NSW HEALTH

Liverpool Hospital Redevelopment - Stage 2

Infrastructure and Ancillary Hospital Works

Project Application and
Environmental Assessment

Appendix N

Geotechnical Investigation Proposed Bridges and Multi Storey Car Park

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For:	Department of Planning
On behalf of :	NSW Health

October 2008

REPORT

TO

CAPITAL INSIGHT PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED BRIDGES AND MULTI-STOREY CAR PARK

AT

LIVERPOOL HOSPITAL, NSW

9 August 2007

Ref: M21170ZArpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed bridges and multi-storey car park at Liverpool Hospital, NSW. The investigation was commissioned by Mr Jeremy Wilson of Capital Insight Pty Ltd by email dated 23 April 2007. The commission was on the basis of our fee proposal, Ref: PM372ZA ("*New Bridges*" and "*Multi-Storey Car Park*") dated 4 December 2006.

Based on the site meeting attended by Mr Jeremy Wilson and Messrs Andrew Jackaman and Adrian Kingswell of Jeffery and Katauskas Pty Ltd (J&K) on 14 November 2006, an unreferenced concept plan showing the "*Stage 1 Construction*" area which was supplied to us at that meeting, and a supplied plan titled "*Revised M Entry Concept Plan, 23 April 2007, Rev. to Incld SubStn*", we understand that the proposed development will comprise the following works:

1. Demolition of the existing single storey "Staff Recreation" building, swimming pool, tennis court, BBQ facility and pavements which are located immediately on the south-eastern side of the Main Southern Railway line.
2. Construction of a road bridge and a pedestrian bridge over the Main Southern Railway line. The architectural and structural designs of the bridges have not yet been finalised. We have assumed that both bridges will be single span. The locations of the proposed bridges are shown on the attached Figure 1.
3. Construction of a multi-storey car park on the south-eastern side of the Main Southern Railway line. The architectural details have not yet been finalised. The outline of the proposed multi-storey car park is shown on Figure 1.

The purpose of the investigation was to assess the subsurface conditions at two borehole locations and at nine Electronic Friction Cone Penetration (EFCP) test locations. Based on the information obtained, we present our comments and recommendations on site earthworks, footings, slab-on-grade, soil aggression and earthquake design parameters.

During the course of the fieldwork, Liverpool experienced a prolonged heavy rainfall period. As a result, the ground surface "softened" thus restricting access to one of the test locations. As such, only six EFCP tests were completed for the proposed multi-storey car park. Nonetheless, an additional EFCP test was completed adjacent to one of the bridge boreholes in order to provide correlation with the borehole information.

We were also commissioned to carry out an environmental site assessment at Liverpool Hospital. This work was carried out by Environmental Investigation Services (EIS) [the environmental consulting division of J&K] who prepared a report, Ref: E21171FK-RPT dated June 2007. This geotechnical report must be read in conjunction with the EIS report. Three additional environmental boreholes (P7, P8 & P9) were drilled for the environmental assessment. The logs of these additional boreholes have not been referred to in this report as they were not prepared nor drilled to sufficient depths for geotechnical purposes.

Since 1989, J&K has completed several geotechnical investigations at Liverpool Hospital. The relevant previous report pertaining to the current proposed development is report Ref: M20303ZArpt dated 13 July 2006. This report was commissioned by the Department of Commerce for the proposed Liverpool Hospital Redevelopment Project, which was at concept stage. However, it was envisaged at that time that the proposed redevelopment would comprise the demolition of some of the older existing buildings and the construction of new buildings and extensions (some with a single basement car parking level), a multi-storey car park, a road bridge and a separate pedestrian spanning over the Main Southern Railway.

The 2006 investigation essentially comprised the drilling of eight boreholes to depths between 6.05m and 23.72m. The bedrock in all eight boreholes was diamond core drilled. The relevant boreholes include BH1001, BH1002 and BH1004 and their logs are attached in Appendix A. The borehole locations have been plotted onto Figure 1. These previous boreholes have been referred to in the preparation of this current report.

2 INVESTIGATION PROCEDURE

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

The fieldwork for the current investigation was carried out on 26 & 27 April 2007 and 23 & 24 May 2007 and comprised two boreholes and nine EFCP tests as outlined below. The test locations are shown on Figure 1.

Borehole Investigation

Two boreholes (B1 & B2) were drilled to depths of 18.83m and 18.58m below existing grade, respectively. The boreholes were initially auger drilled using our truck mounted JK550 drill rig. At 16.16m depth in B1 and 15.97m in B2, the boreholes were extended into the bedrock by rotary diamond coring techniques, using an NMLC triple tube core barrel with water flush.

The relative compaction/strength of the subsoil profile was assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on clayey soils recovered in the SPT split spoon sampler. The strength of the underlying bedrock was assessed by observation of auger penetration resistance, together with examination of recovered rock cuttings. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests.

Groundwater observations were also made in the boreholes.

EFCP Investigation

Nine EFCP tests (B3, B4, P1 to P6, & P10) were completed to refusal depths between 6.8m and 19.3m below existing grade, using our specialised 18 tonne truck mounted EFCP rig. The concrete pavement surface at B3 was diatube cored with water flush. We note that EFCP testing does not provide sample recovery. EFCP testing does, however, provide a continuous plot of the subsoil profile. The subsurface material identification, including material strength/relative density, is by interpretation of the test results based on nearby borehole information (particularly B2) and by empirical correlations. On completion of testing, the EFCP probe holes collapsed to within 1m depth and as such, no meaningful groundwater observations could be made.

The test locations were set out by tape measurements from existing site features. The surface reduced levels indicated on the attached borehole logs and EFCP test results were interpolated between spot level heights shown on the supplied survey plans prepared by John M Daly & Associates Pty Ltd (Ref: 06321DS, Sheets 10, 11 & 12, Issue A, dated 12/2/07), and are therefore only approximate. The survey datum is the Australian Height Datum (AHD). Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineers were present full-time during the fieldwork, to set out the test locations, nominate testing and sampling, to prepare the attached geotechnical borehole logs and to direct the EFCP testing. The Report Explanation Notes define the logging terms and symbols used.

The recovered rock cores from B1 and B2 were photographed and returned to a NATA registered laboratory [Soil Test Services Pty Ltd (STS)] for Point Load Strength Index testing. The photographs are enclosed facing the relevant cored borehole logs.

The Point Load Strength Index test results are plotted on the borehole logs and are also summarised in the attached Table A. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table A.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located immediately either side of the Main Southern Railway line within the Liverpool Hospital grounds, in relatively flat topography. The proposed multi-storey car park will be located in the eastern hospital grounds.

At the time of the fieldwork, the site was generally occupied and surrounded by single storey buildings of brick and/or light weight clad frame construction. Surrounding and between these buildings were concrete and asphaltic concrete (AC) pavements, concrete footpaths, lawns, garden beds, shrubs and scattered trees. Within the proposed multi-storey car park footprint was an in-ground swimming pool and tennis court, as well as a 1m high fill embankment at its north-eastern end.

To the north-east of the proposed multi-storey car park footprint was a steel frame and aluminium clad (Central Energy) building, with steel gas cylinders up to approximately 14m high on its western side. To the west of this building, adjacent to the Main Southern Railway line, was a group of liquid oxygen cylinders up to approximately 7m high.

To the north of the north-western end of the proposed road bridge was a three storey (Ron Dunbier) building which was of concrete frame and brick wall construction. We understand that the western half of this building will be demolished to accommodate the new roadway which will lead onto the proposed bridge.

We understand that a services tunnel extends in a westerly direction from the Central Energy building in the eastern hospital grounds, below the Main Southern Railway, into the western hospital grounds. The location of the services tunnel, as obtained from the survey plans, is shown on Figure 1. The details of the services tunnel (eg. width, invert level, etc.) are unknown. Based on our experience at the hospital, we note that there are a significant number of undocumented buried services which pass below the subject site.

3.2 Subsurface Conditions

The 1:100,000 Series Geological Map of Penrith indicates the site to be underlain by Tertiary alluvium associated with the nearby Georges River.

Generally, the current boreholes and EFCP tests encountered/indicated pavements and/or fill overlying variable and interlayered alluvial soils, then shale bedrock. Reference should be made to the attached borehole logs for details at each specific location. A graphical borehole summary, including BH1001, BH1002 and BH1004 from our previous report, Ref: M20303ZArpt, is presented as Figure 2. A summary of the subsurface characteristics encountered in the boreholes and indicated by the EFCP testing is provided below.

Pavements

A 130mm thick concrete pavement was encountered at B3.

Fill

Fill, comprising clayey and sandy soils, was encountered in all boreholes and indicated by all EFCP tests to depths between 0.2m (B4, P1 & P4) and 1.9m (P2) below existing grade. At (boreholes) B1 and B2, inclusions of ironstone and quartz gravel, and slag fragments were encountered in the fill. Based on the SPT and EFCP test results, the fill was generally assessed to be variably compacted, however, poor compaction was indicated in B3, P5, in the basal profile of P2, and the upper profile of P3.

Alluvial Soils

Variable and interlayered alluvial soils, comprising silty clay, sandy clay, clayey sand, silty sand and sand, were encountered/indicated below the fill in all borehole and EFCP test locations. In boreholes B1 and B2, the alluvial clays were of high plasticity. The strength of the alluvial clays was generally stiff to hard, as encountered by the boreholes and indicated by the EFCP test results. The alluvial sands were generally medium dense to dense, with some localised loose bands.

It is possible that the upper alluvial clay profile in P1 to 1.3m depth and in P5 to 1.5m depth is existing fill. However, based on the limitations of the EFCP, this could not be confirmed.

The EFCP tests also indicated localised "pockets" of very loose alluvial sands in B3, P1, P2 and P3 at varying levels. The thickness of these very loose sand bands ranged from 0.7m to 1.4m.

The EFCP test at P2 also indicated a 0.1m thick firm alluvial clay band at 9.4m depth and a 0.7m thick firm to stiff alluvial clay band at 10.0m depth. At P3, a 50mm thick very soft peat seam was indicated at 10.2m depth.

All EFCP tests terminated within the alluvial soil profile.

Shale Bedrock

Shale bedrock was encountered in B1 and B2 at depths of 16.16m and 14.9m, respectively. In the current boreholes, the shale bedrock was generally fresh and of medium and high strength. The upper shale profile in B2, however, was extremely weathered and of extremely low strength. This upper "weak" profile in B2 was 1.1m thick. The diamond cored portions of the boreholes encountered only a few rock defects (ie. joints).

An engineering classification of the shale bedrock (in accordance with Pells et al. 1978, as revised by Pells et al. 1998) has been carried out for B1 and B2, and also for BH1001, BH1002 and BH1004 from our previous investigations, and is tabulated below.

Borehole	Approx. Surface RL (m) AHD	Indicative Engineering Classification of Shale Bedrock Depths (m)				
		Class V	Class IV	Class III	Class II	Class I
B1	9.4	-	-	16.16-16.4	-	16.4-18.83
B2	9.8	14.9-16.0	-	- 17.2-18.58	-	16.0-17.2
BH1001	10.0	15.55-16.25	-	16.25-16.7	-	16.7-19.25
BH1002	9.7	15.9-16.2	-	16.2-17.5	-	17.5-19.28
BH1004	9.5	-	-	18.8-19.7	-	19.7-20.95

Groundwater

Groundwater observations were made in boreholes B1 and B2 during and on completion of augering at depths between 8.6m and 9.3m. We note that the groundwater levels may not have stabilised within the limited observation period. No groundwater monitoring was carried out.

3.3 Laboratory Test Results

3.3.1 Current Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores correlated well with our field assessment of bedrock strength. The estimated UCSs in the cored rock portions of B1 and B2 generally ranged from 12MPa to 22MPa, however, a value as high as 36MPa was indicated in the upper cored profile of B1.

3.3.2 Previous Results

The estimated UCSs in the cored rock portions of BH1001, BH1002 and BH1004 generally ranged from 14MPa to 34MPa, however, a value as high as 46MPa was indicated in the lower cored profile of BH1004.

The results of the previous chemical soil tests carried out on alluvial clay samples from BH1001 and BH1002, and on an alluvial sand sample from BH1004, are tabulated below.

Borehole	Sample Depth (m)	Soil pH	Soil Sulphate (mg/kg)	Soil Chloride (mg/kg)
BH1001	1.3-1.75	5.5	132	155
BH1002	2.0-2.4	5.1	53	118
BH1004	3.5-4.0	5.6	21	<100

4 COMMENTS AND RECOMMENDATIONS

As the proposed development is at concept stage, the following comments and recommendations are generalised and will need to be reassessed once the development details have been finalised.

4.1 Existing Buried Services

We strongly recommend that a detailed services search be carried out for the proposed site area. The locations of many existing buried services are unknown to the hospital maintenance staff, as was experienced during the set out of previous and current boreholes. The details should then be plotted onto the survey plan for future reference.

4.2 Multi-Storey Car Park

4.2.1 Site Earthworks

All earthworks recommendations provided for the proposed multi-storey car park should be complemented by reference to AS3798-2007 ("Guidelines on Earthworks for Commercial and Residential Developments").

4.2.1.1 **Existing Fill**

No details on the existing fill (ie. placement method, compaction specification, density test records, etc.) have been provided to us. The fill was assessed to be variably compacted, and in the case of B3, P2, P3 and P5, poor compaction was indicated. As such, we consider the existing fill to be "uncontrolled".

Based on the results of the investigation, the existing fill subgrade will probably be suitable to support slabs-on-grade on condition that the subgrade preparation works, as outlined in Section 4.2.1.4 below are carried out. However, we suggest that a generous allowance be made in the contract budget and program for replacement/bridging of poorly compacted fill, which will most likely heave/subside during the proof rolling inspection.

4.2.1.2 **Existing Trees**

Trees were scatted throughout the proposed multi-storey car park footprint and also lined the south-eastern side of the Main Southern Railway line. We note that the existing trees have likely caused localised "drying out" of the surrounding clayey soils. Removal of the trees will therefore lead to the recovery of the soil moisture content, resulting in differential swell movements in the vicinity of the trees. The swell movements generated by the removal of the trees are in addition to the shrink-swell movements, which can occur in the clayey soils due to weather related natural moisture changes and by the reduction in surface evaporation subsequent to covering the site with buildings and slabs. The latter shrink/swell movements are outlined in AS2870-1996 ("Residential Slabs and Footings – Construction").

It is likely that moisture equilibrium in the clayey soils, following removal of the tree stumps and roots, could take one to two years to develop, possibly longer if the current "drought" persists. In order to reduce the effects that removal of the trees will have on the proposed building and slab areas, we recommend they be removed as early as possible ahead of construction.

4.2.1.3 **Site Drainage**

The clay subgrade at the site is expected to undergo substantial loss in strength when wet. Furthermore, the clay subgrade is expected to have a high shrink-swell reactive potential. Therefore, it is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade may become untraffickable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

4.2.1.4 **Subgrade Preparation**

Subgrade preparation will initially comprise:

- Demolition of the existing single storey brick "Staff Recreation" building, swimming pool, tennis court, BBQ facility and pavements;
- Removal of all trees (including their root balls);
- Stripping of all grass, topsoil, root-affected soil and any deleterious or contaminated existing fill;
- Stripping of the site down to design subgrade level; and,
- Possible re-routing of existing buried services.

Stripped topsoil and root affected soils should be stockpiled separately as they are not suitable for reuse as engineered fill. They may however be reused for landscaping purposes. Reference should be made to the EIS report for guidance on the offsite disposal of soil. Care should be taken during site stripping not to undermine or remove support from the adjoining railway corridor.

Following demolition of the swimming pool shell, we recommend that the sides of the localised excavation be battered back at no steeper than 1 Vertical (V) on 1 Horizontal (H) for stability considerations and to facilitate compaction of engineered fill up against the excavation sides, which should be benched in steps no more than 0.4m high. Surcharge loads should be kept well back from the crests of the temporary batter slopes.

Following stripping, we recommend that the exposed subgrade, including the base of the swimming pool excavation, be proof rolled with at least eight passes of a large static smooth drum roller (say, at least 15 tonnes deadweight). The vibratory mode on the roller should not be used due to the close proximity of nearby existing buildings and structures, and the need to limit ground borne vibrations and to maintain patient comfort. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas.

If the proposed multi-storey car park is to be supported on high level footings (refer to Section 4.2.2 below), then all deep fill and upper alluvial soils of limited bearing capacity (ie. very loose sands, firm and stiff clays), where it would be uneconomical to construct deep high level footings founded in the underlying competent alluvial soils, would need to be stripped and replaced with engineered fill. Based on the results of the investigation, we expect that the area in the vicinity of P1 would need to be stripped to 2.0m depth, the area in the vicinity of P2 to 3.3m depth, and the area in the vicinity of P4 to 1.5m depth.

Subgrade heaving during proof-rolling may occur in areas where the clayey soils may have become "saturated". Subgrade heaving should be expected in areas where surficial poorly compacted fill was indicated, such as in the vicinities of P3 and P5. Heaving areas should be locally removed to a stable base and replaced with engineered fill, as outlined below, or further advice could be sought. If existing trench backfill heaves during proof rolling, then it may need to be stripped to a certain depth and replaced with engineered fill. Bridging layer support may also be required. These subgrade stabilisation works must be confirmed and detailed during the proof rolling inspections.

If soil softening occurs after prolonged periods of rainfall, then the subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the clay subgrade exhibits shrinkage cracking, then the surface should be watered and rolled until the shrinkage cracks are no longer evident.

Engineered fill must be used to raise site levels up to design subgrade level.

4.2.1.5 **Engineered Fill**

The stripped clayey and sandy soils may be reused as engineered fill on condition that they are "clean", and free of organic matter and particle sizes greater than 75mm. In order to improve the workability of the stripped clayey soils, we recommend that the stripped sandy soils be mixed in with the clayey soils.

Engineered fill comprising stripped clayey soils should be compacted in maximum 200mm thick loose layers using a large static pad-foot roller to a density strictly between 98% and 102% of Standard Maximum Dry Density (SMDD) and a moisture content within 2% of Standard Optimum Moisture Content (SOMC).

Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved. The frequency of density testing for engineered fill should be at least one test per layer per $500m^2$, or one test per $100m^3$ distributed reasonably evenly throughout full fill depth and area, or three tests per visit, whichever requires the most tests. If high level footings are to be founded within the engineered fill layer, then we recommend that Level 1 testing of fill compaction be adhered to.

4.2.2 Footings

Based on the previous and current investigation results, the proposed multi-storey car park can be supported on high level pad and strip footings founded in very stiff or hard alluvial clays or in engineered fill (to Level 1 control), or on piles founded in the underlying shale bedrock.

We have considered supporting the proposed structure on shallow piles founded in the hard alluvial clays. However, the maximum allowable end bearing pressure would be limited to 600kPa, thus requiring large diameter piles and/or pile groups to support individual column loads. We have also considered supporting the proposed structure on piles founded in the underlying medium dense and dense alluvial sands at approximately RL 3m AHD, however, the limited thickness of this profile as indicated in B4, P1 and P2 would negate the suitability of this option.

4.2.2.1 **High Level Footings**

If the column loads are relatively light, say less than 1500kN, then it may be feasible to support the proposed new structure on high level pad and strip footings founded in very stiff or hard alluvial clays or in engineered fill (to Level 1 control). Such footings may be designed for a maximum allowable bearing pressure of 150kPa.

Due to the expected highly reactive nature of the clay soils, a minimum embedment depth of 0.8m should be adopted, assuming all external areas are fully paved. If any areas are unpaved then the embedment should be increased to 1.2m.

Large footings such as these are likely to settle significantly compared to piled footings. As such, mixed footing types are not recommended unless detailed analyses are completed to avoid potential differential settlement issues.

All high level footing excavations should be cleaned out of loose debris, inspected by a geotechnical engineer and poured without delay. If delays in pouring are envisaged, then we recommend that a concrete blinding layer be poured over the bases to reduce deterioration due to weathering.

4.2.2.2 **Piled Footings**

Due to the presence of sandy soils and groundwater, we recommend that the proposed new structure be supported on continuous flight auger (CFA) piles, which are also known as grout-injected auger piles. CFA piles socketed at least 0.3m into Class III or better quality shale may be designed for a maximum allowable end bearing pressure of 3500kPa. Sockets formed below the minimum 0.3m length requirement may be designed for a maximum allowable shaft adhesion value of 350kPa (compression).

The bearing pressures above are based upon a serviceability criterion of deflections at the pile toe of less than 1% of the pile diameter.

Only BH1004 was drilled within the proposed multi-storey car park footprint, where Class III or better quality shale was encountered at 18.8m depth (RL -9.3m AHD). Borehole B2 was drilled approximately 35m to the north-east of the proposed multi-storey car park footprint, where Class III or better quality shale was encountered at

16.0m depth (RL -6.2m AHD). At the south-western end of the proposed multi-storey car park footprint, EFCP tests P1 and P2 indicated a soil profile to depths of at least 19.3m (RL -9.4m AHD) and 18.5m (RL -8.7m AHD), respectively. We note that we have completed a previous borehole to the south-west of the proposed multi-storey car park footprint, behind the river bank crest of the Georges River. At this borehole, Class III or better quality shale was encountered at RL -10.4m AHD. The surface of the Class III or better quality shale appears to deepen in a south-westerly direction.

We therefore recommend that at least four additional boreholes be drilled to confirm the depth and quality of the shale bedrock once the architectural and structural designs have been finalised. The adoption of bearing pressures much greater than 3500kPa is likely to be feasible if a close grid of cored boreholes is completed as Class I shale was encountered in all the nearby boreholes. However, the cost of completing such a detailed investigation would most likely render this option unfeasible.

Piles on the shale bedrock may also be designed using "Limit State Design" principles as detailed in the paper "Foundations on Sandstone and Shale in the Sydney Region" by Pells, Mostyn and Walker (Australian Geomechanics, Number 33, Part 3, December 1998, Pages 17-29). For limit state design, an ultimate bearing capacity of 30MPa could be adopted for Class III shale bedrock at the site, provided that settlements to 5% of the pile diameter can be tolerated and an extensive pile test program is undertaken. It should be noted that such ultimate bearing pressures must be used in conjunction with an appropriate geotechnical strength reduction factor (ϕ_g) which is dependent upon:

- Both the amount and quality of information available for the founding layer;
- The quality of workmanship and control in the piling process, and;
- The quality of a pile test program.

The strength reduction factor should be selected following reference to Tables 4.1 and 4.2 of AS2159-1995 ("Piling – Design and Installation"). It must be understood that the use of limit state design to adopt relatively high bearing pressures (above the serviceability criteria described above) is not currently standard practice, and there is increased risk of inadequate performance of the piles.

The major limitation when using CFA piles is the maximum available diameter of the pile; usually 0.9m. For high column loads, pile groups may be required to support individual column loads. It is important to keep in mind that there are penetration limitations when using CFA piling rigs and it may not be possible to achieve long load bearing sockets into the Class III or better quality shale. We recommend that only high torque CFA piling rigs be brought to site and that the prospective piling contractor be provided with a full copy of this report.

At the commencement of pile drilling, we recommend that at least two test piles be drilled adjacent to our borehole locations, so that a correlation can be made between drilling penetration rates and the materials encountered in the boreholes. An experienced geotechnical engineer should witness the test pile drilling and initial stages of piling for the proposed extension.

Alternatively, larger diameter conventional bored piles drilled using a high torque rig could be considered, however, due to the presence of groundwater and saturated sandy soils, the piles would have to be drilled under bentonite or polymer fluids to prevent pile shaft collapse and/or be provided with temporary (or permanent) casing. Another option would be to use barrettes, where the shaft would be supported by bentonite or polymer fluids. These two footing options are expected to be costly. Concrete would have to be poured through a tremie. The piling contractors must advise as to the method and proposed equipment for pile/barrette base and socket clean out. If these options are to be further considered, then a detailed work method statement must be compiled by the piling contractor. The work method statement would need to be reviewed by this office.

4.2.3 Slab-on-Grade

Slab-on-grade construction for the ground floor level of the proposed multi-storey car park is considered feasible provided the subgrade is prepared as discussed in Section 4.2.1.4 above. Slabs-on-grade should be constructed independent of the footings and walls (ie. designed as "floating" slabs) to permit relative movement, as a variable subgrade comprising existing fill, engineered fill and alluvial soils is expected. If there are no perimeter footings beams or if external pavements are not sealed against the building, we recommend that slab edge thickening be provided to limit shrink-swell movements around the perimeter.

Based on the results of our previous laboratory testing at Liverpool Hospital, we recommend that the design of proposed slabs-on-grade be based on a CBR of 2.0% or an estimated equivalent Modulus of Subgrade Reaction (k) of 15kPa/mm (750mm diameter plate) for the compacted clay subgrade. For the compacted clay subgrade, a long-term Young's Modulus of 10MPa and a short-term Young's Modulus of 15MPa may be adopted for the slab-on-grade design.

The slabs-on-grade should be supported on at least a 100mm thick sub-base of good quality fine crushed rock such as RTA Specification 3051 unbound base (eg. DGB20) or similar quality, and compacted to a minimum density of 98% of Modified Maximum Dry Density (MMDD). The sub-base material would provide more uniform slab support and would reduce "pumping" of subgrade "fines" at joints. Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

Based on the shrink-swell potential of the clayey soils, we strongly discourage the planting of trees or garden beds in close proximity of the proposed multi-storey car park.

4.3 Bridges

4.3.1 Footings

Based on the results of Boreholes B1 and B2, EFCP tests B3, B4 and P3 and our previous BH1001, BH1002 and BH1004, we recommend that the two proposed bridges be supported on piles founded in the medium dense to dense alluvial sands. As for the proposed multi-storey car park, we recommend that CFA piles be used. We are not in favour of using steel helix screw piles due to their limited lateral load capacity.

For the proposed road bridge, we recommend that the CFA piles be founded at RL 2.5m AHD. For the north-western side of the proposed pedestrian bridge, we recommend that CFA piles be founded at RL 3.5m AHD. For the south-eastern side of the proposed pedestrian bridge, we recommend that CFA piles be founded at RL 2.5m AHD. At these founding levels, the indicative pile design values tabulated below are applicable.

Diameter	Maximum Allowable End Bearing Pressure	Maximum Allowable Shaft Adhesion Value
400mm	1350kPa	10kPa below 3.5m depth
600mm	2050kPa	15kPa below 5.0m depth

If higher bearing pressures are required, then the piles will most likely need to be founded in the underlying Class III or better quality shale, as per the recommendations provided in Section 4.2.2.2 above. If any piles are founded below the above mentioned levels in order to obtain additional shaft adhesion, then further advice should be sought with respect to settlements due to the presence of "weaker" underlying bands.

The design parameters for different strata tabulated below should be adopted in designing the CFA piles to support lateral loads.

Strata	Undrained Shear Strength, C_u (kPa)	Effective Angle of Friction, ϕ (Degrees)	Elastic Modulus (MPa)	Unit Weight (kN/m^3)
Very Stiff to Hard Alluvial Clays	150	-	20 – 30	19
Medium Dense and Dense Alluvial Sands	-	33	20 – 30	20

For individual piles in sands, the unfactored lateral resistance is 3 times the "passive" lateral earth pressure coefficient (K_p) multiplied by the effective vertical stress. For the silty clays, the unfactored lateral resistance is 9 times the undrained shear strength below a depth of 1.5 pile diameters from ground surface. The effects of the overlying existing fill should be ignored.

4.3.2 Road Bridge Abutments

Site preparation for the fill embankments should be carried out in accordance with the recommendations outlined in Section 4.2.1.4.

In bridge construction, reinforced earth walls are usually used to support the sides of fill embankments. Our preferred engineered fill material behind the reinforced earth walls is either well graded crushed concrete or crushed sandstone to a maximum particle size of 75mm. The advantages of using this material is that they are relatively easy to compact and they are a high friction angle material preferred for geogrid tie-back design. Further advice should be sought from the supplier.

Additional advice on the proposed bridges can be provided once the architectural details have been finalised.

4.4 Soil Aggression

The previous laboratory soil pH test results of 5.1 to 5.6 for the alluvial soil samples from BH102, BH1002 and BH1003A indicate mildly acidic subsoil conditions. Reference should therefore be made to the Cement & Concrete Association of Australia Technical Note TN57 and to Section 6 of AS2159-1995 for appropriate precautionary measures.

4.5 Earthquake Design Parameters

For earthquake design in accordance with AS1170.4-1993 ("Minimum Design Loads on Structures, Part 4: Earthquake Loads"), the following design parameters should be adopted:

- Site Factor (S) = 1.0
- Acceleration Coefficient (α) = 0.08

4.6 Additional Investigations

Once the architectural and structural designs have been finalised for the proposed multi-storey car park, we recommend that at least four additional cored boreholes be drilled to obtain an adequate site coverage, as outlined in Section 4.2.2.2. For a development of this nature, boreholes would usually be spaced at no more than 40m apart. The additional boreholes should be drilled post-demolition and could be drilled at high column load locations, if appropriate. We would be happy to provide a cost proposal to carry out the additional boreholes.

If the locations of the proposed bridges and multi-storey car park are altered, then the recommendations provided in this report must be reviewed by this office.

4.7 Additional Geotechnical Input

We summarise below the previously recommended additional work that needs to be carried out:

1. Review of this report once the development details have been finalised.
2. Additional borehole investigation.
3. Proof rolling inspections.
4. Density testing of all engineered fill and sub-base layers.
5. Review of work method statements if conventional bored piles or barrettes are used.
6. Footing inspections.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and Jeffery and Katauskas Pty Ltd accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

Occasionally, the subsurface conditions between the completed borehole and EFCP test locations may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

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

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

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

Andrew Jackaman
Associate
South-Western Sydney Office Manager

Reviewed By:

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

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

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
B1	16.25-16.28	0.8	16
	16.83-16.87	1.8	36
	17.23-17.27	0.8	16
	17.77-17.81	1.1	22
	18.06-18.10	1.0	20
	18.56-18.61	0.8	16
B2	16.06-16.09	1.0	20
	16.86-16.90	1.0	20
	17.04-17.07	0.8	16
	17.64-17.68	0.6	12
	18.13-18.17	1.0	20

NOTES:

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

   U.C.S. = 20 $I_{S(50)}$

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
B1
1/4

Client: CAPITAL INSIGHT PTY LTD

Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK

Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job No. M21170ZA

Date: 24-5-07

Method: SPIRAL AUGER JK550

R.L. Surface: ≈ 9.4m

Datum: AHD

Logged/Checked by: T.M./AJ

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty clay, medium plasticity, dark brown, with a trace of fine to coarse grained sub angular ironstone gravel, slag fragments and root fibres.	MC>PL			GRASS COVER
	DS	N = 7 1,3,4			CH	SILTY CLAY: high plasticity, light brown and light grey, with a trace of fine grained rounded ironstone gravel and roots.	MC>PL	VSt	250 260 280	ALLUVIAL
			1							
	DS	N = 20 5,8,12				SILTY CLAY: high plasticity, red brown and light grey mottled light brown, with fine to coarse grained sub rounded ironstone gravel bands.	MC≈PL	H	>600 >600 >600	
			2							
	DS	N = 32 10,16,16	3		SC	CLAYEY SAND: fine grained, light grey, with orange brown bands, with silty clay seams.	M	D	>600 >600 >600	HP TESTING CARRIED OUT ON SILTY CLAY SEAMS
					SM	SILTY SAND: fine to medium grained, orange brown and light grey, with a trace of clay fines, with occasional sand and clayey sand bands.				
			4							
	DS	N = 31 13,16,15								
			5							
						as above, but red brown.				
	DS	N > 23 18,23/ 150mm END	6							
			7							

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BOREHOLE LOG

Borehole No. **B1**

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job No. M21170ZA
Date: 24-5-07
Method: SPIRAL AUGER JK550
Logged/Checked by: T.M./AJ
R.L. Surface: ≈ 9.4m
Datum: AHD

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					SC	CLAYEY SAND: fine to medium grained, light grey, orange brown and red brown, with occasional sandy clay bands.	M	L		
ON COMPLETION OF AUGERING ▼	DS	N = 9 8,6,3	8							
					SW	GRAVELLY SAND: fine to coarse grained, red brown, fine to coarse grained sub angular to sub rounded ironstone gravel, with a trace of fines.		D		
►	DS	N > 24 8,24/ 150mm END	9				W			
			10		SM	SILTY SAND: fine to coarse grained, red brown, with a trace of fine to coarse grained sub rounded ironstone gravel, with clayey sand bands.		MD		
	DS	N = 16 8,9,7	11							
	DS	N = 10 7,4,6	12							
			13			as above, but with grey bands.				
	DS	N = 24 4,11,13	14							

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BOREHOLE LOG

Borehole No.
B1

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job No. M21170ZA
Date: 24-5-07
Method: SPIRAL AUGER JK550
Logged/Checked by: T.M./AJ
R.L. Surface: ≈ 9.4m
Datum: AHD

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
								SM	SILTY SAND: fine to coarse grained, red brown and grey, with a trace of fine to coarse grained sub rounded ironstone gravel, with clayey sand bands.	W	MD-D		
				■	N > 15 8,15/ 150mm REFUSAL	15							
						16							
									REFER TO CORED BOREHOLE LOG				
						17							
						18							
						19							
						20							


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JOB NO. M21170ZA B1 START CORING AT 16.16m
16
17
18
END OF BOREHOLE AT 18.83m

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CORED BOREHOLE LOG

Borehole No.
B1

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job No. M21170ZA | **Core Size:** NMLC | **R.L. Surface:** ≈ 9.4m
Date: 24-5-07 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK550 | **Bearing:** - | **Logged/Checked by:** T.M./AG

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
		15								
		16		START CORING AT 16.16m						
FULL RET-URN		16–18		SHALE: dark grey, with light grey laminae, bedded at 0-5°.	Fr	M-H			- Be, 0°, P, S - Be, 0°, P, S	
		19		END OF BOREHOLE AT 18.83m						
		20								
		21								

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BOREHOLE LOG

Borehole No.
B2
1/4

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job No. M21170ZA **Method:** SPIRAL AUGER JK550 **R.L. Surface:** ≈ 9.8m
Date: 26-4-07 **Datum:** AHD
Logged/Checked by: T.M./AG

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty clay, high plasticity, brown mottled red brown and grey, with a trace of root fibres, fine to medium grained sub angular ironstone and quartz gravel, and fine to medium grained sand.	MC>PL			GRASS COVER
	DS	N = 21 2,7,14							600 430 570	APPEARS WELL COMPACTED
			1		CH	SILTY CLAY: high plasticity, red brown mottled light grey, with fine to coarse grained sub angular to sub rounded ironstone gravel.	MC>PL	H	-	ALLUVIAL
	DS	N = 20 6,10,10							>600 >600 >600	
			2							
	DS	N = 32 10,17,15	3						>600 >600 >600	
			4		SC	CLAYEY SAND: fine to medium grained, light grey, with a trace of orange brown mottling and clay bands.	M	MD		
	DS	N = 26 8,13,13								
			5							
			6					D		
	DS	N = 31 12,14,17								
						as above, but fine to coarse grained.				
			7							

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BOREHOLE LOG

Borehole No. **B2**

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job No. M21170ZA
Date: 26-4-07
Method: SPIRAL AUGER JK550
Logged/Checked by: T.M./
R.L. Surface: ≈ 9.8m
Datum: AHD

Groundwater Record	Samples (ES/U50/DB/DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					SC	CLAYEY SAND: fine to coarse grained, light grey, with a trace of orange brown mottling.	M	D		
	DS	N = 32 10,16,16								
			8			as above, but with fine to coarse grained sub angular to sub rounded ironstone gravel, with occasional sandy clay bands.		MD		
			9							
ON COMPLETION OF AUGERING	DS	N = 23 3,6,17					W			
			10							
			11							
	DS	N = 23 4,9,14	12							
			13							
			14							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **B2**

BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job No. M21170ZA
Date: 26-4-07
Method: SPIRAL AUGER JK550
Logged/Checked by: T.M./A9
R.L. Surface: ≈ 9.8m
Datum: AHD

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
								SC	CLAYEY SAND: fine to coarse grained, light grey, with a trace of orange brown mottling and fine to coarse grained sub angular to sub rounded ironstone gravel, with occasional sandy clay bands.	W	MD		
						15		-	SHALE: dark grey.	XW	EL	-	
						16			REFER TO CORED BOREHOLE LOG				
						17							
						18							
						19							
						20							


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JOB NO. M21170ZA
B2
START CORING AT 15.97m
16
17
18
END OF BOREHOLE AT 15.58m

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **B2**

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job No. M21170ZA **Core Size:** NMLC **R.L. Surface:** ≈ 9.8m
Date: 26-4-07 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK550 **Bearing:** - **Logged/Checked by:** T.M./AJ

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
		15		START CORING AT 15.97m						
90% RETURN		16		SHALE: dark grey, with light grey laminae, bedded at 0-5°.	Fr	M	×			
		17					× ×		- J, 45°, P, S - J, 45°, P, S - J, 50°, P, S	
		18				VH	× ×		2xJ, 85-90°, P, S - J, 45°, P, S	
				END OF BOREHOLE AT 18.58m		M				
		19								
		20								
		21								

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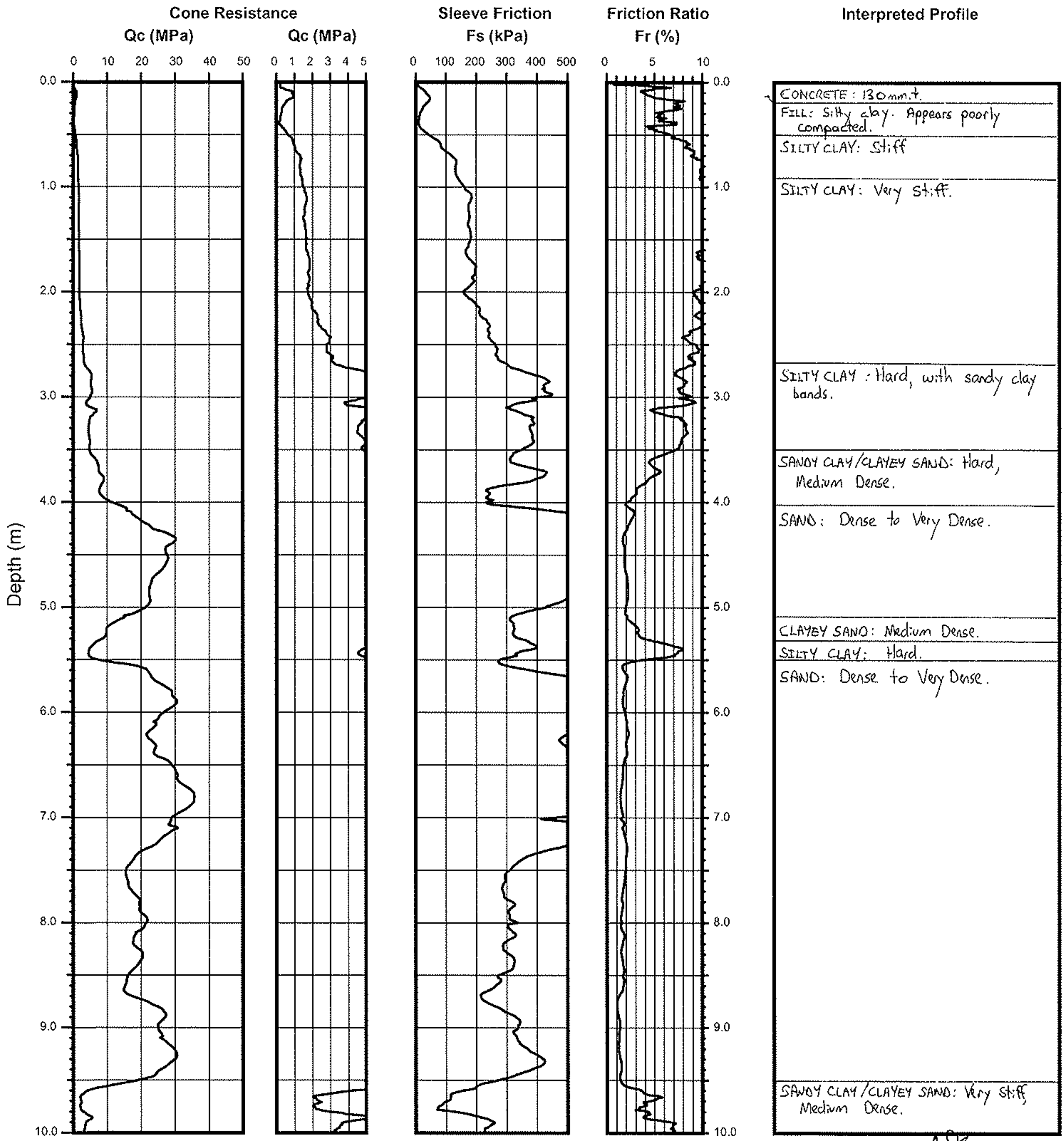
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
B3
1/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.: M21170ZA	**RL Surface:** ~9.6m	**Data File:** M21170ZAcptB3.cpt
Test Date: 23/05/2007	**Datum:** AHD	**Operator:** NES



Interpreted by: A.S.
Checked by: [signature]

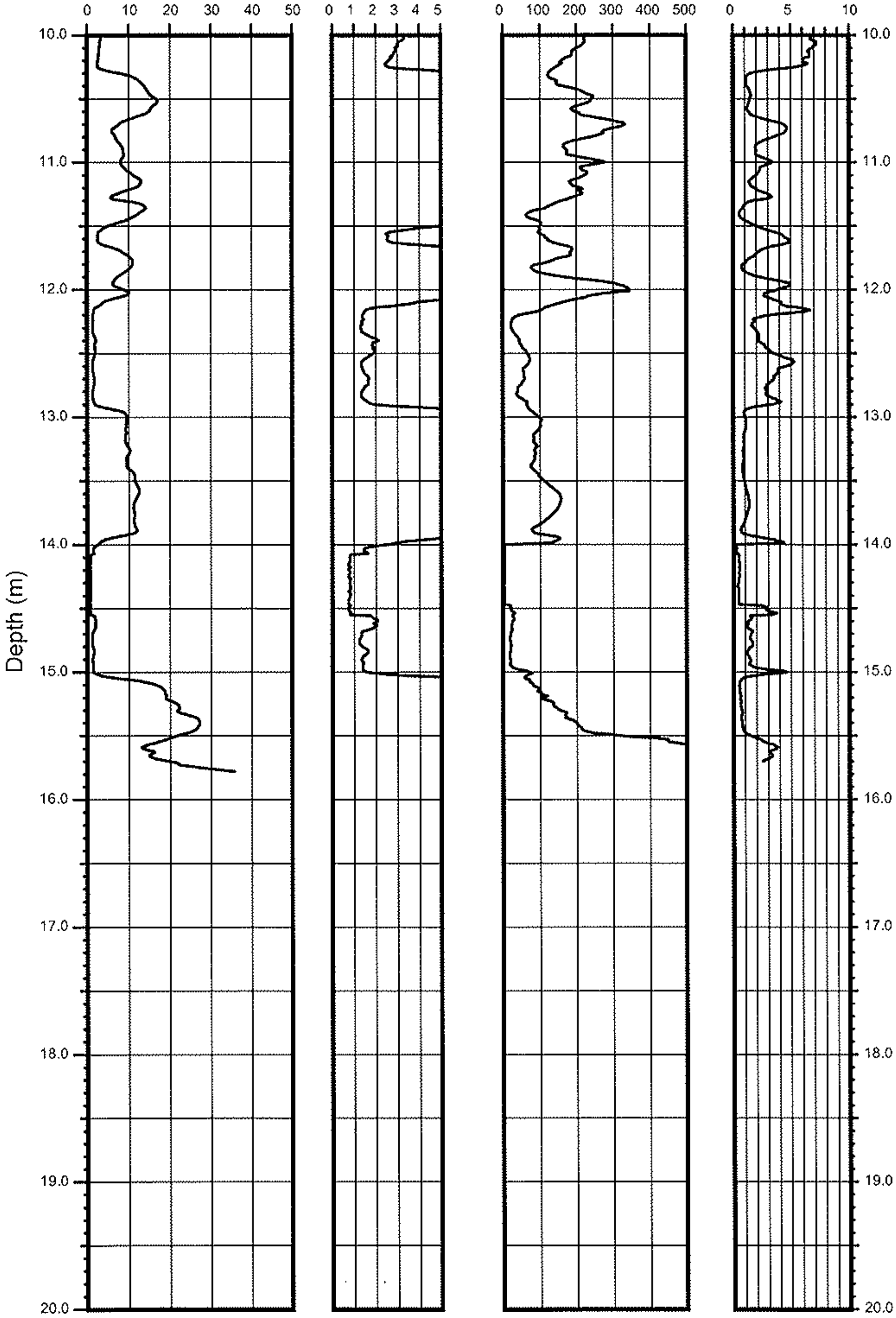
Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
B3
2/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.:	M21170ZA	**RL Surface:**	~9.6m	**Data File:**	M21170ZAcptB3.cpt
Test Date:	23/05/2007	**Datum:**	AHD	**Operator:**	NES



Interpreted Profile

Interpreted Profile
SILTY CLAY: Very Stiff.
SAND/CLAYEY SAND: Medium Dense, with very stiff clay bands.
SAND: Medium Dense.
SAND: Very Loose, with clayey sand seams.
SAND: Dense
SAND: Very Dense
END OF EFCP TESTING AT 15.8m

Interpreted by: A.Y.
Checked by: [signature]

Jeffery and Katauskas Pty Ltd

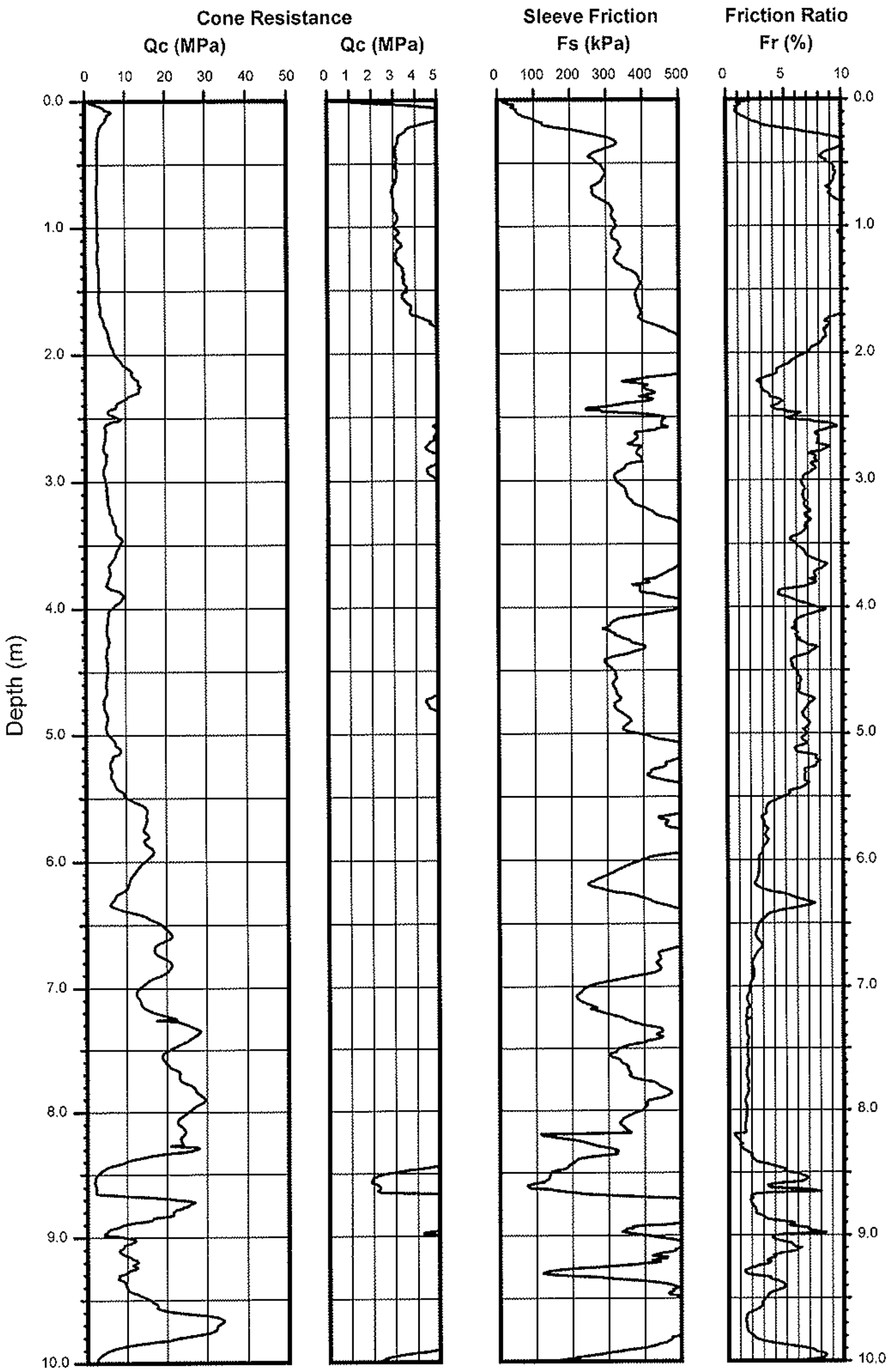
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
B4
1/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.:	M21170ZA	**RL Surface:**	~9.5m	**Data File:**	M21170ZAcptB4.cpt
Test Date:	23/05/2007	**Datum:**	AHD	**Operator:**	NES



Interpreted Profile

GRASS COVER

FILL: Silty sand.
SILTY CLAY: Hard.
CLAYEY SAND/SANDY CLAY: Medium Dense, Hard.
SILTY CLAY: Hard, with sandy clay bands.
CLAYEY SAND: Medium Dense.
SILTY CLAY: Hard.
SAND: Dense.
SANDY CLAY/CLAYEY SAND: Very Stiff to Hard, Medium Dense with dense sand bands.

Interpreted by: AS
Checked by: B

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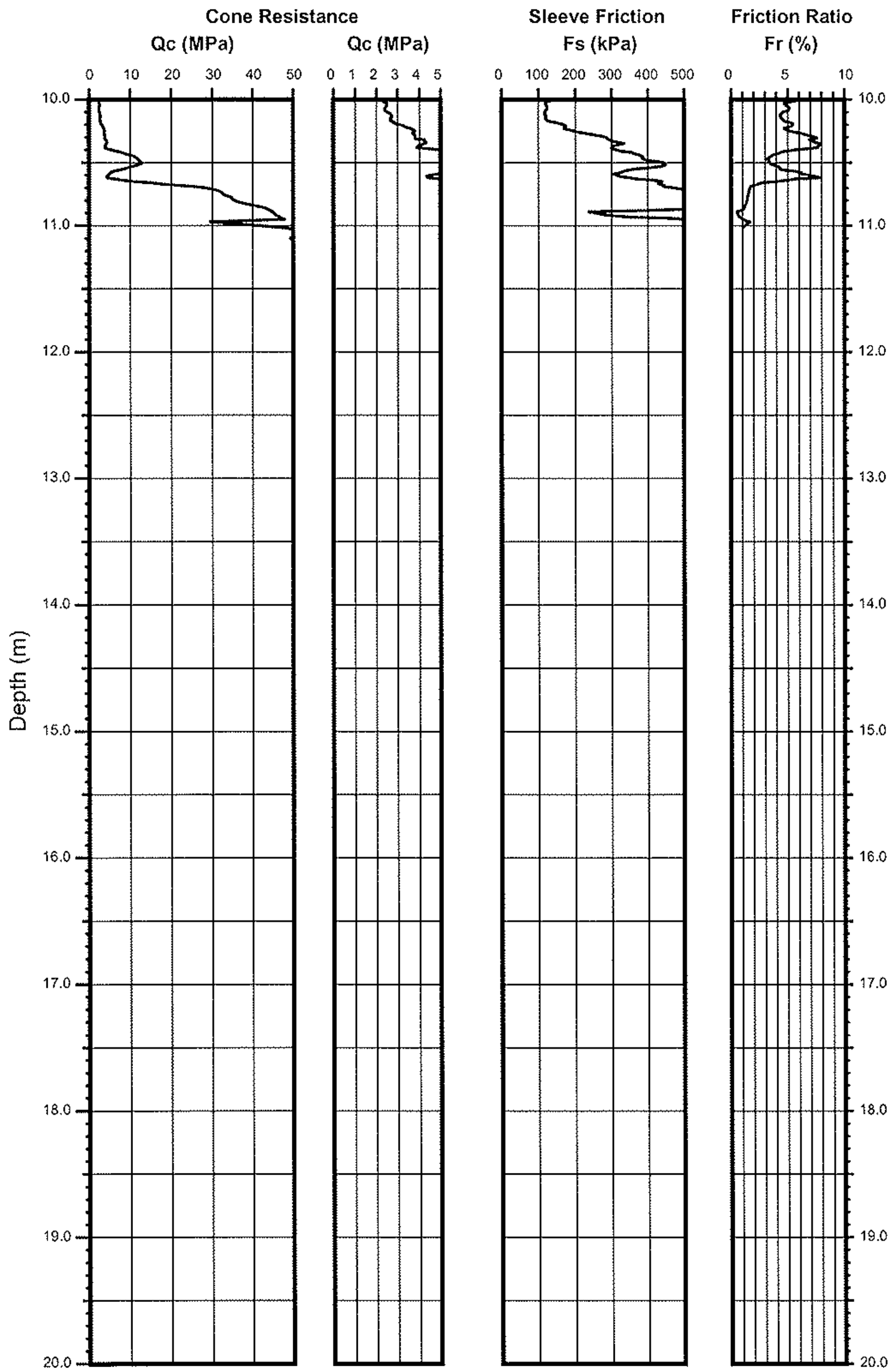
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
B4
2/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.:	M21170ZA	**RL Surface:**	~9.5m	**Data File:**	M21170ZAcptB4.cpt
Test Date:	23/05/2007	**Datum:**	AHD	**Operator:**	NES



Interpreted Profile

SANDY CLAY/SILTY CLAY: Very Stiff to Hard.

SAND: Very Dense.

END OF EFCP TESTING AT 11.1m

Interpreted by: AJ
Checked by: QS

Jeffery and Katauskas Pty Ltd

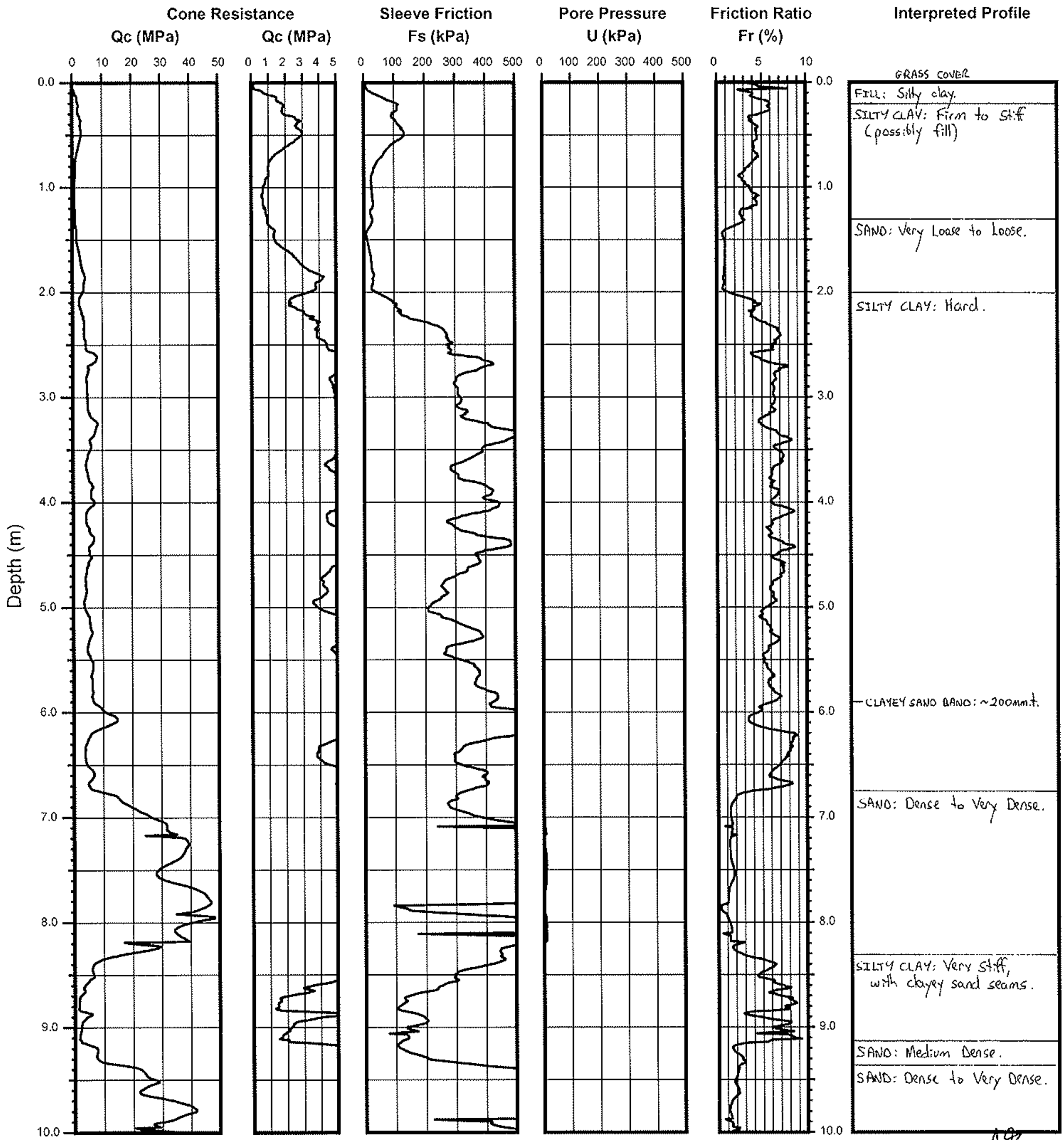
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
P1
1/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.:	M21170ZA	**RL Surface:**	~9.9m	**Data File:**	CPTP1.cpt
Test Date:	26/04/2007	**Datum:**	AHD	**Operator:**	PL



Interpreted by: AG
Checked by: G

Jeffery and Katauskas Pty Ltd

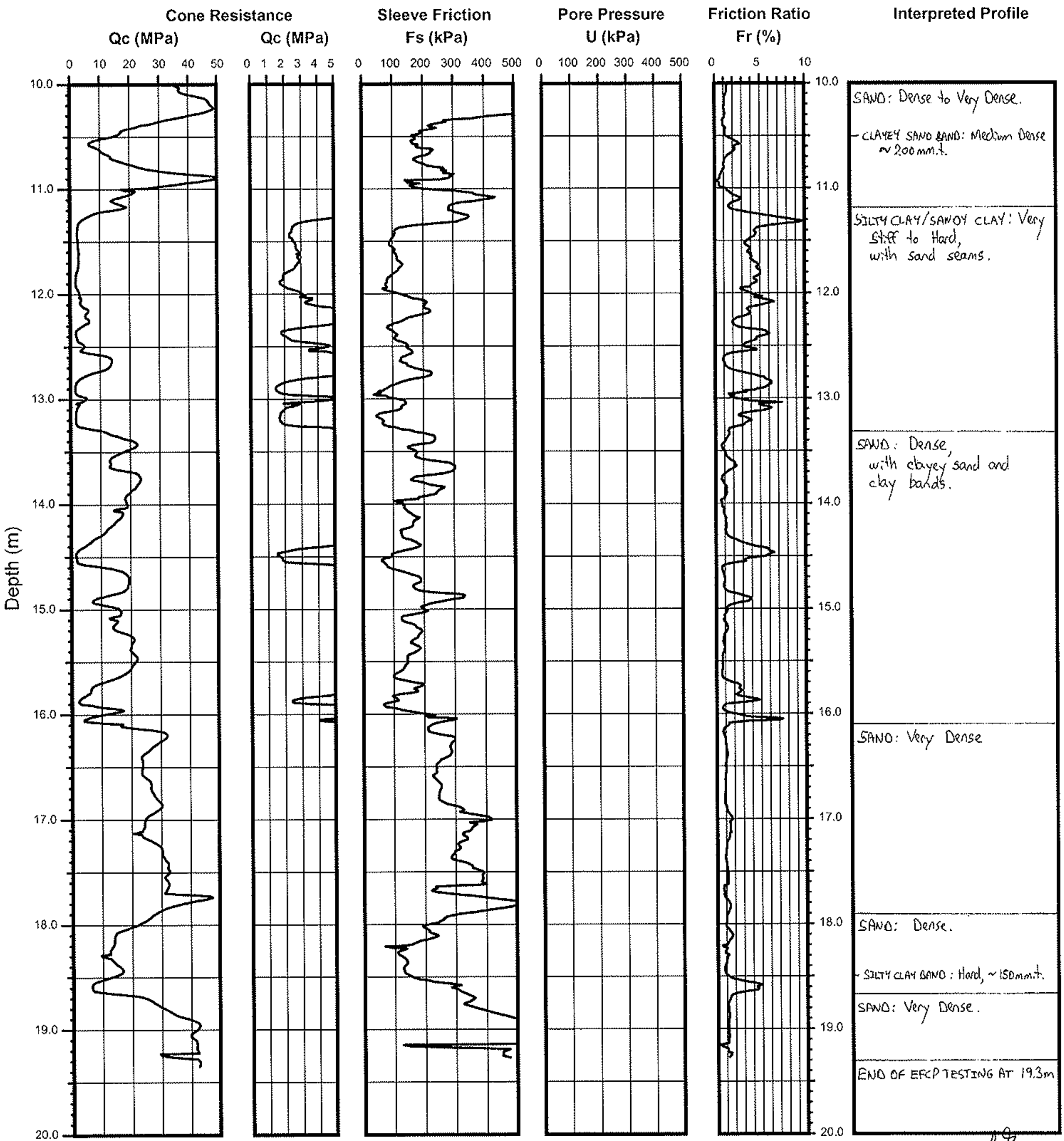
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
P1
2/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.:	M21170ZA	**RL Surface:**	~9.9m	**Data File:**	CPTP1.cpt
Test Date:	26/04/2007	**Datum:**	AHD	**Operator:**	PL



Interpreted by:
Checked by:

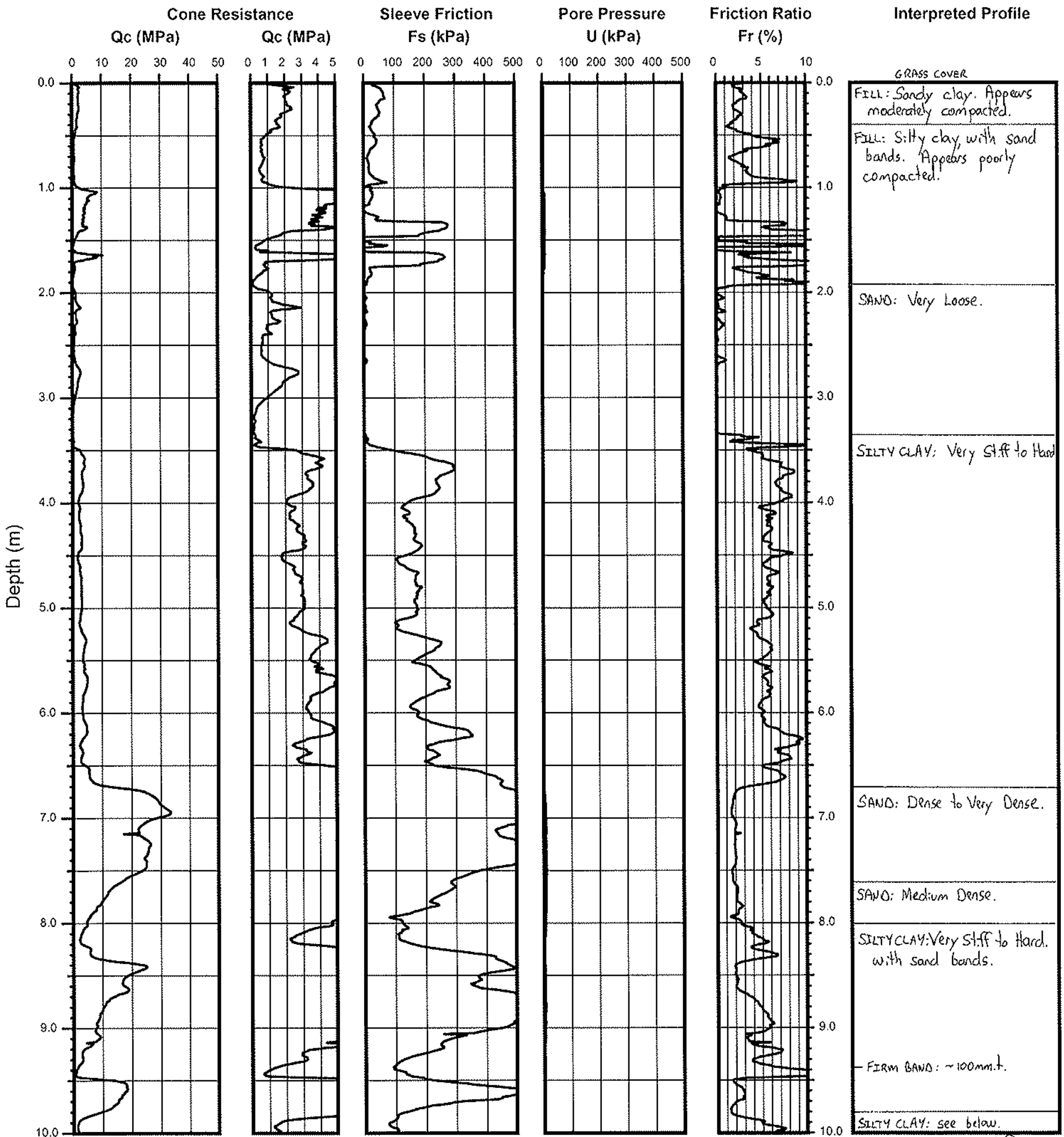
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No. **P2** 1/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD				
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK				
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW				
Job Ref.:	M21170ZA	**RL Surface:**	~9.8m	**Data File:**	CPTP2.cpt
Test Date:	26/04/2007	**Datum:**	ahd	**Operator:**	PL



Interpreted by:
Checked by:

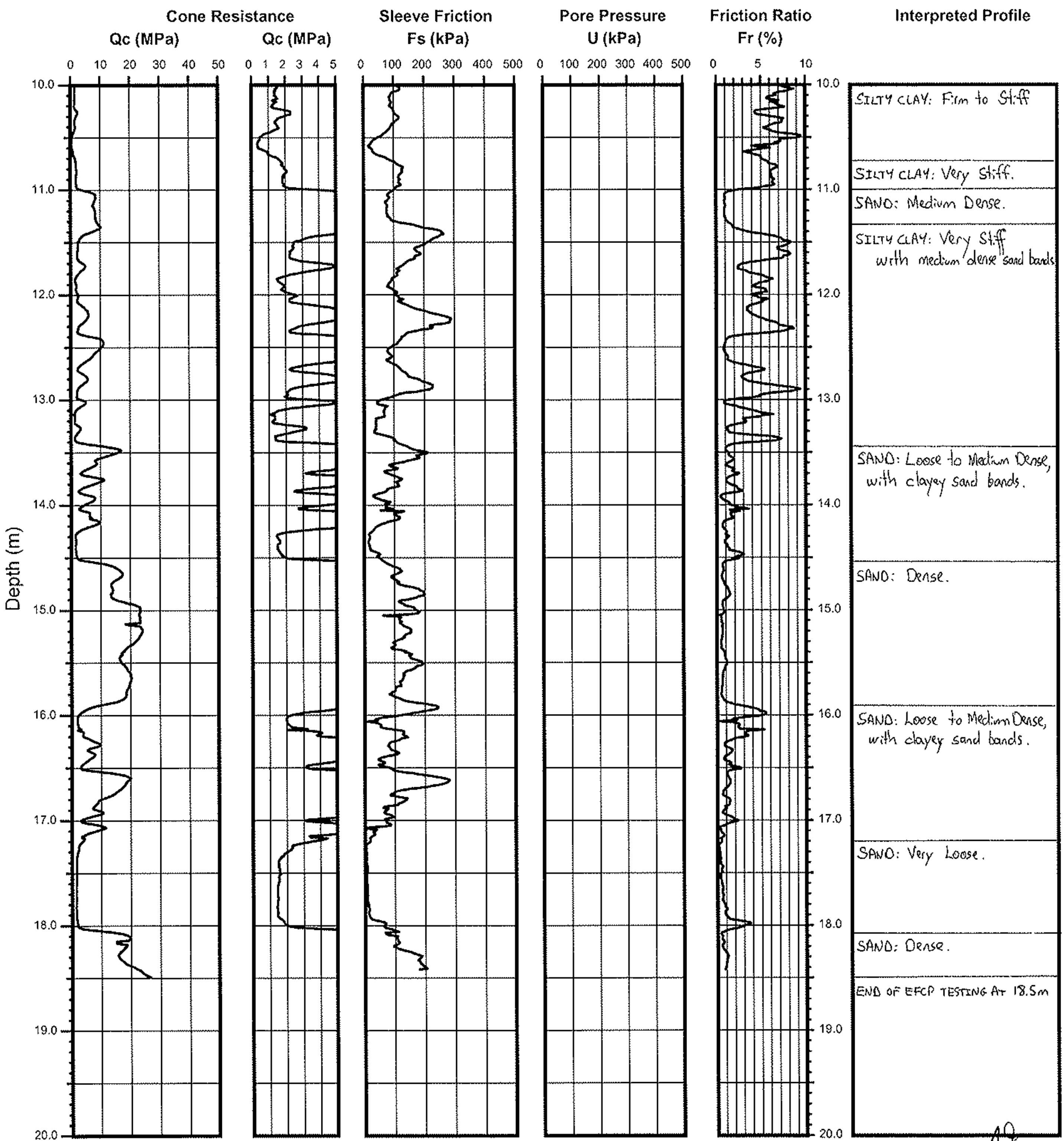
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
P2
2/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD				
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK				
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW				
Job Ref.:	M21170ZA	**RL Surface:**	~9.8m	**Data File:**	CPTP2.cpt
Test Date:	26/04/2007	**Datum:**	ahd	**Operator:**	PL



Interpreted by: AB
Checked by: B

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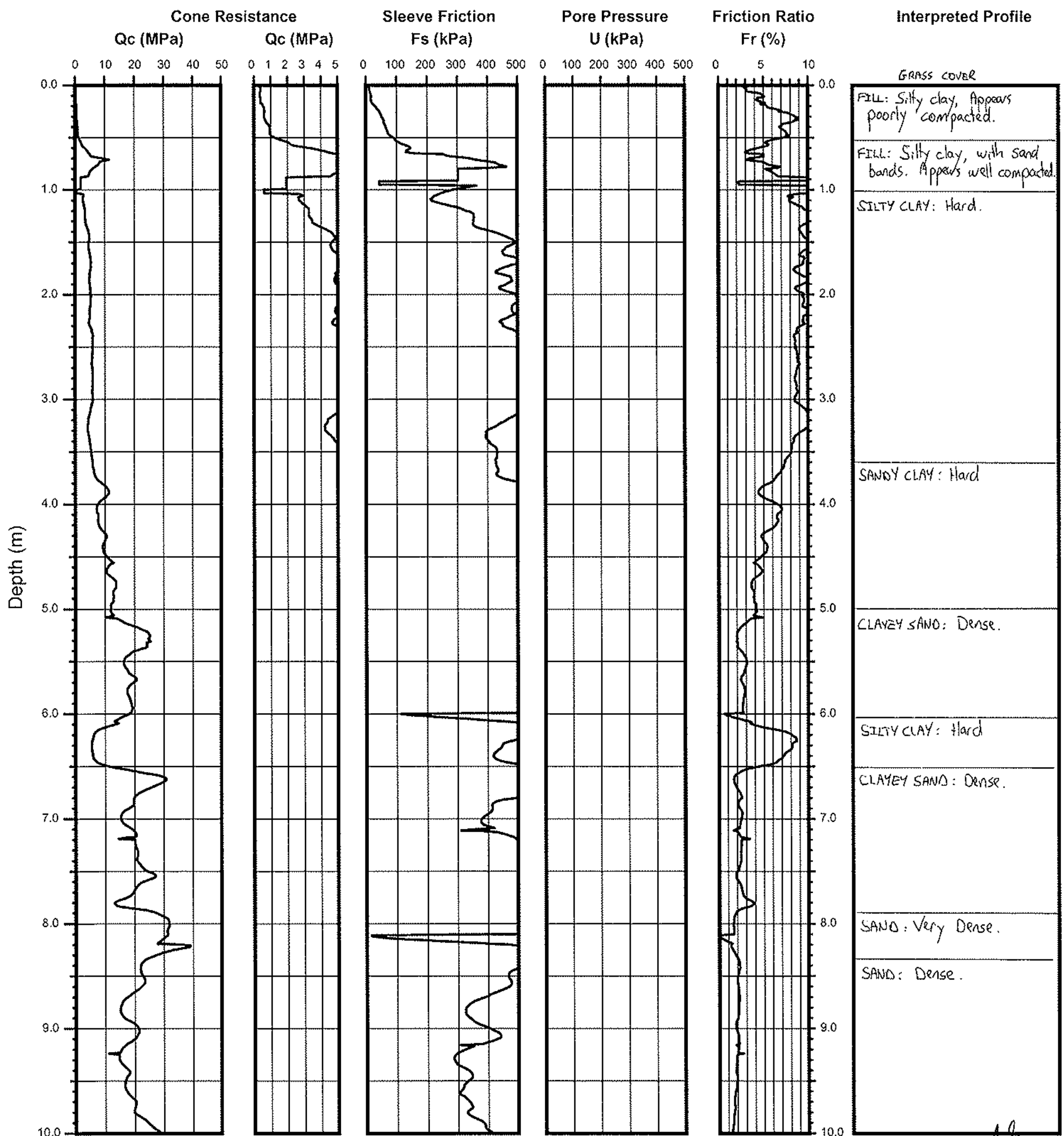
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
P3
1/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.:	M21170ZA	**RL Surface:**	~9.6m	**Data File:**	CPTP3.cpt
Test Date:	27/04/2007	**Datum:**	AHD	**Operator:**	PL



Interpreted by:
Checked by:

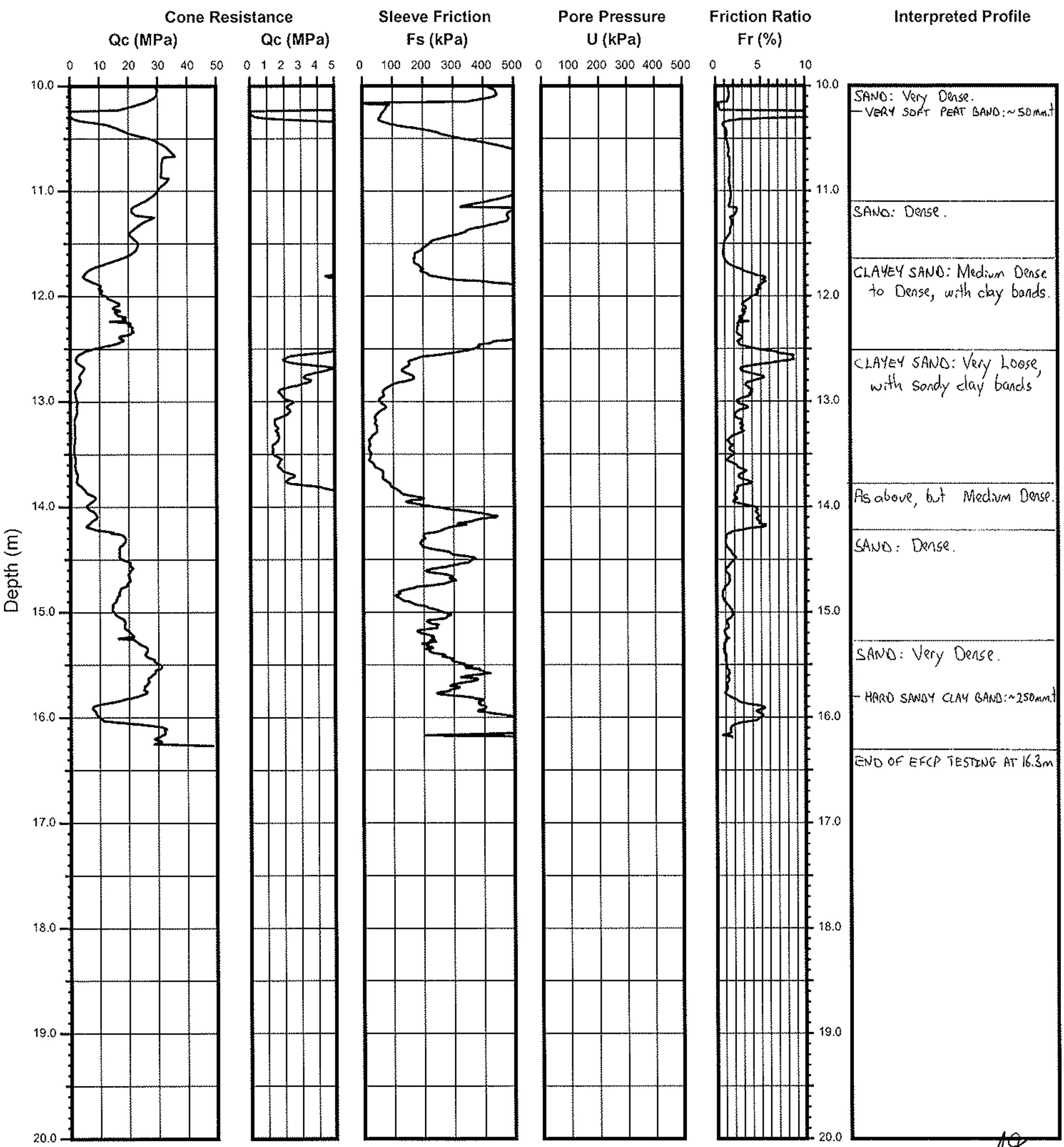
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
P3
2/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD				
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK				
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW				
Job Ref.:	M21170ZA	**RL Surface:**	~9.6m	**Data File:**	CPTP3.cpt
Test Date:	27/04/2007	**Datum:**	AHD	**Operator:**	PL



Interpreted by:
Checked by:

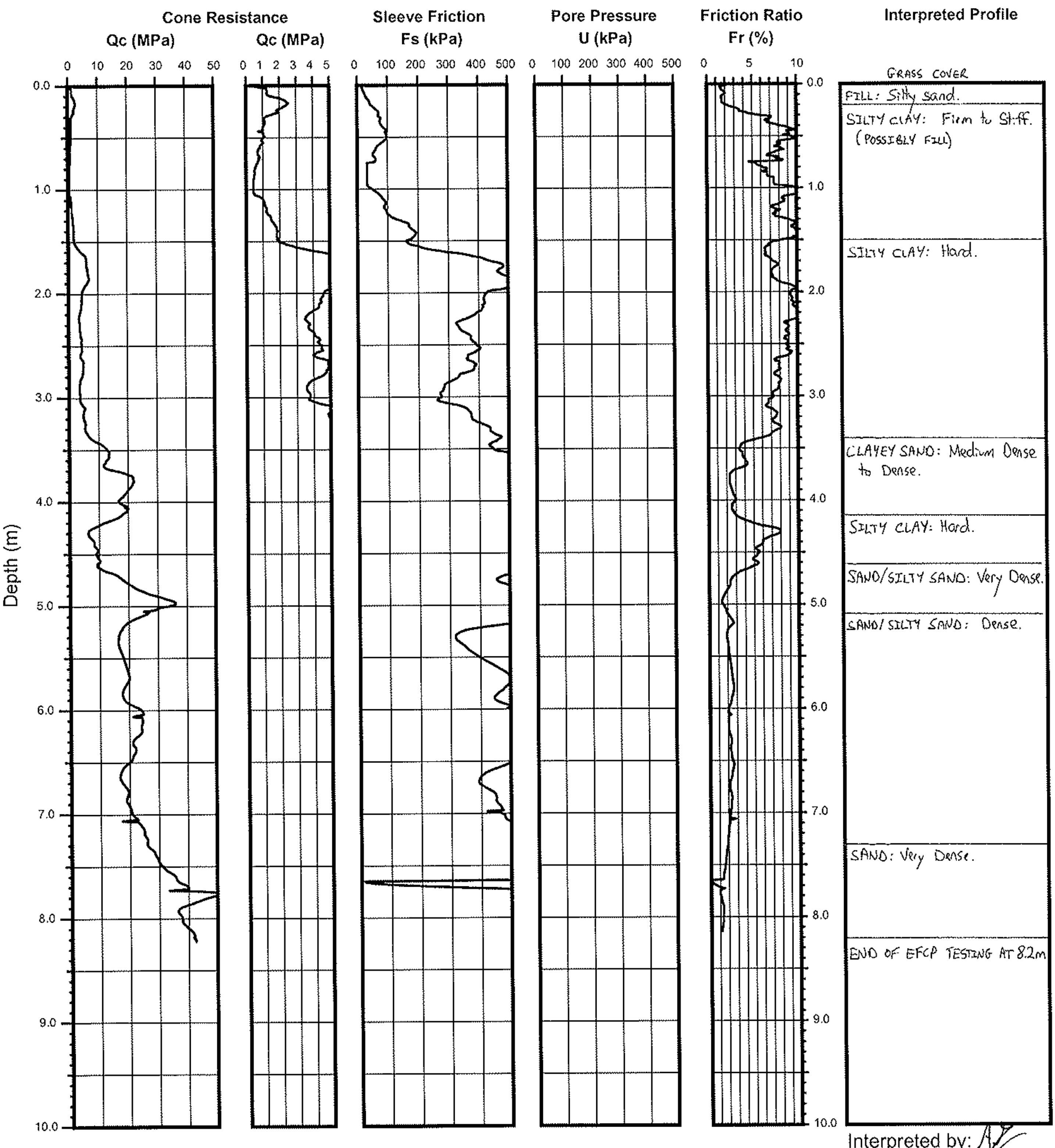
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No. **P4** 1/1

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD				
Project:	PROPOSED BRIDGES AND MULTI STOREY CARPARK				
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW				
Job Ref.:	M21170ZA	**RL Surface:**	~9.5m	**Data File:**	CPTP4.cpt
Test Date:	27/04/2007	**Datum:**	AHD	**Operator:**	PL



Interpreted by:
Checked by:

Jeffery and Katauskas Pty Ltd

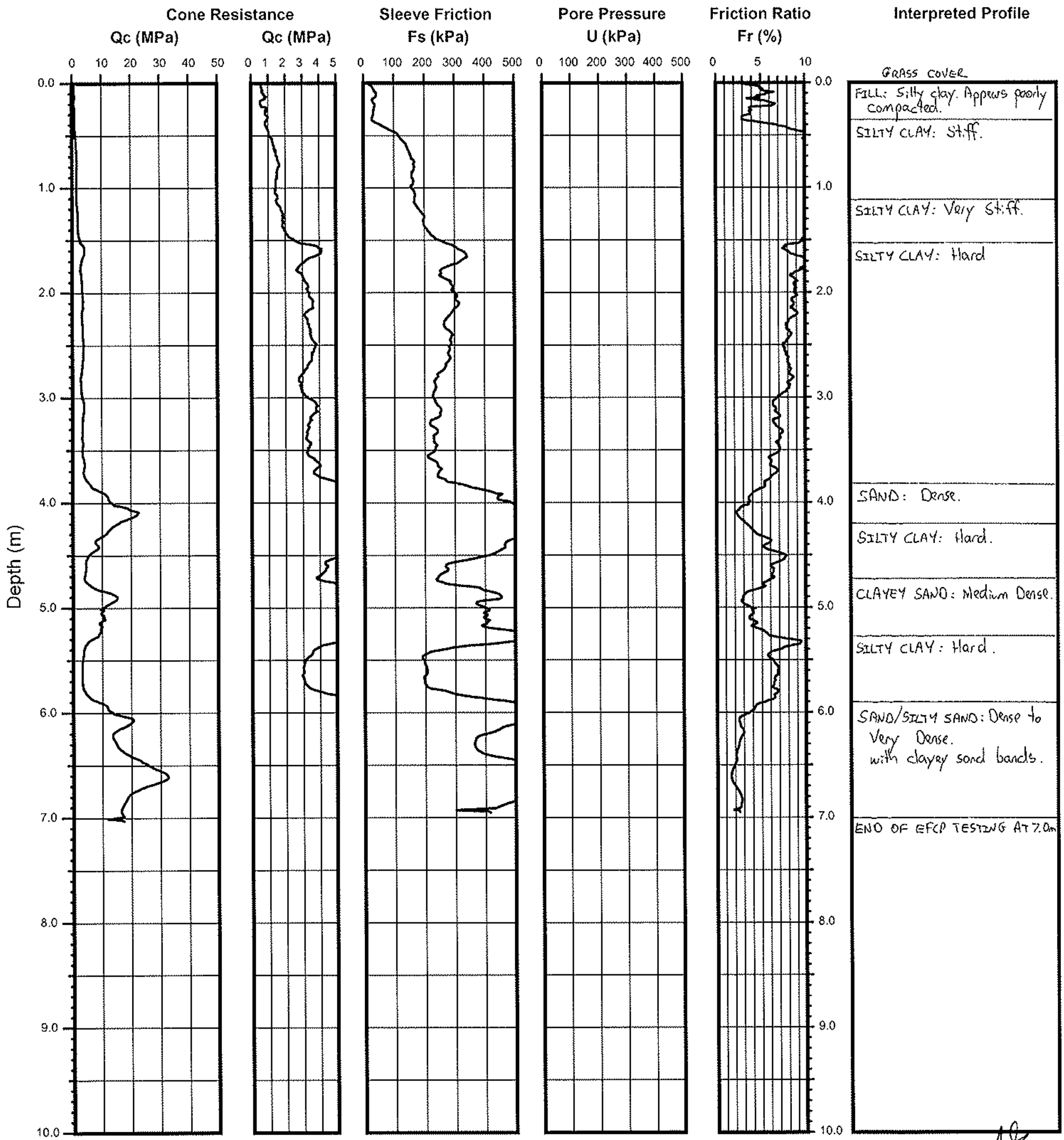
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
P5
1/1

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location: LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.:	M21170ZA	**RL Surface:**	~9.6m	**Data File:**	CPTP5.cpt
Test Date:	27/04/2007	**Datum:**	AHD	**Operator:**	PL



Interpreted by:
Checked by:

Jeffery and Katauskas Pty Ltd

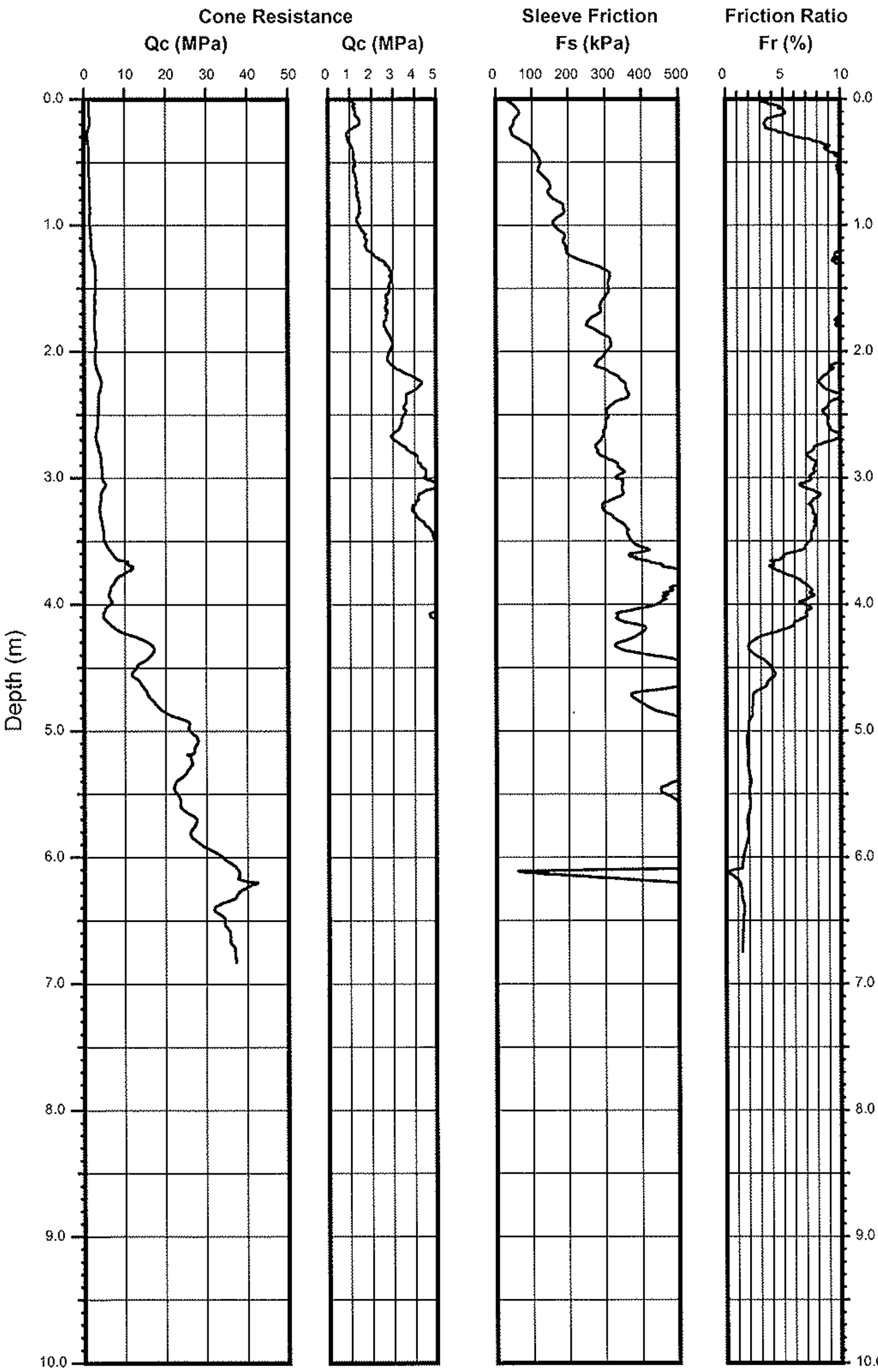
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No. **P6** 1/1

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW

Job Ref.:	M21170ZA	**RL Surface:**	~9.3m	**Data File:**	M21170ZAcptP6.cpt
Test Date:	23/05/2007	**Datum:**	AHD	**Operator:**	NES



Interpreted Profile

GRASS COVER

FILL: Silty Clay. Appears moderately compacted.
SILTY CLAY: Stiff
SILTY CLAY: Very Stiff.
SILTY CLAY: Hard.
– MEDIUM DENSE CLAYEY SAND BAND: ~150mm t.
CLAYEY SAND: Medium Dense.
SAND/SILTY SAND: Dense.
SAND: Very Dense
END OF EFCP TESTING AT 6.8m

Interpreted by:

Checked by:

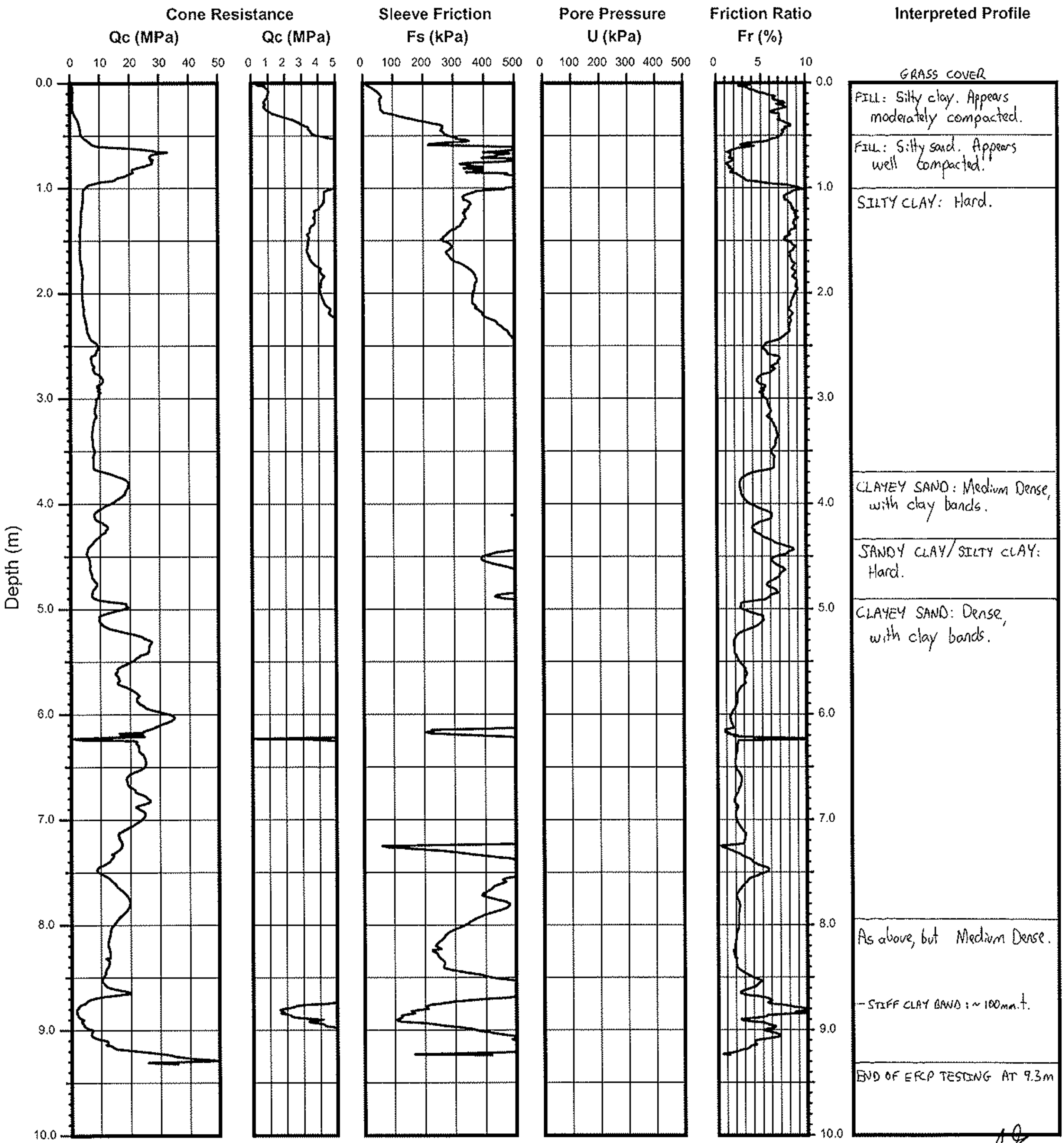
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

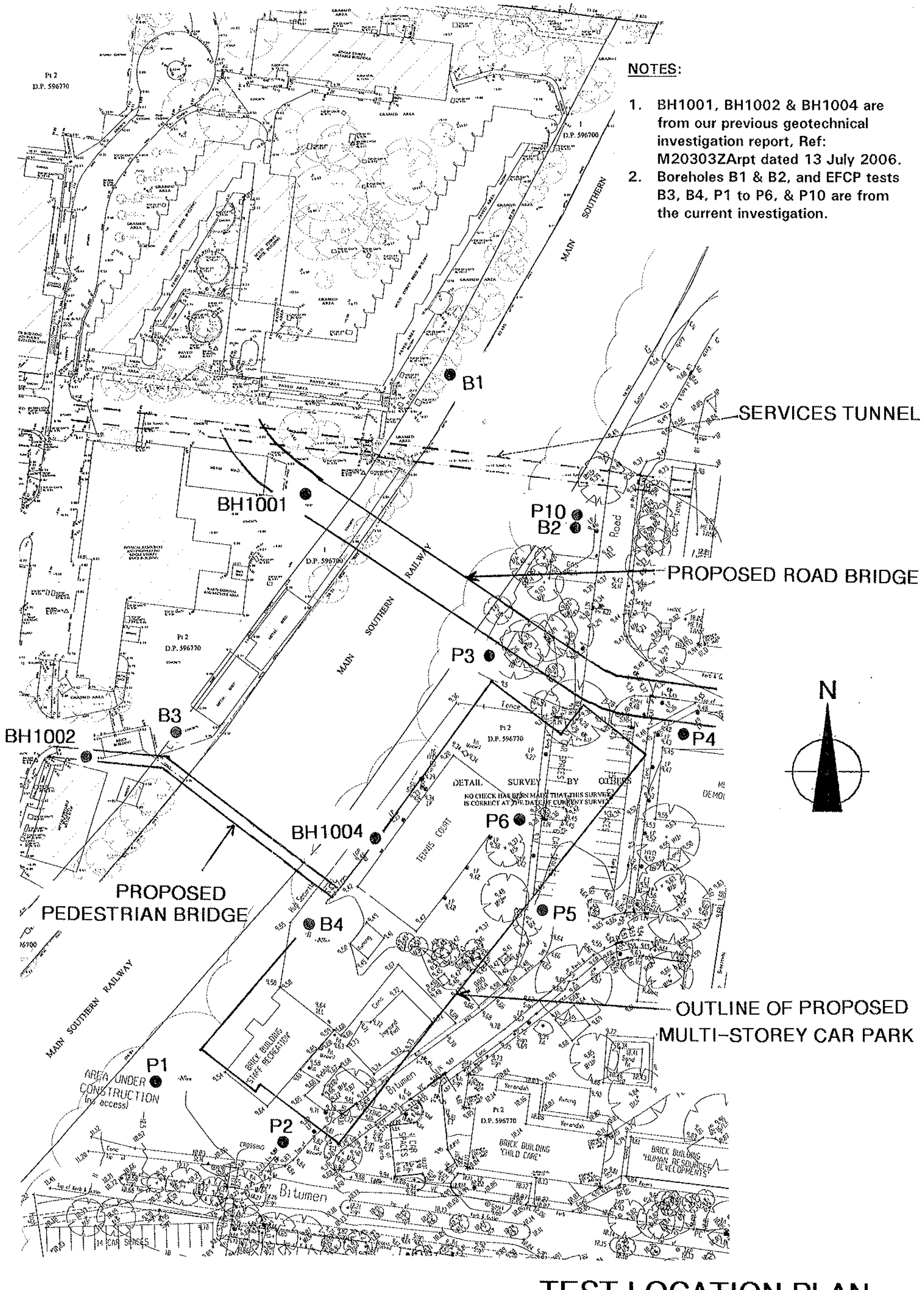
EFCP No.
P10
1/1

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	CAPITAL INSIGHT PTY LTD				
Project:	PROPOSED BRIDGES AND MULTI-STOREY CARPARK				
Location:	LIVERPOOL HOSPITAL, LIVERPOOL, NSW				
Job Ref.:	M21170ZA	**RL Surface:**	~9.5m	**Data File:**	CPTP10.cpt
Test Date:	27/04/2007	**Datum:**	AHD	**Operator:**	PL



Interpreted by:
Checked by:

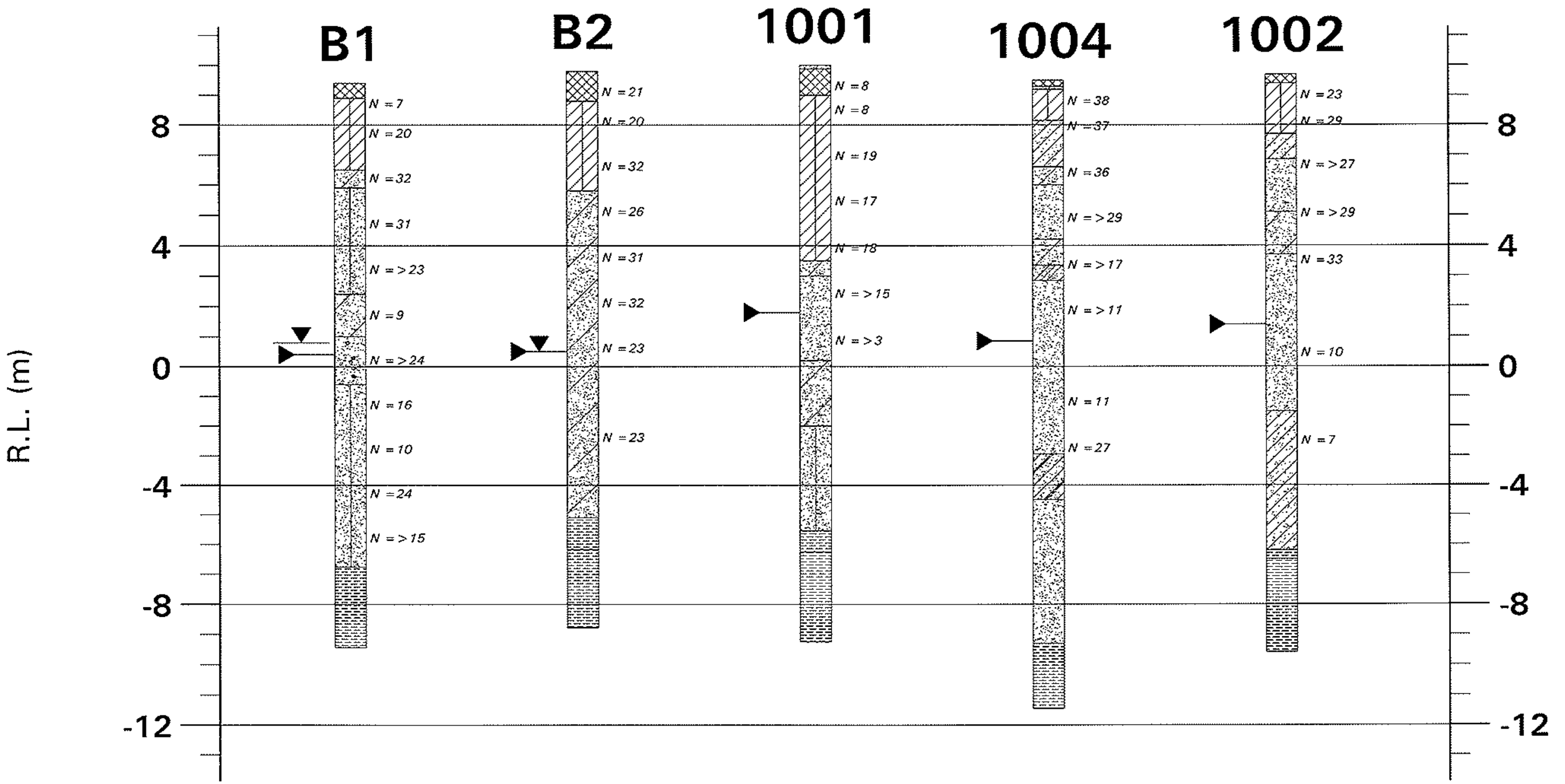


TEST LOCATION PLAN

Jeffery & Katauskas Pty Ltd

Report No. M21170ZA Figure No. 1

GRAPHICAL BOREHOLE SUMMARY



Fill
Silty Clay
Clayey Sand
Silty Sand
Gravelly Sand
Shale
Concrete
Sand
Sandy Clay
Sandy Clay/ Clayey Sand
Observed water level
Groundwater seepage level

N — SPT "N" VALUE
Nc — SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

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

Jeffery and Katauskas Pty Ltd

Job No.: M21170ZA

Figure No.: 2

APPENDIX A

Borehole Logs BH1001, BH1002 & BH1004
From our Previous Report,
Ref: M20303ZArpt dated 13 July 2006

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **1001**
1/4

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA
Date: 30-5-06
Method: SPIRAL AUGER & WASHBORING EDSON 3000
R.L. Surface:
Datum:
Logged/Checked by: M.T./ASH

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		CONCRETE: 120mm.t				7mm DIAMETER REINFORCEMENT, 45mm AND 55mm TOP COVER
		N = 8 7,3,5		-	FILL: Silty sand, fine to medium grained, dark grey, with concrete and brick fragments and igneous gravel.	M	-	-	APPEARS POORLY COMPACTED
		N = 8 3,4,4	1	CH	SILTY CLAY: high plasticity, brown, red and light grey, with root fibres and a trace of ironstone gravel.	MC>PL	VSt	- 310 320 310	ALLUVIAL
		N = 19 5,8,11	3		as above, but with no root fibres.	MC<PL	H	410 410 500	
		N = 17 5,8,9						410 450 460	
		N = 18 6,7,11	6			MC>PL	VSt	280 260 320	
				SC	CLAYEY SAND: fine to medium grained, light grey mottled orange brown.	M	(D)		
			7						

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BOREHOLE LOG

Borehole No. **1001**

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA
Date: 30-5-06
Method: SPIRAL AUGER & WASHBORING EDSON 3000
R.L. Surface:
Datum:
Logged/Checked by: M.T./ASH

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
	DS				SP	SAND: fine to medium grained, orange brown and light grey.	M	(D)		
	DS	N > 15 8,15/ 130mm REFUSAL				as above, but light grey.				
			8			SAND: fine to coarse grained, brown.				
▶	DS						W			
	DS	N > 3 15,3/ 10mm REFUSAL	9					-		CONTINUOUS SPIRAL AUGER AND WASHBORE DRILLING (ie NO INSITU TESTING) FROM 9.16m DOWN TO 15.55m IN ORDER TO PROVE BEDROCK
			10		SC	CLAYEY SAND: fine to medium grained, brown.				
			11							
			12		SM	SILTY SAND: fine to medium grained, light grey, with occasional clay bands.				COMMENCE ROTARY WASHBORE DRILLING
			13			as above, but brown and dark grey.				
			14							

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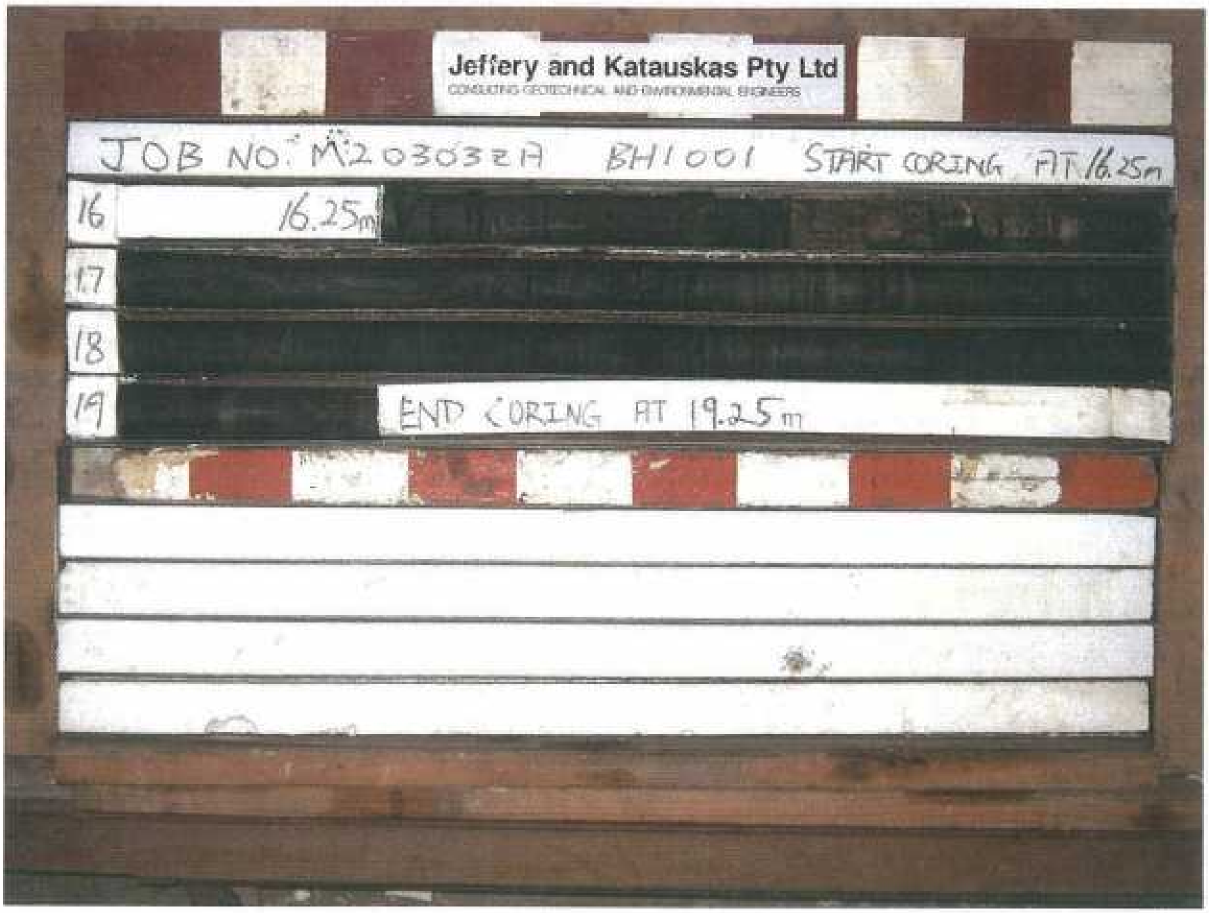
BOREHOLE LOG

Borehole No. **1001**
3/4

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA
Date: 30-5-06
Method: SPIRAL AUGER & WASHBORING EDSON 3000
R.L. Surface:
Datum:
Logged/Checked by: M.T./ASH

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
								SM	SILTY SAND: fine to medium grained, brown and dark grey, with occasional clay bands.	W	-		
						15							
								-	SHALE: dark grey, with clay seams.	XW-DW	EL-VL	-	
						16							
									REFER TO CORED BOREHOLE LOG				
						17							
						18							
						19							
						20							


Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
JOB NO: M20303ZA BH1001 START CORING AT 16.25m
16
16.25m
17
18
19
END CORING AT 19.25m

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Borehole No. 1001
4/4

CORED BOREHOLE LOG

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA | **Core Size:** NMLC | **R.L. Surface:**
Date: 31-5-06 | **Inclination:** VERTICAL | **Datum:**
Drill Type: EDSON 3000 | **Bearing:** - | **Logged/Checked by:** M.T./AJH

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
		15								
		16		START CORING AT 16.25m						
FULL RETURN				SHALE: dark grey, with fine grained, light grey sandstone laminae. bedded at 0-5°, spacing up to 5mm.	Fr	M-H			CS, 20mm.t	
						L-M			CS, 10mm.t	
						M	×			
		17				M-H	×			
									Cr, 5mm.t	
		18					×			
							×			
		19					×			
							×			
				END OF BOREHOLE AT 19.25m						
		20								
		21								

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BOREHOLE LOG

Borehole No. **1002**
1/4

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA
Date: 9-6-06
Method: SPIRAL AUGER & WASHBORING EDSON 3000
R.L. Surface:
Datum:
Logged/Checked by: M.T./ASH

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		FILL: Silty clay, medium plasticity, dark grey and dark brown, with concrete fragments and igneous gravel and a trace of fine to medium grained sand and root fibres.	MC<PL			GRASS COVER
		N = 23 9,11,12		CL	SILTY CLAY: low to medium plasticity, light brown, with a trace of root fibres.	MC<PL	H	- >600 >600 >600	ALLUVIAL
		N = 29 9,13,16	1	CH	as above, but high plasticity, brown, red brown and light grey.			>600 >600 >600	
			2	CH	SANDY CLAY: high plasticity, light grey, light brown and red brown.				
		N > 27 16,22, 5/20mm END	3	SP	SAND: fine to medium grained, light grey, yellow brown and red brown, with a trace of clay fines.	M	D		
					as above, but light brown and light grey.				
		N > 29 14,23, 6/15mm END	4	SC	CLAYEY SAND: fine to coarse grained, light grey, red and yellow brown, with occasional clay seams.				
			5		as above, but fine to medium grained, light grey.				
		N = 33 14,15,18	6	SP	SAND: fine to medium grained, light grey and light brown.				
			7						

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Borehole No. **1002**
2/4

BOREHOLE LOG

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA
Method: SPIRAL AUGER & WASHBORING EDSON 3000
R.L. Surface:
Date: 9-6-06
Datum:
Logged/Checked by: M.T./ASH

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
					SP	SAND: fine to coarse grained, grey and brown, with a trace of clay fines.	M	D		
			8							
▶							W	MD		
			9							
	DS	N = 10 4,6,4				as above, but grey.				
			10							
	DS									
			11							
					CL-CH	SANDY CLAY: medium to high plasticity, light grey and red brown.	MC>PL	(St-VSt)		
	DS									
			12							
	DS	N = 7 4,3,4								
										COMMENCE ROTARY WASHBORE DRILLING
			13							
			14							

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BOREHOLE LOG

Borehole No. 1002
3/4

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA
Date: 9-6-06
Method: SPIRAL AUGER & WASHBORING EDSON 3000
R.L. Surface:
Datum:
Logged/Checked by: M.T./ASH

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
						15		CL-CH	SANDY CLAY: medium to high plasticity, light grey and red brown.	MC>PL	-		CONTINUOUS WASHBORE DRILLING (ie NO INSITU TESTING) FROM 12.45m DOWN TO 15.9m IN ORDER TO PROVE BEDROCK
						16		-	SHALE: dark grey, with clay seams.	XW	EL	-	
						17 18 19 20			REFER TO CORED BOREHOLE LOG				

Jeffery and Katauskas Pty Ltd
JOB NO. M20303ZA BH1002 START CORING AT 16.20m
16.2
17
18
19
END CORING 19.23m

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CORED BOREHOLE LOG

Borehole No. 1002

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA **Core Size:** NMLC **R.L. Surface:**
Date: 9-6-06 **Inclination:** VERTICAL **Datum:**
Drill Type: EDSON 3000 **Bearing:** - **Logged/Checked by:** M.T./KJP

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
		15								
		16		START CORING AT 16.20m						
FULL RET-URN		17 18 19		SHALE: dark grey, with fine grained, light grey sandstone laminae, bedded at 0°-5°, spacing up to 3mm.	Fr	M-H H			- Be, 20°, P, S - Be, 20°, P, S - Be, 20°, P, S - J, 75°, Un, R	
		20 21		END OF BOREHOLE AT 19.28m						

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BOREHOLE LOG

Borehole No.
1004
1/4

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA **Method:** SPIRAL AUGER & WASHBORING EDSON 3000 **R.L. Surface:**
Date: 1-6-06 **Datum:**
Logged/Checked by: M.T./AJ

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty sand, fine to medium grained, dark brown, with igneous gravel and root fibres.	M			GRASS COVER
			SP	SAND: fine to coarse grained, light brown, with a trace of ironstone gravel.	D	(L)	-	ALLUVIAL
	N = 38 13,17,21	1	CH	SILTY CLAY: high plasticity, red brown and light grey, with a trace of fine to medium grained sand and ironstone gravel.	MC<PL	H	>600 >600 >600	
	N = 37 11,16,21	2	CH	SANDY CLAY: high plasticity, red brown and light grey, with a trace of ironstone gravel.			>600 >600 >600	
	N = 36 11,15,21	3	SC	CLAYEY SAND: fine to medium grained, red brown and light grey, with ironstone gravel.	M	D		
		4	SP	SAND: fine to medium grained, light grey and orange brown, with occasional silty clay seams.				
	N > 29 18,25, 4/10mm END	5		as above, but orange brown, red and light grey.				
			SC	CLAYEY SAND: fine to medium grained, red brown and light grey.				
	N > 17 12,17/ 150mm END	6	SC/CL	CLAYEY SAND/SANDY CLAY: fine to medium grained, high plasticity, light grey.	M/ MC<PL	D/ H		
		7	SP	SAND: fine to medium grained, light grey and brown.	M	D		

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Borehole No. **1004**
2/4

BOREHOLE LOG

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA
Date: 1-6-06
Method: SPIRAL AUGER & WASHBORING EDSON 3000
R.L. Surface:
Datum:
Logged/Checked by: M.T./A3H

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
					SP	SAND: fine to coarse grained, light grey and brown.	M	D		
	DS	N > 11 17,11/ 50mm END								
			8							
					SP	SAND: fine to coarse grained, dark grey, with clay seams.				
▶	DS						W			
			9							
						SAND: fine to coarse grained, yellow brown, with clay fines.		MD		
			10							
	DS	N = 11 7,4,7				as above, but with clay bands.				
			11							
			12							
	DS	N = 27 5,13,14								
					SC/CH	CLAYEY SAND/SANDY CLAY: fine to medium grained, high plasticity, brown and light grey.	W/ MC>PL	-		COMMENCE ROTARY WASHBORE DRILLING
			13							
			14							

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BOREHOLE LOG

Borehole No. **1004**

3/4

Client: NSW DEPARTMENT OF COMMERCE

Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT

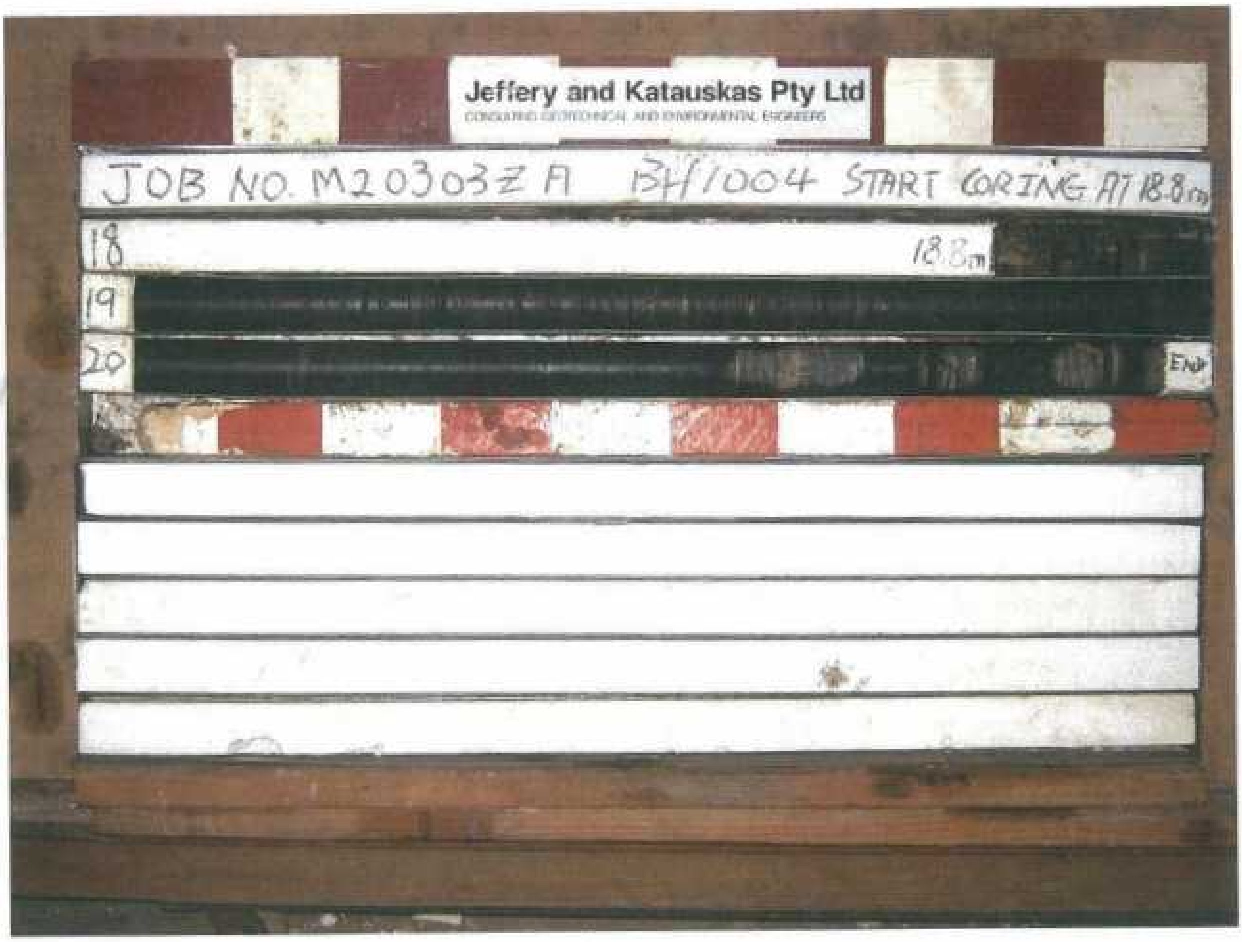
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA **Method:** SPIRAL AUGER & WASHBORING EDSON 3000 **R.L. Surface:**

Date: 1-6-06 **Datum:**

Logged/Checked by: M.T./ASH

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
								SP	SAND: fine to coarse grained, brown and grey.	W	-		CONTINUOUS WASHBORE DRILLING (ie NO INSITU TESTING) FROM 12.5m DOWN TO 18.8m IN ORDER TO PROVE BEDROCK
						15			SAND: fine to coarse grained, dark grey.				
						16							
						17							
						18							
						19			REFER TO CORED BOREHOLE LOG				
						20							


Jeffery and Katauskas Pty Ltd
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JOB NO. M20303ZA BH/1004 START CORING AT 18.8m
18
18.8m
19
20
END

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Borehole No. **1004**
4/4

CORED BOREHOLE LOG

Client: NSW DEPARTMENT OF COMMERCE
Project: PROPOSED LIVERPOOL HOSPITAL REDEVELOPMENT PROJECT
Location: LIVERPOOL HOSPITAL, NSW

Job No. M20303ZA | **Core Size:** NMLC | **R.L. Surface:**
Date: 1-6-06 | **Inclination:** VERTICAL | **Datum:**
Drill Type: EDSON 3000 | **Bearing:** - | **Logged/Checked by:** M.T./AJH

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DESCRIPTION Specific	General
		18		START CORING AT 18.80m						
FULL RET-URN		19		SHALE: dark grey, with fine grained, light grey sandstone laminae, bedded at 0°-5°, spacing up to 20mm.	Fr	H	× ×		- Cr, 3mm.t - Cr, 5mm.t - XWS/Cr, 7mm.t - XWS/Cr, 10mm.t - Cr, 10mm.t - XWS/Cr, 8mm.t	
						M-H	×			
		20		as above, but spacing up to 5mm.		H	×			
		21		END OF BOREHOLE AT 20.95m						
		22								
		23								
		24								

REPORT EXPLANATION NOTES

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
ABN 17 003 550 801

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

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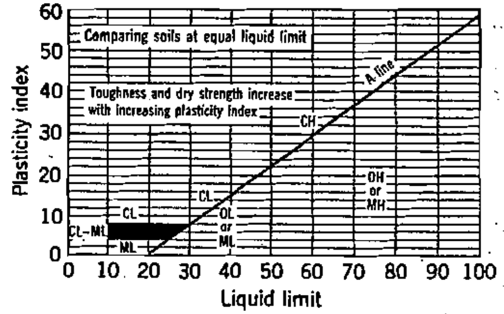
UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 µm and basing fractions on estimated weights)				Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than 75 µm sieve size[b] (The 75 µm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 µm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols. $C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below "A" line, or *PI* less than 4. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	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*)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7

Fine-grained soils More than half of material is *smaller* than 75 µm sieve size	Identification Procedures on Fraction Smaller than 380 µm Sieve Size				Group Symbols	Typical Names	Information Required for Describing Soils
		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)			
	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. GW-GC, well graded gravel-sand mixture with clay fines).

2) Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

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GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

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

ROCK

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

DEFECTS AND INCLUSIONS

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

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

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LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
	—C—	Extent of borehole collapse shortly after drilling.
		Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY - runs freely through fingers.
	M	MOIST - does not run freely but no free water visible on soil surface.
	W	WET - free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT - Unconfined compressive strength less than 25kPa
	S	SOFT - Unconfined compressive strength 25-50kPa
	F	FIRM - Unconfined compressive strength 50-100kPa
	St	STIFF - Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF - Unconfined compressive strength 200-400kPa
	H	HARD - Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose — <15 — 0-4
	L	Loose — 15-35 — 4-10
	MD	Medium Dense — 35-65 — 10-30
	D	Dense — 65-85 — 30-50
	VD	Very Dense — >85 — >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T 60	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

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

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science end 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	