the investigation and their limitations and the logging terms and symbols used.

Selected disturbed samples were recovered from the site and returned to Soil Test Services (STS), a NATA registered laboratory, for moisture content, Atterberg Limit and linear shrinkage tests. The test results are summarised in the attached Table A. The rock core was also returned to STS, where it was photographed and selected sections of core subjected to Point Load Strength Index Tests ($I_{s(50)}$). The core photographs are attached opposite the relevant borehole log and the Point Load Strength Index tests are indicated on the borehole logs and are summarised in Table B.

Environmental screening of the site soils was outside the agreed scope of the investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

We recommend that the following summary of our observations should be read in conjunction with Figure 1, which shows the locations of the existing road, buildings, and some other site features throughout the development site area.

The site is located within hilly topography, which generally slopes towards the south and south-west. The detail survey indicates that ground surface levels fall across the site from around RL 92m to RL 94m at its eastern periphery down to about RL 87m at its north-west corner to about RL 73m towards the southern end of its western boundary. Nield Avenue falls to the west from about RL 98.5m at its intersection with Pacific Highway to about RL92.5m opposite the eastern site boundary, to about RL 86.5m at the western end of the cul-de-sac.

The site consists of several residential lots and will encompass the western end of Nield Avenue. Three properties are on the north-west side, four are on the south-east side, and three are on the western side of Nield Avenue. At the time of the fieldwork, the site had been substantially modified to form building platforms, access driveways, and terraced landscaped gardens.

No. 1 Nield Avenue contains a two storey, brick and rendered house, with a driveway in its north-west corner. The driveway slopes to the north-west at around 5° to 10° and is bounded by a brick retaining wall on its eastern side. The wall is about 1.0m to 1.8m high, retaining the moderately sloping landscaped front yard to the east. A stepped path cuts through the wall providing access to the front of the house. The rear yard is terraced with the upper terrace retained by a timber wall, about 1m high. There is a shed in the south-east corner of the yard. A timber log wall between about 0.5m and 2m high runs along the western boundary of No.1.

No. 2 Nield Avenue is located at the toe of the log wall and is occupied by a two storey, rendered house. The front and rear yards are terraced and retained by brick, concrete and stone walls, generally less than 1m high. The brick wall on the uphill side of the driveway is in a poor, cracked and leaning condition. Apart from the driveway, the ground surface generally falls about 3m to 4m down to the west.

There is a single storey brick house in No.3 Nield Avenue, with a driveway towards its western side. The front and rear yards are grassed, with gardens and several trees. Ground surface levels fall from about RL 89m on the eastern boundary to RL 87m in the north-west corner and to RL 84m in the south-west corner.

Nos 4A and 4B Nield Avenue contains a one and two storey brick building with elevated timber decks on its western side. The northern end of the front yard is relatively flat (with a carport), then falls away steeply to the south at around 45° to 60°. The driveway along its eastern boundary is cut into the hillside and slopes down towards the south. Further to the west of the house, the ground surface falls away steeply, generally down to the west and south-west to a drainage easement. In the north-west corner, the ground rises steeply from the easement to the north-west. This lot is heavy vegetated with several trees.

The one and two storey brick houses in Nos. 5 and 6 Nield Avenue are located towards the north-west end of the lots. The rear yards are relatively flat and at the toe of masonry and brick retaining walls, 1m to 2m high. These walls support the ground in the neighbouring property to the north-west. The front yards contain driveways, landscaped terraces retained by minor walls, grassed areas, gardens and trees. The ground surface in the front yards slopes up to the south, rising about 2m in level towards the street frontage. The south-west portion of No. 5 also slopes steeply down to the south.

No. 7 Nield Avenue is occupied by a brick house with retained terraces on its western and northern sides. The walls supporting the terraces are generally less than 1m high. The large front yard is flat to gently sloping, heavily vegetated, and contains a concrete driveway in poor condition. There is a public concrete pathway between No. 7 and 7A to the east. No.7A contains a two storey brick house, a detached garage and carport, landscaped areas, trees, and a concrete path. Ground surface levels fall steeply from RL 92m in the north-east corner, towards the south to around RL 87.5m at the garage, then is relatively flat. The south-east corner rises steeply to the street frontage.

The brick house on No. 8 Nield Avenue is located on a moderately sloping site which falls from about RL 93m in its north-east corner to about RL 88m adjacent to the south-west corner of the house. The lot contains concrete parking areas in its south-east and south-west corners and a timber walkway to the house. The northern and western sides of the parking area are retained by minor walls. A 1m high retaining wall also runs along the eastern boundary, retaining the site to the east.

The buildings generally appear to be in a fair to good structural condition, although some are in a poor, cracked condition. A variety of trees are scattered throughout the site area and in the island at the end of the cul-de-sac in Nield Avenue, which is surfaced with asphaltic concrete, with concrete kerbing.

The site is bounded by a public walkway and reserve to the north and north-west, blocks of flats to the east and south, and houses to the west.

3.2 Subsurface Conditions

Reference should be made to the borehole logs for specific details of the significantly variable subsurface conditions encountered at each test location. A graphical summary of the borehole information is presented in Figure 2.

In general terms, the boreholes encountered existing pavements, topsoil/fill, shallow, and in places, deep fill over residual silty clays, which grade into weathered bedrock at depths between 2.5m and 7.0m below existing levels. The more pertinent details of the encountered variable subsurface conditions are presented in the following.

Existing Pavements: Reinforced concrete, 100mm and 130mm thick, was encountered from ground surface in BHs 9 and 3, respectively. In BH3, the concrete covered cement mortar and concrete pavers, 100mm in thickness.

Fill: The fill consisted predominantly of silty clay of low to medium plasticity, with localised layers of silty sand, clayey sand and sandy clay. The fill contained varying amounts of gravel and building rubble (glass, ash and slag fragments), and root fibres. Based on the SPT tests and our observations, the fill was assessed to be variably compacted, mainly in the poorly to moderately compacted range. The fill was encountered to depths between 0.5m and 0.75m below existing levels in BHs 3, 6 and 7, increasing to 2.6m in BH4, to 3m in BHs 9 and 10, and to 4.5m in BH1. The fill overlies silty clays.

Residual Silty Clays: The residual silty clays were of medium plasticity with varying sizes and proportions of ironstone and shale gravel. Apart from BH3, the silty clays were predominantly of very stiff to hard strength, with moisture contents generally greater than the plastic limit. In places (BHs 9 and 10), the silty clay was firm to stiff at the base of the existing fill. In BH3, the clays were of soft to firm strength. The silty clays graded into shale.

Weathered Shale Bedrock: The shale was generally on first contact, extremely to distinctly weathered. Poor quality (interbedded hard silty clay/extremely low strength or extremely low to very low strength) shale was generally penetrated at 5.8m in BH1, 6.05m in BH3, 3.45m in BH4, 1.5m in BH6, and at 5.0m in BH10. In BHs 7 and 9, the shale was initially of very low to low strength. The shale generally improved in strength with depth. Low strength or stronger shale was intersected in BHs 1, 4, 6, 7, 9 and 10 at depths ranging from 4.0m to 7.0m and contained extremely weathered bands in places. Iron indurated bands and sandstone laminae were generally distributed through the shale profile of extremely low to low strength.

Reasonable quality medium to high strength shale was encountered at 9.5m in BH1, at 8.7m in BH4, at 8.8m in BH6, and at 9.75m in BH10.

The rock was cored from 8.9m in BH1, 8.72m in BH4, 7.25m in BH6 and 7.35m in BH10. Defects within the cored rock included some extremely weathered seams or clay seams (between 3mm and 200mm thick), or bedding planes, and some (25° to 90°) joints. The core loss zones are inferred to be extremely weathered seams or fractured bands.

Groundwater: The boreholes were 'dry' both during and on completion of auger drilling. The groundwater was not measured after coring as the introduction of water during coring obscures groundwater measurements and is unlikely to be the groundwater level. No long term groundwater monitoring was carried out.

3.3 Laboratory Test Results

The moisture content tests on samples of the rock correlated well with our field assessment of rock strength. The approximate Unconfined Compressive Strengths (UCS) of the rock core, as shown on Table B, varied significantly from 4MPa to 46MPa for the shale, with an average of about 20MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Issues and Further Work

Based on the results of this limited subsurface investigation carried out, the principal geotechnical issues for the development are summarized to be as follows:

- The existing fill is generally variably compacted. We are unaware of records that document the manner of placement, compaction specification, and control of the fill. Hence, the fill is considered to be "uncontrolled". The site would generally be classified as Class "P" in accordance with AS2870. This fill should not be relied upon to provide foundation support to footings and on-ground floor slabs unless it is fully re-compacted (or replaced) to an engineering specification in a controlled manner (refer to Sections 4.2.3 and 4.2.4).

- The proposed development will presumably involve substantial changes to the site including demolition of the existing buildings, retaining walls and other structures, pavements, and excavations of substantial volumes of soil and rock. Good engineering design, construction and maintenance practices should be adopted to maintain stability to adjoining buildings and structures during excavation and in the long term, as well as reducing the risk of vibration damage to adjoining buildings and structures during excavation.
- Groundwater was not encountered during auger drilling. The use of water flush techniques during coring precluded further meaningful groundwater observations. Where the basement is proposed, localised groundwater inflow may occur through defects in the shale exposed in the shale cut faces and the shale floor. We recommend the installation of slotted PVC pipes in additional boreholes to allow further and longer term monitoring of groundwater levels.
- Any proposed lightly loaded structures not connected to the building may be supported on footings founded below the existing fill, either fully within the residual silty clays, or fully within the shale bedrock; we prefer the latter foundation. Any structure founded within the clays should be isolated from structures with footings founded in the bedrock.
- The proposed building of moderate to high loads should be founded on the underlying shale bedrock. Where bedrock is exposed or at shallow depth after site earthworks, pad or strip footings may be used, but piles will be required where the depth to rock is deeper than about 1.5m.
- The proposed pavements may be constructed on an uncontrolled fill subgrade, provided it is prepared and proof rolled as detailed in Section 4.2.3. However, even following proof rolling, and treatment as required, of the fill there will still be a risk of poor pavement performance due to the underlying uncontrolled fill.

The only way to reduce such risks would be to excavate and replace the uncontrolled fill below the pavement area.

Further comments on the above and other issues are provided within the following sections of this report. A summary of additional geotechnical work recommended are provided in Section 5.

Although only a limited subsurface investigation was completed, we believe sufficient information has been gained to be reasonably confident as to subsurface conditions. However, it will be essential during excavation and construction works that regular geotechnical inspections be commissioned to check initial assumptions about excavation and foundation conditions and possible variations that may occur between inspected and tested locations and to provide further relevant geotechnical advice. Irregular or 'milestone' inspections by a geotechnical engineer are often not adequate for excavation, shoring and foundation works. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections. The comments provided in this report should be reviewed following these inspections. A meeting of the design team may be of benefit in order to discuss the geotechnical issues and solutions in more detail.

4.2 Earthworks

4.2.1 Subgrade Preparation and Excavation

Should any large trees require removal, we recommend they be removed well in advance of construction to allow for readjustment of the moisture content of the moderately plasticity (reactive) clay subsoil materials. Removal of any large trees should also include the removal of the tree stumps.

Following this, subgrade preparation for the proposed building areas will require clearance of any other vegetation followed by stripping of root affected topsoil. These materials may be stockpiled or taken off-site as they are not suitable for re-use as engineered fill.

Where floor slab support is required, the existing fill should also be excavated at least 2m beyond the perimeter of the slab, if possible, and re-compacted to form a properly compacted, engineered fill (refer to Sections 4.2.3 and 4.2.4).

Excavation and re-compaction of the fill would not be required where slabs are to be fully suspended and do not rely on the fill for support.

Any remaining existing fill may be left in place below proposed pavements on the condition that the subgrade is proof rolled and appropriately treated. However, there is a chance that some settlement may still occur under pavements bearing on the existing fill, even after it is treated by proof rolling.

The soils can be readily excavated by a small to medium size excavator, a front end loader or dozer. Excavation in extremely low to low strength shale can normally be achieved using either a Caterpillar D7 dozer or equivalent, with some light to medium ripping, or by a ripping hook fitted to medium to large excavators. Much of this material can probably also be excavated using a large bucket excavator. However, localised stronger iron indurated or ironstone bands/zones were encountered in the poorer quality shale, which will require the use of heavier specialised equipment (eg rock hammers or larger dozers or heavy ripping).

Excavation through the shale of medium to high strength will be more difficult, requiring large rock saws in combination with heavy ripping using at least a Caterpillar D10 or similar dozers. A generous allowance should be made for rock hammer assistance to the ripping. Hydraulic rock breaking equipment would also be suitable and would be required for detailed excavations such as footings or services.

The excavatability of the rock and the selection of appropriate excavation equipment have been assessed on the basis of the rock core strength and limited information on the nature and inclination of rock defects. Assessment of excavation characteristics and productivity is not an exact science and contractors must make their own evaluation based on experience with specific equipment, preferably after inspection of the rock cores (we only store these for one month after the formal report is issued unless other arrangements are made). The ease with which excavation of rock is achieved depends upon the equipment used, the skill and experience of the operator and the characteristics of the rock. The contractor must make his own judgement on all of these factors.

The use of heavy rock breakers will cause noise and vibrations. Depending on the locations of buildings and other structures in relation to the excavations, electronic vibration monitoring (i.e. measurement of peak particle velocities) may be required during the period of excavation. As an initial guide, we recommend that peak particle velocities should not exceed those recommended on the attached Vibration Emission Design Goals sheet for buildings in good condition or for heritage buildings. This limit of vibrations should be reviewed once more definite details of the excavation and development staging are known to confirm that they are still suitable. By monitoring vibrations in this way, it will allow some freedom to the excavation contractor in the equipment he adopts, so that a balance can be made between productivity and vibration reduction.

Vibrations induced by excavations can be reduced by alternative methods such as the following.

- Start the rock excavation away from likely critical areas.

- Maintain rock hammer orientation into the face and enlarge excavation by breaking small wedges off faces.
- Operate hammers in short bursts only, to prevent amplification of vibrations.
- Use smaller equipment (offset by a loss in productivity and economy and greater duration of the nuisance).
- Excavate a cut off trench around the site to reduce vibrations from excavation activities; this can be done progressively with the rock saw.
- Use line drilling, especially along excavation boundaries, to aid breaking and trimming.

As a very general guide, we have found on other sites that grinders or rock saws are typically required within about 5m to 10m of the buildings and structures. However the distance is very dependent on specific rock characteristics at each site, the equipment used and the condition of adjoining buildings and, therefore, vibration monitoring is essential.

In addition, we recommend that only excavation contractors with appropriate insurances and experience on similar projects be used. The contractor should also be provided with a copy of this report to make his own judgement on the most appropriate excavation equipment.

4.2.2 Excavation Batters

Construction of the proposed basement will require graded bulk excavation, with the deeper excavations on the eastern side of the site to depths of about 12m (maximum) below existing levels. The proposed excavation is to have boundary setbacks to within 2m to 3m to the east, about 6m to the north, 19m to the west, and 8m to the south. The perimeter of the excavation will be adjacent to the two and four storey buildings to the east.

The excavations in the sandy fill may be cut temporarily to a safe batter no steeper than 1 Vertical (V) in 1.5 Horizontal (H). The silty clay fill in at least a moderately compacted state, silty clay of at least very stiff strength and poor quality (extremely low to very low strength) shale may be battered at 1V in 1H. Low strength shale may be cut at 1V in 0.75H; batters in stronger rock are discussed in the following. A bench at least 2m wide should be provided where cuts in the fill, soils and poor quality shale in excess of 3mto 4m are proposed.

Surcharge loadings (footings, vehicles, etc) should not be within the zone of influence of the excavation. As a guide, surcharge loadings should be no closer than 2H from the top of any batter or the face of any excavation (including footing excavations), where H is the vertical height of the batter or depth of the excavation in the clayey fill, silty clay and low strength or weaker shale.

Flatter batters may be required in the moisture affected and softened clays of soft, firm or stiff strength or where groundwater seepage is encountered. Where possible, water should be drained away from batter slopes and prevented from discharging over batter faces.

Permanent batters would need to be flatter (that is, no steeper than 1V in 2H) and protected from erosion by vegetation or other means.

Good quality shale of at least medium strength may possibly be cut to a temporary batter of about 1V in 0.25H or slightly steeper and the face left temporarily unsupported. However, some allowance should be made for the potential larger scale instability (eg. continuous joints, etc) that occasionally exists within shale bedrock. These continuous joints can be as flat as 40° to 50° and run in north-west/south-east or north-east/south-west directions. Should these joints exist, flatter batters (possibly of the order of 1V in 1H or flatter) or large capacity rock anchors can be required; the cost of the latter would be relatively high and delays to the excavation process with consequential cost implications would occur.

The stability of battered cuts or near vertical cuts, even in good quality, medium strength or stronger shale bedrock, must be subject to confirmation by an inspection by a geotechnical engineer. No excavation face should be allowed to advance more than 1.5m vertically between inspections and the excavation should be staged or stepped so that a whole face is not excavated 1.5m vertically between visits. If adverse defects are identified by the geotechnical engineer during the inspections, then stabilisation or flatter batters will be required. If there are only occasional bedding and joint defects in the medium strength rock, the face may only require protection by dowels, mesh and shotcrete or the permanent basement walls. The extent of shotcrete to temporarily protect the rock faces prior to construction of the permanent walls should be confirmed during the geotechnical inspections. Stabilisation may also require the use of rock bolts, mesh and/or shotcrete protection to support the large blocks or other rock face areas. It would be unusual to complete such an excavation without some form of support being required to the rock faces, though this may take forms other than rock bolting.

The retaining walls would then be constructed at the toe of the temporary batters and subsequent backfilling undertaken. Caution will be required during backfilling to prevent over compaction adjacent to retaining walls and thereby causing excessive forces on the walls.

Where these batter slopes cannot be accommodated, or are not preferred, then the vertical excavation in soils and weathered shale of extremely low to low strength will need to be supported by appropriate shoring systems or properly engineered retaining walls (e.g. soldier pile walls or contiguous pile walls), with due allowance for the slope of the ground behind the walls. Any necessary vertical support system will need to be installed prior to excavation. We recommend that the vertical support system either be anchored or propped. This is discussed further in Section 4.4.

4.2.3 Fill Earthworks

Following excavation to the proposed design levels, the exposed soil subgrade should be proof rolled using a 5 tonne dead weight smooth drum vibratory roller under the supervision of an experienced earthworks superintendent, geotechnician or geotechnical engineer to check for any unstable areas. Proof rolling would not be required below floor slabs, which are to be fully suspended and do not rely on the underlying subgrade for support. During proof-rolling care should be taken to avoid vibration damage to any neighbouring structures or services or improvements. The vibrations should be monitored and the vibrations may need to be reduced or ceased if there is a risk of damage. Where unstable areas are encountered the area should be locally excavated down to a sound base and replaced with engineered fill as detailed in Section 4.2.4.

We expect that at present some sections of the exposed subgrade will comprise clays with an insitu moisture content higher than the plastic limit or have been allowed to become wet due to poor site drainage or prolonged exposure to wet periods. These subgrades may deflect significantly under proof rolling, may exhibit poor trafficability and would not be suitable for construction of new pavements or as a foundation to support building footings or slabs in their present condition. It will therefore be necessary to over-excavate such areas to below the depth of moisture 'softening' and to replace the excavated material with properly compacted engineered fill.

Allowance should be made for either tyning, aerating and drying of the subgrade after over-excavation; or lime to dry out and stabilise the subgrade, or for the use of a heavy grade geogrid/geotextile fabric to act as a bridging and separation over the excavation before placing and compacting the engineered fill. Inspection of the excavated subgrade should be undertaken by a geotechnical engineer to confirm the most appropriate method of treatment.

If 'dry' conditions prevail at the time of construction, the clayey subgrade may become desiccated or have shrinkage cracks prior to sealing with sub-base or base materials. If this occurs then the subgrade must be watered and rolled until the cracks disappear.

We recommend that reference be made to AS2870 for drainage and vegetation precautions on reactive sites.

4.2.4 Engineered Fill

Engineered fill should preferably comprise well-graded granular material (ripped or crushed shale or sandstone), free of deleterious substances and having a maximum particle size of 75mm. The sandy fill materials may be re-used, however, the clay fill and clay materials are less desirable but may be re-used provided unsuitable ('over-wet' and 'over-size') material and any deleterious material is excluded. The well-graded granular fill for backfilling excavations or for raising site levels should be compacted in layers of not greater than 200mm loose thickness, to a density between 98% and 102% of Standard Maximum Dry Density (SMDD). Clayey fill should be compacted to a similar density but within ±2% of Standard Optimum Moisture Content (SOMC). However, it would be wise to have a capping layer of better quality imported fill over the clay fill materials. The use of clay materials for engineered fill will entail more rigorous earthworks supervision and compaction control.

All platform fill or filled road embankments should either be retained or battered to a slope of compacted fill of no steeper than 1V in 2H to prevent instability. Further more detailed geotechnical assessment may be required where fill is to be in excess of 2m to 3m in depth or where fill is to be placed on 'steep' batters. The fill should also be 'keyed in' the existing side batters. All engineered fill areas should be over-filled and compacted and then the loose outer face of the fill should be cut back so that only well-compacted fill remains. We recommend a horizontal compacted fill platform extend beyond the building/pavement periphery by at least 2m. All exposed fill should be protected from erosion by quickly establishing a grass cover.

Density testing should be carried out at not less than the frequencies given in AS3798. At least Level 2 testing (but Level 1 where fill is to support building footings or movement-sensitive floor slabs/pavements) of earthworks should be carried out in accordance with AS3798. Preferably, the geotechnical testing authority should be engaged directly on behalf of the client and not as part of the earthworks contract. We can complete these tests if you wish to commission us.

The earthworks recommendations provided here should be complemented by reference to AS3798.

4.3 Groundwater and Drainage

We expect that localised seepage may possibly occur into the excavations along the soil/bedrock boundary and along existing defects, such as bedding planes and joints, which we surmise exist in the rock. Localised seepage may also occur through the fill or permeable gravelly layers in the clay, especially during and following periods of heavy rainfall. We anticipate that seepage would be controllable using conventional sump and pump techniques.

Complete and permanent drainage and appropriate waterproofing are recommended for the walls and floors close to or in contact with the excavated areas.

If basement excavations are proposed, some under-floor drainage will be required for on-ground slabs constructed over the shale, though this should be reviewed following after inspection of the completed excavation. The drains should incorporate a sump and gravity or an automatic pump-out system for discharge of collected seepage to the stormwater system.

The clayey subgrade is likely to soften with an increase in moisture content. Therefore, good and effective site drainage should be provided both during construction and for long term site maintenance. Earthworks platforms should be graded to maintain cross-falls during construction. The principal aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade will also become untrafficable when wet. We recommend that if soil 'softening' occurs, the subgrade be over-excavated to below the depth of moisture 'softening' and that the excavated material be replaced with engineered fill, compacted as specified in Section 4.2.4.

4.4 Shoring Systems and Retaining Walls

A suitable method of retention to support vertical cuts, prior to bulk excavation, would be bored cast in-situ or augered, grout injected (CFA), soldier pile walls with infill panels where movement is not of concern, or alternatively, anchored contiguous pile walls.

Construction of the contiguous pile walls should be of high quality, taking the uttermost care to prevent soil loss through gaps that may occur between the piles as this would add to the possibility of settlement occurring outside the excavation. Such gaps should be rectified without delay, such as by mass concrete infill.

Conventional driven sheet-pile walls would not be suitable as there is a need to minimise noise and avoid ground vibration damage to the neighbouring buildings.

We advise that cantilevered walls may be used for supporting retained heights of around 3m to 4m and only where some higher lateral and vertical movements of adjoining ground can be tolerated. If greater height walls are required, or, where only minimal movements can be tolerated, then anchored or propped walls would normally be required.

The excavation along the eastern side of the building will require substantial cuts which should be supported by a contiguous pile wall, progressively anchored or propped during staged bulk excavation.

The piles of the shoring walls should be suitably embedded below the base of the excavation. Props or anchors will also be needed to restrain the upper sections of the walls and these must be installed progressively and immediately once the propping point has been uncovered, and prior to excavation adjacent to neighbouring structures and sensitive services which are located within the 2H zone of influence of the excavation perimeter (discussed in Section 4.2.2).

Drilling of rock sockets will be difficult through the iron indurated bands and medium to high strength rock requiring the use of heavy drilling rigs equipped with rock augers and a coring bucket. Some groundwater inflow is expected into bored pile footings and we expect that this inflow will be controllable by conventional pumping methods. Alternatively, concrete may be poured using tremie methods.

4.4.1 Retaining Wall Design Parameters

Design of the retaining walls may be on the basis of an 'active' lateral pressure coefficient, K_a, of at least 0.35 for the fill, clayey soils, extremely low, and extremely low to very low strength shale, provided some deflection is tolerable. The K value may be reduced to about 0.2 for shale of at least low strength rock. Subject to geotechnical inspection, no K values need to be taken into account for the shale of at least medium strength. Approximate bulk unit weights of 20kN/m^3 for the soils and 21-22kN/m^3 for extremely low to low strength rock may be adopted. Walls which are to be subsequently propped by the permanent structure (e.g. by the upper ground floor slab) should be designed based on a higher lateral pressure coefficient, K, of at least 0.6 (or about 0.4 for low strength shale). These coefficients assume almost horizontal ground surfaces behind the crest of the walls.

For propped or anchored walls, we recommend the use of a trapezoidal lateral earth pressure of at least 4H (kPa), where H is the retained height in metres in the soils and shale. For propped or anchored walls in areas, which are highly sensitive to lateral movement (such as adjacent to neighbouring building footings located within 2H metres of the excavation), a greater trapezoidal lateral earth pressure of at least 8H (kPa) should be used. These 4H and 8H pressures should be assumed to be uniform over the central 50% of the full, retained height in the soils and shale. Alternatively, more sophisticated computer based shoring design (such as Wallap) generally results in cost savings compared to designs based on simplified assumptions regarding earth pressure distributions. These detailed numerical analyses can model the progressively anchored or propped shoring walls as they are constructed. The lateral earth pressure coefficients nominated for the cantilever wall may be adopted to confirm the minimum depth of embedment of the wall toe and the likely order of magnitude of wall movements during the various phases of construction when using Wallap.

The recommended lateral earth pressure coefficients and trapezoidal pressures assume almost horizontal ground surfaces behind the crest of the walls. If inclined backfill surfaces are to be designed, then the above factors would have to be increased or the inclined section of backfill should be taken as a surcharge load in the design.

Applicable hydrostatic pressures should be added to the lateral earth pressures, unless specific measures are taken to introduce complete and permanent drainage of the ground behind the walls. Any surcharge affecting the walls (e.g. footings, retaining walls and their backfill, the ground slope behind the wall, etc.) should also be taken into account in design.

Anchors may be designed for an allowable bond stress of 350kPa for shale bedrock of at least low strength. All ground anchors should be proof tested to 1.3 times the working load under the supervision of an experienced engineer independent of the anchor contractor. Anchors must be bonded behind a 45° line drawn upwards from the base of the excavation. Anchor group interaction must also be taken into account. Permanent anchors should have appropriate corrosion provisions.

4.4.2 Excavation Induced Movements

It is inevitable that the excavation will induce movements of the adjacent ground that falls within the area of influence of the excavation.

Lateral and horizontal movements could occur within about 2H back from the anchored wall. With a less rigid support system, excavation induced movements should be expected to be of a higher order. Settlements may also be caused by the wall construction itself (e.g. loss of ground during anchor drilling, etc).

As excavation of the rock progresses, the rock mass will also tend to move inwards towards the excavation along bedding planes, clay seams, etc. as it is stress relieved. With increasing depth of excavation, the bed undergoing excavation will also drag overlying beds with it as the lower bed moves towards the excavation. The extent of movement will depend on the strength of the rock between the bedding planes and the spacing of joints or other defects. As the beds move inwards, joints, etc. will start opening behind the excavated face and any structures on or in the rock also move. These stress-relief movements will decrease away from the excavated face, however, their magnitude will increase as the depth of excavation increases.

Experience with excavations in residual clay and weathered shale indicates that lateral and vertical ground movements of around 2 to 5mm/m of excavation depth may occur, mostly as a result of stress relief, depending on the rigidity and construction practice of the shoring system.

It may not be practicable to prevent significant vertical and lateral ground displacements immediately beyond the limits of the excavation, so the effects of the inevitable excavation induced movements on the adjoining buildings and structures and also on the permanent structure should be assessed.

The objective with properly engineered retaining walls is to keep the adjacent ground movements within tolerable limits. The actual wall movements are highly dependent on the construction sequence, detailing and quality of installation and should be assessed by the structural engineer for the system to be adopted. Hence, any existing adjoining structures, or buried services, which fall within the area of influence of the excavations, should be assessed for risks of damage due to excavation-induced movements and whether underpinning is required. The underpinning should be designed for lateral earth pressures, any surcharge loadings and hydrostatic pressures.

The risk of architectural or structural damage to adjoining buildings and structures will depend on their sensitivity to horizontal and vertical deformations, structural load, type and founding elevations of the floor slabs and footings and foundation conditions. All these factors should be carefully investigated and evaluated prior to excavation commencing.

In addition, we recommend that an excavation/retention methodology be prepared prior to bulk excavation commencing. The methodology must include but not be limited to proposed excavation, retention and underpinning techniques, the proposed excavation equipment, excavation/retention/underpinning sequencing, geotechnical inspection intervals or hold points, vibration monitoring procedures, monitor locations, monitor types, contingency plans in case of non-compliance. Preferably, this methodology should be shown on the structural engineer's drawings. The excavation/retention/underpinning methodology should be reviewed and approved by the geotechnical engineer.

4.5 Footing Design

Footings should be uniformly founded on either silty clay of at least very stiff strength or uniformly on the shale to limit the potential for differential settlements. Any structure founded within engineered fill or clays should be isolated from structures with footings founded in the bedrock.

4.5.1 Footings on Engineered Fill and Natural Clays

The recommendations given in the following assume that the existing fill will be full re-compacted or replaced with engineered fill in accordance to recommendations provided in Sections 4.2.3 and 4.2.4.

The residual silty clays have a moderate potential for shrink-swell reactive movement. If any lightly loaded footings are founded in the residual silty clays, we recommend that they should be designed to cater for shrink-swell movements equivalent to those experienced on a "Class M" site (i.e. about 20mm-40mm free surface movements).

If existing trees are to be removed or if the site is to be filled with reactive clays (eg. excavated from elsewhere on-site), the effect of the readjustment in soil moisture in the underlying clays should be carefully assessed. Should any large trees require removal, we recommend they be removed well in advance of construction to allow for readjustment of the moisture content of the moderately reactive silty clay subsoil. Removal of any large trees should also include the removal of the tree stumps.

Shallow footings, including the edge and internal beams of stiffened raft slabs, founded within natural clay of at least very stiff strength may be designed for an allowable bearing pressure of 200kPa for an embedment of at least 0.7m (or deeper to suit the type of structure in accordance with AS2870) below the surrounding ground surface. A lower bearing pressure of 150kPa may also be adopted for footings founded in a building platform consisting of properly and uniformly compacted engineered fill prepared and compacted in accordance with the procedures outlined in Section 4.2 and under Level 1 geotechnical supervision. The footing embedment in the engineered filled platform should generally be not less than 0.7m. However, the effects of reactive movements and the latter footing embedment depth should be reviewed if on-site or reactive clay materials are used as fill or if less than 1m of granular fill covers the underlying natural silty clays. Reference should also be made to AS2870 for design, construction, performance criteria and maintenance precautions on reactive clay sites.

Flexible and movement tolerant forms of construction should be adopted. Attention is drawn to other precautionary, site and foundation maintenance measures outlined in AS2870.

4.5.2 Footings on Bedrock

The building and retaining walls should preferably be supported by strip or pad footings, or bored, cast in-situ piles or augered, grout injected piles founded in the underlying shale bedrock.

A possible further pile alternative could be steel screw piles, which could have similar working bearing pressures to a grout injected pile. However, the working bearing pressure is dependent on the pile diameter and embedment depth as well as the strength/stiffness of the pile itself. Consideration should be given to long term corrosion and advice should be sought from the manufacturer. Also it is important to ensure that steel screw piles can penetrate to achieve an adequate embedment into the weathered shale.

Strip and pad footings or bored piles or augered, grout injected (CFA) piles may be designed for maximum allowable working bearing pressures for the shale given in Table 1.

Table 1 – Footing Bearing Pressures and Depth

Borehole Number	Depth (in metres) below existing ground level for Safe Bearing Pressure of 700kPa	Depth (in metres) below existing ground level for Safe Bearing Pressure of 1500kPa	Depth (in metres) below existing ground level for Safe Bearing Pressure of 3500kPa
1	7.3	8.3	9.2
3	6.5	8.0	-
4	4.0	7.3	9.5
6	6.1	7.7	9.0
7	2.8	4.3	-
9	5.8	8.3	-
10	5.5	8.1	9.3

Rock sockets in piled footings below the indicative founding levels specified above may be designed for a safe adhesion value of 10% of the appropriate safe bearing pressure under compressive vertical loading. Two-thirds of these adhesion values may be adopted in uplift. These adhesion values assume excavation is not carried out within the zone of influence of the footing. The bearing and adhesion values assume footing bases have been cleaned of loosened or softened materials and sockets are free of smeared material (a special roughening tool is normally required to achieve this in bored piers).

For footings fully embedded into the underlying bedrock below the lowest building floor level, an allowable lateral stress in the rock socket equal to one third of the allowable bearing pressure may be adopted. These passive resistance values assume excavation is not carried within the zone of influence of the wall toe and the rock does not contain unfavourable defects etc. The upper 0.3m depth of the socket should not be taken into account to allow for disturbance effects during excavation.

Where footings are founded close to the top of a rock face, the allowable bearing pressure below these footings will need to be carefully assessed. The safe bearing pressure would need to take into account rock strength, the inclination of the rock face, jointing and the influence of clay seams as well as the magnitude and inclination of the applied loadings.

If the designer wishes to adopt the limit state design methods, such as in the Piling Code, AS2159-1995, then the ultimate values of end bearing pressure may be estimated by multiplying the above recommended allowable bearing and lateral stress values by Factors of Safety of 3. A Factor of Safety of 2 should be applied to the shaft adhesion values. We recommend that the ultimate values be multiplied by a geotechnical strength reduction factor, Φ_g, of 0.5. Higher reduction factors may be adopted but these will depend on the intensity and type of proving of the footings and their foundation. An appropriate load factor should also be applied to the proposed footing loadings.

The rock bearing pressures given in Table 1 are based on a serviceability criteria of deflections at the footing base/pile toe of less than or equal to 1% of the least footing dimension (or pile diameter). Footing settlements may be estimated using the Elastic Moduli given in Table 2.

Footings on rock can also be designed using 'Limit State Design' principles as detailed in the paper "Foundation on Sandstone and Shale in the Sydney Region' by Pells, Mostyn and Walker, Australian Geomechanics, Number 33, Part 3, December 1998 (Pages 17-29). It must be emphasised that the use of limit state design to adopt relatively high bearing pressures (above the serviceability criteria described above) is not currently standard practice, and there is an increased risk of inadequate footing performance.

Table 2 – Elastic Moduli for Footings in Rock

Strata	Bulk Unit Weight (kN/m^3)	Poisson's Ratio	Elastic Modulus (MPa)
Shale – extremely low to very low strength with iron indurated bands	22	0.25	100 – 150
Shale – low strength	23	0.25	400 – 500
Shale – low to medium strength	23	0.2	500 – 700
Shale – medium or medium to high strength	23	0.2	1000 – 2000

If construction proceeds during a relatively 'dry' period, the beams between piles should be designed to withstand potential uplift pressures associated with possible subsequent swell of the clay fill or clay subgrade as it 'wets up'. Alternatively, the beams should be underlain with void formers or similar (at least 40mm thick) to minimise the impact of uplift pressures. A degree of uplift protection can be achieved by tyning/loosing the soil below the ground beams for say 120mm depth.

4.5.3 Footing Construction

In order to minimise potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the proposed footing design and construction requirements and how to identify the weathered rock materials at the indicative founding levels so as to minimise over-drilling of the piles during construction.

If bored or augered grout piles are to be socketed into the shale then we recommend that heavy drilling rigs with rock augers be used to drill the piles. Heavy drill rigs with coring buckets may be required for drilling through medium strength or stronger rock or through the iron indurated bands.

Some groundwater seepage can be expected during the construction of piers and we recommend that trials should be undertaken to confirm piers can be successfully constructed at the site, otherwise augered, grout injected piles should be used. Piers should be dewatered (by conventional pumping methods) prior to concreting or the concrete may be poured using tremie methods.

All footings should be drilled, cleaned, inspected and poured with minimal delay, on the same day or the base of the footing should be protected by a concrete blinding layer after cleaning of loose spoil and inspection. 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.

In addition to inspection, the shale foundation may also need to be spoon tested or cored in boreholes if footings are designed using a safe bearing pressure of 3.5MPa. This testing is to confirm that seams or defects present below the founding levels are within tolerable limits. The presence of such seams would require a reduction in allowable bearing capacity or an increase in footing depth. The amount of testing should be addressed when structural design is more advanced.

The initial stages of footing excavation/drilling, particularly if bored piles are adopted, should be inspected by a geotechnical engineer/engineering geologist to ascertain that the recommended foundation material has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit.

4.6 Basement Floor Slab

On-ground floor slabs will be partly constructed over the shale and no special treatment is required other than the removal of loose and softened material. Areas, which have to be built-up to infill low points in the excavations should be filled with properly compacted sub-base material.

Although we expect that some under-floor drainage will be required, this should be reviewed following further monitoring of groundwater seepage during and on completion of the excavations. The under-floor drainage (such as perimeter drains and/or a free draining gravel bed) should be installed with sumps for gravity or automatic pumped discharge of groundwater. If under-floor drainage is not installed, then the on-ground floor slab may be subjected to uplift pressures from the groundwater; this may require additional mass or ground anchors.

The basement floor slab, where subject to traffic loadings, should have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 (1994) unbound base material (or equivalent good quality durable fine crushed rock) which is compacted to at least 100%SMDD.

4.7 Floor Slabs and Pavements

The on-ground floor slab for the buildings and pavements may be founded on the engineered fill or the proof rolled clayey subgrade on condition that the subgrade is prepared in accordance to the recommendations provided in Section 4.2.

The design of pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of new fill imported to the site, as well as vehicle loadings and use.

Lightly loaded pavements may tentatively be designed using a lower bound characteristic CBR value of 2.5% or a coefficient of subgrade reaction of 20kPa/mm (750mm plate) or a long term Young's modulus of 10MPa for the proof rolled and treated clay subgrade. These preliminary design values should be confirmed by CBR tests once initial earthworks design is complete and by inspection and testing during construction.

On-ground floor slabs should be incorporated in a stiffened slab or raft footing system designed to allow for movements in the underlying fill or silty clays, which will generally have a moderate shrink/swell potential. Slabs constructed over the treated fill or clay subgrade must be isolated from slab sections founded on the shale.

For flexible pavements, in-situ lime stabilisation of the clayey subgrade could be undertaken to reduce total pavement thickness. Alternatively, an appropriate select fill layer comprising good quality well-graded granular material may be used below the pavement.

Improvement of the subgrade CBR design value and consequent reduction of the crushed rock pavement thickness may be achieved by stabilising the clay subgrade with lime to a minimum depth of say 200mm to 300mm. To determine the optimum lime addition rate to achieve the beneficial effect desired, laboratory tests should be carried out. However, an indicative proportion to achieve a CBR of 6% would probably be the addition of 4% of quick lime by dry weight of the clay. The lime must be thoroughly mixed with the clay using specialist blending machines and then compacted to not less than 98% SMDD at ±2% of SOMC.

Only contractors experienced with lime stabilisation should be used. We note that use of lime close to pedestrian and adjacent building areas is generally not preferred unless an acceptable method of dust suppression can be adopted.

Concrete pavements and on-ground floor slabs subject to traffic loadings should be supported on a sub-base layer of RTA Specification 3051 unbound or equivalent good quality crushed rock, compacted to a density of at least 100% SMDD.

Concrete pavements should be provided with effective shear connection at joints by using dowels or keys. Concrete pavements should preferentially be used in areas where heavy vehicles manoeuvre such as garbage bin and truck unloading areas.

Subsoil drains should generally be provided on the uphill side and along the perimeter of pavements, with inverts not less than 0.3m below clay subgrade level. The drainage trench should be excavated with a longitudinal fall to appropriate discharge points so as to minimise the risk of water ponding. The pavement subgrade should be graded to promote water flow or infiltration towards subsoil drains.

5 SUMMARY OF FURTHER GEOTECHNICAL WORK

Excavation and retention recommendations provided in this report should be complemented by reference to the Code of Practice Excavation Work, Cat. No. 312 by WorkCover NSW.

As detailed in this report, further geotechnical work is recommended as follows:

- Assessment of the effects of excavation on the nearby building footings and whether underpinning is required.
- Quantitative monitoring of transmitted vibrations during rock excavation using rock hammers.
- Assessment of groundwater inflow to confirm drainage requirements following excavation. We also recommend the installation of slotted PVC pipes in boreholes to allow further and longer term monitoring of groundwater levels.
- Inspection of the excavations to confirm batter slopes and rock face treatment for cuts in the medium or higher strength rock.
- Inspection of footing excavations to ascertain that the recommended foundation has been reached and to check initial assumptions regarding foundation conditions and possible variations that may occur.
- Inspect proof rolling of fill/silty clay subgrade to detect soft spots requiring treatment.
- Carry out laboratory CBR testing of clay subgrade parameters for pavement design.
- Carry out laboratory tests to establish the optimum lime addition rates for pavement/floor slab subgrades.
- This investigation has been limited to boreholes spread throughout site and where access permitted. Additional boreholes may need to be drilled to address

particular design issues once design work is commenced and to provide a better coverage across the proposed building and to confirm the variation in depth to rock, and rock quality, especially if bored piers are adopted. For example, where it is proposed to adopt the 3.5MPa bearing pressure, additional cored boreholes may be required.

We recommend that Jeffery & Katauskas Pty Ltd view the proposed earthworks and structural drawings and section details in order to confirm they are within the guidelines of this report.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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.

The long-term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgement from an experienced engineer. Such judgement often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the completed boreholes 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.

The offsite disposal of soil will most likely require classification in accordance with the Department of Environment & Conservation (NSW) guidelines as inert, solid, industrial or hazardous waste. We can complete the necessary classification and testing if you wish to commission us. As testing requires about seven days to complete, allowance should be made for such testing in the construction program unless testing is completed prior to construction. If contamination is found to be present then substantial further testing and delays should be expected. We strongly recommend this issue be addressed prior to commencement of excavation on site.

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.

Tony Walker
Associate

QA Review by:

Fernando Vega
Senior Associate
For and on behalf of
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SOIL TEST SERVICES
ABN 43 002 145 173

Ref No:22027VT
Table A: Page 1 of 1

TABLE A
SUMMARY OF LABORATORY TEST RESULTS

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
3	1.50-1.95	13.7				
3	3.00-3.45	25.6	48	21	27	13.0
3	6.05-6.45	13.1				
3	7.00-7.50	10.6				
7	1.50-1.95	17.3	45	18	27	12.0
7	5.50-6.00	10.4				
7	7.00-7.50	9.4				
9	7.00-7.50	12.1				
9	8.50-9.00	9.0				
10	3.00-3.45	25.8	40	18	22	12.0

Notes:

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

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Ref No: 22027VT
Table B: Page 1 of 2

TABLE B
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	9.20-9.23	0.3	6
	9.84-9.88	0.8	16
	10.23-10.25	1.1	22
	10.74-10.78	0.6	12
	11.17-11.20	1.6	32
	11.75-11.79	1.2	24
4	8.80-8.83	0.6	12
	9.53-9.56	0.8	16
	10.17-10.20	0.6	12
	11.22-11.25	1.3	26
	11.71-11.74	0.9	18
6	7.36-7.40	0.3	6
	7.72-7.75	0.6	12
	8.33-8.36	0.2	4
	8.85-8.89	0.6	12
	9.29-9.32	0.8	16
	9.76-9.79	0.9	18
	10.26-10.30	2.3	46
	10.94-10.97	1.3	26
	11.28-11.31	1.1	22
	11.69-11.72	0.4	8
	12.19-12.23	1.3	26
	12.59-12.63	0.9	18

NOTES:See page 2 of 2

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Table B: Page 2 of 2

TABLE B
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
6	13.33-13.36	1.5	30
	13.77-13.80	1.3	26
	14.23-14.25	1.4	28
	14.79-14.82	1.5	30
	15.21-15.24	2.2	44
	15.82-15.86	2.2	44
10	7.79-7.82	0.4	8
	8.07-8.12	0.4	8
	8.68-8.71	0.7	14
	9.35-9.38	0.7	14
	9.76-9.79	1.1	22
	10.23-10.27	1.4	28
	10.70-10.74	1.4	28

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S(50)}$

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER
JK300
R.L. Surface: ≈ 85.9m
Datum: AHD
Logged/Checked by: W.W./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0			FILL: Silty clay, medium plasticity, orange brown, with a trace of roots.	MC>PL			GRASS COVER APPEARS POORLY COMPACTED
				■	N = 11 3,5,6	1			FILL: Silty clay, medium plasticity, light grey mottled orange brown and red brown, with fine to medium grained gravel. FILL: Silty clay, low plasticity, light grey mottled red brown.	MC>PL MC≥PL	-	520 240 >600	APPEARS MODERATELY COMPACTED
				■	N = 4 2,2,2	2			FILL: Silty clay, medium plasticity, orange brown mottled light grey, brown, with fine to medium grained gravel.	MC>PL		270 190 170	APPEARS POORLY TO MODERATELY COMPACTED
				■		3 4			as above, but with shale gravel.				LOW 'TC' BIT RESISTANCE IN SHALE BAND
				■	N = 8 4,3,5	5		CH	SILTY CLAY: medium plasticity, light grey mottled orange brown, with fine to medium grained gravel.		VSt	250 320 260	
				■	N = 23 3,8,15	6 7			SILTY CLAY/SHALE: medium plasticity, grey, with sandstone laminae.		VSt-H/EL	450 510 210	RESIDUAL

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
2/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER
JK300
R.L. Surface: ≈ 85.9m
Datum: AHD
Logged/Checked by: W.W./

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 light grey sandstone laminae and iron indurated bands.	DW-SW	L		VERY LOW 'TC' BIT RESISTANCE
													LOW TO MODERATE RESISTANCE
						8					L-M		
						9			REFER TO CORED BOREHOLE LOG				
						10							
						11							
						12							
						13							
						14							

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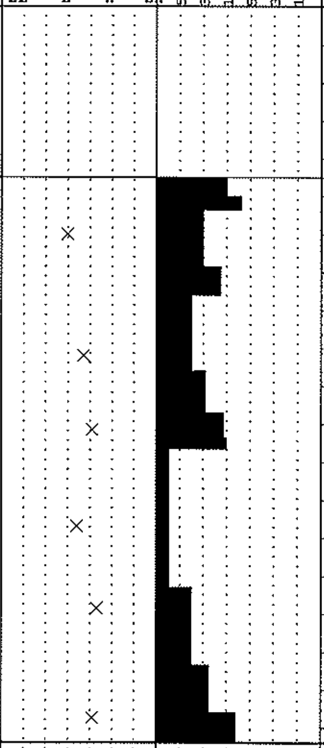
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
1
3/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Core Size: NMLC
R.L. Surface: ≈ 85.9m
Date: 1-4-08
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: J.C./



Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		8		START CORING AT 8.90m						
FULL RET-URN		9		SHALE: dark grey.	DW	L			- Be, 0°, P, R, IS - XWS, 5mm.t - Be, 15°, P, S, IS	
						M-H			- J, 45-50°, P, S	
		10							- J, 70-75°, P, S - J, 35-45°, P, R - J, 45-50°, P, S - J, 45-50°, P, S	
		11		SHALE: light grey.					- J, 30-35°, P, R - J , 25-30°, P, S - CS, 20mm.t	
		12		END OF BOREHOLE AT 11.88m						
		13								
		14								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JOB NO: 22027VT
BH1 START CORING AT 8.90m
8
START CORING AT 8.90 m
9
10
11
END
11.83m

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/2

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 83.6m
Date: 1-4-08
Datum: AHD
Logged/Checked by: W.W./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			CONCRETE: 130mm.t				CORED BY DIATUBE 6mm DIAMETER REINFORCEMENT, 50-55mm TOP COVER
									CONCRETE PAVERS AND MORTAR				
				■				-	FILL: Silty clay, high plasticity, light grey, mottled red brown, with fine to medium grained sub-angular ironstone gravel.	MC>PL	-	-	
						1		SC	CLAYEY SAND: fine to medium grained, light grey mottled orange brown and red brown, with fine to medium grained sub-angular ironstone gravel.	MC>PL	-	-	
				■	N = 13 9,10,3						St- VSt	180 210 270	
						2							
				■	N = 5 1,2,3	3		CL	SILTY CLAY: medium plasticity, light brown mottled grey, with fine to medium grained sub-angular ironstone gravel.		F	100 80	
									SILTY CLAY: medium plasticity, dark grey mottled brown, with fine to medium grained sub angular ironstone gravel.			50 70 80	
						4			as above, but brown, with shale gravel.				
				■	N = 2 1,1,1						S-F	50 30 50	
						5							
				■	N = 30 6,14,16	6		-	SHALE: grey, with iron indurated bands, clay seams, and light grey sandstone laminae.	XW	EL-VL		
						7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
2/2

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER
JK300
R.L. Surface: ≈ 83.6m
Datum: AHD
Logged/Checked by: W.W./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■ □ ■		8 9			SHALE: grey, with iron indurated bands and clay seams, with light grey sandstone laminae.	DW	VL-L		LOW 'TC' BIT RESISTANCE LOW TO MODERATE RESISTANCE
						10 11 12 13 14			END OF BOREHOLE AT 9.0m				

BOREHOLE LOG

Borehole No.
4
1/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 2-4-08
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 86.6m
Datum: AHD
Logged/Checked by: J.C./

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING		0		FILL: Clayey sand, fine to medium grained, dark grey, with fine to medium grained gravel.	M			APPEARS POORLY COMPACTED
	N = 11 2,5,6			as above, but brown.	MC≥PL		380 500 415	APPEARS MODERATELY COMPACTED
		1		FILL: Silty clay, low to medium plasticity, brown mottled orange brown, with dark grey ash and slag and fine to medium grained ironstone gravel.	MC≤PL		355 350 380	
	N = 14 6,7,7			FILL: Silty clay, high plasticity, light grey mottled orange brown, red brown, with dark grey ash and fine to medium grained ironstone gravel.			480 >600 >600	
		2		FILL: Silty clay, low to medium plasticity, brown and orange brown, with glass fragments and dark grey ash.				
			CL	SILTY CLAY: medium plasticity, light grey mottled orange brown.	MC>PL	VSt -H	-	RESIDUAL
	N = 15 3,6,9	3					330 430 430	
		4	-	SHALE: dark grey.	XW	EL-VL	-	
		5						
		6		SHALE: dark grey, with light grey and occasional orange brown sandstone laminae.	DW	VL-L		LOW 'TC' BIT RESISTANCE
		7						

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BOREHOLE LOG

Borehole No.
4
2/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 2-4-08
Method: SPIRAL AUGER
JK300
Logged/Checked by: J.C./
R.L. Surface: ≈ 86.6m
Datum: AHD

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■		8			SHALE: dark grey, with light grey sandstone laminae.	DW	L		
						9 10 11 12 13 14			REFER TO CORED BOREHOLE LOG				

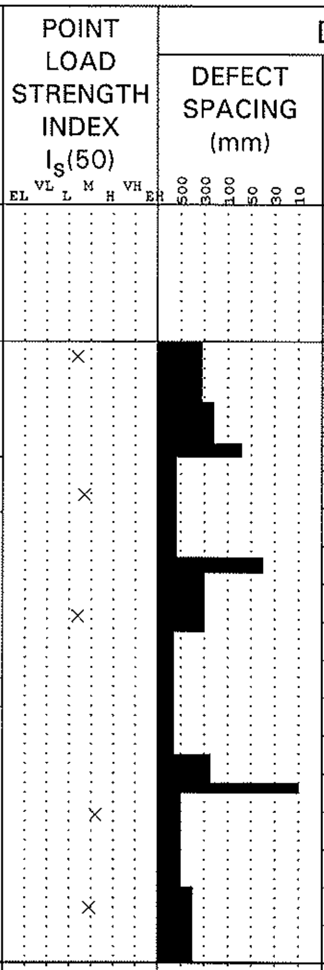
CORED BOREHOLE LOG

Borehole No. 4
3/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 2-4-08
Drill Type: JK300
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 86.6m
Datum: AHD
Logged/Checked by: J.C./

Water Loss/Level	Barrel Lift	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		8	START CORING AT 8.72m				
FULL RET-URN			SHALE: dark grey.	DW-SW	M-H		
		9				- XWS, 200mm.t - XWS, 30mm.t - XWS, 70mm.t	
					L-M		
					M-H	- J, SUBVERTICAL, P, S	
		10				- Be, 20°, P, R, IS - CS, 35mm.t - CS, 50mm.t	
				XW-DW	EL-VL		
				DW-SW	M-H	- J, 80°, P, S	
		11				- J, 65°, P, S - J, 80°, P, S - XWS, 5mm.t	
		12	END OF BOREHOLE AT 12.0m				
		13					
		14					

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JOB No: 22027 V.T BH4 START CORING AT 8.72m
8
START CORING AT 8.72 m.
9
10
11

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BOREHOLE LOG

Borehole No.
6
1/4

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 2-4-08
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 91.9m
Datum: AHD
Logged/Checked by: J.C./

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION OF AUGERING						0			FILL: Clayey sand, fine to medium grained, dark grey, with root fibres and fine to medium grained gravel.	M			GRASS COVER APPEARS POORLY COMPACTED
					N = 18 4,7,11	1		CL	as above, but brown. SILTY CLAY: medium plasticity, light grey mottled red brown, and orange brown, with fine to medium grained ironstone gravel.	MC<PL	H	430 >600 560	RESIDUAL
					N > 21 8,11, 10/50mm REFUSAL	2			SILTY CLAY/SHALE: low to medium plasticity, light grey and red brown, with VL-L shale bands.	MC<PL /XW	H/EL	500 >600 580	
						3 4 5			as above, but with M-H strength iron indurated bands.	MC<PL/ XW	H/EL		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS FRIABLE
						6 7		-	SHALE: dark grey.	DW-SW	L		LOW 'TC' BIT RESISTANCE

BOREHOLE LOG

Borehole No.
6
2/4

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 2-4-08
Method: SPIRAL AUGER
JK300
R.L. Surface: ≈ 91.9m
Datum: AHD
Logged/Checked by: J.C./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■					SHALE: dark grey.	DW-SW	M-H		
									REFER TO CORED BOREHOLE LOG				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

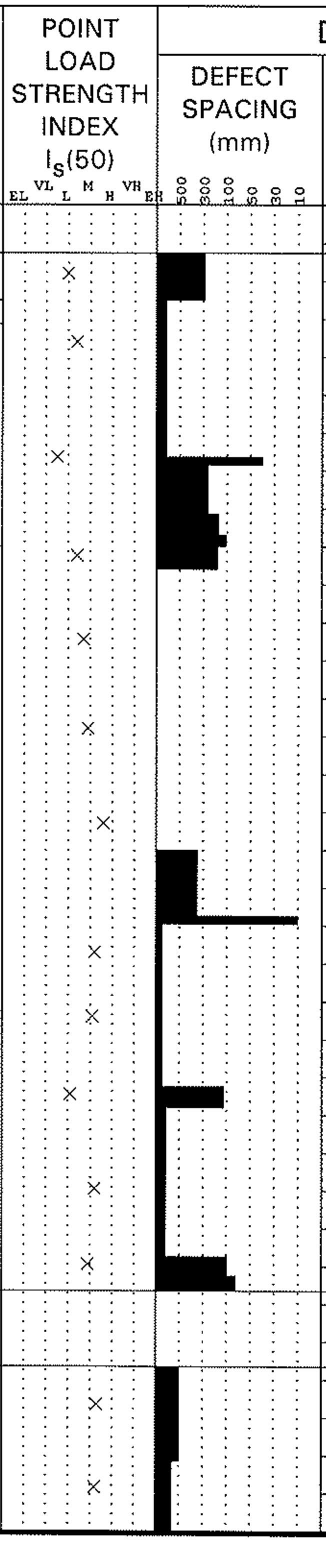
CORED BOREHOLE LOG

Borehole No. 6
3/4

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 2-4-08
Drill Type: JK300
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 91.9m
Datum: AHD
Logged/Checked by: J.C./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		7		START CORING AT 7.25m				
FULL RET- URN		8 9 10 11 12		SHALE: dark grey, with light grey laminae, bedded at 0-5°.	DW-SW SW	L-M VL-L L-M M-H	- CS, 20mm.t - VL BAND - CS, 5mm.t - CS, 3mm.t - CS, 15mm.t - CS, 3mm.t - CS, 5mm.t - Be, 0°, P, R - J, 50°, P, S - J, 65°, P, S - CS, 20mm.t - CS, 20mm.t - XWS, 3mm.t - XWS, 5mm.t	
		13		CORE LOSS 0.4m				
				SHALE: dark grey, with light grey laminae, bedded at 0-5°.	SW	H	- J, 65°, P, S	



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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

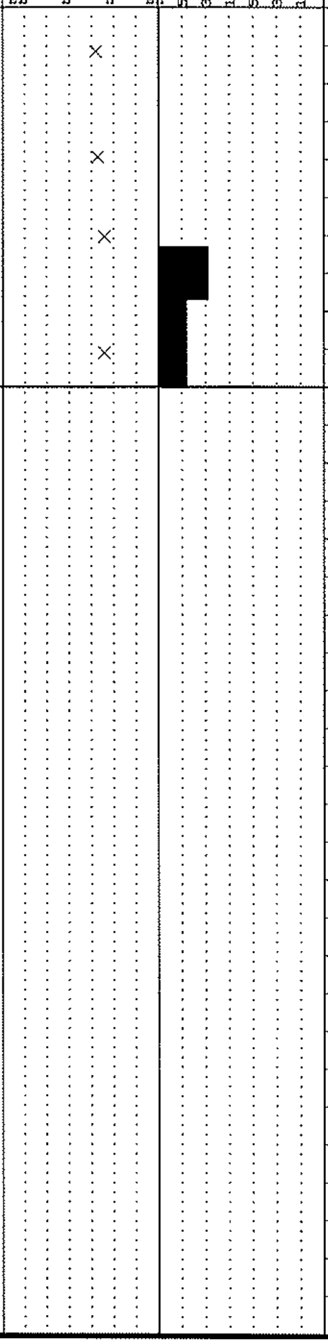
CORED BOREHOLE LOG

Borehole No. 6
4/4

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Core Size: NMLC
R.L. Surface: ≈ 91.9m
Date: 2-4-08
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: J.C./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
			▨	SHALE: dark grey, with light grey laminae, bedded at 0-5°.	SW	H	×		- J, 60°, P, S	
							×			
		15					×		- J, 75°, P, S - J, 80°, P, S	
							×			
		16		END OF BOREHOLE AT 16.0m						
		17								
		18								
		19								
		20								

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
JOB No. 22027 VT BH6 START CORING AT 7.25m.
7
START CORING AT 7.25 m.
8
9
10
11
12
CORE LOSS
13
0.4m
14
15

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
7
1/2

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER JK250
Logged/Checked by: W.W./
R.L. Surface: ≈ 87.6m
Datum: AHD

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						0			FILL: Silty clay, medium plasticity, light brown, with root fibres and fine to medium grained sub angular ironstone gravel.	MC>PL			GRASS COVER APPEARS POORLY COMPACTED
				■	N = 8 6,4,4	1		CL	SILTY CLAY: medium plasticity, brown, with fine to medium grained sub-angular ironstone gravel.	MC>PL	VSt-H	350 490 450	
				■	N = 13 5,5,8	2			SILTY CLAY: medium plasticity, light grey mottled orange brown, with fine to medium grained sub-angular ironstone gravel.	MC≈PL	VSt -H	460 310 270	
				■		3		-	SHALE: dark grey.	DW	VL-L		LOW 'TC' BIT RESISTANCE
				■		4 5			SHALE: dark grey, with iron indurated bands.		L		LOW TO MODERATE RESISTANCE
				■		6			SHALE: dark grey, with light grey sandstone laminae.				
						7					L-M		MODERATE RESISTANCE

BOREHOLE LOG

Borehole No.
7
2/2

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 87.6m
Datum: AHD
Logged/Checked by: W.W./

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 and light grey.	DW	L		
						8			END OF BOREHOLE AT 8.0m				
						9							
						10							
						11							
						12							
						13							
						14							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
9
1/2

BOREHOLE LOG

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 91.2m
Datum: AHD
Logged/Checked by: J.C./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			CONCRETE: 100mm.t				CORED BY DIATUBE 6mm DIAMETER REINFORCEMENT, 45mm TOP COVER
				■				-	FILL: Silty clay, medium plasticity, brown, with fine to medium grained gravel.	MC>PL	-	-	APPEARS POORLY COMPACTED
				■	N = 9 5,4,5	1			as above, but mottled red brown, with shale fragments and fine to medium grained sandstone gravel.			450 360 550	APPEARS MODERATELY COMPACTED
				■	N = 2 1,SUNK,2	2			FILL: Clayey sand, fine to medium grained, brown, with slag.	M			APPEARS POORLY COMPACTED
									FILL: Silty clay, medium plasticity, light grey mottled orange brown, with slag.	MC>PL			
				■	N = 6 2,3,3	3		CL	SILTY CLAY: medium plasticity, light grey mottled orange brown, with root fibres.	MC>PL	F-St	130 110 90	
				■	N = 11 3,4,7	4 5		CL-CH	SILTY CLAY: medium to high plasticity, dark grey, with EL shale bands.	MC>PL	VSt-H	300 450 480	RESIDUAL
				■		6 7		-	SHALE: dark grey, with orange brown sandstone laminae.	DW-SW	VL-L		LOW 'TC' BIT RESISTANCE

BOREHOLE LOG

Borehole No.
9
2/2

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 91.2m
Datum: AHD
Logged/Checked by: J.C./

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 light grey sandstone laminae XW bands.	DW-SW	L-M		LOW TO MODERATE RESISTANCE
				■		8	▤		SHALE: dark grey, with light grey and orange brown sandstone laminae.				MODERATE RESISTANCE
						9			END OF BOREHOLE AT 9.0m				
						10							
						11							
						12							
						13							
						14							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
10
1/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 87.7m
Datum: AHD
Logged/Checked by: W.W./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0	▨		FILL: Silty sand, fine to medium grained, dark grey, with root fibres.	M			GRASS COVER
							▨		FILL: Clayey sand, fine to medium grained, light brown.	M	-	-	APPEARS POORLY COMPACTED
				■	N = 9 3,4,5		▨		FILL: Silty clay, medium plasticity, light grey.	MC≥PL		480	
						1	▨		FILL: Silty clay, low to medium plasticity, dark brown, with fine to medium grained gravel.	MC≥PL		320 340 300 280 200	APPEARS MODERATELY COMPACTED
				■	N = 11 3,7,4	2	▨		FILL: Clayey sand, fine to medium grained, orange brown mottled red brown, with sandy clay seams.	M			
				■	N = 10 5,6,4	3	▨	CL	SILTY CLAY: medium plasticity, orange brown mottled light grey and red brown, with a trace of fine to medium grained gravel and sand.	MC>PL	F-St	70 180 70	
				■	N = 29 13,13,16	4	▨	CL	SILTY CLAY: medium plasticity, brown, light grey and orange brown, with shale bands.	MC≥PL	VSt -H	490 390 240	RESIDUAL
						5	▤	-	SHALE: dark grey, with orange brown sandstone laminae.	XW-DW	EL-VL		NO RESISTANCE
				■		6	▤			DW-SW	L-M		VERY LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						7	▤						VERY LOW TO LOW RESISTANCE

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
10
2/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Date: 1-4-08
Method: SPIRAL AUGER
JK300
Logged/Checked by: W.W./
R.L. Surface: ≈ 87.7m
Datum: AHD

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
				■			▤		SHALE: dark grey, with light grey sandstone laminae, with EL bands	DW	L-M		
									REFER TO CORED BOREHOLE LOG				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

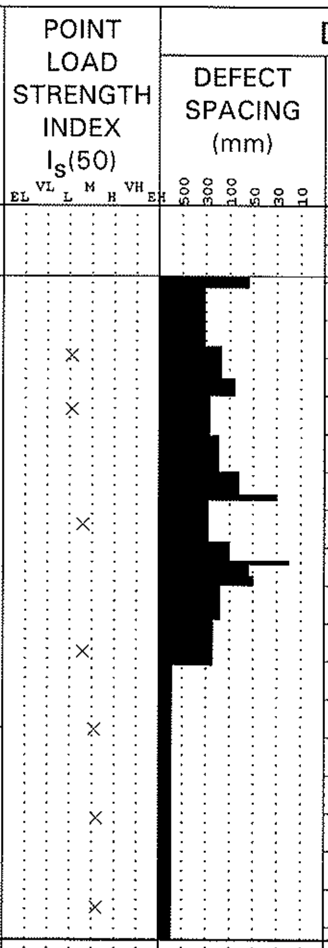
Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
10
3/3

Client: WATERBROOK AT GREENWICH PTY LTD
Project: PROPOSED PRIVATE HOSPITAL
Location: 1-8 NIELD AVENUE, GREENWICH, NSW

Job No. 22027VT
Core Size: NMLC
R.L. Surface: ≈ 87.7m
Date: 1-4-08
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: W.W./Q



Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		7		START CORING AT 7.37m				
FULL RET-URN		8 9		SHALE: dark grey, with light grey laminae, bedded at 5-15°.	DW	L-M	- CS, 300mm.t (HP 60,80,140kPa) - CS, 30mm.t - EL-VL SEAMS, 80mm.t - VL-L BAND - XWS, 5mm.t - J, 80°, P, R - CS, 20mm.t - XWS, 5mm.t - J, SUBVERTICAL, P, R - XWS, 3mm.t - XWS, 3mm.t - J, 75-80°, P, S - CS, 50mm.t - J, 40-45°, P, S - CS, 100mm.t - 2xJ, 40-45°, P, S	
		10		SHALE: light grey, with dark grey laminae, bedded at 5-10°.	SW	H	- J, 30-40°, P, S	
		11 12 13		END OF BOREHOLE AT 10.87m				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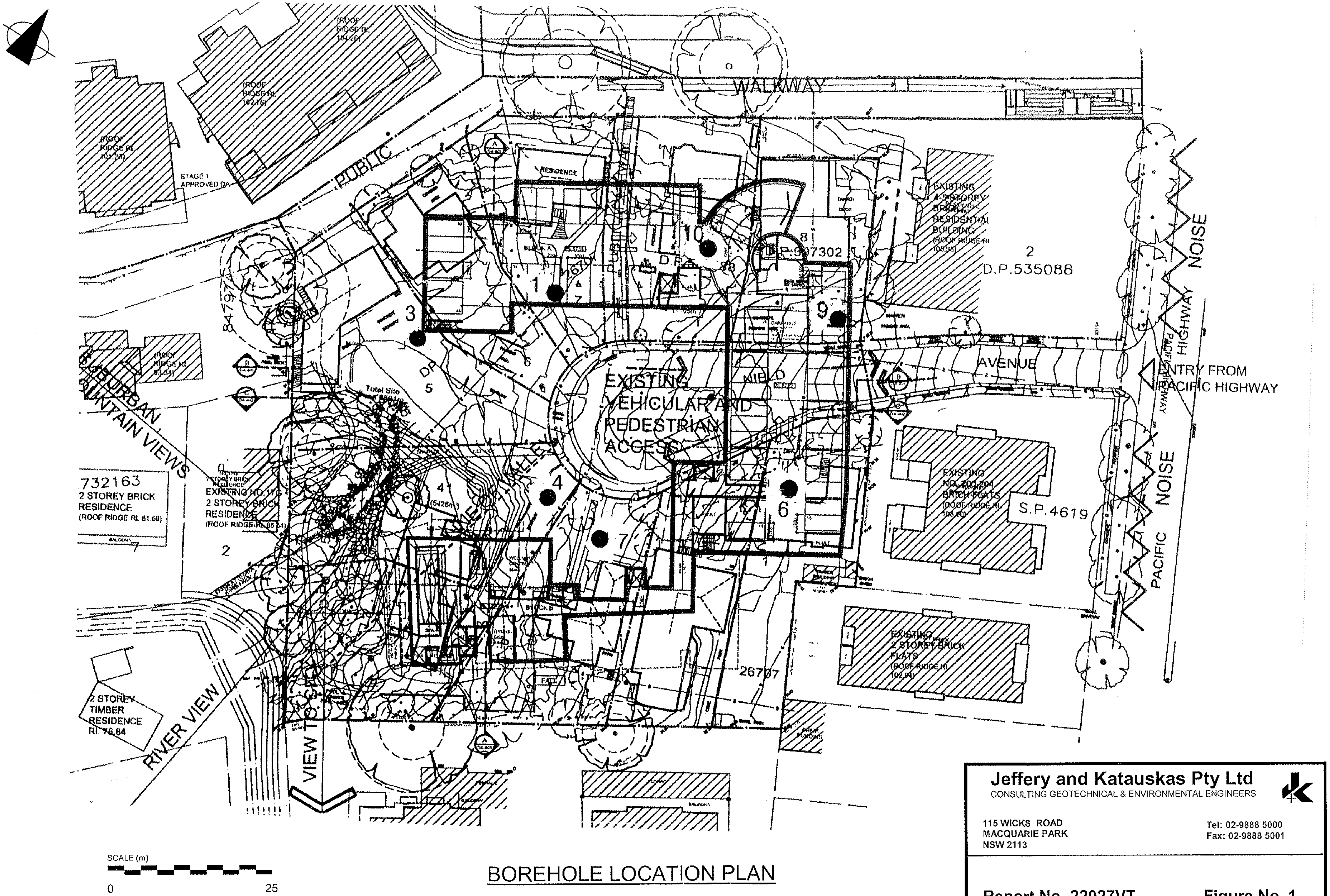
JOB NO: 22027VT
BH10 START CORING AT 7.37 m
7
START CORING AT 7.37 m
8
9
10
END BH AT 10.87m



BOREHOLE LOCATION PLAN

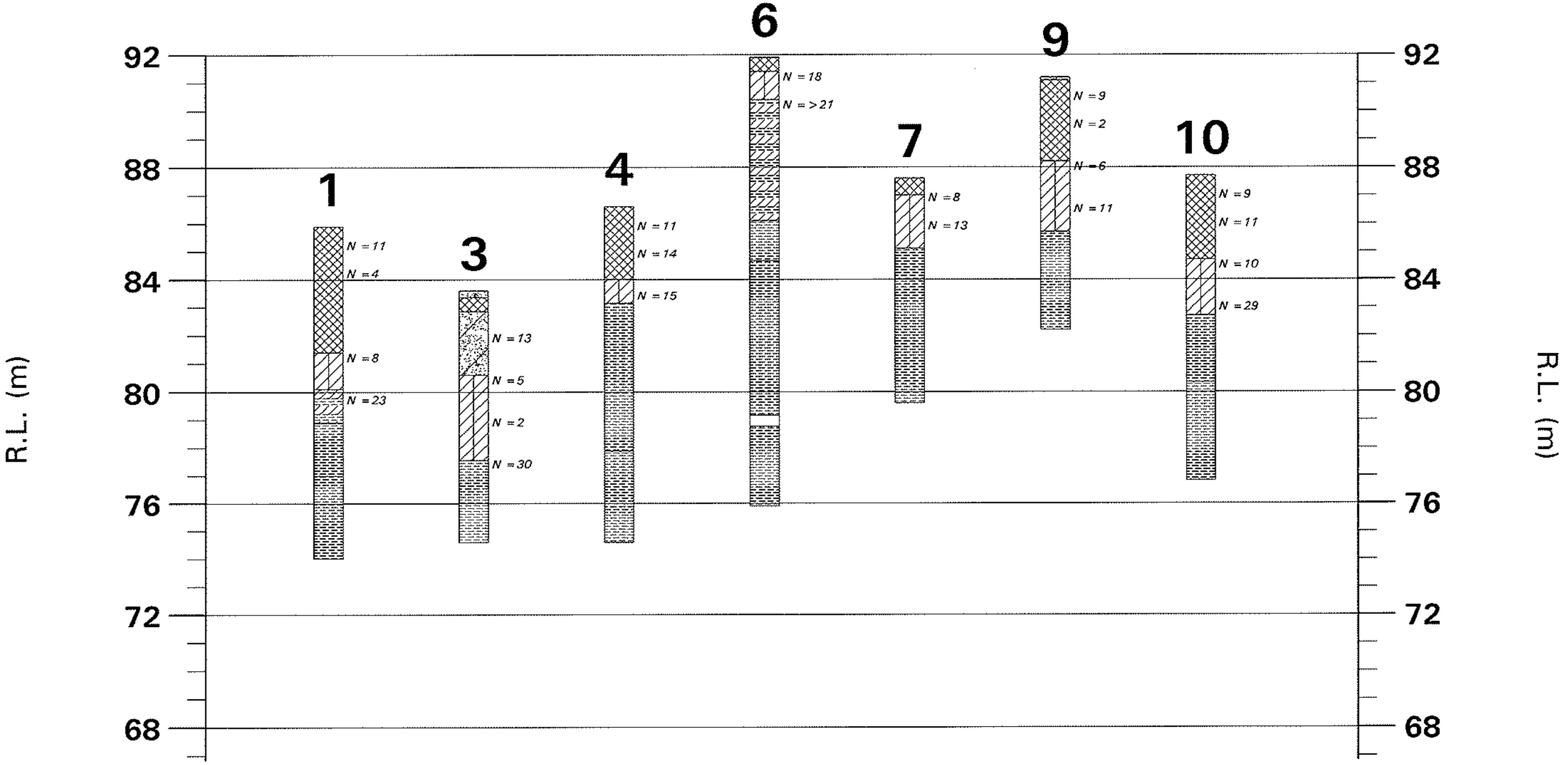
Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

115 WICKS ROAD
MACQUARIE PARK
NSW 2113

Tel: 02-9888 5000
Fax: 02-9888 5001

Report No. 22027VT **Figure No. 1**

GRAPHICAL BOREHOLE SUMMARY



▩ Fill

▨ Silty Clay

▤ Interbedded Shale and Clay

▦ Shale

▒ Concrete

▧ Clayey Sand

☐ Core Loss/ Empty

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; NTS (horiz)

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Job No.: 22027VT

Figure No.: 2

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VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1986 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite "safe", depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are "safe limits", up to which no damage due to vibration effects has been observed for the particular class of building. "Damage" is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the "safe limits" then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the "safe limits" are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1 DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level At a Frequency of			Plane of Floor of Uppermost Storey
		Less than 10 Hz	10 Hz to 50 Hz	50 Hz to 100 Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100 Hz, the higher values in the 50 Hz to 100 Hz column should be used.

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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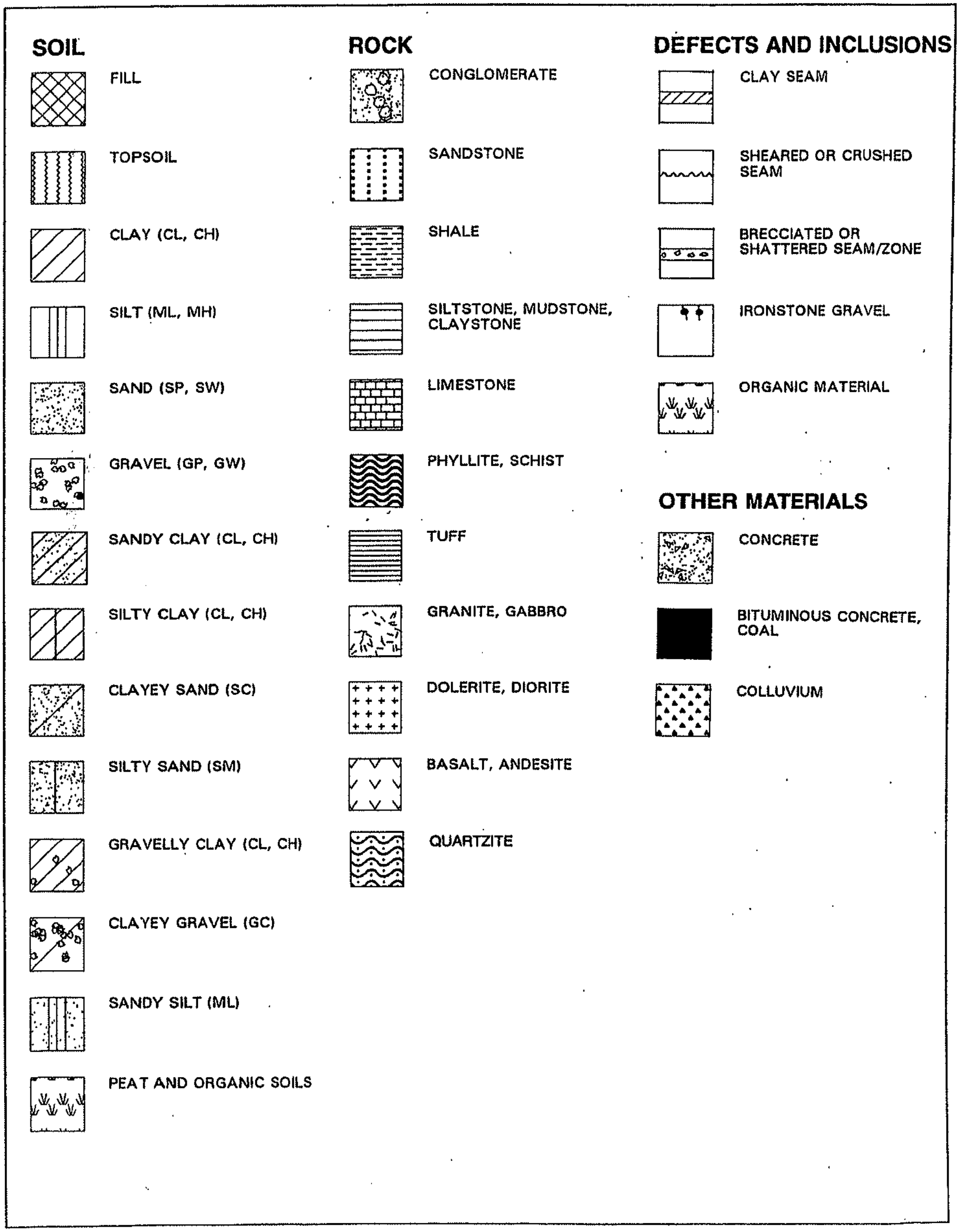
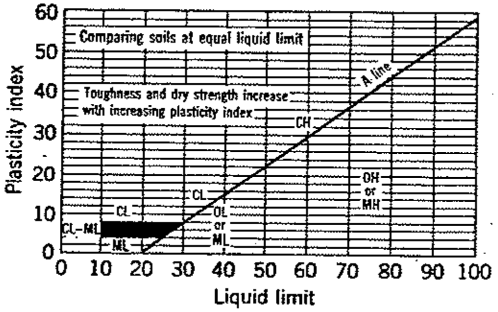
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



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			
(The 75 µm sieve size is about the smallest particle visible to naked eye)	Coarse-grained soils More than half of material is *larger* than 75 µm sieve size[b]	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	
	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 characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)							
		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*)			 Plasticity chart for laboratory classification of fine grained soils	
				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					

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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A.B.N. 17 003 550 801 A.C.N. 003 550 801

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION		
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.		
	—C—	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.		
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.		
	⊤ 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
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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	

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