REPORT

TO
SIR MOSES MONTEFIORE JEWISH HOME

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED ALTERATIONS AND ADDITIONS

AT
36 DANGAR STREET, RANDWICK, NSW

29 June 2016
Ref: 17167ZRrpt5

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5003
www.jkgeotechnics.com.au
Jeffery & Katauskas Pty Ltd, trading as
JK Geotechnics ABN 17 003 550 801

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Paul Roberts

Report prepared by:

Paul Roberts
Senior Associate | Engineering Geologist

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed alterations and additions at the Sir Moses Montefiore Jewish Home, 36 Dangar Street, Randwick, NSW. The assessment was commissioned on behalf of Sir Moses Montefiore Jewish Home by Ms Connie Argyrou of Jackson Teece in an email dated 19 May 2016. The commission was on the basis of our fee proposal (Ref. P42293ZR rev1) dated 19 May 2016.

We have been provided with the following information:

- Unreferenced and undated architectural plans prepared by Jackson Teece.
- Site survey plan (Ref. No. 090512 Issue H, dated 9 May 2016) prepared by Denny Linker & Co.

Based on the provided information we understand the proposed alterations will comprise:

- Construction of three new five to seven level buildings (Blocks D, E and F) over the southern portion of the site. Two levels of basement car parking will be provided below Blocks E and F and the western end of the Basement Level 2 car park will also include a detention basin. The finished floor reduced levels (RLs) of Basement Level 2 and 1 will be at RL35.5m and RL38.5m, respectively. The existing car parking area over the south-eastern corner of the site will be extended to the north, east and south at a similar surface level (RL41.5m). Excavations to maximum depths of about 6.5m and 4.5m will be required to achieve the design subgrade levels over the south-western and south-eastern portions of the site, respectively.
- A new tunnel connection to the Basement Level 2 car park below proposed Block E (proposed floor level at RL35.5m). The tunnel will extend south from Block A (current floor level at RL38.3m) and excavations to a maximum depth of about 3m below the current Block A floor level will also be required to achieve design subgrade levels.
- Landscaping including paved courtyards will be provided over the south-western 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.
- Report (Ref. E17167KBlet-rev1.2) dated 13 July 2010 regarding potential urban salinity risks/hazards at the site prepared by our specialist environmental investigation services division (EIS).
- Report (Ref. 17167ZR3rpt) dated 13 July 2010 for the Part 3A Concept Plan Proposal and Project Application, which comprised a desktop study of the available geotechnical information for the site sourced from our previous geotechnical investigations and reports.
- Report (Ref. 17167ZR4rpt) dated 14 March 2011 for a previous configuration of the proposed alterations and additions which included Blocks D and E, which comprised an additional geotechnical investigation and included the results of previous geotechnical investigations.

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

The purpose of the investigation was to obtain additional geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation, retention, drainage, footings, on-grade floor slabs, groundwater and external paved areas.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out between 24 and 27 May 2016 and comprised:

- Five boreholes (BH201 to BH205) auger drilled to depths ranging between 4.1m and 5.35m below existing ground level using our track mounted JK205 and JK308 drill rigs. Four of the boreholes (BH201 to BH204) were extended by diamond core drilling using NMLC coring techniques to final depths ranging between 8.61m and 12.31m.
- Two test pits (TP206 to TP207) excavated adjacent to existing building walls in an attempt to expose existing footings. The test pits were excavated to respective depths of 0.65m and 0.7m below existing surface levels.
- One borehole (BH206) hand auger drilled from the base of TP206 to a refusal depth of 0.7m below existing surface levels.
- Two Dynamic Cone Penetrometer (DCP) tests (DCP206 and DCP207) carried out adjacent to the test pits. The DCP tests were extended to respective refusal depths of about 0.3m and 1.1m below existing surface levels.

Prior to commencement of the fieldwork, the test locations were electro-magnetically scanned for the presence of buried services by a specialist sub-contractor.

The test locations, as indicated on the attached Figure 2, were set out by taped measurements from existing surface features. The approximate surface RLs at the test locations were interpolated between spot levels shown on the provided survey plan. The survey datum is the Australian Height Datum (AHD).

The state of compaction of the fill and density of the natural sands were assessed from the Standard Penetration Test (SPT) ‘N’ values and the DCP test results. The strength of the weathered bedrock within the augered portions of the boreholes was assessed from observation of drilling resistance when using a tungsten carbide (‘TC’) bit and examination of the recovered rock cuttings. The strength of the bedrock within the cored portion of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with laboratory Point Load Strength Index testing.

Groundwater observations were made in the boreholes and test pits during drilling/excavation and on completion of auger drilling, coring and excavation. We note that water is used as part of the coring process, and therefore water levels at the completion of coring may not have stabilised in the short time period after drilling. However, standpipes (together with data loggers) were installed in BH201, BH202, BH203 and BH205. The standpipes were installed to the following depths:

- BH201: 8.8m depth, with a response zone in the sandstone bedrock from 4.5m to 8.8m depth.
- BH202: 4.3m depth, with a response zone in the soil profile from 0.4m to 4.3m depth.
- BH203: 8.6m depth, with a response zone in the sandstone bedrock from 4m to 8.6m depth.
- BH205: 3.9m depth, with a response zone in the soil profile from 0.5m to 3.9m depth.

In addition, the standing water levels were also recorded in the standpipes installed during our previous investigation in BH101, BH104 and BH106.

We note that we returned to the site on 23 June 2016 to download the data loggers and record the standing water levels in the standpipes installed during our previous investigation. No further longer term groundwater monitoring has been carried out. For more details of the investigation procedures reference should be made to the attached Report Explanation Notes.

The fieldwork was carried out under the direction of our geotechnical engineer (Tristan Piat), who was present full-time on site, and set out the test locations, directed the buried services scan, logged the encountered subsurface profile, prepared the test pit cross sectional sketches, nominated in-situ testing and sampling and directed installation of the standpipes. The borehole logs (which include field test results, Point Load Strength Index test results and groundwater observations) test pit cross sectional sketches (Figures 6 and 7) and the DCP test results are attached, together with a glossary of logging terms and symbols used. .

The recovered rock core was returned to the Soil Test Services (STS) NATA registered laboratory where it was photographed and Point Load Strength Index Tests completed. A summary of the Point Load Strength Index tests and estimated Unconfined Compressive Strengths are presented in the attached STS Table B. The core photographs are included opposite the relevant cored borehole log. Selected rock chip samples were also returned to the STS NATA registered laboratory, for moisture content testing. The results are summarised in the attached STS Table A.

A contamination screen of site soils and groundwater was outside the agreed scope of the investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

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; see Figure 1.

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' [Block C]). 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^o; a trace of a dilapidated steel soldier pile wall with timber infill panels was also 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^o. 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^o.
- 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 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 and presented in our previous report dated 14 March 2011 were based on the results of boreholes JK6 and JK11 to JK15 (drilled in 1991), boreholes ML1, ML3 to ML5, ML7 to ML10 and ML12 (drilled in 2002) and BH101 to BH107 (drilled in 2011). Reference should be made to the borehole logs presented in Appendix A for detailed descriptions of the subsurface conditions at each borehole location. 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.

Our current subsurface investigations have revealed a similar subsurface profile as described above. A summary of the pertinent subsurface conditions encountered during our current subsurface investigation and, where relevant, details from our previous subsurface investigation are presented below. The attached Figures 3, 4 and 5 present sections providing graphical borehole summaries; their locations are indicated on the attached Figure 2.

Fill

Sandy fill with varying gravel content was encountered in all the boreholes for the current investigation and the thickness ranged between 0.4m (BH201) and 3.2m (BH204). We note that BH206 was terminated within the fill at a hand auger refusal depth of 0.7m. TP206 and TP2-7 were terminated within the fill at respective depths of 0.65m and 0.7m. The fill was assessed to be moderately or poorly compacted.

In our previous investigations sandy or clayey fill with varying gravel content was encountered from surface level or beneath paved surfaces in boreholes JK6, JK11 to JK15, ML1, ML9 and BH101 to BH107. BH107 was terminated in the fill at 1.3m depth. 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, moderately or well compacted.

Relic (Old) Topsoil

We note that our previous investigations encountered a relic topsoil layer 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). No relic topsoil was encountered in the current investigations.

Natural Soils

In this current investigation natural sands (occasionally silty) were encountered beneath the fill in all the boreholes except BH206. The sands were generally medium dense, but were loose on first contact to 2.2m depth in BH201, below which they improved to medium dense.

The natural sands encountered in our previous investigations were, on first contact, generally loose or medium dense (very loose in borehole ML3). In BH102 the sands were loose over the full depth of the borehole. In a selection of the boreholes drilled in 2002, dense sands were encountered from depths ranging between about 3m (ML5) and 6m (ML7) and in BH104 a dense layer (1.5m thick) was encountered at 3m depth.

In boreholes JK6 and ML12 drilled during our previous investigations, 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

In this current investigation weathered sandstone bedrock was encountered in all the boreholes except BH206 beneath the natural sands at depths ranging between 3.8m (BH203) and 6m (BH204). The recorded RLs of the top surface of the bedrock indicate that the bedrock surface steps down to the north-west from the south-eastern corner. The top surface of the bedrock was encountered at about RL41.2m (BH203) and RL33.6m (BH204).

On first contact the sandstone bedrock was generally assessed to be distinctly weathered and of at least low strength. With depth, the sandstone generally improved to distinctly or slightly weathered and medium and high strength.

Within the cored sections of the boreholes the following defects were recorded:

- Occasional bedding partings dipping at between 8° and 23°.
- Occasional sub-horizontal extremely weathered seams and clay seams, ranging between about 1mm and 110mm thickness were encountered.
- A few planar and occasionally undulating joints dipping at between 25° and 90°.

The following core loss zones were also encountered:

- In BH201 at 8.65m depth; about 0.04m thick.
- In BH202 at 8.22m depth; about 0.04m thick.
- In BH204 at 6.17m depth; about 1.61m thick.

The above zones of core loss may be interpreted as representing clay seams, extremely weathered seams, fractured bands and/or or crushed zones.

In our previous investigations weathered sandstone bedrock was encountered in boreholes JK6, JK11, JK15, ML4, ML5, ML7 to ML10, ML12 and BH101 to BH106 at depths ranging between 0.6m (JK11) to 6.5m (ML7). The recorded RLs of the top surface of the bedrock indicated that it stepped down to the west from the eastern site boundary to the eastern side of The ‘Burger Centre’.

The quality of bedrock encountered in the previous cored boreholes was broadly similar to that encountered in this current investigation, although occasionally extremely weathered bedrock of extremely low strength was encountered. A summary of the defects encountered in the cored portions of the previous boreholes is presented below:

- The sandstone bedrock was horizontally bedded with cross bedding dipping at 20^{o}.
- Occasional planar jointing was recorded with dips ranging from 40^{o} to vertical.
- Sub-horizontal extremely weathered zones and clay seams were encountered and ranged between 2mm and 180mm thickness.
- Zones of core loss were noted in boreholes ML5, ML9, BH102, BH103 and BH106 were about 0.11m, 0.05m, 0.49m, 0.44m and 1.66m thick, respectively. .

In accordance with Table 1a of the “Engineering Classification of Shales and Sandstones in the Sydney Region”, as revised by Pells et al 1998, below the top surface of the bedrock, the sandstone may be classified as follows:

BH	Surface RL (m AHD)	Depth(m)/RL Top of Class V	Depth(m)/RL Top of Class IV	Depth(m)/RL Top of Class III	Depth(m)/RL Top of Class II	Depth(m)/RL Top of Class I
JK6*	44.2	-	6.1/38.1	-	-	-
JK11*	38.5	-	0.6/37.9	-	-	-
ML4	42.5	-	5.7/36.8	6.4/36.1	9.3/33.2	-
ML5	37	6.1/30.9	3.4/33.6 6.7/30.3	-	-	-
ML8	43.9	-	5.1/38.8	6.3/37.6	6.8/37.1	-
ML9	43.8	4.3/39.5	4.9/38.9	8.1/35.7	-	-
ML10	42.8	5.1/37.7 7.7/35.1	5.8/37	-	8.8/34	-
101	42.01	-	4.3/37.71	6/36.01	7.8/34.21	-
102	45.58	3.3/42.28	4.5/41.08	-	-	-
103	41.95	5.6/36.35	7.6/34.35	9.7/32.25	-	-
104	40.52	5.4/35.12	5.9/34.62	8.4/32.12	-	-
105	38.38	-	5.1/33.28	5.5/32.88	-	-
106	41.95	4.6/37.35	7.9/34.05 & 10.4/31.55	8.8/33.15 & 11.5/30.45	-	-
201	39.8	-	4.5/35.3 8.12/31.7	5.35/34.45	8.69/31.1	-
202	41.2	4.7/36.5	6.38/34.82	8.22/32.98	7.2/34.0 8.7/32.5	-
203	45.0	-	3.8/41.2	7.22/37.78	7.71/37.29	-
204	39.6	6/33.6	-	9.17/30.43	-	-
205*	39.5	4/35.5	-	-	-	-

* Classification based on auger drilled borehole.

Groundwater

In this current investigation, groundwater was encountered whilst auger drilling BH201, BH202, and BH24 at depths of 3.5m, 4m, and 4.9m, respectively. On completion of auger drilling, standing water levels were recorded in BH201, BH202 and BH204 at depths of 3.5m, 4.4m and 4.7m, respectively. These depths are equivalent to RL36.3m, RL36.8m and RL34.9m, respectively.

In the cored boreholes, standing water levels were recorded within a short time of completion core drilling in BH201, BH202, BH203 and BH204, at depths of 4.8m, 4.4m, 2.1m and 5.1m. 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 to near full water flush returns were noted in all the cored boreholes indicating a relatively low permeability rock mass.

On the day the data loggers were installed in the standpipes in BH201, BH202, BH203 and BH205 standing water levels were recorded at 3.6m depth (RL36.2m) in BH201 and 4.1m in BH203 (RL40.9m). The standpipes in BH202 and BH205 were noted as ‘dry’. On 23 June 2016, standing water levels were recorded in BH201, BH202, BH203 and BH205 at depths of 3.05m (RL36.75m), 4.26m (RL36.94m), 3.88m (RL41.12m) and 3.8m (RL35.8m).

In our previous investigations groundwater was encountered during auger drilling in boreholes JK15, ML3, ML4, ML10, BH104, BH105 and BH106 at depths of 2.7m, 1.2m, 5.3m, 5.1m, 3.5m, 2.8m and 3.3m, respectively. On completion of auger drilling, standing water levels were recorded in boreholes JK6, JK12, BH103, BH104, BH105 and BH106 at depths of 5.1m, 2.8m, 2.6m, 3.5m, 2.8m and 3.3m, respectively equivalent to approximately RL39.1m, RL36.2m, RL39.35m, RL37.02m, RL35.58m and RL38.65m, respectively. 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, ML12, BH101, BH102, BH103 and BH105 at depths of 5m, 6.2m, 1.4m, 8.0m, 2.0m, 4.8m, 2.7m, 1.6m, 1.4m, 0m and 2.3m, respectively. These depths are equivalent to RL37.5m, RL35.6m, RL37.2m, RL35.9m, RL41.8m, RL38.0m, RL41.2m, RL40.4m, RL44.18m, RL41.05m and RL36.08m, respectively. 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 except BH105 below 9m depth, indicating a relatively low permeability rock mass.

Groundwater levels were recorded in the standpipes installed in BH101, BH104 and BH106 on 18 February 2011 at depths of 4.68m, 4.02m and 4.1m, respectively equivalent to approximately RL37.33m, RL36.5m and RL37.85m. On 23 June 2016, BH104 and BH106 were ‘dry’ and a standing water at 3.96m depth was recorded in BH101. We note that the standpipes in BH104 and BH106 had ‘silted up’ to depths of about 4.9m and 2m, respectively.

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. Over the subject site (Blocks D, E and F), the monitoring of groundwater levels during the geotechnical investigation and subsequent recording of groundwater levels in the standpipes between 2011 and the current fieldwork has indicated hydraulic gradients down to the west and north from the eastern and southern sides of the site, respectively.

During the monitoring period for the data loggers installed in BH201, BH202, BH203 and BH204, we note the following:

- BH201: the attached Figure 8 indicates the highest recorded groundwater level in the standpipe installed in the bedrock was about 3m depth (RL36.8m) on 7 June 2016, following rainfall events on 4, 5 and 6 June 2016 of 74.8mm, 102mm and 61mm, respectively. At the end of the current monitoring period the groundwater level had remained essentially at RL36.8m.
- BH202: the attached Figure 9 indicates that groundwater was recorded in the standpipe installed in the soil profile from 6 June 2016 during and following rainfall events on 4, 5 and 6 June 2016 of 74.8mm, 102mm and 61mm, respectively. From 6 June 2016 to the end of the monitoring period the groundwater level has risen from about 3.8m depth (RL37.4m) to 3.4m (RL37.8m).
- BH203: the attached Figure 10 indicates the highest recorded groundwater level in the standpipe installed in the bedrock was about 4.5m depth (RL40.5m) on 9 June 2016, following rainfall events on 4, 5 and 6 June 2016 of 74.8mm, 102mm and 61mm, respectively. At the end of the current monitoring period the groundwater level has slightly decreased to 4.8m depth (RL40.2m).
- BH205: the attached Figure 11 indicates that groundwater was recorded in the standpipe installed in the soil profile from 5 June 2016 during and following rainfall events on 4 and 5 and 6 June 2016 of 74.8mm, 102mm and 61mm, respectively. From 5 June 2016 to the end of the monitoring period the groundwater level rose to a maximum of about 3.3m depth (RL36.2m) on 8 June 2016 with only a gradual reduction to 3.4m (RL36.1m).

3.3 Test Pit Results

The details exposed in TP206 and TP207 are indicated on Figures 6 and 7.

TP206 indicated that at this location, the existing Block C wall was supported on concrete founded at a depth of about 0.6m within poorly compacted sandy fill. The concrete stepped out a horizontal distance of 0.33m from the wall. Based on our previous involvement, Block A was supported on pile footings founded in bedrock. The concrete therefore most likely represents a pile capping beam. See Figure 6.

TP206 indicated that at this location the existing clad wall was supported on concrete which stepped out a horizontal distance of 0.08m from the wall then extended down to a concrete surface that covered the base of the test pit at 0.7m depth. Based on the results of DCP207, the concrete extended out a maximum of about 1.3m from the face of the wall. The refusal of DCP207 at about 1.1m depth may indicate a further concrete surface or an inclusion within the fill. Based on our previous involvement, Block C was supported on pile footings founded in bedrock. The concrete immediately below the wall therefore most likely represents a pile capping beam although the purpose of the concrete extending out about 1.3m from the wall face is not known. See Figure 7.

3.4 Potential Contamination Issues

We note that in our previous investigation in 2011, hydrocarbon odours were recorded in BH103, BH104 and BH105 together with some creosote inclusions towards the base of the fill in BH103. This information was passed on to McLachlan Lister Pty Ltd in an email dated 15 February 2011 and we understand that the environmental consultants have been provided with this information.

Over the south-eastern corner of the site there are two existing pits that have been used to store and contain contaminated materials sourced from previous bulk excavations within the site. It is likely that a number of proposed pile footings will penetrate these pits with associated potential for detrimental environmental and WH&S impacts. Further advice from the environmental consultants will need to be sought. We note that to reduce the amount of spoil generated by the piling operations, specialised displacement piles may be considered. Further advice from the piling contractor would need to be sought in relation to the quantities of spoil that would be generated.

3.5 Laboratory Test Results

The moisture content tests on recovered rock chip samples confirmed our field assessment of rock strength.

The point load test results indicated that the rock cored ranged between very low and high strength but was generally of medium strength with estimated Unconfined Compressive Strengths (UCS) varying from 2MPa to 30MPa.

Previous laboratory test results in 2011 indicated the following:

- The four day soaked CBR test of the sandy fill sample in BH107 compacted to 98% of the Standard Maximum Dry Density (SMDD) has returned a value of 25%. This value is at the upper end of the expected range for such sandy soils. We therefore recommend that a lower value of 15% be adopted for design of pavements to allow for variability in the density of the sandy soil subgrade and with regard for previous CBR testing completed during our previous investigations. The natural moisture content of the sample tested was about 4.1% 'dry' of the Standard Optimum Moisture Content (SOMC).

- The natural sand in BH101 was slightly acidic with a recorded pH value of 6.7, had a recorded chloride content value of <10mg/kg and a recorded sulphate content value of <10mg /kg.
- The natural sand in BH102 was slightly alkaline with a recorded pH value of 7.5, had a recorded chloride content value of <10mg/kg and a recorded sulphate content value of <10mg /kg.
- The natural sand in BH105 was moderately alkaline with a recorded pH value of 8.4, had a recorded chloride content value of <10mg/kg and a recorded sulphate content value of <10mg /kg.
- The natural silty sand in BH106 was slightly alkaline with a recorded pH value of 7.5, had a recorded chloride content value of 57mg/kg and a recorded sulphate content value of <10mg /kg.
- The point load test results indicated that the rock cored ranged between very low and high strength but was generally of medium to high strength with estimated Unconfined Compressive Strengths (UCS) varying from <1MPa to 36MPa.

Previous laboratory test results in 1991 and 2002 indicated the following:

- The laboratory soil pH test results on natural sand samples ranged between 7.2 and 7.4 indicating slightly acidic 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 and Excavation

4.1.1 General

To maintain the stability of sections of existing buildings that will remain and neighbouring existing buildings, structures and/or paved surfaces lining and/or close to the western site boundary and the

King Street frontage, demolition and excavation will need to be completed with care. This will be of particular concern below the south-western portion of Block A (where the tunnel access is proposed), adjacent to Block C (the ‘Burger Centre’) and over the south-western corner of the site.

The structural engineer will need to detail any propping of existing sections of buildings and structures that will remain and/or line the site boundaries over the south-western corner of the site.

Based on the size of the existing buildings, our past experience at the site and the results of TP206 and TP207, Blocks A, B and C were supported on pile footings socketed into bedrock with on-ground floor slabs provided. Care will need to be exercise in order to support remaining sections of pile capping beams, on-grade floor slabs adjacent to proposed excavations. We also note the following:

- Areas of buried concrete immediately to the west of Block C are likely to be encountered. The purpose of the concrete is not known and all available ‘as built’ drawings for Block C be sourced to provide further information. If such records are not available, then excavations will need to be carefully completed at the commencement of works to determine the extent and function of the concrete. The concrete will pose difficulties when installing shoring piles and will need to be removed.
- The proposed tunnel excavation will extend the Block A floor slab level. We note that a series of gravel drains was installed below this slab and localised dewatering may well be required in addition to maintaining the integrity of the existing drainage.

Prior to commencement of removal of 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 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. However, within Block A, had held equipment may well be required and/or smaller tracked excavators. Further comments regarding use of rock breakers are provided in the Section 4.1.3, below.

4.1.2 Excavation

Excavation recommendations provided below should be completed by reference to the Safe Work Australia Code of Practice ‘Excavation Work’, dated July 2015.

The outline of the proposed basement levels below Blocks E and F and the outline of the proposed buildings are indicated on the attached Figure 2. Excavations to maximum depths of about 6.5m and 4.5m will be required to achieve the design subgrade levels over the south-western and south-eastern portions of the site, respectively. In addition, access tunnel excavations to a maximum depth of about 3m below the current Block A floor level will also be required to achieve design subgrade levels.

On the basis of the investigation results, following demolition, the proposed excavations will encounter the soil profile and penetrate weathered sandstone bedrock towards the base of the Basement Level 2 excavation and over the northern and south-eastern portions of the Basement Level 1 excavation below Block D. Topsoil and/or root affected soils should be stripped and separately stockpiled for re-use in landscape areas as such soils are not suitable for re-use as engineered fill.

The areas of 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 would be beneficial with respect to trafficking the subgrade. In this regard there will also be the need to provide an appropriate working platform for piling rigs. The working platform may be significantly thicker than the above mentioned sacrificial layer; further advice is provided in Section 4.3.3, below.

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, we expect to be excavated using rock breakers, rock grinders and ripping attachments to the tracked excavator. Rock breakers would assist in completing detailed rock excavations for footings, service trenches, lift pits etc and also trimming any rock excavation faces.

Care will be required to control ground vibrations associated with the use of rock breakers, such as the provision of rock saw cuts (see Section 4.1.3, below). Rock saws may also be used to create ‘smooth’ finishes on cut faces and aid in detailed excavation of footings, services trenches etc. Where rock breakers, rock saws and/or rock grinders are used, the resulting dust should be suppressed with water.

A review of the relevant Acid Sulphate Soil Risk Map for Botany Bay (Edition 2) dated December 1997 published by the Department of Land and Water Conservation (now DECCW) indicates that the subject site lies in a zone of ‘No Known Occurrence’ of acid sulphate soils. Consequently, we consider that an Acid Sulphate Soil Management Plan will not be required. However, we recommend that this be confirmed by the project environmental consultants.

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 should be taken where rock breakers are used during demolition and for excavation of sandstone bedrock so that ground vibrations do not adversely affect nearby neighbouring structures and paved surfaces and adjacent sections of existing buildings within the site that will remain. If there is any cause for concern then demolition and/or excavation should cease and further geotechnical advice sought.

While the rock breakers are being used, continuous vibration monitoring of the neighbouring buildings and structures to the west will be required, to confirm that peak particle velocities (PPV) fall within acceptable limits. Subject to the results of the dilapidation reports (see Section 4.1.4, below), we would recommend that the PPV along the western site boundary do not exceed 5mm/sec during bedrock excavation using rock breakers. Should higher vibrations be measured they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be acceptable depending on the vibration frequency. We note that this vibration limit will reduce the risk of vibration damage to the neighbouring building and structures. However, these vibrations may still result in discomfort to occupants of the neighbouring buildings and residents and staff members within the site. If excessive vibrations are confirmed, it will be necessary to use lower energy equipment such as smaller rock breakers and/or use rock saw cuts with the base of the slot maintained below the level at which the rock breaker is being used.

Where rock breakers are used, to reduce vibrations we recommend that the rock breaker be continually orientated towards the face, and be operated one at a time and in short bursts only to reduce amplification of vibrations.

4.1.4 Dilapidation Surveys

Detailed dilapidation reports will need to be completed on the neighbouring buildings and structures adjacent to the southern half of the western site boundary. In addition, Council may also require that dilapidation survey reports be completed on their assets lining the street frontages, i.e. the paved footpaths, the roadways and kerbs and gutters lining the southern and eastern site boundaries. The property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the client in defending themselves from unfair damage claims.

4.2 Seepage and Dewatering

The pattern of groundwater levels recorded in our previous investigations within and adjacent to the subject site tended to indicate a hydraulic gradient down from the eastern and southern sides of the site which reflected the fall of topography and generally followed the fall of the underlying bedrock profile. Groundwater levels recorded in the current investigation have indicated a generally similar pattern.

We note that during construction of the existing buildings over the northern portion of the site (Blocks A and B), 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. The groundwater levels recorded in the standpipes installed in 2011 indicated a groundwater level ranging between approximately RL36.5m and RL37.85m.

The current data loggers have indicated that following the heavy and prolonged rainfall between 4 and 6 June 2016 groundwater levels rose to maximum levels of between RL36.2m and RL37.8m in the sandy soil profile and between RL36.8m and RL40.5m in the bedrock. We note that the standing water levels recorded in BH201, BH202 and BH205 are above the proposed Basement Level 2 design subgrade level. However, the standing water level recorded in BH201 may indicate the piezometric head of groundwater within water carrying defects within the rock mass which may not be encountered in the excavation. The elevated groundwater in the bedrock over the south-eastern corner of the site (RL40.5m in the bedrock) is below the proposed Lower Ground Level of Block D (RL41.5m). Again, the standing water level recorded in BH203 may indicate the piezometric head of groundwater within water carrying defects within the rock mass.

Based on the above, groundwater seepage into the basement excavations below Blocks E and F can be expected within the sandy soil profile at, or above, the bedrock surface profile, particularly during and following periods of heavy or prolonged rainfall; as indicated by the data loggers. Depending on the amount of rainfall preceding bulk excavation, the proposed excavations may well extend below the groundwater level in the sandy soil profile over the Basement Level 2 excavation and the tunnel excavation below Block A and possibly below the Block D Lower Ground Level excavation. Where bedrock is encountered, concentrated flows may occur if water carrying defects daylight into the excavation face.

Based on the above, we recommend that contingency be made for dewatering the excavations, particularly Basement Level 2 over the south-western portion of the site, and the tunnel excavation below Block A.

De-watering of excavations can lead to lowering of groundwater levels outside of the site. The zone of groundwater drawdown can potentially extend well in excess of 50m outside the site boundaries and lowering of groundwater levels may cause settlement of nearby high level footings, pavements, buried services, slabs-on-grade etc. As a guide, for a 2m lowering of the groundwater level, settlements of less than 10mm are expected, depending on the thickness and composition of the underlying soils. Consequently, dewatering will need to be carried out with care and we recommend that computer based modelling of the proposed dewatering be undertaken.

The proposed basement and tunnel shoring systems will need to be extended to sufficient depth below basement level in order to provide a cut-off and reduce potential drawdown outside the excavation. The sandstone bedrock should provide an appropriate cut-off.

Recharge of extracted water outside the basement excavations can assist in reducing potential drawdown and we would expect this to be achievable within the site.

Where required, groundwater would need to be lowered at least to 1m below bulk excavation levels prior to excavation and continue throughout the construction period. The quantity of groundwater to be extracted per day may be estimated based on completion of computer based modelling of the dewatering, as recommended above. We can complete infiltration tests in the standpipes, if required, to assist in computer based groundwater modelling.

The rate at which the groundwater can be extracted will be a function of the mass permeability of the soil and bedrock, the capacity and number of pumps used. The disposal of extracted groundwater will be affected by practical considerations such as whether or not the groundwater can be discharged to the stormwater or sewer system (with or without on-site treatment). If the groundwater has to be treated before discharge into the sewer or stormwater system and/or if the water has to be removed from site using tankers then this will further slow down the rate at which groundwater can be extracted. Careful sequencing of the de-watering and discharge of extracted groundwater would clearly be required.

With regard to the above, the NSW Office of Water (now Department of Primary Industries Water [DPIW]) are likely to impose a number of conditions, including the following:

- No extraction of groundwater other than for temporary construction purposes.
- Prior to excavation, groundwater measurements from at least three standpipes to be completed and a report provided to DPIW.
- An estimate of the total volume of groundwater to be extracted to be provided to DPIW together with the details of the analyses used to determine the volume of water.
- Testing of groundwater to assess its suitability for disposal into the stormwater or sewerage system.
- A groundwater monitoring and testing program for the dewatering to be completed and submitted for review to DPIW.
- Implementation of the approved groundwater monitoring and testing program and a final report submitted to DPIW on completion.

4.3 Retention

4.3.1 Temporary Batters

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. However, due to the expected groundwater levels, such temporary batters are unlikely to be feasible, particularly over the south-western portion of the site

Some instability of temporary excavation batters may occur at the soil-bedrock interface within excavations if groundwater seepage occurs and especially after rain periods. Sand bagging may be required to stabilise the toe of batter slopes through the soils. This is likely to be more applicable over the south-eastern portion of the site.

4.3.2 Retention Options

Where battering cannot be accommodated within the site geometry, or is not preferred (e.g. where dewatering of the excavation is expected to be required; Basement Level 2 and the tunnel access), a full depth engineered retention system will need to be installed prior to excavation commencing.

A full depth engineered retention system is likely to be required to support the tunnel sides although access will be difficult for piling equipment below Block A and advice will be required from specialist piling contractors.

A piled wall retention system is suitable for the site. Due to the potentially collapsible nature of the soil profile, groundwater levels, the need to control potential movement of adjacent or neighbouring ground surfaces, buildings and structures and dewatering considerations, a grout injected (cfa) secant piled wall will be required. As the proposed excavation will extend below groundwater level, maintaining verticality of the secant piles will be critical in order to reduce the potential for seepage and/or loss of retained sands through gaps in the secant piles. We therefore recommend that a 'double rotary' cfa pile installation system be adopted. Any gaps between adjacent secant piles must be grouted immediately as soil loss and seepage could cause settlements of the ground adjoining the retention system.

An alternative suitable retention system would be a proprietary in-situ soil and grout mixing system within a low strength cement/bentonite slurry such as provided by Wagstaff or Bachy Soletanche. The system involves construction of 'interlocking' panels of up to about 2.4m width which are formed in a 'hit 1 miss 1' sequence. This is also a low vibration method, generates nominal amounts of spoil and can provide a satisfactory final wall surface. However, the panels would need to uniformly key into bedrock in order to provide an adequate 'cut-off'. This would be difficult to achieve if the bedrock surface is particularly uneven and/or of variable strength; further advice should be sought from specialist contractors.

Over the south-eastern portion of the site, consideration could be given to a contiguous pile wall. 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 be able to be incorporated into the footing system. In addition, in some instances the embedment depth required for stability considerations may extend into the sandstone bedrock which the sheet piles would not be able to penetrate.

The toe of the piled walls should be embedded below bulk excavation level to sufficient depth and/or appropriately keyed into bedrock to satisfy stability and bearing considerations and provide an adequate ‘cut-off’ to groundwater. In addition, retention piles and/or the soil/grout mix panel walls which penetrate sandstone bedrock may be incorporated into the footing system. Further comments regarding the load carrying capacity of the piled walls are provided in Section 4.4, below.

The full depth retention system 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 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.

Construction of the retaining system and anchors should be of a high quality and only experienced contractors should be employed.

4.3.3 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 into 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.

We note that as the cfa piles will be socketed 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.

We recommend that a site trial be undertaken well away from any potentially affected structures with the geotechnical engineer in attendance.

The piling rigs may need to be provided with a suitable working platform determined by a geotechnical engineer. The design of the working platform will need to be based on the loadings and track dimensions supplied by piling contractor for the specific piling rig being proposed. Further, the assessment of the working platform thickness will need to be based on the methodology outlined in BRE 2004 *'Working Platforms for Tracked Plant'*.

The working platform will need to be constructed using DGB20 (or a similar durable granular material approved by the geotechnical engineer) compacted to 95% Modified Maximum Dry Density (MMDD) using a large roller. Areas of soil subgrade will also need to be prepared in a similar manner as described in Section 4.6.1, below.

Density tests should be regularly carried out on the working platform materials to confirm the above density has been achieved. The frequency of density testing should be at least one test per layer per 2500m^2 or three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. However, our preference would be for Level 1 control of fill placement and compaction, in accordance with AS3798-2007.

4.3.4 Retention Design 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 basement shoring system should be uniformly founded below bulk excavation level in weathered sandstone bedrock. Allowable bearing pressure recommendations are provided in Section 4.4, below.
- For progressively anchored or propped walls, and the tunnel walls, where lateral movements need 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 progressively anchored or internally propped walls, where minor movements can be tolerated (such as along the south-eastern portions of Block D, the northern and southern sides of Block F and the southern side of Block E), we recommend the use of a uniform

rectangular earth pressure distribution of 6H kPa for the soil profile, where 'H' is the retained height in metres.

- For design of cantilever walls which will be propped by the structure, 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 soil profile and any extremely weathered sandstone bedrock, assuming a horizontal backfill surface.
- For new landscape retaining walls, where we assume some minor movements of the walls may be tolerated, they may be designed using a triangular lateral earth pressure distribution and 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 and reduced to 10kN/m^3 below the groundwater level.
- Any surcharge affecting the walls (including adjacent high level footings, traffic, landscaping, compaction stresses, sloping retained surfaces etc) should be allowed in the design using the appropriate earth pressure coefficient from above.
- The basement retention system will need to be designed to withstand hydrostatic pressures. A groundwater level of at least RL37.8m should be adopted for the design of Basement Levels 1 and 2 and the tunnel access, although this will need to be confirmed following further monitoring of groundwater levels. In addition, the basement design should be checked for an 'ultimate' design case of the groundwater level coincident with the surrounding surface level. Reference should be made to Section 4.5, below for further comments in this regard.
- Contiguous piled walls over the south-eastern corner of the site (if selected) must be designed as permanently drained and PVC pipes should be installed at nominal 1.2m horizontal spacing just above the adjacent floor level and bedrock levels, where appropriate. 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.
- Any conventional retaining walls and landscape retaining walls 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 key design.

- For the design of shoring piles socketed into sandstone of at least very low strength, it is recommended that maximum allowable lateral toe resistance of 200kPa be used for the design of rock sockets. 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^o 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.

4.3.5 Tanking

Groundwater levels may be at or just below design subgrade levels for Basement Level 2 and the access tunnel. Heavy rainfall and/or flood events will raise groundwater levels above design subgrade levels. If preferred, the basement levels below Blocks E and F and the tunnel may be designed as tanked and a design groundwater level at RL38m, based on the data loggers, is recommended. However, this design groundwater level should be reviewed in with regard to 50 year ARI and 100 year ARI flood events to determine the most appropriate design groundwater levels; further advice from a hydraulic engineer and/or Council will be required in this regard. Further groundwater monitoring would also assist in verifying appropriate design groundwater levels. 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 4.4.3, above if the self-weight of the buildings does not provide sufficient resistance.

4.3.6 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.

4.4 Footings

4.4.1 Overview

As outlined in Section 3.2 above, the expected quality of sandstone bedrock across Blocks D, E and F and the tunnel are as follows:

BH	Building Location	Surface RL (m AHD)	Depth(m)/RL Top of Class V	Depth(m)/RL Top of Class IV	Depth(m)/RL Top of Class III	Depth(m)/RL Top of Class II
JK6	**BLOCK D FFL RL41.5m**	44.2	-	6.1/38.1	-	-
ML4		42.5	-	5.7/36.8	6.4/36.1	9.3/33.2
ML8		43.9	-	5.1/38.8	6.3/37.6	6.8/37.1
ML9		43.8	4.3/39.5	4.9/38.9	8.1/35.7	-
ML10		42.8	5.1/37.7 7.7/35.1	5.8/37	-	8.8/34
101		42.01	-	4.3/37.71	6/36.01	7.8/34.21
102		45.58	3.3/42.28	4.5/41.08	-	-
203		45.0	-	3.8/41.2	7.22/37.78	7.71/37.29
JK11	**BLOCKS E AND F (INCLUDING TUNNEL BELOW BLOCK A) FFL RL35.5m**	38.5	-	0.6/37.9	8/101.5	12.3/97.2
ML5		37	6.1/30.9	3.4/33.6 6.7/30.3	-	-
103		41.95	5.6/36.35	7.6/34.35	9.7/32.25	-
104		40.52	5.4/35.12	5.9/34.62	8.4/32.12	-
105		38.38	-	5.1/33.28	5.5/32.88	-
106		41.95	4.6/37.35	7.9/34.05 & 10.4/31.55	8.8/33.15 & 11.5/30.45	-

Weathered sandstone bedrock is expected to be encountered at, or a short depth below bulk excavation level and locally within a maximum of about 2m depth below bulk excavation levels over the north-western portion of Block F and the northern end of the tunnel. With regard to Block D, bedrock is expected to be a maximum of about up to 4.5m below design subgrade levels. Over the southern portion of Block D, the bedrock surface is expected to be a maximum depth of about 6.1m below existing surface levels.

Pad or strip footings will be appropriate over the majority of Blocks E and F locally pile footings will be required over the north-western portion of Block F and the northern end of the tunnel. Piles will be required over the footprint of Block D. Provided the excavations are appropriately dewatered then bored piles may be feasible over Blocks E and F and the tunnel and may be feasible over the footprint of Block D. However, provision would need to be made for the use of temporary liners to support the drill holes. Alternatively, cfa piles may be used.

For the various Classes of sandstone bedrock the pad, strip and pile footings may be designed using the following parameters:

Sandstone Class	Allowable End Bearing Pressure	Allowable Shaft Adhesion (for piles)	
		Compression	Tension
V	800kPa	80kPa	40kPa
IV (or better)	3,000kPa	300kPa	150kPa

We note that higher allowable end bearing pressures may be feasible for the Class IV (or better) sandstone bedrock. However, with regard to the difficulties confirming the bedrock quality in pile footing drill holes, in particular with cfa piles, and the variable nature of the upper portion of the sandstone bedrock profile, in particular the presence of poor quality zones (such as extremely weathered seams), we consider that the allowable bearing pressure of 3,000kPa for Class IV (or better) sandstone bedrock is appropriate.

With regard to allowable ‘peak’ pressures for temporary live and wind loads of short duration the above values may be increased as follows:

Sandstone Class	‘Peak’ Allowable End Bearing Pressure	‘Peak’ Allowable Shaft Adhesion	
		Compression	Tension
V	1,000kPa	100kPa	50kPa
IV (or better)	3,500kPa	350kPa	175kPa

We provide below guidance on piled and shallow footings.

4.4.2 Pile Footings

As mentioned above, cfa piles are most suited to the site. However, the piles will need to penetrate medium and high strength sandstone (maximum estimated UCS value of 36MPa; see Section 3.5, above) and we recommend large capacity drilling rigs be used. The proposed piling contractor must be given a copy of this report so that piling rigs of appropriate size and capacity to penetrate medium and high strength bedrock are established at the site. We note that 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 which may detrimentally impact nearby surface levels or existing buildings within the site or paved surfaces lining the eastern and southern site boundaries.

We reiterate that over the south-eastern corner of the site proposed pile footings will likely penetrate the existing pits that have been used to store and contain contaminated materials sourced from previous bulk excavations within the site. To reduce the amount of spoil generated by the piling operations, specialised displacement piles may be considered. However, displacement piles may have difficulty penetrating medium or high strength bedrock. Further advice from the piling contractor would need to be sought in relation to the quantities of spoil that would be generated and the ability of such piles to penetrate medium or high strength bedrock.

Retention systems that are constructed can be incorporated into the footing system for the structure provided they are socketed into sandstone bedrock using the above allowable bearing pressures.

Alternatively, if the designer wishes to adopt limit state design methods, in accordance with Section 4.3.2 of AS2159-2000 “Piling-Design and Installation”, we have assessed the Average Risk Rating (ARR) to be 2.3 (which assumes on-site pile load testing would be carried out). Accordingly, the basic geotechnical strength reduction factor ($Ø_{gb}$) should be 0.56 or 0.64 for low or high redundancy systems, respectively. Advice on pile testing is provided in section 4.4.5, below.

Alternatively, if on-site pile load testing is not carried out, then the basic geotechnical strength reduction factor ($Ø_{gb}$) should be 0.52 or 0.60 for low or high redundancy systems, respectively.

In addition, the following ultimate values and ‘E’ values should be adopted for the sandstone bedrock classes when completing limit state design.

Sandstone Class	Ultimate End Bearing Pressure	Ultimate Shaft Adhesion		E
		Compression	Tension	
V	3,000kPa	300kPa	150kPa	50MPa
IV (or better)	9,000kPa	4,500kPa	2,250kPa	400MPa

If it is intended to incorporate existing piles into the new footing system, we note that it is our understanding that these piles were socketed into sandstone bedrock and designed on the basis of an allowable end bearing pressure of 3,000kPa. Consequently, the load carrying capacity of these piles should be checked against this value and any rock sockets that were formed may be assessed using the above allowable shaft adhesion values.

Pile footing sockets should also be checked by the structural engineer for control of potential uplift pressures based on the design groundwater levels outlined in Section 4.3.5.

4.4.3 Shallow Footings

Pad or strip footings founded in Class V or IV bedrock may be designed on the basis of the above allowable end bearing pressures. However, due to the potential for significant bands of poor quality sandstone (extremely weathered seams) to be present below the bases of any pad or strip footings, spoon testing of all pad or strip footings designed for an allowable bearing pressure of 3,000kPa should be carried out. If such poor quality bands are revealed during spoon testing of footing locations then the geotechnical engineer may request that either footings go deeper to encounter better quality bedrock or there be an increase in the plan area of pad or strip footings. Some allowance should be made for these types of variations.

4.4.4 Settlements

In accordance with Table 5a of the “Engineering Classification of Shales and Sandstones in the Sydney Region”, as revised by Pells et al 1998 ‘typical’ settlements for footings founded on bedrock with the above allowable end bearing pressures would be less than 1% of the pile diameter or footing width.

Consideration should be given to provision of movement control joints where portions of new buildings connect to existing buildings.

4.4.5 General

Excavations for 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, particularly if dewatering of the excavation below Bocks E and F and the tunnel is not implemented.

In addition, we recommend that any shallow footing excavation bases in sandstone bedrock be inspected (and spoon tested) by a geotechnical engineer to confirm the quality of the bedrock.

We recommend that the initial installation of cfa piled footings be witnessed by a geotechnical engineer. However, we note that the bases of cfa piles cannot be inspected and sample recovery from the drill hole is limited. Confirmation that the required quality of bedrock has been penetrated would therefore need to be based on the depth of the pile hole, reference to nearby borehole logs and information provided by the piling contractor, in particular, torque readings.

We note that the ARR provided in Section 4.4.2 above, assumed that on-site pile load testing would be carried out. In this regard, we recommend that at least 15% of all piles be tested and assumes that the piling contractor provides comprehensive records of the monitoring information for each pile. Pile testing should be completed in accordance with the requirements of AS2159-2009. Alternatively, as outlined in Section 4.4.2 above, on-site pile load testing may not be carried out and revised basic geotechnical strength reduction factors have been provided. We also recommend that whether or not pile load testing is carried out the piling contractor be requested to certify the load carrying capacity of the piles.

All footings should be drilled/excavated, cleaned, inspected (where applicable) and poured with minimal delay, (preferably on the same day as drilling/excavating). Water should be prevented from ponding in the base of pad or strip footing excavations bases in sandstone bedrock as this may lead to a softening of the base, particularly in areas of more weathered bedrock. If a delay in pouring is expected, consideration may be given to provision of a blinding layer of concrete to protect the base of pad or strip footing excavations, otherwise, over-excavation of any water softened material may be required.

4.5 Concrete Durability

Based on the advice provided in AS2159-2009 “Piling Design and Installation” for corrosion protection and durability and AS3600-2009 “Concrete Structures” the previous laboratory chemical test results have indicated the following:

For the natural sands and silty sands an A2 or ‘Mild’ Exposure Classification may be assigned based on Table 4.8.1 of AS3600 and Table 6.4.2(C) of AS2159, respectively.

4.6 Earthworks

Earthworks recommendations presented below should be complemented by reference to AS3798-2007 “Guidelines on Earthworks for Commercial and Residential Developments”.

4.6.1 Subgrade Preparation

Over areas of proposed piling rig working platforms, 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). We note that if larger capacity piling rigs are intended to be used then the roller size should be increased to say 10 tonne deadweight.
- 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 and/or over the northern and north-western portion of Block E where current site surface levels are below the design subgrade level.

4.6.2 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.

Backfill to conventional retaining walls should also comprise engineered fill. The excavated sands or well graded granular materials such as ripped or crushed sandstone and demolition rubble would be suitable for this purpose. Such fill should be compacted in horizontal layers as above using a hand held plate compactor. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

We note that if single sized granular material (or ‘no fines’ gravel) is used as backfill to retaining walls then only nominal compaction (with no compaction testing) will be required.

Piling rig working platforms, where required, should comprise well graded granular materials such as ripped or crushed sandstone and demolition rubble and compacted using a minimum 5 tonne deadweight smooth drum vibratory roller. Density testing of such materials may not be feasible and so placement and compaction of the fill should be completed under the direction of an experienced earthworks supervisor or geotechnical engineer.

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. For backfilling of localised excavations, such as behind retaining walls, service trenches or localised soft spots, testing should consist of one test per two layers per 50m^2. At least Level 2 testing of earthworks should be carried out in accordance with AS3798-2007. Any areas of insufficient compaction will require reworking.

4.7 On-Grade Floor Slabs, External Pavements and Drainage

4.7.1 General

Slab-on-grade construction is feasible for on-grade floor slabs and external pavements provided the areas of exposed sand subgrade are prepared, and any engineered fill is placed, in accordance with the advice outlined in Section 4.6, above.

Based on the previous four day soaked CBR test results and following proof rolling of the subgrade and compaction of engineered fill, as outlined in Section 4.6 above, we recommend that the design of floor slabs and external pavements over a sandy soil subgrade be based on a CBR value of 15%.

For design of concrete on-grade floor slabs and rigid pavement design, Short and Long Term Young’s Modulus values of 45MPa and 35MPa may be adopted, respectively for the sandy soils.

4.7.2 On Grade Floor Slabs and Drainage

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.

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 Basement 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 of Blocks E and F and the tunnel.

The proposed concrete on-grade floor slabs in a drained basement should be separated from all walls, footings etc. (i.e. 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.

4.7.3 Pavement Design and Drainage

We provide below advice on the design of both rigid and flexible pavements using the ‘Guide to Pavement Technology’ Part 2: Pavement Structural Design (AUSTROADS May 2008). Our advice has assumed a design CBR of 15%, a design period of 40 years, a design traffic value of 1.5 x 10^5 Equivalent Standard Axles (ESA), an ESA/HVAG (Heavy Vehicle Axle Group) ratio of 0.3 and a HVAG value of 5 x 10^5.

A flexible pavement design would comprise:
- An asphalt seal of maximum 40mm thickness.
- A minimum 100mm thick layer of unbound base material (DGB20).
- A 50mm thick layer of unbound subbase (DGS20).

For a rigid pavement with a concrete shoulder, a project design reliability of 95% (Load Safety Factor (LSF) of 1.2) and a design concrete flexural strength 4MPa, the rigid pavement would comprise:
- A 165mm thick concrete base.
- A minimum 125mm thick unbound subbase layer (DGS20).

The materials comprising the above mentioned DGB20 and DGS20 base and subbase materials, respectively should comply with the QA Specification 3051 “Granular Base and Subbase materials for Surfaced Pavements" (Edition 6/Revision 1) dated October 2010 prepared by the Roads and Traffic Authority NSW (RTA).

The unbound base and subbase materials should be compacted to at least 98% and 95% of Modified Maximum Dry Density (MMDD), respectively and compaction control testing should be carried out in accordance with the advice provided Section 4.5.2, above.

4.8 Earthquake Design

Based on the advice provided in AS 1170.4-2007 "Structural Design Actions Part 4: Earthquake Actions the site may be assigned a Class C_e (Shallow Soil) classification and a Hazard Factor (Z) of 0.08 adopted.

4.9 Proposed Tunnel

With regard to the proposed tunnel we note the following issues and constraints:

- The proposed excavations will extend across the existing road; buried services will need to be re-located
- The stability of the Block A building will need to be maintained during excavations. Our preference would be for the construction of a grout injected (cfa) secant piled wall retention system prior to excavation commencing. Access will be difficult for piling equipment below Block A and advice will be required from specialist piling contractors (see Section 4.3.2,above).
- The tunnel excavation may 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.
- 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.
- The tunnel invert will likely comprise a combination of weathered sandstone bedrock and natural sands over the southern and northern portions of the tunnel, respectively.

4.10 Further Geotechnical Work

The following summarises the scope of further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Groundwater monitoring and infiltration testing in the standpipes and possible groundwater modelling of basement excavation dewatering.
- Monitoring of groundwater seepage into bulk excavations.
- Inspection of test pits revealing existing buried concrete to the west of Block C.
- Piling rig working platform design.
- Witnessing drilling of cfa piles.
- Continuous quantitative vibration monitoring during rock excavation using rock breakers.
- Inspection of footing excavation bases.
- Proof rolling of exposed sub-grade.
- Density testing of engineered fill.
- Qualitative vibration monitoring during proof rolling and engineered fill compaction.

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

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

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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 JK Geotechnics. 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.

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT TEST REPORT

Client: JK Geotechnics
Project: Proposed Alterations and Additions
Location: Montefiore Jewish Home,
Dangar Street, Randwick, NSW

Ref No: 17167ZR5
Report: A
Report Date: 1/06/2016
Page 1 of 1

AS 1289	**TEST METHOD**	**2.1.1**
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
203	3.80-4.00	7.0
205	4.00-4.25	6.0

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Alterations and Additions
Location: Montefiore Jewish Home,
Dangar Street, Randwick, NSW

Ref No: 17167ZR5
Report: B
Report Date: 1/06/2016
Page 1 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
201	5.46-5.50	0.4	8
	5.88-5.93	0.5	10
	6.34-6.38	0.6	12
	6.89-6.93	0.3	6
	7.25-7.29	0.4	8
	7.83-7.87	0.5	10
	8.22-8.26	0.8	16
	8.87-8.88	0.7	14
	9.13-9.17	0.8	16
202	6.64-6.68	0.3	6
	7.19-7.22	0.4	8
	7.79-7.82	1.1	22
	8.32-8.36	1.2	24
	8.86-8.91	1.5	30
	9.42-9.47	1.1	22
	9.89-9.93	1.0	20
	10.19-10.24	1.3	26
203	4.34-4.38	0.3	6
	4.91-4.96	0.2	4
	5.33-5.37	0.5	10
	5.78-5.82	0.3	6
	6.25-6.29	0.3	6
	6.82-6.85	0.3	6
	7.22-7.26	0.6	12
	7.71-7.75	0.7	14
	8.17-8.21	1.1	22

NOTES: See Page 2 of 2

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	17167ZR5
Project:	Proposed Alterations and Additions	**Report:**	B
Location:	Montefiore Jewish Home, Dangar Street, Randwick, NSW	**Report Date:**	1/06/2016
		Page 2 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
204	7.96-8.00	0.1	2
	8.53-8.57	0.1	2
	9.17-9.21	0.4	8
	10.05-10.08	0.5	10
	10.45-10.49	0.6	12
	11.16-11.19	0.4	8
	11.76-11.80	0.5	10
	12.18-12.21	0.5	10

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\,(50)}$

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
201
1 / 2

BOREHOLE LOG

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR **Method:** SPIRAL AUGER **R.L. Surface:** ~39.8 m
Date: 25/5/16 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** T.P./P.R.

Groundwater Record	ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									-	FILL: Silty sand, fine to medium grained, brown, trace of medium grained ironstone gravel.	M			GRASS COVER
				■	N = 7 3,3,4	39	1		SP	SAND: fine to medium grained, light grey.	M	L		
				■	N = 8 5,4,4	38	2			as above, but medium to coarse grained.				
ON COMPLETION OF AUGERING				■	N = 14 4,6,8	37 36	3 4			as above, but light brown.	W	MD		GROUNDWATER RECORDED IN STANDPIPE AT 3.0m DEPTH ON 7/6/16
				■	N > 9 9,9/ 80mm REFUSAL									
						35	5		-	SANDSTONE: medium to coarse grained.	DW	L		LOW 'TC' BIT RESISTANCE
												L - M		LOW TO MODERATE RESISTANCE
						34 33	6			REFER TO CORED BOREHOLE LOG				

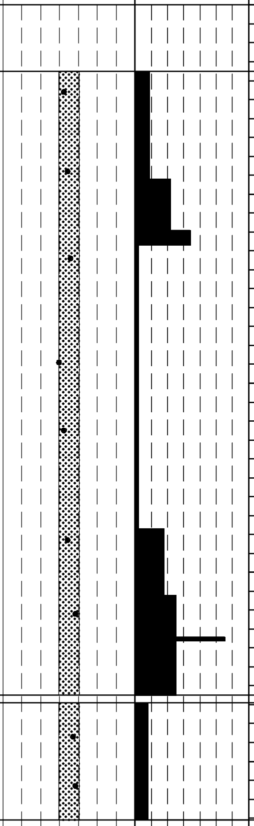

JK Geotechnics
JOB# 17167ZR BH201 START CORING AT 5.35m
5
6
7
8
CL
0.04
9
END OF BH AT 9.31m

CORED BOREHOLE LOG

Borehole No.
201
2 / 2

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Core Size: NMLC
R.L. Surface: ~39.8 m
Date: 25/5/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: T.P./P.R.



Water Loss\Level	Barrel Lift	RL (m AHD)	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
					START CORING AT 5.35m			
100% RETURN		34 33	6 7		SANDSTONE: medium to coarse grained, light grey with orange brown laminae, bedded at 0-15°.	DW	M	(5.92m) CS, 0°, 3 mm.t (6.19m) Be, 15°, P, R (6.27m) J, 25°, Un, R, IS
		32	8		SANDSTONE: medium to coarse grained, grey with dark grey laminae, bedded at 0-15°.	SW		(7.77m) Be, 10°, P, R (8.12m) CS, 5°, 8 mm.t (8.34m) J, 55°, P, S (8.36m) J, 55°, CLAY INFILL 9mm.t (8.51m) J, 75°, Un, R
					CORE LOSS 0.04m			
		31	9		SANDSTONE: medium to coarse grained, light grey with dark grey laminae, bedded at 0-15°.	SW	M	
		30 29 28	10 11		END OF BOREHOLE AT 9.31 m			Class 18 PVC standpipe installed to 8.8m depth. Machine slotted from 8.8m to 4.6m. Unslotted from 4.6m to 0.1m depth. 2mm sand filter pack from 8.8m to 4.5m. Bentonite plug from 4.5m to 0.1m. Gatic cover installed.

Borehole No.
202
1 / 2

BOREHOLE LOG

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Method: SPIRAL AUGER
R.L. Surface: ~41.2 m
Date: 25/5/16
Datum: AHD
Plant Type: JK308
Logged/Checked By: T.P./P.R.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	DS		41		▨		FILL: Sand, fine to medium grained, brown, trace of fine to medium grained sandstone gravel, and ash.	M			GRASS COVER APPEARS POORLY COMPACTED
	DS	N = 5 1,1,4			▨						
			40	1	⁚	SP	SAND: fine to medium grained, light brown.	M	L		
	DS	N = 7 3,4,3	39	2	⁚						
	DS	N = 14 5,6,8			⁚				MD		
			38	3	⁚		SAND: medium to coarse grained, yellow brown.				
ON COMPLETION OF AUGERING	DS	N > 27 6,13,14/ 100mm REFUSAL	37	4	⁚						GROUNDWATER RECORDED IN STANDPIPE AT 3.4m DEPTH ON 23/6/16
				5	⁚	-	SANDSTONE: medium to coarse grained, orange brown.	DW	L		LOW TO MODERATE 'TC' RESISTANCE
			36				REFER TO CORED BOREHOLE LOG				
			35	6							


JK Geotechnics
JOB# 17167ZR BH202 START CORING AT 5.17m
5
CORE LOSS 1.21m
6
7
8
CL 0.04m
9
10
END OF BH AT 10.23m

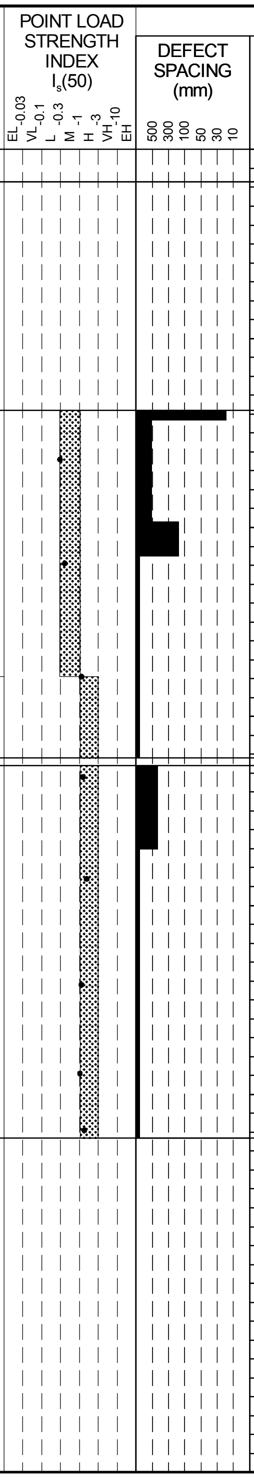
CORED BOREHOLE LOG

Borehole No.
202
2 / 2

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR	**Core Size:** NMLC	**R.L. Surface:** ~41.2 m
Date: 25/5/16	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** T.P./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	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
					START CORING AT 5.17m				
100% RETURN		36 35	6		CORE LOSS 1.21m				
		34	7		SANDSTONE: medium to coarse grained, light grey, with orange brown bands, bedded at 0-10°.	DW	M	(6.40m) CS, 0°, 40 mm.t (6.43m) J, 90°, P, R	
					SANDSTONE: medium to coarse grained, light grey, with grey laminae, bedded at 0-17°.	SW		(6.97m) Be, 17°, P, R (7.15m) Be, 17°, P, R, CLAY INFILL	
					as above, but fine to medium grained.				
		33	8			FR	H		
		32 31	9 10		CORE LOSS 0.04m SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-10°.	FR	H	(8.70m) J, 90°, P, R	
		30	11		END OF BOREHOLE AT 10.23 m			Class 18 PVC standpipe installed to 4.3m depth. Machine slotted from 4.3m to 0.5m. Unslotted from 0.5m to 0.1m depth. 2mm sand filter pack from 4.3m to 0.4m. Bentonite plug from 0.4m to 0.1m. Gatic cover installed.	



Borehole No.
203
1 / 2

BOREHOLE LOG

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR **Method:** SPIRAL AUGER **R.L. Surface:** ~45.0 m
Date: 26/5/16 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** T.P./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING										FILL: Silty sand, fine to medium grained, dark brown, trace of fine to medium grained sandstone and igneous gravel, roots and root fibres.	M			GRASS COVER APPEARS MODERATELY COMPACTED
					N = 10 3,5,5					as above, but medium to coarse grained.				
						44	1							
									SP	SAND: fine to medium grained, light grey.	M	MD		
					N = 12 4,5,7				SM	SILTY SAND: fine to medium grained, dark orange brown.				
						43	2			as above, but orange brown.				
					N = 17 7,8,9	42	3							
						41	4		-	SANDSTONE: medium to coarse grained, light grey.	DW	L - M		MODERATE 'TC' BIT RESISTANCE
										REFER TO CORED BOREHOLE LOG				
						40	5							
						39	6							

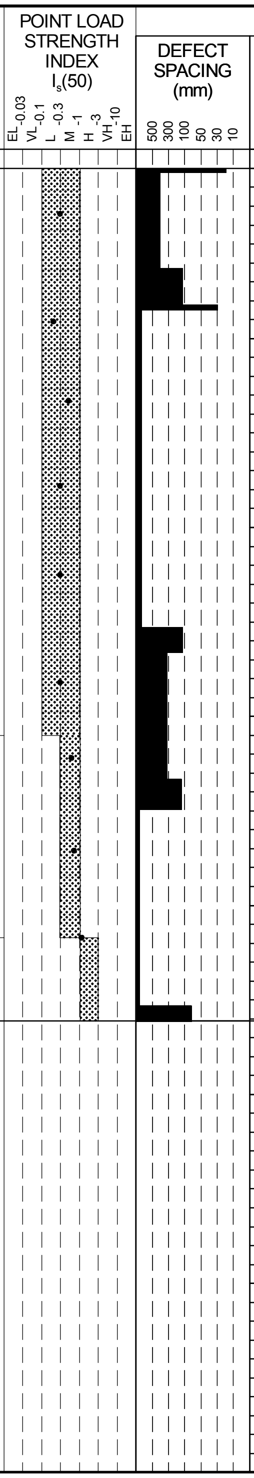

JK Geotechnics
JOB # 17167ZR BH203 START CORING AT 4.10m
4
5
6
7
8
END OF BH AT 8.61m

CORED BOREHOLE LOG

Borehole No.
203
2 / 2

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Core Size: NMLC
R.L. Surface: ~45.0 m
Date: 26/5/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: T.P./P.R.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components. START CORING AT 4.10m	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
100% RETURN		40	5		SANDSTONE: medium to coarse grained,orange brown with red brown bands, bedded at 0-23°.	DW	L - M	(4.12m) J, 38°, Un, S GROUNDWATER RECORDED IN STANDPIPE AT 4.5m DEPTH ON 9/6/16 (4.63m) Be, 23°, P, R (4.82m) J, 45°, Un, R (4.85m) CS, 15°, 3 mm.t (5.69m) XWS, 0°, 16 mm.t
		39 38	6 7		SANDSTONE: medium to coarse grained, light grey with grey laminae, bedded at 0-20°.			(6.53m) CS, 34°, 45 mm.t (6.66m) J, 80°, Un, R (6.97m) Be, 8°, Un, R
		37	8		as above, but bedded at 5-30°.	SW	M H	(7.33m) CS, 30°, 1 mm.t (7.49m) Be, 20°, P, S (8.53m) CS, 6°, 1 mm.t
		36 35	9 10		END OF BOREHOLE AT 8.61 m			Class 18 PVC standpipe installed to 8.6m depth. Machine slotted from 8.6m to 4.6m. Unslotted from 4.6m to 0.1m depth. 2mm sand filter pack from 8.6m to 4.0m. Bentonite plug from 4.0m to 3.5m. Backfilled from 3.5m to 0.1m with gatic cover installed.

Borehole No.
204
1 / 2

BOREHOLE LOG

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR **Method:** SPIRAL AUGER **R.L. Surface:** ~39.6 m
Date: 24/5/16 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.P./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				■						FILL: Silty sand, fine to coarse grained, dark brown, trace of wire, concrete fragments, fine to medium grained sandstone gravel and root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
				■	N = 5 2,3,2	39								
							1							
										as above, but trace of fine to coarse grained sandstone gravel.				
				■	N = 4 2,2,2	38								
							2							
				■		37								
				■			3							
				■	N = 21 6,10,11				SP	SAND: fine to medium grained, yellow brown, trace of fine grained ironstone gravel.	M	MD		
				■		36				SAND: medium to coarse grained, light brown.				
							4							
▼ ON COMPLETION OF AUGERING				■	N = 22 6,9,13	35								
							5				W			
						34								
					N=SPT 1/ 1mm REFUSAL		6		-	SANDSTONE: medium to coarse grained, yellow brown.	DW	M - H		MODERATE TO HIGH 'TC' BIT RESISTANCE
						33				REFER TO CORED BOREHOLE LOG				


JK Geotechnics
JOB# 17167ZR BH204 START CORING AT 6.17m
6
CORE LOSS 1.61m
7
8
9
10
11
12
END OF BH AT 12.31m

CORED BOREHOLE LOG

Borehole No.
204
2 / 2

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Core Size: NMLC
R.L. Surface: ~39.6 m
Date: 24/5/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: T.P./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL -0.03, VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
					START CORING AT 6.17m						
100% RETURN		33 32	7		CORE LOSS 1.61m						
		31	8		SANDSTONE: medium to coarse grained, yellow brown with orange brown bands.	DW	VL			(8.07m) J, 90°, P, R (8.34m) J, 90°, P, R, CLAY INFILL	
		30 29 28	9 10 11 12		SANDSTONE: medium to coarse grained, grey with dark grey laminae, bedded at 5-15°.		M			(9.80m) CS, 0°, 110 mm.t, HP: 220kPa (10.98m) J, 85°, P, R	
		27			END OF BOREHOLE AT 12.31 m						

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
205
1 / 1

BOREHOLE LOG

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR **Method:** SPIRAL AUGER **R.L. Surface:** ~39.5 m
Date: 24/5/16 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.P./P.R.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION					N = 8 5,3,5	39				FILL: Silty sand, fine to medium grained, brown, trace of fine grained sandstone and igneous gravel.	D			GRASS COVER APPEARS MODERATELY COMPACTED
					N=SPT 13/ 150mm REFUSAL	38 37	1 2			as above, but with concrete fragments.	M			
					N = 27 8,12,15	36	3		SP	SAND: medium to coarse grained, orange brown.	M	MD		GROUNDWATER RECORDED IN STANDPIPE AT 3.3m DEPTH ON 8/6/16
							4		-	SANDSTONE: medium to coarse grained, light grey.	SW	M		MODERATE TO HIGH 'TC' BIT RESISTANCE
						35 34 33	5 6			END OF BOREHOLE AT 4.25 m				'TC' BIT REFUSAL Class 18 PVC standpipe installed to 3.9m depth. Machine slotted from 3.8m to 0.9m. Unslotted from 0.9m to surface. 2mm sand filter pack from 3.9m to 0.5m. Bentonite plug from 4.5m to 3.8m. Gatic cover installed.

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

HOLE No.

206

1 / 1

Client: JACKSON TEECE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 36 DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR

Method: HAND AUGER

R.L. Surface: N/A

Date: 27/5/16

Datum: AHD

Plant Type:

Logged/Checked By: T.P./P.R.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS									
					REFER TO DCP TEST RESULTS				REFER TO TEST PIT 206 CROSS SECTIONAL SKETCH				
								-	FILL: Silty sand, fine to medium grained, light brown, with concrete fragments (up to cobble size), trace of roots and root fibres.	D			HAND AUGER REFUSAL ON OBSTRUCTION IN FILL
						1			END OF HOLE AT 0.70 m				
						2							
						3							
						4							
						5							
						6							

JK_LIB_CURRENT - V8.00.GLB Log J & K AUGERHOLE - MASTER 17167ZR RANDWICK.GPJ <<DrawingFile>> 29/06/2016 13:21 Produced by gINT Professional, Developed by Datgel

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client: JACKSON TEECE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 36 DANGAR STREET, RANDWICK. NSW

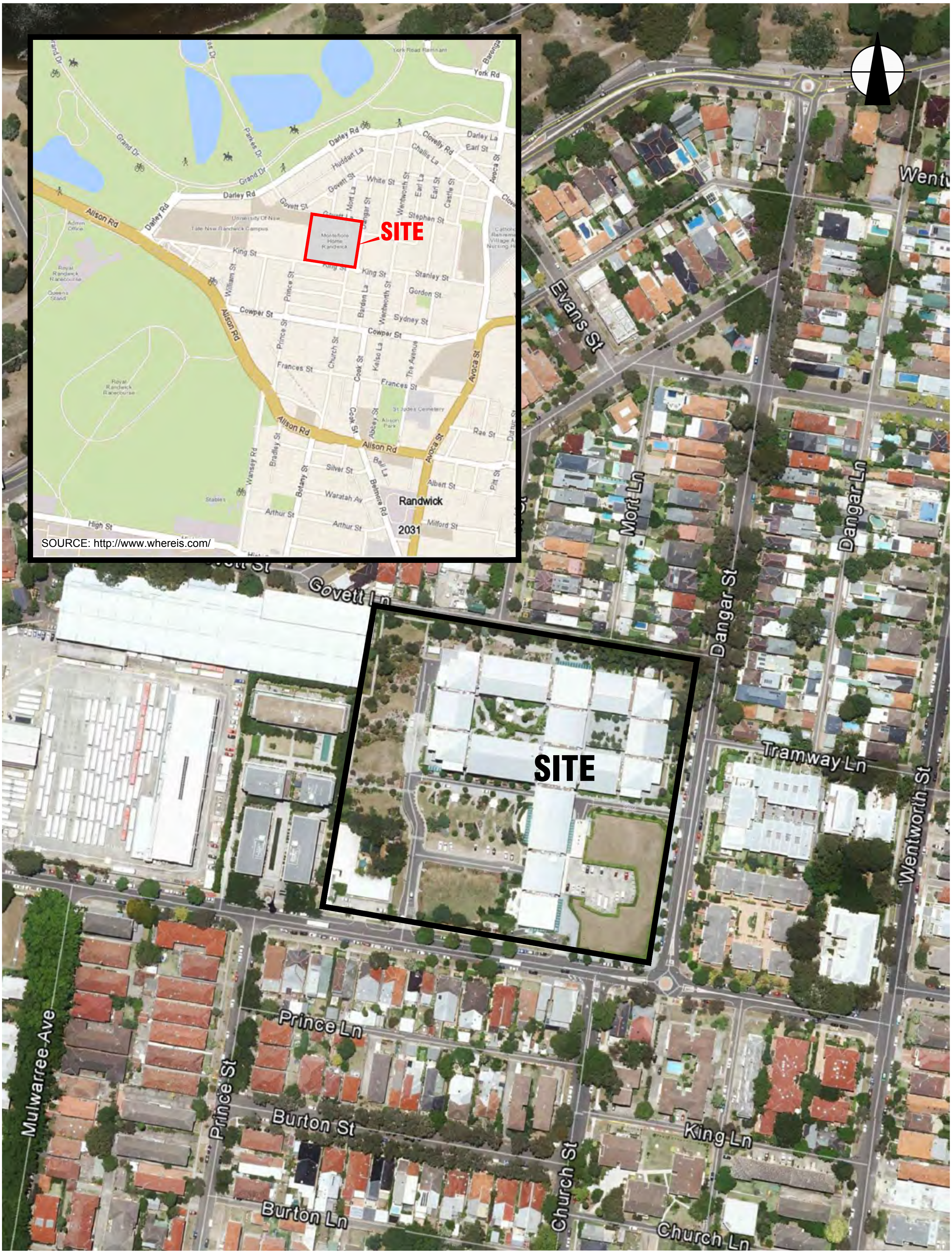
Job No. 17167ZR
Date: 27-5-16
Tested By: T.P.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Number of Blows per 100mm Penetration

Test Location	RL ~41.7m	RL ~37.8m					
Depth (mm)	**206**	**207**					
0 - 100	1	1					
100 - 200	3	9					
200 - 300	13	9					
300 - 400	REFUSAL	6					
400 - 500		8					
500 - 600		7					
600 - 700		8					
700 - 800		9					
800 - 900		11					
900 - 1000		17					
1000 - 1100		14/50mm					
1100 - 1200		REFUSAL					
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD.



AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

This plan should be read in conjunction with the JK Geotechnics report.

Title:
SITE LOCATION PLAN

Location:
36 DANGAR STREET
RANDWICK, NSW

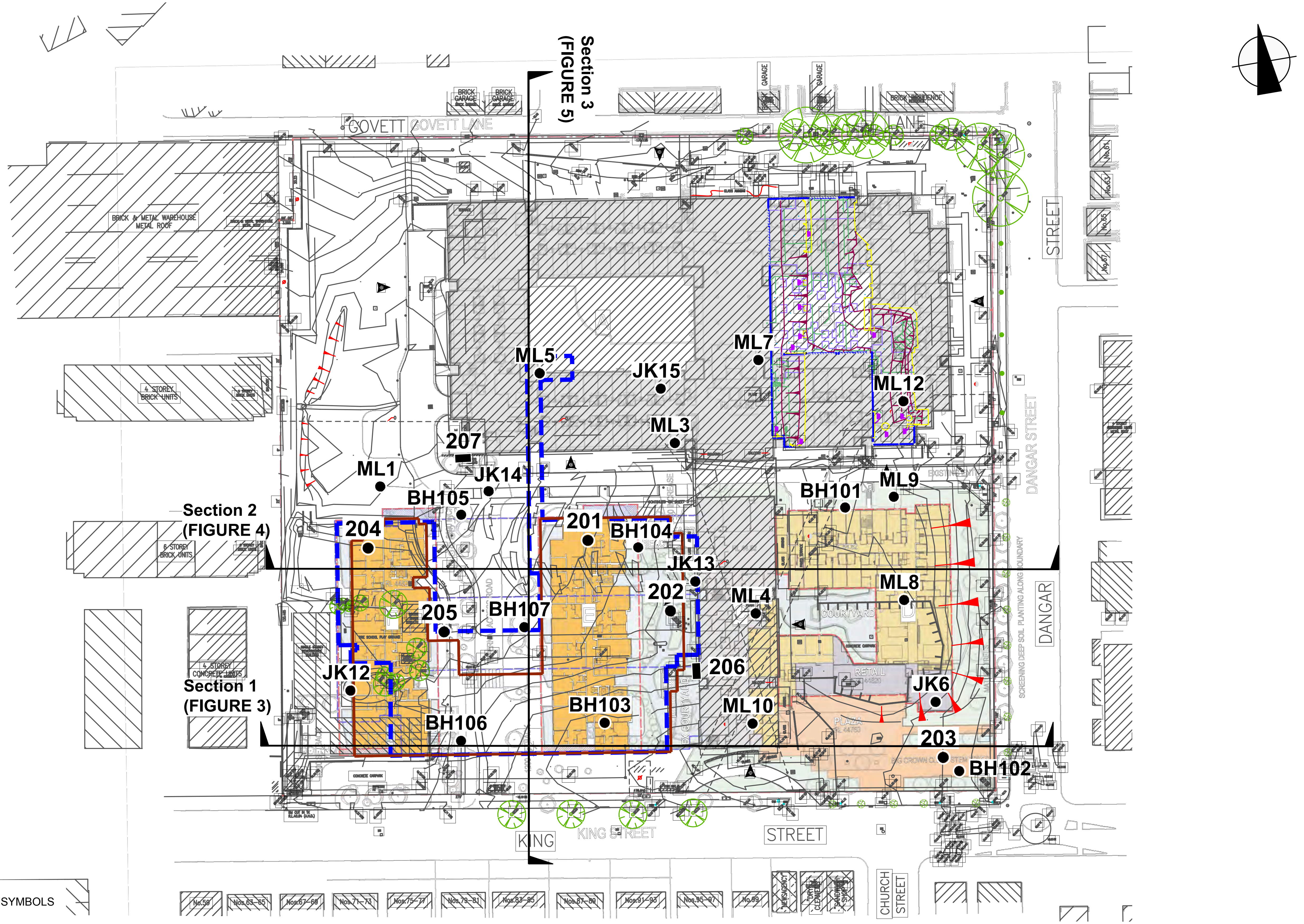
Report No: 17167ZR

Figure No: 1

JK Geotechnics

PLOT DATE: 29/06/2016 11:36:28 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\17000'S\17167ZR RANDWICK\CAD\17167ZR.DWG

PLOT DATE: 29/06/2016 11:38:19 AM   DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\17000'S\17167ZR RANDWICK\CAD\17167ZR.DWG



NOTE:

- FOR EXPLANATION OF MAPPING SYMBOLS SEE FIGURE 2.
- BOREHOLES ML AND JK ARE FROM OUR PREVIOUS GEOTECHNICAL INVESTIGATIONS.
- BOREHOLES BH101 TO BH107 ARE FROM THE PREVIOUS GEOTECHNICAL INVESTIGATION. DATED 14 MARCH 2011
- BOREHOLES BH201 TO BH205, AND TP206 TO TP207 ARE FROM THE CURRENT GEOTECHNICAL INVESTIGATIONS

LEGEND

●	BOREHOLE
▬	TEST PIT
▬ ▬	APPROXIMATE OUTLINE OF BASEMENT LEVEL 2 CARPARK
▬	APPROXIMATE OUTLINE OF BASEMENT LEVEL 1 CARPARK

0 10 20 30 40 50
SCALE 1:1000 @A3 METRES

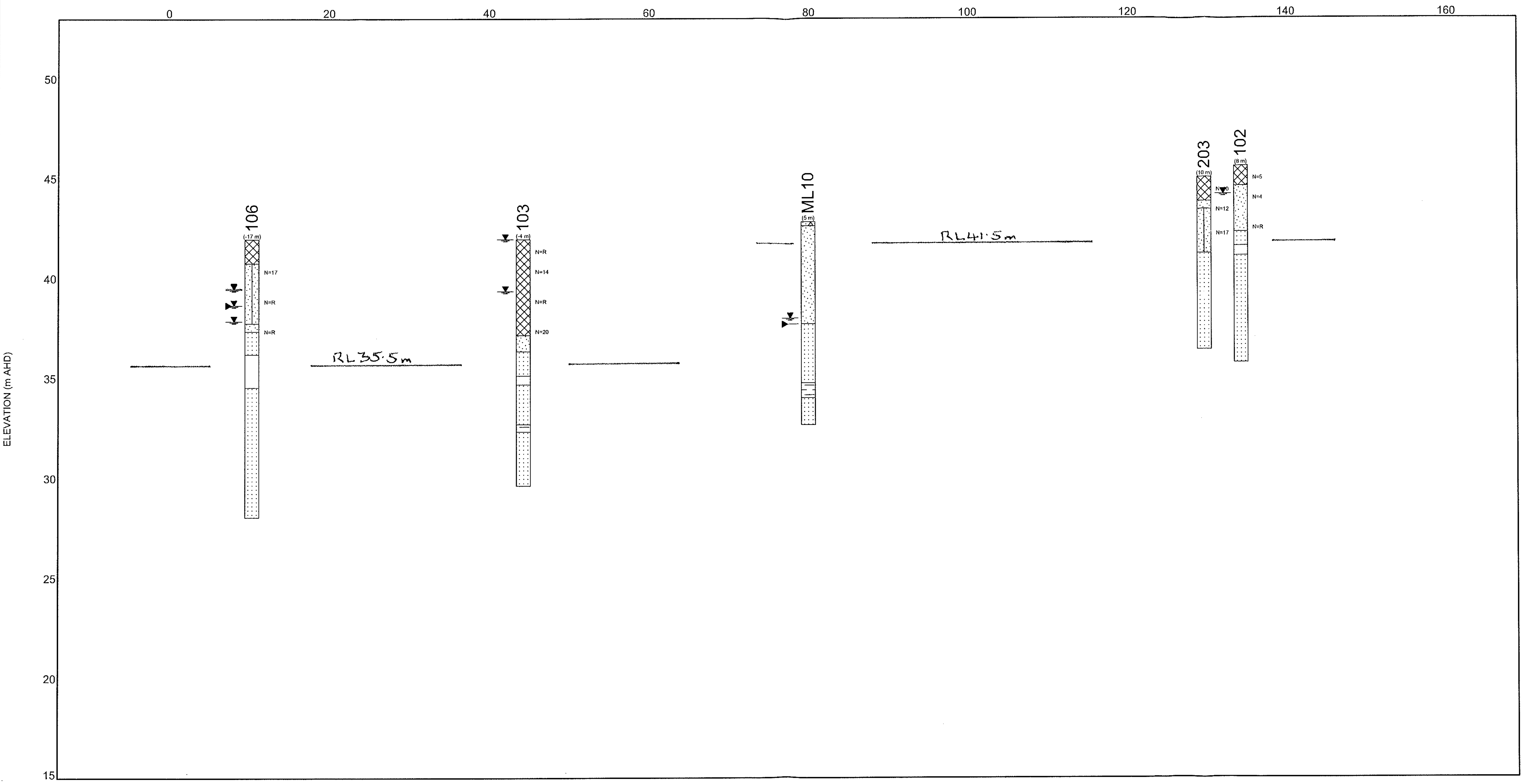
This plan should be read in conjunction with the JK Geotechnics report.

Title:
INVESTIGATION LOCATION PLAN

Location: 36 DANGAR STREET
RANDWICK, NSW

Report No: 17167ZR
Figure No: 2

JK **Geotechnics**



▨ CLAYEY SAND
▥ SILTY SAND
▒ SANDSTONE
☐ CORE LOSS
▨ CONCRETE
☰ SHALE
▒ SAND
▩ FILL

Scales(A3)

H 1:500

V 1:200

Title: SECTION 1

Location: 36 DANGAR STREET, RANDWICK, NSW

Report No: 17167ZR

Figure No: 3

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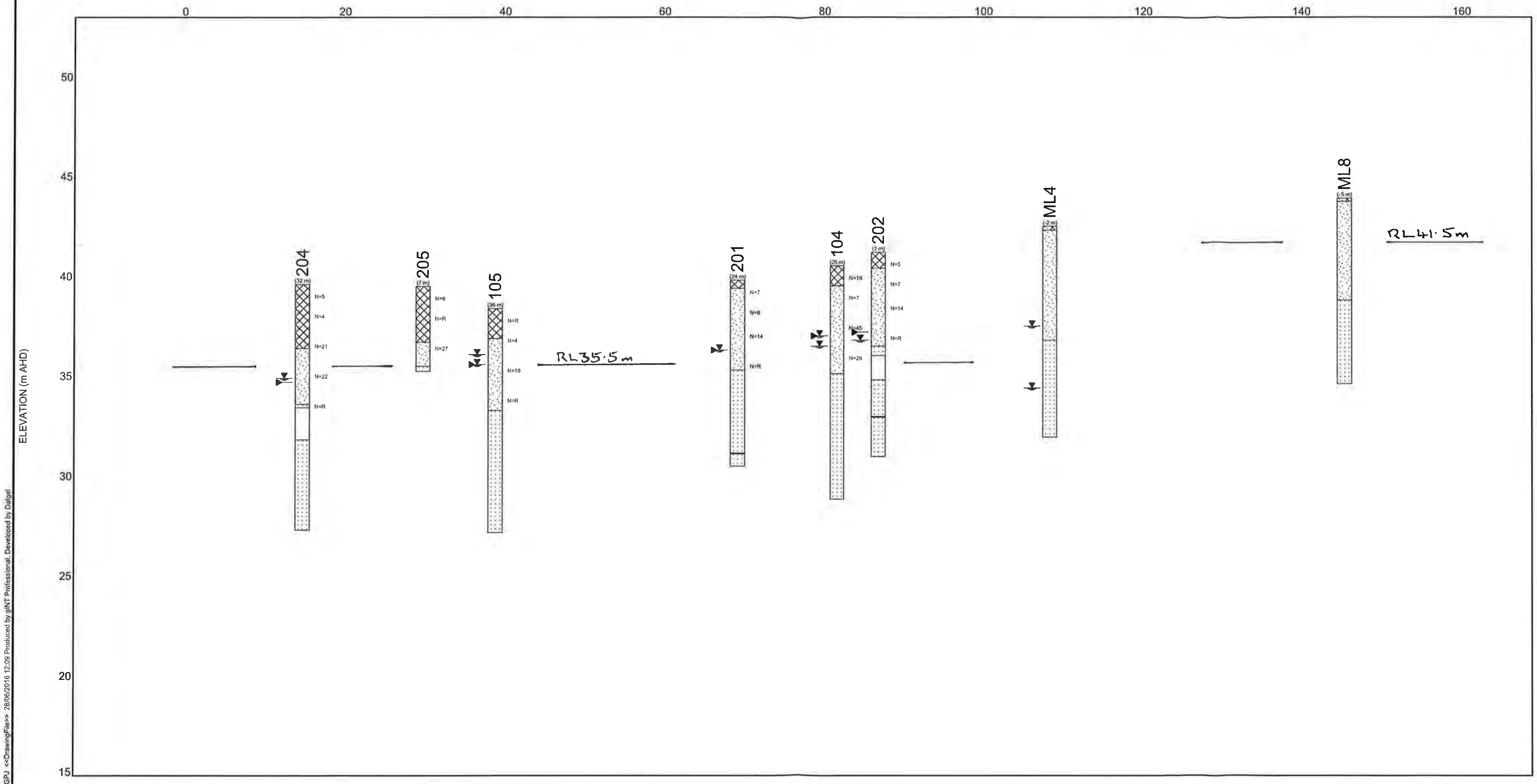
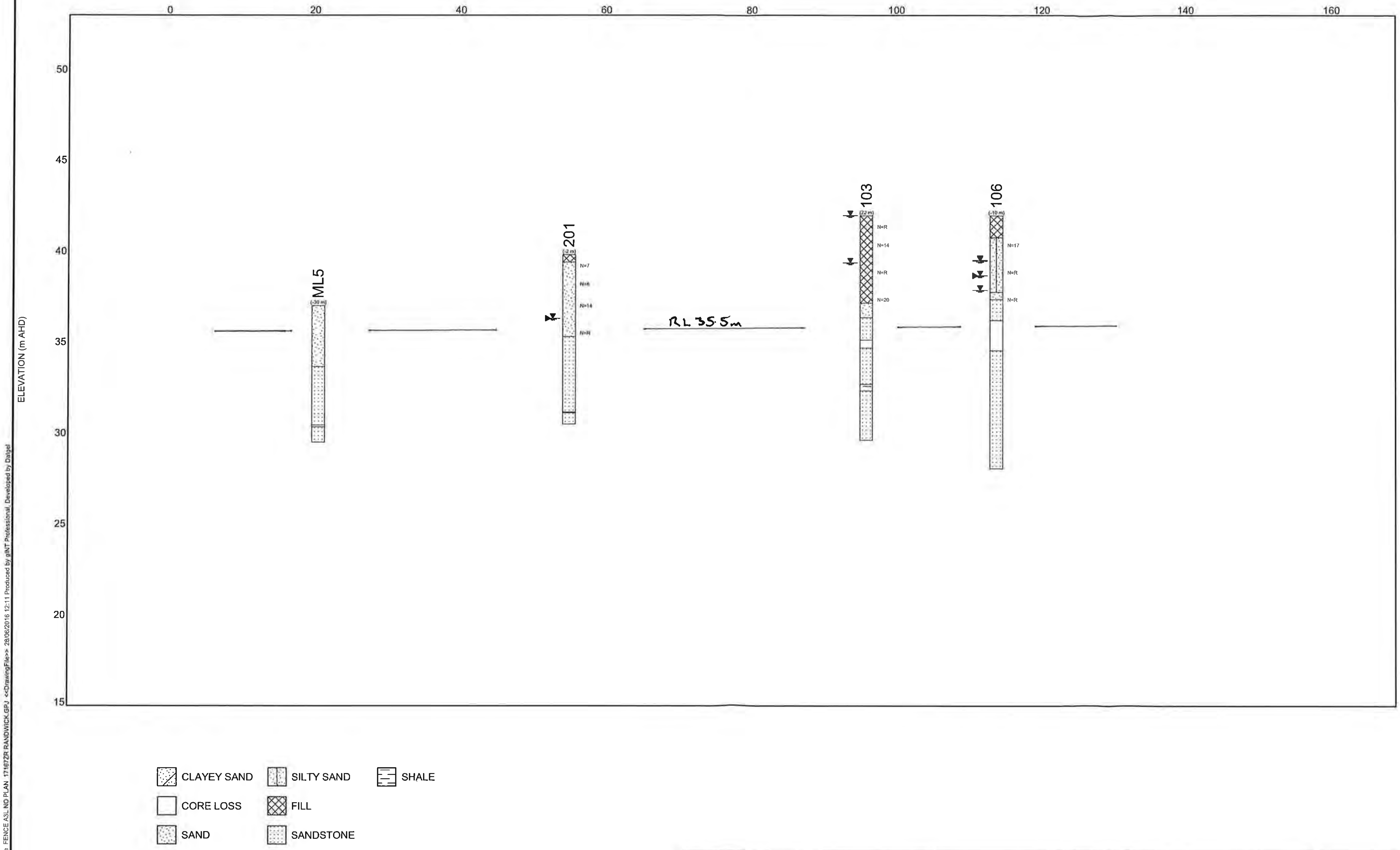
JK_LIB_CURRENT - V8.00.GLB Fence FENCE A3L NO PLAN 17167ZR RANDWICK.GPJ <<DrawingFile>> 28/06/2016 12:04 Produced by gINT Professional, Developed by Datgel



▢ CORE LOSS
▨ FILL
▒ SAND
⁘ SANDSTONE
▒ CONCRETE

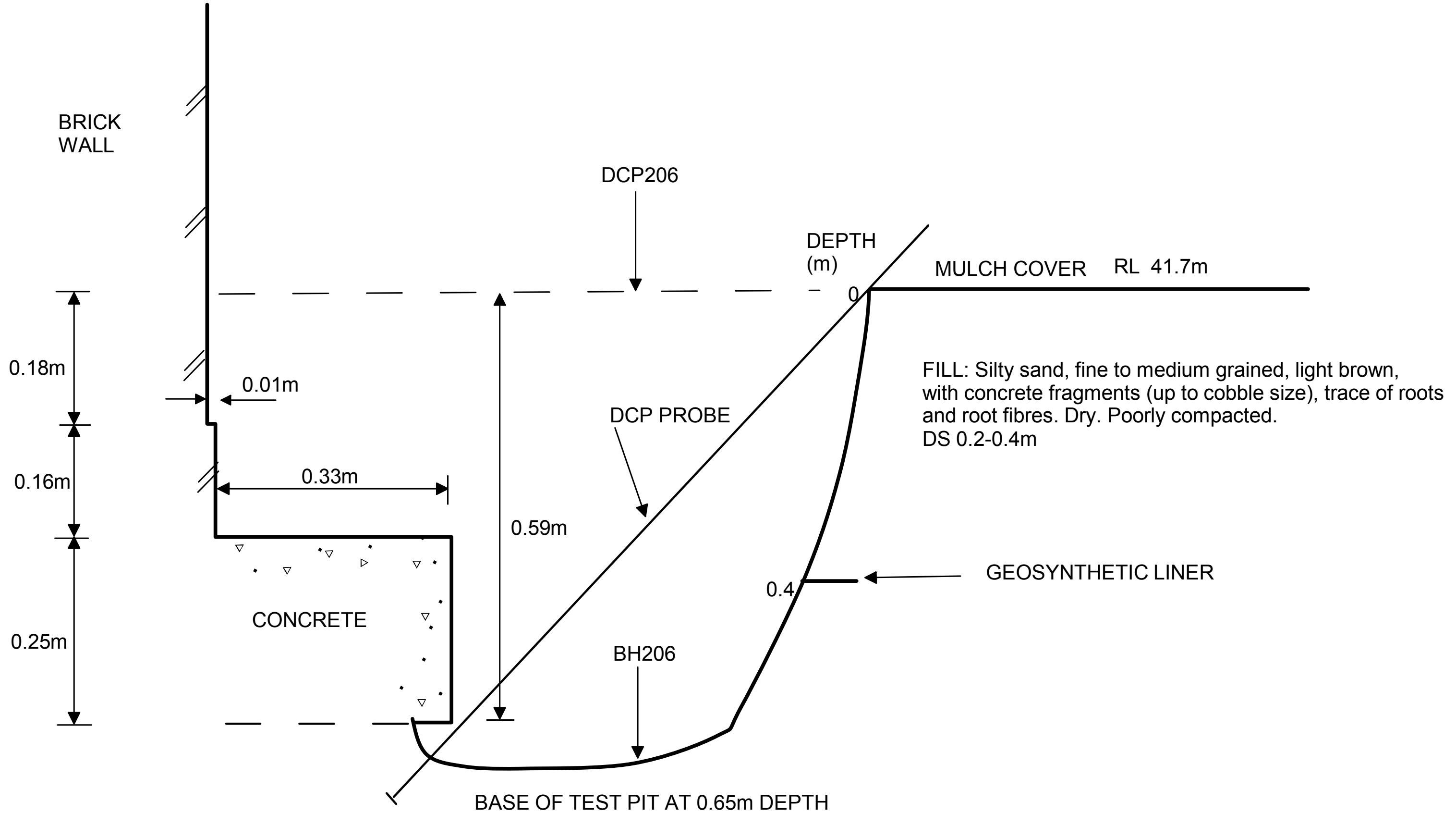
Scales(A3)	**Title:** **SECTION 2**
H 1:500	**Location:** **36 DANGAR STREET, RANDWICK, NSW**
V 1:200	**Report No:** **17167ZR** **Figure No:** **4**
	JK Geotechnics

JK_LIB_CURRENT - V8.00.GLB Fence FENCE A3L NO PLAN 17167ZR RANDWICK.GPJ <<DrawingFile>> 28/06/2016 12:09 Produced by gINT Professional, Developed by Datgel



JK_LIB_CURRENT - V8.00.GLB Fence FENCE A3L NO PLAN 17167ZR RANDWICK.GPJ <<DrawingFile>> 28/06/2016 12:11 Produced by gINT Professional, Developed by Datgel

Scales(A3)	Title: SECTION 3	
H 1:500	Location: 36 DANGAR STREET, RANDWICK, NSW	
V 1:200	Report No: 17167ZR	Figure No: 5
	JK Geotechnics	



FILL: Silty sand, fine to medium grained, light brown,
with concrete fragments (up to cobble size), trace of roots
and root fibres. Dry. Poorly compacted.
DS 0.2-0.4m

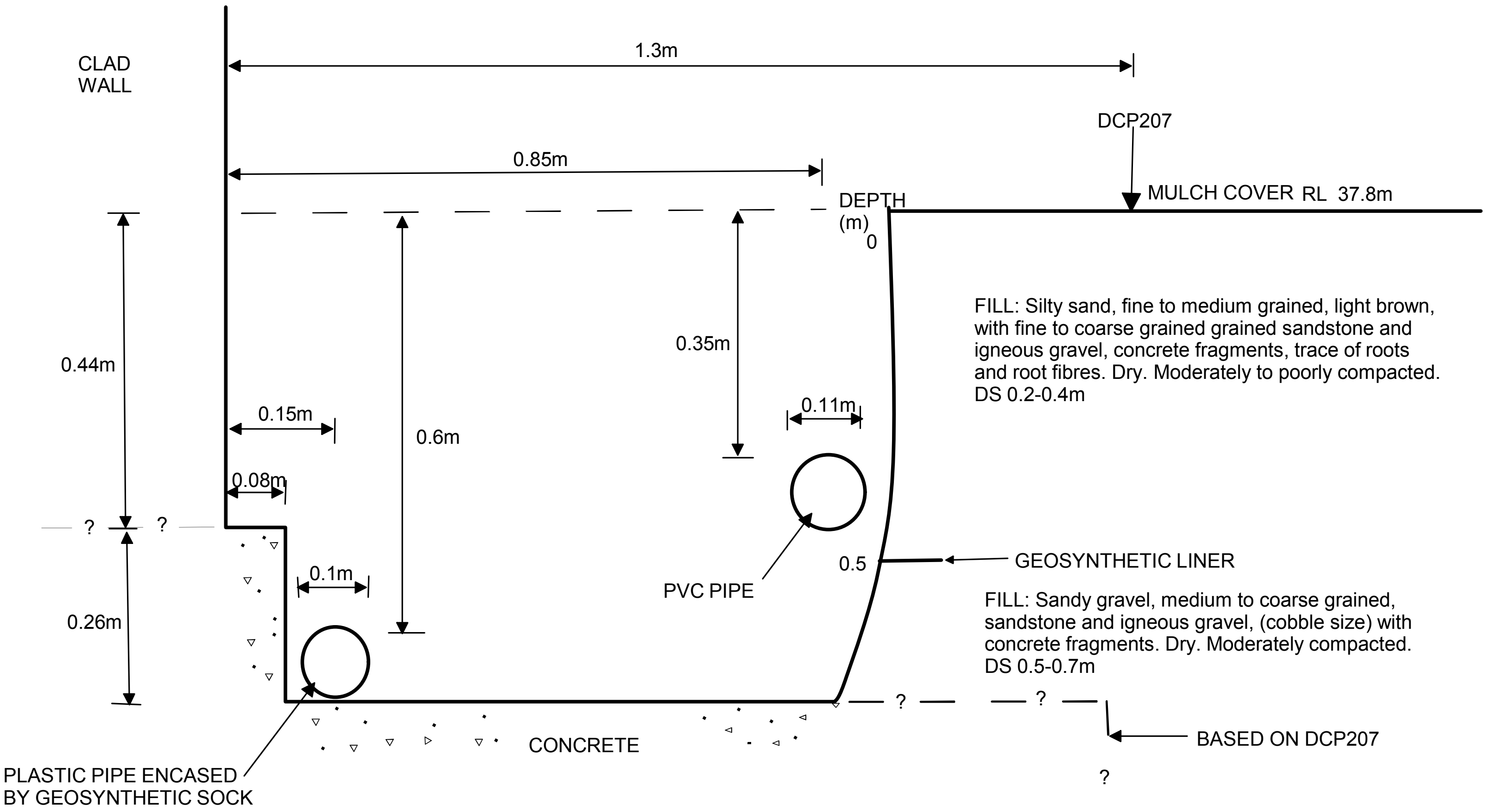
BASE OF TEST PIT AT 0.65m DEPTH
BASE OF CONCRETE AT 0.59m DEPTH
'DRY' ON COMPLETION
REFER TO BOREHOLE LOG AND DCP TEST RESULTS

TEST PIT 206
CROSS SECTIONAL SKETCH
LOOKING SOUTH

0 0.5
Scale 1:10 @A4 Metres

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 17167ZR Figure No. 6



FILL: Silty sand, fine to medium grained, light brown, with fine to coarse grained grained sandstone and igneous gravel, concrete fragments, trace of roots and root fibres. Dry. Moderately to poorly compacted. DS 0.2-0.4m

FILL: Sandy gravel, medium to coarse grained, sandstone and igneous gravel, (cobble size) with concrete fragments. Dry. Moderately compacted. DS 0.5-0.7m

BASE OF TEST PIT AT 0.7m DEPTH
BASE OF CONCRETE UNKNOWN
'DRY' ON COMPLETION
REFER TO DCP TEST RESULTS

TEST PIT 207
CROSS SECTIONAL SKETCH
LOOKING EAST

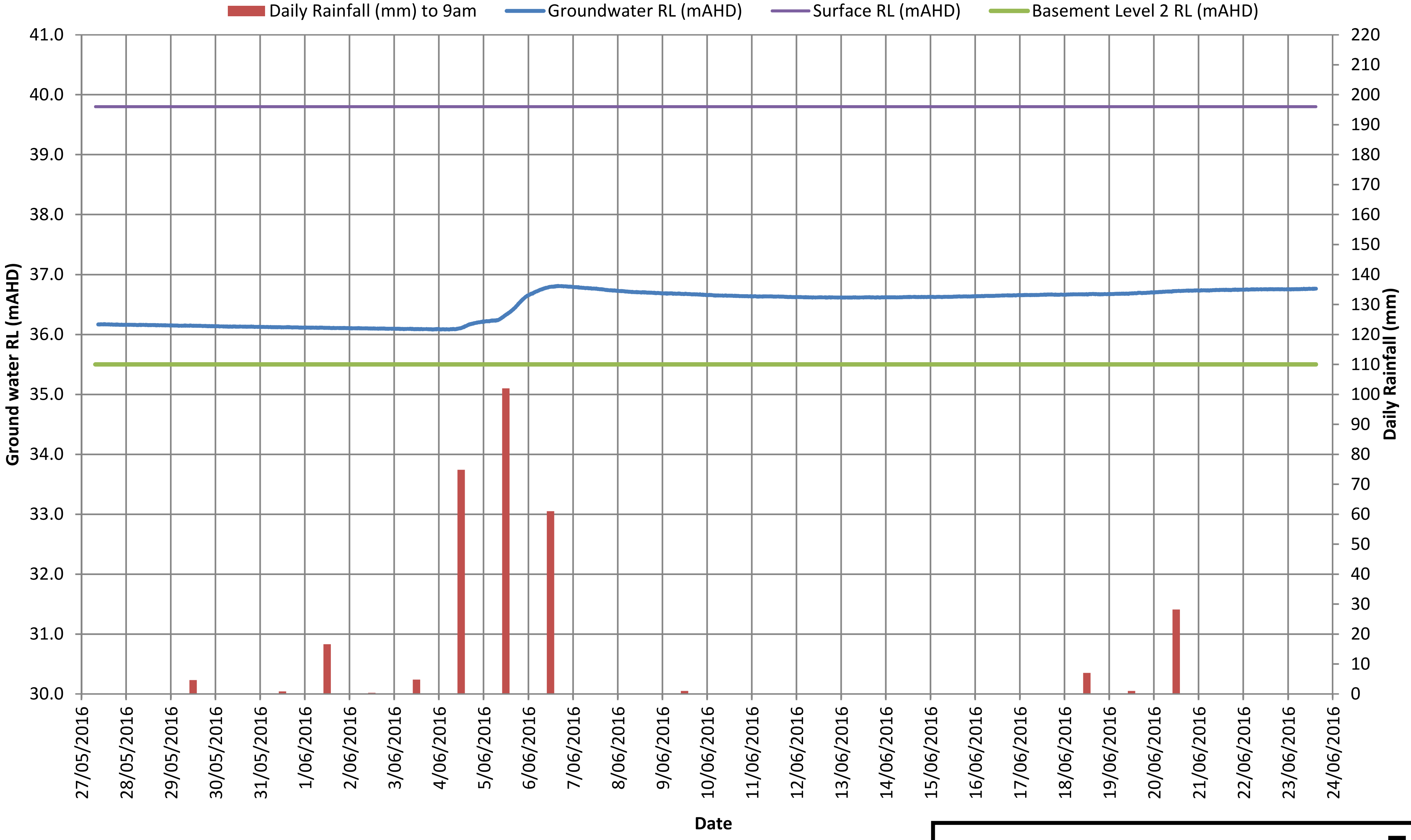
0 0.5
Scale 1:10 @A4 Metres

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 17167ZR
Figure No. 7

File Name: 17167ZR BH201 chart
Date Printed: 28/06/2016

Ground Water Level and Daily Rainfall -v- Time Plot
BH201

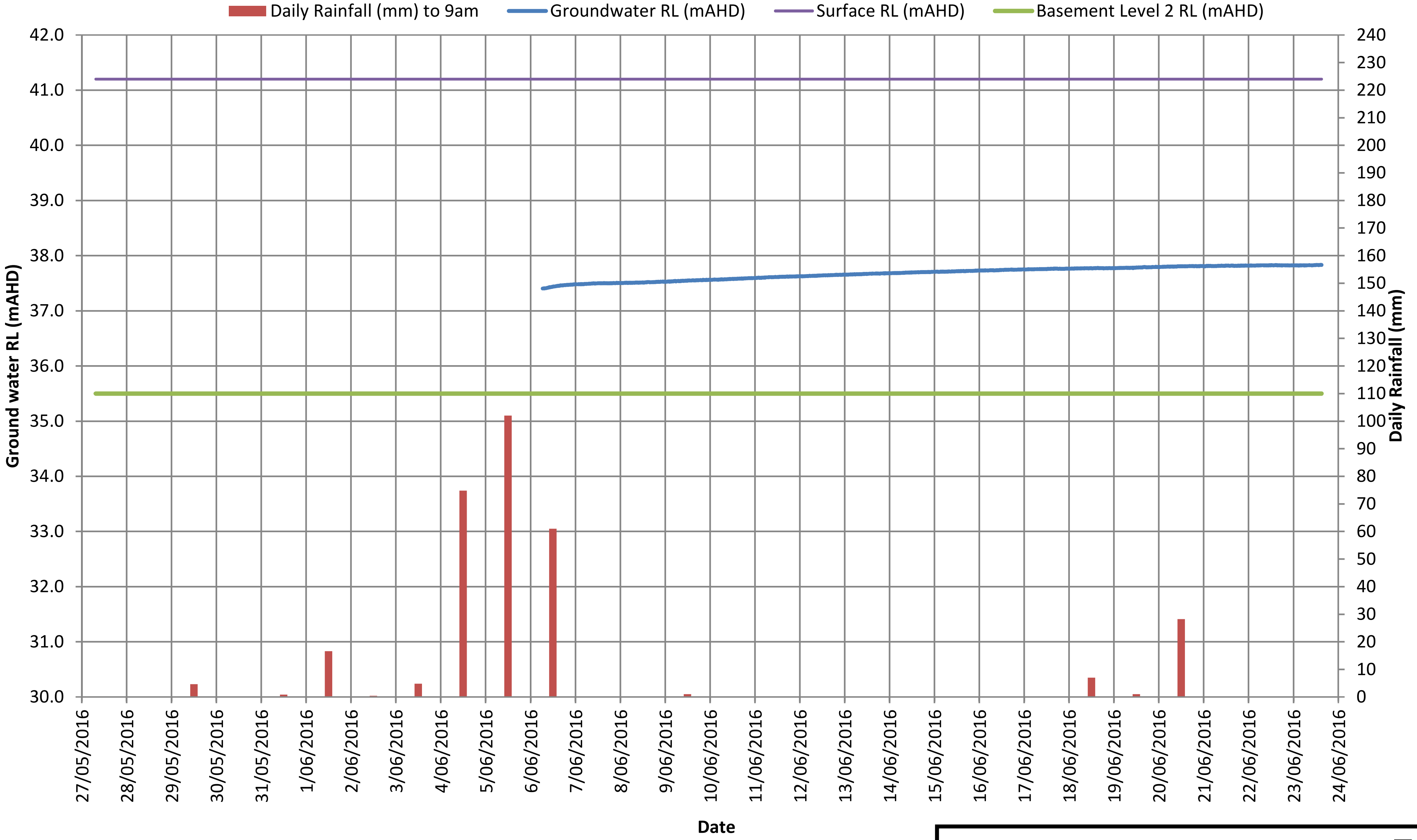


Rainfall data from Randwick Street, Station No. 66052

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
Report No. 17167ZR **Figure No. 8**

File Name: 17167ZR BH202 chart
Date Printed: 28/06/2016

Ground Water Level and Daily Rainfall -v- Time Plot
BH202



Rainfall data from Randwick Street, Station No. 66052

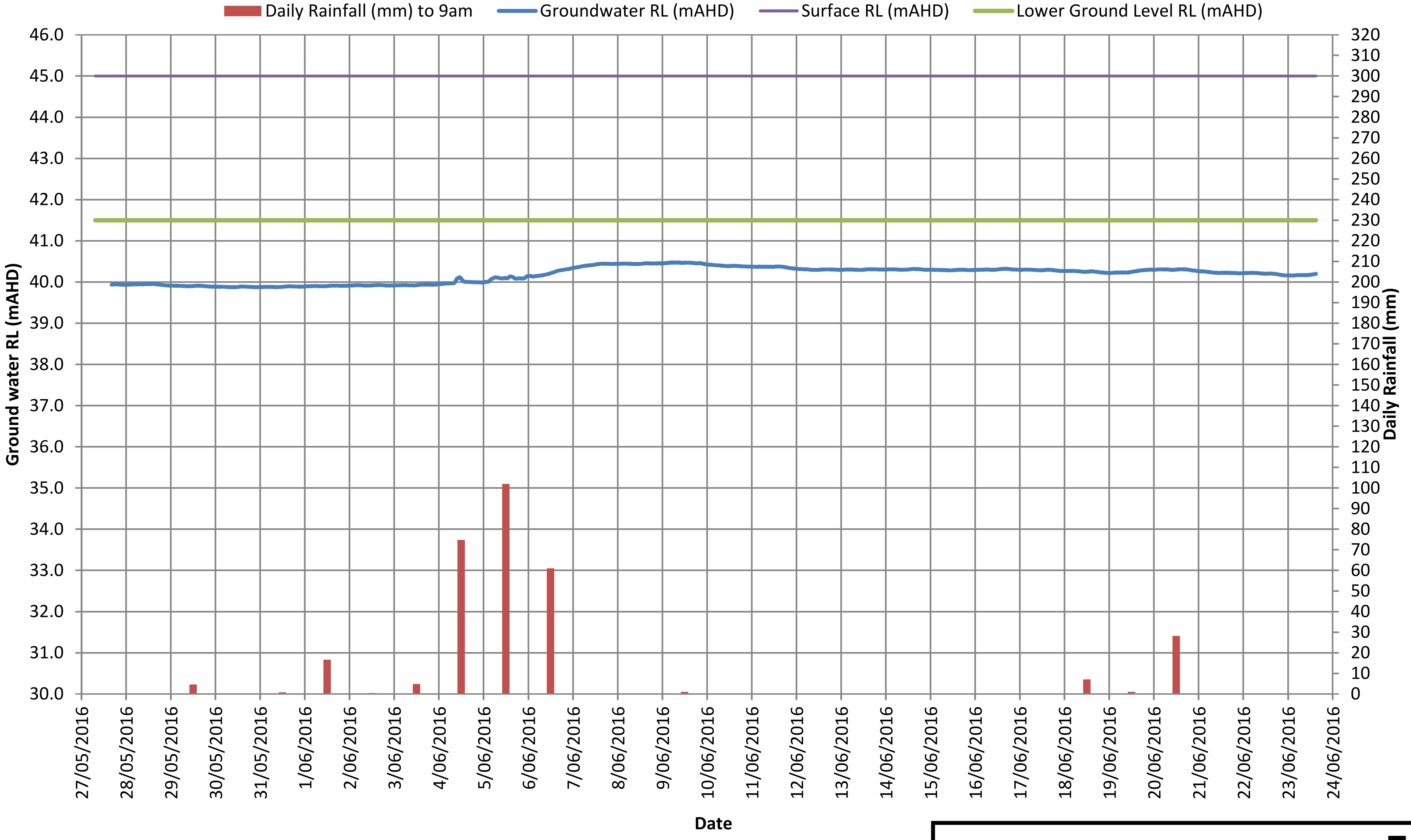
JK Geotechnics

GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 17167ZR **Figure No. 9**

File Name: 17167ZR BH203 chart
Date Printed: 28/06/2016

Ground Water Level and Daily Rainfall -v- Time Plot
BH203



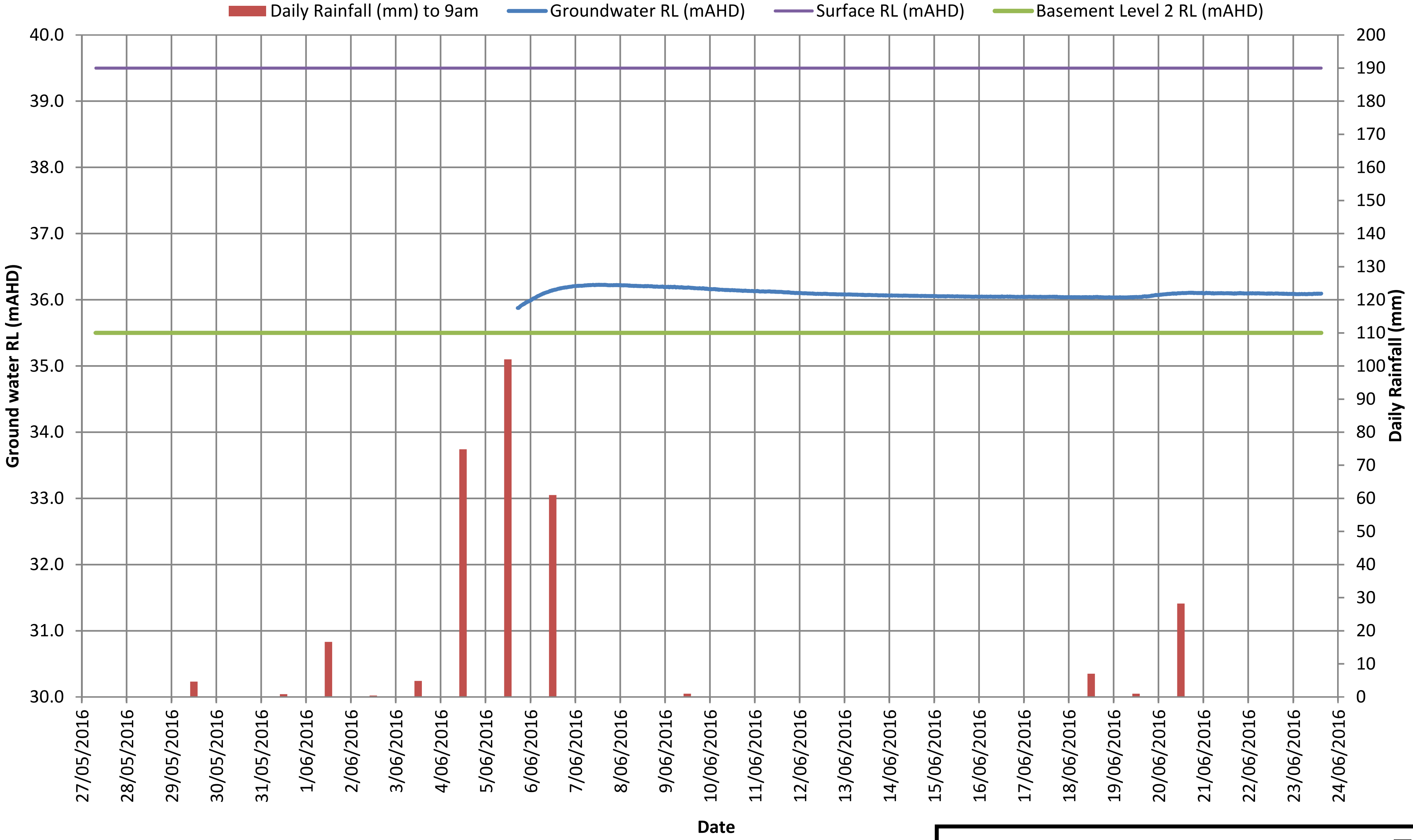
Rainfall data from Randwick Street, Station No. 66052

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 17167ZR **Figure No. 10**

File Name: 17167ZR BH205 chart
Date Printed: 28/06/2016

Ground Water Level and Daily Rainfall -v- Time Plot
BH205



Rainfall data from Randwick Street, Station No. 66052

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
Report No. 17167ZR **Figure No. 11**

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

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

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

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

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

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

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

APPENDIX A

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL ENGINEERS

BOREHOLE LOG

Borehole No. JK6

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

Job No. 7940S
Date: 10 - 1 - 91
Method: SPIRAL AUGER
G.C.H. RIG
R.L. Surface: 44.2m
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.		
	DS	N = 12 5, 6, 6	2							
			3					MD		
	DS	N = 25 8, 10, 15	4							
							M/W			
			5		SC	CLAYEY SAND: fine to medium grained, light grey occasional orange brown ironstaining.				
ON DRILLING ON 25-1-91	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. LOW RESISTANCE.
			7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL ENGINEERS

Borehole No.
JK6
2/2

BOREHOLE LOG

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

Job No. 7940S
Date: 10 - 1 - 91
Method: SPIRAL AUGER
G.C.H. RIG
R.L. Surface: 44·2m
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 10 11 12 13			END OF BOREHOLE AT 8·4m.				

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL ENGINEERS

BOREHOLE LOG

Borehole No. JK11

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 3 4 5 6 7			END OF BOREHOLE AT 1·6m.				

Jeffery and Katauskas Pty Ltd
CONS ULTING GEOTECHNICAL ENGINEERS

Borehole No.
JK12

BOREHOLE LOG

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

Job No. 7940S Method: SPIRAL AUGER Estimated RL : 39·0m
Date: 11 - 1 - 91 G.C.H. RIG Datum : AHD

undwater 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	⁘						
ON DRILLING				⁘						SLIGHT OIL ODOUR.
			3			END OF BOREHOLE AT 3·0m.	W			
			4							
			5							
			6							
			7							

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Borehole No.
JK13

BOREHOLE LOG

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

Job No. 79405
Date: 11 - 1 - 91
Method: SPIRAL AUGER
G.C.H. RIG
Estimated RL : 42·8m
Datum : AHD

oundwater record	Samples	Field Tests	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
				░		CONCRETE SLAB: 190mm				
				▨		FILL: Sand, fine to medium grained, light brown to white.	M			
	DS	N=8 2,3,5	1	▨						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 No. JK14

BOREHOLE LOG

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							

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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							

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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							
					N = 39 6,18,21						D		
▼						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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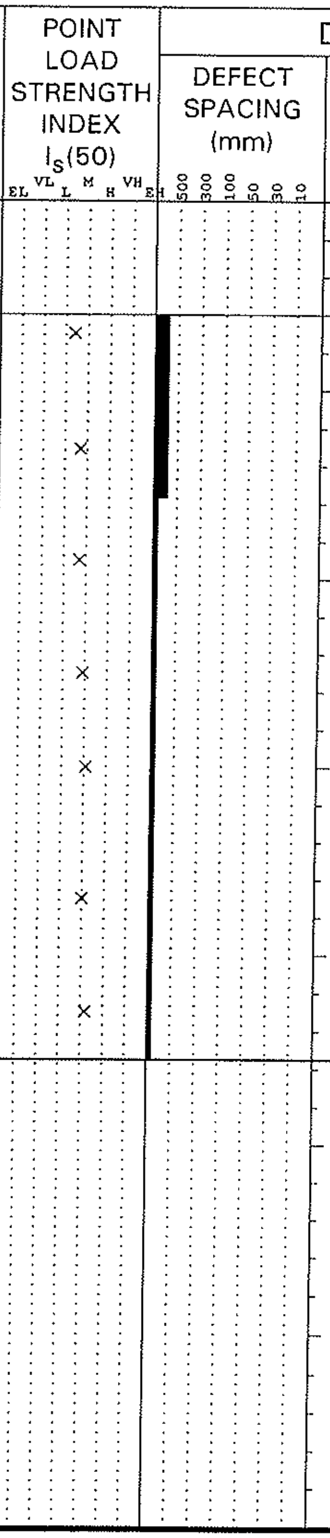
CORED BOREHOLE LOG

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	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						

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
[illegible] ON C[illegible]PLETION 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							
									as above, but light grey.				
				■	N = 9 3,4,5								
						2							
									as above, but light brown.				
						3					MD		
				■	N = 11 5,5,6								
									as above, but light yellow brown.				
						4							
				■	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							

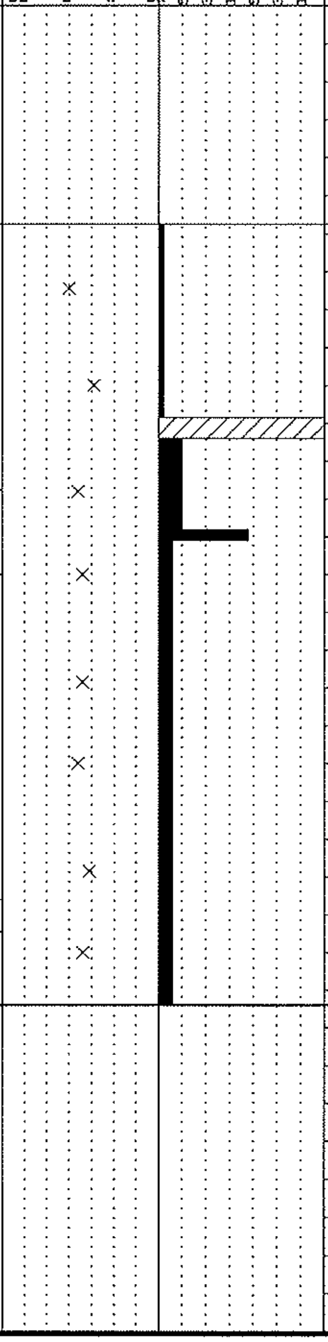
CORED BOREHOLE LOG

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
Core Size: NMLC
R.L. Surface: ≅ 43.9m
Date: 26-9-02
Inclination: VERTICAL
Datum: AHD
Drill Type: EDSON 3000
Bearing: -
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 $I_s(50)$	DEFECT SPACING (mm)	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	
L 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 LOG

Borehole No. ML9 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: ≅ 43.8m
Datum: AHD
Logged/Checked by: A.J.H./

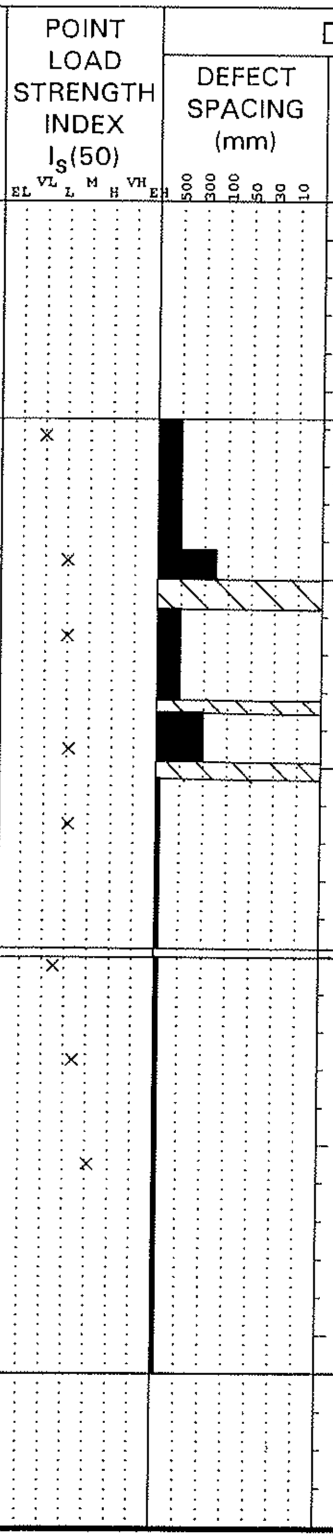
Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
Y ON COMPLETION OF AUGERING						0			FILL: Sand, fine to coarse grained, dark grey, with a trace of sandstone gravel and bricks.	M			
								SP	SAND: fine to coarse grained, light grey.	M	L	-	
				■	N = 7 4,4,3								
						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							
								-	SANDSTONE: fine to coarse grained, orange brown, with clay seams.	XW	EL-VL	-	VERY LOW 'TC' BIT RESISTANCE
				■	N > 8 16,8/ 50mm REFUSAL								
									SANDSTONE: fine to medium grained, orange brown.	DW	VL-L		LOW RESISTANCE
						5			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		
		6				M	- XWS, 0°, 40mm.t - XWS, 0°, 150mm.t	
				as above, but cross bedded at 0-15°.			- XWS, 0°, 60mm.t - XWS, 0°, 100mm.t	
		7		SANDSTONE: fine to medium grained, light grey, massive.	SW			
FULL RET-URN								
		8		CORE LOSS 0.05m			- CS, 0°, 10mm.t	
				SANDSTONE: fine to medium grained, light grey, massive.	SW	L-M		
		9				M-H	- 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	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						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 ▼						4							
				■	N = 44 11,18,26						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
									REFER TO CORED BOREHOLE LOG				
						6							
						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				
		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.	
		7					- CS, 3mm.t., 0°	
FULL RET- RN				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						
		11		END OF BOREHOLE AT 10.14m				

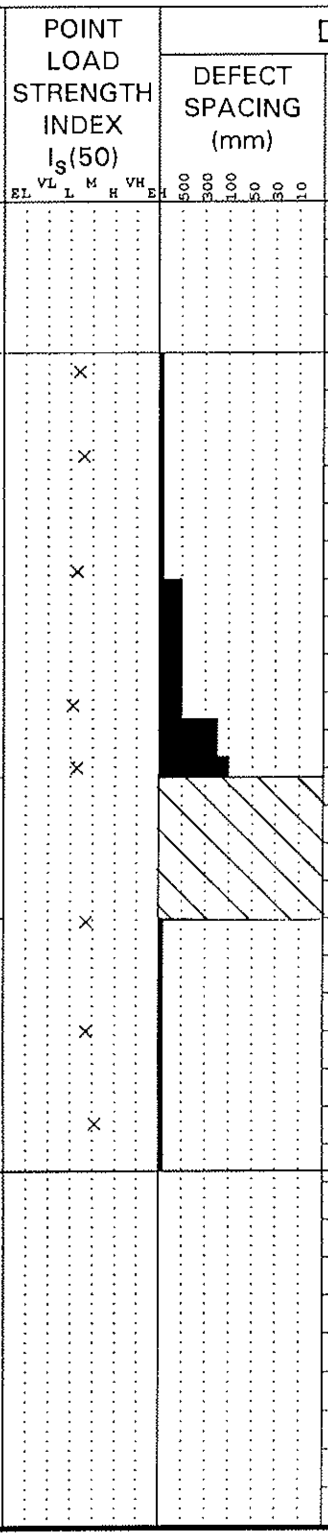

TABLE A
SUMMARY OF LABORATORY TEST RESULTS

AS 1289	TEST METHOD	4.3.1
SAMPLE	SAMPLE DEPTH m	pH
ML 6	0.50 - 0.95	7.4
ML 11	1.50 - 1.95	7.2

4/10/2002

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Ref No: 17167S
Table B: Page 1 of 1

TABLE B
SUMMARY OF FOUR DAY SOAKED C.B.R.TEST RESULTS

SAMPLE LOCATION	ML 1	ML 2	ML 3
DEPTH (m)	0.50 - 1.50	0.00 - 0.00	0.00 - 0.00
Surcharge (kg)	4.5	4.5	4.5
Maximum Dry Density (t/m^3)	1.71 STD	1.73 STD	1.73 STD
Optimum Moisture Content (%)	8.3	12.6	11.3
Moulded Dry Density (t/m^3)	1.68	1.71	1.70
Sample Density Ratio (%)	98	99	98
Sample Moisture Ratio (%)	99	95	96
Moisture Contents			
Insitu (%)	4.2	10.9	12.4
Moulded (%)	8.2	11.9	10.8
After soaking and After Test, Top 30mm(%)	15.6	15.5	14.9
Remaining Depth (%)	15.4	15.3	14.8
Material Retained on 19mm Sieve (%)	0	0	0
Swell (%)	0.0	0.0	0.5
C.B.R. value: @2.5mm penetration	17	25	20

NOTES:

(1) Refer to appropriate notes for soil descriptions
(2) Test Methods :
    (a) Soaked C.B.R. : AS 1289 6.1.1
    (b) Standard Compaction : AS 1289 5.1.1
    (c) Moisture Content : AS 1289 2.1.1

4/10/2002
Authorised Signature
LAB No. 1327

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TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
ML 4	6.70-6.74	0.6	12
	7.31-7.35	0.8	16
	7.90-7.94	0.8	16
	8.50-8.53	0.9	18
	9.00-9.04	1.1	22
	9.70-9.74	1.0	20
	10.30-10.34	1.3	26
ML 5	3.66-3.70	0.1	2
	4.16-4.19	0.3	6
	4.67-4.70	0.6	12
	5.21-5.26	0.2	4
	5.70-5.73	0.7	14
	6.27-6.31	0.1	2
	6.96-7.00	0.3	6
	7.40-7.43	0.2	4
ML 6	3.94-3.98	0.2	4
	4.42-4.46	0.5	10
	4.46-5.00	0.5	10
	5.43-5.46	0.6	12
	5.93-5.97	0.2	4
	7.30-7.34	0.1	2
ML 7	6.63-6.66	0.8	16
	7.30-7.33	0.5	10
	7.90-7.94	0.6	12
	8.50-8.54	1.2	24
	9.00-9.05	0.8	16

NOTES: SEE PAGE 3

TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
ML 7	9.70-9.74	0.9	18
	10.40-10.43	0.5	10
ML 8	5.49-5.52	0.2	4
	6.0-6.04	1.2	24
	6.56-6.60	0.6	12
	7.00-7.03	0.7	14
	7.57-7.60	0.7	14
	8.00-8.03	0.6	12
	8.57-8.61	0.9	18
	9.0-9.03	0.7	14
ML 9	5.24-5.28	0.1	2
	5.90-5.94	0.3	6
	6.30-6.34	0.3	6
	6.90-6.94	0.4	8
	7.30-7.34	0.4	8
	8.05-8.09	0.2	4
	8.55-8.59	0.6	12
	9.10-9.15	1.1	22
ML 10	5.90-5.94	0.6	12
	6.35-6.39	0.7	14
	6.96-7.00	0.5	10
	7.67-7.71	0.4	8
	8.00-8.03	0.5	10
	8.82-8.86	0.8	16
	9.40-9.43	0.8	16
	9.89-9.92	1.1	22

NOTES: SEE PAGE 3

TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
ML 11	5.42-5.45	0.1	2
	5.75-5.78	0.3	6
	6.95-7.00	0.7	14
	7.42-7.47	0.5	10
	7.78-7.81	0.3	6
	8.47-8.50	0.6	12
	8.75-8.78	0.3	6
	9.79-9.83	0.2	4
	10.30-10.34	0.8	16
	10.80-10.83	1.2	24
	11.30-11.34	1.5	30
ML 12	6.10-6.14	1.3	26
	6.60-6.63	1.4	28
	7.20-7.24	1.1	22
	7.79-7.83	1.5	30
	8.40-8.43	1.0	20
	8.93-8.97	1.4	28
	9.41-9.45	0.8	16
ML 13	7.70-7.75	0.3	6
	8.25-8.28	0.3	6
	9.30-9.34	0.2	4
	9.80-9.84	0.3	6
	10.25-10.29	0.2	4
	10.80-10.84	0.4	8
	11.25-11.30	0.3	6
	11.55-11.59	0.2	4

NOTES:

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

Report No: NAA02-1648

08 OCT 2002

LabPoint

LabPoint Pty Ltd
ABN 82 096 903 749
Phone: (02) 9648 6498
Fax: (02) 9648 6512
E-Mail: labpoint@bigpond.net.au
11-13 Byrne Street,
Auburn NSW 2144
P.O. Box 177
Kings Langley NSW 2147

Date Received: 03/10/2002

Order No: C.O.C 02/10/02

Attention: Mr. Ashwin Tatikonda

Jeffery and Katauskas Pty Ltd
39 Buffalo Road
GLADESVELLE NSW 2111

Type of Samples: One soil - project 17167 S

Tests	**ML 8** **3.0-3.45 m**	**Methods**
Sulphate	<50	AS 1289 D2.1 & APHA 4500 SO^{2-}_4 - E

Note: Units: mg/kg dryweight for soils. Analysed "as received".

Samples will be disposed of seven days after issue of this report unless otherwise notified.

The above soil samples have been prepared by customer as follows:

(a) Oven dried at 50 C

(b) Sieved over 2.36 mm sieve

Dr Rama Bhat
Manager Environmental Services
Date Issued: 04/10/2002

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LABORATORY NO 11111

TABLE D: SUMMARY OF LABORATORY SULPHATE CONTENT TEST RESULT

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
101
1 / 2

BOREHOLE LOG

Client: MCLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Method: SPIRAL AUGER
R.L. Surface: ~42.01 m
Date: 08/02/11
Datum: AHD
Plant Type: JK300
Logged/Checked By: D.S./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING								▨	-	FILL: Sand, fine to medium grained, light grey and brown, with fine to medium grained igneous gravel and concrete fragments.	D			GRASS COVER APPEARS MODERATELY COMPACTED
				■	N = 12 11,7,5									
						41	1							
									SP	SAND: fine to medium grained, light grey.	D	MD		
ON COMPLETION OF CORING				■	N = 13 13,7,6									
									SM	SILTY SAND: fiine to medium grained, dark brown.				
						40	2							
									SP	SAND: fine to medium grained, yellow brown.				
						39	3							
				■	N = 16 6,7,9									
						38	4							
										SANDSTONE: fine to medium grained, red brown.	DW	L - M		LOW TO MODERATE 'TC' BIT RESISTANCE
										REFER TO CORED BOREHOLE LOG				
						37	5							
						36	6							

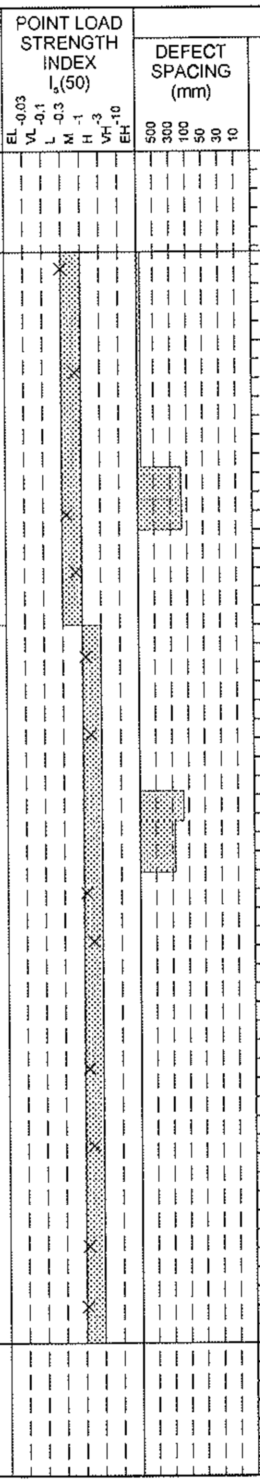
Borehole No.
101
2 / 2

CORED BOREHOLE LOG

Client: McLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Date: 08/02/11
Plant Type: JK300
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: ~42.01 m
Datum: AHD
Logged/Checked By: D.S./

Water Loss\Level	Barrel Lift	RL (m AHD)	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
					START CORING AT 4.53m			CLASS 18 SLOTTED PVC INSTALLED TO 5.6m, BACKFILLED WITH 2mm SAND, BENTONITE SEAL BETWEEN 0.2m AND 0.5m, GATIC COVER CONCRETED AT SURFACE	
▼ 18-2-11					SANDSTONE: fine to medium grained, red brown.	SW	M		
		37	5						
								XWS, 0°, 50 mm.t	
								XWS, 0°, 10 mm.t	
								XWS, 0°, 3 mm.t	
		36	6						
							H		
					SANDSTONE: fine to medium grained, light grey, with dark grey laminae bedded at 0-10°.				
100% RETURN		35	7						
								J, 80°, Un, R, IS	
								J, 80', Un, R, IS	
								J, 60°, P	
		34	8						
		33	9						
								Be, 0°, Un, S	
		32	10						
					END OF BOREHOLE AT 10.30 m				



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JEFFERY & KATAUSKAS PTY LTD
JOB No: 17167ZR
BH 101
START CORING AT 4.53m
4
5
6
7
8
9
10
END BH

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
102
1 / 2

BOREHOLE LOG

Client: MCLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Method: SPIRAL AUGER
R.L. Surface: ~45.58 m
Date: 08/02/11
Datum: AHD
Plant Type: JK300
Logged/Checked By: D.S./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING				■	N = 5 3,3,2	45			-	FILL: Silty sand, fine to medium grained, brown, with concrete fragments.	D			GRASS COVER APPEARS POORLY COMPACTED
▼ ON COMPLETION OF CORING				■	N = 4 2,2,2	44	1		SP	SAND: fine to medium grained, light grey.	D	L		
							2			as above, but brown.	M			
						43				as above, but yellow brown.				
				■	N > 11 3,11/ 150mm REFUSAL		3			as above, but red brown.				
						42			-	SANDSTONE: fine to coarse grained, light grey.	DW	L		VERY LOW 'TC' BIT RESISTANCE
											SW	L - M		LOW TO MODERATE RESISTANCE
						41 40 39	4 5 6			REFER TO CORED BOREHOLE LOG				

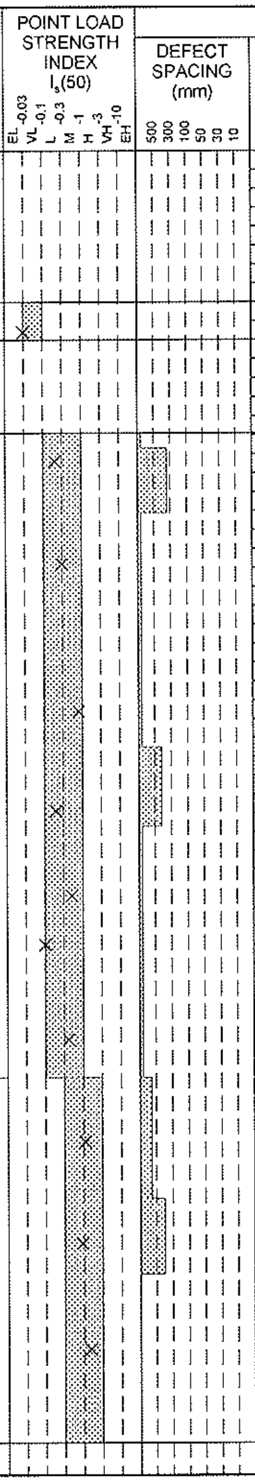
CORED BOREHOLE LOG

Borehole No.
102
2 / 2

Client: McLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Date: 08/02/11
Plant Type: JK300
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: ~45.58 m
Datum: AHD
Logged/Checked By: D.S./

Water Loss\Level	Barrel Lift	RL (m AHD)	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
		42			START CORING AT 3.80m			
100% RETURN			4		SANDSTONE: fine to coarse grained, light grey and orange brown.	DW	VL	
					CORE LOSS 0.49m			
		41 40 39	5 6		SANDSTONE: fine to coarse grained, light grey and orange brown banded.	DW	L - M	J, 40°, P J, 60°, Un, R XWS, 0°, 5 mm.t XWS, 20°, 25 mm.t
		38	7		SANDSTONE: fine to coarse grained, light grey, with dark grey laminae bedded at 0-20°.			XWS, 20°, 40 mm.t
		37	8		as above, but fine to medium grained.		M - H	CS, 0°, 3 mm.t XWS, 0°, 10 mm.t
		36	9		as above, but with sub rounded clay inclusions.			XWS, 0°, 100 mm.t
					END OF BOREHOLE AT 9.83 m			



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Ref: 17167ZR Borehole 102

Borehole No.
103
1 / 2

BOREHOLE LOG

Client: MCLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Method: SPIRAL AUGER
R.L. Surface: ~41.95 m
Date: 04/02/11
Datum: AHD
Plant Type: JK300
Logged/Checked By: D.S./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
ON COMPLETION OF CORING ▼				■						FILL: Silty gravelly sand, fine to medium grained, brown, with fine to medium grained sandstone and igneous gravel, concrete and brick fragments.	D			GRASS COVER APPEARS MODERATELY COMPACTED
					N > 4,0/ 150mm REFUSAL									
						41	1			FILL: Silty sand, fine to medium grained, brown, with a trace of fine gravel sized cemented nodules and ash.	D	MD		
				■	N = 14 5,6,8									
						40	2							
ON COMPLETION OF AUGERING ▼														
				■	N > 30 11,17,13/ 100mm REFUSAL	39	3							
						38	4							
				■	N = 20 6,7,13					FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained sandstone and ironstone gravel, fine gravel sized cemented nodules, brick fragments, ash and cresote inclusions.				HYDROCARBON ODOUR
						37	5		SP	SAND: fine to medium grained, yellow brown.		MD		
									-	SANDSTONE: fine to medium grained, light grey.	DW	VL - L		VERY LOW TO LOW 'TC' BIT RESISTANCE
						36	6					L - M		LOW TO MODERATE RESISTANCE
										REFER TO CORED BOREHOLE LOG				
						35								

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Borehole No.
103
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CORED BOREHOLE LOG

Client: McLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Core Size: NMLC
R.L. Surface: ~41.95 m
Date: 04/02/11
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: D.S./

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components. START CORING AT 6.13m	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
100% RETURN					SANDSTONE: fine to medium grained, light grey.	DW	L - M				
		35	7		CORE LOSS 0.44m						
		34 33	8 9		SANDSTONE: fine to medium grained, light grey, with fine to coarse grained bands.	DW	L - M				
					SHALE: dark grey.	DW	VL				
		32 31 30	10 11 12		SANDSTONE: fine to medium grained, light grey, bedded at 5-10°.	FR	M			XWS, 0°, 5 mm.t XWS, 0°, 3 mm.t	
		29			END OF BOREHOLE AT 12.33 m						

Ref: 17167ZR Borehole 103

Borehole No.
104
1 / 2

BOREHOLE LOG

Client: MCLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Method: SPIRAL AUGER
R.L. Surface: ~40.52 m
Date: 07/02/11
Datum: AHD
Plant Type: JK300
Logged/Checked By: D.S./

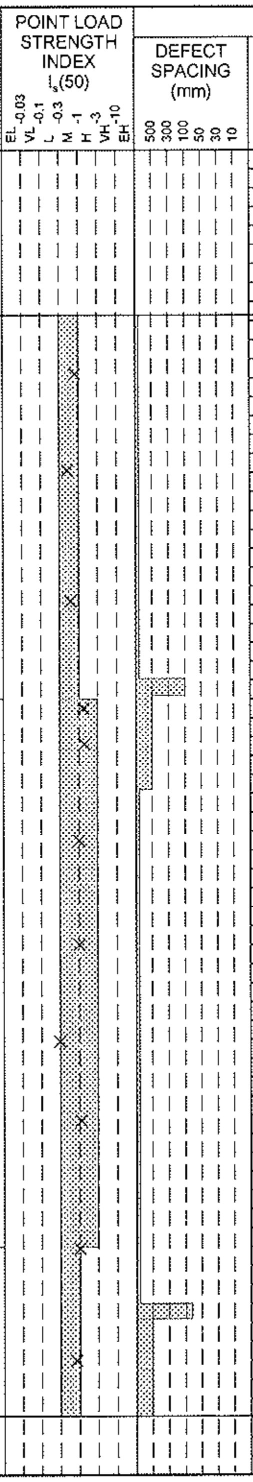
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
							0	▨		FILL: Silty sand, fine to medium grained, light brown and dark brown, with a trace of root fibres.	D			GRASS COVER APPEARS WELL COMPACTED
				■	N = 16 8,8,8	40		▨						
							1	░	SP	SAND: fine to medium grained, yellow brown.	D	L		
				■	N = 7 3,3,4	39		░			M			
						38	2	░						
				■	N = 45 11,20,25		3	░				D		
ON COMPLETION OF AUGERING						37		░			W			HYDROCARBON ODOUR
18-02-11							4	░						
				■	N = 28 5,13,15	36		░				MD		
							5	░						
						35		⁙		SANDSTONE: fine to medium grained, orange brown.	DW	L - M		LOW TO MODERATE 'TC' BIT RESISTANCE
							6			REFER TO CORED BOREHOLE LOG				
						34								

Borehole No.
104
2 / 2

CORED BOREHOLE LOG

Client: McLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Date: 07/02/11
Plant Type: JK300
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: ~40.52 m
Datum: AHD
Logged/Checked By: D.S./



Water Loss\Level	Barrel Lift	RL (m AHD)	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
		35			START CORING AT 5.87m				CLASS 18 SLOTTED PVC INSTALLED TO 5.6m, BACKFILLED WITH 2mm SAND, BENTONITE SEAL BETWEEN 0.2m AND 0.5m, GATIC COVER CONCRETED AT SURFACE
95% RETURN			6		SANDSTONE: fine to medium grained, orange brown.	SW	M		
		34 33	7		SANDSTONE: fine to medium grained, light grey.	FR		XWS, 0°, 30 mm.t XWS, 0°, 10 mm.t	
		32 31 30	8 9 10				M - H	XWS, 0°, 35 mm.t XWS, 0°, 5 mm.t	
		29	11				M	XWS, 0°, 5 mm.t XWS, 0°, 15 mm.t	
					END OF BOREHOLE AT 11.69 m				

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Ref: 17167ZR Borehole 104

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
105
1 / 2

BOREHOLE LOG

Client: MCLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Method: SPIRAL AUGER
R.L. Surface: 38.38 m
Date: 07/02/11
Datum: AHD
Plant Type: JK300
Logged/Checked By: D.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
										FILL: Silty sand, fine to medium grained, brown, with fine to medium grained ironstone, sandstone and igneous gravel, concrete and brick fragments and root fibres.	D			APPEARS MODERATELY COMPACTED
						38								
				■	N > 8 10,8/ 150mm REFUSAL									
							1							
						37								
				■	N = 4 1,2,2				SP	SAND: fine to medium grained, yellow brown.	D	L		
							2							
ON COMPLETION OF CORING														
						36								
ON COMPLETION OF AUGERING											W			HYDROCARBON ODOUR
							3					MD		
				■	N = 19 4,7,12									
						35								
							4							
						34								
				■	SPT 16/ 150mm REFUSAL									
							5							
										SANDSTONE: fine to medium grained, orange brown.	DW	L - M		LOW TO MODERATE 'TC' BIT RESISTANCE
						33								
										REFER TO CORED BOREHOLE LOG				
							6							
						32								

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

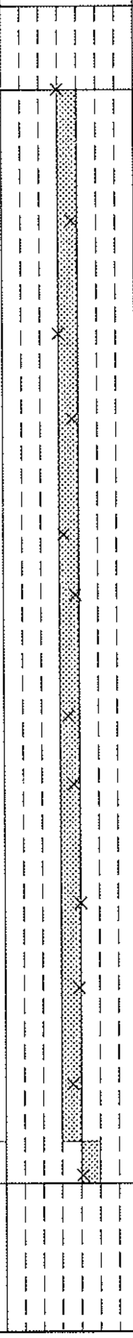
Borehole No.
105
2 / 2

CORED BOREHOLE LOG

Client: McLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Core Size: NMLC
R.L. Surface: 38.38 m
Date: 07/02/11
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: D.S./

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL -0.03, VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		33			START CORING AT 5.44m						
100% RETURN					SANDSTONE: fine to medium grained, orange brown.	DW	M				
		32	6		SANDSTONE: fine to medium grained, light grey, wth dark grey laminae, bedded at 0-20°.						
										XWS, 0°, 10 mm.t	
		31	7								
			8							XWS, 0°, 5 mm.t	
90% RETURN		30									
0% RETURN		29	9							XWS, 0°, 5 mm.t	
		28	10							XWS, 0°, 5 mm.t	
			11				H				
		27			END OF BOREHOLE AT 11.22 m						



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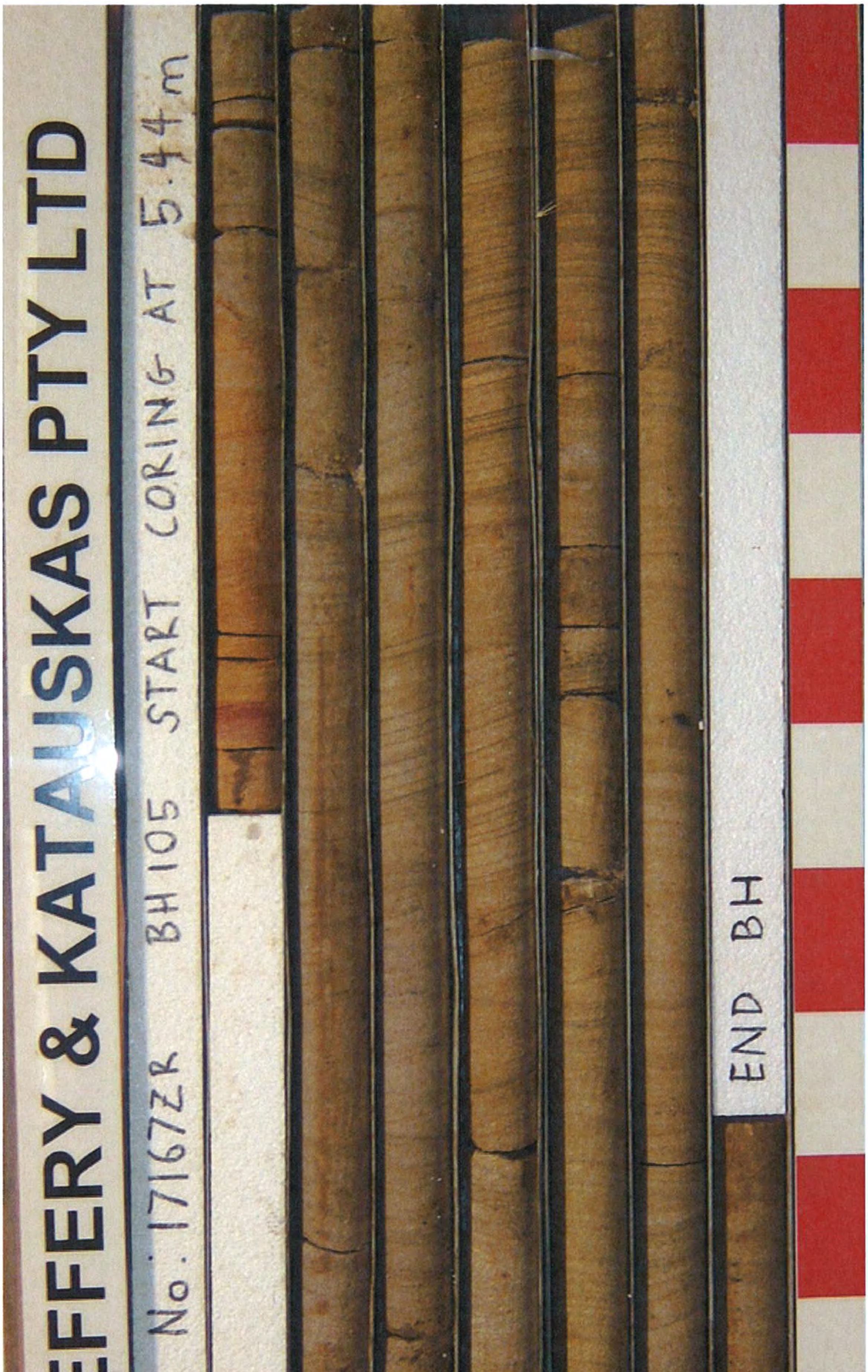

Borehole No.
106
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BOREHOLE LOG

Client: MCLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Method: SPIRAL AUGER
R.L. Surface: 41.95 m
Date: 04/02/11
Datum: AHD
Plant Type: JK350
Logged/Checked By: D.S./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
							0			FILL: Silty gravelly sand, fine to medium grained, brown, fine to medium grained sandstone and igneous grave and concrete fragments.	D			GRASS COVER
				■		41	1							
									SM	SILTY SAND: fine to medium grained, light grey and brown, with fine grained cemented nodules.	D	MD		
				■	N = 17 6,8,9					SILTY SAND: fine to medium grained, brown, with a trace of fine grained cemented nodules.				
						40	2							
ON COMPLETION OF AUGERING														
				■	N > 24 5,10,14/ 130mm REFUSAL	39	3				M			
AFTER 34 HRS														
18-2-11						38	4							
									SC	CLAYEY SAND: light grey.	W			
				■	SPT 6,0,0/ 100mm REFUSAL					SANDSTONE: fine to medium grained, light grey.	XW - DW	EL - VL		VERY LOW 'TC' BIT RESISTANCE
						37	5							
						36	6			REFER TO CORED BOREHOLE LOG				
						35								

Borehole No.
106

CORED BOREHOLE LOG

Client: McLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Core Size: NMLC
R.L. Surface: 41.95 m
Date: 04/02/11
Inclination: VERTICAL
Datum: AHD
Plant Type: JK350
Bearing: N/A
Logged/Checked By: D.S./

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL -0.03 VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific General
					START CORING AT 5.75m					CLASS 18 SLOTTED PVC INSTALLED TO 5.7m, BACKFILLED WITH 2mm SAND, BENTONITE SEAL BETWEEN 0.2m AND 0.5m, GATIC COVER CONCRETED AT SURFACE
100% RETURN		36	6		CORE LOSS 1.66m					
		35	7							
					SANDSTONE: fine to coarse grained, light grey.	DW	VL			XWS, 0°, 180 mm.t
		34	8			DW	L - M			
							M			CS, 0°, 70 mm.t
		33	9		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-20°.					
		32	10		as above, but fine to coarse grained.					
					SANDSTONE: fine to medium grained, light grey.		VL - L			XWS, 0°, 5 mm.t J, 70°, Un, R XWS, 25°, 5 mm.t J, 50°, Un, R XWS, 50°, 60 mm.t
		31	11							J, 90°, Un, R XWS, 0°, 90 mm.t J, 70°, P, S
							M			
		30								

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Borehole No.
106
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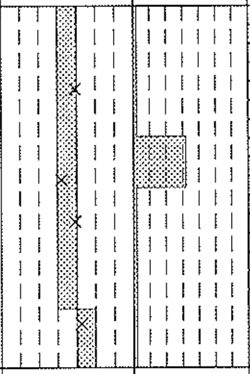
CORED BOREHOLE LOG

Client: McLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Date: 04/02/11
Plant Type: JK350
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: 41.95 m
Datum: AHD
Logged/Checked By: D.S./

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
100% RETURN		29	13		SANDSTONE: fine to medium grained, light grey, bedded at 0-15°.	DW	M H			CS, 0°, 3 mm.t CS, 0°, 2 mm.t CS, 0°, 1 mm.t	
		28	14		END OF BOREHOLE AT 13.91 m						
		27	15								
		26	16								
		25	17								
		24	18								
		23									



Ref: 17167ZR Borehole 106

Borehole No.
107
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BOREHOLE LOG

Client: MCLACHLAN LISTER PTY LTD
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MONTEFIORE JEWISH HOME, DANGAR STREET, RANDWICK, NSW

Job No.: 17167ZR
Method: SPIRAL AUGER
R.L. Surface: N/A
Date: 04/02/11
Datum: AHD
Plant Type: JK350
Logged/Checked By: D.S./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION							▨		FILL: Silty sand, fine to medium grained, brown, with fine grained igneous gravel and concrete fragments.				GRASS COVER
			■			1	▨		FILL: Sand, fine to medium grained, brown, with a trace of quartz cobbles.				
						2 3 4 5 6			END OF BOREHOLE AT 1.30 m				

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

STS
SOIL TEST SERVICES
ABN 43 002 145 173

Ref No: 17167ZR
Table A: Page 1 of 3

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	4.62-4.65	0.3	6
	5.17-5.21	0.7	14
	5.92-5.96	0.4	8
	6.22-6.26	0.7	14
	6.67-6.72	1.2	24
	7.08-7.12	1.6	32
	7.92-7.96	1.2	24
	8.18-8.22	1.8	36
	8.85-8.89	1.3	26
	9.26-9.29	1.7	34
	9.79-9.83	1.2	24
	10.11-10.14	1.1	22
102	3.96-3.99	0.03	<1
	4.64-4.68	0.2	4
	5.18-5.23	0.3	6
	5.96-6.00	0.8	16
	6.49-6.53	0.2	4
	6.94-6.98	0.5	10
	7.20-7.24	0.1	2
	7.70-7.74	0.4	8
	8.24-8.26	1.1	22
	8.78-8.82	0.9	18
	9.34-9.38	1.5	30
103	6.57-6.60	0.3	6
	7.58-7.61	0.04	<1

NOTES: See Page 3 of 3

Ref No: 17167ZR
Table A: Page 2 of 3

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
103	8.02-8.07	0.4	8
	8.60-8.64	0.4	8
	9.00-9.04	0.2	4
	9.66-9.71	0.6	12
	10.10-10.14	1.0	20
	10.68-10.72	0.3	6
	11.22-11.26	0.5	10
	11.68-11.71	0.5	10
104	6.18-6.22	0.8	16
	6.69-6.74	0.5	10
	7.38-7.42	0.6	12
	7.95-8.00	1.3	26
	8.14-8.19	1.4	28
	8.65-8.70	1.0	20
	9.20-9.24	1.0	20
	9.71-9.74	0.3	6
	10.13-10.18	1.1	22
	10.81-10.84	1.0	20
	11.40-11.44	0.8	16
105	5.44-5.48	0.3	6
	6.13-6.17	0.7	14
	6.73-6.77	0.3	6
	7.18-7.22	0.7	14
	7.79-7.7.83	0.4	8
	8.11-8.16	0.8	16

NOTES: See Page 3 of 3

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 988B 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
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Ref No: 17167ZR
Table A: Page 3 of 3

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
105	8.75-8.79	0.5	10
	9.11-9.15	0.7	14
	9.74-9.78	1.0	20
	10.19-10.23	0.9	18
	10.70-10.74	0.6	12
	11.18-11.22	1.1	22
106	7.49-7.53	0.03	<1
	8.11-8.15	0.7	14
	8.72-8.76	0.2	4
	9.29-9.33	0.7	14
	9.83-9.87	0.5	10
	10.19-10.22	0.3	6
	10.80-10.84	0.04	<1
	11.34-11.38	0.2	4
	11.90-11.95	0.8	16
	12.44-12.48	0.9	18
	12.92-12.97	0.4	8
	13.14-13.18	0.9	18
	13.68-13.72	1.3	26

NOTES:

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

115 Wicks Road
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North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
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Ref No: 17167ZR
Table B: Page 1 of 1

TABLE B
SUMMARY OF FOUR DAY SOAKED C.B.R.TEST RESULT

BOREHOLE NUMBER	107
DEPTH (m)	0.20 - 0.90
Surcharge (kg)	4.5
Maximum Dry Density (t/m^3)	1.76 STD
Optimum Moisture Content (%)	11.4
Moulded Dry Density (t/m^3)	1.72
Sample Density Ratio (%)	98
Sample Moisture Ratio (%)	102
Moisture Contents	
Insitu (%)	7.3
Moulded (%)	11.7
After soaking and	
After Test, Top 30mm(%)	12.9
Remaining Depth (%)	12.6
Material Retained on 19mm Sieve (%)	0
Swell (%)	0.0
C.B.R. value: @5.0mm penetration	25

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  (a) Soaked C.B.R. : AS 1289 6.1.1
  (b) Standard Compaction : AS 1289 5.1.1
  (c) Moisture Content : AS 1289 2.1.1

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Approved Signatory
(A.Tatikonda)

Date: 23/12/11

TABLE C
SUMMARY OF LABORATORY RESULTS
SOIL CHEMISTRY - pH, SULPHATE & CHLORIDES

Borehole Number	**Sample Depth (m)**	**Sample Description**	**pH Units**	**Sulphate (mg/kg)**	**Chloride (mg/kg)**
BH101	1.5 - 1.95	SAND	6.7	<10	<10
BH102	3.0 - 3.45	SAND	7.5	<10	<10
BH105	1.5 - 1.95	SAND	8.4	<10	<10
BH106	3.0 - 3.45	Silty SAND	7.9	<10	57

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 (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

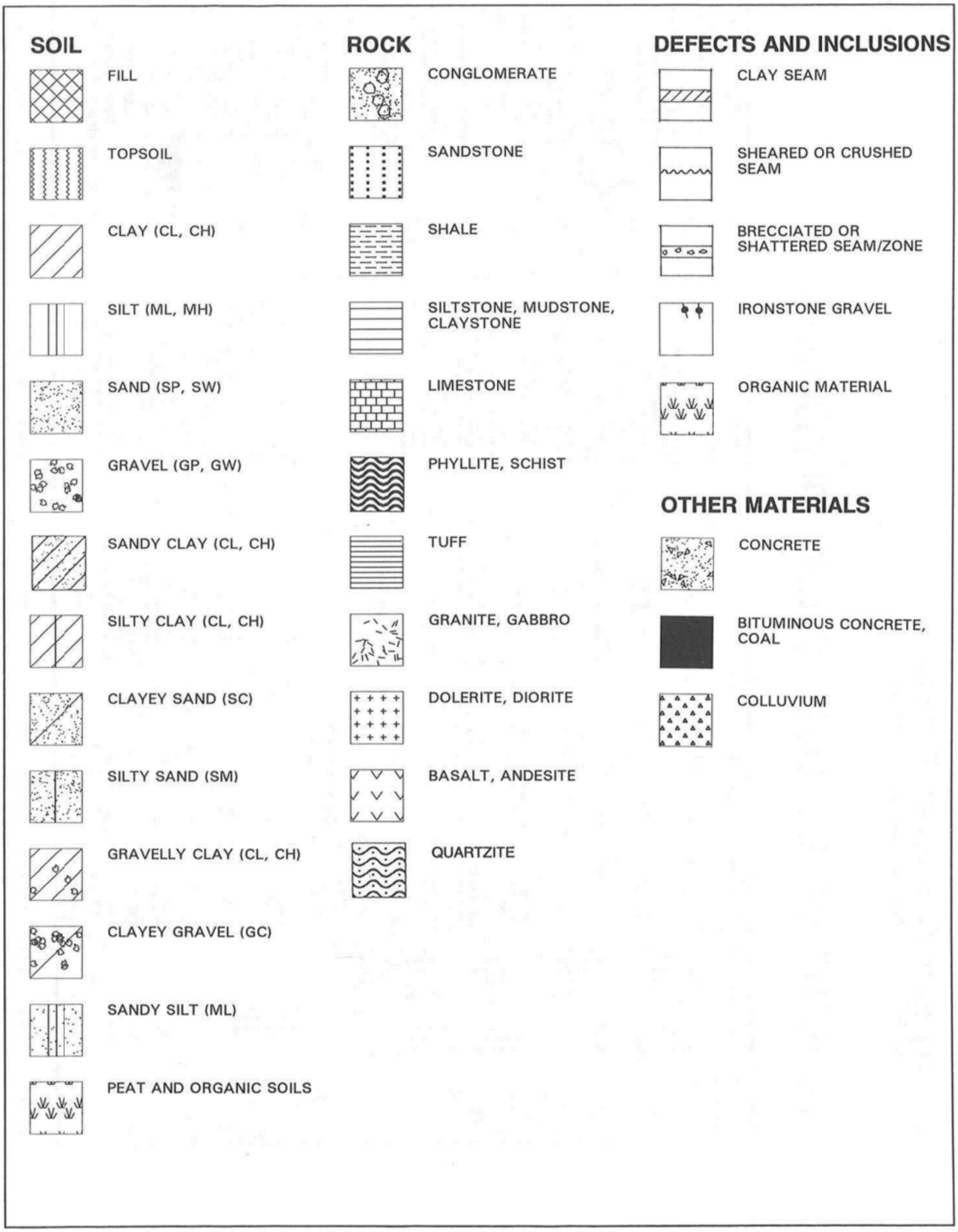
Requirements could range from:

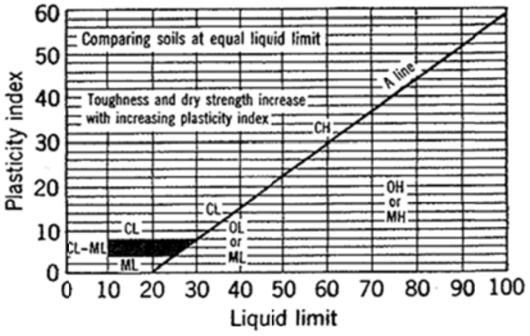
i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

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

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



<table>
<tr><th colspan="7">Field Identification Procedures
(Excluding particles larger than 75 μm and basing fractions on estimated weights)</th><th>Group Symbols[a]</th><th>Typical Names</th><th>Information Required for Describing Soils</th><th colspan="4">Laboratory Classification Criteria</th></tr>
<tr><td rowspan="8">Coarse-grained soils
More than half of material is *larger* than 75 μm sieve size[b]</td><td rowspan="16">(The 75 μm sieve size is about the smallest particle visible to naked eye)</td><td rowspan="4">Gravels
More than half of coarse fraction is larger than 4 mm sieve size</td><td rowspan="2">Clean gravels (little or no fines)</td><td colspan="3">Wide range in grain size and substantial amounts of all intermediate particle sizes</td><td>*GW*</td><td>Well graded gravels, gravel-sand mixtures, little or no fines</td><td rowspan="8">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*)</td><td rowspan="17">Use grain size curve in identifying the fractions as given under field identification</td><td rowspan="8">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</td><td colspan="2">$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4
$C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>*GP*</td><td>Poorly graded gravels, gravel-sand mixtures, little or no fines</td><td colspan="2">Not meeting all gradation requirements for *GW*</td></tr>
<tr><td rowspan="2">Gravels with fines (appreciable amount of fines)</td><td colspan="3">Nonplastic fines (for identification procedures see *ML* below)</td><td>*GM*</td><td>Silty gravels, poorly graded gravel-sand-silt mixtures</td><td>Atterberg limits below "A" line, or *PI* less than 4</td><td rowspan="2">Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures, see *CL* below)</td><td>*GC*</td><td>Clayey gravels, poorly graded gravel-sand-clay mixtures</td><td>Atterberg limits above "A" line, with *PI* greater than 7</td></tr>
<tr><td rowspan="4">Sands
More than half of coarse fraction is smaller than 4 mm sieve size</td><td rowspan="2">Clean sands (little or no fines)</td><td colspan="3">Wide range in grain sizes and substantial amounts of all intermediate particle sizes</td><td>*SW*</td><td>Well graded sands, gravelly sands, little or no fines</td><td colspan="2">$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6
$C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>*SP*</td><td>Poorly graded sands, gravelly sands, little or no fines</td><td colspan="2">Not meeting all gradation requirements for *SW*</td></tr>
<tr><td rowspan="2">Sands with fines (appreciable amount of fines)</td><td colspan="3">Nonplastic fines (for identification procedures, see *ML* below)</td><td>*SM*</td><td>Silty sands, poorly graded sand-silt mixtures</td><td>Atterberg limits below "A" line or *PI* less than 5</td><td rowspan="2">Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures, see *CL* below)</td><td>*SC*</td><td>Clayey sands, poorly graded sand-clay mixtures</td><td>Atterberg limits below "A" line with *PI* greater than 7</td></tr>
<tr><td rowspan="8">Fine-grained soils
More than half of material is *smaller* than 75 μm sieve size</td><td colspan="5">Identification Procedures on Fraction Smaller than 380 μm Sieve Size</td><td rowspan="2"></td><td rowspan="2"></td><td rowspan="2"></td><td colspan="3" rowspan="9">
Plasticity chart for laboratory classification of fine grained soils</td></tr>
<tr><td colspan="2" rowspan="4">Silts and clays liquid limit less than 50</td><td>Dry Strength (crushing characteristics)</td><td>Dilatancy (reaction to shaking)</td><td>Toughness (consistency near plastic limit)</td></tr>
<tr><td>None to slight</td><td>Quick to slow</td><td>None</td><td>*ML*</td><td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity</td><td rowspan="7">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*)</td></tr>
<tr><td>Medium to high</td><td>None to very slow</td><td>Medium</td><td>*CL*</td><td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td></tr>
<tr><td>Slight to medium</td><td>Slow</td><td>Slight</td><td>*OL*</td><td>Organic silts and organic silt-clays of low plasticity</td></tr>
<tr><td colspan="2" rowspan="3">Silts and clays liquid limit greater than 50</td><td>Slight to medium</td><td>Slow to none</td><td>Slight to medium</td><td>*MH*</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td></tr>
<tr><td>High to very high</td><td>None</td><td>High</td><td>*CH*</td><td>Inorganic clays of high plasticity, fat clays</td></tr>
<tr><td>Medium to high</td><td>None to very slow</td><td>Slight to medium</td><td>*OH*</td><td>Organic clays of medium to high plasticity</td></tr>
<tr><td colspan="4">Highly Organic Soils</td><td colspan="3">Readily identified by colour, odour, spongy feel and frequently by fibrous texture</td><td>*Pt*</td><td>Peat and other highly organic soils</td></tr>
</table>

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	—▼—	Standing water level. Time delay following completion of drilling may be shown.
	—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.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

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	