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REPORT

TO

MCLACHLAN LISTER PTY LTD

ON

PRELIMINARY GEOTECHNICAL ASSESSMENT

FOR

PART 3A CONCEPT PLAN PROPOSAL AND PROJECT APPLICATION

AT

MONTEFIORE JEWISH HOME,
DANGAR STREET, RANDWICK, NSW

13 July 2010
Ref: 17167ZR3rpt

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
Postal Address: PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 • Fax: 02 9888 5003 • Email: engineers@jkgroup.net.au • ABN 17 003 550 801

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FIGURE 1: GEOTECHNICAL SITE PLAN

FIGURE 2: GEOTECHNICAL MAPPING SYMBOLS

FIGURE 3: GRAPHICAL BOREHOLE SUMMARY

APPENDIX A: SELECTED BOREHOLE LOGS FROM OUR PREVIOUS GEOTECHNICAL REPORTS

REPORT EXPLANATION NOTES

1 INTRODUCTION

This report presents the results of our preliminary geotechnical assessment for the proposed Part 3A Concept Plan Proposal and Project Application at the Montefiore Jewish Home, Dangar Street, Randwick, NSW. The assessment was commissioned on behalf of Sir Moses Montefiore Jewish Home by Mr Clive Chandler of McLachlan Lister Pty Ltd in a letter dated 24 May 2010. The commission was on the basis of our email fee proposal (Ref. P17167ZRextra) dated 19 May 2010.

We have been provided with the following information:

- Architectural plans (Drawing Numbers DA120 to DA127, DA310 and DA311, DA350, DA412 and DA413 Issue A dated 7 July 2010) prepared by Jackson Teece.
- The unreferenced "Part 3A Concept Plan" dated July 2010, prepared by Jackson Teece.

Based on the provided information we understand the proposed Concept Plan will comprise construction of three new four to six level buildings over the southern portion of the site (Blocks D, E and F) and some re-configuration of the existing Block C. The new buildings will be constructed over one level of basement with a proposed finished floor reduced level (RL) at RL41.3m (Blocks D and E) and RL38.3m (Block F), requiring excavations to a maximum depth of about 3.5m. In addition, a new tunnel will connect the southern end of Block A to the northern end of Block C. The tunnel will have a finished floor level at RL37.7m and excavations to a maximum depth of about 4m will be required. Blocks D and E will connect to the existing 'Burger Centre' which occupies the central section of the southern portion of the site.

We have not been provided with structural loads and have assumed typical loadings for this type of development.

We confirm that we have previously prepared the following reports for the site:

- Report (Ref. 7940S/vm) dated 14 February 1991, which comprised a combined geotechnical and contamination investigation.
- Report (Ref. 15587Srpt) dated 4 December 2000, which comprised a desktop study of the geotechnical information contained in the above report.
- Report (Ref. 15587S2 Let) dated 12 June 2002, which presented the results of in-situ permeability testing at the site.
- Report (Ref. 17167Srpt) dated 27 September 2002, which comprised a desktop study of the available hydrogeological information for the site, including the contents of our previous reports.
- Report (Ref. 17167S2rpt) dated 22 October 2002, which comprised an additional geotechnical investigation and included the results of previous geotechnical and environmental investigations.
- Various advice provided during 2003 contained in site reports during construction of the buildings now occupying the northern and southern central portion of the site.

For specific details regarding site conditions at the time of the above investigations and the investigation procedures adopted, reference should be made to these previous reports.

The purpose of this preliminary assessment was to review the above reports as a basis for assessing the likely subsurface conditions expected to be encountered at the site as a basis for comments and recommendations on excavation, retention, footings, hydrogeology, drainage, on-grade floor slabs and external pavements.

We note that out specialist environmental investigation services division (EIS) is preparing a report (Ref. E17167KBlet-rev1.2) dated 13 July 2010 regarding potential urban salinity risks/hazards at the site.

2 ASSESSMENT PROCEDURE

A desk-top study was undertaken and included a review of our previous geotechnical reports outlined in Section 1, above. We note that the previous reports covered the entire site area occupied by the current aged care facility. For ease of reference, we have attached the relevant geotechnical boreholes in Appendix A

A walkover assessment was carried out by an Associate level Engineering Geologist on 1 May 2009, in order to gain an appreciation of the site setting. An additional site visit was completed on 2 June 2010 to cover the area of the proposed Block F. This assessment is based upon a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs.

A summary of our observations and assessment of subsurface conditions are presented in below Sections 3.1 and 3.2, respectively.

The attached Figure 1 presents a geotechnical site plan showing the principle geotechnical features present at the site and the locations of previous boreholes included in our assessment. Figure 1 is based on a provided architectural plan. Additional features on Figure 1 have been measured by hand held inclinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. An explanation of geotechnical mapping symbols is presented as Figure 2.

In addition, a series of photographs of the site were taken and have been retained in our files for record purposes.

3 RESULTS OF ASSESSMENT

3.1 Site Observations

The site is located at the toe of a concave hillside that slopes down to the west at a maximum of 15°.

The site has southern, eastern and northern frontages onto King Street, Dangar Street and Govett Lane.

At the time of the assessment, the site was an aged care facility constructed since preparation of our geotechnical report in October 2002. The site surfaces had gentle to moderate slopes down to the west and south-west; the site surface level stepped down about 4m from the south-eastern corner of the site to the south-western corner of the site.

The northern half of the site was occupied by a maximum 5 level brick building and the central section of the southern portion of the site was occupied by a maximum four level building (The 'Burger Centre'). The buildings were surrounded by asphalt, concrete and asphaltic concrete (AC) paved access roads and footpaths, grass surfaced landscaped areas and planter beds. The paved area adjacent to the south-eastern corner of The 'Burger Centre' was uneven with some vertical displacement of the order of 10mm at selected paver interfaces.

The subject site comprises the southern portion of the site, i.e. to the east and west of The 'Burger Centre'. The pertinent site features are as follows:

- The southern portion of the eastern site boundary comprised a grass surfaced batter which sloped down to the west at a maximum of about 15°.

- The southern portion of the eastern side of The 'Burger Centre' was lined by a paved area.

- The central portion of The 'Burger Centre' was lined by an AC paved car park which extended west under the building to an access road and extended to the east. The eastern portion of the car park was lined by concrete block retaining walls of about 3.5m maximum height which supported the grass surfaced slopes and the paved area to the south.

- The northern portion of the eastern side of The 'Burger Centre' was lined by an AC paved driveway with a deck area suspended over the western side of the driveway.

- The northern subject site boundary was lined by asphalt and AC paved driveways.

- The northern and central portions of the western side of The 'Burger Centre' were lined by gently sloping landscaped areas.

- The southern portion of the western side of The 'Burger Centre' was lined by what appeared to be an elevated yard area a maximum of about 3m above surrounding landscaped surface levels. Observations were limited due to the presence of a timber screen of about 5m maximum height.

- The south-western corner of the site was occupied by child-care centre comprising clad frame buildings which were set-back about 2m from the concrete block retaining wall (maximum height about 2m) which supported the southern portion of the western side of the driveway entrance into the site.

- The child care centre was accessed from the King Street frontage by a suspended concrete deck supported on concrete columns. Below the southern end of the suspended deck a sand batter (about 2.5m maximum height) sloped down to the north at a maximum of about 40°; traces of a dilapidated steel soldier pile wall with timber infill panels was evident.

- A raised landscaped area extended north along the western site boundary from the northern end of the childcare centre. The landscaped area was supported by a concrete retaining wall (maximum height about 1.5m). A portion of the southern side of the landscaped area and the entire length of the western side of the landscaped area sloped down to the south and west at a maximum of about 30°. The remainder of the north-western portion of the site comprised a grass surfaced landscaped area which sloped down to the east and south at a maximum of about 20°.
- The southern end of the western site boundary was lined by a concrete block wall (maximum height about 3m). The central portion of the western site boundary was lined by a concrete block fence (maximum height about 1.5m). The face of the fence contained a number of rusted 24mm diameter bolt heads and plates and occasional cracks up to about 4mm width were recorded. The northern end of the western site boundary was lined by a concrete block wall (maximum height about 2.5m) which supported the subject site; occasional hairline to 2mm wide cracks were observed.
- Neighbouring four and five level brick residential unit buildings were set-back about 5m to 10m from the southern and central portions of the western site boundary; occasional sections of render were missing from a unit building wall adjacent to the central portion of the western site boundary. A brick saw-tooth factory building was set-back about 5m from the northern portion of the western site boundary. Neighbouring grass surfaced and paved yard areas lined the western site boundary.

Based on a cursory inspection from within the site, the existing buildings, paved surfaces and structures were generally in good condition except where otherwise detailed above.

3.2 Expected Subsurface Conditions

Reference to the 1:100,000 geological map of Sydney indicates that the site is underlain by dune sand deposits of Quaternary age. These sands form part of the Botany Basin deposits, which extend to the south and west of the site area. It is known that the depth of sand and other alluvial deposits increases to the south, with bedrock generally occurring at depths in excess of 20m in the Mascot area.

The subsurface conditions expected to be encountered at the site are based on the results of boreholes JK6 and JK11 to JK15 (drilled in 1991) and boreholes ML1, ML3 to ML5, ML7 to ML10 and ML12 (drilled in 2002). Reference should be made to the borehole logs presented in Appendix A for detailed descriptions of the subsurface conditions at each borehole location. A graphical borehole summary is presented as Figure 3. The boreholes disclosed a generalised subsurface profile that comprised a limited thickness of fill over natural sands then sandstone bedrock at depths ranging between 0.6m and 6.5m. A summary of the subsurface conditions encountered in our previous investigations is outlined below.

Paved Surfaces

Concrete and asphalt paved surfaces were encountered in boreholes JK13, JK14, ML4, ML8 and ML10 and ranged between about 50mm and 200mm thickness.

Fill

Sandy or clayey fill with varying gravel content was encountered from surface level or beneath paved surfaces in boreholes JK6, JK11 to JK15, ML1 and ML9. In borehole JK14 a crushed sandstone fill (450mm thick) interpreted to represent pavement foundation material was encountered beneath the paved surface. The fill was generally assessed to be poorly (occasionally moderately) compacted.

Relic (Old) Topsoil

A sandy layer with roots interpreted to be a relic topsoil layer was encountered at the base of the fill in boreholes JK6, JK13 and JK14 and ranged in thickness from "thin" (JK13, no thickness recorded) to 0.3m (JK14).

Natural Soils

The natural soils generally comprised sands and were encountered from surface level or beneath the fill or old topsoil in all boreholes except JK11. On first contact the sands were generally loose or medium dense (very loose in borehole ML3). In a selection of the boreholes drilled in 2002 ('ML' series), dense sands were encountered from depths ranging between about 3m (ML5) and 6m (ML7).

In boreholes JK6 and ML12, residual medium dense clayey sand (1.1m thick) and sandy clay of medium plasticity and very stiff strength/medium dense clayey sand (0.8m thick), were encountered at 5.0m and 4.7m depth, respectively and extended down to the bedrock surface.

Weathered Sandstone Bedrock

Sandstone bedrock was encountered in boreholes JK6, JK11, JK15, ML4, ML5, ML7 to ML10 and ML12 at depths ranging between 0.6m (JK11) to 6.5m (ML7). Where surface RLs have recorded on the borehole logs, we note that close to the eastern site boundary the bedrock surface ranges between about RL38.16 (JK6) and RL39.5 (ML9). To the west, at approximate locations close to the eastern side of The 'Burger Centre', the bedrock surface had stepped down to about RL36.8m (ML4), RL36.9m (ML7) and RL37.7m (ML10). No boreholes penetrated weathered sandstone bedrock over the central or western portions of the subject site.

On first contact the sandstone was typically assessed to be extremely to distinctly weathered (equivalent to highly weathered recorded on the 1991 logs) and of extremely weathered to medium strength. Weak and very weak strengths recorded on the 1991 logs are broadly equivalent low and very low strengths.

From commencement of core drilling in boreholes ML5, ML8, ML9, ML10 and ML12 the sandstone bedrock was generally distinctly or slightly weathered and of medium or high strength. However, in boreholes ML5 and ML11, the sandstone bedrock was of poorer quality; distinctly weathered and of very low to low strength. The sandstone bedrock did not typically improve in quality with depth; in ML12 the bedrock was broadly of good quality, whilst in the remaining cored boreholes, the sandstone was of variable quality and contained extremely weathered seams (of shale and sandstone), core loss zones and strengths ranging from extremely low to medium. A summary of the defects encountered in the cored portions of the boreholes is presented below:

- The sandstone bedrock was horizontally bedded with cross bedding dipping at 20°.
- Occasional planar jointing was recorded with dips ranging from 60° to vertical.
- Horizontal extremely weathered zones were encountered in all cored boreholes except ML12 and ranged between 2mm and 100mm thickness.
- Zones of core loss were noted in boreholes ML5 and ML9 and were about 110mm and 50mm long, respectively. These zones may be interpreted as extremely weathered seams or clay seams.

Groundwater

Groundwater was encountered during auger drilling in boreholes JK15, ML3, ML4 and ML10 at depths of 2.7m, 1.2m, 5.3m and 5.1m, respectively. On completion of auger drilling, standing water levels were recorded in boreholes JK6 and JK12 at depths of 5.1m and 2.8m, respectively equivalent to approximately RL39.1m and RL36.2m. In borehole ML3, a collapse depth was recorded at 1.2m on completion of auger drilling. In sandy soils, borehole collapse often occurs at, or close to, the standing groundwater levels.

In the cored boreholes, standing water levels were recorded within a short time of completion core drilling in ML4, ML5, ML7, ML 8, ML 9, ML 10 and ML12, at depths of 5m, 6.2m, 1.4m, 8.0m, 2.0m, 4.8m and 2.7m, respectively. These depths are equivalent to RL37.5m, RL35.6m, RL37.2m, RL35.9m, RL41.8m, RL38.0m and RL41.2m. However, we note that water flush is used as part of the core drilling process thereby preventing a meaningful assessment of groundwater levels in the cored boreholes as groundwater levels would not have stabilised over the short monitoring period. Full water flush returns were noted in all the cored boreholes indicating a relatively low permeability rock mass.

We note that no longer term groundwater monitoring has been undertaken. However, based on advice provided in our previous hydrogeological report dated September 2002 and monitoring of groundwater levels in 2003 during construction of the existing buildings at the site, plotting of all available groundwater data at the site indicated a hydraulic gradient down to the west and north-west. However, over the subject site, the monitoring of groundwater levels indicated a hydraulic gradient down to the west-north-west.

3.3 Laboratory Test Results

Previous laboratory test results indicated the following:

- The laboratory soil pH test results on natural sand samples ranged between 7.2 and 7.4 indicating near neutral soil conditions.
- A laboratory sulphate content test result on a natural sand sample indicated a value of <50 mg/kg.

- The four-day soaked CBR test results returned values of ranging from 17% to 25% for the natural sands.
- The point load test results indicated that the rock cored ranged between low to high strength with estimated Unconfined Compressive Strengths (UCS) varying from 2MPa to 30MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Demolition, Excavation Conditions, Groundwater

We note that the previous boreholes drilled within and surrounding the subject site provided limited site coverage, particularly over the western end of the site. The comments and recommendations which follow are therefore of a preliminary nature and will need to be confirmed by further geotechnical investigation (see Section 4.8, below).

4.1.1 Demolition and Dilapidation Reports

Demolition of any structures, access roads and paved surfaces currently adjoining The 'Burger Centre' or lining the margins of the subject site will need to be carried out with care so as not to damage or de-stabilise the sections of existing buildings, structures and/or paved surfaces (including the King Street frontage) that will remain.

Prior to commencement of removal of any portions of existing sections of structures, access roads or paved surfaces, we recommend that saw cuts be provided at the interface with the portions of buildings or structures that are to remain. This will assist in controlling potential damage to the existing structures, access roads and paved surfaces (including the King Street frontage) associated with expected demolition activities. We expect a saw attachment to say at least medium sized (15 tonne) excavator would be used then removal of the structures, access roads and paved surfaces completed using a ripping tyne attachment and possibly a rock breaker attached to the tracked excavator.

Further comments regarding use of rock breakers are provided in the section 4.1.3, below.

Prior to demolition commencing we recommend that detailed dilapidation reports be completed on the neighbouring buildings and structures lining the western site boundary. Council may also require dilapidation survey reports to be completed on the neighbouring paved surfaces to the south and east. The owners should be asked to sign the reports and agree that they are a fair assessment of existing conditions, as these can then be used as a benchmark in assessing potential future damage claims (due to ground surface movements and/or vibration damage).

The proposed excavations have the potential to de-stabilise the adjoining 'Burger Centre', the southern end of the central portion of Blocks A and B and any paved surfaces lining the margins of the site that will remain (including the King Street frontage). We recommend that all available 'as built' drawings for The 'Burger Centre' and Blocks A and B be obtained in order to check the nature and extent of existing footings supporting the buildings. If such records are not available, we recommend that test pits be excavated to reveal existing footings and the nature of the foundation materials. We also recommend that similar test pits be excavated adjacent to the concrete block walls lining the western site boundary. The test pits must be inspected by a structural and geotechnical engineer to assess the need for, and if appropriate to detail, underpinning and/or other temporary support measures. Further details can then be provided.

We also note the presence of a steep sand batter slope and dilapidated sections of s steel soldier pile wall and timber infill panels beneath the elevated concrete paved area adjacent to the child care centre over the south-western corner of the site.

Following demolition of the concrete paved area we recommend that the stability of this batter slope be assessed by a geotechnical engineer; the need and extent of any stabilisation measures (such as temporary soil nails and shotcrete and/or a permanent retaining wall) may then be detailed.

4.1.2 Excavation

All excavation work should be complemented by reference to the Code of Practice, 'Excavation Work', Cat. No 312 dated 31 March 2000 by WorkCover.

The outline of the proposed excavations for Blocks D, E and F are indicated on the attached Figure 1. The proposed finished floor levels will be at RL41.3m (Blocks D and E), RL38.3m (Block F) and RL37.7m (tunnel between Blocks A and C); excavations to achieve design subgrade levels will extend to maximum depths of about 3.5m (new buildings) and 4m (new tunnel).

The excavations will extend through the soil profile and are not expected to penetrate weathered sandstone bedrock. The expected generally sandy subgrade should remain trafficable to tracked earthmoving plant. Wheeled vehicles (trucks etc) may become 'bogged' in areas of relatively loose sands, or following heavy rain periods. Preparatory compaction with tracked excavators and rollers and the placement of a sacrificial surface layer of crushed demolition rubble along the access routes would be beneficial with respect to trafficking the subgrade.

Excavations through the soil profile and any extremely weathered sandstone may be readily completed using bucket attachments to the tracked excavators. More competent (medium and high strength) sandstone bedrock, if encountered, we expect to be excavated using ripping tynes fitted to the tracked excavators and/or rock breaker attachments. Rock breakers would assist in completing detailed rock excavations for footings, service trenches, lift pits etc and also trimming any rock excavation faces.

4.1.3 Potential Vibration and Ground Surface Movement Risks

We note that any surficial poorly compacted fill and/or loose sands are likely to extend beyond the site boundaries. We advise that sudden stop/start movements of tracked equipment should be avoided in order to reduce transmission of ground vibrations to adjoining buildings, structures and paved surfaces.

Care is required with demolition and excavation as these may result in direct transmission of ground vibrations to neighbouring existing buildings, structures and paved surfaces. If there is any cause for concern then demolition and/or excavation should cease and further advice sought. In this regard, we recommend that rock breakers initially be restricted to say less than maximum 500kg size.

Use of rock breakers is expected to be limited and so we suggest that qualitative vibration monitoring of adjacent buildings, structures and paved surfaces within the site be regularly undertaken by site staff while the rock breakers are being used. However, for neighbouring buildings and structures, we recommend that periodic quantitative vibration monitoring be undertaken to confirm that peak particle velocities fall within acceptable limits. Subject to viewing the above mentioned dilapidation reports, we recommend that the peak particle velocities along the site boundaries do not exceed say 5mm/sec. We note that this vibration limit will reduce the risk of vibration damage to the neighbouring buildings and structures. However, these vibrations may still result in discomfort to occupants of the neighbouring buildings.

With regard to both qualitative and quantitative vibration monitoring, if potentially damaging vibrations are occurring it will be necessary to use lower energy equipment such as smaller hammers or grinder attachments on hydraulic excavators. Alternatively grid-sawing techniques can be used to dampen ground vibrations.

Where rock breakers are used, to reduce vibrations we recommend that the rock breaker be continually orientated towards the face, be operated one at a time and in short bursts only to reduce amplification of vibrations. In addition, rock saw cuts around the perimeter of any rock excavations would assist in reducing vibrations, provided the saw cut depth was continually maintained below the depth of the adjacent rock excavation. When using the rock breakers, the resulting dust should be suppressed by spraying with water.

4.1.4 Seepage

Within and adjacent to the subject site, groundwater levels recorded during our previous investigations ranged between approximately RL41.8m (ML9) and RL35.6m (ML5). The pattern of recorded groundwater levels tends to indicate a groundwater flow down from the east which reflects the fall of topography and generally follows the fall of the underlying bedrock profile. In addition, we note that during construction of the existing buildings over the northern portion of the site, site staff reported groundwater inflow into the bulk excavations from the eastern side of the site. Further, during construction, standing water levels within the temporary storage ponds and the bulk excavation were recorded at or close to approximately RL38m.

Based on the above past experience, groundwater seepage down from the east can be expected into the proposed excavations within the sandy soil profile, at or above the bedrock surface profile, particularly during periods of heavy or prolonged rainfall. If any bedrock is encountered in the excavations, concentrated flows may be encountered where defects daylight into the excavation face. Some instability of temporary excavation batters may occur at the soil-bedrock interface within excavations, especially after rain periods and sand bagging may be required to stabilise the toe of batter slopes through the soils.

We note that during heavy and prolonged periods of rainfall the groundwater level will rise. Studies within the Botany Basin, of which the site forms a part, have indicated maximum fluctuations of 1m to 2m. We therefore recommend that the initial stages of the excavation are carefully monitored by site staff, and if substantial flows are encountered, appropriate drainage measures may be detailed at the time.

At this stage we expect that any seepage that does occur within the excavation will be controlled using conventional sump and pump techniques and, locally, gravity drainage. However, locally de-watering using spear points may be required to control groundwater seepage into the tunnel excavation and to maintain batter slope stability. We would expect the water could be satisfactorily discharged within the site. We recommend that further investigation of the groundwater conditions be completed as part of the additional geotechnical investigation outlined in Section 4.8, below.

4.2 Retention

4.2.1 Temporary Batters

Where required, temporary excavation batters through the sandy soil profile of 1 Vertical (V) in 1.5 Horizontal (H) are appropriate. Where any clayey residual soils and extremely weathered bedrock are encountered temporary excavation batters of 1V in 1H are appropriate. Such batters are generally expected to be accommodated within the site and where battering can be accommodated, a conventional retaining wall may be constructed at the base of the batter and subsequently backfilled.

Where battering cannot be accommodated within the site geometry, or is not preferred, a full depth engineered retention system will need to be installed prior to excavation commencing. Such a system may be required where the excavation margins will extend to the sides of the southern end of The 'Burger Centre' and the access road adjacent to the adjacent Block E and F excavations (depending on relative levels). A full depth engineered retention system is also expected to be required to support the tunnel sides adjacent to existing buildings. The recommendations for determining the existing footing depths outlined in Section 4.1.1 are pertinent in this regard. Should the founding depth of the existing footings be assessed to lie above proposed bulk excavation levels then the full depth engineered retention system will be required.

A piled wall retention system is suitable for the site. Due to the potentially collapsible nature of the soil profile and the need to control potential movement of adjacent or neighbouring ground surfaces, buildings and structures we consider that bored piles are not suited to this site. Our preference, therefore, is for a grout injected (cfa) secant piled walls as they will limit the potential for ground collapse into the pile hole and the potential loss of sandy soils through gaps between the piles. Alternatively, a contiguous pile wall may be constructed. However, if a contiguous pile wall is selected, allowance must be made for making good gaps between the piles in order to reduce the loss of retained soils and consequent inducement of adjacent ground surface movements. In this regard, consideration may be given to providing a shotcrete face to the contiguous pile wall. The shotcrete facing would need to be applied in 'lifts' of maximum 1.5m vertical height and must be applied on the same day as completion of excavation in front of the contiguous pile wall.

A sheet piled wall retention system has been considered, but is not considered to be a suitable alternative as the sheet pile walls would not penetrate the bedrock expected at shallow depth below the subgrade levels of Blocks D and E, and also would not be able to be incorporated into the footing system. A slurry wall is also considered to be suitable but would but would not be cost effective on a relatively small site.

The toe of the piled walls should be embedded below bulk excavation level to sufficient depth to satisfy stability and bearing considerations. In this regard, the retention piles are expected to intersect bedrock and may be incorporated into the footing system. Further comments regarding the load carrying capacity of the piled walls are provided in Section 4.3, below.

The piled wall will require temporary propping and this may be achieved by using ground anchors. We assume that permanent propping of the retention system will be provided by the proposed floor slabs. Any temporary anchors which extend below neighbouring properties (believed to be unlikely), would require permission from the neighbours. Alternatively, where space permits, temporary propping of the walls may be achieved by using a temporary bench of sand left in front of the retention system. The bench should have a 3m minimum horizontal width just below the crest of the pile capping beam and should be graded down to bulk excavation level at 1V in 1.5H. The bench can be removed once the floor slabs of the proposed building provide permanent support to the retaining walls.

4.2.2 Construction Issues

If small cfa pile rigs are used at this site they may have difficulty drilling through any cemented bands within the soil profile and/or bedrock; further advice from the piling contractor should be sought in this regard. This work would need to be completed with care, using suitably experienced (and insured) contractors. Should any of the secant or contiguous piles penetrate bedrock below bulk excavation level, then every fourth pile should be terminated about 0.5m, above the bedrock surface to allow 'through-flow' of groundwater.

We note that if cfa piles are socketted into bedrock, care will be required whilst drilling the piles into the bedrock so as not to cause excessive sand draw-down and possibly induce ground surface movements around the excavation perimeter. The ground surface adjacent to the pile drill hole must be continually monitored by the piling contractor or site supervisor. If settlement indicating draw-down is detected, pile drilling must stop and further geotechnical advice sought.

4.2.3 Earth Pressure Parameters

The major consideration in the selection of earth pressures for the design of the retaining walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of temporary or permanent retention systems:

- The perimeter pile walls should be uniformly founded below bulk excavation level in sands of at least medium relative density and/or weathered sandstone bedrock. Allowable bearing pressure recommendations are provided in the Section 4.3 below.
- For progressively anchored or propped walls, say adjacent to The 'Burger centre' and the tunnel walls, where lateral movements are to be controlled, we recommend the use of a uniform rectangular earth pressure distribution of 8H kPa, where H is the retained height in metres.
- For design of cantilever walls which will be propped by the structure (and possibly any tunnel walls that do not lie close to adjoining buildings) and any underpins supporting a soil profile, we recommend the use of a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient (k_o) of 0.55 for the retained sands and extremely weathered sandstone profile, assuming a horizontal backfill surface.

- For new landscape retaining walls (and if a new wall is provided to support the western end of the King Street frontage), where we assume some minor movements of the walls may be tolerated, they may be designed for a coefficient of 'active' earth pressure, k_a, of 0.35 for the retained soil and extremely weathered sandstone (if encountered) profile, assuming a horizontal backfill surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile and extremely weathered sandstone above the groundwater level.
- Any surcharge affecting the walls (including adjacent high level footings, traffic, landscaping, compaction stresses etc) should be allowed in the design using the appropriate earth pressure coefficient from above.
- The piled walls and any underpins supporting a soil profile must be designed as permanently drained and UPVC pipes should be installed at nominal 1.2m horizontal spacing just above the adjacent floor level. Holes will need to be drilled to allow installation of the pipes and/or use of gaps between contiguous piles. The end of the pipe penetrating the retained sands behind the wall must be wrapped in a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The pipes should discharge into the perimeter drainage system.
- The free standing retaining walls constructed at the toe of the batters should be designed as drained and provision made for permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate the non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion.
- The passive toe resistance of the retaining walls may be estimated using a triangular lateral earth pressure distribution and a "passive" earth pressure coefficient, K_p, of 3 for the sands (but with a factor of safety of at least 2 to limit deformations), assuming horizontal ground in front of the wall.

The passive pressure due to the upper 0.3m below bulk excavation should be ignored in the analysis to take excavation tolerances into account. Any localised excavations in front of the walls (such as for buried services, footings, lift pits etc) must be taken into account in the wall design. Alternatively, where the walls are founded on bedrock, toe resistance may be achieved by keying the footing into bedrock. An allowable lateral stress of 200kPa may be adopted for design.

- For the design of any piles socketed into sandstone of at least very low strength, it is recommended that maximum allowable lateral toe resistance of 200kPa be used. Where bedrock is penetrated, due to strain incompatibility between the sands and bedrock, lateral restraint must be wholly provided by the rock socket.
- Temporary anchors bonded into the medium dense sands or weathered sandstone bedrock of at least low strength can be designed based on an effective friction angle of 33° or an allowable bond strength of 200kPa, respectively. All anchors should be proof tested to 1.3 times the working load under the supervision of an experienced engineer or construction superintendent, independent of the anchor contractor. We recommend that only experienced contractors be considered for the anchor installation as they will most likely extend below the groundwater level.

4.2.4 Tanking

Groundwater levels may be at or just below design subgrade levels and flood events may raise groundwater levels above design subgrade levels. If preferred, the basement levels may be designed as tanked and the proposed tunnel will most likely need to be designed as tanked. A design water level commensurate with a 50 or 100 year flood event would be appropriate; further advice from a hydraulic engineer or Council will be required in this regard. In addition, uplift pressures acting on the on-grade floor slabs would need to be resisted by ground anchors designed in accordance with the advice provided in Section 4.2.3, above.

4.2.5 Permanent Batters

Permanent batter slopes within the sandy soil profile should be formed at no steeper than 1V in 2.5H and planted with rapid growing vegetation to improve near surface stability and reduce erosion.

4.3 Footings

We expect that generally medium dense sands will be exposed over the majority of the base of the bulk excavations for Blocks D and F, loose sands over the base of the bulk excavations for Block E and very loose over the base of the tunnel bulk excavation. Allowable bearing pressures for the sands will be reduced due to the influence of the groundwater level and for the envisaged large column loads, pad footings of the order of 3m square may be required. We therefore recommend that the buildings be entirely founded within weathered bedrock. Weathered sandstone bedrock is expected to be encountered between about 1.5m and 3m below the bulk excavation level for Blocks D and E. However, we have no information as to the likely depth to bedrock over the footprint of Block F. We provide below guidance on shallow footings and piled footings.

4.3.1 Shallow Footings

Where bedrock is expected a short depth below bulk excavation level (say Block D), shallow footings founded in sandstone bedrock of at least low to medium strength may be tentatively designed for an allowable bearing pressure of 1,500kPa. Higher allowable bearing pressures may be appropriate provided an additional cored borehole investigation and/or spoon testing of pad footing bases is completed.

Excavations for pad or strip footings extending through sands should be supported with formwork, as vertical cuts will be potentially unstable. In addition, spear points may be required for localised drainage of footing excavations. We recommend that the shallow footing excavation bases be inspected by a geotechnical engineer to confirm the quality of the bedrock and will also be required for pile footing bases socketted into bedrock (see Section 4.2.2, below).

4.3.2 Pile Footings

As mentioned above, any piled wall retention systems that are constructed can be incorporated into the footing system for the structure. However, due to the likely depths to bedrock, we expect that the piles will penetrate bedrock. On this basis, for perimeter piles socketted a nominal 0.3m into sandstone of at least low to medium strength may be tentatively designed for an allowable end bearing pressure of 1,500kPa.

For piles socketted into the sandstone, we recommend large capacity drilling rigs with rock drilling equipment (such as coring buckets) be used. The proposed piling contractor must be given a copy of this report so that appropriate equipment with sufficient power is brought to site.

For internal footings, consideration could be also be given to auger grout injected piles socketed into weathered sandstone bedrock and tentatively designed for a similar allowable end bearing pressure of 1,500kPa.

4.3.3 Settlements

For individual piles socketed into bedrock or pad footings founded in bedrock, maximum total settlements are expected to be less than 1% of the pile diameter or minimum footing width, i.e. of the order of 5mm or 10mm, assuming a maximum pile diameter of 0.5m or a minimum pad footing width of 1m, respectively.

Differential settlements between the various footing types founded within sandstone bedrock are expected to be a maximum of about 5mm depending on the footing type. Such settlements are expected to be instantaneous (i.e. occur as the building is constructed). In addition, the effects of differential settlements may also be mitigated by delaying of final surface finishes.

We recommend that movement control joints be provided where any portions of new buildings connect to existing buildings.

4.4 Earthworks

Earthworks recommendations presented below should be complemented by reference to AS3798.

4.4.1 Subgrade Preparation

Over areas of proposed on-grade floor slabs, external pavements and over areas where fill is to be placed to raise site levels, preparation of the subgrade should consist of the following:

- Proof roll the sandy subgrade with a minimum 5 tonne deadweight smooth drum vibratory roller to achieve a minimum density index (I_D) of 65% or a minimum density of 98% Standard Maximum Dry Density (SMDD).
- Proof rolling should be closely monitored by the site supervisor to detect soft or unstable areas which should be removed and replaced with engineered fill (as outlined below).

- Care should also be taken when using vibrating equipment not to cause damage to any adjacent structures. The vibrations should be qualitatively monitored by site personnel and if there is any cause for concern then proof rolling should cease and further advice sought.

Where floor slabs are suspended then subgrade preparation would not be required and this may be preferred should groundwater levels be encountered at bulk excavation level.

4.4.2 Engineered Fill

Engineered fill should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm. We expect the excavated natural sands and any weathered bedrock may be used as engineered fill. Engineered fill should be placed in layers of maximum 100mm loose thickness and compacted with the above mentioned roller to achieve a minimum I_D of 70% for the sandy soils or a minimum density of 98% SMDD and within 2% of for clayey soils and weathered bedrock. However, the I_D or SMDD may be reduced to 65% or 95%, respectively in landscaped areas.

Density tests should be carried out at a frequency of one test per layer per $500m^2$ or three tests per visit, whichever requires the most tests, to confirm the above specification has been achieved. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

4.5 On-Grade Floor Slabs, External Pavements and Drainage

Slab-on-grade construction is feasible for on-grade floor slabs and external pavements provided the areas of exposed sand subgrade are prepared as outlined above and any engineered fill is placed in accordance with the above guidelines.

We recommend that the proposed on-ground floor slabs within cuts and over areas of bedrock subgrade be provided with under-floor drainage unless tanked basements are preferred. The under-floor drainage should comprise a high strength, durable, single sized washed aggregate, such as 'blue metal' gravel. The under-floor drainage should connect with the wall drainage (where appropriate) and lead to a sump for disposal to the stormwater system. However, we note that during construction of the existing buildings, groundwater levels were at or close to the proposed subgrade level (approximately RL38m), i.e. similar to Level 1. It may be that additional longitudinal drains similar to those provided under the existing buildings may be required in order to promote drainage of the subgrade and prevent leakage of groundwater into the basement level, tunnel and/or to control potential uplift pressures. If the basement and/or the tunnel are tanked then design for uplift pressures due to a groundwater level at least at RL38m will be required and may be of the order of 2m higher, depending on further advice with regard to local flood levels.

The proposed concrete on-grade floor slabs in a drained basement should be separated from all walls, footings etc (ie. designed as 'floating') to permit relative movement. Slab joints should be capable of resisting shear forces but not bending moments by providing dowels or keys. In addition, close to the interface between soil and bedrock subgrade conditions, additional joints and dowels will be required.

4.6 Soil Aggression

A sulphate content of less than 50mg/kg was returned for a natural sand sample. The soil pH values of 7.2 and 7.4 for the natural sands indicate near neutral subsurface conditions. For further information reference should be made to the Cement and Concrete Association's Technical Note TN57.

4.7 Tunnel Design and Construction Issues

We understand that the proposed tunnel will be formed using 'cut and cover' techniques and we note the following issues and constraints:

- The proposed excavations will extend across the existing road; buried services will need to be re-located

- There is the potential to de-stabilise adjoining buildings during excavations. Our preference would be for the construction of a grout injected (cfa) secant piled wall retention system prior to excavation commencing (see Section 4.2).

- The tunnel excavation is likely to encounter the groundwater table and there is likely to be the need for de-watering of the excavation during tunnel construction. Consideration will need to be given to design groundwater levels and whether or not a tanked design is appropriate. The scope of the proposed additional geotechnical investigations outlined in Section 4.8, below are pertinent in this regard.

- De-watering of the tunnel excavation during construction also has the potential to cause settlement of surrounding building footings founded in the sandy soils; review of the 'as built' footing details of the surrounding buildings will assist in assessing this potential for damage to the existing nearby buildings.

4.8 Further Geotechnical Investigations and Advice

The comments and recommendations provided above will need to be confirmed following completion of an additional geotechnical investigation, particularly over the south-eastern and south-western corners of the site and over the location of the proposed tunnel. Of particular importance will be the following aspects of the subsurface conditions:

- Bedrock depths and the quality of the bedrock with regard to allowable bearing pressures for footing design. Cored boreholes will be required.
- Groundwater levels across the site and their potential impact on excavation stability, retention design and possible need for tanking of basements and the tunnel. Installation of groundwater monitoring wells will be required together with on-going monitoring of groundwater levels. Further an assessment of the potentially detrimental impact of excavation de-watering may be required depending on the encountered groundwater levels.

5 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.

Occasionally, the subsurface conditions between and below 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.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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

Reviewed by:

Paul Roberts

Paul Roberts
Senior Associate

Agi Zenon
Senior Associate

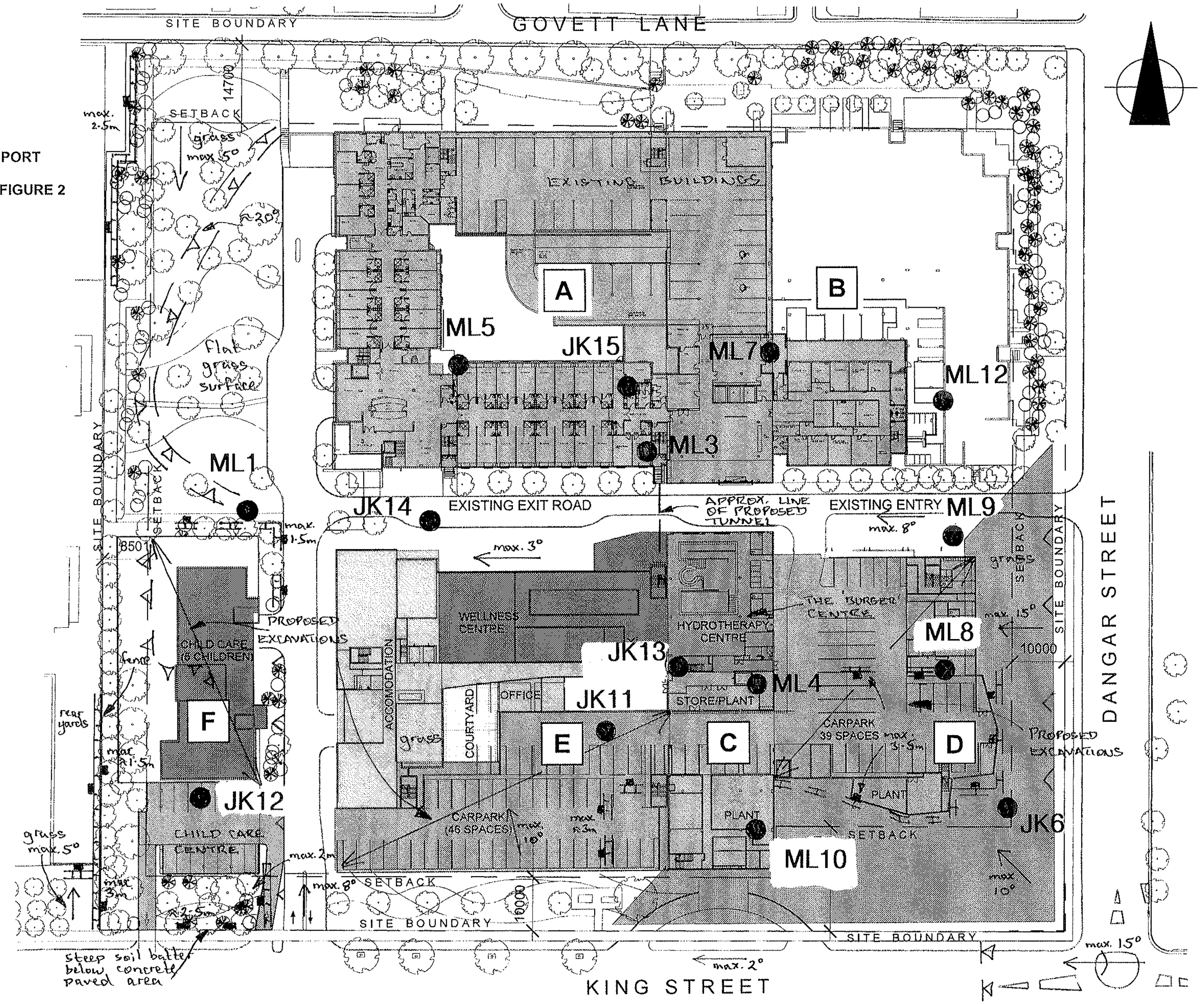
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

LEGEND

TO BE READ IN CONJUNCTION WITH TEXT OF REPORT

FOR EXPLANATION OF MAPPING SYMBOLS SEE FIGURE 2

● **BOREHOLE**



SCALE (m)

0m 37.5

GEOTECHNICAL SITE PLAN

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CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 17167ZR **Figure No. 1**

TOPOGRAPHY

Symbol *Ground Profile*

convex
concave
well defined or angular break of slope

convex
concave
poorly defined or smooth change of slope

breaks of slope
changes of slope
convex and concave too close together to allow the use of separate symbols

sharp
rounded
ridge crest

3 Cliff or escarpment or sharp break 40° or more (estimated height in metres)

15 → Uniform Slope
10 → Concave Slope
8 → Convex Slope
Slope direction and angle (Degrees)

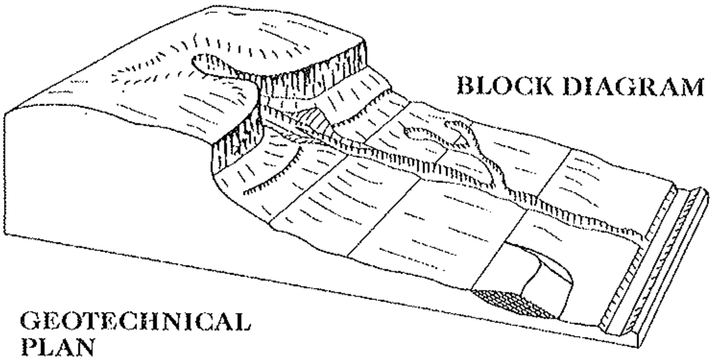
Top
Bottom
Cut or fill slope, arrows pointing down slope

Hummocky or irregular ground

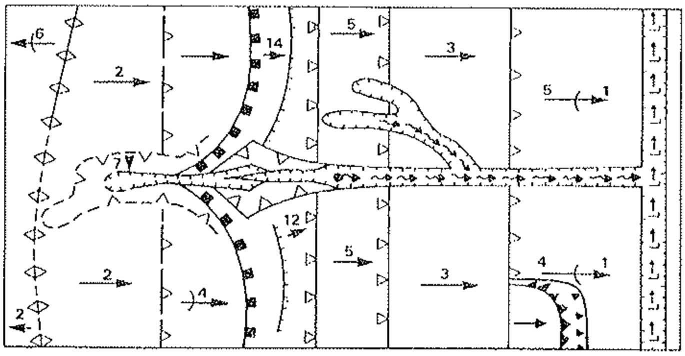
OTHER FEATURES

Boulder
Seepage/spring
Swallow hole for runoff
Natural water course
Open drain, unlined
Open drain, lined
Fenceline
Property boundary
Dry Stone Wall
J —— J 200 Major joint in rock face (opening in millimetres)
- T - T - 10 Tension crack (opening in millimetres)
Masonry or concrete wall
Ponding water
Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



GEOTECHNICAL PLAN

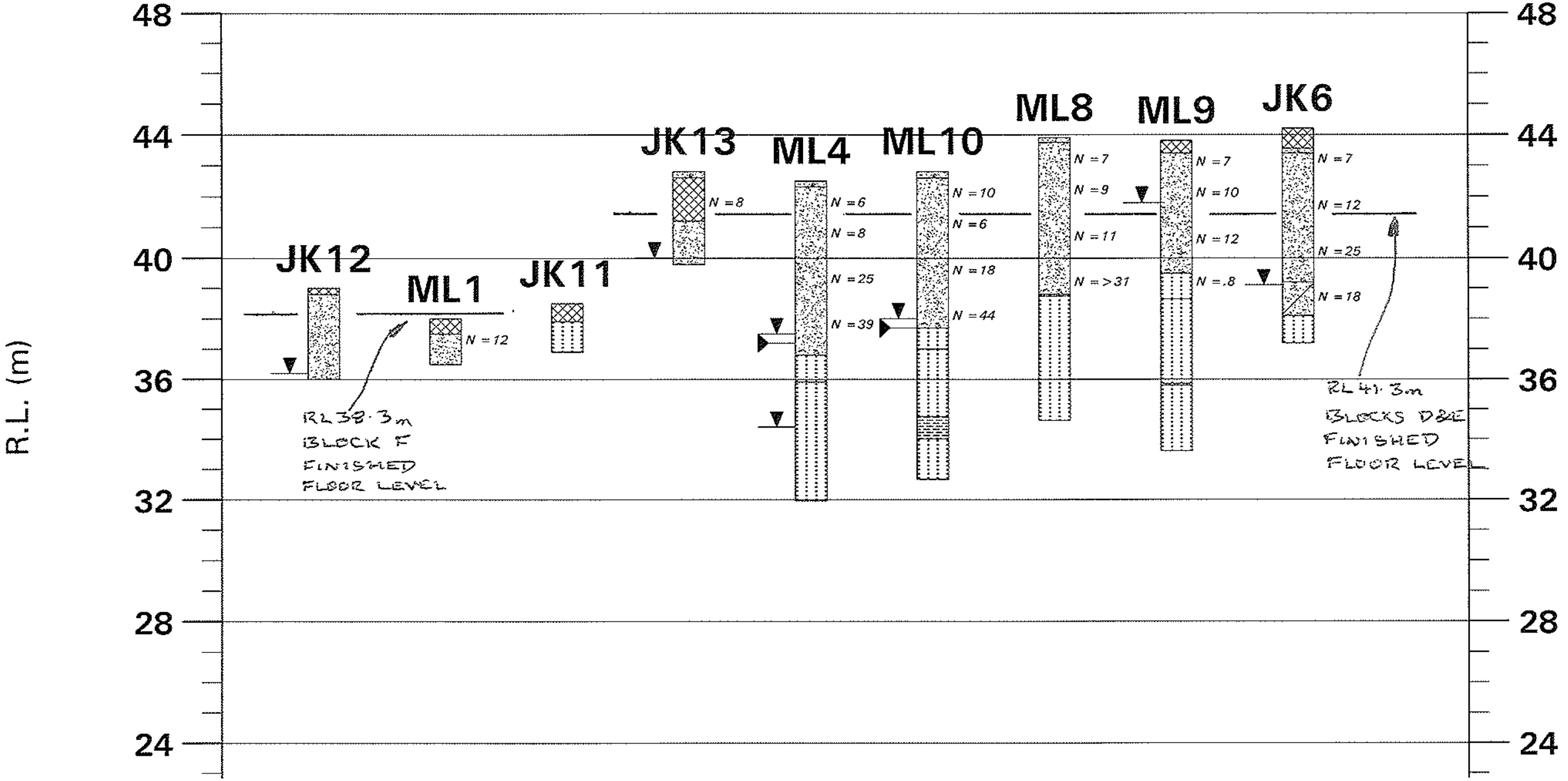


(After Gardiner, V & Dackombe, R.V. (1983), Geomorphological Field Manual; George Allen & Unwin).

GEOTECHNICAL MAPPING SYMBOLS

Jeffery and Katauskas Pty Ltd
Report No. 17167ZR Figure No. 2

GRAPHICAL BOREHOLE SUMMARY



RL38·3m
BLOCK F
FINISHED
FLOOR LEVEL

RL41·3m
BLOCKS D&E
FINISHED
FLOOR LEVEL

▩	Fill	▨	Concrete	▥	Silty Sand	▶—	Groundwater seepage level	Nc	SOLID CONE BLOW COUNTS PER 150mm
▒	Sand	▤	Shale	▧	Clayey Sand	N	SPT "N" VALUE		
⋮	Sandstone/ Greywacke	☐	Core Loss/ Empty	▼	Observed water level				

NOTE: REFER TO BOREHOLE LOGS

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

Jeffery and Katauskas Pty Ltd

Job No.: 17167ZR

Figure No.: 3

APPENDIX A

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BOREHOLE LOG

Borehole No. JK6

Client:
Project: GEOTECHNICAL AND CONTAMINATION INVESTIGATION
Location: RANDWICK BUS DEPOT

Job No. 7940S
Method: SPIRAL AUGER G.C.H. RIG
R.L. Surface: 44.2m
Date: 10 - 1 - 91
Datum: AHD

ndwater record	Samples	Field Tests	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
						FILL: Sand, fine to medium grained, light grey some fragments of bricks and concrete.				
					SM	SILTY SAND: fine grained, brown, some roots.	M	L		OLD TOPSOIL.
	DS	N = 7 4, 3, 4	1		SP.	SAND: fine to medium grained light grey to yellow white, rare orange brown ironstaining, trace of silty fines.				
								L/MD.		
			2							
	DS	N = 12 5, 6, 6								
			3							
								MD		
	DS	N = 25 8, 10, 15								
			4							
							M/W			
▼ ON DRILLING ON 25-1-91			5		SC	CLAYEY SAND: fine to medium grained, light grey occasional orange brown ironstaining.				
	DS	N = 18 8, 8, 10								
			6			SANDSTONE: fine to medium grained, light grey occasional orange ironstaining highly weathered, weak with very weak bands 50-100mm				MODERATE 'TC' BIT RESISTANCE
										LOW RESISTANCE.
										MODERATE RESISTANCE.
			7							LOW RESISTANCE.

Borehole No.
JK6
2/2

BOREHOLE LOG

Client:
Project: GEOTECHNICAL AND CONTAMINATION INVESTIGATION
Location: RANDWICK BUS DEPOT

Job No. 7940S
Method: SPIRAL AUGER
G.C.H. RIG
R.L. Surface: 44·2m
Date: 10 - 1 - 91
Datum: AHD.

ndwater record	Samples	Field Tests	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
	DS		8	░		SANDSTONE: as above.				MODERATE RESISTANCE WITH LOW BANDS.
			9			END OF BOREHOLE AT 8·4m.				
			10							
			11							
			12							
			13							

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Borehole No. JK11

BOREHOLE LOG

Client:
Project: GEOTECHNICAL AND CONTAMINATION INVESTIGATION
Location: RANDWICK BUS DEPOT

Job No. 7940S
Date: 11 - 1 - 91
Method: SPIRAL AUGER
G.C.H. RIG
Estimated RL: ~ 38·5 m
Datum: AHD

ndwater record	Samples	Field Tests	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
DRY ON COMPLETION				▩		FILL: Sand, fine to medium grained, some crushed rock.	M			APPEARS MODERATELY COMPACTED. FAINT OIL ODOUR.
	DS		1	░		SANDSTONE: white, fine to medium grained, some orange brown ironstaining, highly weathered, weak.				LOW TO MODERATE 'TC' BIT RESISTANCE.
			2			END OF BOREHOLE AT 1·6m.				
			3							
			4							
			5							
			6							
			7							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL ENGINEERS

BOREHOLE LOG

Borehole No. JK12

Client:
Project: GEOTECHNICAL AND CONTAMINATION INVESTIGATION
Location: RANDWICK BUS DEPOT

Job No. 7940S
Date: 11 - 1 - 91
Method: SPIRAL AUGER G.C.H. RIG
Estimated RL : 39.0m
Datum : AHD

oundwater record	Samples	Field Tests	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
				▩		FILL: Sand, fine to medium grained, brown.				
				⠿		SAND: fine to medium grained, white some orange brown ironstaining (decreasing with depth)	M	MD		
	DS	N = 18 7, 9, 9	1	⠿						
			2	⠿						
				⠿						SLIGHT OIL ODOUR.
▼ ON DRILLING			3			END OF BOREHOLE AT 3.0m.	W			
			4							
			5							
			6							
			7							

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CONSULTING GEOTECHNICAL ENGINEERS

Borehole No.
JK13

BOREHOLE LOG

42·8

Client:
Project: GEOTECHNICAL AND CONTAMINATION INVESTIGATION
Location: RANDWICK BUS DEPOT

Job No. 7940S
Date: 11 - 1 - 91
Method: SPIRAL AUGER
G.C.H. RIG
Estimated RL : 42·8m
Datum : AHD

Groundwater record	Samples	Field Tests	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
				▒		CONCRETE SLAB: 190mm				
	DS	N = 8 2,3,5	1	▨		FILL: Sand, fine to medium grained, light brown to white.	M			APPEARS POORLY COMPACTED.
			2	░	SP	SAND: fine to medium grained, light yellow white.	M	L/MD		POSSIBLY THIN LAYER OF OLD TOPSOIL OVER SAND.
			3			END OF BOREHOLE AT 3·0m.				
			4							
			5							
			6							
			7							

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CONSULTING GEOTECHNICAL ENGINEERS

BOREHOLE LOG

Borehole No. JK14

Client:
Project: GEOTECHNICAL AND CONTAMINATION INVESTIGATION
Location: RANDWICK BUS DEPOT

Job No. 7940S Method: SPIRAL AUGER UNKNOWN
Date: 11 - 1 - 91 G.C.H. RIG

undwater record	Samples	Field Tests	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
DRY ON COMPLETION				▩		ASPHALT 50mm FILL: Crushed sandstone.				
	DS	N=10 8, 5, 5	1	▩		FILL: Sand, fine to medium grained, light grey.				APPEARS POORLY TO MODERATELY COMPACTED.
				▨	SC	CLAYEY SAND: fine to medium grained, brown, occasional root.				OLD TOPSOIL.
			2	⋯	SP	SAND: fine to medium grained, light yellow white.				
			3			END OF BOREHOLE AT 3.0m.				
			4							
			5							
			6							
			7							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL ENGINEERS

Borehole No. JK15

BOREHOLE LOG

Client:
Project: GEOTECHNICAL AND CONTAMINATION INVESTIGATION
Location: RANDWICK BUS DEPOT

Job No. 7940S
Date: 11 - 1 - 91
Method: SPIRAL AUGER
G.C.H. RIG
UNKNOWN

ndwater record	Samples	Field Tests	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
				▨		FILL: Sand, fine to medium grained, grey, high proportion of crushed basalt.				DIESEL FUEL STAIN OF SURFACE -DIESEL ODOUR IN SOIL
	DS	N=8 4,4,4	1	▨		FILL: Sand, fine to medium grained, grey to brown.	M			APPEARS POORLY COMPACTED.
			2	⋯	SP	SAND: fine to medium grained, light yellow white.		L/MD		
SLOW.			3	⋯		SANDSTONE: fine to medium grained, white ironstained orange brown, highly weathered, very weak.				
			4 5 6 7			END OF BOREHOLE AT 3.0m				

Borehole No.
ML1 1/1

BOREHOLE LOG

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 24-9-02
Method: SPIRAL AUGER
JK250
R.L. Surface: ≅ 38.0m
Datum: AHD
Logged/Checked by: A.J.H./

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
Y ON COMPLET- ION & AFTER 5 MINS						0			FILL: Silty clay, low plasticity, light red brown, with a trace of sandstone gravel.	MC<PL			
			●		N = 12 7,5,7			SP	SAND: fine to coarse grained, light yellow brown.	M	MD		
			●			1							
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
ML3 1/1

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 24-9-02
Method: SPIRAL AUGER
JK250
R.L. Surface: ≅ 37.7m
Datum: AHD
Logged/Checked by: A.J.H./

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		SP	SAND: fine to coarse grained, brown, with a trace of rootlets and silt fines.	M	VL		GRASS COVER
		N = 3 1,1,2				as above, but grey and brown, without rootlets and with silt fines.				
			1							
▶							W			
						END OF BOREHOLE AT 1.5m				
			2							
			3							
			4							
			5							
			6							
			7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
ML4 1/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 24-9-02
Method: SPIRAL AUGER
JK250
R.L. Surface: ≅ 42.5m
Datum: AHD
Logged/Checked by: A.J.H./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						0			CONCRETE: 200mm.t				NO OBSERVED REINFORCEMENT
								SP	SAND: fine to medium grained, light brown.	M	L	-	AEOLIAN
				■	N = 6 2,3,3				as above, but with dark brown seams.				
						1							
									SAND: fine to coarse grained, yellow brown, with a trace of silt fines.				
				■	N = 8 3,4,4								
						2							
						3					MD		
				■	N = 25 12,12,13								
15 HRS AFTER COMPLETION OF CORING						4							
											D		
▼				■	N = 39 6,18,21								
						5			as above, but with a trace of clay fines.				
►										W			
								-	SANDSTONE: fine to medium grained, light grey, with clay bands.	DW	VL	-	VERY LOW 'TC' BIT RESISTANCE
						6							
									SANDSTONE: fine to coarse grained, light grey.	SW	M		MODERATE RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						7							

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CORED BOREHOLE LOG

Borehole No. ML4 2/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 24-9-02
Drill Type: JK250
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≅ 42.5m
Datum: AHD
Logged/Checked by: A.J.H./

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
		6		START CORING AT 6.60m						
			⁝	SANDSTONE: fine to medium grained, light grey, massive.	SW	M	×	▮	- CS, 0°, 25mm.t	
		7	⁝				×	▮	- CS, 0°, 2mm.t	
▼ ON COMPLETION OF CORING FULL RETURN		8	⁝				×			
			⁝				×		- CS, 0°, 2mm.t	
		9	⁝		SW-Fr	H	×			
			⁝			VL-L			- J, SUBVERTICAL, Un, R	
			⁝			H	×			
		10	⁝				×		- XWS, 0°, 5mm.t	
				END OF BOREHOLE AT 10.55m						
		11								
		12								

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BOREHOLE LOG

Borehole No.
ML5 1/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 25-9-02
Method: SPIRAL AUGER
JK250
R.L. Surface: ≅ 37.0m
Datum: AHD
Logged/Checked by: A.J.H./

Groundwater Record	ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						0		SP	SAND: fine to coarse grained, brown, with a trace of rootlets and silt fines.	M	L		GRASS COVER
									as above, but without rootlets.				SLIGHT HYDROCARBON ODOUR FROM 0.5m TO 1.5m DEPTH
ON COMPLETION OF CORING ▼				■	N = 9 1,3,6	1			as above, but grey brown, with silt fines.				
▶ DURING DRILLING				■	N = 14 4,7,7	2				W	MD		
									as above, but light brown and yellow brown, with a trace of silt fines.				
				■	N > 31 7,20, 11/50mm	3			as above, but light yellow brown, without silt fines.		D		
					END			-	SANDSTONE: fine to coarse grained, light orange brown.	DW	L	-	MODERATE 'TC' BIT RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						4							
						5							
						6							
						7							

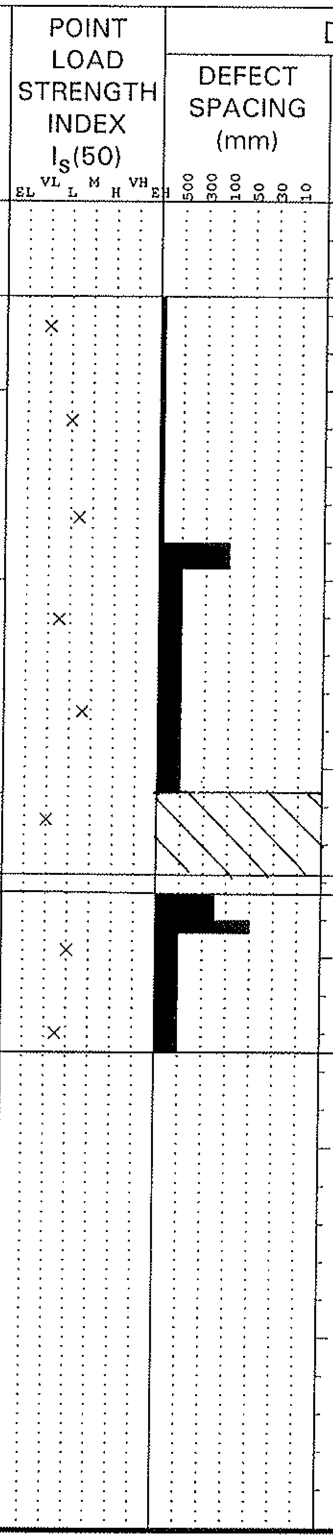
CORED BOREHOLE LOG

Borehole No.
ML5 2/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 25-9-02
Drill Type: JK250
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≅ 37.0m
Datum: AHD
Logged/Checked by: A.J.H./

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
		3		START CORING AT 3.50m				
FULL RETURN				SANDSTONE: fine to coarse grained, light orange brown, bedded at 0-15°, with a trace of fine grained subangular and sub-rounded quartz gravel.	DW	L		
		4				M		
				as above, but light grey and light orange brown.				
							- VERY LOW STRENGTH SEAM, 70mm.t - XWS, 0°, 30mm.t - XWS, 0°, 20mm.t	
		5		SANDSTONE: fine to medium grained, light grey bedded at 0-10°.		L-M		
							- XWS, 0°, 5mm.t	
		6			XW	EL		
				CORE LOSS 0.11m				
				SANDSTONE: fine to medium grained, light grey, bedded at 0-10°.	DW	VL-L	- XWS, 0°, 20mm.t - XWS, 0°, 3mm.t	
		7				L-M		
				END OF BOREHOLE AT 7.50m				
		8						
		9						



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BOREHOLE LOG

Borehole No.
ML7 1/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 25-9-02
Method: SPIRAL AUGER
JK250
R.L. Surface: ≅ 43.4m
Datum: AHD
Logged/Checked by: A.J.H./

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
▽ ON COMPLETION OF AUGERING						0		SP	SAND: fine to medium grained, dark brown, with silt fines.	D	L		
				■	N = 6 3,3,3				as above, but light grey brown.	M			
						1							
				■	N = 6 6,3,3				as above, but light grey.				
						2							
				■	N = 19 5,6,13	3			as above, but with dark brown seams.		MD		
						4							
				■	N = 20 6,10,10				as above, but light grey.				
						5			as above, but light yellow brown.				
▼ ON COMPLETION OF CORING				■	N = 36 7,16,20	6			as above, but light brown, with light orange brown seams.		D		
								-	SANDSTONE: fine to coarse grained, brown and grey. REFER TO CORED BOREHOLE LOG	DW	M	-	MODERATE 'TC' BIT RESISTANCE
						7							

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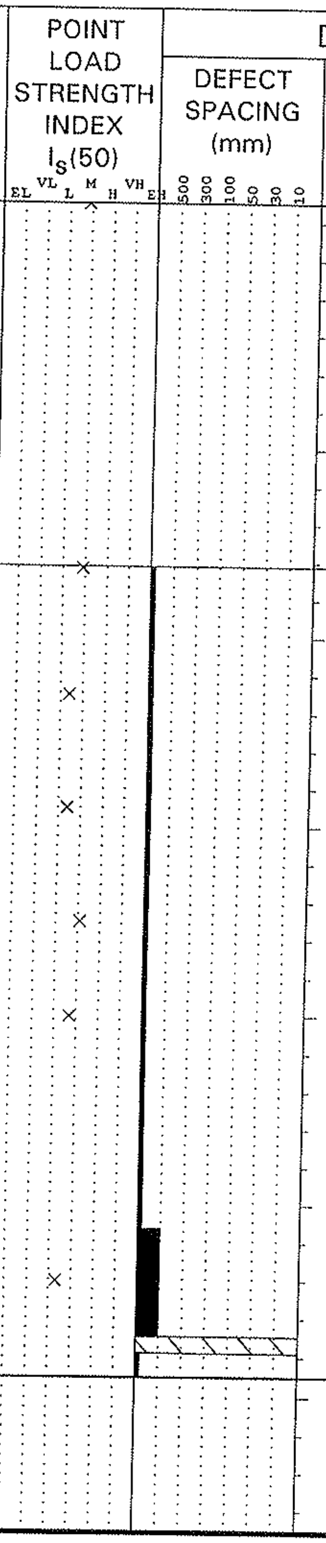
CORED BOREHOLE LOG

Borehole No. ML7 2/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 25-9-02
Drill Type: JK250
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≅ 43.4m
Datum: AHD
Logged/Checked by: A.J.H./

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
		5					
		6					
			START CORING AT 6.62m				
FULL RET-URN		7	SANDSTONE: fine to medium grained, brown and orange brown, bedded at 0-5°.	DW	M		
		8	SANDSTONE: fine to medium grained, light grey, massive, with occasional light orange laminae.	SW	M-H		
					M		
		9	as above, but without orange laminae.				
		10				- XWS, 0°, 40mm.t	
						- XWS, 0°, 80mm.t	
		11	END OF BOREHOLE AT 10.90m				



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BOREHOLE LOG

Borehole No.
ML8 1/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 26-9-02
Method: SPIRAL AUGER
EDSON 3000
R.L. Surface: ≅ 43.9m
Datum: AHD
Logged/Checked by: A.J.H./

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
ON COMPLETION OF AUGERING						0			CONCRETE: 160mm.t				
								SP	SAND: fine to coarse grained, light brown.	M	L	-	
				■	N = 7 3,3,4				as above, but dark brown.				
						1							
				■	N = 9 3,4,5				as above, but light grey.				
						2			as above, but light brown.				
				■	N = 11 5,5,6	3					MD		
						4			as above, but light yellow brown.				
				■	N > 31 12,17, 14/50mm END						D		
						5		-	SANDSTONE: fine to medium grained, light brown. REFER TO CORED BOREHOLE LOG	DW	M	-	MODERATE RESISTANCE
						6							
						7							

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CORED BOREHOLE LOG

Borehole No.
ML8
2/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 26-9-02
Drill Type: EDSON 3000
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≅ 43.9m
Datum: AHD
Logged/Checked by: A.J.H./

er 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
		4								
		5		START CORING AT 5.15m						
				SANDSTONE: fine to medium gained, light brown, bedded at 0-10°.	DW	M	×			
		6		as above, but light grey and brown.			×		- XWS, 0°, 110mm.t	
				SANDSTONE: fine to medium grained, light grey, massive.	SW	H	×		- CS, 0°, 2mm.t - CS, 0°, 5mm.t	
LL RET- URN		7				M	×			
							×		- XWS, 0°, 5mm.t	
AFTER REMOVING CASING		8					×		- J, 70°, P, R	
						L	×			
		9				M	×		- J, 80°, P, R	
				END OF BOREHOLE AT 9.28m						
		10								

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Borehole No.
ML9 1/2

BOREHOLE LOG

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 26-9-02
Method: SPIRAL AUGER
EDSON 3000
R.L. Surface: ≅ 43.8m
Datum: AHD
Logged/Checked by: A.J.H./

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
Y ON COMPLETION OF AUGERING						0	▨		FILL: Sand, fine to coarse grained, dark grey, with a trace of sandstone gravel and bricks.	M			
				■	N = 7 4,4,3			SP	SAND: fine to coarse grained, light grey.	M	L	-	
						1							
				■	N = 10 3,3,7						MD		
▼ ON COMPLETION OF CORING						2			as above, but light yellow brown and light brown.				
									as above, but yellow brown, with a trace of clay fines.				
				■	N = 12 6,6,6	3							
						4							
				■	N > 8 16,8/ 50mm REFUSAL			-	SANDSTONE: fine to coarse grained, orange brown, with clay seams.	XW	EL-VL	-	VERY LOW 'TC' BIT RESISTANCE
						5			SANDSTONE: fine to medium grained, orange brown.	DW	VL-L		LOW RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						6							
						7							

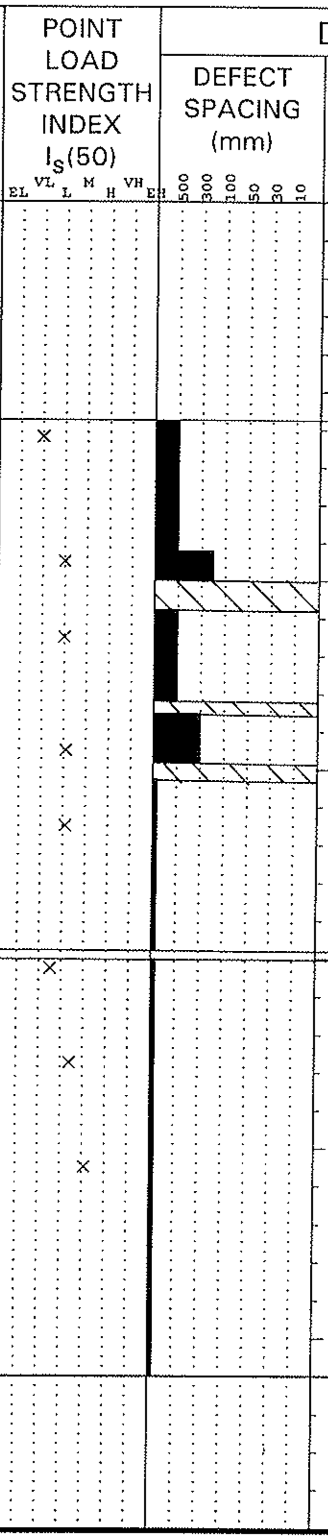
CORED BOREHOLE LOG

Borehole No.
ML9 2/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 26-9-02
Drill Type: EDSON 3000
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≅ 43.8m
Datum: AHD
Logged/Checked by: A.J.H./

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
		4						
		5		START CORING AT 5.15m				
				SANDSTONE: fine to medium grained, orange brown and light grey, bedded at 0-5°.	DW	VL-L		
						M		
		6					XWS, 0°, 40mm.t XWS, 0°, 150mm.t	
				as above, but cross bedded at 0-15°.				
							XWS, 0°, 60mm.t	
		7					XWS, 0°, 100mm.t	
FULL RETURN				SANDSTONE: fine to medium grained, light grey, massive.	SW			
		8		CORE LOSS 0.05m			CS, 0°, 10mm.t	
				SANDSTONE: fine to medium grained, light grey, massive.	SW	L-M		
						M-H		
		9						
							XWS, 0°, 40mm.t	
		10						
				END OF BOREHOLE AT 10.20m				



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BOREHOLE LOG

Borehole No.
ML10 1/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 27-9-02
Method: SPIRAL AUGER
EDSON 3000
R.L. Surface: ≅ 42.8m
Datum: AHD
Logged/Checked by: C.T./

Groundwater Record	ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						0			CONCRETE: 200mm.t				
								SP	SAND: fine to medium grained, light grey.	M	L-MD	-	-
				■	N = 10 3,5,5								
						1			as above, but light brown.				
				■	N = 6 1,2,4						L		
						2			as above, but yellow brown.				
				■	N = 18 6,8,10	3					MD		
3 HRS AFTER COMPLETION OF CORING ▼				■	N = 44 11,18,26	4					D		STANDPIPE INSTALLED TO 5.5m DEPTH ON COMPLETION OF CORING
▶ DURING DRILLING						5		-	SANDSTONE: fine to medium grained, red brown, with clay bands.	XW-DW	EL-VL	-	VERY LOW 'TC' BIT RESISTANCE
						6			REFER TO CORED BOREHOLE LOG				
						7							

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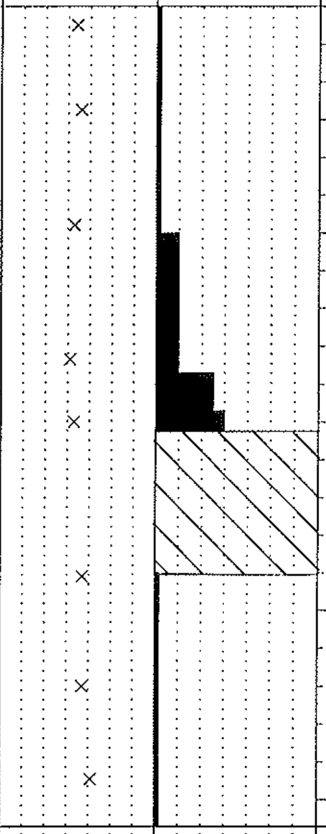
CORED BOREHOLE LOG

Borehole No.
ML10 2/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 27-9-02
Drill Type: EDSON 3000
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≅ 42.8m
Datum: AHD
Logged/Checked by: C.T../



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
		5		START CORING AT 5.80m				
FULL RET-URN		6		SANDSTONE: fine to medium grained, orange brown.	DW	M		
				as above, but fine to coarse grained, light grey, with occasional shale lenses.	SW		XWS, 0°, 2mm.t. CS, 3mm.t., 0°	
		7		as above, but with shale bands.			XWS, 0°, 40mm.t. XWS, 0°, 5mm.t.	
		8		SHALE: dark grey, with very low to low strength seams.	XW	EL		
		9		SANDSTONE: fine to medium grained, light grey, with some shale lenses, massive.	SW	M-H	XWS, 0°, 5mm.t.	
		10		END OF BOREHOLE AT 10.14m				
		11						

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BOREHOLE LOG

Borehole No.
ML12 1/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 27-9-02
Method: SPIRAL AUGER JK350
R.L. Surface: ≅ 43.9m
Datum: AHD
Logged/Checked by: C.T./

roundwater 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
/ ON COMPLETION OF CORING						0		SM	SILTY SAND: fine to medium grained, dark grey.	M	MD		
				■	N = 10 3,5,5								
						1		SP	SAND: fine to medium grained, light grey.				
				■	N = 5 1,2,3						L		
						2							STANDPIPE INSTALLED TO 6.0m DEPTH ON COMPLETION OF CORING
▼ 2 HRS AFTER ORING													
				■	N = 10 1,4,6	3					MD		
									as above, but yellow brown.				
						4		SM	SILTY SAND: fine to medium grained, red brown.				
				■	N = 16 4,7,9			CL/SC	SANDY CLAY/CLAYEY SAND: medium plasticity, fine to medium grained, light grey, yellow brown and red brown.	MC>PL /M	(VSt) / MD		
						5							
								-	SANDSTONE: fine to medium grained, red brown and grey.	DW	VL-L	-	VERY LOW TO LOW 'TC' BIT RESISTANCE
						6			REFER TO CORED BOREHOLE LOG				
						7							

CORED BOREHOLE LOG

Borehole No.
ML12 2/2

Client: McLACHLAN LISTER PTY LIMITED
Project: PROPOSED AGED CARE FACILITY
Location: CORNER KING AND DANGAR STREETS, RANDWICK, NSW

Job No. 17167S
Date: 27-9-02
Drill Type: JK350
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≅ 43.9m
Datum: AHD
Logged/Checked by: A.J.H./

Vater 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
		5								
				START CORING AT 6.00m						
		6		SANDSTONE: fine to medium grained, red brown, massive.	DW	H	×			
							×			
				SANDSTONE: fine to medium grained, light grey, with occasional dark grey laminae, cross bedded at 0-20°.	SW					
		7					×			
									CS, 10mm.t., 0°	
FULL ET- RN							×			
		8								
							×			
		9					×			
						M	×			
		10		END OF BOREHOLE AT 10.0m						
		11								

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ABN 17 003 550 801

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

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

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

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

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

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

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from "feel" and rate of penetration.

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

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

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

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

In the tests, a 35mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

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

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer.

SITE INSPECTION

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

Requirements could range from:

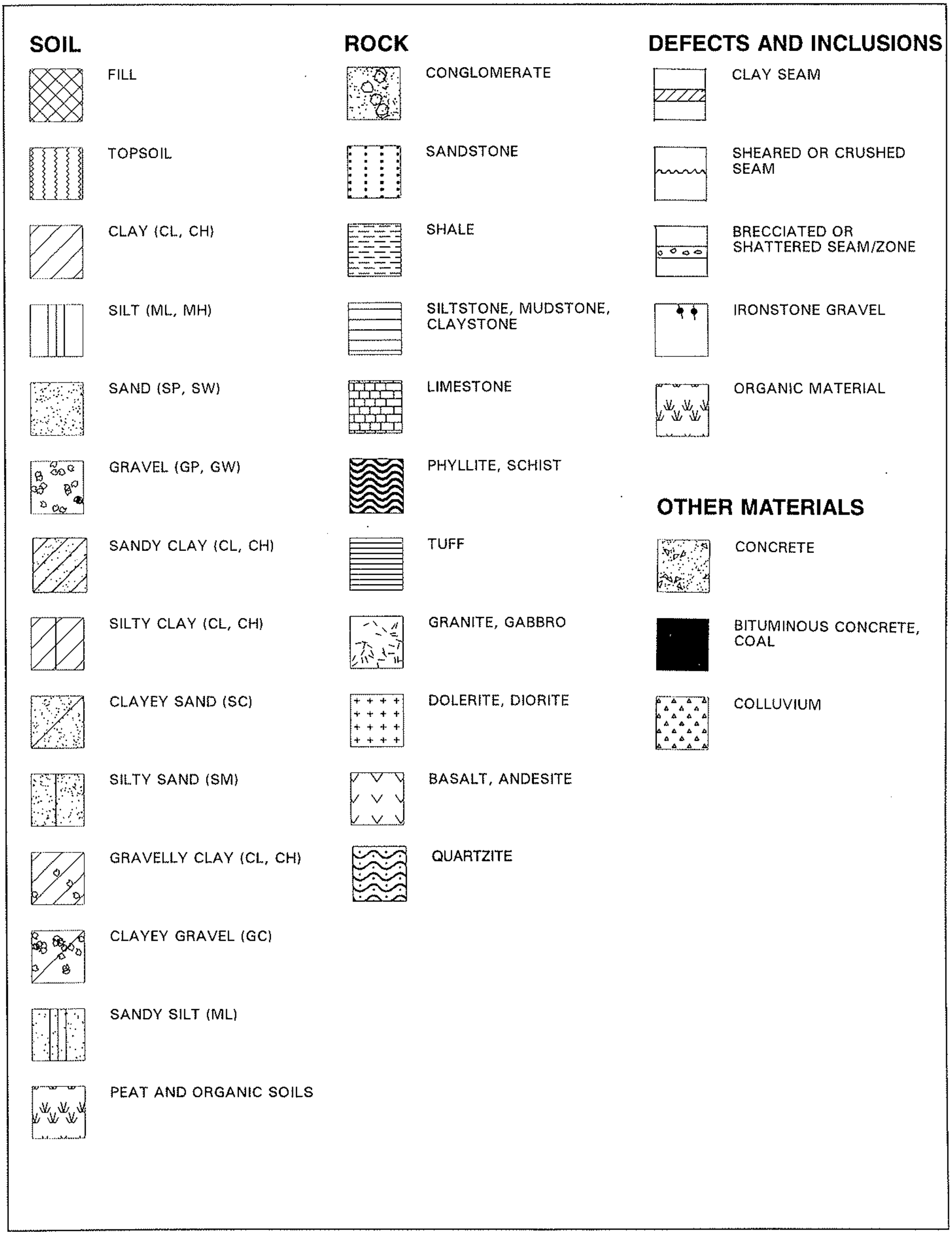
i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or

iii) full time engineering presence on site.

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



SOIL

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

ROCK

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

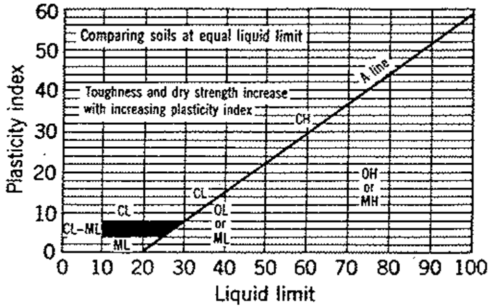
DEFECTS AND INCLUSIONS

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

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)						Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria			
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses 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									 Plasticity chart for laboratory classification of fine grained soils		
			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*)				
			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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LOG SYMBOLS

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

Ref: Standard Sheets/Log Symbols
November 2007

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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

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

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

ABBREVIATIONS USED IN DEFECT DESCRIPTION

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

Ref: Standard Sheets/Log Symbols
November 2007