REPORT TO

HAMMONDCARE

ON

ADDITIONAL GEOTECHNICAL INVESTIGATION

FOR

PROPOSED HOSPITAL REDEVELOPMENT

AT

97-115 RIVER ROAD, GREENWICH, NSW

Date: 10 May 2022
Ref: 32507R2rpt Rev2

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1 INTRODUCTION

This report presents the results of an additional geotechnical investigation for the proposed hospital redevelopment at 97-115 River Road, Greenwich, NSW.

This Additional Geotechnical Investigation Report is submitted to the Department of Planning, Industry and Environment (DPIE) in support of a State Significant Development Application (SSD-13619238) for the redevelopment of Greenwich Hospital into an integrated hospital and seniors living facility on land identified as 97-115 River Road, Greenwich (the site). The extent of the site is shown below.

The subject proposal is for the detail design and construction of the facility following its concept approval under SSD-8699. Specifically, SSD-13619238 seeks approval for the following:

- Demolition of the existing hospital building and associated facilities at the site;
- Construction of a new hospital facility and integrated healthcare campus comprising of hospital, residential aged care, seniors housing, overnight respite, across:
  - A new main hospital building up to RL 80.0;
  - Two new seniors living buildings, Northern building up to RL 56.36, and Southern building up to RL 60.65;
  - A new 2-3 storey respite care building up to RL 56.9;
- Construction of associated site facilities and services, including pedestrian and vehicular access and basement parking;
- Site landscaping and infrastructure works; and
- Preservation of Pallister House which will continue to host dementia care and administrative functions.

JK Geotechnics note that the development plans issued to JK Geotechnics on 1 April 2022 indicate the new 7 storey main hospital building is to be constructed above set-down and mezzanine levels. The buildings will be terraced to account for the slope of the site.

We have been provided with architectural drawings (Drawing Numbers DD-HST-0100 to 0110 Rev. P6 to P11, dated 25 January 2021, DD-SLN-0200 to 0205 Rev. P1 or P2, dated 9 December 2021, DD-SLS-0200 to 0206 Rev. P1 or P2, dated 9 December 2021 and DD-SW-0200 to 0210 Rev. P16, P18 or P8, dated 1 April 2022) prepared by Bickerton Masters Architecture Pty Ltd.

Based on a review of the provided architectural drawings, we understand that following demolition of all existing buildings and structures (excluding Pallister House), the proposed staged hospital redevelopment will include:

- The main hospital building and two serviced seniors living buildings constructed over one or two levels of carparking. The proposed Level 1 car park finished floor reduced level (RL) will be formed at between RL37.95m and RL38.6m. Bulk excavations to a maximum depth of about 14.0m to the east and north into the hillside will be required to achieve design surface levels;
- Construction of a new two to three storey respite care building to the east of the main building with a lower floor level suspended over the hillside possibly requiring localised excavations to the west into the hillside to a maximum depth of about 1.0m; and
- Reconfiguration of the surrounding areas to include new access roads, external parking areas, walkways and landscaped areas.

Structural loads have not been provided and we have assumed typical loadings for this type of development apply.

We note that we have prepared the following previous geotechnical reports at the site on behalf of HammondCare:

1. Preliminary geotechnical assessment report (Ref. 23789ZRrpt) dated 19 February 2010 (JK2010).
2. Geotechnical report for a proposed car park (Ref. 23789ZRrpt2) dated 20 June 2014 (JK2014).
3. Additional geotechnical assessment for a proposed car park (Ref. 23789ZRrpt3) dated 30 November 2016 (JK2016).
4. Geotechnical report for Hospital Redevelopment (Ref. 32507Rrpt) dated 19 September 2019 (JK2019).

The relevant geotechnical information from our previous geotechnical reports is presented in the attached Appendix A.

The purpose of the investigation was to obtain additional geotechnical information on the subsurface conditions and to use this as a basis for updated comments and recommendations on demolition, excavation, retention, footing design, drainage, hydrogeology, earthworks, floor slabs, external paved areas and soil aggression.

This geotechnical investigation was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate reports by JKE for the results of the environmental site assessment:

- The acid sulfate soil assessment report: E32507BRrpt3.
- The salinity report: E32507BRrpt4.
- The contamination report: E32507BRrpt5.

In accordance with section 4.39 of the Environmental Planning & Assessment Act 1979 (EP&A Act), the Secretary's Environmental Assessment Requirements (SEARs) for SSD-13619238 were issued on 24 February, 2021. This report has been prepared to respond to the following SEARs:

SEAR	Relevant section of report
Plans and Documents • Geotechnical and Structural Report.	This report relates to the additional geotechnical investigation required to prepare the Geotechnical Report and to assist the preparation of the Structural Report by the structural engineering consultant. The results of the investigation are presented in Section 3 and our comments and recommendations are presented in Section 4.

2 INVESTIGATION PROCEDURE

The fieldwork for the additional geotechnical investigation was carried out between 27 and 30 September 2021 and comprised:

- Six boreholes (BH105 to BH110) auger drilled to depths between 1.24m and 3.13m using our track mounted JK205 or JK305 drill rigs. The boreholes were extended by diamond core drilling using NMLC coring techniques with water flush to final depths between 7.83m and 15.00m.
- In BH105, BH106, BH107 and BH109, groundwater monitoring wells were installed to their final depths, and comprised 50mm diameter Class 18 slotted PVC pipes. Further details of the monitoring well installations are provided on the attached borehole logs.

We also note that as part of the additional JKE investigation for fieldwork carried out on 1 and 6 October 2021, four boreholes (BH101, BH102, BH104 and BH119) were auger drilled to depths between 2.05m and 4.37m using our track mounted JK205 or JK305 drill rigs. The boreholes were extended by diamond core drilling using NMLC coring techniques with water flush to final depths between 5.75m and 7.5m. Groundwater monitoring wells were installed to their final depths in all these environmental boreholes, and comprised 50mm diameter Class 18 slotted PVC pipes. Further details of the monitoring well installations are provided on the attached borehole logs

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

The borehole locations as shown on the attached Figure 2 were set out by taped measurements from existing surface features. The borehole locations from our previous JK2014 and JK2019 investigations are also included on Figure 2. Figure 2 is based on aerial imagery sourced from 'Nearmap' with the outline of the proposed Level 1 (the basement), Seniors Living North and South, Health and Care and Respite buildings superimposed. The approximate surface RL's at the borehole locations were interpolated between spot levels shown on the previously provided survey plan (Ref. 32677 008DT Rev. B dated 4 July 2019) prepared by LTS Lockley. The survey datum is the Australian Height Datum (AHD).

The state of compaction of the fill and the relative density/strength of the residual sands/clays were assessed from the Standard Penetration Test (SPT) 'N' values augmented by the results of hand penetrometer readings on cohesive soil samples recovered in the SPT split tube. 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, examination of the recovered rock cuttings and correlations with subsequent laboratory moisture content test results. The assessment of rock strength in this way is approximate, and variations of about one order of strength should not be unexpected. The strength of the bedrock within the cored portion of the boreholes was assessed by examination of the recovered rock core and in BH105 to BH110 (the geotechnical boreholes) subsequent correlation with laboratory Point Load Strength Index testing.

Groundwater observations were also made in the boreholes during and on completion of auger drilling and on completion of core drilling. On completion of the geotechnical investigation, the groundwater monitoring wells were purged (water pumped out of the wells) by JKE. We returned to site on 20 and 22 October 2021 (approximately three weeks after completion of the geotechnical fieldwork and two weeks after the completion of the environmental fieldwork) to record the groundwater levels in the monitoring wells. No further groundwater level monitoring has been carried out.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

The fieldwork for the most recent investigation was carried out under the direction of our geotechnical engineer (Joanne Lagan), who was present full-time on site, and set out the borehole locations, directed the buried services scans, logged the encountered subsurface profile, nominated in-situ testing and sampling, and directed installation of the groundwater monitoring wells. The borehole logs (which include field test results, Point Load Strength Index test results and groundwater observations) are attached, together with a glossary of logging terms and symbols used.

Selected soil samples were returned to the Soil Test Services Pty Ltd (STS) NATA registered laboratory, for moisture content, Atterberg Limits and linear shrinkage testing. The results are summarised in the attached STS Table A. The recovered rock core was returned to the JK Geotechnics (JKG) offices 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 Table A. The core photographs for BH105 to BH110 are included opposite the relevant cored borehole log.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

Based on our recent site observations, the site description presented in Section 3.1 of the previous report, Ref.32507Rrpt, dated 19 September 2019 is still valid and for ease of reference is reproduced below.

The site is located within undulating terrain towards the crest of a hillside that slopes and steps down to the south, with localised slopes down to the east and west defining gully features orientated approximately north-south.

The subject site comprises the grounds of Greenwich Hospital and has northern and eastern frontages onto River Road and St. Vincents Road, respectively. The southern site boundary and the northern portion of the western site boundary are lined by the residential yard areas. The south-western portion of the site boundary is lined by Gore Creek Reserve.

At the time of the assessment, the northern-central portion of the site was occupied by one to five level (typically one and two level) brick (occasionally fibro) hospital buildings. The central-southern portion of the site was occupied by a two-storey brick building (Pallister House).

The hospital buildings were connected by bitumen and asphaltic concrete (AC) surfaced driveways, access roads and footpaths. Grassed covered and landscaped areas formed covered much of the remainder of the site with the exception of the bitumen and gravel surfaced car park areas scattered across the site.

The site topography was characterised by a relatively flat elevated central and north-eastern portion that extended north-south across the site. Site surface levels generally stepped and sloped down to the south-east, south, west and south-west from the higher elevation area. The slopes and steps were formed by fill batter slopes (typically 0.9m to 2.0m height and battered at about 15^o), sandstone bedrock outcrops (typically between about 0.5m to 3.5m height) and retaining walls (described in more detail below). The building platforms appear to have exploited relatively flat elevated areas with some localised cut and fill earthworks to extend the building platforms and to create parking and landscape areas.

The retaining walls within the site were typically less than 1.5m high and of sandstone masonry, stacked sandstone, brick, concrete, and concrete segmental block construction. Generally, these walls were in good condition, although some walls contained hairline to 30mm wide cracking; the larger crack widths appeared to be associated with tree root growth. The more significant retaining structures comprised:

- Sandstone masonry walls of maximum 3.5m and 3m heights which supported areas lining the eastern side of the driveway (below the Main Hospital Wing) and the north-western and western side of Pallister House, respectively and had faces sloping at about 35^o to 40^o. The walls were generally in reasonably good condition with the exception of occasional missing or displaced blocks. However, our observations were limited as significant lengths of the walls were overgrown.
- A concrete crib wall supporting the western side of the car park adjacent to the Service Wing which was a maximum height of about 2.6m, with a face that sloped down to the west at about 70^o. The wall appeared to be in reasonably good condition with some erosion of the sand backfill (which included gravel to small cobble sized inclusions of brick, sandstone and fragments of fibro sheeting).

Over the western side of the site, an overgrown uneven surfaced fill batter slope lined the western and south-western side of the existing car park and sloped down to the west and south-west at between about 30° and 45°. Our site inspections in 2014 and 2016 noted that stepped sandstone outcrop faces (between about 2.5m and 7.0m height were situated at the toe of the southern and central portions of the fill batter slope. The outcrop faces were typically defined by a sub-vertical planar joint orientated approximately 110° with occasional overhang features noted. The outcrop face extended to the south and increased in height to a maximum of about 10.0m. The toe of the southern section of the outcrop face was lined by a bushland area that gently sloped and stepped down to the south-west beyond the site boundary to the cliff face (about 25.0m height) that lined the northern side of the Gore Creek Reserve and formed the southern portion of the western site boundary.

The western and south-western sides of the car park were lined by a dilapidated low height metal fence that was leaning over from vertical and had an uneven alignment. No tension cracks orientated parallel with the crest of the slope were evident at the time of the current or previous inspections.

An abandoned pool was situated close to the western end of the southern site boundary. The southern and western sides of the pool area were supported by brick walls of about 1.5m maximum height. The south-western corner of the wall was in poor condition; what appeared to be a previously collapsed section of wall (about 0.5m wide and 1m high) was evident.

Our previous inspection in 2010 followed a period of heavy and prolonged rainfall. It appeared from surface erosion traces over the fill batter slope above the north-eastern corner of the abandoned pool that surface run-off from the nearby car park surface discharged down the fill batter slope. At the time of the previous inspection, significant quantities of water were discharging from a stormwater pipe (about 0.7m diameter) within a sandstone masonry headwall located at the crest of the northern end of the cliff face lining the southern half of the western site boundary. The discharged water cascaded down the cliff face to Gore Creek below.

In 2016 following sewer pipe line works over the north-western portion of the site, we noted areas of what appeared to be poorly compacted fill forming the fill batter slope (formed at a maximum slope of about 35°) with erosion rills (maximum depth about 0.5m) and tension cracks (maximum 40mm wide and 60mm deep) parallel, and approximately perpendicular to, the crest of the slope, identified. Fill debris had collected at the toe of the slope at the location of the neighbouring rear yard fence. The chain link fence was bowing out and leaning over.

Based on a cursory inspection, the hospital buildings generally appeared to be in good condition although some cracking of the rendered walls supporting the verandah over the south-eastern corner of Pallister House was evident. The existing asphaltic concrete (AC) paved car park surfaces were in variable condition but were typified by numerous cracking (maximum 10mm width), with pot holes and areas of previous car park surface repair evident. The AC driveways leading into the site were generally in good condition.

Site surface levels were generally similar across the northern and eastern site boundaries. However, a sandstone outcrop (about 0.5m high) was located at the southern end of the eastern site boundary. In addition, immediately to the south of the eastern driveway entry, the chain/metal fence was leaning and bulging; the base of the fence was supporting sandstone and brick rubble fill.

The southern site boundary was lined by neighbouring residential properties. Brick or rendered one to three level buildings lined or were set-back about 5.0m from the southern site boundary. Based on a cursory inspection from within the site the neighbouring buildings generally appeared to be in good external condition within only occasional hairline to 2mm wide cracking evident. Site surface levels were generally similar across the central and eastern portions of the boundary although the sandstone outcrop to the east of Pallister House extended south-west into the neighbouring properties. The central-western portion of the southern site boundary was lined by brick and timber retaining walls (about 1.0m height) which supported the subject site and appeared to be in reasonable condition, based on limited observations (due to access restrictions and vegetative cover).

The northern and central portion of the western site boundary was lined by neighbouring residential yards and pools; the toe of one of the above-mentioned fill batter slopes extended to this portion of the western site boundary. Two and three level rendered and timber houses were set-back at least 1.0m from this portion of the western site boundary. The northern end of the western site boundary was lined by a concrete wall (maximum height about 1.5m) which supported the subject site. Based on a cursory inspection from within the site, and where observations were possible, the neighbouring buildings and structures appeared to be in good condition.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone. The generalised subsurface profile disclosed by the boreholes from our previous and current investigations comprised a variable thickness of fill overlying an intermittent thin layer of residual soils with weathered sandstone bedrock encountered at shallow to moderate depth. Groundwater seepage was not recorded during the investigation but seepage was recorded at depth in the monitoring wells after completion of the fieldwork. For detailed subsurface conditions at each location, reference should be made to the attached borehole logs and the boreholes logs from our previous investigations presented in Appendix A, which also includes the previous laboratory test results. The borehole logs, geotechnical site plan, sections and laboratory test results from our previous investigations (JK2014 and JK2019) are presented in Appendix A. A summary of some of the more pertinent subsurface conditions is outlined below:

Paved Surface

The thickness of the existing AC paved surfaces at the locations of BH101, BH102, BH104, BH105, BH106, BH109, BH119 and in BH3 to BH6 (JK2019) ranged between 10mm and 65mm. The JK2014 boreholes situated over the south-western portion of the site encountered an AC paved surface 30mm to 40mm thick.

Fill

From surface level or below the AC paved surfaces, sandy and clayey fill with varying silt 'fines' and gravel content extended to depths between 0.3m (BH106 and BH119) and 3.4m (BH104). Based on the SPT 'N' values, the fill was assessed to be moderately or poorly compacted.

The JK2019 boreholes also encountered fill from surface level or below the AC paved surfaces and comprised sand with varying silt 'fines' and gravel content, or occasionally silty sandy gravel. The fill extended to depths between 0.3m and 3.9m and was assessed to be moderately or poorly compacted. The JK2014 boreholes situated over the south-western portion of the site encountered similar fill which extended to depths between 2.3m and 5.6m below the AC paved surface.

Residual Soils

Residual soils were encountered below the fill in all boreholes except BH101, BH102, BH104, BH108 and BH119, and comprised silty clay in BH106 and BH107, sandy clay in BH105 and BH109, and silty sand in BH107 overlying the silty clay. The residual clays were generally assessed to be of low (occasionally medium) plasticity and very stiff to hard (occasionally stiff) strength. The residual soil in BH107 comprised silty sands of very loose relative density. The residual soils extended to the sandstone bedrock surface at depths of 1.4m (BH105), 0.7m (BH106), 1.3m (BH107), and 1.0m (BH109).

Residual soils were only encountered below the fill in BH4, BH5 and BH6 (JK2019) and extended to the sandstone bedrock surface at respective depths of 2.3m, 1.0m and 0.9m. In BH4 the residual soils comprised clayey sands with an inferred loose relative density. In BH5 and BH6, the residual soils comprised sandy clay assessed to be of low to medium plasticity and very stiff or hard strength. The JK2014 boreholes situated over the south-western portion of the site encountered an intermittent layer of residual sandy or silty clay below the fill. The residual clays were assessed to be of medium plasticity and stiff to very stiff strength.

Weathered Sandstone Bedrock

Weathered sandstone bedrock was encountered in all the boreholes below the residual soils or fill at depths between 0.3m (BH119) and 3.4m (BH104). Weathered sandstone bedrock was also encountered in all the JK2019 boreholes at depths between 0.3m (BH8) and 3.9m (BH1).

The recorded RLs of the top surface of the bedrock in the current geotechnical and environmental investigations and the JK2019 and JK2014 investigations indicate that the bedrock surface steps down to the east, west and south from BH7 (RL51.6m) and BH107 (RL50.3m) over the central-northern portion of the site, to about RL50.5m (BH8) and RL50.1m (BH108), RL48.1m (BH9 and BH109), RL48.6m (BH6) and RL48.4m (BH106), RL47.4m (BH110), RL42.2m (BH119), RL43.5m (BH5) and RL43.4m (BH105), RL40.4m (BH101), RL38.2m (BH104), RL36.5m (BH102), RL33.4m (BH1) and RL29.9m (BH6 from JKG2014 drilled over the south-western portion of the site).

On first contact, the sandstone bedrock in the current investigations was of variable quality ranging from extremely weathered and of dense to very dense relative density or hard soil strength to distinctly or moderately weathered and medium to high strength. With depth, the sandstone was typically assessed to be slightly weathered or fresh and of medium or high strength. Occasional variable strength siltstone bands (maximum thickness about 0.5m) were encountered in BH107 and BH110.

In the JK2019 investigation, on first contact the sandstone bedrock was also variable; assessed to be highly to slightly weathered and of very low to medium to high strength. With depth, the sandstone was typically assessed to be slightly weathered or fresh and of medium or high strength. The JK2014 boreholes drilled over the south-western portion of the site encountered similar quality sandstone bedrock.

Within the cored sections of the boreholes from our current investigation and our previous JK2019 and JK2014 investigations, the following defects were recorded:

- Bedding partings dipping between 0° and 30°.
- Typically sub-horizontal clay seams, extremely weathered seams and occasional crushed seams ranging between about 1mm and 80mm thickness were encountered. A number of these defects were recorded as dipping at between about 5° and 30°.
- Occasional planar and undulating joints dipping at between 15° and 85°, and with clay or carbonaceous infill (maximum 15mm thick).

Within the cored sections of the boreholes from our current investigation and our previous JK2019 and JK2014 investigations, the following no core zones were also encountered:

- In BH104 at 3.4m depth (about 0.74m thick)
- In BH107 at 2.87m depth (about 0.13m thick) and at 10.83m depth (about 0.22m thick).
- In BH3 (JK2019) at 2.22m depth (about 0.42m thick).
- In BH9 (JK2019) at 1.93m depth (about 0.19m thick).
- In BH6 (JK2014) at 7.65m depth (1.12m thick).

The no core zones may be interpreted as representing clay seams, extremely weathered seams and/or fractured bands of bedrock which have been eroded by the coring process.

The sandstone exposed in the outcrop faces was assessed to be moderately weathered and of medium strength with sub-vertical planar joints orientated (striking) at approximately 110°.

Groundwater

All boreholes were 'dry' during, and on completion of auger drilling. We note that water is introduced during core drilling and obscures groundwater measurements. Water used whilst coring was pumped out after the completion of fieldwork. The groundwater levels measured on the 20 or 22 October 2021 (approximately two to three weeks after completion of the geotechnical and environmental phases of the fieldwork) in BH101, BH102, BH105, BH106, BH107 and BH109 were 3.7m (RL38.4m), 4.35m (RL33.3m), 6.04m (RL38.8m), 9.32m (RL39.8m), 9.94m (RL41.7m) and 7.69m (RL41.4m), respectively. No groundwater was recorded in BH104 and BH119. We note that no longer term groundwater level monitoring has been carried out.

Water flush returns in the current and previous both investigations were estimated to be between 0% and 100% which indicates a variable permeability rock mass.

3.3 Laboratory Test Results

Based on the Liquid Limit and Linear Shrinkage determinations the residual clays in BH106 and BH109 were confirmed to be of low plasticity with an assessed slight potential for shrink/swell reactivity with changes in moisture content.

Based on the Liquid Limit and Linear Shrinkage determinations the clayey fill in BH1 and BH2 (JK2019) was confirmed to be of low to medium plasticity with an assessed slight potential for shrink/swell reactivity with changes in moisture content.

A summary of the previous JK2019 laboratory chemical test results is provided in the table below:

Borehole Number	Sample Depth (m)	Description	pH Units	Sulfate (mg/kg)	Chloride (mg/kg)	Resistivity (ohm cm)
1	0.7 – 0.95	FILL: silty clay	8.0	28	<10	13,000
2	0.5 – 0.95	FILL: silty sand	8.0	29	<10	11,000
4	0.5 – 0.95	FILL: silty sand	8.7	23	10	11,000
5	0.5 – 0.95	Sandy CLAY	4.7	360	<10	5,300
5	1.0 – 1.2	SANDSTONE	5.2	39	<10	29,000
5	2.1 – 2.3	SANDSTONE	5.1	43	<10	25,000

The laboratory chemical test results carried out on fill and residual clay soil samples recovered from the boreholes completed in the previous JK2014 investigation indicated that:

- The soil pH ranged between 4.4 and 8.8.
- The sulfate content ranged between 31 and 460 mg/kg, and
- The chloride content ranged between <10 and 100 mg/kg.

The point load test results from the current geotechnical investigation indicated that the rock cored in the boreholes were predominantly of medium to high strength, with some low strength bands. The estimated Unconfined Compressive Strengths (UCS) ranged between 6MPa and 48MPa, with approximately 70% of the estimated UCS values ranging between 20MPa and 50MPa.

The point load test results from the previous JK2014 and JK2019 investigations indicated that the rock cored in the boreholes was of low to high strength, with estimated Unconfined Compressive Strengths (UCS) ranging between 2MPa and 50MPa, with approximately 60% of the estimated UCS values ranging between 20MPa and 50MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Engineering Classification of Sandstone Bedrock

Based on Pells et al (2019), an indicative engineering classification of the sandstone bedrock has been carried out based on the boreholes and the laboratory test results, as tabulated below.

Borehole	**Indicative Sandstone Class**			
	Depth/(RL) Top of Class V	**Depth/(RL) Top of Class IV**	**Depth/(RL) Top of Class III**	**Depth/(RL) Top of Class II (or better)**
101	-	1.7m*# (RL40.4m)	5.7m# (RL36.4m)	-
102	-	1.2m*# (RL36.5m)	4.6m# (RL33.1m)	-
104	3.4m# (RL38.2m)	4.4m# (RL37.2m)	-	-
105	1.4m* (RL43.4m)	3.1m (RL41.7m)	-	-
106	0.7m* (RL48.4m)	6.3m (RL42.8m)	1.2m (RL47.9m)	10.2m (RL38.9m)
107	10.3m (RL41.3m)	1.3m* (RL50.3m)	3.2m (RL48.4m) 12.2m (RL39.4m)	-
108	-	0.4m* (RL50.1)	-	2.4m (RL48.1m)
109	-	-	1.0m* (RL48.1)	11.5m (RL37.6m)
110	-	-	-	1.4m (RL47.1m)
119	-	0.3m*# (RL42.2m)	-	-
1 (JK2019)	-	-	3.9m* (RL33.4m)	5.6m (RL31.7m)
2 (JK2019)	-	2.4m** (RL36.6m)	-	-
3 (JK2019)	1.2m (RL41.6m)	2.7m (RL40.1m)	-	4.2m (RL38.6m)
4 (JK2019)	2.5m* (RL39.0m)	-	-	4.0m (RL37.5m)
5 (JK2019)	-	1.0m** (RL43.5m)	-	-
6 (JK2019)	-	-	-	0.9m* (RL48.6m)
7 (JK2019)	-	0.4m* (RL51.6m)	-	3.0m (RL49.0m)
8 (JK2019)	-	-	0.3m** (RL50.5m)	-

Borehole	Indicative Sandstone Class			
	Depth/(RL) Top of Class V	Depth/(RL) Top of Class IV	Depth/(RL) Top of Class III	Depth/(RL) Top of Class II (or better)
9 (JK2019)	0.9m* (RL48.1m)	3.8m (RL45.2m)	2.2m (RL46.8m) 4.9m (RL44.1m)	-
10 (JK2019)	-	-	0.2m* (RL49.1m)	1.9m (RL47.4m)
1 (JK2014)	-	2.3m** (RL35.9m)	2.7m** (RL35.5m)	-
2 (JK2014)	-	17.3m (RL20.7m)	3.8m (RL34.2m)	4.5m (RL33.5m)
3 (JK2014)	-	-	2.95m* (RL76.5m)	-
4 (JK2014)	-	4m (RL33.8m)	4.3m (RL33.5m) 6.6m (RL31.2m)	4.8m (RL33m) 10.4m (RL27.4m)
5 (JK2014)	-	-	4.2m** (RL33.6m)	-
6 (JK2014)	7.6m (RL29.9m)	-	9.2m (RL28.3m)	10m (RL27.5m)

* **tentative classification of upper section of the borehole estimated from auger drilling.**

** **tentative classification estimated from an augered borehole.**

tentative classification based on tactile assessment of recovered rock core.

4.2 Site Stability

We note that the existing fill batter slope over the western portion of the site is over-steep and is considered marginally stable. On-going near surface creep indicated by the uneven slope surfaces and the misaligned fence lining the western side of the car park is currently evident. We have inferred that the fill has been placed over a stepped sandstone cliff face and so deep seated ‘global’ instability affecting the full height of the fill batter slope would be less likely to occur where bedrock outcrops are present across the slope, or are inferred to be at shallow depth below the batter slope surface.

To manage the potential impact of fill batter slope instability, there are two options:

- Re-profile the batter slope to a flatter slope angle [no steeper than 1 Vertical (V) in 2 Horizontal (H)] and re-vegetate.
- From the toe of the fill batter slope, project a theoretical failure plane at an angle no steeper than 1V in 2H and locate structures east of this theoretical failure plane line, or extend footings below this theoretical failure plane line and suspended paved areas, sections of buildings etc between these footings.

With regard to the batter slope over the area of sewer pipe line works completed in 2016, we reiterate the comments provided in our report JK2016:

- The provided photographs indicate that the works were completed using a tracked excavator with the trench sides supported by trench boxes.
- The settlement of the fill around the concrete slab, presence of erosion rills and tension cracks indicates that the fill was, in all probability, not placed and compacted in a controlled manner. In this regard, we have not been provided with any details of backfill specification requirements or any site records of fill compaction tests.
- The maximum 35° fill batter slope formed as part of reinstatement works for the trench may also be regarded as over-steep, particularly as the fill does not appear to have been compacted. The presence of tension cracks will allow surface run-off to penetrate the fill which could lead to softening of the fill and a reduction in shear strength which could lead to slope instability. This would be exacerbated if water was 'trapped' in the fill and a hydrostatic pressure then developed, further reducing the shear strength of the backfill materials.

On the basis of the above, possible remediation works for the sewer backfill area may comprise one of the following options:

- Re-profile the batter slope to a flatter slope angle (no steeper than 27°) and re-vegetate.
- Excavate the backfill materials, replace in a controlled manner to an appropriate specification and form a flatter slope angle as noted above and re-vegetate.
- Leave the slope 'as is', re-vegetate and provide erosion protection measures.

However, we have no information to confirm if the above remediation works were completed.

Under existing conditions, and in accordance with the criteria given in Reference 1, the JK2016 report assessed the levels of risk to property to vary between 'acceptable' and 'tolerable', and the levels of risk to life to be 'acceptable'.

Provided the geotechnical advice provided in this report is adopted in full, then we consider that the stability of the site will be maintained and improved and risk levels reduced or maintained at an 'acceptable' level.

4.3 Demolition and Excavation

4.3.1 Dilapidation Surveys

Prior to demolition and excavation commencing, detailed dilapidation reports should be compiled on the neighbouring residences to the west that abut the western site boundary. Consideration may also be given to compiling a similar dilapidation report on Pallister House. In addition, Council may also require that dilapidation survey reports be completed on their assets lining the street frontages, i.e. the paved footpaths, roadways and kerbs and gutters.

The dilapidation survey reports can be used as a benchmark against which to set vibration limits for rock excavation, and for assessing possible future claims for damage arising from the works.

The respective owners of the adjoining properties should be asked to confirm in writing that the dilapidation survey report on their property presents a fair assessment of the existing conditions. As dilapidation survey reports are relied upon for the assessment of potential future damage claims, they must be carried out thoroughly with all defects rigorously described (i.e. defect type, defect location, crack width, crack length etc) and defects photographed where practical.

4.3.2 Demolition and Excavation Methods

The excavation recommendations provided below should be complemented by reference to the NSW Government "Code of Practice Excavation Work" dated January 2020.

The outline of the proposed Level 1 (the basement), Seniors Living North and South, Health and Care and Respite buildings are indicated on the attached Figure 2. To achieve design surface levels, excavations are expected to extend to a maximum depth of about 14.0m below existing surface levels.

Demolition and excavation will need to be carefully sequenced and completed in order to maintain the stability of the adjacent sections of existing buildings and structures within the site that will remain during the staged construction, the neighbouring buildings and structures and the fill batter slope over the western end of the site. This work will need to be completed using suitably experienced (and insured) contractors. In this regard, we note that the excavations may extend below the base of adjacent footings supporting existing buildings and structures. We assume that the buildings and structures have generally been founded on bedrock. However, this must be confirmed during demolition by excavating test pits in order to expose the existing footings and confirm the foundation materials. Based on inspection of these test pits by the structural and geotechnical engineers, the need and extent of underpinning, propping and/or wall strengthening measures can then be determined and detailed. Any underpins that will be supporting the soil profile will need to be designed in accordance with the advice provided in Section 4.4.3 below, to resist lateral loading.

We also reiterate the comments in Section 4.2 above in relation to the existing fill batter slope over the western portion of the site and the options to manage the potential impact of fill batter slope instability on the proposed development. During construction, plant, equipment or stockpiles of material must not operate and/or be located west of an exclusion zone defined by a theoretical failure plane line projected up from the toe of the fill batter slope at an angle no steeper than 1V in 2H. This exclusion zone must be clearly marked out on site. In addition, to prevent additional erosion and potential instability, any temporary or permanent surface run-off must not be discharged over the fill batter slope.

On-going monitoring of the fill batter slope and the existing surface levels close to the crest of the fill batter slope must be completed during the works by a nominated site staff member on a daily basis and after periods of heavy or prolonged rainfall. The inspection should check for signs of tension cracking within the surfaces lining the crest of the slope, leaning trees or fences and/or areas of disturbed fill batter slope surface.

The monitoring inspections must be formally documented and include the date of the inspection, weather conditions on, and immediately preceding, the inspection, and any comments/observations and photographs should also be provided. A copy must be provided to the geotechnical consultant for review. If there are concerns regarding stability, then the need for slope remediation works, re-location of the exclusion zone etc may need to be considered.

On the basis of the investigation results, following demolition, the proposed excavations will encounter the soil profile and penetrate weathered sandstone bedrock over the central and eastern portions of the proposed basement. Any topsoil or root affected soils should be stripped and separately stockpiled for re-use in landscape areas or appropriately disposed of as such soils are not suitable for re-use as engineered fill.

Tree root systems dry out the surrounding clayey soils and their removal will result in localised moisture recovery leading to swelling which may have a detrimental impact on the performance of any nearby buildings and paved surfaces founded/supported in the clayey soil profile within the site. Therefore, trees should only be removed where absolutely necessary and as soon as practicable, in order for the moisture content of the clayey subsoils to recover; ideally this would be years in advance of construction though this may not be practical for this site.

Due to the presence of poorly compacted fill, which may extend below Pallister House, we do not recommend the use of rock breakers during demolition or rock excavation in close proximity to the building due to the potential for transmission of vibrations which could cause damage, unless the building is founded on, or underpinned to, bedrock. Based on the results of the test pit inspections described above, underpinning of the building may be required. Similar concerns may apply in the unlikely event that the test pit investigations indicate that other existing buildings adjacent to the staged works are not founded on bedrock. In such instances, if underpinning of any footings not founded on bedrock is not carried out, we recommend that the removal of concrete floor slabs and footings be completed using a diamond saw followed by removal of the concrete pieces using a bucket attachment to the tracked excavator.

We expect the excavation of the soil profile and extremely weathered bedrock to be readily completed using bucket attachments to tracked excavators. We expect that excavation of low and higher strength bedrock will require small to medium size rock breakers and ripping attachments to the tracked excavators and possibly dozers with ripping tyne attachments. Rock saws may also be used to create ‘smooth’ finishes on cut faces and aid in detailed excavation of footings, services trenches, lift pits, etc.

When using the rock breakers, saws and ripping attachments, the resulting dust should be suppressed by spraying with water. Care will be required to control ground vibrations associated with the use of rock breakers during bedrock excavation, and further advice is provided in Section 4.3.3 below. Alternative excavation techniques to reduce vibrations and therefore reduce vibration monitoring could include using a rock grinder on the excavator, or a large excavator mounted rock saw to grid saw the bedrock into blocks that could then be removed using a ripping tyne attachment to the excavator, or locally by the use of drill and split techniques.

We also note that 'dropping' of large sections of existing structure during demolition should also be avoided in order to prevent the generation of potentially damaging vibrations.

4.3.3 Potential Vibration and Ground Surface Movement Risks

Due to the presence of poorly compacted surficial fill which may extend beyond the staged work areas and across 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 the adjacent sections of neighbouring buildings and structures that may not be founded in bedrock, and/or the fill batter slope over the western portion of the site.

There is a possibility that vibrations from excavation equipment and other site activities may cause damage to adjoining structures within or neighbouring the site if these adjoining structures are not founded on bedrock. Our preference is to underpin any adjacent structures to rock. This is not always practical or possible, however if it is not carried out, the client/contractor will be responsible for any damage that does occur. Where adjoining structures are founded on and/or underpinned to rock, the limit for vibrations provided below should be assessed by the structural engineer following review of the dilapidation reports.

Where rock breakers are used during demolition and to excavate bedrock, continuous quantitative vibration monitoring of the neighbouring buildings and structures to the west will be required, to confirm that the peak vibration velocity (Vi, max) falls within acceptable limits. Subject to review of the dilapidation reports described in Section 4.3.1 above, and assuming adjoining structures are founded and/or underpinned on bedrock, we tentatively recommend that the Vi, max does not exceed 5mm/sec during bedrock excavation using rock breakers, subject to confirmation by the structural engineer. We also recommend that consideration be given to similar vibration monitoring of the adjacent sections of hospital buildings that will remain during bedrock excavation using rock breakers. Subject to confirmation by the structural engineer, we tentatively recommend that Vi, max's do not exceed 3mm adjacent to Pallister House and 10mm/sec for the remaining hospital buildings.

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 the vibration limits recommended above will reduce the risk of vibration damage to the neighbouring and/or adjacent buildings and structures. However, these vibrations may still result in perceived discomfort or concern to occupants of the neighbouring buildings and/or the hospital buildings. If excessive vibrations are confirmed, it will be necessary to use lower energy equipment such as rock grinders, smaller rock breakers and/or use rock saw cuts (in conjunction with the use of rock breakers or ripping tynes) 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 potential amplification of vibrations.

4.4 HYDROGEOLOGICAL EVALUATION

No discernible seepage was recorded in the augered portions of the boreholes and seepage was not recorded emanating from the various outcrop faces within the site particularly over the south-western portion of the site. However, standing water levels were recorded in the majority of the groundwater monitoring wells at depths between 3.7m and 9.32m (approximate RL33.3m to RL41.7m), which indicates that there is the potential for groundwater seepage to be encountered in the basement excavation. The response zones of the wells extended through the bedrock profile and into the lower portion of the soil profile. On this basis we note the following:

- The wells are likely acting as sumps and collecting localised perched seepage at the soil-bedrock interface and localised seepage at discrete defects in the rock mass such as open joints or bedding partings, fractured bands etc. The ability for water to collect in the wells therefore being a function of the connectivity of the various defects in the rock mass, the quantity of water they contain and how many have been intersected by the well. That is, the varying water levels in the wells do not represent a 'true' groundwater level.
- The site is situated in an elevated area and surface levels stepped down to the south-west beyond the south-western corner of the site to a cliff face (about 25.0m height) that lined the northern side of the neighbouring Gore Creek Reserve. This topographic feature would be expected to drawdown water collecting in defects in the rock mass by allowing water flowing through intersecting defects to eventually discharge from the cliff face into the reserve. On this basis, an elevated groundwater level in the rock mass within the area of the proposed development would be very unlikely to develop as water in the defects would be expected to eventually flow down to the south-west.

Notwithstanding the above there is the potential for ephemeral seepage inflows to be encountered in the excavations, particularly after periods of heavy rain, close to the soil-rock interface and through defects within the rock mass. In general, we expect that inflows, to be relatively small and managed by conventional sump and pump techniques or gravity drainage. Inspection and monitoring of groundwater seepage during excavation is recommended and should form a routine part of the geotechnical rock cut face inspections described in Section 4.5.2 below, so that any unexpected conditions, which may be revealed can be incorporated into the drainage design. In the unlikely event that larger volumes or concentrated areas of seepage are encountered then additional dewatering measures such as additional sumps and pumps may be required and a contingency for this eventuality should be made in the contract documents. Any water collected from the excavation must not be discharged over the existing fill batter slope to the west as this could cause instability.

Any seepage through defects (joints, bedding partings etc) within the rock mass would be expected to reduce from initially higher seepage rates, as the water perched in defects seeps out relatively quickly. The seepage rate would then reduce as recharging water from the surface would be slow as the water infiltrates through the soil profile and rock mass via interconnecting defects. As noted above, locally seepage rates may increase during and following rainfall. Our expectation for this hydrogeological setting would be for a sump and pump system to adequately control seepage during construction and for a drained basement to be applicable.

We note that numerical modelling to estimate water ingress volumes was outside the agreed scope of the investigation. To make any prediction or estimation of groundwater volume ingress during construction would require numerical modelling which we can carry out, if commissioned.

4.5 Temporary Excavation Support and Retention

4.5.1 Temporary Batters and Retention

Temporary batter slopes through the sandy soil profile no steeper than 1 Vertical (V) in 1.5 Horizontal (H) are considered to be appropriate, though some surficial instability could still occur. However, steeper temporary batters of 1V in 1H will be appropriate in areas of clayey soils and extremely weathered bedrock. These temporary batters are expected to be achievable within the site geometry. This also assumes that adjoining buildings and structures within the site are founded on, or, where necessary, have been underpinned to bedrock of at least low strength. Stockpiles of construction materials, excavated materials, etc should be kept well clear of the batter crests to avoid surcharging the slopes.

Some instability of temporary soil batters may occur at, or below, the level of any seepage, especially after rain periods, and sand bagging may be required to stabilise the lower portion of these batters. Conventional retaining walls may be constructed at the base of the batters and subsequently backfilled. It is likely that the retaining walls could be founded at the crest of rock cut faces, subject to confirmation by geotechnical inspections as described below.

4.5.2 Sandstone Cut Face Stability

Competent sandstone bedrock (low or higher strength) may be cut vertically, subject to confirmation by geotechnical inspection at maximum 1.5m depth intervals during excavation to check for any adverse defects that may require stabilisation, as described further below.

For walls and/or underpins founded at the crest of excavation faces, lateral restraint may be provided by starter bars drilled and grouted to a depth of at least 0.5m into the sandstone bedrock. The starter bars should be installed at a downward angle into the rock face and be provided with a vertical cogged length. Where cross bedded units within the sandstone bedrock are identified during geotechnical inspections and slope down into the excavation, then the starter bars may have to be extended further into the bedrock to stabilise the potentially unstable cross bedded units.

The presence of potentially unstable wedges, clay seams and extremely weathered seams within the sandstone bedrock may adversely affect the stability of the cut faces, footings and/or underpins located close to the crests of cut faces. Such features may require shotcreting and bolting (subject to permission from the neighbours). However, in some instances the prompt construction of full height retaining walls may remove the need for use of shotcrete and rock bolts (provided there is no need to access the void such as for the application of waterproofing); this could only be confirmed following geotechnical inspection. Provision should be made in the contract documents (budget and programme) for such inspections and stabilisation measures.

4.5.3 Retention Design Parameters

The following earth pressure coefficients and soil parameters may be adopted for the design of retaining walls, underpins supporting a soil profile or landscape walls:

- For design of retaining walls that will be temporarily propped, backfilled and permanently supported by the structure and any underpins supporting a soil profile, we recommend the use of a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient (k_o) of 0.5 for the retained profile, assuming a horizontal backfill surface.
- Where some minor movements of retaining walls may be tolerated (e.g. landscape walls), 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 soil profile, assuming a horizontal backfill surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the retained profile.
- Any surcharge affecting the walls (e.g. nearby footings, compaction stresses, sloping retained surfaces, construction loads etc) should be allowed for in the design using the appropriate earth pressure coefficient from above.
- 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 a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system.
- Any underpins supporting a soil profile must be designed as permanently drained and PVC pipes should be installed at nominal 1.2m horizontal spacing just above the adjacent floor level or the bedrock surface. The end of the pipe penetrating the retained soils 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.
- Lateral restraint of landscape walls founded in the soil profile below adjacent surface levels may be provided by the passive pressure of the soil below these levels. A 'passive' earth pressure coefficient, K_p, of 3 may be adopted, using a triangular pressure distribution and provided a Factor of Safety of at least 2 is used in order to reduce the high deflections that are associated with achieving a full passive case. Localised excavations in front of the walls e.g. for buried services etc must also be taken into account in the design. Where these footings are on bedrock, the lateral resistance may be calculated using a friction angle of 35° for the footing/bedrock interface.
- For retaining walls keyed into the sandstone bedrock below bulk excavation level and/or adjacent surface levels, an allowable lateral stress of 200kPa may be adopted for sandstone of at least low strength. The socket length should commence below the base of any nearby excavations such as for service trenches, and also below a nominal allowance for over-excavation or fracturing during excavation.
- Any permanent rock bolts or dowels supporting retaining wall footings at the crests of bedrock cut faces should be designed for an allowable bond stress of 200kPa assuming they are installed into sandstone bedrock of at least low strength. Permanent rock bolts will need to be designed with due regard for long term corrosion protection, i.e. fully grouted, hot dipped galvanised and provided with a sacrificial

thickness or using stainless steel bars. Rock bolts extending across the site boundaries will require permission from the neighbouring property owners and in accordance with the Design and Building Practitioners Act (2020), easements for temporary ground anchors may also be required for this type of development.

4.6 Soil Reactivity and Footing Design

4.6.1 Soil Reactivity

The results of the investigation have indicating that where present, the residual clay soils have a slight potential for shrink/swell reactivity with changes in moisture content, i.e. similar to those expected for a Class ‘S’ site as defined in AS 2870 – 2011, “Residential Slabs and Footings” (maximum 20mm). The clayey fill is also expected to be slightly reactive.

As noted in Section 4.3.2, the removal of tree root systems will dry out clay soils resulting in localised moisture recovery leading to swelling which may have a detrimental impact on the performance of any nearby buildings and paved surfaces founded/supported in the clayey soil profile. These effects can be managed by removing the trees well in advance of construction but this is not expected to be feasible for this site.

The residual clays, where present, have a limited thickness and encountered below existing predominantly granular (non-reactive) fill; only limited amounts of clay fill were encountered in our various phases of investigation. The base of the residual clays and bands of clay fill were encountered well above 1.8m depth (the extent of moisture content changes in the Sydney region). On this basis, any reactive soil movements would be expected to be well below 20mm and more likely to be of the order of about 5mm. Provided any new fill placed to raise surface levels is inert (i.e. non-reactive) and considering the depth of the proposed cuts and limited thickness and extent of the slightly reactive clay soils, we consider that there will be negligible impact on the predicted reactive soil movements.

Where trees are removed, we would expect any associated swelling associated with localised moisture content recovery would be less than 20mm.

4.6.2 Footing Design

Based on the investigation results, sandstone bedrock is expected to be exposed over the majority of the bulk excavation level , or is otherwise expected to be present at depths of less than 1.0m below bulk excavation level over the western portion of the basement. On this basis, we expect that pad or strip footings founded in the weathered sandstone bedrock will be generally be suitable. However, over the western portion of the proposed development pile footings will be required.

Bored piles could be used, but would most likely require sacrificial liners to support the sandy soil profile. We also note that there is a greater likelihood of collapse of the sides of bored piles, particularly if seepage close to bedrock is encountered, and could cause adjacent ground surface movements which extend beyond the immediate site area. In addition, the allowable bearing pressure (ABP) of the bedrock would be limited to those applicable for a Class IV sandstone (see below).

If bored piles are considered, a site trial should be completed away from any critical areas to confirm their suitability. Alternatively, grout injected (continuous flight auger [CFA]) piles could be used. The piles would need to penetrate bedrock and sand mining could occur when forming rock sockets using conventional CFA piling techniques and/or decompression if any groundwater is encountered. A site trial in the centre of the site would need to be undertaken under the direction of a geotechnical engineer to assess potential sand mining/decompression as this could detrimentally impact adjacent buildings and structures if they are not founded on competent bedrock. Alternatively, sand mining/decompression effects would be satisfactory controlled by using double rotary CFA piles, which includes a casing system to support the soil being installed concurrently with the auger. However, considering the expected limited area of the basement where bedrock is a maximum of 1.0m below bulk excavation level the use of CFA piles is unlikely to be economic.

Based on the preliminary engineering classification presented in Section 4.1 above, we note the following:

- Over the western portion of the site, the sandstone is variable, ranging between Class V and Class III on first contact. In addition, the footings are likely to be close to a step down in the bedrock surface where the proposed buildings are close to the existing fill batter slope.
- Over the central and eastern portion of the site, Class II or better sandstone is expected at bulk excavation level.
- Over the area of the proposed 'Respite' Building, the sandstone is expected to be at a maximum depth of about 1.4m below existing surface levels, and is variable, ranging between Class V and Class III on first contact. In addition, the footings are likely to be close to an eastward step down in the bedrock surface.

The following allowable bearing pressures (ABPs) may be adopted for the various sandstone Classes:

- Class V sandstone: 1MPa; close to a bedrock cut face or outcrop face, a reduced ABP of 500kPa applies.
- Class IV sandstone: 2MPa.
- Class III sandstone: 3.5MPa.
- Class II (or better): 6MPa

For footings founded in Class IV or better sandstone close to the crest of bedrock outcrop/cut faces, we recommend a reduced allowable bearing pressure of 1MPa.

For pile footings, we recommend a maximum ABP of 3.5MPa be adopted for Class III or better sandstone. Further advice from the piling contractor should be sought in regard to the most suitable piling rig to form the footings socketed into bedrock. If bored piles are adopted, the footings would require the use of small but powerful pile drilling plant with rock augers fitted with tungsten carbide teeth rather than using pendulum auger attachments to tracked excavators.

The above ABP's will need to be confirmed by geotechnical inspection of all footing excavations and, where ABP's in excess of 3.5MPa are adopted, spoon testing of at least 50% of the pad or strip footing excavations.

This is of particular importance where footings or underpins are founded on or near the crests of rock cut faces, as outlined in Section 4.5.2 above.

All pad and strip footings, pile footings and any underpins should be excavated/drilled, inspected, spoon tested (where appropriate) and poured with minimal delay. All footings should be free from all loose or softened materials prior to placement of the reinforcement cage and pouring. If water ponds in the base of the footings, they should be pumped dry and then re-excavated to remove all loose and any water softened materials.

We note that little recovery of rock chips is obtained from CFA pile holes (if selected) and so determination of bedrock depths and strength would be based on witnessing drilling of CFA piles by a geotechnical engineer together with reference to the borehole logs and torque readings provided by the piling rig operator. We also reiterate the warnings in Section 4.4.1 above, regarding sand mining or decompression when forming conventional CFA piles. The CFA piles will need to be certified by the piling contractor.

4.7 Floor Slabs and External Paved Areas

Slab-on-grade construction is theoretically feasible for floor slabs over soil subgrade areas, expected towards the western end of the basement. However, it would require removing and re-compacting the existing fill. Also, the basement slab would be in contact with a mix of existing fill, residual clays and weathered bedrock resulting in differential deflections, which may be exacerbated by reactive soil movements outlined in Section 4.6.1 above, and potential cracking of the floor slab.

Over soil subgrade areas our recommendation is to suspend the floor slab between footings founded in bedrock and void formers should be provided to accommodate potential reactive movements (maximum 20mm). The need for void formers can be determined by geotechnical inspection of the soil subgrade areas.

We note that the ‘Respite’ Building will be partially suspended over the slope to the east and the subgrade is expected to comprise a mix of bedrock and fill. We recommend that the entire floor slab be suspended between footings founded in bedrock over the soil subgrade and the slope.

For floor slabs suspended over soil subgrade areas, the subgrade preparation would comprise the removal of any topsoil and/or any soil containing organics, completion of the bulk excavation and the nominal tracking of ‘formwork fill’ to the required subgrade level.

Where sections of on-grade floor slabs are expected to directly overly sandstone bedrock, we recommend that under-floor drainage be provided. The under-floor drainage should comprise perimeter drains comprising a geofabric wrapped perforated drainage pipes in a gravel filter. Additional drains below the floor slabs in the basement (orientated north-south) should also be provided. The under-floor drainage must connect to the stormwater system for controlled disposal.

The floor slabs over the bedrock subgrade should also be provided with at least a 20mm to 30mm thick 'clean' sand bedding layer to provide a de-bonding layer to limit the potential for shrinkage cracking or curling of the slabs.

The basement slab (unless suspended) should be designed to be separated from all walls, columns, footings, etc to permit relative movements (i.e. 'floating'). The concrete floor slab should be provided with effective shear connection of joints by using dowels or keys. Additional dowels may be required at the interface between bedrock and soil.

For external paved areas (including new access roads and driveways) slab-on-grade-construction is feasible, although it will be difficult to complete high quality earthworks over small site areas. Even if the earthworks are completed in accordance with the recommendations in Section 4.8.1 below, it is likely there will be a significant variation in subgrade conditions, resulting in some degree of differential deflection, and potential cracking of the external paved areas. If this cannot be accepted, the external paved areas should be suspended from the bedrock.

On-grade concrete pavements (if selected) should be provided with effective shear connection at joints by using dowels or keys.

Sub-soil drains should be provided along the perimeter of external pavements (where constructed on-grade), with inverts not less than 0.2m below design subgrade level. The drainage trenches should be excavated with a longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The pavement subgrade should be graded to promote water flow or infiltration towards sub-soil drains.

A new kerb line around the crest of the existing fill batter slope over the western portion of the site should be formed to prevent surface run-off discharging down the fill batter slope, which could otherwise lead to erosion and possibly trigger instability.

4.8 Earthworks

The following earthworks recommendations should be complemented by reference to AS3798-2007 "Guidelines on Earthworks for Commercial and Residential Developments".

4.8.1 Subgrade Preparation

Prior to construction of any external on-grade paved areas, the soil subgrade should be prepared as follows:

- Following removal of existing paved surfaces, and stripping of topsoil and/or any root affected soils, any remaining poorly compacted fill should be removed, as directed by the geotechnical engineer. The poorly compacted fill may then be used to reinstate the area back to the design surface level but would need to compacted as engineered fill as outlined in Section 4.8.3 below. The poorly compacted fill must be removed and re-compacted in layers in order to achieve effective compaction. Compacting the poorly compacted fill in-situ will not achieve effective compaction and long-term settlements can be expected, particularly where thicker areas of existing fill are present.

- Proof roll the soil subgrade with a minimum 10 tonne deadweight smooth drum roller, using the static (non-vibration) mode, following thorough moistening of the sand subgrade.
- To assist with proof rolling, we recommend that a thin layer of road base (at least 75mm thick) be placed over the sand subgrade to improve near surface compaction and prevent shearing during rolling.
- Sections of clay subgrade that contain shrinkage cracks should be lightly watered and rolled until the shrinkage cracks disappear.
- Proof rolling should be closely monitored by the geotechnical engineer 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.

If it is preferred not to carry out the above subgrade preparation in areas of a thicker existing fill subgrade under areas of proposed pavements and/or other paved areas, then the subgrade would need to be improved by the use of a ‘bridging’ layer comprising ‘over-size’ durable and coarse rock (at least 75mm size) possibly in conjunction with a geogrid reinforcement layer. Details of the bridging layer, if required, must be provided by the geotechnical engineer following the proof rolling inspection.

4.8.2 Subgrade Drainage During Construction

In places, the subgrade will comprise clay soils. The clays may be found to be unstable if proper site drainage is not implemented during construction. It is therefore important to provide good drainage in order to promote run-off and reduce ponding. Earthworks platforms should be graded to maintain cross-falls during construction. If the clays are exposed to periods of rainfall, softening may result and site trafficability will be poor. If softening occurs, the subgrade should be over-excavated to below the depth of moisture softening. The material removed should be replaced with engineered fill. Trafficability may be improved by the use of a sacrificial surface layer of crushed demolition rubble.

4.8.3 Engineered Fill

For treatment of poor subgrade areas to replace poorly compacted fill and for raising of any site surface levels (if required), engineered fill should be used.

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 soils and bedrock may be used as engineered fill. Engineered fill should be compacted using the above-mentioned roller in layers not exceeding 100mm loose thickness to a density between 98% and 102% of Standard Maximum Dry Density (SMDD), and within 2% of their Standard Optimum Moisture Content (SOMC). The density may be reduced to 95% of SMDD in soft landscaped areas, where the designer considers that settlements are not critical.

Backfill to retaining walls should also comprise engineered fill. Well graded granular materials such as crushed sandstone and demolition rubble would be suitable for this purpose. This granular fill should be free of deleterious substances and should have a maximum particle size not exceeding 40mm. Such fill should be compacted in horizontal layers as above using a hand-held plate compactor (e.g. whacker packer). Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

Density tests should be carried out at the frequencies outlined in AS3798 (Table 8.1) for the volume of fill involved. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking and retesting. The Geotechnical Testing Authority should be directly engaged by the client or their representative and not by the earthworks contractor.

As an alternative, single sized granular material (or 'no fines' gravel) may be used as backfill to retaining walls and this would also act as the drainage behind the wall and would only require nominal compaction (with no compaction testing). The drainage material should be wrapped in a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion. Further, retaining wall backfill should be provided with a clay cap of at least 0.3m thickness at surface level to reduce the likelihood of surface water entering the backfill and surcharging the retaining walls.

4.8.4 Pavement Materials

Concrete pavements, if selected, should have a sub-base layer of at least 100mm thickness of crushed rock complying with the latest revision of Transport for NSW QA specification 3051 unbound base material which is compacted using a heavy roller to at least 98% of Modified Maximum Dry Density (MMDD).

For the pavement materials we recommend that:

- All base course materials for flexible pavements comprise DGB20 in accordance with the latest revision of Transport for NSW QA specification 3051. The DGB20 should be compacted in a single layer using a large smooth drum roller to at least 98% of MMDD.
- All sub-base materials for flexible and rigid pavements should comprise DGS40, DGS20 or DGB20 in accordance with the latest revision of Transport for NSW QA specification 3051. Recycled materials may be used provided they conform to the specification requirements. However, recycled materials can be self-cementing, which can then cause reflective cracking through the pavement surface, which would then require crack sealing. While this may be an aesthetic issue, it would not necessarily cause significant reduction in the pavement life provided the cracks are appropriately sealed. The sub-base should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to at least 95% of MMDD.

For all of the above pavement construction materials, adequate moisture conditioning to within 2% of MOMC should be provided during placement so as to reduce the potential for material breakdown during compaction.

Density tests should be carried out on the granular pavements materials to confirm the above specifications are achieved. The frequency of density testing should be as per the requirements outlined in AS3798 and Level 2 testing of fill compaction is recommended. The Geotechnical Testing Authority should be directly engaged by the client or their representative and not by the earthworks contractor.

4.9 Soil Aggression

Based on the advice provided in AS2159-2009 “Piling Design and Installation” for corrosion protection and durability and in AS3600-2009 “Concrete Structures” we note that the laboratory chemical test results have indicated that the following Exposure Classifications are applicable:

- ‘Moderate’ (based on Table 6.4.2 (C), in AS2159-2009), and
- B1 (based on Table 4.8.1 in AS3600-2009).

4.10 Working Platform

Tracked piling rigs (if used) will 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 the contractor for the specific equipment 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’. Due to the close proximity to the slopes over the western portion of the site, the design of the working platforms may need to be based on first principles and geotechnical design software such as SLOPE/W, PLAXIS etc will need to be used to assess global stability.

Following demolition, the subgrade will need to be proof rolled in accordance with the advice provided in Section 4.8 above.

We recommend that the platform material be formed using select fill of 40mm to 70mm particle size, comprising strong durable rock or recycled concrete with a minimum compressive strength of 25MPa. Such fill will be suitable for use with geogrids, if required. For particle sizes above 40mm, a method specification would need to be determined and strictly directed by the project geotechnical engineer. This will be of particular importance if geogrids are used as an appropriate grading will be required to engage the apertures in the geogrid.

Once the piling works from the existing surface level are completed the platform materials will need to be removed. If a platform is required at BEL the platform may be left in place to improve trafficability and act as ‘formwork fill’ for the suspended basement floor slab. Where the select fill is removed, then it would require a waste classification before removal from site; refer to JKE reports for further advice.

4.11 Further Geotechnical Input

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.

- Dilapidation reports of adjoining buildings and structures.
- Inspection of test pits exposing footings supporting adjacent buildings within the site that will remain (permanently, or for a limited period through the staged construction).
- Continuous vibration monitoring during use of rock breakers during demolition and for excavation of bedrock.
- Inspection of rock cut faces at maximum 1.5m depth intervals and directing stabilisation measures, if required.
- Monitoring of groundwater seepage into bulk excavations.
- Witnessing installation of pile footings.
- Inspection of footing excavations.
- Witnessing proof rolling.
- Advice on treatment of poor subgrade areas using bridging layers.
- Density testing of engineered fill.
- Working platform design, if required.

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

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

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of 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.

Reference 1:	Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.

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, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Hospital Redevelopment
Location: 97-115 River Road, Greenwich, NSW

Report No.: 32507R2 - A
Report Date: 25/10/2021
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
105	2.20 - 2.60	8.6	-	-	-	-
105	3.00 - 3.10	4.5	-	-	-	-
106	1.00 - 1.30	4.1	-	-	-	-
106	0.50 - 0.70	11.8	25	13	12	4.5
107	1.30 - 1.60	6.0	-	-	-	-
108	1.00 - 1.30	3.4	-	-	-	-
109	0.80 - 0.95	10.2	27	11	16	5.0
109	1.00 - 1.20	3.8	-	-	-	-

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 21/10/2021.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
This document shall not be reproduced except
In full without approval of the laboratory. Results relate only to
the items tested or sampled.

25/10/2021
Authorised Signature / Date
(D. Treweek)

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: TSA

Ref No: 32507R2

Project: Proposed Hospital Redevelopment

Report: A

Location: 97-115 River Road, GREENWICH, NSW

Report Date: 1/10/21

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BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH105	3.26 - 3.30	0.6	12	A
	3.79 - 3.82	0.5	10	A
	4.14 - 4.18	0.9	18	A
	4.71 - 4.76	0.7	14	A
	5.34 - 5.38	0.6	12	A
	5.78 - 5.82	0.9	18	A
	6.22 - 6.25	1.1	22	A
	6.61 - 6.65	1.4	28	A
	7.10 - 7.14	0.7	14	A
	7.52 - 7.56	0.7	14	A
	7.71 - 7.74	1.6	32	A
BH106	1.36 - 1.39	1.4	28	A
	1.78 - 1.82	1.6	32	A
	2.10 - 2.13	1.6	32	A
	2.91 - 2.94	1.7	34	A
	3.21 - 3.24	1.8	36	A
	3.77 - 3.81	1.9	38	A
	4.16 - 4.18	1.6	32	A
	4.62 - 4.66	1.5	30	A
	5.35 - 5.38	1	20	A
	5.74 - 5.78	1	20	A
	6.06 - 6.09	1	20	A
	6.82 - 6.85	0.9	18	A
	7.15 - 7.18	0.7	14	A
	7.64 - 7.68	0.9	18	A

NOTE: SEE PAGE 6

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	TSA	**Ref No:**	32507R2
Project:	Proposed Hospital Redevelopment	**Report:**	A
Location:	97-115 River Road, GREENWICH, NSW	**Report Date:**	1/10/21

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BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH106	8.24 - 8.27	0.6	12	A
	8.83 - 8.86	0.7	14	A
	9.15 - 9.19	0.9	18	A
	9.69 - 9.73	0.8	16	A
	10.24 - 10.27	1.3	26	A
	10.79 - 10.82	1.4	28	A
	11.08 - 11.12	1.5	30	A
	11.73 - 11.76	1.1	22	A
	12.05 - 12.08	1.8	36	A
	12.42 - 12.45	1.4	28	A
BH107	1.87 - 1.90	0.4	8	A
	2.09 - 2.13	0.6	12	A
	2.67 - 2.69	1.1	22	A
	3.07 - 3.10	2	40	A
	3.77 - 3.81	1.5	30	A
	4.22 - 4.26	1.8	36	A
	4.68 - 4.71	2	40	A
	5.39 - 5.42	1.9	38	A
	5.85 - 5.89	1.9	38	A
	6.33 - 6.38	1.3	26	A
	6.84 - 6.88	1.7	34	A
	7.10 - 7.13	1.8	36	A
	7.59 - 7.62	1.2	24	A
	8.24 - 8.27	1.5	30	A
	8.63 - 8.67	1.2	24	A

NOTE: SEE PAGE 6

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: TSA **Ref No:** 32507R2

Project: Proposed Hospital Redevelopment **Report:** A

Location: 97-115 River Road, GREENWICH, NSW **Report Date:** 1/10/21

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BOREHOLE NUMBER	DEPTH (m)	$I_{S(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH107	8.83 - 8.86	0.3	6	A
	9.07 - 9.10	1.7	34	A
	9.76 - 9.79	1.2	24	A
	10.15 - 10.18	1.7	34	A
	10.66 - 10.69	1.1	22	A
	11.36 - 11.40	1.3	26	A
	11.85 - 11.89	1.7	34	A
	12.20 - 12.23	1.5	30	A
	12.62 - 12.65	1.8	36	A
	13.08 - 13.11	1.9	38	A
	13.75 - 13.79	1.4	28	A
	14.32 - 14.35	1.5	30	A
	14.81 - 14.84	1.7	34	A
BH108	1.63 - 1.66	1.8	36	A
	2.10 - 2.13	1.1	22	A
	2.62 - 2.65	1	20	A
	3.25 - 3.28	1.1	22	A
	3.76 - 3.79	1.7	34	A
	4.06 - 4.09	1.6	32	A
	4.77 - 4.81	0.7	14	A
	5.18 - 5.21	1.5	30	A
	5.73 - 5.75	1.1	22	A
	6.25 - 6.28	0.9	18	A
	6.72 - 6.76	1	20	A
	7.34 - 7.37	1.4	28	A

NOTE: SEE PAGE 6

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	TSA	**Ref No:**	32507R2
Project:	Proposed Hospital Redevelopment	**Report:**	A
Location:	97-115 River Road, GREENWICH, NSW	**Report Date:**	1/10/21

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BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH108	7.79 - 7.82	1.3	26	A
	8.15 - 8.18	2	40	A
	8.74 - 8.77	1.3	26	A
	9.13 - 9.16	1.1	22	A
	9.60 - 9.63	1.3	26	A
	10.26 - 10.30	1.4	28	A
	10.65 - 10.69	1.5	30	A
	11.22 - 11.25	2.4	48	A
	11.64 - 11.68	1.9	38	A
	12.21 - 12.26	1.5	30	A
	12.60 - 12.64	1.7	34	A
	13.26 - 13.31	1.4	28	A
	13.69 - 13.73	1.7	34	A
BH109	1.60 - 1.63	1.2	24	A
	2.07 - 2.11	1.7	34	A
	2.71 - 2.74	0.8	16	A
	3.10 - 3.14	0.9	18	A
	3.74 - 3.78	0.8	16	A
	4.20 - 4.24	0.8	16	A
	4.59 - 4.63	0.7	14	A
	5.13 - 5.17	1.1	22	A
	5.65 - 5.69	0.8	16	A
	6.35 - 6.38	1.5	30	A
	6.81 - 6.85	1.1	22	A
	7.07 - 7.11	0.8	16	A

NOTE: SEE PAGE 6

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: TSA

Ref No: 32507R2

Project: Proposed Hospital Redevelopment

Report: A

Location: 97-115 River Road, GREENWICH, NSW

Report Date: 1/10/21

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BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH109	7.75 - 7.78	0.8	16	A
	8.17 - 8.20	1	20	A
	8.82 - 8.85	1.3	26	A
	9.14 - 9.18	1.2	24	A
	9.74 - 9.77	0.6	12	A
	10.08 - 10.11	1.5	30	A
	10.65 - 10.69	1.1	22	A
	11.25 - 11.29	0.3	6	A
	11.81 - 11.84	1	20	A
	12.05 - 12.08	1.3	26	A
	12.38 - 12.41	1.4	28	A
BH110	1.36 - 1.39	0.8	16	A
	1.92 - 1.95	0.9	18	A
	2.15 - 2.18	0.4	8	A
	2.74 - 2.77	1.1	22	A
	3.24 - 3.27	0.3	6	A
	3.59 - 3.63	0.4	8	A
	4.31 - 4.34	0.6	12	A
	4.96 - 5.00	1.1	22	A
	5.09 - 5.12	0.9	18	A
	5.71 - 5.74	0.9	18	A
	6.31 - 6.35	1	20	A
	6.86 - 6.89	1.6	32	A
	7.16 - 7.19	1	20	A
	7.75 - 7.78	1.1	22	A

NOTE: SEE PAGE 6

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	TSA	**Ref No:**	32507R2
Project:	Proposed Hospital Redevelopment	**Report:**	A
Location:	97-115 River Road, GREENWICH, NSW	**Report Date:**	1/10/21

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BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH110	8.08 - 8.11	1	20	A
	8.75 - 8.79	1.4	28	A
	9.09 - 9.12	0.3	6	A
	9.71 - 9.74	0.3	6	A
	10.10 - 10.14	0.9	18	A
	10.79 - 10.82	1.1	22	A
	11.22 - 11.25	1.4	28	A
	11.96 - 12.00	1.4	28	A
	12.08 - 12.12	1.3	26	A
	12.66 - 12.69	1.7	34	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
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 based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{S(50)}$.

SOURCE: http://www.whereis.com/

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:

SITE LOCATION PLAN

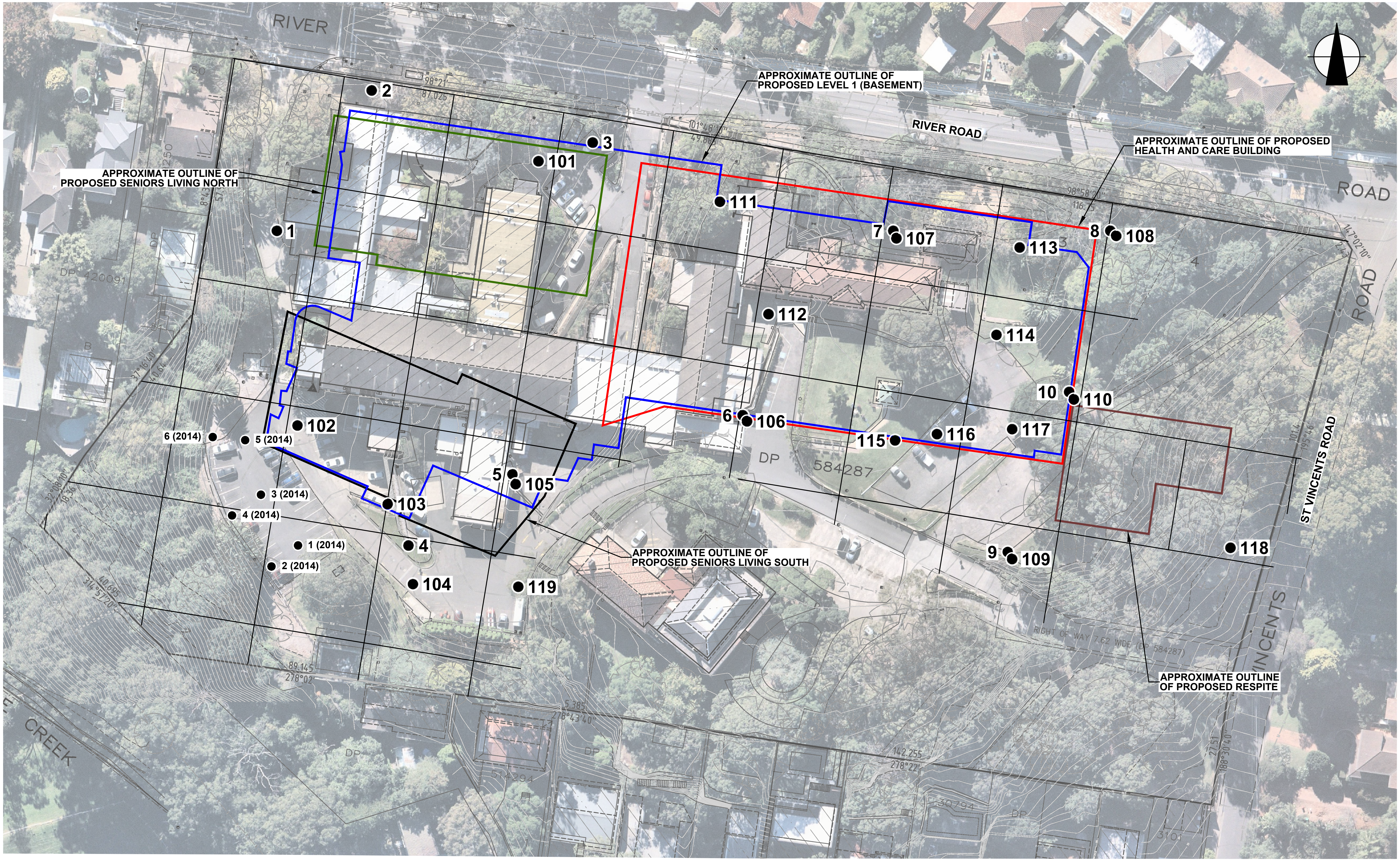
Location: 95-115 RIVER ROAD,
GREENWICH, NSW

Report No: 32507R2

Figure No: 1

JKGeotechnics

PLOT DATE: 21/10/2021 12:12:52 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32507R GREENWICH\CAD\32507R2\32507R2.DWG



NOTES:

1. BOREHOLES 1 TO 10 ARE FROM OUR PREVIOUS INVESTIGATION IN 2019.
2. BOREHOLES 1 (2014) TO 6 (2014) ARE FROM OUR PREVIOUS INVESTIGATION IN 2014.
3. BOREHOLES 105 TO 110 ARE FROM OUR CURRENT INVESTIGATION.
4. BOREHOLES 101, 102, 104 AND 119 FROM JKE INVESTIGATION (WITH ROCK CORE SAMPLES).
5. BOREHOLES 103 AND 111 TO 118 SEE JKE REPORT.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 8 16 24 32 40

SCALE 1:800 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN**

Location: 95-115 RIVER ROAD,
GREENWICH, NSW

Report No: 32507R2

Figure No: 2

JKGeotechnics

PLOT DATE: 21/03/2022 10:23:17 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32507R GREENWICH\CAD\32507R2\32507R2.DWG

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.

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 1726:2017 *'Geotechnical Site Investigations'*. 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 soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

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 (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	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 fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

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 shrink-swell behaviour, 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 methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock 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 a large 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. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, 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 limited 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 cuttings. 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 assessed 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. 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, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, 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 NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom 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 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

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 63.5kg 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.

A modification to the SPT 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.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm 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 a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm 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. The CPT does not provide soil sample recovery.

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. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- 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 CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT 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.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

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 reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised 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 assess 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 Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. 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.

Reasonable 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.
- Details of the development that the Company could not reasonably be expected to anticipate.

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 rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

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

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 and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

▩	FILL
≋	TOPSOIL
▨	CLAY (CL, CI, CH)
▥	SILT (ML, MH)
░	SAND (SP, SW)
◌	GRAVEL (GP, GW)
▨	SANDY CLAY (CL, CI, CH)
▨	SILTY CLAY (CL, CI, CH)
▨	CLAYEY SAND (SC)
░	SILTY SAND (SM)
▨	GRAVELLY CLAY (CL, CI, CH)
▨	CLAYEY GRAVEL (GC)
▥	SANDY SILT (ML, MH)
ψ	PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

◌	CONGLOMERATE
░	SANDSTONE
▤	SHALE/MUDSTONE
▥	SILTSTONE
▤	CLAYSTONE
■	COAL
▥	LAMINITE
▦	LIMESTONE
≋	PHYLLITE, SCHIST
▤	TUFF
⋰	GRANITE, GABBRO
+	DOLERITE, DIORITE
⋁	BASALT, ANDESITE
≈	QUARTZITE

OTHER MATERIALS

▦	BRICKS OR PAVERS
△	CONCRETE
■	ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory **Classification** Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\, D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

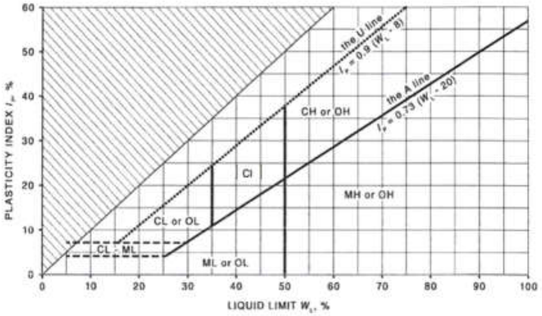
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition		
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.		
	—C—	Extent of borehole/test pit collapse shortly after drilling/excavation.		
	▶—	Groundwater seepage into borehole or test pit noted during drilling or excavation.		
Samples	ES	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 analysis.		
	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. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	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° 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 (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.		
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.		
	w < PL	Moisture content estimated to be less than plastic limit.		
	w ≈ LL	Moisture content estimated to be near liquid limit.		
	w > LL	Moisture content estimated to be wet of liquid limit.		
(Coarse Grained 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 ≤ 25kPa.		
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.		
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.		
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.		
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.		
	Hd	HARD – unconfined compressive strength > 400kPa.		
	Fr	FRIABLE – strength not attainable, soil crumbles.		
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.		
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 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		

Log Column	Symbol	Definition
Remarks	‘V’ bit	Hardened steel ‘V’ shaped bit.
	‘TC’ bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as: RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock. EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock. ALLUVIAL – soil deposited by creeks and rivers. ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents. MARINE – soil deposited in a marine environment. AEOLIAN – soil carried and deposited by wind. COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description ‘slopewash’ is used for thinner surficial deposits. LITTORAL – beach deposited soil.

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered		XW		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately Weathered		MW		The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

APPENDIX A

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, ATTERBERG LIMIT AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Hospital Redevelopment
Location: 97-115 River Road, Greenwich, NSW

Ref No: 32507R
Report: A
Report Date: 8/08/2019
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	1.50 - 1.95	10.5	35	17	18	6
2	1.50 - 1.95	13.1	29	16	13	5

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 30/07/2019.
- Sampled and supplied by client. Samples tested as received.

08/08/2019

Authorised Signature / Date
(D. Treweek)

NATA

NATA Accredited Laboratory Number:1327

115 Wicks Road
Macquarie Park NSW 2113
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SOIL TEST SERVICES

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Hospital Redevelopment
Location: 97-115 River Road, Greenwich, NSW

Ref No: 32507R
Report: B
Report Date: 5/08/2019
Page 1 of 4

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	4.34 - 4.38	0.7	14
	4.80 - 4.83	0.4	8
	5.21 - 5.24	1.0	20
	5.72 - 5.75	1.0	20
	6.32 - 6.35	1.2	24
	6.72 - 6.75	1.3	26
	7.15 - 7.18	1.0	20
3	1.32 - 1.35	1.1	22
	1.74 - 1.77	0.2	4
	2.15 - 2.18	0.3	6
	2.94 - 2.97	0.2	4
	3.06 - 3.09	0.9	18
	3.42 - 3.46	0.7	14
	4.07 - 4.10	0.4	8
	4.38 - 4.42	2.2	44
	5.09 - 5.12	1.3	26
	5.61 - 5.65	1.5	30
	6.23 - 6.26	1.5	30
	6.77 - 6.81	2.0	40
	7.09 - 7.12	1.6	32
4	3.12 - 3.16	0.2	4
	3.83 - 3.87	0.3	6
	4.25 - 4.28	1.0	20
	4.70 - 4.73	0.7	14
	5.20 - 5.24	1.0	20

NOTES: See Page 4 of 4

115 Wicks Road
Macquarie Park NSW 2113
Telephone: 02 9888 5000
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SOIL TEST SERVICES

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Hospital Redevelopment
Location: 97-115 River Road, Greenwich, NSW
Ref No: 32507R
Report: B
Report Date: 5/08/2019
Page 2 of 4

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
4	5.76 - 5.79	1.8	36
	6.17 - 6.21	0.7	14
	6.68 - 6.72	1.7	34
	7.16 - 7.19	1.6	32
6	1.34 - 1.37	1.3	26
	1.72 - 1.75	1.7	34
	2.17 - 2.20	2.0	40
	2.60 - 2.63	1.1	22
	3.29 - 3.32	1.9	38
	3.64 - 3.66	0.7	14
	4.07 - 4.10	1.8	36
	4.47 - 4.51	1.4	28
	5.27 - 5.30	0.7	14
	5.70 - 5.73	0.8	16
	6.38 - 6.41	1.0	20
	6.66 - 6.69	0.8	16
	7.19 - 7.23	0.7	14
7	2.79 - 2.82	1.6	32
	3.19 - 3.22	1.6	32
	3.65 - 3.68	1.2	24
	4.27 - 4.31	1.1	22
	4.71 - 4.74	1.7	34
	5.38 - 5.40	0.9	18
	5.64 - 5.67	1.7	34
	6.29 - 6.32	1.5	30

NOTES: See Page 4 of 4

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SOIL TEST SERVICES

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32507R
Project:	Proposed Hospital Redevelopment	**Report:**	B
Location:	97-115 River Road, Greenwich, NSW	**Report Date:**	5/08/2019
		Page 3 of 4	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
7	6.69 - 6.72	2.0	40
	7.05 - 7.08	1.9	38
9	1.37 - 1.40	0.4	8
	1.78 - 1.81	0.5	10
	2.27 - 2.30	0.9	18
	2.68 - 2.71	0.6	12
	3.12 - 3.15	1.0	20
	3.68 - 3.71	0.6	12
	4.08 - 4.11	0.09	2
	4.61 - 4.65	0.2	4
	5.19 - 5.22	1.3	26
	5.67 - 5.71	0.7	14
	6.33 - 6.36	1.1	22
	6.89 - 6.92	0.6	12
	7.18 - 7.22	1.0	20
10	1.36 - 1.39	0.6	12
	1.87 - 1.90	1.0	20
	2.29 - 2.32	1.4	28
	2.78 - 2.81	1.1	22
	3.27 - 3.31	1.0	20
	3.88 - 3.92	0.9	18
	4.29 - 4.33	1.6	32
	4.78 - 4.81	2.0	40
	5.08 - 5.11	0.9	18
	5.68 - 5.71	1.0	20

NOTES: See Page 4 of 4

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SOIL TEST SERVICES

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32507R
Project:	Proposed Hospital Redevelopment	**Report:**	B
Location:	97-115 River Road, Greenwich, NSW	**Report Date:**	5/08/2019
		Page 4 of 4	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
10	6.18 - 6.21	0.9	18
	6.68 - 6.71	0.7	14
	7.07 - 7.10	1.1	22

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 $I_{S\ (50)}$

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www.envirolab.com.au

CERTIFICATE OF ANALYSIS 222885

Client Details

Client	JK Geotechnics
Attention	Stephen Mosad
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**32507R, Greenwich**
Number of Samples	6 Soil
Date samples received	01/08/2019
Date completed instructions received	01/08/2019

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	08/08/2019
Date of Issue	06/08/2019

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil						
Our Reference		222885-1	222885-2	222885-3	222885-4	222885-5
Your Reference	UNITS	BH5	BH5	BH5	BH1	BH2
Depth		1.0-1.2	2.1-2.3	0.5-0.95	0.7-0.95	0.5-0.95
Date Sampled		23/07/2019	23/07/2019	23/07/2019	22/07/2019	23/07/2019
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	05/08/2019	05/08/2019	05/08/2019	05/08/2019	05/08/2019
Date analysed	-	05/08/2019	05/08/2019	05/08/2019	05/08/2019	05/08/2019
pH 1:5 soil:water	pH Units	5.2	5.1	4.7	8.0	8.0
Electrical Conductivity 1:5 soil:water	µS/cm	34	39	190	79	87
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	39	43	360	28	29
Resistivity in soil*	ohm m	290	250	53	130	110

Misc Inorg - Soil		
Our Reference		222885-6
Your Reference	UNITS	BH4
Depth		0.5-0.95
Date Sampled		23/07/2019
Type of sample		Soil
Date prepared	-	05/08/2019
Date analysed	-	05/08/2019
pH 1:5 soil:water	pH Units	8.7
Electrical Conductivity 1:5 soil:water	µS/cm	94
Chloride, Cl 1:5 soil:water	mg/kg	10
Sulphate, SO4 1:5 soil:water	mg/kg	23
Resistivity in soil*	ohm m	110

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

Client Reference: 32507R, Greenwich

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			05/08/2019	[NT]	[NT]	[NT]	[NT]	05/08/2019	[NT]
Date analysed	-			05/08/2019	[NT]	[NT]	[NT]	[NT]	05/08/2019	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	104	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	102	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	108	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.
Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.	

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R **Method:** SPIRAL AUGER **R.L. Surface:** 37.3 m
Date: 22/7/19 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** S.M./P.R.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF CORING		37			CONCRETE: 170mm.t				8mm DIA. REINFORCEMENT, 12mm & 35mm BOTTOM COVER
				-	FILL: Silty sandy gravel, fine to medium grained, igneous, angular, dark grey, fine to coarse grained sand.	M D			60mm VOID UNDERNEATH SLAB
	N = 11 4,7,4				FILL: Clayey sand, medium to coarse grained, yellow brown and orange brown, with sub-angular fine to medium grained sandstone gravel.	w>PL		490 390 360	APPEARS MODERATELY COMPACTED
		36	1		FILL: Silty clay, medium to high plasticity, light grey mottled brown and dark grey, trace of ash, slag, and angular fine to medium grained igneous gravel.				
	N = 8 3,4,4	35	2		FILL: Silty clay, medium plasticity, light grey, with angular medium to coarse grained ironstone gravel.			250 270 190	APPEARS MODERATELY COMPACTED
ON COMPLETION	N = 2 1,1,1	34	3					200 180 140	
					FILL: Sand, medium to coarse grained, light grey and brown.	M W			
			4	-	SANDSTONE: fine to medium grained, light grey.	DW SW	L M - H		HAWKESBURY SANDSTONE LOW TO MODERATE 'TC' BIT RESISTANCE HIGH RESISTANCE
		33 32 31	5 6		REFER TO CORED BOREHOLE LOG				

SAMPLES: ES U50 DB DS

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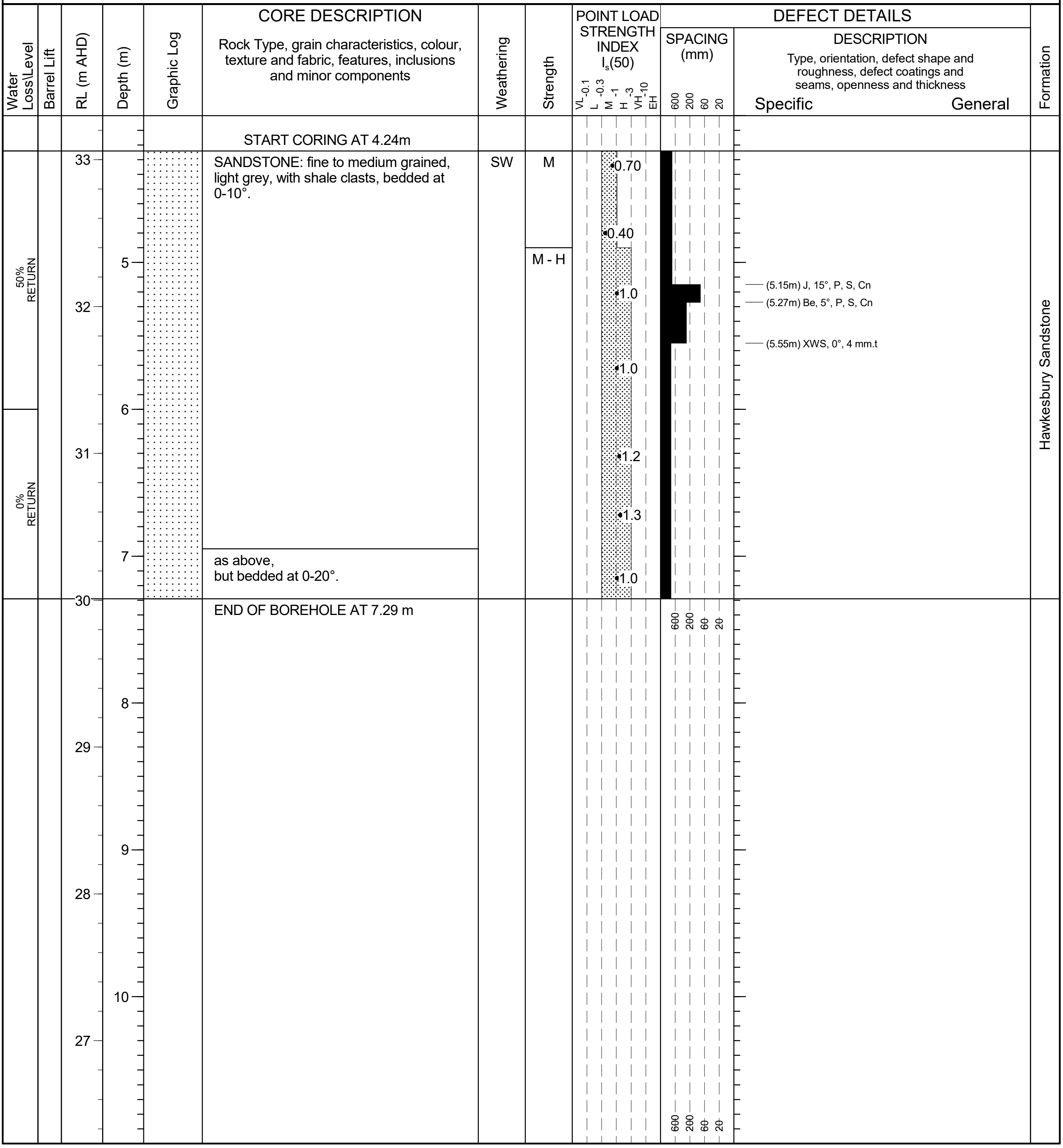
JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1
2 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Core Size: NMLC
R.L. Surface: 37.3 m
Date: 22/7/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: S.M./P.R.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					START CORING AT 4.24m							
50% RETURN		33 32	5		SANDSTONE: fine to medium grained, light grey, with shale clasts, bedded at 0-10°.	SW	M M - H	0.70 0.40 1.0 1.0		(5.15m) J, 15°, P, S, Cn (5.27m) Be, 5°, P, S, Cn (5.55m) XWS, 0°, 4 mm.t		Hawkesbury Sandstone
0% RETURN		31	6 7					1.2 1.3				
					as above, but bedded at 0-20°.			1.0				
		30			END OF BOREHOLE AT 7.29 m							

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 32507R BH1 STARTED CORING AT 4.24m
4
5
6
7
END OF BOREHOLE AT 7.29m

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 1

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R **Method:** SPIRAL AUGER **R.L. Surface:** 39 m
Date: 23/7/19 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** S.M./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										FILL: Silty sand, fine to medium grained, dark brown, with root fibres.	M			GRASS COVER APPEARS MODERATELY COMPACTED
					N = 21 2,12,9					FILL: Silty sand, medium to coarse grained, brown, with clay fines an sub-angular medium to coarse grained sandstone gravel, trace of ash.				
						38	1			FILL: Silty sandy clay, low to medium plasticity, brown mottled orange brown, medium to coarse grained sand, with sub-angular medium to coarse grained ironstone and sandstone gravel, trace of slag and ash.				APPEARS MODERATELY COMPACTED
					N = 4 2,2,2									
						37	2							
									-	SANDSTONE: fine to medium grained, light grey.	SW	M - H		HAWKESBURY SANDSTONE HIGH 'TC' BIT RESISTANCE WITH VERY LOW RESISTANCE BANDS
											DW	VL		
						36	3							
										as above, but yellow brown.	SW	M		
										as above, but orange brown.	DW	VL		
						35	4			as above, but light grey.	SW	M		MODERATE RESISTANCE WITH VERY LOW RESISTANCE BANDS
											DW	VL		
											SW	M		GROUNDWATER MONITORING WELL INSTALLED TO 2.3m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 0.4m TO 2.3m. CASING 0.11m TO 0.4m. 2mm SAND FILTER PACK 0.4m TO 2.3m. BENTONITE SEAL 0.11m TO 0.4m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						34	5							
						33	6							
										END OF BOREHOLE AT 6.30 m				

BOREHOLE LOG

Borehole No.
3
1 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R **Method:** SPIRAL AUGER **R.L. Surface:** 42.8 m
Date: 23/7/19 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** S.M./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 OF CORING	■							■	-	ASPHALTIC CONCRETE: 45mm.t	D			APPEARS MODERATELY COMPACTED
	■							▨		FILL: Silty gravelly sand, medium to coarse grained, dark grey, angular medium to coarse grained igneous gravel.				
	■			■	N = 11 8,4,7	42		▨		FILL: Sand, medium to coarse grained, light grey, with dark grey silty clay nodules, trace of ash.				
	■						1	▨	-	FILL: Silty sand, medium to coarse grained, brown and light grey, with sub-angular medium to coarse grained sandstone gravel, trace of clay fines, slag and ash.	SW	M - H		HAWKESBURY SANDSTONE HIGH 'TC' BIT RESISTANCE
										FILL: Clayey sand, medium to coarse grained, light brown.				
						41				SANDSTONE: fine to medium grained, light grey.				
							2			REFER TO CORED BOREHOLE LOG				
						40	3							
						39	4							
						38	5							
						37	6							
						36								

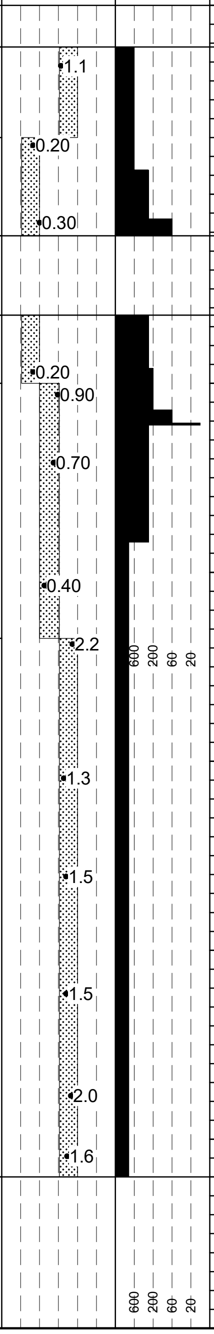
CORED BOREHOLE LOG

Borehole No.
3
2 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Core Size: NMLC
R.L. Surface: 42.8 m
Date: 23/7/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: S.M./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
					START CORING AT 1.22m					
ON COMPLETION		41	2		SANDSTONE: fine to medium grained, dark red brown, light grey and orange brown, bedded at 0-20°.	MW	H L	1.1 0.20 0.30	(1.87m) XWS, 20°, 3 mm.t (2.13m) XWS, 10°, 8 mm.t (2.21m) XWS, 10°, 15 mm.t	Hawkesbury Sandstone
					NO CORE 0.42m					
100% RETURN		40 39	3 4		SANDSTONE: fine to medium grained, light grey and dark red brown, bedded at 0-10°.	MW	L M	0.20 0.90 0.70 0.40	(2.65m) XWS, 0°, 25 mm.t (2.92m) Cb, 0°, 5mm.t (3.14m) XWS, 0°, 3-8mm.t (3.21m) Be, 0°, P, S, Cb Sn (3.22m) J, 30°, P, S, Cb Sn, 5 mm.t (3.67m) Be, 0°, P, S, Cb Sn, 5 mm.t (3.84m) XWS, 0°, 75 mm.t	Hawkesbury Sandstone
		38 37	5 6		as above, but light grey and orange brown.	SW	H	2.2 1.3 1.5 1.5	(6.57m) XWS, 0°, 65 mm.t	
		36	7		as above, but light grey, red brown and yellow brown, with slump bedding structure.			2.0 1.6		
		35			END OF BOREHOLE AT 7.20 m					



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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32507R GREENWICH.GPJ <<DrawingFile>> 13/09/2019 16:02 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20


JK Geotechnics
JOB No. 32507R BH3 STARTED CORING AT 1.22m
1
2
NO CORE 0.42m
3
4
5
6
7
END OF BOREHOLE AT 7.20m

BOREHOLE LOG

Borehole No.
4
1 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R **Method:** SPIRAL AUGER **R.L. Surface:** 41.5 m
Date: 23/7/19 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** S.M./P.R.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF CORING		0		ASPHALTIC CONCRETE: 65mm.t				
			-	FILL: Silty gravelly sand, medium to coarse grained, dark grey, angular fine to coarse grained igneous gravel.	M W			APPEARS POORLY COMPACTED
	N = 3 4,2,1			FILL: Silty sand, medium to coarse grained, brown and dark yellow brown, with sub-angular medium to coarse grained sandstone gravel.	M			
	N = 4 3,2,2	1		as above, but orange brown and yellow brown.				
		2		as above, but brown, with clay fines.				
			SC	Clayey SAND: fine to medium grained, red brown and orange brown.	M	(L)		RESIDUAL
			-	SANDSTONE: fine to medium grained, light grey.	DW	VL - L		HAWKESBURY SANDSTONE LOW 'TC' BIT RESISTANCE
		3		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 2.2m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 0.1m TO 2.2m. 2mm SAND FILTER PACK 0.3m TO 2.2m. BENTONITE SEAL 0.1m TO 0.3m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

RL (m AHD): 41, 40, 39, 38, 37, 36, 35
Depth (m): 1, 2, 3, 4, 5, 6

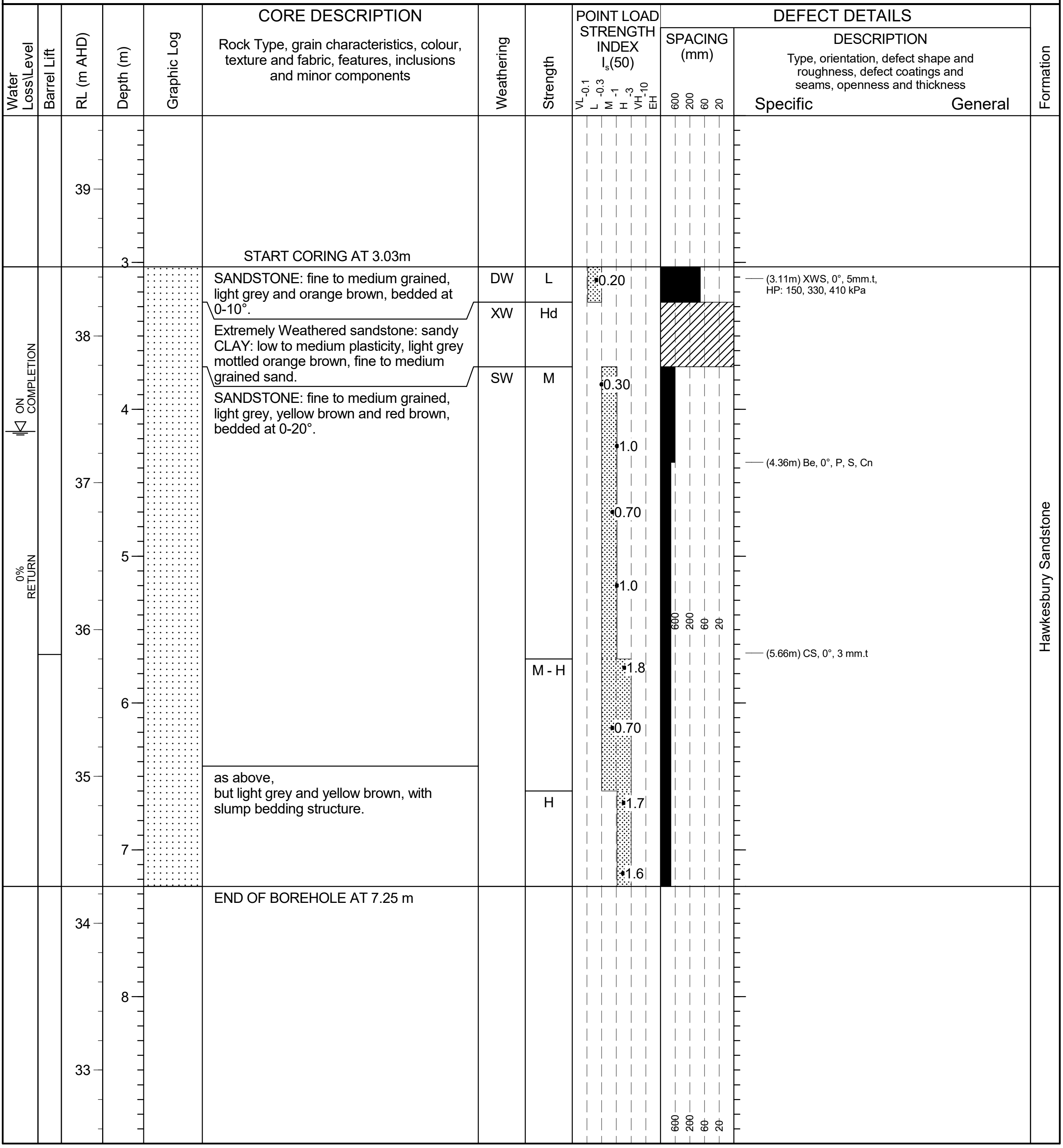
SAMPLES: ES, U50, DB, DS

CORED BOREHOLE LOG

Borehole No.
4
2 / 2

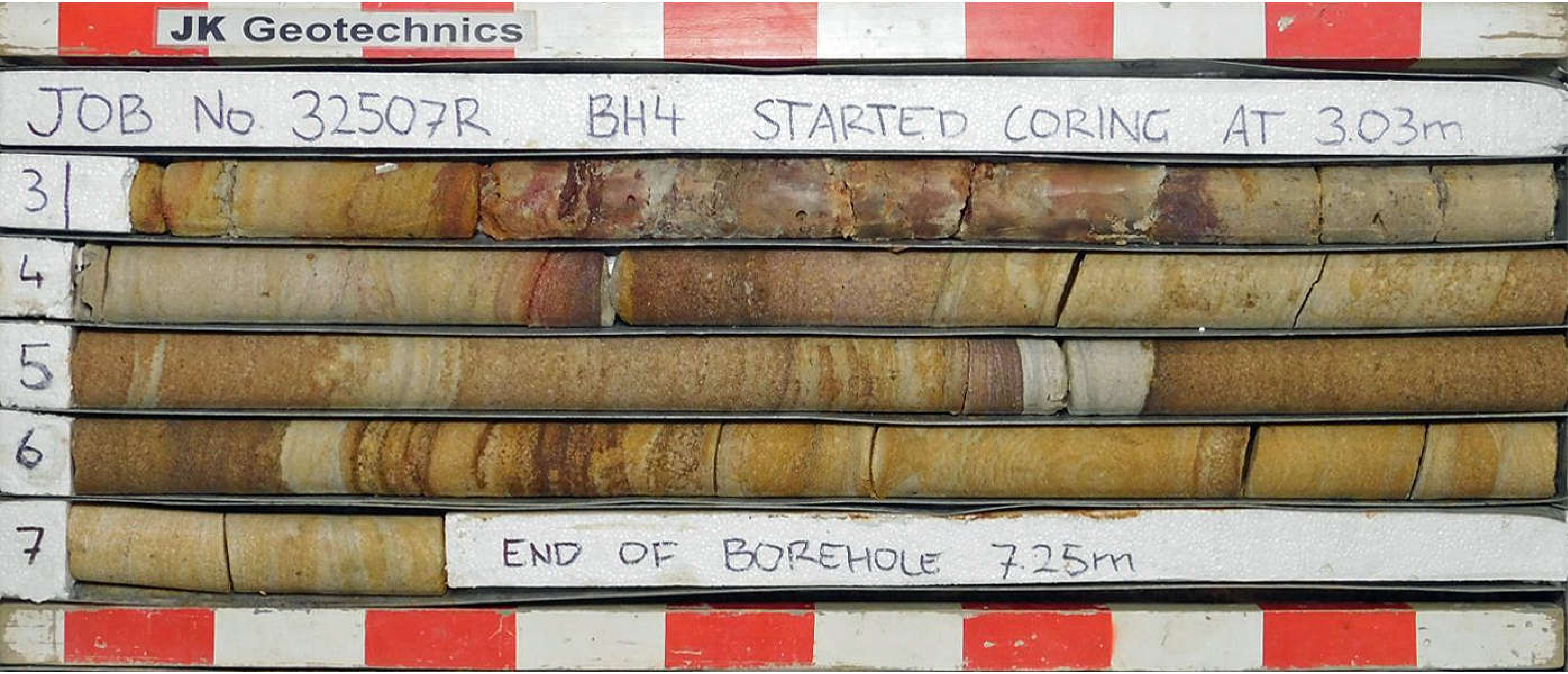
Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Core Size: NMLC
R.L. Surface: 41.5 m
Date: 23/7/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: S.M./P.R.



CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
START CORING AT 3.03m						
SANDSTONE: fine to medium grained, light grey and orange brown, bedded at 0-10°.	DW	L	0.20	(3.11m) XWS, 0°, 5mm.t, HP: 150, 330, 410 kPa		Hawkesbury Sandstone
Extremely Weathered sandstone: sandy CLAY: low to medium plasticity, light grey mottled orange brown, fine to medium grained sand.	XW	Hd				
SANDSTONE: fine to medium grained, light grey, yellow brown and red brown, bedded at 0-20°.	SW	M	0.30 1.0 0.70 1.0	(4.36m) Be, 0°, P, S, Cn		
		M - H	1.8 0.70	(5.66m) CS, 0°, 3 mm.t		
as above, but light grey and yellow brown, with slump bedding structure.		H	1.7 1.6			
END OF BOREHOLE AT 7.25 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No 32507R BH4 STARTED CORING AT 3.03m
3
4
5
6
7
END OF BOREHOLE 7.25m

JKGeotechnics

BOREHOLE LOG

Borehole No.
5
1 / 1

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Method: SPIRAL AUGER
R.L. Surface: 44.5 m
Date: 23/7/19
Datum: AHD
Plant Type: JK205
Logged/Checked By: S.M./P.R.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION					ASPHALTIC CONCRETE: 65mm.t				
				-	FILL: Silty gravelly sand, medium to coarse grained igneous, angular, dark grey, medium to coarse grained sand.	D			
	N = 14 5,5,9	44		CL-CI	FILL: Sand, medium to coarse grained, orange brown, with clay fines, and sub-angular medium to coarse grained sandstone gravel.	w<PL	Hd	460 600 570	RESIDUAL
			1	-	Sandy CLAY: low to medium plasticity, orange brown and yellow brown, fine to medium grained.	DW	L		HAWKESBURY SANDSTONE
		43			as above, but light grey, with low strength iron indurated bands.	SW	M		LOW 'TC' BIT RESISTANCE HIGH RESISTANCE
			2		SANDSTONE: fine to medium grained, light grey and orange brown.	DW	VL		VERY LOW RESISTANCE
					as above, but red brown and light grey.	SW	M		HIGH RESISTANCE
		42			END OF BOREHOLE AT 2.30 m				

SAMPLES: ES, U50, DB, DS

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BOREHOLE LOG

Borehole No.
6
1 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R **Method:** SPIRAL AUGER **R.L. Surface:** 49.5 m
Date: 23/7/19 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** S.M./P.R.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF CORING										ASPHALTIC CONCRETE: 60mm.t				
	■ ■								-	FILL: Silty gravelly sand, medium to coarse grained, dark grey, sub-angular medium to coarse grained igneous gravel.	D			
	■				N=SPT 2/ 20mm REFUSAL	49			CI-CH	FILL: Clayey sand, medium to coarse grained, with sub-angular medium to coarse grained sandstone gravel.	w<PL	(VSt)		RESIDUAL
							1		-	Sandy CLAY: low plasticity, light grey, fine to medium grained, with low strength iron indurated bands.	HW	L		HAWKESBURY SANDSTONE
										SANDSTONE: fine to medium grained, light grey.	SW	M - H		LOW 'TC' BIT RESISTANCE
						48								HIGH RESISTANCE
										REFER TO CORED BOREHOLE LOG				
							2							
						47								
							3							
						46								
							4							
						45								
							5							
						44								
							6							
						43								

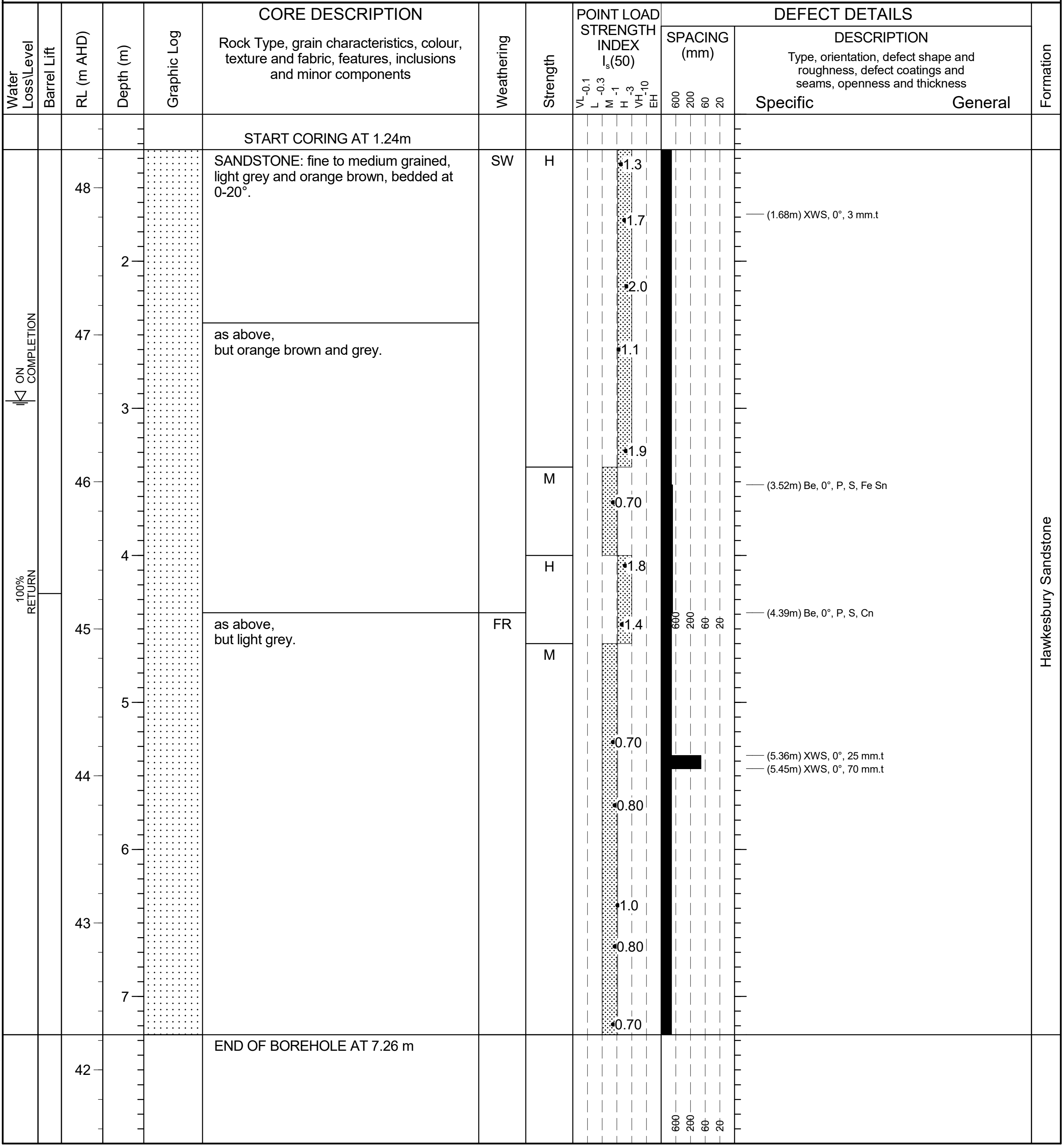
JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 32507R GREENWICH.GPJ <<DrawingFile>> 13/09/2019 16:02 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

Borehole No.
6
2 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Core Size: NMLC
R.L. Surface: 49.5 m
Date: 23/7/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: S.M./P.R.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					START CORING AT 1.24m						
ON COMPLETION 100% RETURN		48	2		SANDSTONE: fine to medium grained, light grey and orange brown, bedded at 0-20°.	SW	H	1.3 1.7 2.0	(1.68m) XWS, 0°, 3 mm.t		Hawkesbury Sandstone
		47 46	3 4		as above, but orange brown and grey.		H M H	1.1 1.9 0.70 1.8	(3.52m) Be, 0°, P, S, Fe Sn		
		45 44 43	5 6 7		as above, but light grey.	FR	M	1.4 0.70 0.80 1.0 0.80 0.70	(4.39m) Be, 0°, P, S, Cn (5.36m) XWS, 0°, 25 mm.t (5.45m) XWS, 0°, 70 mm.t		
		42			END OF BOREHOLE AT 7.26 m						

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 32507R BH6 STARTED CORING AT 1.24m
1
2
3
4
5
6
7
END OF BOREHOLE AT 7.29m

JKGeotechnics

BOREHOLE LOG

Borehole No.
7
1 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R **Method:** SPIRAL AUGER **R.L. Surface:** 52 m
Date: 23/7/19 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** S.M./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 OF CORING	■							▨		FILL: Silty sand, fine to medium grained, dark brown, with root fibres.	D			
	■			■		51	1	░	-	SANDSTONE: fine to medium grained, yellow brown and brown, with extremely weathered bands.	DW	L		HAWKESBURY SANDSTONE LOW 'TC' BIT RESISTANCE
				■				░				VL		VERY LOW RESISTANCE
								░				L		LOW RESISTANCE
						50	2	░		as above, but red brown and light grey.	SW	M		MODERATE RESISTANCE
				■				░		as above, but light grey.				
						49 48 47 46	3 4 5 6			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Core Size: NMLC
R.L. Surface: 52 m
Date: 23/7/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: S.M./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					START CORING AT 2.72m							
100% RETURN					SANDSTONE: fine to medium grained, light grey, brown ad orange brown, bedded at 0-20°.	SW	H	1.6		(2.85m) Be, 5°, P, S, Fe Sn (2.87m) Be, 10°, P, S, Fe Sn (2.89m) Be, 0°, P, S, Cn		Hawkesbury Sandstone
		49	3					1.6				
								1.2				
		48	4							(4.12m) Be, 0°, Ir, R, Cn		
								1.1		(4.42m) XWS, 0°, 5 mm.t		
								1.7				
		47	5				M			(5.27m) Be, 0°, Ir, R, Cn		
								0.90	600 200 60 20			
							H	1.7				
		46	6		as above, but light grey, with brown bands.							
								1.5				
								2.0				
		45	7					1.9				
					END OF BOREHOLE AT 7.14 m							
		44	8						600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 32507R BH7 STARTED CORING AT 2.72m
3
4
5
6
7
END OF BOREHOLE AT 7.14m

BOREHOLE LOG

Borehole No.
8
1 / 1

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Method: SPIRAL AUGER
R.L. Surface: 50.8 m
Date: 23/7/19
Datum: AHD
Plant Type: JK205
Logged/Checked By: S.M./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	■							▨		FILL: Silty sand, fine to medium grained, dark brown, with root fibres.	D			GRASS COVER
				■				⠿	-	SANDSTONE: fine to medium grained, light grey and red brown.	SW	M - H		HAWKESBURY SANDSTONE
						50				END OF BOREHOLE AT 0.70 m				HIGH 'TC' BIT RESISTANCE
							1							
						49								
							2							
						48								
							3							
						47								
							4							
						46								
							5							
						45								
							6							
						44								

BOREHOLE LOG

Borehole No.
9
1 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Method: SPIRAL AUGER
R.L. Surface: 49 m
Date: 24/7/19
Datum: AHD
Plant Type: JK205
Logged/Checked By: S.M./P.R.

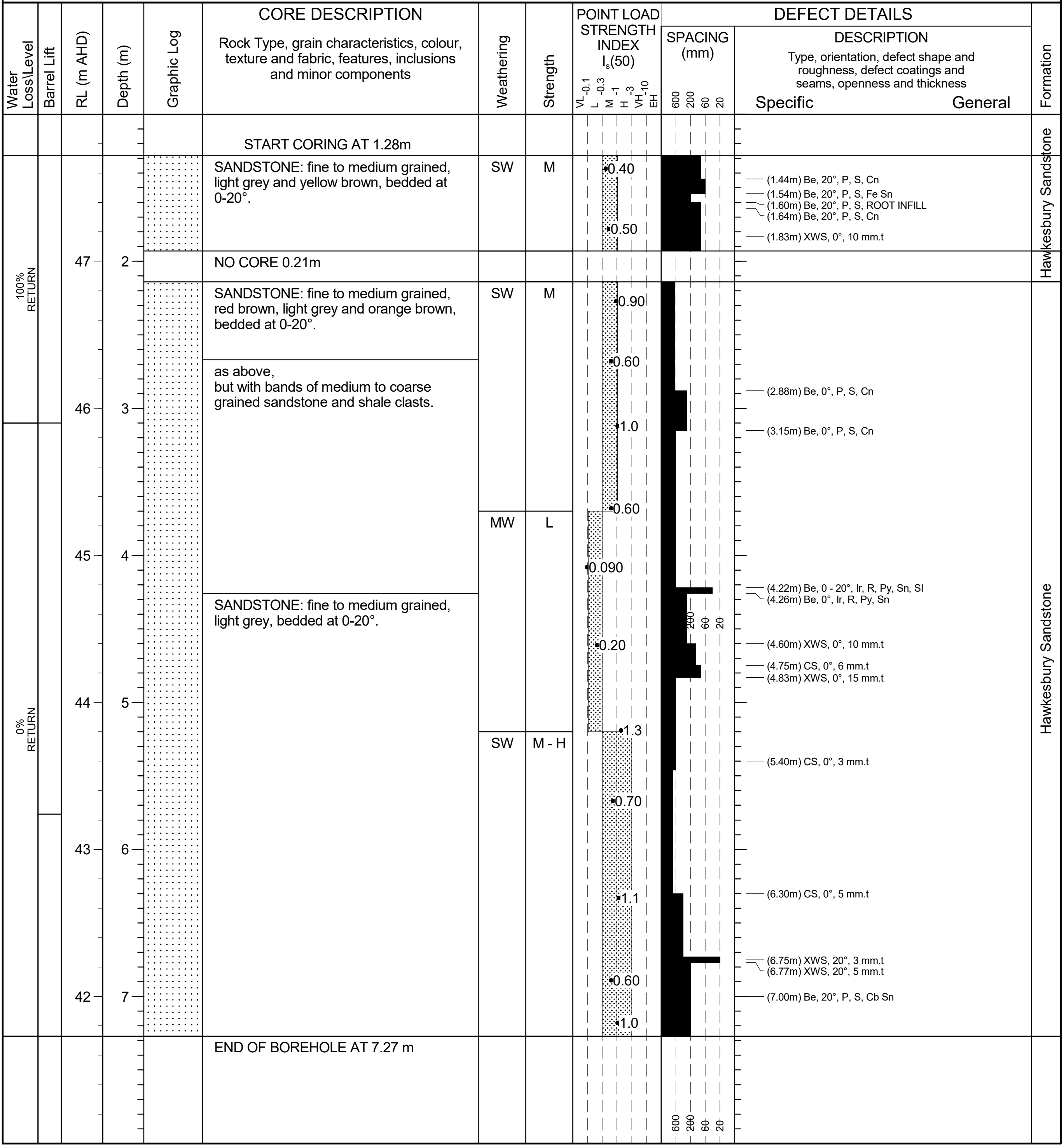
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
ON COMPLETION ▽	■			■	N > 13 10,2,11/ 120mm REFUSAL			▨		FILL: Silty sandy gravel, fine to medium grained, dark grey, medium to coarse grained sand, with slag.	D			ROAD BASE APPEARS MODERATELY COMPACTED
	■			■				▨		FILL: Silty gravelly sand, medium to coarse grained, red brown, brown and dark grey, medium to coarse grained igneous and sandstone gravel.				
	■					48	1	⁙	-	FILL: Silty sand, medium to coarse grained, brown and orange brown, with clay fines, and rounded medium to coarse grained sandstone gravel, trace of ash.	SW	M - H		HAWKESBURY SANDSTONE
										SANDSTONE: fine to medium grained, light grey and yellow brown.				
										REFER TO CORED BOREHOLE LOG				
						47	2							
						46	3							
						45	4							
						44	5							
						43	6							

CORED BOREHOLE LOG

Borehole No.
9
2 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Core Size: NMLC
R.L. Surface: 49 m
Date: 24/7/19
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: S.M./P.R.



Water Loss\Level	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific	Formation
			START CORING AT 1.28m					
100% RETURN			SANDSTONE: fine to medium grained, light grey and yellow brown, bedded at 0-20°.	SW	M	0.40 0.50	(1.44m) Be, 20°, P, S, Cn (1.54m) Be, 20°, P, S, Fe Sn (1.60m) Be, 20°, P, S, ROOT INFILL (1.64m) Be, 20°, P, S, Cn (1.83m) XWS, 0°, 10 mm.t	Hawkesbury Sandstone
	47	2	NO CORE 0.21m					
			SANDSTONE: fine to medium grained, red brown, light grey and orange brown, bedded at 0-20°.	SW	M	0.90		Hawkesbury Sandstone
	46	3	as above, but with bands of medium to coarse grained sandstone and shale clasts.			0.60 1.0 0.60	(2.88m) Be, 0°, P, S, Cn (3.15m) Be, 0°, P, S, Cn	
0% RETURN	45	4		MW	L	0.090	(4.22m) Be, 0 - 20°, Ir, R, Py, Sn, Sl	
	44	5	SANDSTONE: fine to medium grained, light grey, bedded at 0-20°.			0.20	(4.26m) Be, 0°, Ir, R, Py, Sn (4.60m) XWS, 0°, 10 mm.t (4.75m) CS, 0°, 6 mm.t (4.83m) XWS, 0°, 15 mm.t	
	43 42	6 7		SW	M - H	1.3 0.70 1.1 0.60 1.0	(5.40m) CS, 0°, 3 mm.t (6.30m) CS, 0°, 5 mm.t (6.75m) XWS, 20°, 3 mm.t (6.77m) XWS, 20°, 5 mm.t (7.00m) Be, 20°, P, S, Cb Sn	
			END OF BOREHOLE AT 7.27 m					

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 32507R BH9 STARTED CORING AT 1.28m
1
2
NO CORE 0.19m
3
4
5
6
7
END OF BOREHOLE AT 7.27m

JKGeotechnics

BOREHOLE LOG

Borehole No.
10
1 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Method: SPIRAL AUGER
R.L. Surface: 49.3 m
Date: 24/7/19
Datum: AHD
Plant Type: JK205
Logged/Checked By: S.M./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 OF CORING										FILL: Silty sand, fine to medium grained, dark brown, with root fibres.	D			GRASS COVER
						49			-	SANDSTONE: fine to medium grained, light grey and red brown.	SW	M - H		HAWKESBURY SANDSTONE
										as above, but light grey and orange brown.				HIGH 'TC' BIT RESISTANCE
							1							
						48				REFER TO CORED BOREHOLE LOG				
							2							
						47								
							3							
						46								
							4							
						45								
							5							
						44								
							6							
						43								

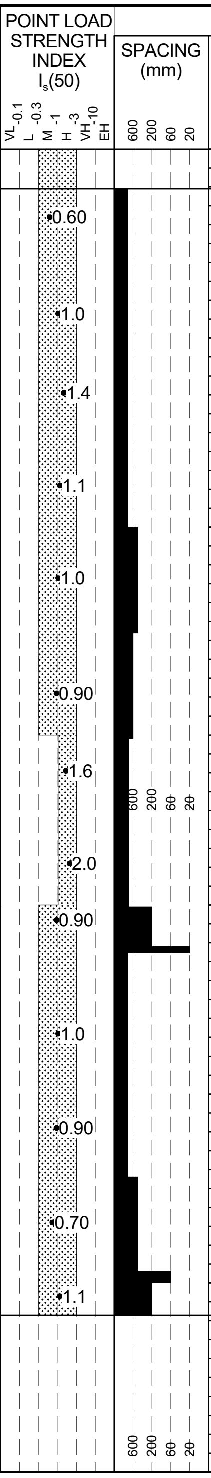
CORED BOREHOLE LOG

Borehole No.
10
2 / 2

Client: HAMMOND CARE
Project: PROPOSED HOSPITAL REDEVELOPMENT
Location: 97-115 RIVER ROAD, GREENWICH, NSW

Job No.: 32507R
Date: 24/7/19
Plant Type: JK205
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: 49.3 m
Datum: AHD
Logged/Checked By: S.M./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	Formation
				START CORING AT 1.21m			
100% RETURN ON COMPLETION		48 47	2 3	SANDSTONE: fine to medium grained, light grey, orange brown and dark red brown, bedded at 0-20°.	SW	M - H	Hawkesbury Sandstone
0% RETURN		46 45	4	as above, but dark grey and red brown.		H	
		44 43	5 6 7	as above, but light grey.	FR	M - H	
		42		END OF BOREHOLE AT 7.17 m			



DEFECT DETAILS
DESCRIPTION
Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness

Specific	General
(1.71m) Be, 0°, P, S, Cn	
(3.00m) XWS, 0°, 5 mm.t	
(3.56m) Be, 0°, Ir, R, Fe Sn	
(4.12m) Be, 20°, P, S, Cb Sn	
(5.01m) Be, 20°, P, S, Fe Sn	
(5.22m) J, 50°, P, S, Cn	
(5.25m) Be, 0°, P, S, Cn	
(6.44m) XWS, 0°, 20 mm.t	
(6.94m) XWS, 20°, 5 mm.t	
(7.00m) XWS, 0°, 3 mm.t	

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 32507R BH10 STARTED CORING AT 1.21m
1
2
3
4
5
6
7
END OF BOREHOLE AT 7.17m

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
enquiries@envirolabservices.com.au
www.envirolabservices.com.au

CERTIFICATE OF ANALYSIS 110514

Client:

Environmental Investigation Services
PO Box 976
North Ryde BC
NSW 1670

Attention: David Schwarzer

Sample log in details:

Your Reference:	**23789ZR, Greenwich**
No. of samples:	6 Soils
Date samples received / completed instructions received	27/05/2014 / 27/05/2014

Analysis Details:

Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	3/06/14 / 2/06/14
Date of Preliminary Report:	Not Issued

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Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

Jacinta Hurst
Laboratory Manager

ACCREDITED FOR TECHNICAL COMPETENCE

Miscellaneous Inorg - soil						
Our Reference:	UNITS	110514-1	110514-2	110514-3	110514-4	110514-5
Your Reference	-------------	1	2	3	4	5
Depth	------------	1.5-1.95	0.5-0.95	1.5-1.95	1.5-1.95	3.5-4.0
Date Sampled		20/05/2014	20/05/2014	21/05/2014	21/05/2014	23/05/2014
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	28/05/2014	28/05/2014	28/05/2014	28/05/2014	28/05/2014
Date analysed	-	28/05/2014	28/05/2014	28/05/2014	28/05/2014	28/05/2014
pH 1:5 soil:water	pH Units	6.0	8.4	8.2	8.8	7.4
Chloride, Cl 1:5 soil:water	mg/kg	100	<10	84	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	460	30	340	31	250

Miscellaneous Inorg - soil		
Our Reference:	UNITS	110514-6
Your Reference	-------------	6
Depth	------------	6-6.45
Date Sampled		23/05/2014
Type of sample		Soil
Date prepared	-	28/05/2014
Date analysed	-	28/05/2014
pH 1:5 soil:water	pH Units	4.4
Chloride, Cl 1:5 soil:water	mg/kg	25
Sulphate, SO4 1:5 soil:water	mg/kg	170

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA 22nd ED, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 22nd ED, 4110 -B.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			28/05/2014	[NT]	[NT]	LCS-1	28/05/2014
Date analysed	-			28/05/2014	[NT]	[NT]	LCS-1	28/05/2014
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	97%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	107%

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NA: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

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
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	23789ZR
Project:	Proposed Car Park	**Report:**	A
Location:	Greenwich Hospital, River Road, Greenwich, NSW	**Report Date:**	2/06/2014
		Page 1 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	4.42-4.45	1.1	22
	4.77-4.79	1.7	34
	5.13-5.15	1.3	26
	5.79-5.82	1.7	34
	6.45-6.47	1.3	26
	7.20-7.22	1.8	36
	7.75-7.77	1.4	28
	8.37-8.40	1.5	30
	8.80-8.83	1.1	22
	9.50-9.53	1.4	28
	10.21-10.24	1.4	28
	10.75-10.78	1.6	32
	11.50-11.52	1.5	30
	12.23-12.26	1.0	20
	12.72-12.75	1.4	28
	13.46-13.48	1.4	28
	14.35-14.39	2.2	44
	14.82-14.84	1.7	34
	15.61-15.64	1.3	26
	16.22-16.46	1.7	34
	16.75-16.80	0.8	16
	17.28-17.31	1.2	24
	17.60-17.63	0.3	6
	18.10-18.12	1.1	22
	18.69-18.72	1.1	22
	19.33-19.37	2.5	50

NOTES: See Page 2 of 2

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Car Park
Location: Greenwich Hospital, River Road, Greenwich, NSW

Ref No: 23789ZR
Report: A
Report Date: 2/06/2014
Page 2 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
4	4.70-4.74	0.4	8
	5.09-5.12	1.0	20
	5.75-5.80	1.2	24
	6.47-6.50	0.7	14
	7.23-7.27	1.0	20
	7.73-7.76	1.0	20
	8.56-8.59	1.0	20
	9.49-9.52	0.8	16
	10.78-10.82	1.2	24
	11.38-11.42	1.5	30
	12.21-12.25	0.6	12
	12.84-12.87	1.0	20
	13.47-13.51	0.9	18
	13.95-13.99	1.5	30
6	8.90-8.94	0.2	4
	9.27-9.29	0.5	10
	9.75-9.79	0.9	18
	10.55-10.59	1.6	32
	11.22-11.26	0.7	14
	11.77-11.79	0.9	18
	12.50-12.54	1.4	28
	13.30-13.33	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: 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 $I_{S\,(50)}$

BOREHOLE LOG

Borehole No.
1
1/1

Client: HAMMOND CARE
Project: PROPOSED CAR PARK
Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Method: SPIRAL AUGER
JK308
R.L. Surface: ≈ 38.2m
Date: 21-5-14
Datum: AHD
Logged/Checked by: D.S./P.R.

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 AND AFTER 6.25HRS						0		-	ASPHALTIC CONCRETE: 30mm.t.	D			APPEARS MODERATELY COMPACTED
	■								FILL: Silty sand, fine to medium grained, red brown, with fine to medium grained igneous gravel.				
				■	N = 11 6,5,6	1			FILL: Silty clayey sand, fine to medium grained, brown, light grey and dark brown, with fine to medium grained sandstone gravel.				
				■	N = 8 3,3,5	2			FILL: Silty clay, medium plasticity, brown mottled light grey, with fine to medium grained ironstone and sandstone gravel.	MC>PL		280 300 250	
				■				-	SANDSTONE: fine to medium grained, yellow brown and orange brown.	DW	L-M		LOW TO MODERATE 'TC' BIT RESISTANCE
				■		3			SANDSTONE: fine to medium grained, light grey mottled orange brown.		H		HIGH RESISTANCE
						4 5 6 7			END OF BOREHOLE AT 3.5m				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/4

Client: HAMMOND CARE
Project: PROPOSED CAR PARK
Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Method: SPIRAL AUGER JK308
R.L. Surface: ≈ 38.0m
Date: 21-5-14
Datum: AHD
Logged/Checked by: D.S./P.R.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0	▨	-	ASPHALTIC CONCRETE: 30mm.t. FILL: Silty sandy clay, low plasticity, brown, fine to medium grained sand with fine to medium grained sandstone gravel.	MC>PL			APPEARS MODERATELY TO POORLY COMPACTED
				■	N = 5 3,2,3		▨					250 200 300	
						1	▨						
							▨		as above, but with fine to coarse grained ironstone gravel, brick fragments and trace roots.	MC<PL			APPEARS MODERATELY COMPACTED TOO FRIABLE FOR HP TESTING
				■	N = 15 7,5,10		▨						
						2	▨						LOW 'TC' BIT RESISTANCE (POSSIBLE BOULDER)
							▨		FILL: Silty clay, low plasticity, light grey, with fine grained sand and fine to medium grained sandstone and ironstone gravel.				
ON COMPLETION OF CORING ▼				■	N = 20 6,10,10	3	▨						TOO FRIABLE FOR HP TESTING
				■		4	⁙	-	SANDSTONE: fine to medium grained, orange brown mottled light grey.	SW	M-H		MODERATE TO HIGH RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						5							
						6							
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
2
2/4

Client: HAMMOND CARE

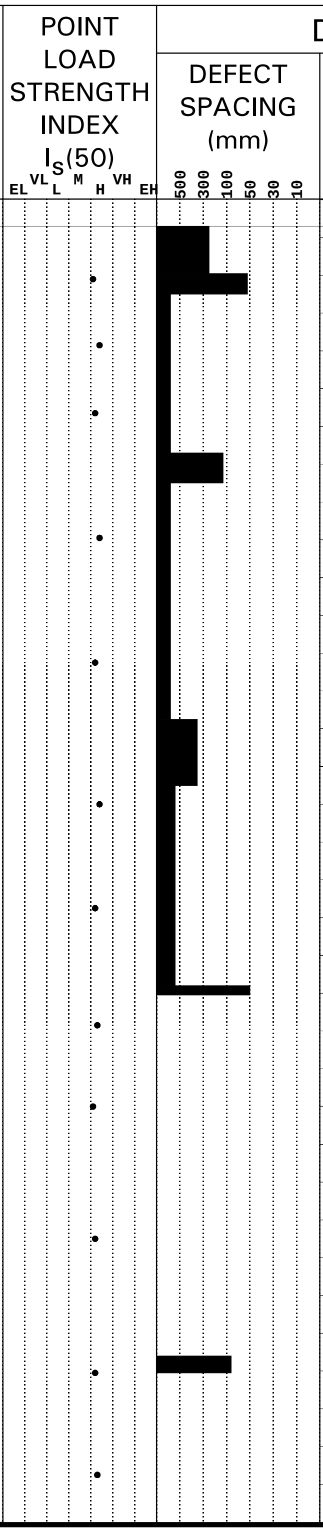
Project: PROPOSED CAR PARK

Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR | **Core Size:** NMLC | **R.L. Surface:** ≈ 38.0m

Date: 21-5-14 | **Inclination:** VERTICAL | **Datum:** AHD

Drill Type: JK308 | **Bearing:** - | **Logged/Checked by:** D.S./P.R.



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		START CORING AT 4.14m				
				SANDSTONE: fine to medium grained, orange brown mottled light grey, bedded at 20°.	SW	M-H	- CS,10°,3mm.t - Be,20°,P,S,IS - CS,5°,2mm.t	
		5					- Be,20°,P,S	
				as above, but light grey mottled light brown.			- CS,10°,5mm.t - CS,0°,3mm.t	
		6						
							- Be,20°,P,S,clay infill 1mm.t	
		7					- CS,10°,5mm.t	
				SANDSTONE: fine to coarse grained, light grey mottled dark grey and brown.				
		8					- J,30°,Un,R - J,30°,Un,R	
				SANDSTONE: fine to medium grained, light grey.				
		9						
				SANDSTONE: fine to medium grained, light grey and light brown, with dark brown laminae, bedded at 0°-30°.				
		10					- CS,10°,3mm.t - CS,10°,5mm.t	
				SANDSTONE: fine to medium grained light grey, with dark grey laminae, bedded at 0°-10°.	FR			

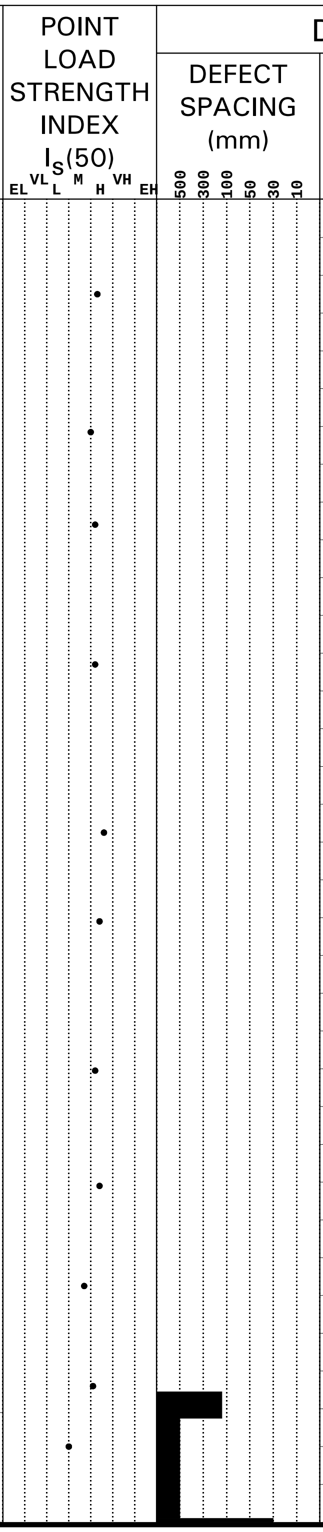
CORED BOREHOLE LOG

Borehole No.
2
3/4

Client: HAMMOND CARE
Project: PROPOSED CAR PARK
Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Core Size: NMLC
R.L. Surface: ≈ 38.0m
Date: 21-5-14
Inclination: VERTICAL
Datum: AHD
Drill Type: JK308
Bearing: -
Logged/Checked by: D.S./P.R.

Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
95% RE TURN	12 13	SANDSTONE: fine to medium grained, light grey, massive.	Fr	M-H	- CS,0°,15mm.t
	14	SANDSTONE: fine to medium grained, light grey, with occassionaldark grey laminae, bedded at 10°-30°.			- CS,0°,5mm.t
	15 16	SANDSTONE: fine to medium grained, light grey, cross bedded at 20°.			
	17	SANDSTONE: fine to coarse grained, light grey, cross bedded at -20°.			- Be,10°,P,R
		SANDSTONE: fine grained light grey, with dark grey laminae bedded at 0°-10°.	FR	M	- XWS,0°,70mm.t - J,85°,Un,R



CORED BOREHOLE LOG

Borehole No.
2
4/4

Client: HAMMOND CARE

Project: PROPOSED CAR PARK

Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR

Core Size: NMLC

R.L. Surface: ≈ 38.0m

Date: 21-5-14

Inclination: VERTICAL

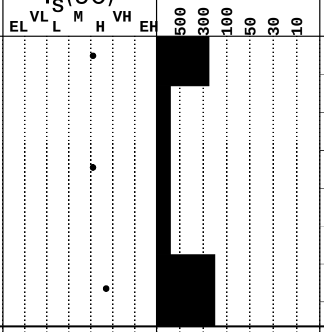
Datum: AHD

Drill Type: JK308

Bearing: -

Logged/Checked by: D.S./P.R.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		19		SANDSTONE: fine grained, light grey, with dark grey laminae bedded at 0°-10°.	Fr	M			- XWS,0°,20mm.t - CS,0°,40mm.t - XWS,0°,30mm.t - XWS,0°,60mm.t	
		20 21 22 23 24		END OF BOREHOLE AT 19.53m						




JK Geotechnics
JOB No.: 23789ZR BH2 START CORING AT 4.14m
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
END BH AT 19.53m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: HAMMOND CARE
Project: PROPOSED CAR PARK
Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Date: 22-5-14
Method: SPIRAL AUGER
JK308
R.L. Surface: ≈ 38.0m
Datum: AHD
Logged/Checked by: D.S./P.R.

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 COMPL -ETION AND AFTER 6HRS						0		-	ASPHALTIC CONCRETE: 30mm.t.	D			
									FILL: Silty sand, fine to medium grained, red brown, with fine to medium grained igneous gravel.	MC>PL			APPEARS MODERATELY COMPACTED
				■	N = 6 4,3,3				FILL: Silty sandy clay, low plasticity, brown, fine to medium grained sand with fine to medium grained igneous and sandstone gravel.			230 270 280	
						1							
				■	N = 7 2,4,3				FILL: Silty clay, medium plasticity, brown, with fine to medium grained ironstone and sandstone igneous gravel, trace fine to medium grained sand.			250 250 250	
						2							
				■	N >4 5,4,/100mm REFUSAL				FILL: Silty clayey sand, fine to medium grained, red brown mottled light grey, with fine to medium grained sandstone and ironstone gravel.	D			
						3		-	SANDSTONE: fine to medium grained, light grey.	SW	M H		MODERATE 'TC' BIT RESISTANCE HIGH RESISTANCE
				■									
						4			END OF BOREHOLE AT 4.0m				
						5							
						6							
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/4

Client: HAMMOND CARE

Project: PROPOSED CAR PARK

Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Date: 22-5-14

Method: SPIRAL AUGER
JK308

R.L. Surface: ≈ 37.8m
Datum: AHD

Logged/Checked by: D.S./P.R.

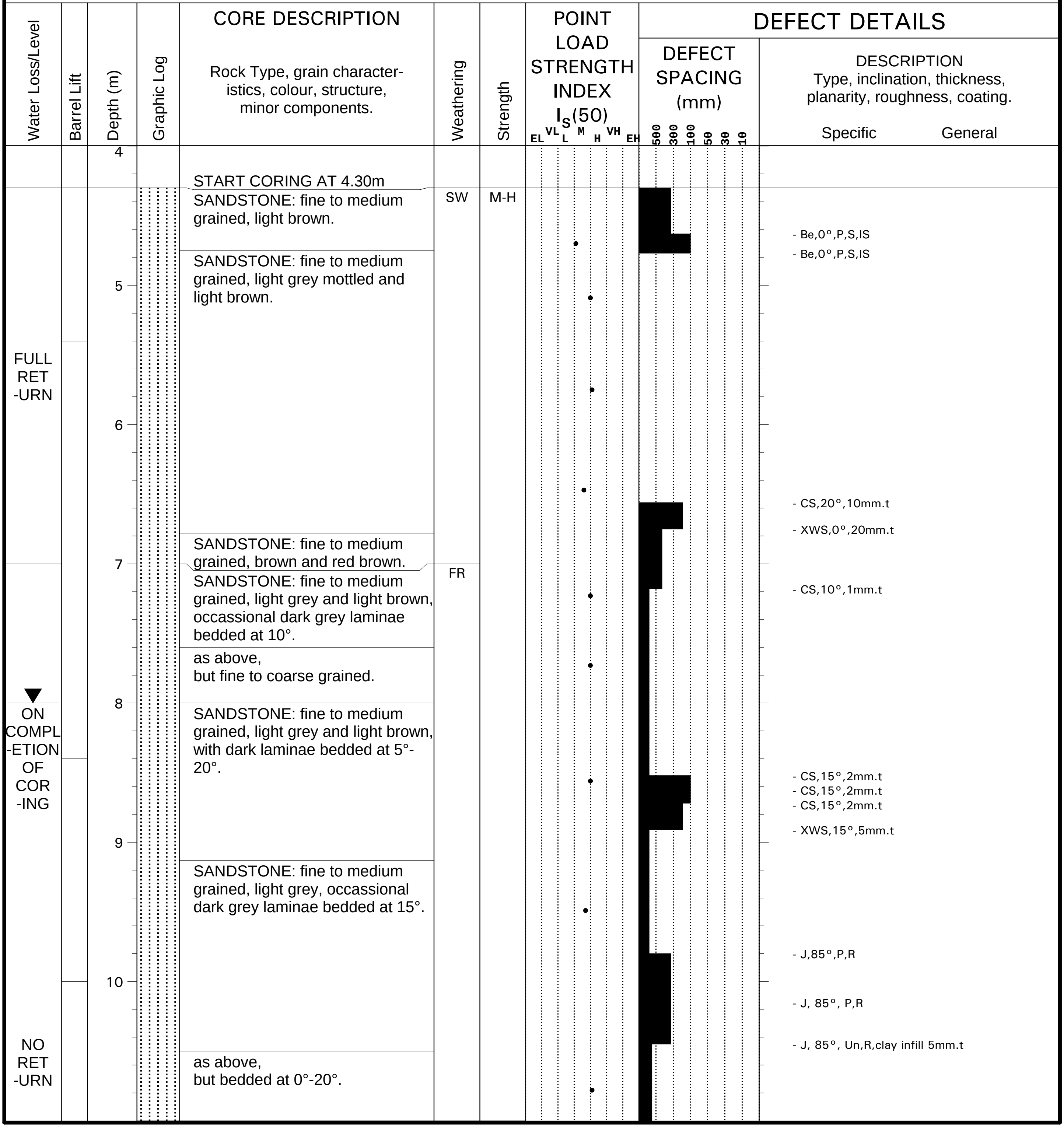
Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPL -ETION OF AUGER -ING						0	■	-	ASPHALTIC CONCRETE: 40mm.t.	D			APPEARS POORLY COMPACTED
							▨		FILL: Silty sand, fine to medium grained, red brown, with fine to medium grained igneous gravel.	MC>PL			
	■			■	N = 3 3,1,2		▨		FILL: Silty sandy clay, low plasticity, brown, fine to medium grained sand, with fine to medium grained sandstone and igneous gravel, trace of brick fragments.				
						1	▨						
							▨		FILL: Silty clay, medium plasticity, light grey, with fine to medium grained shale and sandstone gravel.				
				■	N = 13 8,6,7		▨					170 210 210	APPEARS MODERATELY COMPACTED
						2	▨						
				■	N = 15 6,10,5	3	▨		FILL: Silty clay, brown with fine to medium grained sandstone gravel and sandstone cobbles and boulders.	MC>PL			
						4	⁙	-	SANDSTONE: fine to medium grained, light brown.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						5							
						6							
						7							

CORED BOREHOLE LOG

Borehole No.
4
2/3

Client: HAMMOND CARE
Project: PROPOSED CAR PARK
Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Core Size: NMLC
R.L. Surface: ≈ 37.8m
Date: 22-5-14
Inclination: VERTICAL
Datum: AHD
Drill Type: JK308
Bearing: -
Logged/Checked by: D.S./P.R.



Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific
	4	START CORING AT 4.30m			
FULL RETURN		SANDSTONE: fine to medium grained, light brown.	SW	M-H	- Be,0°,P,S,IS - Be,0°,P,S,IS
	5 6	SANDSTONE: fine to medium grained, light grey mottled and light brown.			- CS,20°,10mm.t - XWS,0°,20mm.t
	7	SANDSTONE: fine to medium grained, brown and red brown.			
		SANDSTONE: fine to medium grained, light grey and light brown, occassional dark grey laminae bedded at 10°.	FR		- CS,10°,1mm.t
▼		as above, but fine to coarse grained.			
ON COMPLETION OF CORING	8 9	SANDSTONE: fine to medium grained, light grey and light brown, with dark laminae bedded at 5°-20°.			- CS,15°,2mm.t - CS,15°,2mm.t - CS,15°,2mm.t - XWS,15°,5mm.t
	10	SANDSTONE: fine to medium grained, light grey, occassional dark grey laminae bedded at 15°.			- J,85°,P,R - J, 85°, P,R - J, 85°, Un,R,clay infill 5mm.t
NO RETURN		as above, but bedded at 0°-20°.			

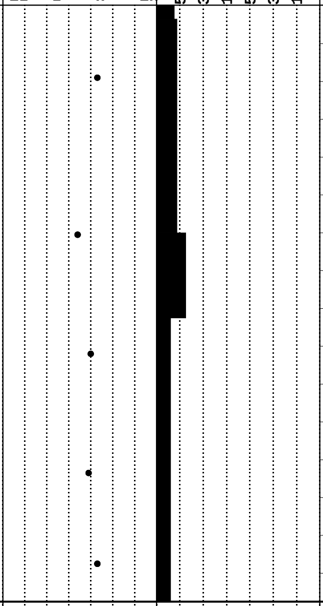
CORED BOREHOLE LOG

Borehole No.
4
3/3

Client: HAMMOND CARE
Project: PROPOSED CAR PARK
Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Core Size: NMLC
R.L. Surface: ≈ 37.8m
Date: 22-5-14
Inclination: VERTICAL
Datum: AHD
Drill Type: JK308
Bearing: -
Logged/Checked by: D.S./P.R.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific General
		12 13 14		SANDSTONE: fine to medium grained, light grey, with dark grey laminae bedded at 0°-20°.	FR	M-H			- J,85°, P,R - J,83°,P,S,clay infill 15mm.t - CS,0°,10mm.t - CS,30°,0-15mm.t
		15 16 17		END OF BOREHOLE AT 14.15m					




JK Geotechnics
JOB No: 23789ZR BH4 START CORING AT 4.90m
4
5
6
7
8
9
10
11
12
13
14
END BH AT 14.16m

BOREHOLE LOG

Borehole No.
5
1/1

Client: HAMMOND CARE
Project: PROPOSED CAR PARK
Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Date: 23-5-14
Method: SPIRAL AUGER
JK308
R.L. Surface: ≈ 37.8m
Datum: AHD
Logged/Checked by: D.S./P.R.

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 COMPL -ETION AND AFTER 6HRS						0	■	-	ASPHALTIC CONCRETE: 30mm.t.	MC<PL			APPEARS MODERATELY COMAPCTED
	■			■	N = 10 4,6,4	1	▨		FILL: Silty sandy clay, low plasticity, light grey, fine to medium grained sand, trace of fine to coarse grained shale gravel.			250 250 210	
				■	N = 6 3,4,2	2	▨		FILL: Silty clay, low plasticity, orange brown, with fine to medium grained sand, fine to medium grained ironstone gravel, trace of fine to coarse grained igneous gravel.			220 250 220	
				■	N = 6 2,2,4	3	▨		FILL: Silty sandy clay, low plasticity, light grey mottled orange brown, fine to medium grained sand, with fine to coarse grained sandstone and ironstone gravel.	MC>PL		350 350 420	
	■			■		4	▧	CL	SILTY CLAY: medium plasticity, orange brown, trace of fine to medium grained ironstone gravel.	MC>PL	St	150 150 150	
				■	N = SPT 8/50mm REFUSAL		⁝	-	SANDSTONE: fine to medium grained, light grey mottled light brown.	SW	M		MODERATE 'TC' BIT RESISTANCE
						5 6 7			END OF BOREHOLE AT 5.0m				

BOREHOLE LOG

Borehole No.
6
1/3

Client: HAMMOND CARE

Project: PROPOSED CAR PARK

Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR **Method:** SPIRAL AUGER & WASHBORING JK308 **R.L. Surface:** ≈ 37.5m

Date: 23-5-14 **Datum:** AHD

Logged/Checked by: D.S./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPL -ETION OF AUGER -ING						0		-	ASPHALTIC CONCRETE: 30mm.t	D			
									FILL: Silty sand, fine to medium grained, red brown, trace of fine to medium grained igneous gravel.	MC<PL			APPEARS POORLY COMPACTED
				■	N = 4 2,2,2	1			FILL: Silty sandy clay, low plasticity, brown, fine to medium grained sand, with fine to medium grained sandstone and igneous gravel.			280 250 280	
				■	N >9 2,3,6/ 100mm REFUSAL	2			FILL: Silty sandy clay, low plasticity, brown, fine to medium grained sand, with fine to medium grained sandstone gravel.			350 350 400	APPEARS MODERATELY COMPACTED
				■	N = 6 4,3,3	3			FILL: Silty gravelly clay, low plasticity, grey, fine to coarse grained shale gravel.			150 120 200	COMMENCE WASH BORE DRILLING
				■	N = 8 3,3,5	4 5			FILL: Gravelly clayey sand, fine to medium grained, light grey and light brown, fine to medium grained sandstone and ironstone gravel.	D			APPEARS MODERATELY COMPACTED
▼ ON COMPL -ETION OF COR -ING				■	N = 7 3,3,4	6 7		CL	SANDY CLAY: medium plasticity, light grey, fine to medium grained sand, with fine to medium grained ironstone gravel.	MC<PL	VSt	350 350 350	

BOREHOLE LOG

Borehole No.
6
2/3

Client: HAMMOND CARE
Project: PROPOSED CAR PARK
Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR
Date: 23-5-14
Method: SPIRAL AUGER & WASHBORING JK308
R.L. Surface: ≈ 37.5m
Datum: AHD
Logged/Checked by: D.S./P.R.

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
							▨	CL	SANDY CLAY: medium plasticity, light grey, fine to medium grained sand, with fine to medium grained ironstone gravel.	MC<PL	VSt		
				■	N >21 4,21/ 150mm REFUSAL		⁘	-	SANDSTONE: fine to medium grained, light grey mottled light brown	DW	L		LOW 'TC' BIT RESISTANCE
						8			REFER TO CORED BOREHOLE LOG				
						9							
						10							
						11							
						12							
						13							
						14							

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
6
3/3

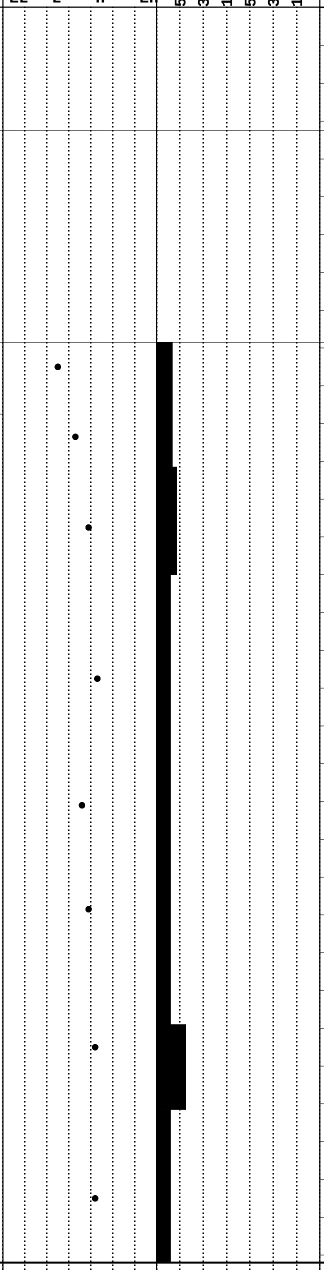
Client: HAMMOND CARE

Project: PROPOSED CAR PARK

Location: GREENWICH HOSPITAL, RIVER ROAD, GREENWICH, NSW

Job No. 23789ZR	**Core Size:** NMLC	**R.L. Surface:** ≈ 37.5m
Date: 23-5-14	**Inclination:**	**Datum:** AHD
Drill Type: JK308	**Bearing:**	**Logged/Checked by:** D.S./P.R.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		7		START CORING AT 7.65m						
		8		CORE LOSS: 1.12m						
		9		SANDSTONE: fine to coarse grained, light grey and light brown.	DW	L				
				as above, but light grey, occassional dark grey laminae bedded at 0°-20°.	SW	M			- CS, 20°,1mm.t	
		10							- CS,20°, 3mm.t	
		11							- XWS,0°,80mm.t	
		12							- CS, 0°,1mm.t	
		13							- CS,5°,5mm.t	
				END OF BOREHOLE AT 13.64m						



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JK Geotechnics
JOB No: 23789ZR BH 6 START CORING AT 7.55m
7
8
CORE LOSS 1.12m
9
10
11
12
13
END BH AT 13.64m

NVIRONMENTAL LOG

Borehole No.
7

ironmental logs are not to be used for geotechnical purposes

lient: HAMMOND CARE
roject: WASTE CLASSIFICATION
ocation: GREENWICH HOSPITAL

ob No. E23789K **Method:** HAND AUGER **R.L. Surface:** N/A
ate: 21-5-14 **Datum:**
Logged/Checked by: R.M./

Record	SAMPLES ES	ASS	ASB	SAL	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
Y ON IPLET- ON	■		■			0	▨		FILL: Silty clay, brown, with brick fragments, plastic, roots and organic matter.	MC>PL			
						0.5			END OF BOREHOLE AT 0.5m				HAND AUGER REFUSAL
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

VIRONMENTAL INVESTIGATION SERVICES
ISULTING ENVIRONMENTAL ENGINEERS

EIS

Borehole No.
8

NVIRONMENTAL LOG

ronmental logs are not to be used for geotechnical purposes

:lient: HAMMOND CARE
'roject: WASTE CLASSIFICATION
.ocation: GREENWICH HOSPITAL

ob No. E23789K
Method: HAND AUGER
R.L. Surface: N/A
)ate: 21-5-14
Datum:
Logged/Checked by: R.M./

Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	ASS	ASB	SAL									
Y ON PLET-ON	■		■			0 0.5	▨		FILL: Silty clay, medium plasticity, brown, trace of ironstone and shale gravel, metal fragments, plant roots and organic matter.	MC>PL			
						1 1.5 2 2.5 3 3.5			END OF BOREHOLE AT 0.6m				HAND AUGER REFUSAL

NVIRONMENTAL LOG

Borehole No.
9

ronmental logs are not to be used for geotechnical purposes

:lient: HAMMOND CARE
'roject: WASTE CLASSIFICATION
.ocation: GREENWICH HOSPITAL

ob No. E23789K
Method: HAND AUGER
R.L. Surface: N/A
)ate: 21-5-14
Datum:
Logged/Checked by: R.M./

Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	ASS	ASB	SAL									
Y ON PLET- ON						0	▨		FILL: Silty clay, medium plasticity, brown, with organic material and roots, trace of gravel and sand.	MC>PL			
	■		■			0.5	▨		FILL: Silty clay, medium to high plasticity, grey.	MC≈PL			
						1 1.5 2 2.5 3 3.5			END OF BOREHOLE AT 0.6m				HAND AUGER REFUSAL

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.

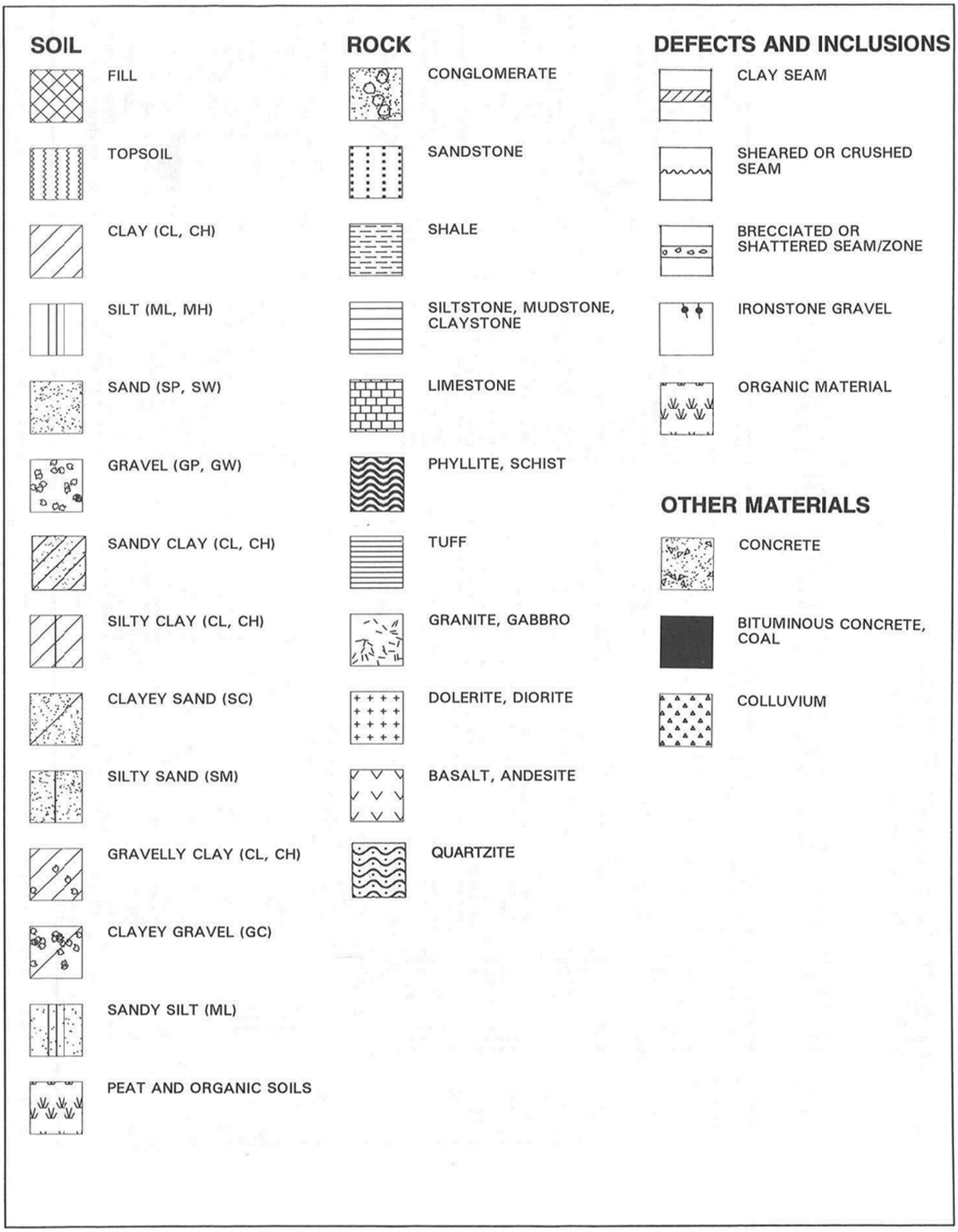
SITE INSPECTION

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



SOIL

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

ROCK

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

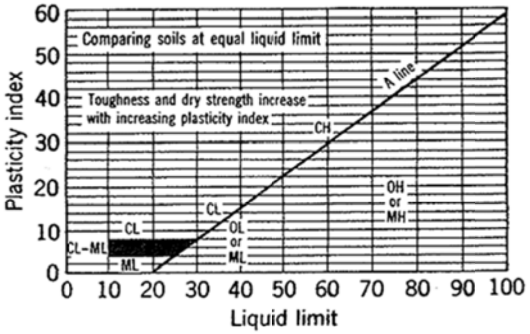
DEFECTS AND INCLUSIONS

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

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

<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 character-istics)</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.

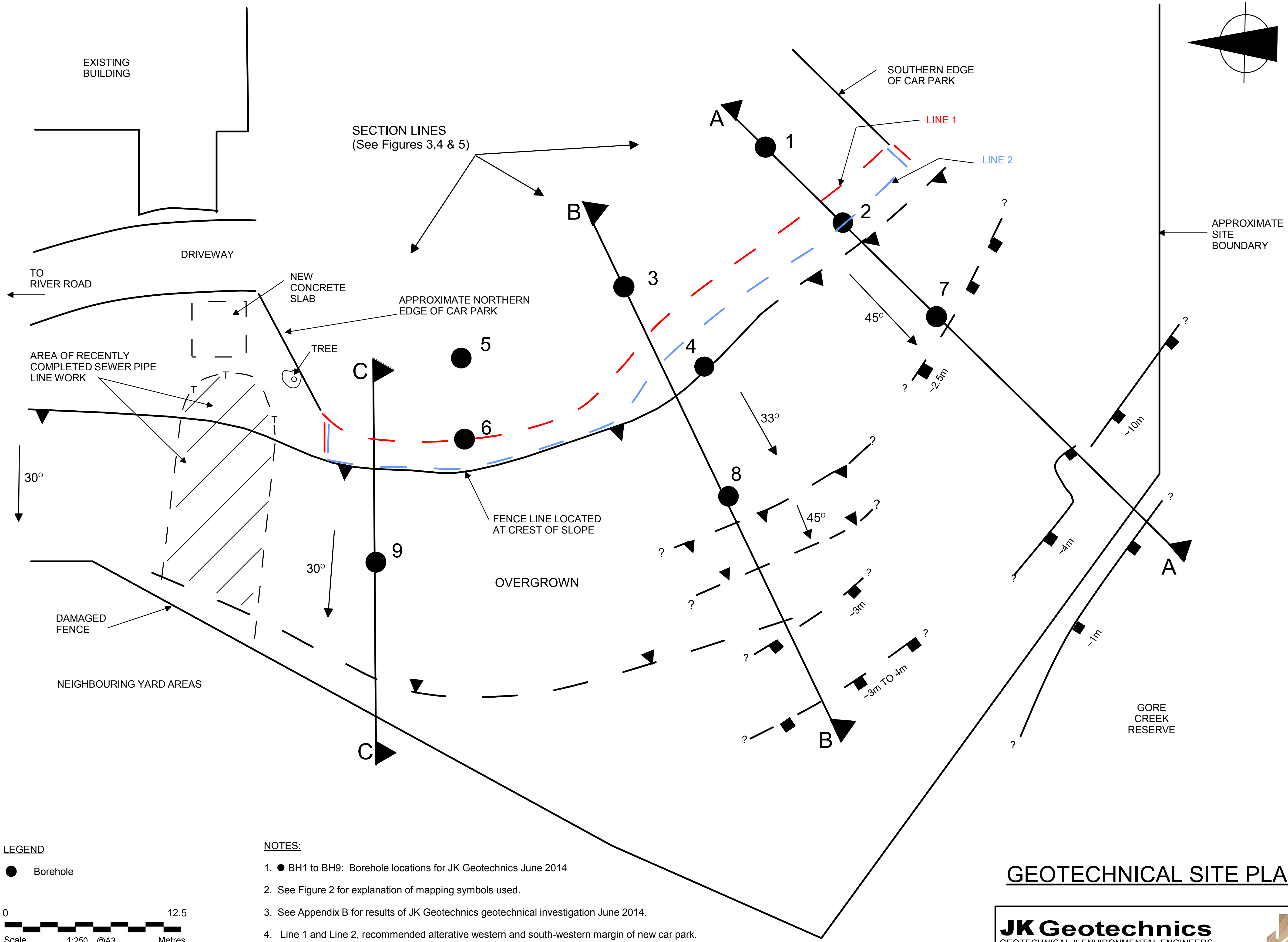
ROCK STRENGTH

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	



LEGEND

● Borehole

0 12.5
Scale 1:250 @A3 Metres

NOTES:

1. ● BH1 to BH9: Borehole locations for JK Geotechnics June 2014
2. See Figure 2 for explanation of mapping symbols used.
3. See Appendix B for results of JK Geotechnics geotechnical investigation June 2014.
4. Line 1 and Line 2, recommended alterative western and south-western margin of new car park.

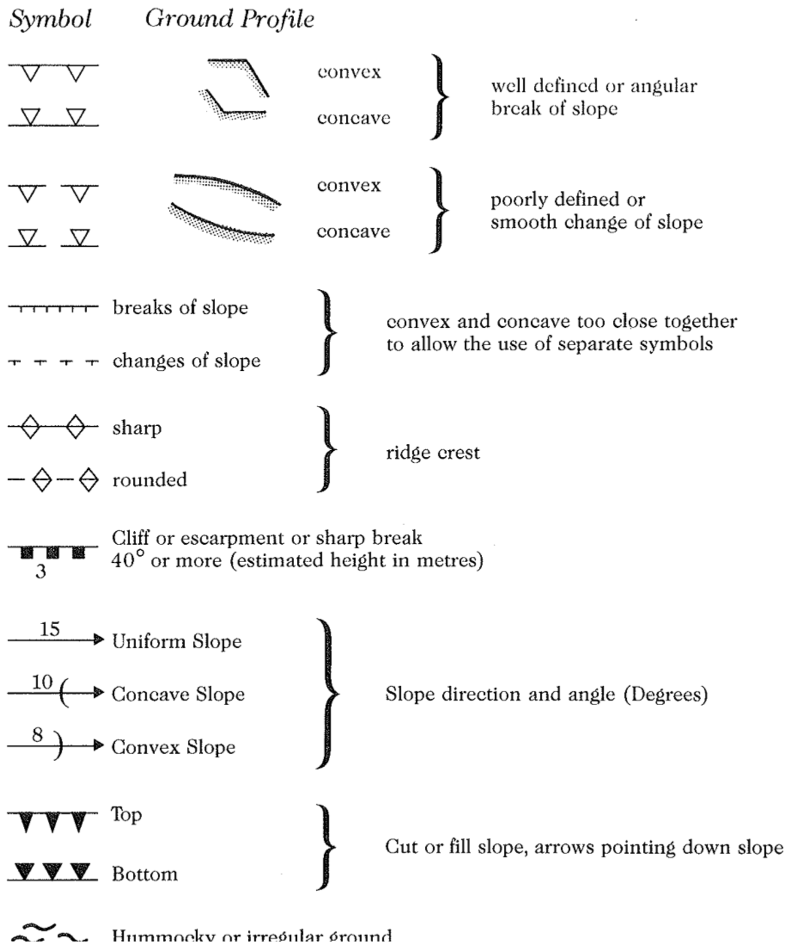
GEOTECHNICAL SITE PLAN

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

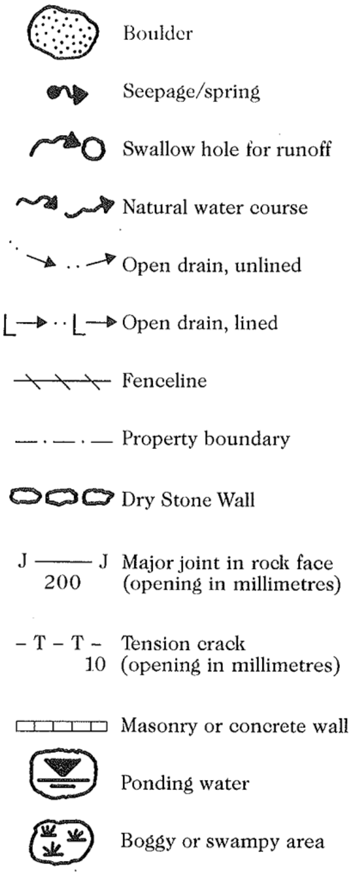
Report No. 23789ZR Figure No. 1

GEOTECHNICAL MAPPING SYMBOLS

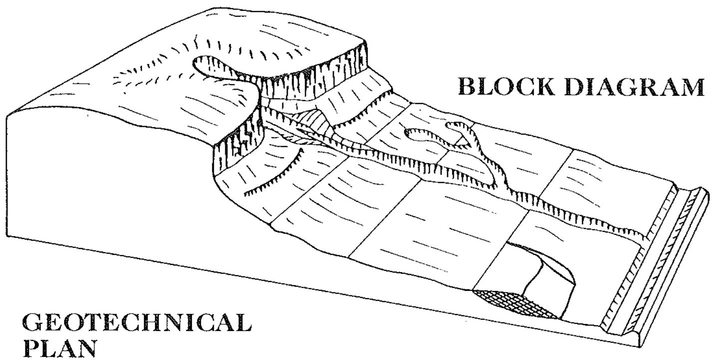
TOPOGRAPHY



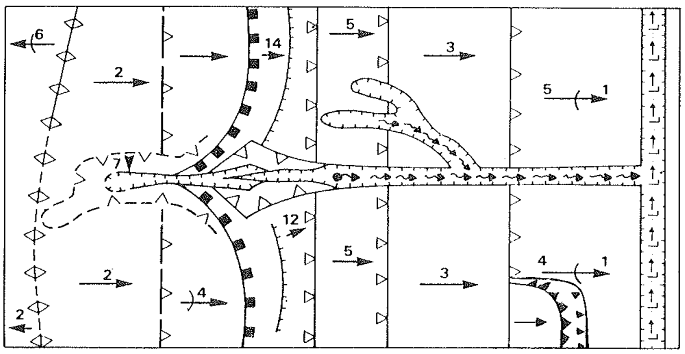
OTHER FEATURES



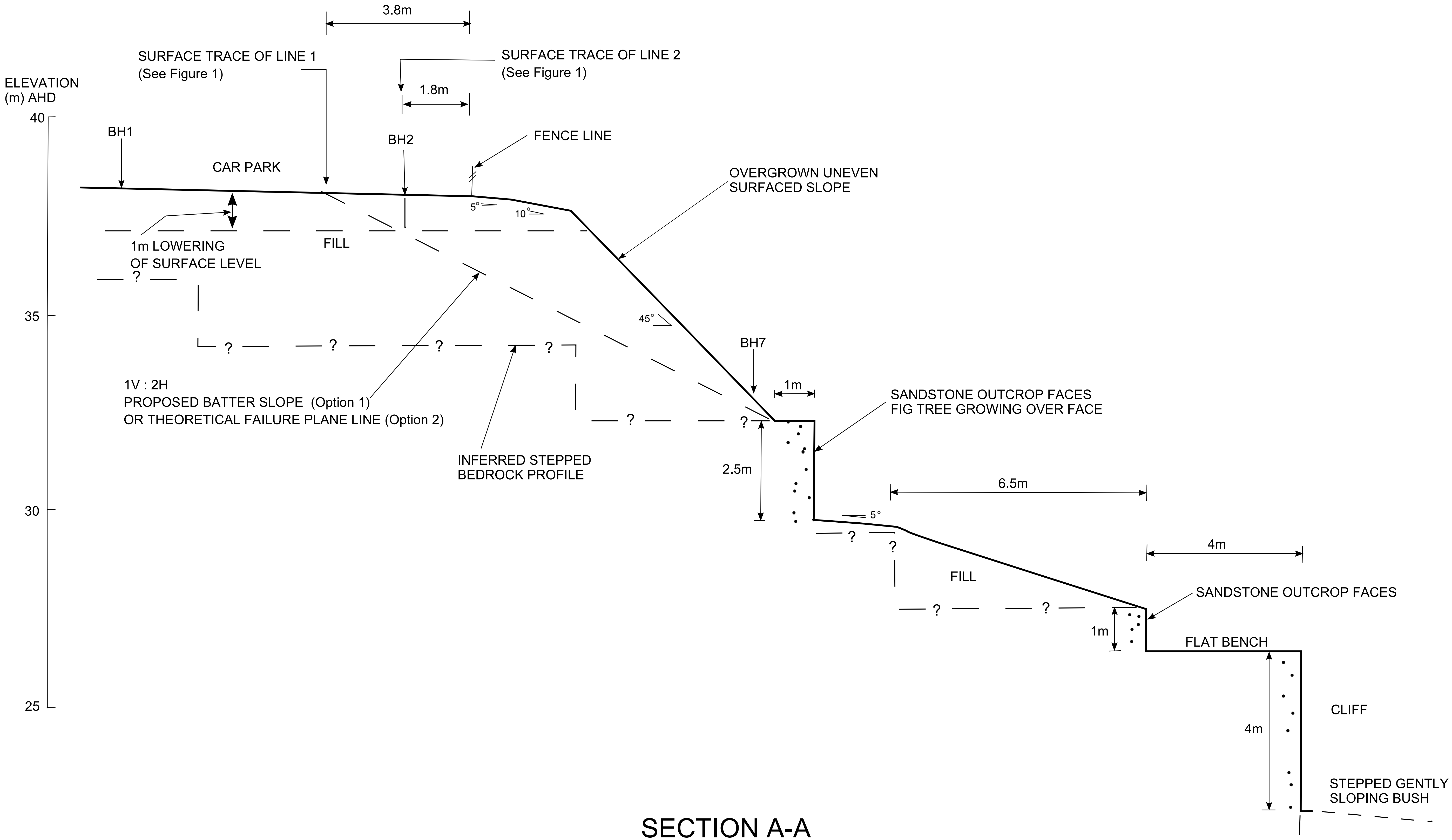
EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



GEOTECHNICAL PLAN



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



SECTION A-A
CROSS SECTIONAL SKETCH
(LOOKING SOUTH)

SCALE (m)

0 5

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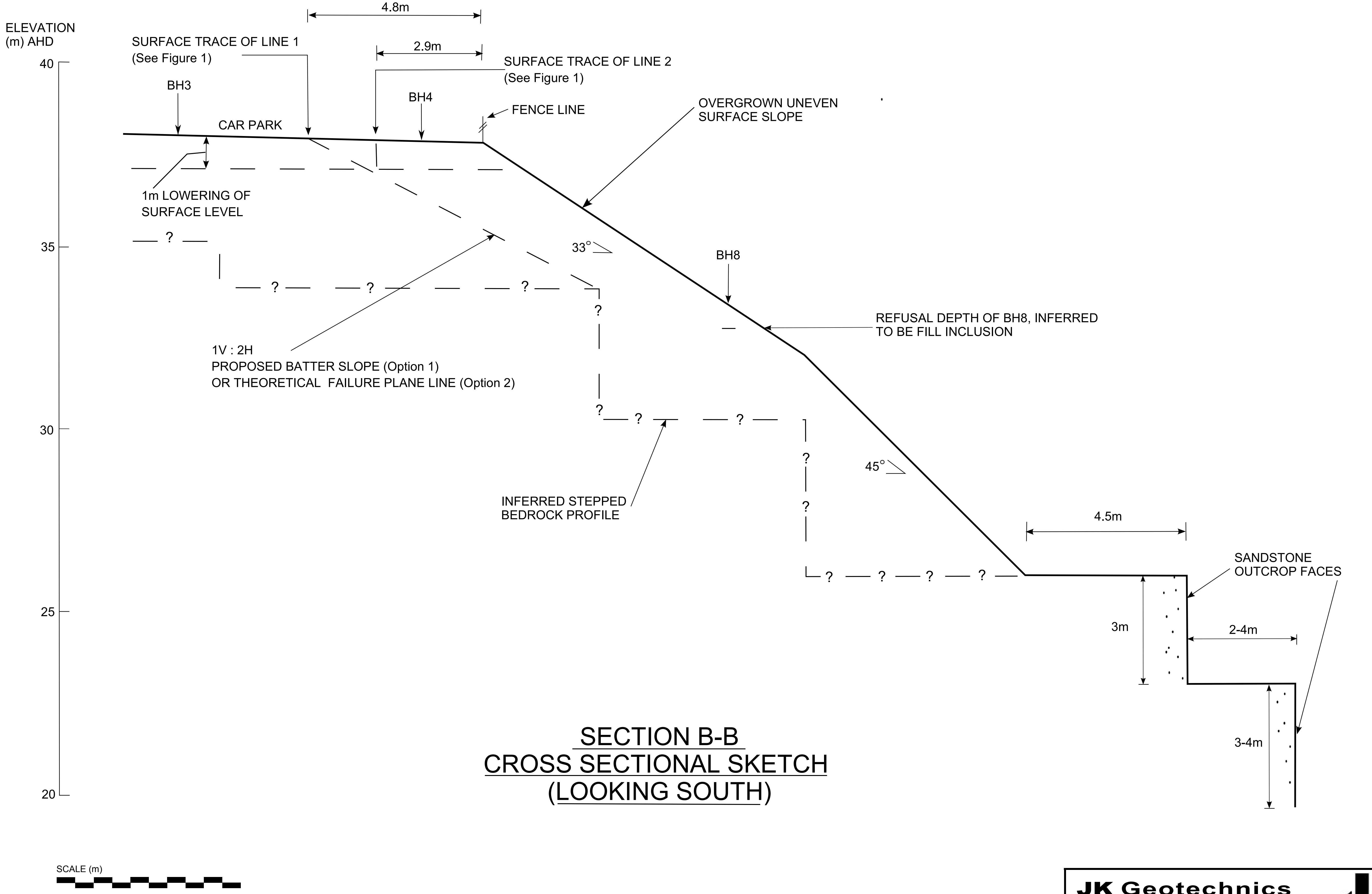
Figure No. 3



SECTION B-B
CROSS SECTIONAL SKETCH
(LOOKING SOUTH)

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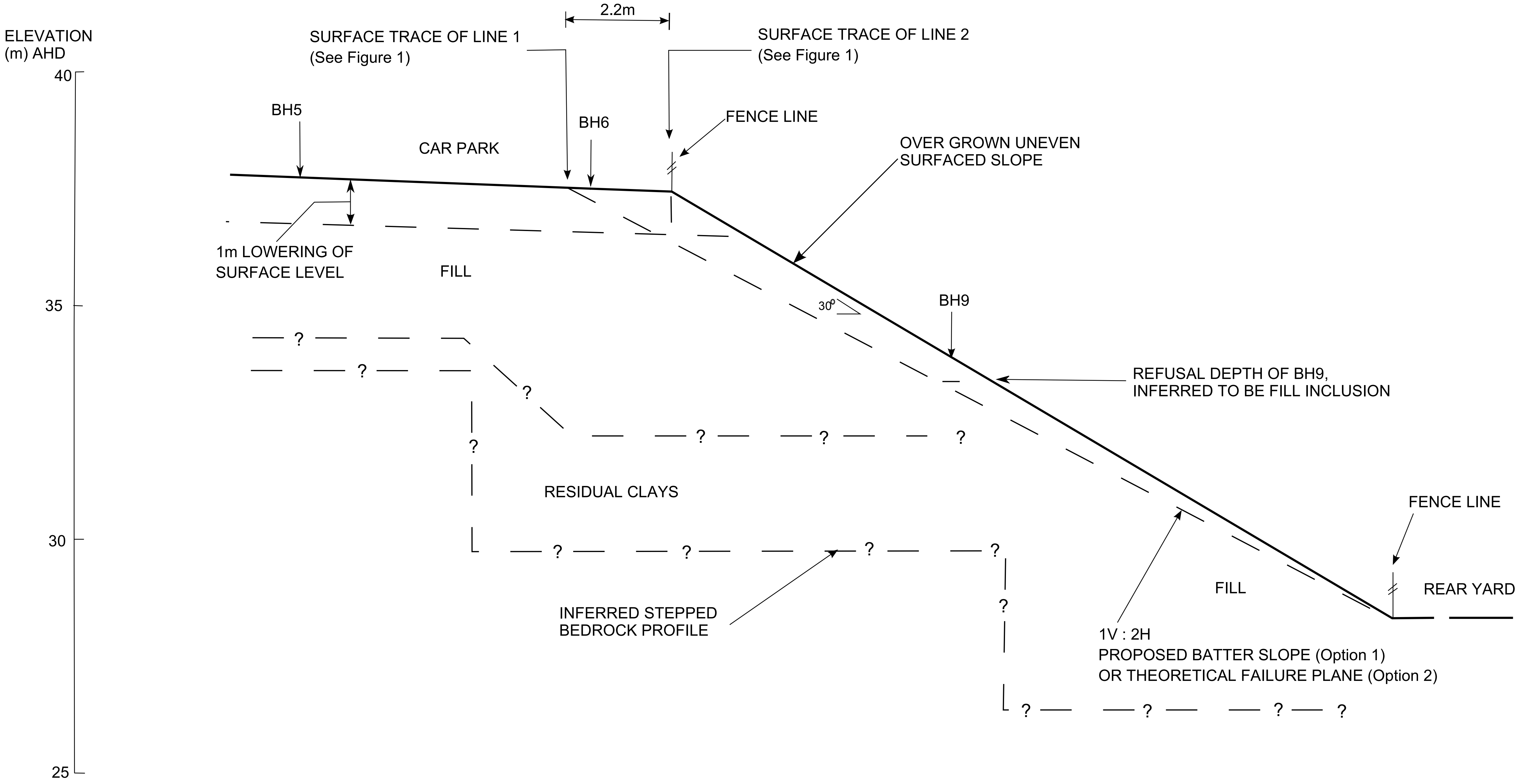
Report No 23789ZR Figure No. 4



SECTION C-C
CROSS SECTIONAL SKETCH
(LOOKING SOUTH)

SCALE (m)
0 5

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Report No 23789ZR Figure No. 5