REPORT

TO
MANLY CIVIC CLUB JV

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

AT
2 WEST PROMENADE, MANLY, NSW

24 May 2016
Ref: 21496LBrpt

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1 INTRODUCTION

This report presents the results of geotechnical investigations for the proposed redevelopment of the Manly Civic Club at 2 West Promenade, Manly, NSW. This investigation was commissioned by Mr Stewart Nettleton of the Manly Civic Club, in consultation with Mr Jim Simons of Eastview (Aust) Pty Ltd.

Geotechnical investigations have been carried out at the site in several stages due to revisions to the proposed development and due to access constraints before and after demolition of the previous club building on the site. Three previous geotechnical investigation reports have been prepared by Jeffery and Katauskas Pty Ltd and JK Geotechnics as follows:

- Report dated 14 September 2007, Ref: 21496SBrpt, involving the Electrical Friction Cone Penetrometer tests EFCP1 and EFCP2 and borehole BH3.
- Report dated 5 November 2010, Ref: 21496SB1rpt, involving EFCP101, EFCP102, EFCP104, EFCP105, BH101 and BH103.
- Report dated 17 September 2012, Ref: 21496SB2rpt, involving EFCP201, EFCP202, BH203 and BH204.

The current investigation comprised Dilatometer testing (DMT) to better profile the natural sands to assist with the design of a raft slab. This report incorporates all geotechnical investigation results obtained for the site and this information has been used as a basis for comments and recommendations on excavations, groundwater, retention and footings. This report supersedes all our previous geotechnical reports for the site.

The geotechnical investigations have been carried out in conjunction with various environmental investigations by our specialist division, Environmental Investigation Services (EIS). Reference should be made to the reports by EIS, Ref: E21496K, for the results of the environmental investigations.

2 PROPOSED DEVELOPMENT

The current proposed development is shown in the supplied architectural drawings by Mijollo International Pty Ltd (Ref: 1548, Drawing Nos A201 to A207, A301 to A304 and A401 to A402, revision P7, dated 12 May 2016). Based on these drawings the new building will have six above ground levels, containing a ground floor Manly Civic Club and five levels of residential units above, over two basement levels. The ground floor of the new building will join to the rear of the existing service station located in the south-eastern corner of the site. The rear portion of the existing building will be removed to allow construction of the basement and allow the existing building to be incorporated as part of the new club.

The lowest basement will grade along parking ramps ranging from RL-0.85m to RL-2.89m, which will require excavation to depths ranging from about 5m to 7.5m. The basement will extend to the site boundaries, apart from the south-eastern corner due to the existing service station building.

3 INVESTIGATION PROCEDURE

The fieldwork for the geotechnical investigation has been carried out over several stages from 2007 to 2016. The subsurface investigations carried out at the site are summarised as follows:

- Electrical Friction Cone Penetrometer (EFCP) tests EFCP1, EFCP2, EFCP101, EFCP102, EFCP104, EFCP105, EFCP201 and EFCP202. EFCP1 and EFCP2 were carried out in 2007; EFCP101, EFCP102, EFCP104 and EFCP105 were carried out in 2010; and EFCP201 and EFCP202 were carried out in 2012. EFCP1, EFCP101, EFCP102, EFCP104, EFCP105, EFCP201 and EFCP202 were carried out to refusal at depths of 16m, 23.42m, 15.91m, 28.19m, 30.47m, 13.76m and 20.31m, respectively, below the existing ground surface. EFCP2 was terminated without refusal at a depth of 19.7m.
- Boreholes BH3, BH101, BH103, BH203 and BH204 drilled using our track mounted JK300 and truck mounted JK500 rigs. BH3 was drilled in 2007 to a depth of 9m. BH102 and BH103 were drilled in 2010; and BH203 and BH204 were drilled in 2012. These boreholes were auger drilled to depths of 23.63m, 16m, 27.51m and 15.35m, respectively, and were then continued by diamond coring techniques using a NMLC core barrel with water flush, to depths of 29.34m, 23.62m, 33.94m and 18.36m, respectively.
- Dilatometer tests (DMT) 301, 302, 304 and 305 completed in 2016. These tests were carried out to refusal at depths of 11.14m, 26.6m, 15.8m and 24.2m, respectively. DMT303 was attempted, but refused at a shallow depth of 2.4m and was moved slightly and DMT305 completed. Therefore, the results from DMT303 are not included herein.

The investigation locations, as shown on Figure 1, were set out by taped measurements from existing surface features and inferred site boundaries. The approximate surface levels, as shown on the EFCP test results, borehole logs and DMT results, were estimated by interpolation between spot levels shown on the supplied survey plan by Hill & Blume (Ref: 50188, dated 6/8/07). The datum of the level is Australian Height Datum (AHD). All investigation locations were checked for buried services at the start of each phase of the fieldwork by reference to available service plans and scanning of the locations using electronic service detection equipment as required.

EFCP testing involves continuously pushing a testing probe with a 35mm diameter conical tip into the soil using the hydraulic rams of our purpose built ballasted truck mounted EFCP rig. Measurements of the cone tip resistance and the frictional resistance of a separate 134mm long sleeve located directly behind the cone are made at 10mm intervals during testing. EFCP testing does not provide sample recovery. The subsurface material identification, including material strength/relative density, is by interpretation of the test results based on past experience, empirical correlations and comparisons with available borehole logs.

In BH3, the apparent compaction of the fill and the relative density of the natural sand was assessed from Standard Penetration Test (SPT) 'N' values. In BH101, BH103, BH203 and BH204 only limited testing within the upper soil profile was undertaken as the boreholes were located adjacent to or close to EFCP tests.

The strength of the cored sandstone was assessed with reference to Point Load Strength Index ($I_{S(50)}$) carried out on the recovered rock core. The point load strength index test results are summarised on the cored borehole logs and in the attached STS Tables B and B1.

DMT involves pushing a Dilatometer blade into the soil using the hydraulic rams of our purpose built ballasted truck mounted rig. The blade is stopped at intervals of 0.2m and the flexible membrane on the blade is inflated using pressurised gas. The pressure required to initially lift the membrane and the pressure to expand the membrane 1.1mm is recorded and from this several geotechnical parameters of the soil can be determined. Graphs of some of these parameters from the DMT are attached for each test and combined for the four tests completed.

Groundwater measurements were made within the boreholes during drilling and on completion of drilling. The EFCP test holes were checked for groundwater on completion of the tests. Slotted PVC standpipes were installed in BH3, BH103, BH203 and BH204 on completion to allow groundwater measurements to be taken and samples collected by EIS. Groundwater monitoring data loggers were installed within the standpipes during the following time periods to measure groundwater levels:

- In BH3 and BH103 groundwater data loggers were installed on 8 September 2010, downloaded on 27 September and removed on 15 October 2010. The results of this groundwater monitoring are shown in Figures 2 and 3.
- In BH3, BH103, BH203 and BH204 groundwater data loggers were installed on 29 August 2012 and removed on 12 September 2012. The results of this groundwater monitoring are shown in Figures 4 to 7.
- In BH3, BH103, BH203 and BH204, groundwater data loggers were installed on 21 March 2016 and removed on 3 May 2016. The results of this groundwater monitoring in BH3, BH103 and BH203 are shown in Figures 8 to 10. Unfortunately, the logger installed in BH204 malfunctioned and no useful data could be obtained.

The groundwater levels within the standpipes were measured during installation and removal of the data loggers and were compared with the levels measured by the data loggers. Daily rainfall records were obtained from the Australian Bureau of Meteorology for the monitoring station located at nearby Collaroy and are shown on Figures 2 to 10.

Our geotechnical engineers and engineering geologists set out the EFCP test, borehole and DMT locations, nominated the sampling and testing locations and prepared logs of the subsurface conditions encountered. The EFCP test results, including our interpreted subsurface profile, the borehole logs and DMT results are attached to this report, together with a set of report explanation notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Eight samples of the natural sands recovered from the boreholes were tested by Soil Test Services Pty Ltd (STS), a NATA registered laboratory, to determine particle size distributions. The results of these tests, and the point load strength index tests, are provided in the attached STS Tables A, B, A1 and B1.

4 RESULTS OF INVESTIGATION

4.1 Site Description

The site is located near the base of a moderately sloping hillside that slopes down toward the east, with the base of the hill located about 50m to the west of the site. The site itself is relatively level.

At the time of the 2012 and 2016 fieldwork, the previous Manly Civic Club building had been demolished and the majority of the site was covered with concrete floor slabs of the building and concrete parking areas. In the south-eastern corner of the site is a single storey rendered former service station building. The former service station building appeared to be in fair condition with a sub-horizontal crack observed within the north-western wall of the building.

To the east of the site is West Promenade, to the south is Gilbert Street and to the west is Eustace Street. On the far side of West Promenade is a grassed park and on the far side of Eustace Street is a residential unit building, which had been constructed between our 2012 and 2016 fieldwork. On the southern side of Gilbert Street is a ten storey apartment building, in good condition, with basement car parking. Access was possible within this basement and the lowest basement floor was measured to be about 3m lower than Gilbert Street. To the north of the site are two, three storey brick apartment buildings, located about 1.3m to 2m from the common boundary. These buildings appeared to be in good to fair external condition when viewed from within the subject site. From experience of similar buildings nearby it is likely that these buildings are founded upon high level footings.

4.2 Subsurface Conditions

In summary, the EFCP tests and boreholes encountered concrete pavements and fill covering natural sands with sandstone bedrock at depth. Further comments on the subsurface conditions encountered are provided below. Reference should be made to the EFCP test results and borehole logs for detailed descriptions of the subsurface conditions encountered.

Concrete

Concrete was penetrated at each location ranging from 60mm to 170mm thick. However, in BH101 two concrete slabs were penetrated of 140mm and 170mm thick, with a 100mm thick void between.

Fill

Fill was encountered in the boreholes to depths ranging from 1m to 2m. Fill was inferred within the EFCP tests to similar depths ranging from 1.4m to 2m. The fill observed in the boreholes comprised gravelly sand, clayey sand and silty sand with igneous and sandstone gravel and a trace of sandstone cobbles and brick and tile fragments. Based on the SPT ‘N’ values and the EFCP test results, the fill was generally assessed to be poorly compacted.

Natural Sand

The natural soils comprised sand, but some thin clay and silty sand bands were encountered in the EFCP tests between depths of about 6.2m and 9.9m. We note that such thin clay and silt layers could not be picked up within the boreholes, particularly where only limited testing was carried out in this case, and the EFCP plots. From the EFCP tests, the sands were initially of loose relative density becoming medium dense, dense and very dense with depth. However, some very loose or loose bands were encountered within the EFCP tests at the following depths:

- EFCP1 between depths of 7.6m and 9.3m.
- EFCP2 between depths of 6.9m and 10.5m.
- EFCP101 between depths of 7.8m and 10.6m.
- EFCP102 between depths of 7.1m and 8.8m.
- EFCP104 between depths of 8.7m and 12.8m.
- EFCP105 between depths of 6.25m and 10.3m.
- EFCP201 between depths of 6.3m and 9.65m.
- EFCP202 between depths of 6.7m and 11.85m.

These results show that the very loose or loose bands occur to greater depths within the eastern tests. Below the above depths, the relative density of the sands was medium dense and then dense, with some very dense bands.

The DMT results also indicate similar loose bands within the depths given above, when the test results are compared with the nearby EFCP test results.

Sandstone

Sandstone was encountered in BH101, BH103, BH203 and BH204 at depths of 23.45m, 16m, 27.51m, and 15.2m, respectively. However, in BH203 banded clayey sand and sandstone was encountered from a depth of 25.5m. EFCP1, EFC101, EFCP102, EFCP104, EFCP105, EFCP201 and EFCP202 refused at depths of 16m, 23.42m, 15.91m, 28.19m, 30.47m, 13.76m, and 20.31m, respectively, which by comparison with the borehole results appears to have occurred on the surface of the sandstone. The lower portions of the EFCP tests before refusal indicate that the soils may be residual. EFCP2 was terminated at a depth of 19.7m without refusal and so would be above the level of the sandstone. Overall the depth of the sandstone increases from the western end of the site to the eastern end, from about 16m to about 30m.

In BH101 and BH103, the sandstone was of variable quality with significant section of core loss up to 1.4m thickness. The recovered sandstone was distinctly or slightly weathered and was predominantly of low or medium strength, but with bands of extremely low strength sandstone and high strength sandstone. The core loss zones are likely to represent extremely weathered or clay bands and could be due to undercuts being present in buried cliff lines below the site. High strength sandstone was encountered in BH101 and BH103 below depths of 28.4m and 23.2m, respectively, but this was only proven for short lengths.

In BH203 and BH204, the sandstone was of better quality, with no core loss zones encountered. This sandstone was distinctly weathered to fresh and was generally of medium or high strength, with occasional thin low strength bands.

Defects within the recovered core comprised extremely weathered seams, clay seams and joints inclined at 40° to 90°.

Groundwater

Groundwater seepage was encountered during the drilling of BH3, BH101, BH103, BH203 and BH204 at depths of 3.5m (≈RL0.4m), 3.6m (≈RL0.2m), 3.5m (≈RL0.3m), 3.0m (≈RL1.3m), and 3.0m (≈RL1.0m), respectively. The EFCP test holes collapsed on completion at depths ranging from 2.6m to 3.4m (≈RL1.2m to ≈RL0.7m). We note that in sands, collapse usually occurs at about the groundwater level.

The results of the groundwater monitoring within the standpipes within the three monitoring periods are summarised as follows:

- During the monitoring period between 8 September 2010 and 15 October 2010 (Figures 2 and 3), the groundwater levels in BH3 and BH103 were generally constant, with groundwater levels ranging between about RL0.7m and RL0.8m.
- During the monitoring period between 29 August 2012 and 12 September 2012 (Figures 4 to 7), the groundwater levels in BH3, BH103, BH203 and BH204 had a slight drop in levels over the monitoring period, from about RL0.7m-RL0.75m to about RL0.5m-RL0.6m, but then rose at the end of the period to RL0.85m-RL1.1m due to rainfall that occurred.
- During the monitoring period between 21 March 2016 and 4 May 2016 (Figures 8 to 10), the groundwater levels in BH3, BH103 and BH203 were generally consistent, with groundwater levels ranging between about RL0.7m and RL0.95m, with a slight fall over the monitoring period most likely due a lack of to rainfall over the monitoring period.

4.3 Laboratory Test Results

The particle size distributions on samples of the sand from BH101, BH103, BH203 and BH204 all showed that the samples were sand, with between 0% and 3% fine gravel (i.e. larger than 2.36mm, but smaller than 6.7mm) and between 1% and 6% fines (i.e. smaller than 0.075mm).

The point load strength index tests results generally confirmed our field assessment of rock strength. The Unconfined Compressive Strength (UCS) of the sandstone, estimated from the point load strength index test results, generally ranged from 2MPa to 36MPa, with some higher results in BH204 of 46MPa to 52MPa.

5 COMMENTS AND RECOMMENDATIONS

5.1 Geotechnical Issues

Based on the results of the geotechnical investigations carried out at the site, the main geotechnical issues for the proposed development are as follows. Further comments on these issues are provided within the following sections of this report.

- The excavations for the proposed basement will encounter sandy fill and natural sands that will not be self supporting and full depth retention systems will need to be installed prior to the start of the excavations. Where these walls are required adjacent to the buildings to the north and next to the former service station building, the walls will need to be as rigid as possible to limit adjacent ground movements. Lateral support of the retaining walls will be required, which will need to be installed progressively during excavation.
- If anchors are installed for lateral support, the lower anchors will need to be constructed with care as they will be below the groundwater table. If the anchors are not drilled with care then loss of soils through the anchor holes can occur, which may result in settlement behind the walls. Our preference would be for the adoption of top-down construction methods, which includes internal propping using the floor slabs as the excavation deepens. This would reduce the risk of unacceptable wall movements.
- Groundwater was encountered at levels ranging from ≈RL0.5m to ≈RL1.1m. The lowest basement is currently designed to be at RL-0.85m to RL-2.89m, which is below the Groundwater table. Therefore, dewatering will be required in order to lower the groundwater level and allow construction of the basement. Such dewatering will need to be carried out with care due to the risk of the drawdown of the groundwater causing settlement below the footings of the adjoining buildings. Analysis has shown that in order to limit the groundwater drawdown outside of the excavation some form of barrier of limited permeability will be required below the retention system. The groundwater levels around

the basement will need to be carefully monitored during the works to assess the amount of drawdown induced. In the long term, the retaining walls and the basement floor slab will need to be designed to resist hydrostatic uplift forces, i.e. a 'tanked' basement. Significant uplift loads will need to be resisted.

- The use of a raft slab appears feasible, but raft slabs are suited to a uniform excavation level and uniform building loads. Therefore, the basement will need to be formed with a uniform bulk excavation level and the car park ramps constructed off the raft slab. Detailed analysis of a raft slab will be required to assess settlements. During construction careful preparation of the subgrade for the raft slab will be required.
- The depth to the sandstone bedrock is variable ranging from about 15m to 16m on the western end of the site to about 30m at the eastern end. Due to these variable depths the surface of the sandstone is likely to comprise a series of steps or buried cliff lines and may contain undercut sections within those cliff lines. The core loss zones encountered in BH101 and BH103 may be due to these buried undercut cliffs. Therefore, if piles founded within the rock are adopted, care would be required that the piles are socketed into the continuous bedrock and not on undercut sections. The designer/constructor may require additional cored boreholes at specific locations to profile the sandstone in more detail and assess the presence of the undercut sections.
- The excavated fill, natural sands and groundwater will need to be disposed of appropriately taking into account the results of the environmental investigation by EIS.

The main issues for this development are associated with groundwater and the construction of basement levels below the groundwater table which will be difficult and costly. Detailed design of the retention system, raft slab, dewatering system and the tanked basement will be required, including detailed geotechnical analysis. If the architectural design can be revised to raise the bulk excavation level this would make design and construction of the basement less complicated and less costly.

5.2 Excavations and Adjacent Buildings

Prior to the start of excavations, detailed dilapidation surveys should be carried out on the adjacent buildings to the north of the site. Even if such reports were prepared prior to demolition of the previous club building there has been a few years since demolition and we recommend that the reports be updated to record the conditions immediately prior to the start of excavation. These dilapidation surveys should comprise detailed inspection of the buildings both externally and internally with all defects rigorous described, i.e. defect type, location, crack widths, crack lengths, etc. The respective owners of the buildings should be asked to confirm that the dilapidation surveys represent a fair record of actual conditions. Such reports will also help to guard against opportunistic claims for damage that was present prior to the start of excavations.

Excavations to the expected depths of 5m to 7.5m will encounter surface fill and natural sands and such soils will be able to be excavated using conventional earthmoving equipment, such as the buckets of hydraulic excavators. However, the sands will not be self supporting and full depth retention systems will be required as described in Section 5.4 below.

The excavated fill and natural soils will need to be disposed of appropriately. Reference should be made to the reports by EIS for appropriate disposal of the excavated materials.

5.3 Groundwater

The results of the groundwater monitoring show groundwater levels ranging from about RL0.5m to RL1.1m, or about 3m to 4m below the ground surface. As excavations are proposed to depths of about 5m to 7.5m temporary dewatering will be required to allow construction of the basement. Such dewatering must be carried out with care due to the risk of settlement of the soils below the buildings to the north and the service station building due to the resulting drawdown of the groundwater. Given the age of the buildings, it is likely that they are supported on shallow footings, which would be susceptible to settlement.

We have carried out some preliminary groundwater analysis of one section across the site to give a preliminary indication of the depth of shoring walls needed to achieve adequate 'cut-off' to limit groundwater drawdown.

The location of the section analysed is shown on Figure 1. This section was adopted in order to pass through BH101 to assist with assessing the subsurface profile. The seepage analysis was carried out using the 2D finite element computer program Seep/W 2012 (from Geo-Slope International Ltd). The overall methodology used for the seepage analysis was as follows:

- Construct the surface and subsurface model for the section utilising existing surface levels from the supplied survey and EFCP and borehole information.
- Carry out steady state seepage analysis for the section after excavation and during dewatering with various wall toe levels.

The analysis was based on the following assumptions:

- The subsurface profile is horizontally constant across the section and is represented by the results of EFCP101/BH101 and EFCP202 and BH203. We note that the surface of the sandstone has been modelled at RL-19.7m.
- The groundwater level before dewatering was assumed to be constant at RL1.0m, which is close to the highest groundwater level measured within the standpipes.
- Shoring walls will be located on both boundaries and will be completely waterproof.
- The bulk excavation level for the basement will be RL-3.4m, which allows for about a 0.5m thick raft slab below the lowest basement floor level of RL-2.89m. We note that the basement level varies throughout the site, but for simplicity we have adopted this constant bulk excavation level. We also note that it is expected that a raft slab will be adopted, which will require a horizontal bulk excavation level.
- The location and spacing of the dewatering points have been uniformly spaced across the section. Optimisation of the locations, spacing and depths of the dewatering points was outside the scope of this analysis.
- Estimation of the amount of water that will be removed from the excavation was outside the scope of this analysis.
- The groundwater level inside the basement has been drawn down to RL-4.0m, i.e. 0.6m below the assumed bulk excavation level.
- The horizontal and vertical permeability has been assumed to be equal.

In order to carry out the analysis the coefficient of permeability for the sands and for the sandstone have been assumed. Based on the investigation results, including the particle size distributions, we have adopted a saturated coefficient of permeability for the sands of 1×10^{-4} m/sec and for the sandstone of 1×10^{-7} m/sec. We note that the permeability of the sandstone will be governed by the defects within the sandstone as the majority of the flow will occur through joints and bedding partings within the sandstone.

For the adopted section, analysis was completed for different shoring toe levels so that the groundwater and the drawdown at the site boundary could be assessed. The results of the analysis are summarised in the table below and in Figures 11 to 13.

Shoring Toe Level	Initial Groundwater Level	Groundwater Level During Dewatering at Boundary	Drawdown in Groundwater Level at Boundary
RL-10m (in sand)	RL1.0m	RL-1.5m	2.5m
RL-17.5m (in sand)	RL1.0m	RL-0.5m	1.5m
RL-20.2m (0.5m into sandstone)	RL1.0m	RL1.0m	0m

The above results show that if the shoring walls are terminated within the sands significant drawdown of the groundwater outside of the basement will occur. Such drawdown will result in settlement of the sands below the adjoining buildings. However, extension of the shoring to cut-off into the sandstone will be costly and may be difficult to achieve due to the potential for undercut sections within buried sandstone cliff lines and the potential for variable sandstone depths.

Options that could be considered to limit the drawdown could be the use of grout injection to form impermeable layers within the soils below the site to impede water flow. Construction of a horizontal grout blanket is likely to be difficult as it would need to be quite thick to resist the hydrostatic uplift pressures and would be extensive as it would need to extend fully between all shoring walls. Grouting below the toe of the shoring walls may be possible to create a grout curtain around the perimeter of the basement to impede water flow between the toe of the shoring and the underlying bedrock.

To assess the effectiveness of a grout curtain we have carried out analysis with the shoring toe at RL-10 and a lower permeability layer between the shoring and the surface of the rock (i.e. a grout curtain). We have analysed this for a permeability of the grout curtain of 1×10^{-6} m/sec and 1×10^{-7} m/sec and the results are summarised in the table below and in Figures 14 and 15.

Grout Curtain Permeability	Initial Groundwater Level	Groundwater Level During Dewatering at Boundary	Drawdown in Groundwater Level at Boundary
1×10^{-6} m/sec	RL1.0m	RL0.2m	0.8m
1×10^{-7} m/sec	RL1.0m	RL0.8m	0.2m

From these results the use of a grout curtain would be effective in limiting the drawdown outside of the basement. Obviously, the lower the permeability of the grout curtain the lower drawdown outside of the basement would occur.

Further groundwater seepage analysis should be carried out as part of the design of the dewatering system, which should include estimates of the volume of water that will need to be removed to lower the groundwater levels and then maintain them during construction. The sections used for that analysis should be prepared once the architectural and structural designs have been finalised and the bulk excavation levels and the required level of groundwater drawdown inside the basement are known. Information should also be sought on the permeability that can be achieved for the grout curtain in order to limit groundwater flow.

During dewatering the groundwater levels outside of the basement must be monitored to assess if the drawdown is excessive. We recommend a limit on drawdown outside of the basement of no more than 1m. If it is found that the drawdown outside of the basement is excessive, measures may need to be taken to limit the drawdown, such as by further grouting or the injection of water into the ground outside the basement. However, since the proposed basement will extend to the boundaries, space will not be available for any monitoring and reinjection wells inside the site and permission will need to be obtained from the owners of the adjoining properties to install monitoring wells within their properties. We recommend that at least two monitoring wells be installed on the northern side of the site, and one each to the west, south and east. The wells should be monitored continuously during initial dewatering inside the basement and then daily during steady state dewatering to maintain the lowered groundwater at the required level. If these daily results show little change in groundwater levels then the frequency of the readings may be decreased.

In the long term, the proposed basement will need to be designed to resist hydrostatic uplift forces, i.e. a tanked basement. Allowance for a rise of at least 1m above the measured groundwater levels should be allowed for in the design, i.e. at least RL2m. This will result in quite large uplift forces as the basement will be several metres into the groundwater. Raising of the basement level will reduce these forces and may result in significant cost savings. The dewatering will need to continue during construction until the weight of the structure is sufficient to resist the hydrostatic uplift forces. The structural engineer will need to advise when in the construction program the dewatering can ease.

5.4 Retention

The sands will not be self supporting and a full depth retention system will need to be installed prior to the start of excavation. The retention system will need to be quite rigid where it will be constructed adjacent to the existing service station building on the south-eastern corner of the site and on the northern side close to the adjoining buildings. The retention system will also need to be quite deep to extend below the base of the proposed excavation and to achieve sufficient toe level to maintain stability and limit groundwater drawdown.

Retention systems that could be considered for this site include the following:

- Diaphragm walls, which comprise concrete cast-in-place walls formed within a trench supported by bentonite slurry. Such walls are practically impermeable if constructed properly and the surface finish may be of sufficient quality that they can be used as the final basement walls.
- Secant pile retaining walls formed by overlapping piles constructed using auger, grout injected (CFA) piles. Given the depth of the retention system care must be taken that the piles are straight and have sufficient overlap to impede groundwater flow. This would be almost impossible to achieve with conventional uncased piles, which could also cause issues with settlement due to sands being drawn into the auger during drilling. Therefore, if secant pile walls were to be adopted we recommend the use of cased, double rotary drilling methods, where steel casing is drilled with the auger so that the auger is at all times contained within the casing. This also has the advantage of helping keep the piles straight.
- Cutter Soil Mix (CSM) which are formed in place by cutting and mixing the soils with a cement to form a 'watertight' wall. The walls are constructed in sections or panels with reinforcement placed in each panel after mixing. The walls may be formed with a relatively high finish so they can be used as the permanent basement walls.

No matter which retention system is adopted, it will need to be designed and constructed in conjunction with a grout curtain below the toe of the shoring extending to the rock to cut-off the groundwater inflow and limit drawdown outside of the basement as discussed in Section 5.3.

Lateral restraint of the retention systems will be required in the form of internal props or external anchors, which must be installed progressively as each restraining point is uncovered. Permanent lateral support would be provided by the floor slabs.

Our preference would be to adopt top-down type construction methods, where each slab is constructed before excavating below it so that the unexcavated material provides passive support while the slabs are constructed, and the slab provides permanent lateral support. For this type of construction the ground floor slab would be constructed prior to excavation, excavations then carried out to the first basement slab level and that slab constructed and so on until the lowest basement slab is constructed. Such methods would eliminate the risks of external anchors as discussed below. Alternatively, an internal propping system could be used, but this would need to be carefully designed and planned so that the props do not interfere with slab construction. In addition, the props would need to be able to be stressed.

If external anchors are used they must be installed with care as drilling of anchors in sand below the water table can result in loss of soil and resulting settlement of the ground above. If anchors are adopted, permission will need to be obtained from the owners of the adjoining properties prior to installation of anchors below those properties. Such permission can take some time to obtain and should be sought at an early stage to allow for negotiation. The design of anchors will also need to take into account the location and depth of any nearby basements and buried services.

We do not recommend the use of uncased anchor holes as these would almost certainly result in loss of soil and resulting surface settlement. All anchor holes should be cased to limit the flow of sands from around the anchor hole. Only the services of an anchoring contractor with a proven record of installing anchors in sand below the water table should be considered. Even with experienced contractors, there will still be some risk of settlements during anchor construction due to the inherent difficulties of construction of anchors in sand below the water table.

Propped or anchored walls may be preliminarily designed using a rectangular earth pressure distribution of magnitude 6H kPa, where H is the retained height in metres. In areas that are sensitive to adjacent movements, such as near the buildings to the north and the former service station building that is to remain, a higher earth pressure distribution of 8H kPa should be used to limit deflections. These pressures assume horizontal backfill surfaces and where inclined backfill is proposed the backfill would need to be taken as a surcharge load. All surcharge loads should be allowed for in the design, including full hydrostatic pressures. We recommend that for these walls that detailed analysis of the retention system be undertaken using software such as Wallap or Plaxis. We have found that the completion of such analysis usually results in significant saving in construction costs. We can assist with such analysis if required.

A preliminary effective friction angle of 26° may be adopted for anchors bonded into loose sands or 30° for anchors bonded into medium dense sand. The anchor design and construction of anchors should also satisfy the following conditions:

- Anchor bond length of not less than 3m, beyond the ‘active’ zone of the retention system, which may be taken as a line drawn up at 45° from the base of the excavation.
- Overall stability, including anchor group interaction, is satisfied.

- All anchors are respectively proof loaded to at least 1.3 times their design working load before locking off at about 80% of their working load.

Shoring wall designs need to assess wall movements, bending moments and shear forces at each stage of the excavation and propping/anchoring. The designer must then assess the potential impact of the wall movements on adjoining structures and services. Where wall movements cause resulting ground movements in excess of what the adjoining structures and services can resist without damage, then underpinning of the adjoining structures would be required. In our opinion it will be difficult to design and construct a showing wall in these subsurface conditions without at least a moderate risk to the nearby structures. Therefore, we recommend that some underpinning of the adjoining structures be provided prior to commencement of shoring wall construction. The extent of underpinning would need to be further assessed once initial shoring wall design and expected wall movements have been determined.

5.5 Footings

5.5.1 Raft Slab

We understand that a raft slab is being considered to support the proposed building. Based on the results of the investigation the use of a raft slab would be feasible. However, the use of a raft slab would only be possible for a uniform excavation depth. The current lowest basement level varies between RL-0.85m and RL-2.89m and such a variation would not be able to be accommodated by a raft slab. This could be overcome by excavating to a common level for the raft slab and then constructing the final basement car park ramps elevated above the raft slab. The use of a raft slab also requires relatively uniform loading and high load concentrations will need to be avoided.

If a uniform excavation level and slab loading cannot be achieved then a piled footing system would be more appropriate.

Detailed analysis of a raft slab would be required to determine the settlements below the raft. Such analysis requires the building loads and raft layout to be known and may be based on the DMT results obtained. We note that the DMT results show lower density layers just below the expected bulk excavation level, which is not ideal, but provided the subgrade is properly compacted during construction we consider that these layers will not make a raft slab unfeasible. If excessive settlements are estimated below the raft the use of a piled raft could be considered in order to limit settlements.

We can provide estimates of the settlement below a raft slab once the building loads and raft layout are known. We recommend that such estimates be made using the finite element software, Plaxis, which is able to analyse the variable depth to the rock. Since the depth of the rock varies from about 15m to about 30m from one end of the site to the other, if a simple linear elastic analysis of the settlements was carried out this may overestimate the settlement below the raft due to the variable depth of soil.

The performance of raft slabs is dependent on the subgrade preparation below the raft, and if carried out correctly this preparation will improve the density of the lower density layers encountered just below the expected bulk excavation level. The use of a granular layer will be required to allow compaction of the natural sands and protect them from loosening during construction. We recommend that a base quality granular layer of at least 200mm thickness be placed and compacted over the subgrade. Following compaction of the subgrade it will need to be inspected by a geotechnical engineer and Dynamic Cone Penetrometer (DCP) testing carried out to assess the quality of the subgrade.

5.5.2 Piles

If settlement of the raft is excessive or if high concentrated loads are to be supported, the proposed structure will need to be supported on piles. Piles founded within the sands will have limited capacity at the western end of the site as there will not be sufficient depth of sand to achieve pile embedment due to the depth of the rock. Therefore, if piles were adopted, at the western end they would probably need to be founded within the sandstone so the remaining piles should also be founded within the sandstone to provide uniform support.

Due to the sands and the groundwater, auger, grout injected (CFA) piles will be required for this site. Alternatively, auger screw displacement type piles (i.e. ‘Atlas’ or ‘Omega’ type piles) could be used. The piles will be quite deep at the eastern end of the site and large piling equipment will be needed to reach such depths. The sandy subgrade would not be suitable to support large piling equipment and a working platform of granular material would be required. The working platform may be quite thick given the size of the equipment that would be required to reach the sandstone.

The upper sandstone in BH101 and BH103 is of variable quality with many core loss zones encountered. Therefore, reduced bearing pressures recommended for this upper sandstone. In BH203 and BH204 core loss zones were not encountered and the sandstone was more consistent within these boreholes. The sandstone encountered in each borehole has been classified in the table below.

Borehole	Depth and Reduced Level of the start of Each Class of Sandstone			
	Class V		Class IV	
	Depth (m)	≈RL (mAHD)	Depth (m)	≈RL (mAHD)
BH101	23.5m	-19.7m	26.3m	-22.5m
BH103	16.0m	-12.2m	21.5m	-17.7m
BH203	27.5m	-23.2m	27.5m	-23.2m
BH204	15.2m	-11.2m	15.7m	-11.7m

Piles may be designed based on the following parameters for each class of sandstone given above.

Sandstone Class	Allowable End bearing Pressure for Serviceability	Allowable Shaft Adhesion in Compression	Ultimate End Bearing Pressure	Ultimate Shaft Adhesion in Compression	Elastic Modulus
Class V	1000kPa	100kPa	3MPa	150kPa	80MPa
Class IV	2000kPa	200kPa	10MPa	500kPa	400MPa

For the design of piles in uplift, shaft adhesions of half the shaft adhesions given for compressive loads may be adopted. The above serviceability parameters are based on settlement at the pile toe of less than 1% of the pile diameter. The ultimate parameters may be used on the understanding that settlement of the piles may be 5% or more of the pile diameter and hence the ultimate limit state calculations must be coupled with serviceability limit state calculations. The pile settlements should be assessed using the modulus values given. Appropriate load factors and geotechnical strength reduction factors, in accordance with As2159-2009, must be used in the design.

As can be seen the piles on the southern side of the site will need to be socketed well into the sandstone and will need to penetrate through high strength sandstone layers to reach the Class IV sandstone. Therefore, large capacity rigs will be required in order to drill through these layers and to reach the required depths. Advice should be obtained from specialist piling contractors on the equipment required to construct the piles.

Higher bearing pressures, say allowable bend bearing pressures of 3500kPa to 6000kPa, would also be possible within the medium or high strength sandstone for individual piles. However, if higher bearing pressures are adopted, we recommend that additional cored boreholes be drilled at specific pile locations to confirm the depth where such sandstone is present. Given the depth of the sandstone and the presence of weak zones within the sandstone, even if the parameters given for Class IV sandstone are adopted we recommend that additional cored boreholes be drilled at specific pile locations to determine the founding depths of the piles.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed boreholes and 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 JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended.

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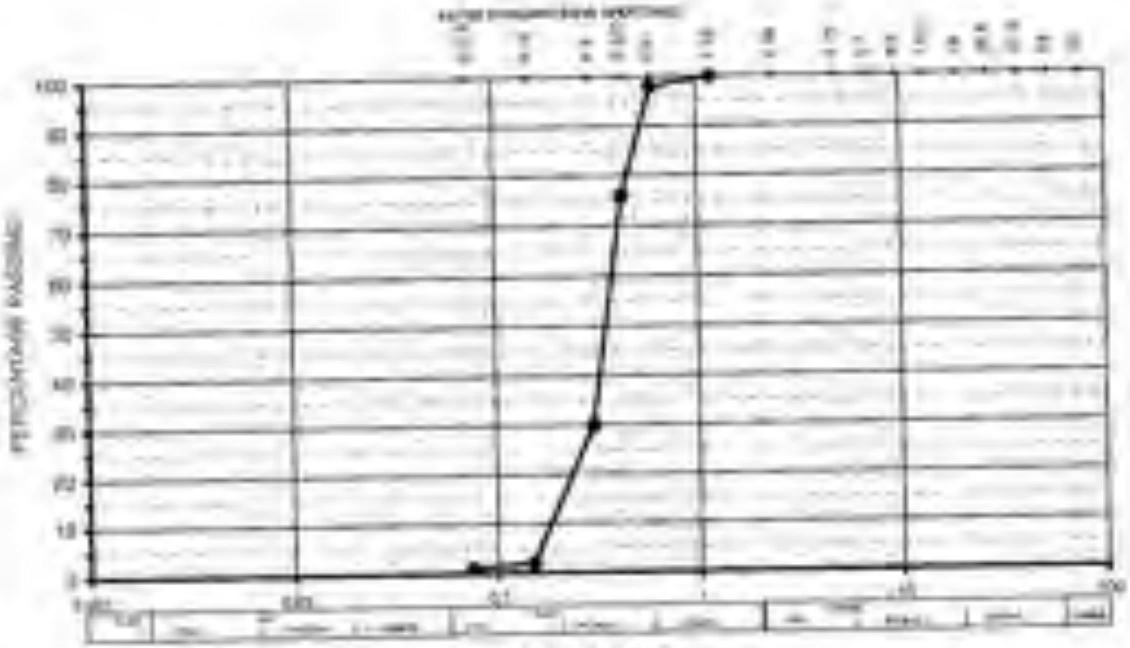


SIEVE ANALYSIS RESULTS

SIEVE SIZE	% PASSING
2.36 mm	100
1.18 mm	100
600 um	98
425 um	76
300 um	30
150 um	2
75 um	1

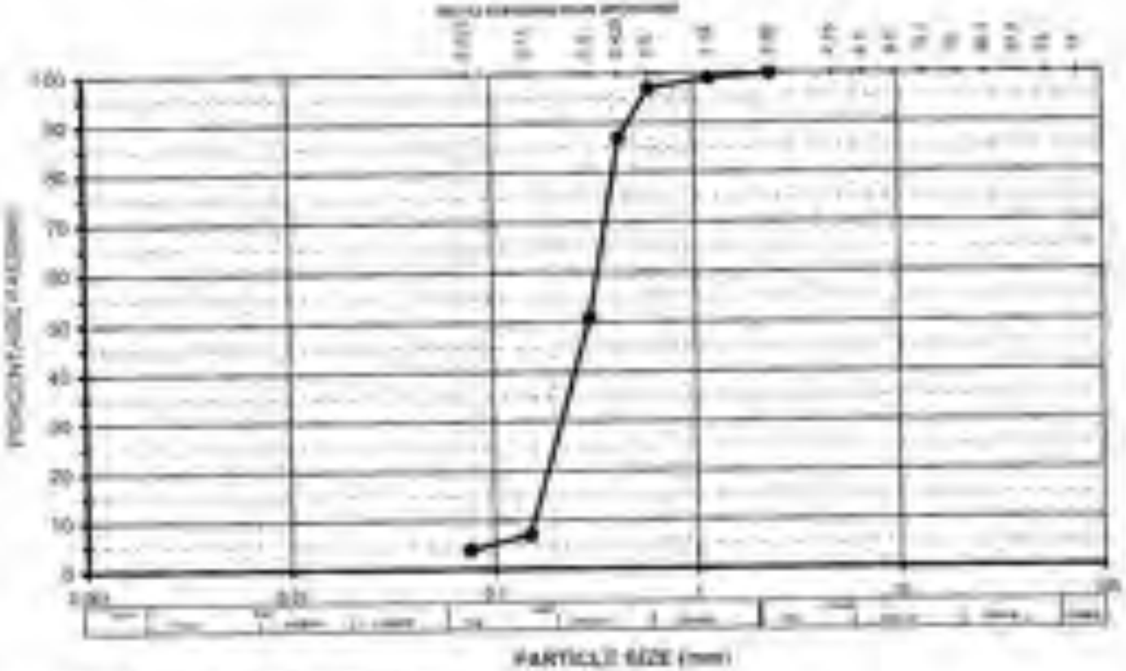
Test Method: AS1289.3.6.1 Dry Sieve(Washed)

- Notes:
- Please refer to appropriate notes for soil descriptions

PARTICLE SIZE DISTRIBUTION CURVE

Ref No: 21496SB1
Borehole Number: 101
Depth(m): 4.50-5.00
Table A Page 1 of 4

NATA Accredited Laboratory
Number 1327



Test Method: AS1289 3.6.1 Dry Sieve(Washed)

* **Notes:**
* Please refer to appropriate notes for soil descriptions

SIEVE ANALYSIS RESULTS

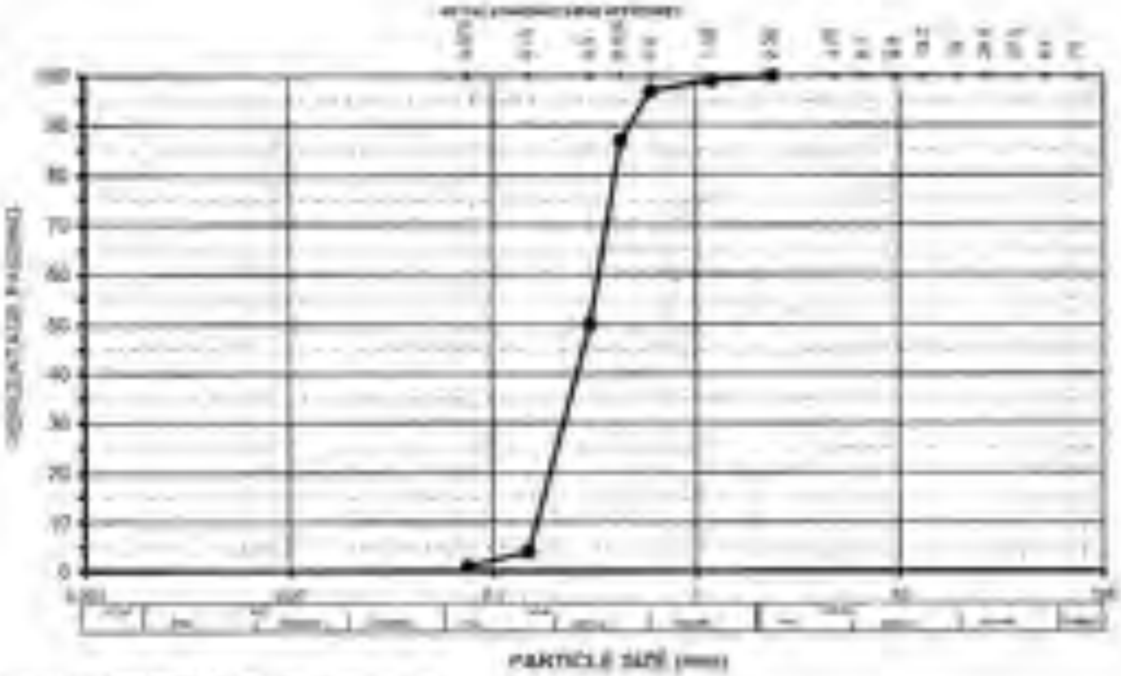
SIEVE SIZE	% PASSING
2.36 mm	100
1.18 mm	99
600 um	97
425 um	87
300 um	51
150 um	7
75 um	4

PARTICLE SIZE DISTRIBUTION CURVE

Ref No: 21495SB1
Borehole Number: 101
Depth(m): 7.50-8.00
Table A:Page 2 of 4

NATA

Approved Signatory / Date
28/9/00



SIEVE ANALYSIS RESULTS

SIEVE SIZE	% PASSING
2.36 mm	100
1.18 mm	98
600 µm	97
425 µm	87
300 µm	50
150 µm	4
75 µm	1

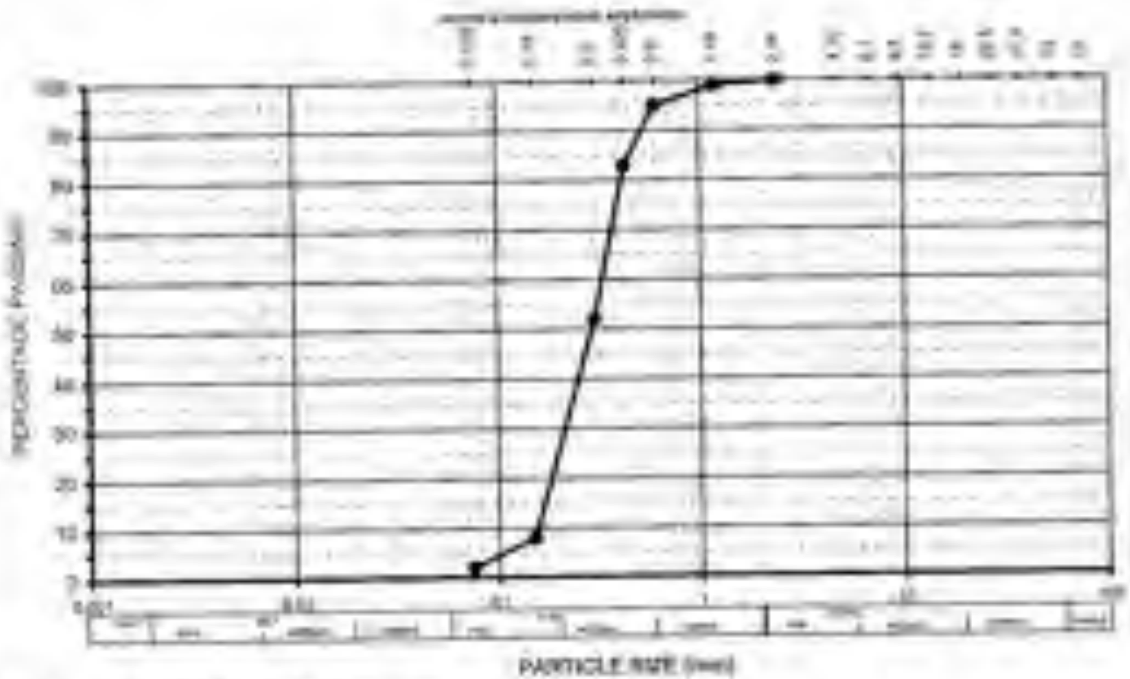
Test Method: AS1289 3.6.1 Dry Sieve/Washed

• Notes:

• Please refer to appropriate notes for soil descriptions

PARTICLE SIZE DISTRIBUTION CURVE

Ref No: 21496SB1
Borehole Number: 103
Depth(m): 7.50-8.00
Table A Page 3 of 4



SIEVE ANALYSIS RESULTS

SIEVE SIZE	% PASSING
2.36 mm	100
1.18 mm	99
600 um	95
425 um	83
300 um	52
150 um	8
75 um	2

Test Method: AS1289.3.6.1 Dry Sieve(Washed)

• Notes:

• Please refer to appropriate notes for soil descriptions

PARTICLE SIZE DISTRIBUTION CURVE

Ref No: 21498S01

Borehole Number: 103

Depth(m): 11.50-12.00

Table A Page 4 of 4

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 475
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

TABLE B
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{s(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	23.71-23.74	0.2	4
	23.95-24.00	0.4	8
	24.81-24.85	0.4	8
	26.48-26.52	0.3	6
	26.89-26.93	0.3	5
	27.26-27.30	0.2	4
	27.79-27.83	0.1	2
	28.59-28.64	1.2	24
	29.29-29.34	1.6	32
105	16.14-16.17	0.9	18
	16.68-16.71	0.6	12
	17.48-17.53	1.8	[illegible]
	18.09-18.14	1.3	26
	18.77-18.82	1.0	20
	19.32-19.37	0.5	10
	21.76-21.80	0.2	4
	22.11-22.16	0.5	10
	23.47-23.52	1.3	26

NOTES

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

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

TABLE A1
PARTICLE SIZE DISTRIBUTION CURVE

Client: JK Geotechnics Pty Ltd
Project: Proposed Redevelopment of Manly Civic Centre
Location: 2 West Promenade, Manly, NSW

Ref No: 21498582
Report No: A1
Report Date: 5/09/2012
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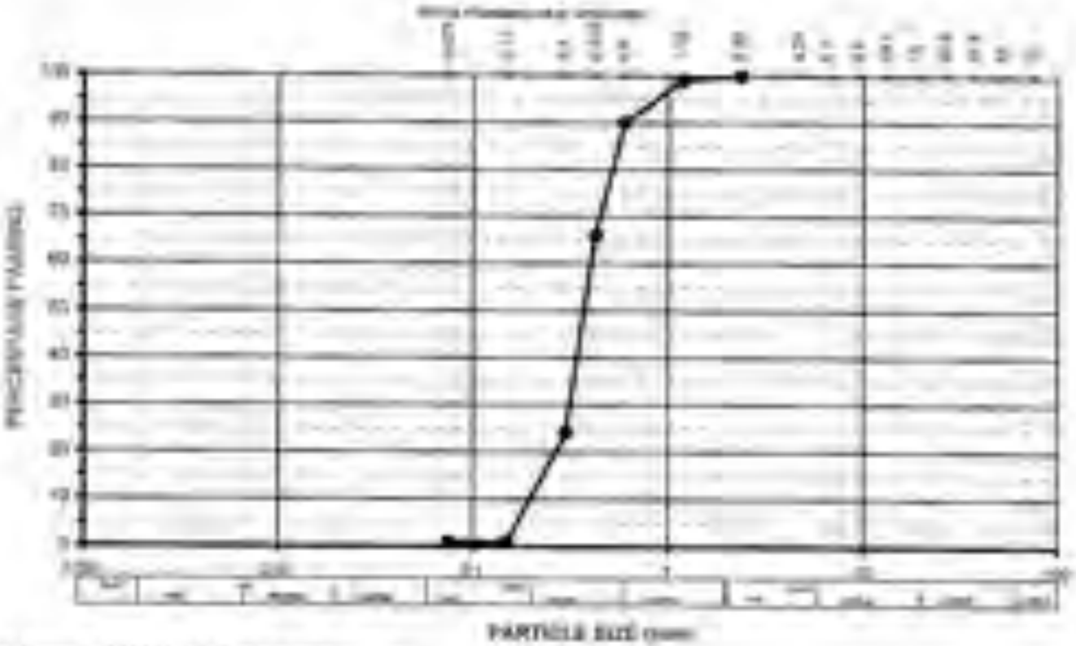


SIEVE ANALYSIS RESULTS

SIEVE SIZE	% PASSING
2.36 mm	100
1.18 mm	99
600 um	90
425 um	66
300 um	24
150 um	1
75 um	1

Borehole Number: 203
Depth (m): 5.3-5.5

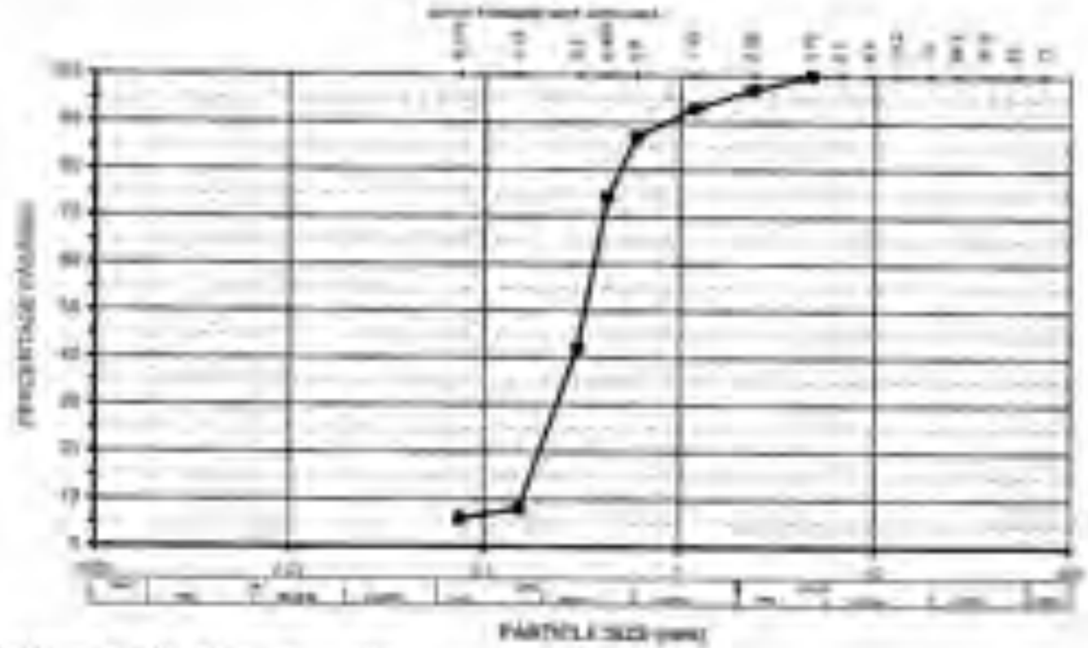
Test Method: AS1289.3.6.1 (Dry Sieve(washed))

NATA Accredited Laboratory

Approved Signatory / Date
5/9/12

TABLE A1
PARTICLE SIZE DISTRIBUTION CURVE

Client:	Jk Geotechnics Pty Ltd	**Ref No:**	21490SRC
Project:	Proposed Redevelopment of Manly Civic Centre	**Report No:**	A1
Location:	2 West Promenade, Manly, NSW	**Report Date:**	5/09/2012
		Page 2 of 4	



Test Method: AS1289.3.6.1 Dry Sieve(washed)

SIEVE ANALYSIS RESULTS

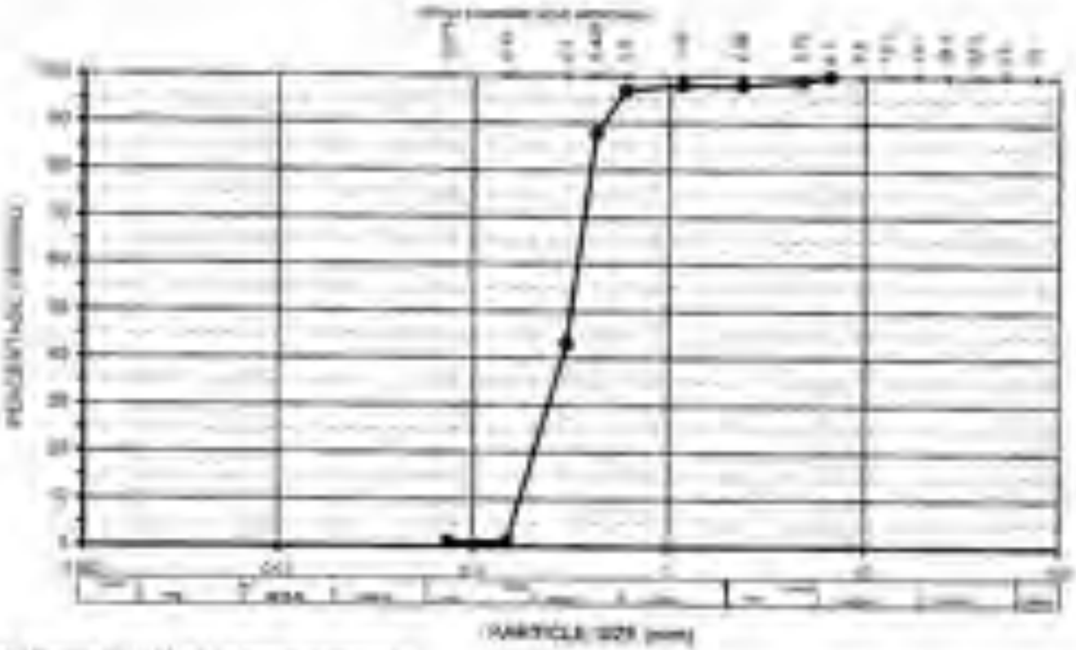
SIEVE SIZE	% PASSING
4.75 mm	100
2.36 mm	97
1.18 mm	93
600 µm	87
425 µm	74
300 µm	42
150 µm	8
75 µm	6

Borehole Number: 205
Depth (m): 6.5-7.0

5/9/12

TABLE A1
PARTICLE SIZE DISTRIBUTION CURVE

Client:	Jk Geotechnics Pty Ltd	**Ref No:**	21496SB2
Project:	Proposed Redevelopment of Manly Civic Centre	**Report No:**	A1
Location:	2 West Promenade, Manly, NSW	**Report Date:**	5/09/2012
			Page 3 of 4



SIEVE ANALYSIS RESULTS

SIEVE SIZE	% PASSING
6.70 mm	100
4.75 mm	99
2.36 mm	99
1.18 mm	98
600 µm	97
425 µm	88
300 µm	43
150 µm	1
75 µm	1

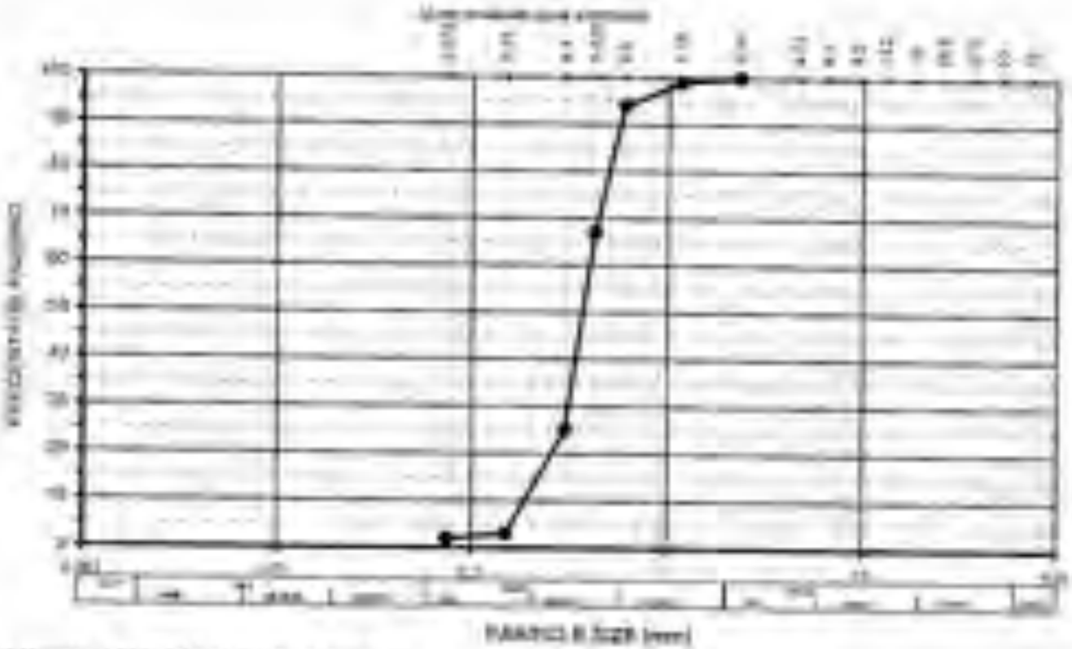
Borehole Number: 204
Depth (m): 2.0-2.8

Test Method: AS1289.3.6.1 Dry Sieve(2009)

TABLE A1
PARTICLE SIZE DISTRIBUTION CURVE

Client: Jk Geotechnics Pty Ltd
Project: Proposed Redevelopment of Manly Civic Centre
Location: 2 West Promenade, Manly, NSW

Ref No: 214[illegible]
Report No: A1
Report Date: 5/09/2012
Page 4 of 4



SIEVE ANALYSIS RESULTS

SIEVE SIZE	% PASSING
2.36 mm	100
1.18 mm	99
600 µm	93
425 µm	67
300 µm	25
150 µm	3
75 µm	2

Borehole Number: 204
Depth (m): 5.0-6.0

Test Method: AS1289.3.6.1 (Dry Sieve/washed)

NATA

5/9/12

[illegible]

SOIL TEST SERVICES

TABLE B1
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	[illegible]
Project:	Proposed Redevelopment	**Report:**	B1
Location:	2 West Promenade, Manly, NSW	**Report Date:**	[illegible]2012
		Page 1 of 1	

BOREHOLE NUMBER	DEPTH m	$I_{s(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
203	27.51-27.55	0.7	14
	28.00-28.04	1.2	24
	28.55-28.59	1.6	32
	29.00-29.04	1.4	28
	29.50-29.53	1.0	20
	30.00-30.03	1.2	24
	30.50-30.54	1.1	22
	31.05-31.09	1.6	32
	31.54-31.59	0.8	16
	32.02-32.06	1.1	23
	32.60-32.64	0.2	4
	33.00-33.04	0.6	12
	33.75-33.78	0.6	15
204	15.42-15.46	0.4	8
	15.64-15.68	0.1	2
	16.03-16.06	1.6	32
	16.53-16.58	2.3	46
	17.00-17.04	2.6	52
	17.51-17.55	2.3	46
	18.26-18.31	0.9	18

NOTES

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

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

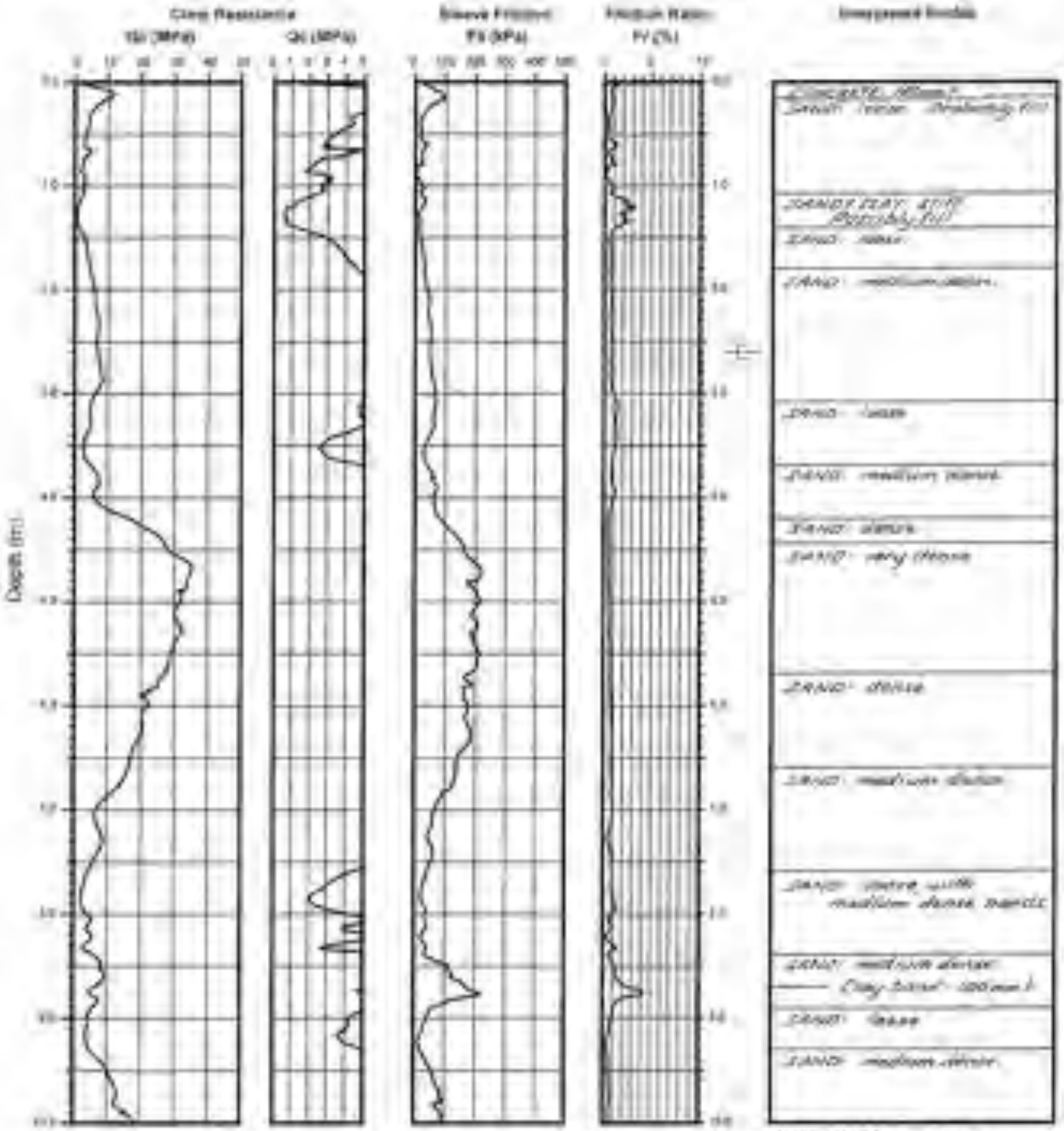
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
1
1/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	MANLY CIVIC CLUB LIMITED				
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB				
Location:	2 WEST PROMENADE, MANLY, NSW				
Job Ref.:	21498SB	**RL Surface:**	[illegible]	**Data File:**	21498SBcpt1.cal
Test Date:	31/08/2007	**Datum:**	[illegible]	**Operator:**	NES



Interpreted by: [illegible]
Checked by: [illegible]

Jeffery and Katauskas Pty Ltd

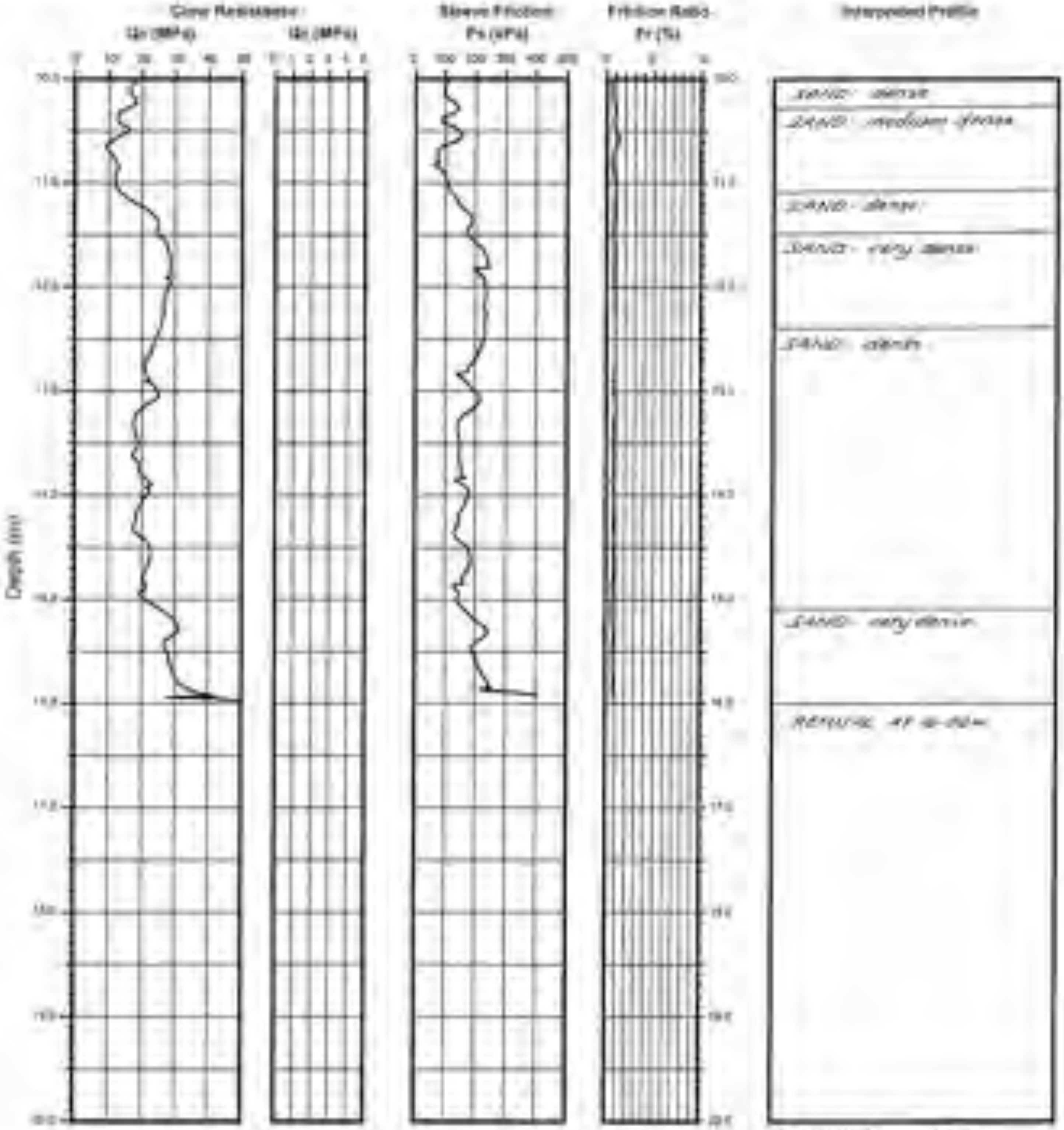
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No. 1
2/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job Ref.: 21488SB	**RL Surface:** ≈2.8m	**Data File:** 21488SBcpt1.cdt
Test Date: 11/08/2007	**Datum:** AHD	**Operator:** NES



Interpreted by: NES
Checked by: G.B.

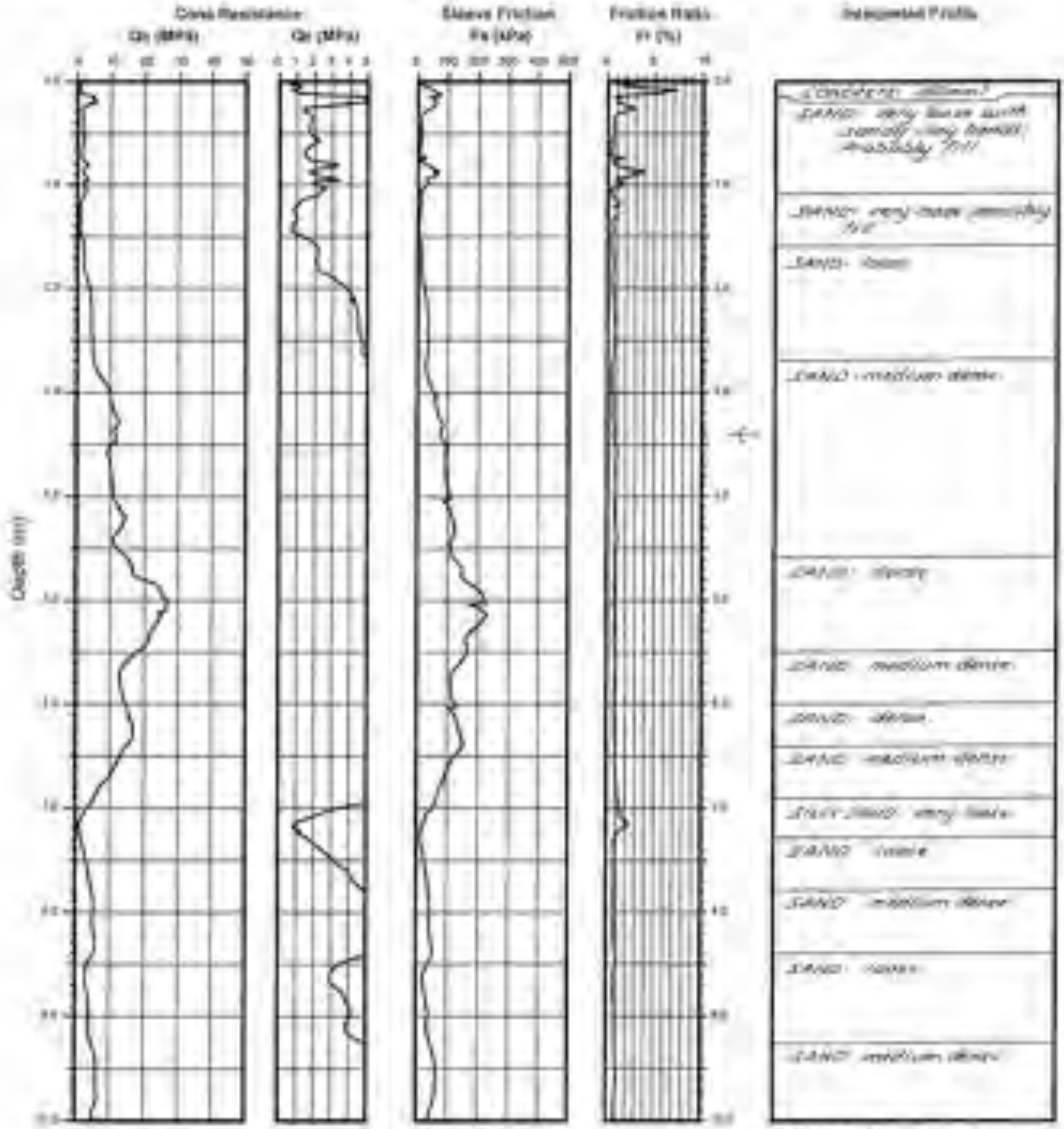
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
2
1/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	MANLY CIVIC CLUB LIMITED				
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB				
Location:	2 WEST PROMENADE, MANLY, NSW				
Job Ref.:	21496SE	**RL Surface:**	[illegible]	**Data File:**	21496SEcpt2.cpt
Test Date:	31/08/2007	**Datum:**	[illegible]	**Operator:**	NES



Interpreted by: N.E.S.
Checked by: [illegible]

Jeffery and Katauskas Pty Ltd

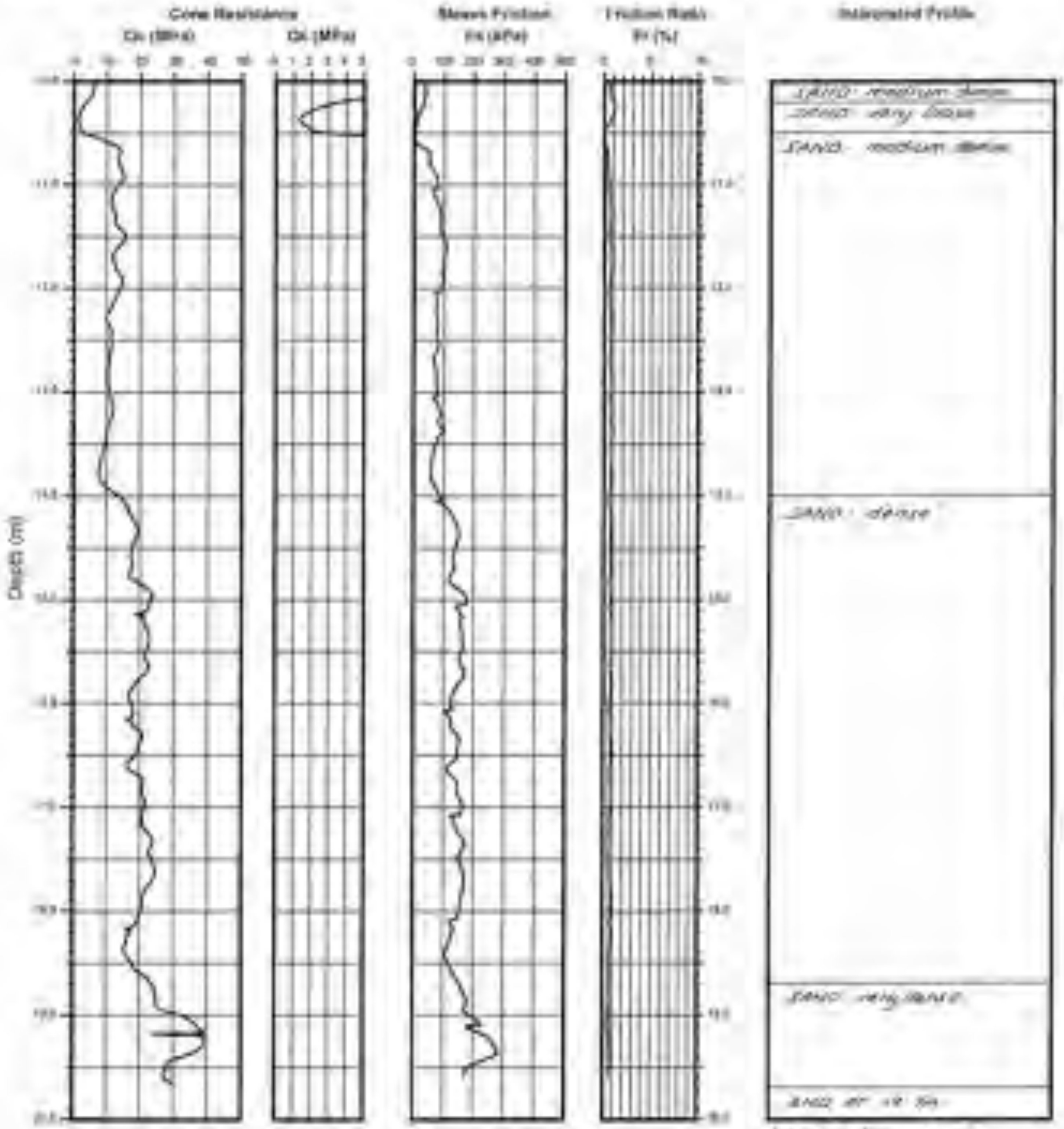
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
2
2/2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job Ref.:	21496SB	RL Surface:	[illegible]	Data File:	21496SBEFCP2 [illegible]
Test Date:	31/08/2007	Datum:	[illegible]	Operator:	NES



Interpreted by: NES
Checked by: GF

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
101
1 / 3

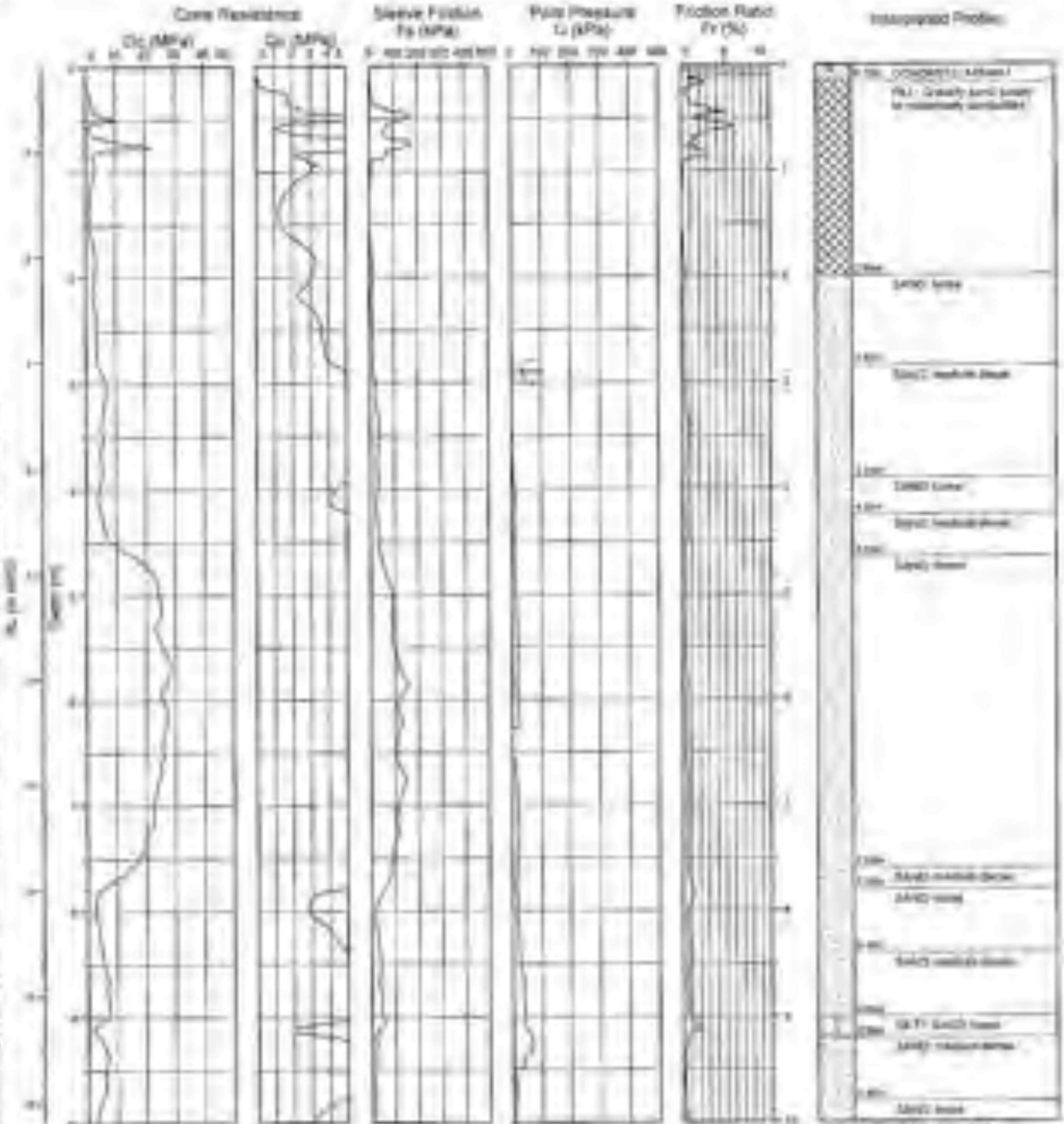
ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB1	**R.L. Surface:** ~3.8 m	**Data File:** 21496SB1_101.CEF
Date: 06/09/10	**Datum:** AHD	**Operator:** L.Y.



Interpreted by: D.B.
Checked by: N.E.S

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
101
2 / 3

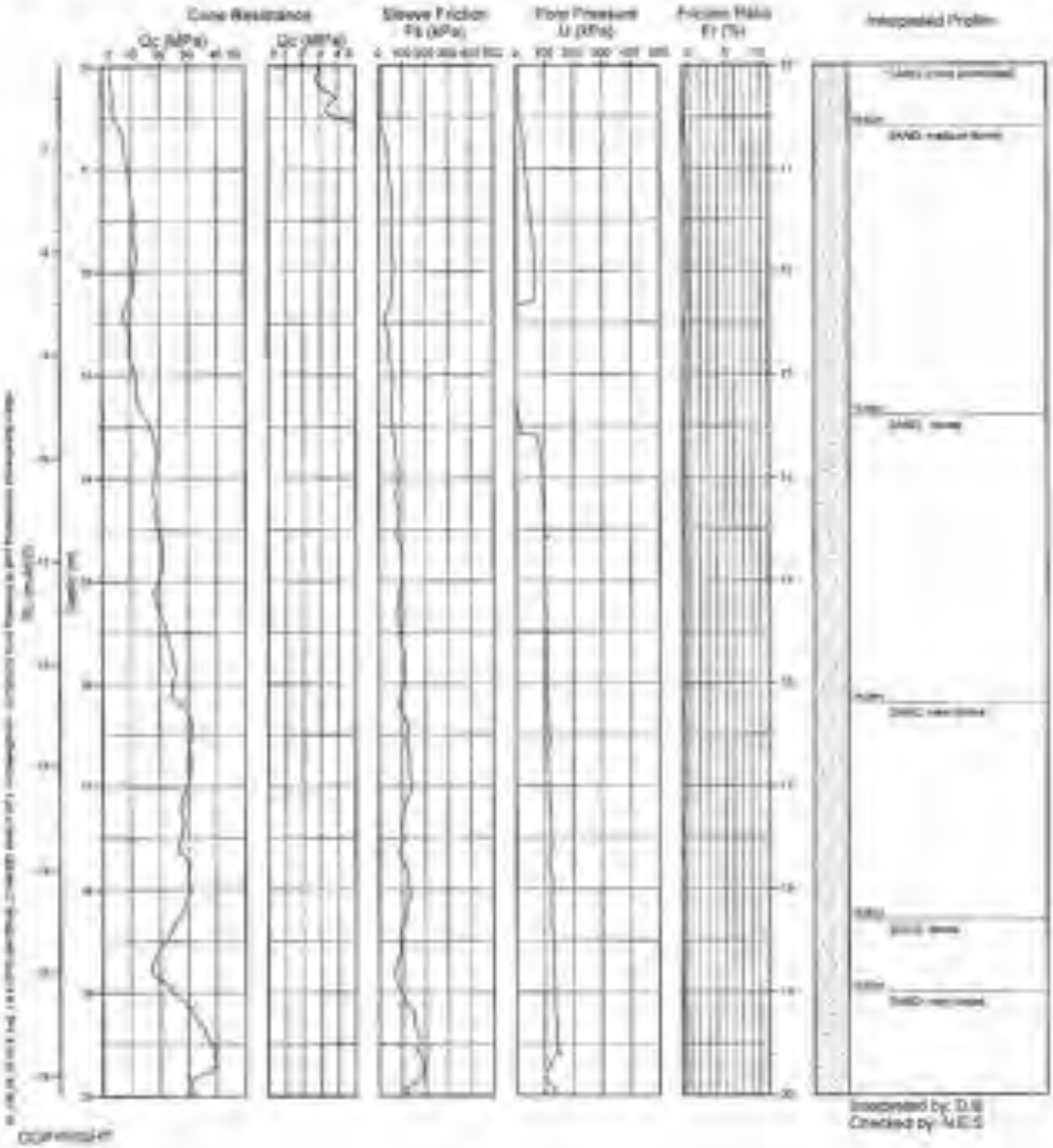
ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21498SB1	**R.L. Surface:** ~3.8 m	**Data File:** 21498SB1_101.GEF
Date: 08/03/10	**Datum:** AHD	**Operator:** L.Y



Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

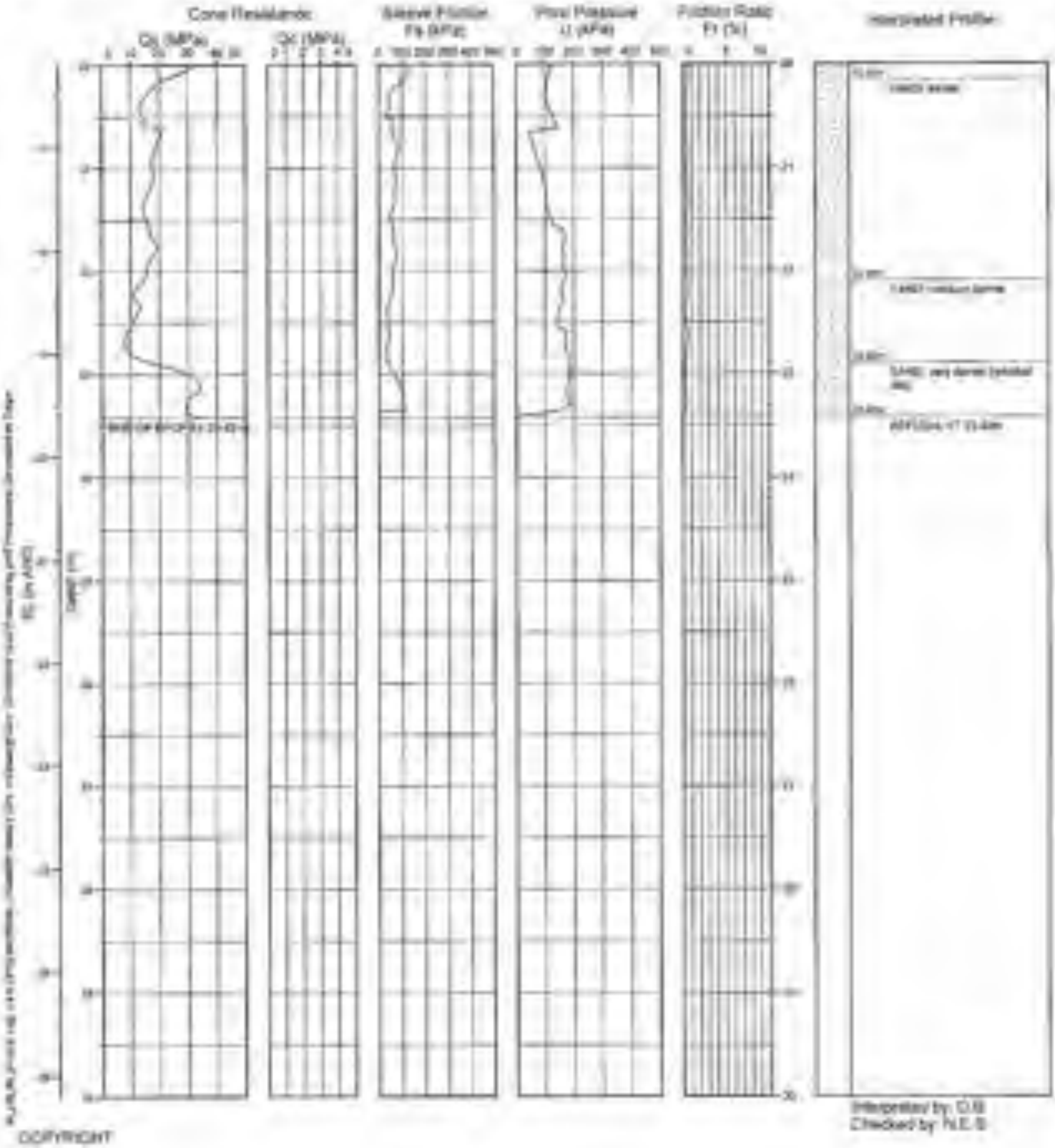
EFCP No. **101**

3 / 3

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	MANLY CIVIC CLUB LIMITED
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location:	2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB1	**R.L. Surface:** ~3.5 m	**Data File:** 21496SB1_101.GEF
Date: 06/09/10	**Datum:** AHD	**Operator:** L.Y.



Interpreted by: D.B.
Checked by: N.E.S.

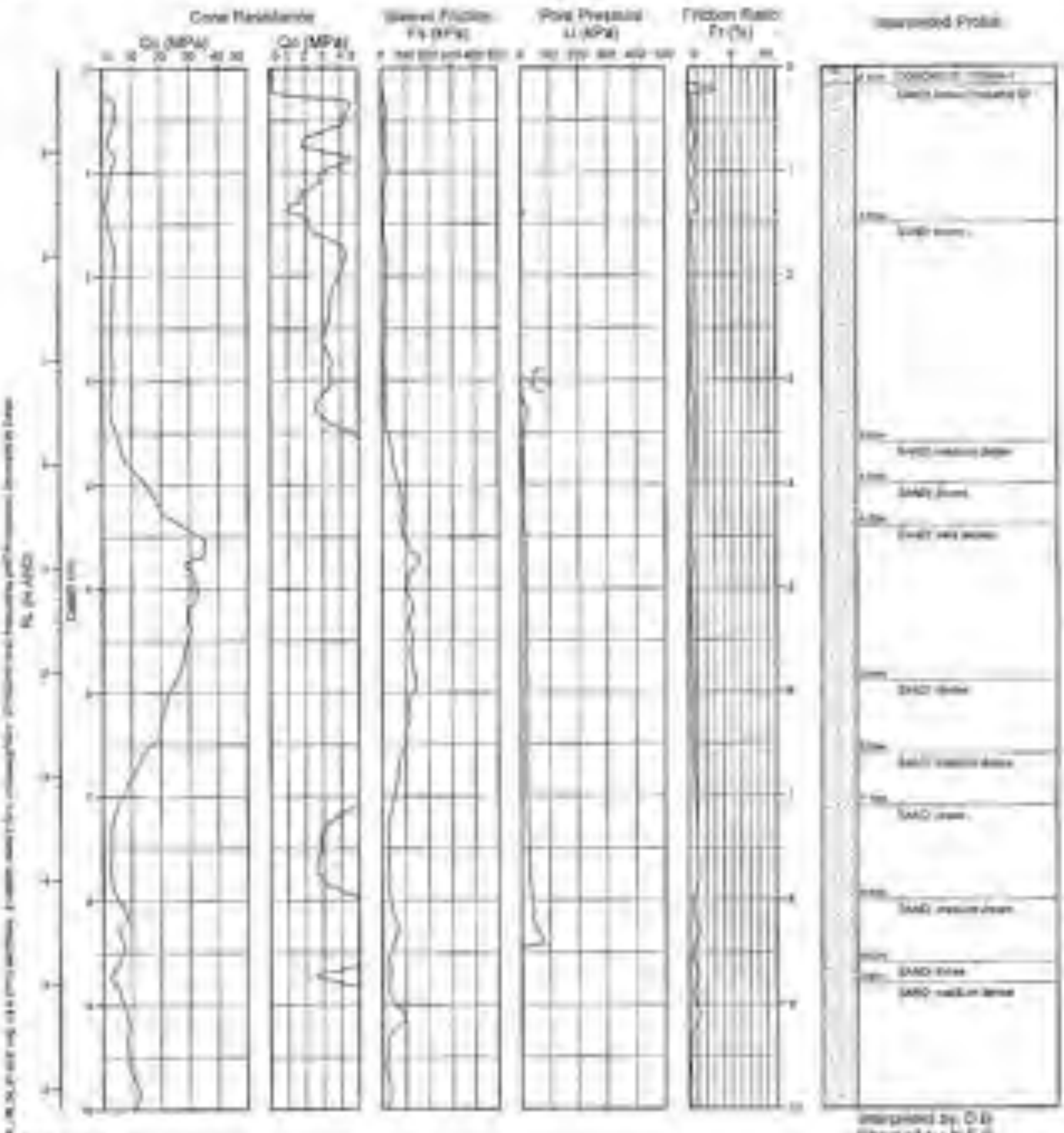
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
102
1 / 2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	MANLY CIVIC CLUB LIMITED		
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB		
Location:	2 WEST PROMENADE, MANLY, NSW		
Job No.: 21490SB1	**R.L. Surface:** ~3.5 m	**Data File:** 21490SB1_102.CEP	
Date: 06/09/10	**Datum:** AHD	**Operator:** L.Y.	



Interpreted by: D.B.
Checked by: N.E.S.

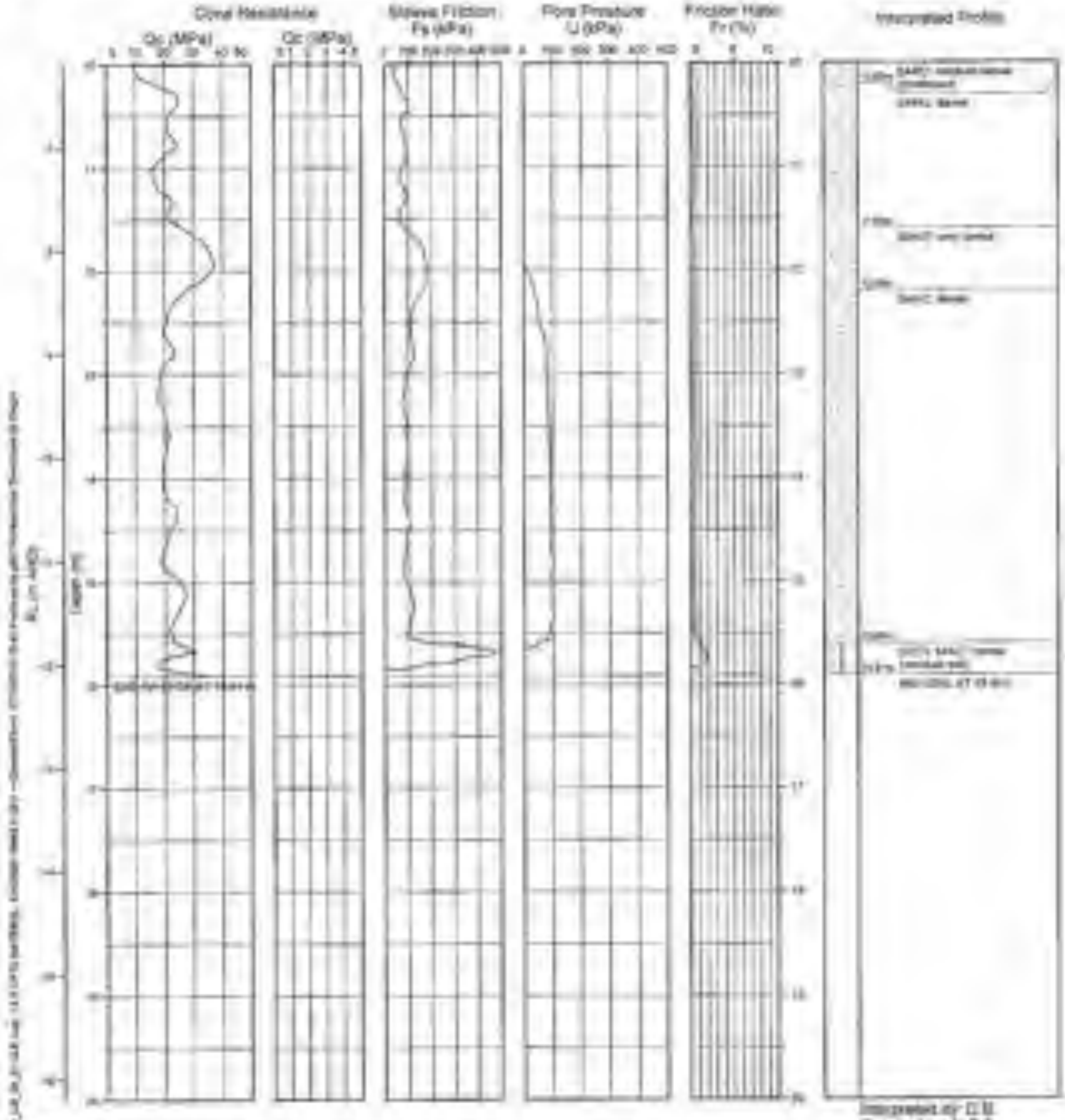
Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
102
2 / 2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	MANLY CIVIC CLUB LIMITED		
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB		
Location:	2 WEST PROMENADE, MANLY, NSW		
Job No.: 21496SB1	R.L. Surface: ~3.8 m	Data File: 21496SB1_102.GEF	
Date: 06/09/10	Datum: AHD	Operator: L.Y	



Interpreted by: D.B.
Checked by: N.E.S

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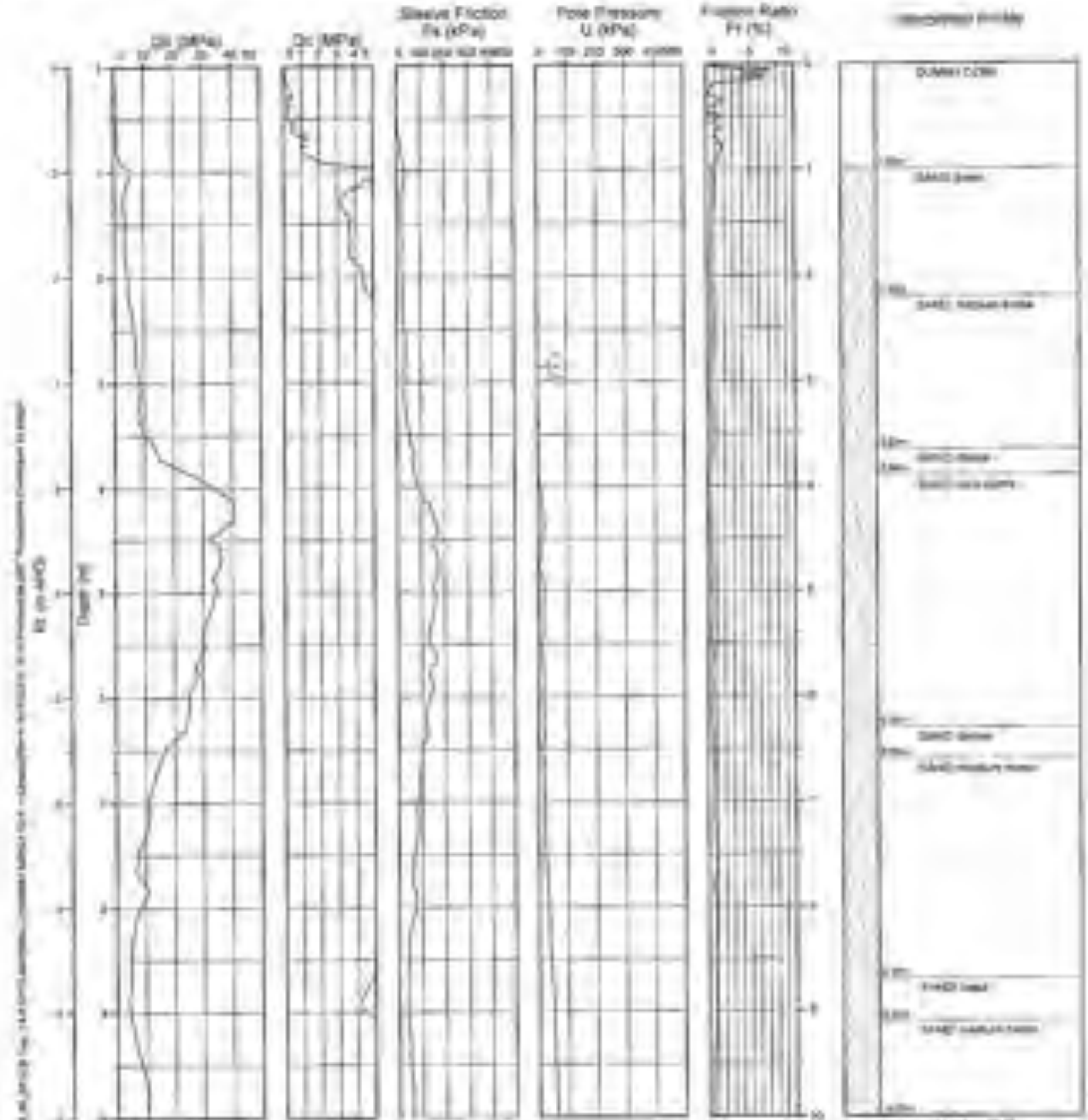
EFCP No.
104

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB1
Date: 01/11/10
R.L. Surface: ~4 m
Datum: AHD
Data File: 21496SB1_CPT104.GEF
Operator: M.T.



Interpreted by: D.B.
Checked by: N.E.S.

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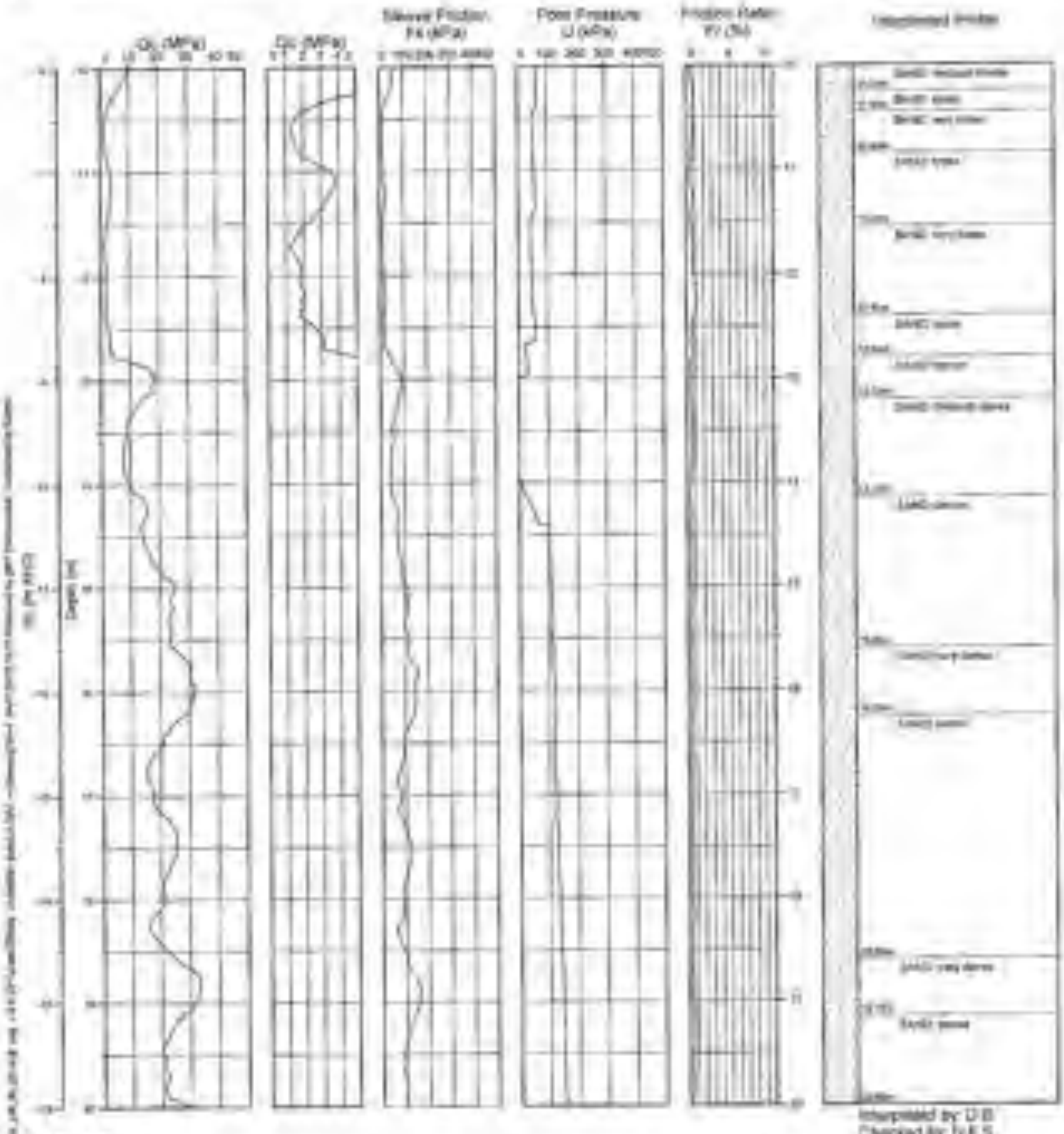
EFCP No. **104**

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21406SB1	**R.L. Surface:** ~4 m	**Data File:** 21406SB1_CPT104.GEF
Date: 01/11/10	**Datum:** AHD	**Operator:** M.T.



Interpreted by: D.B.
Checked by: N.E.S.

Jeffery and Katauskas Pty Ltd

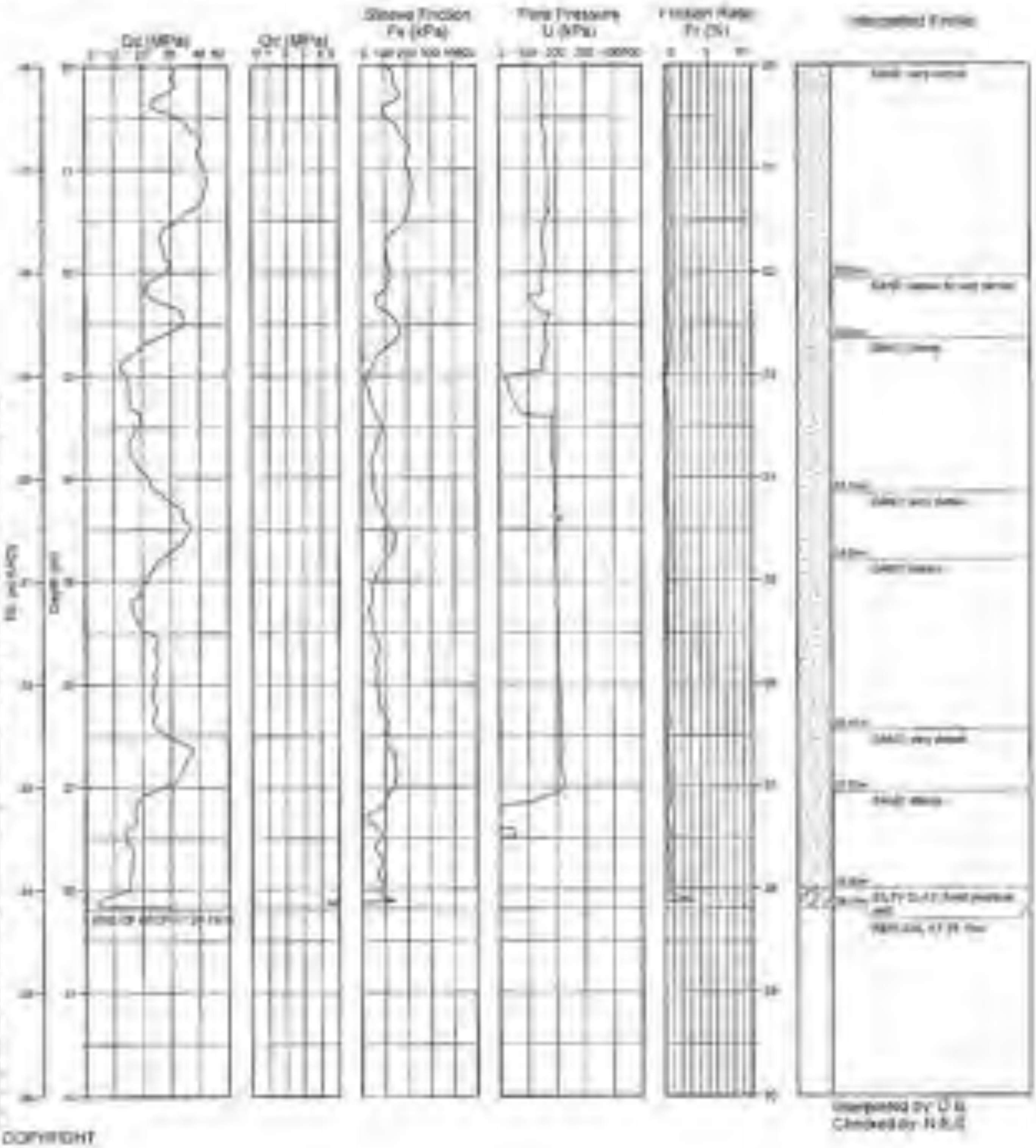
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No. **104**

3 / 3

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	MANLY CIVIC CLUB LIMITED				
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB				
Location:	2 WEST PROMENADE, MANLY, NSW				
Job No.:	21496SB1	**R.L. Surface:**	~4 m	**Data File:**	21496SB1_CPT104.GEF
Date:	01/11/10	**Datum:**	AHD	**Operator:**	M.T.



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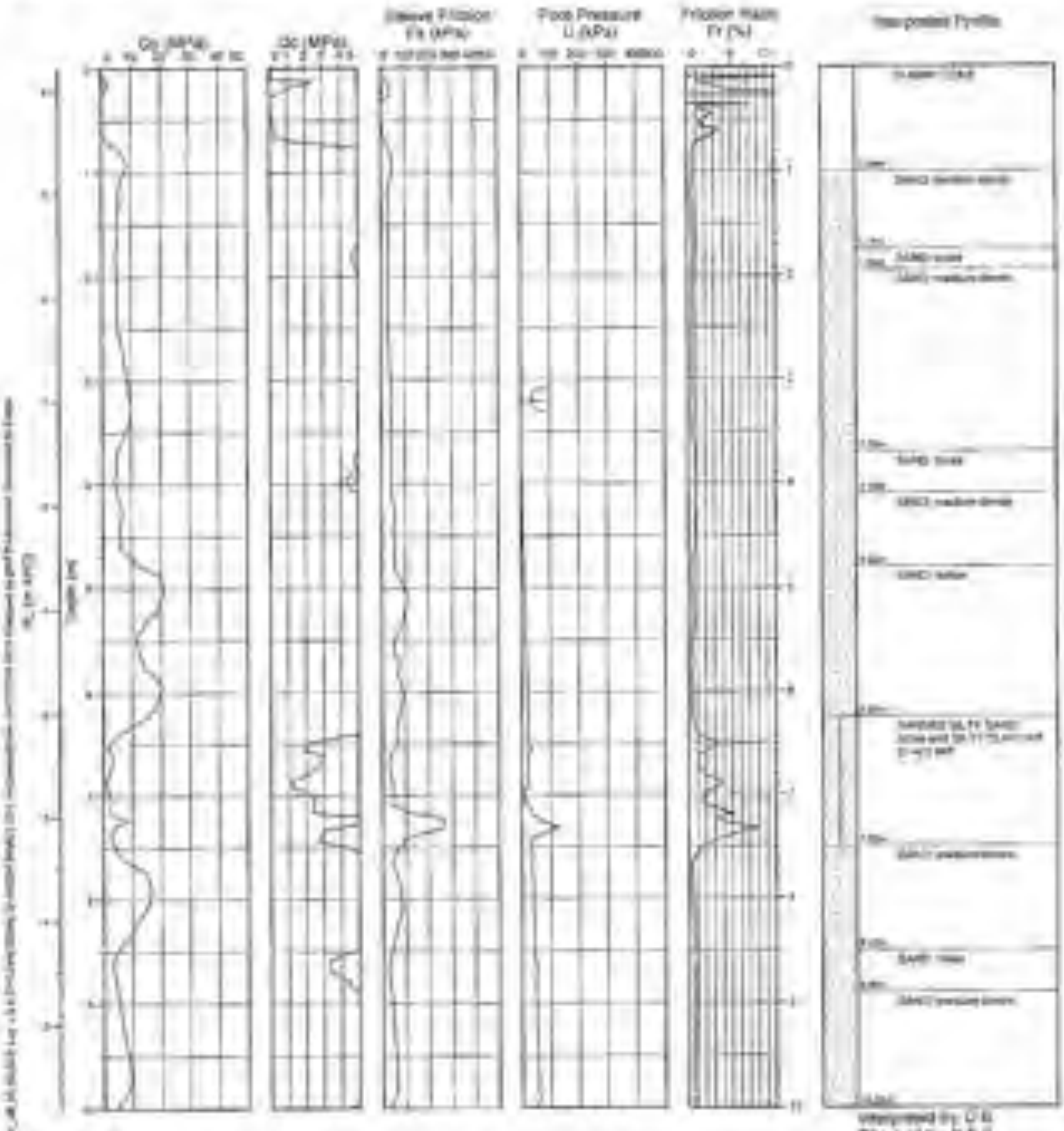
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No. **105**
1 / 4

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB1	**R.L. Surface:** ~4.2 m	**Data File:** 21496SB1_EFCP105.GEF
Date: 01/11/10	**Datum:** AHD	**Operator:** M.T.



Interpreted By: D.B.
Checked By: N.B.S.

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

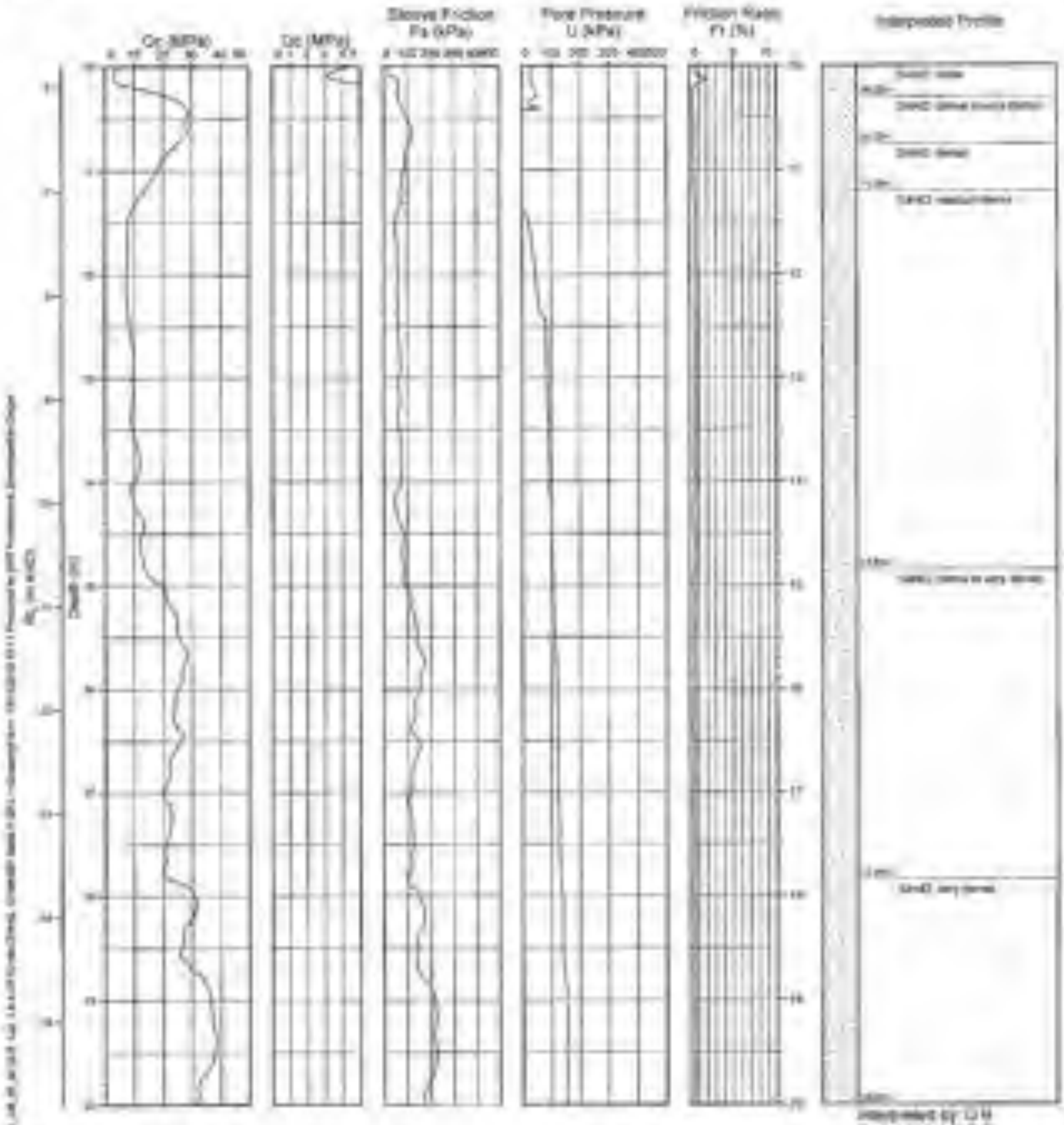
ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21486SB1	**R.L. Surface:** ~4.2 m	**Data File:** 21486SB1_EFCP105.GEF
Date: 01/11/10	**Datum:** AHD	**Operator:** M.T



Interpreted by: [illegible]
Checked by: [illegible]

Jeffery and Katauskas Pty Ltd

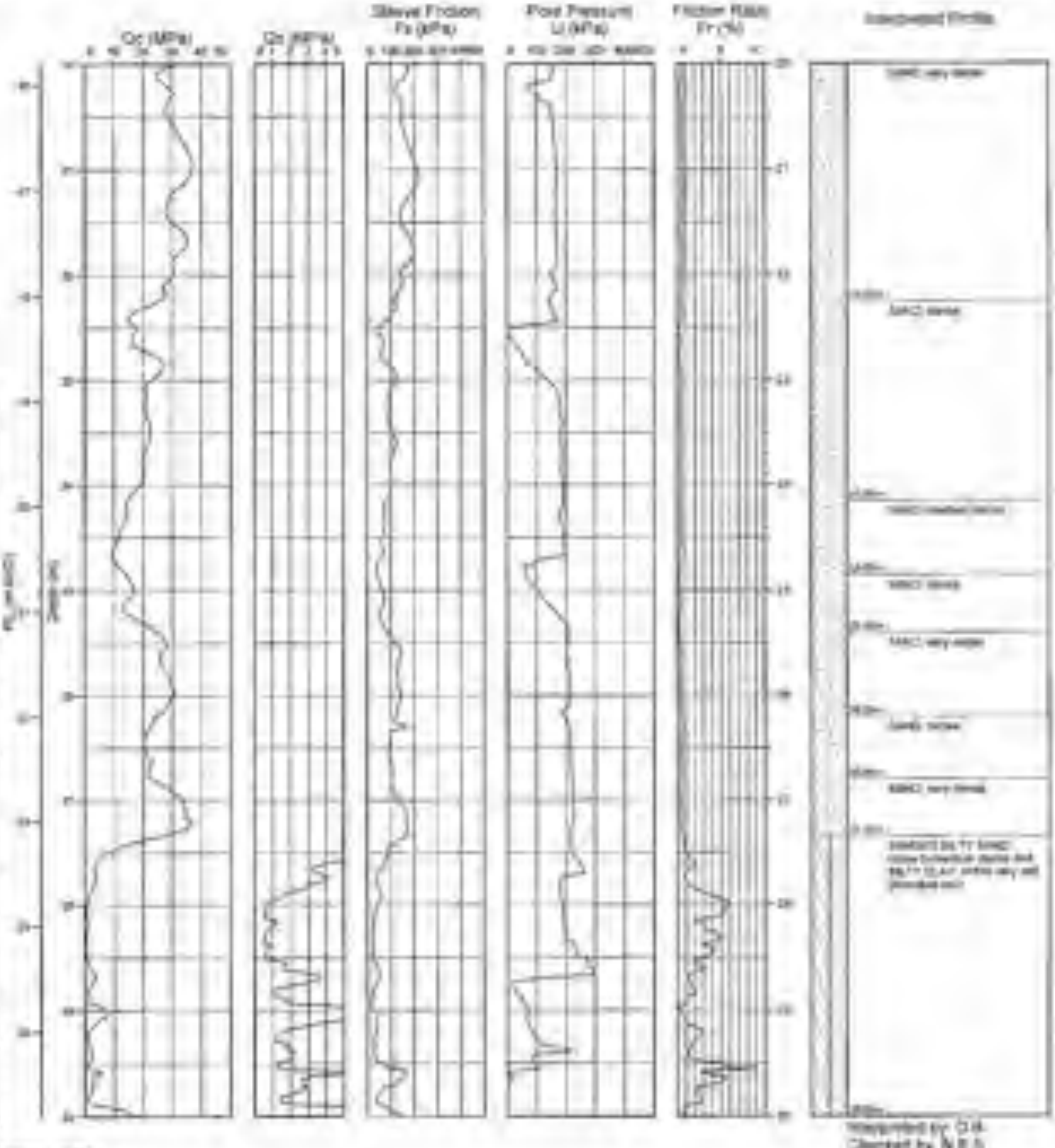
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No. **105**

3 / 4

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED			
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB			
Location: 2 WEST PROMENADE, MANLY, NSW			
Job No.: 21496SB1	**R.L. Surface:** ~4.2 m	**Data File:** 21496SB1_EFCP105.GEF	
Date: 01/11/10	**Datum:** AHD	**Operator:** M.T.	



Interpreted by: D.R.
Checked by: N.E.S.

Jeffery and Katauskas Pty Ltd

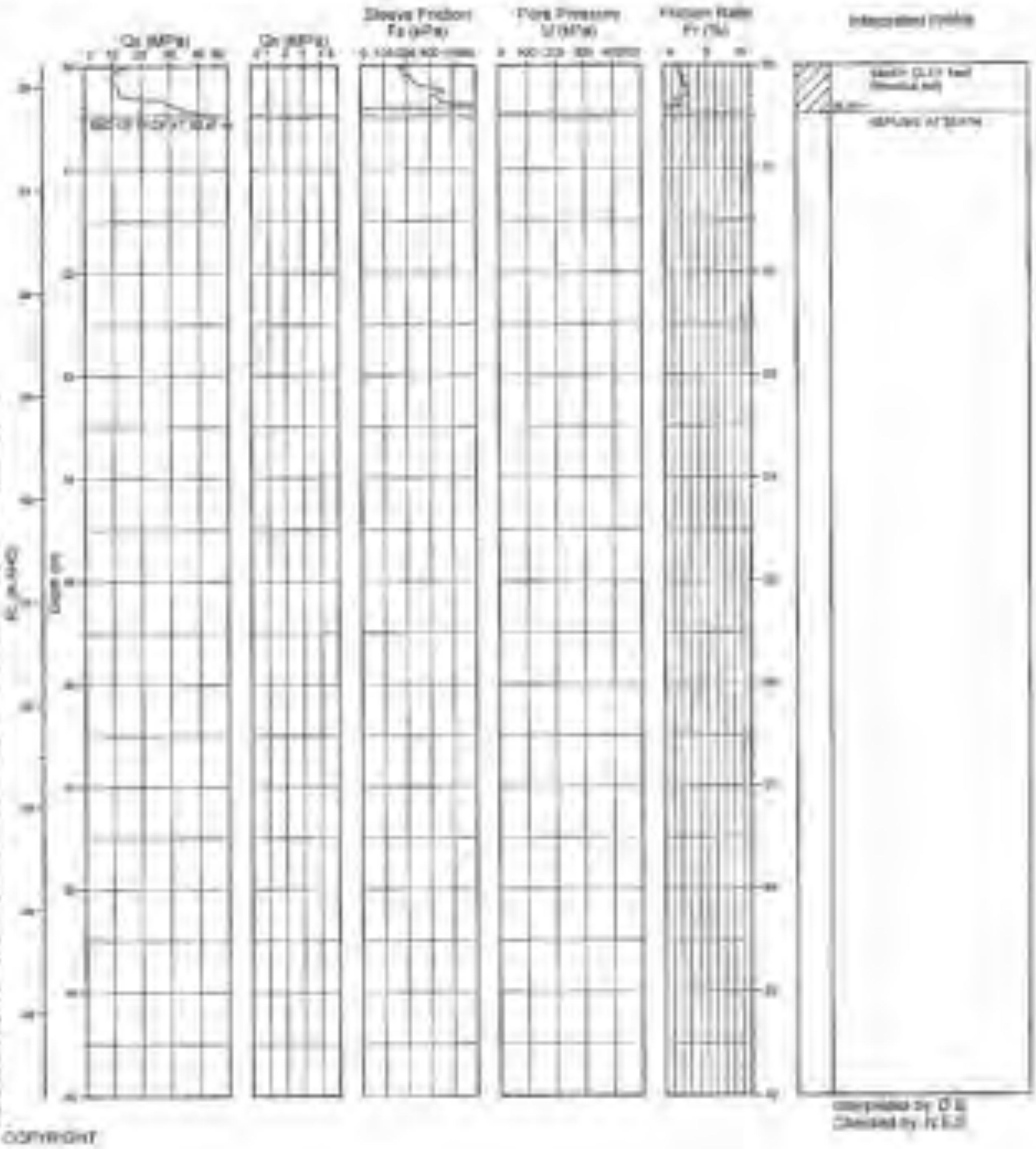
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

EFCP No.
105

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB1	R.L. Surface: ~4.2 m	Data File: 21496SB1_EFCP105.GEF
Date: 01/11/10	Datum: AHD	Operator: M.T.



Interpreted by: D.E.
Checked by: N.E.S.

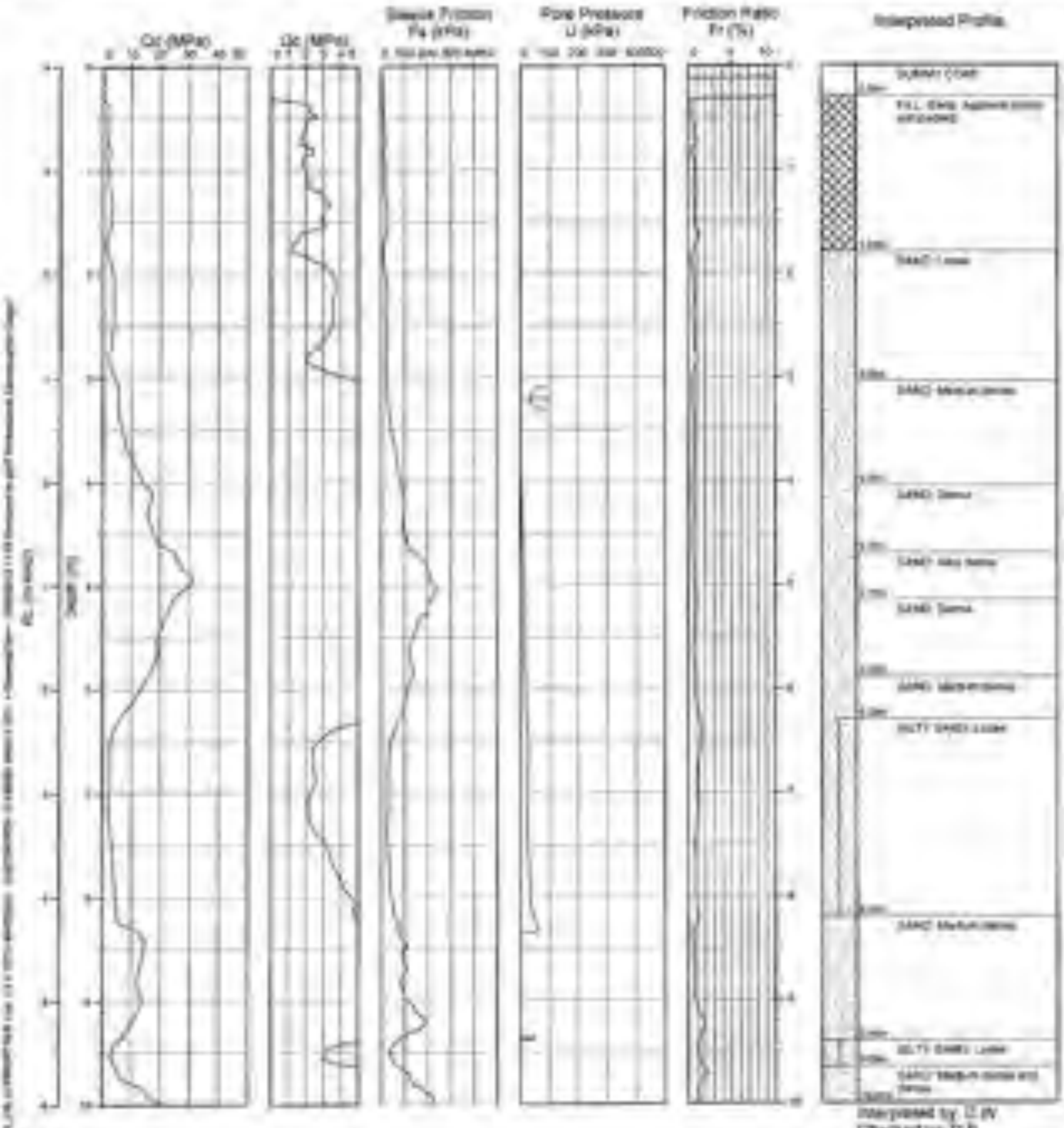
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No. 201

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	MANLY CIVIC CLUB LIMITED
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location:	2 WEST PROMENADE, MANLY, NSW

Job No.: 21498SB2	R.L. Surface: ~4 m	Data File: 21498SB2_201.GEF
Date: 21/8/12	Datum: AHD	Operator: D.W



Interpreted by: D.W
Checked by: D.B

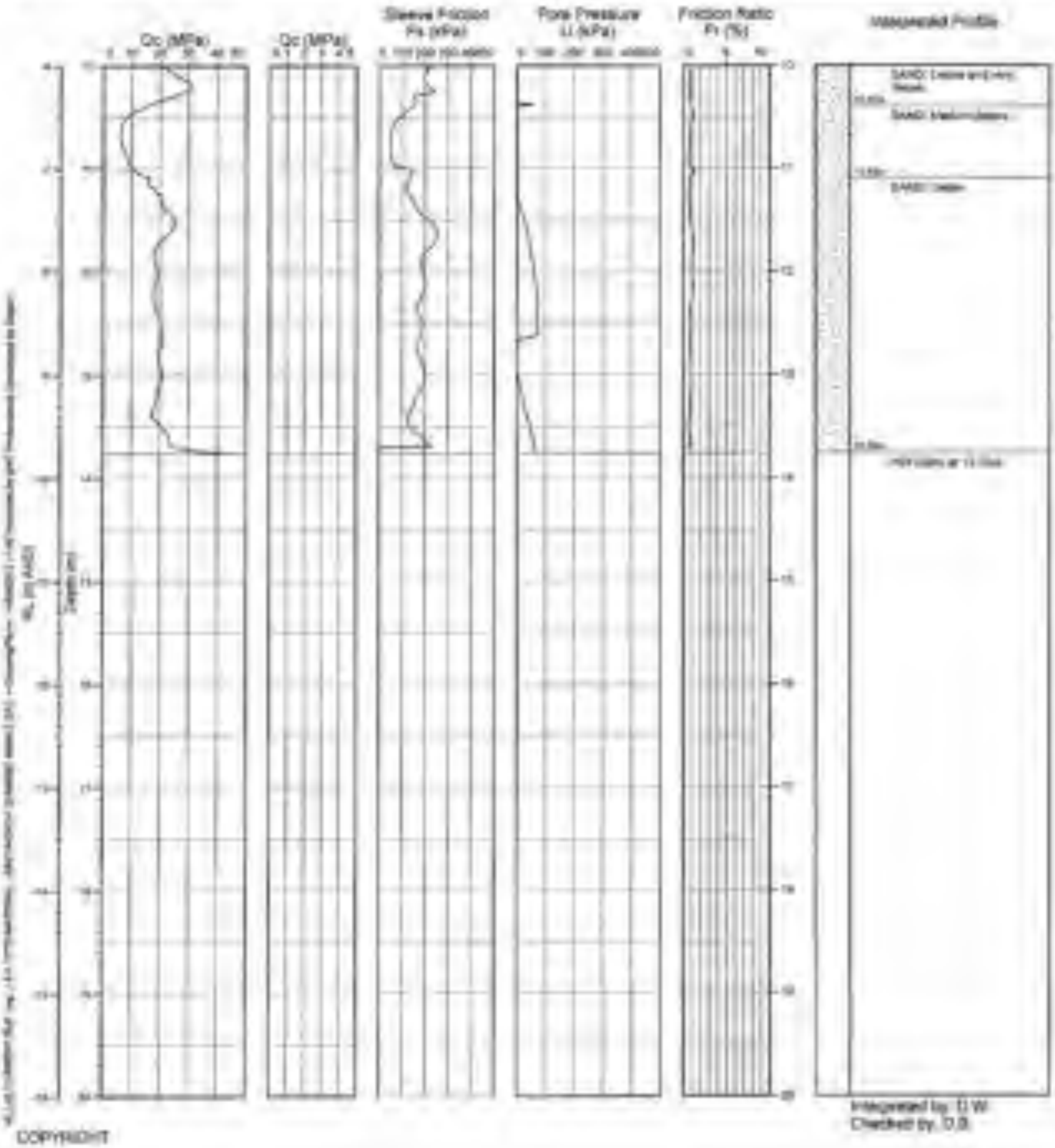
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
201
2 / 2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	MANLY CIVIC CLUB LIMITED
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location:	2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB2	**R.L. Surface:** ~4 m	**Data File:** 21496SB2_201.GEF
Date: 21/8/12	**Datum:** AHD	**Operator:** D.W.



Interpreted by: D.W.
Checked by: D.B.

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
202
1 / 3

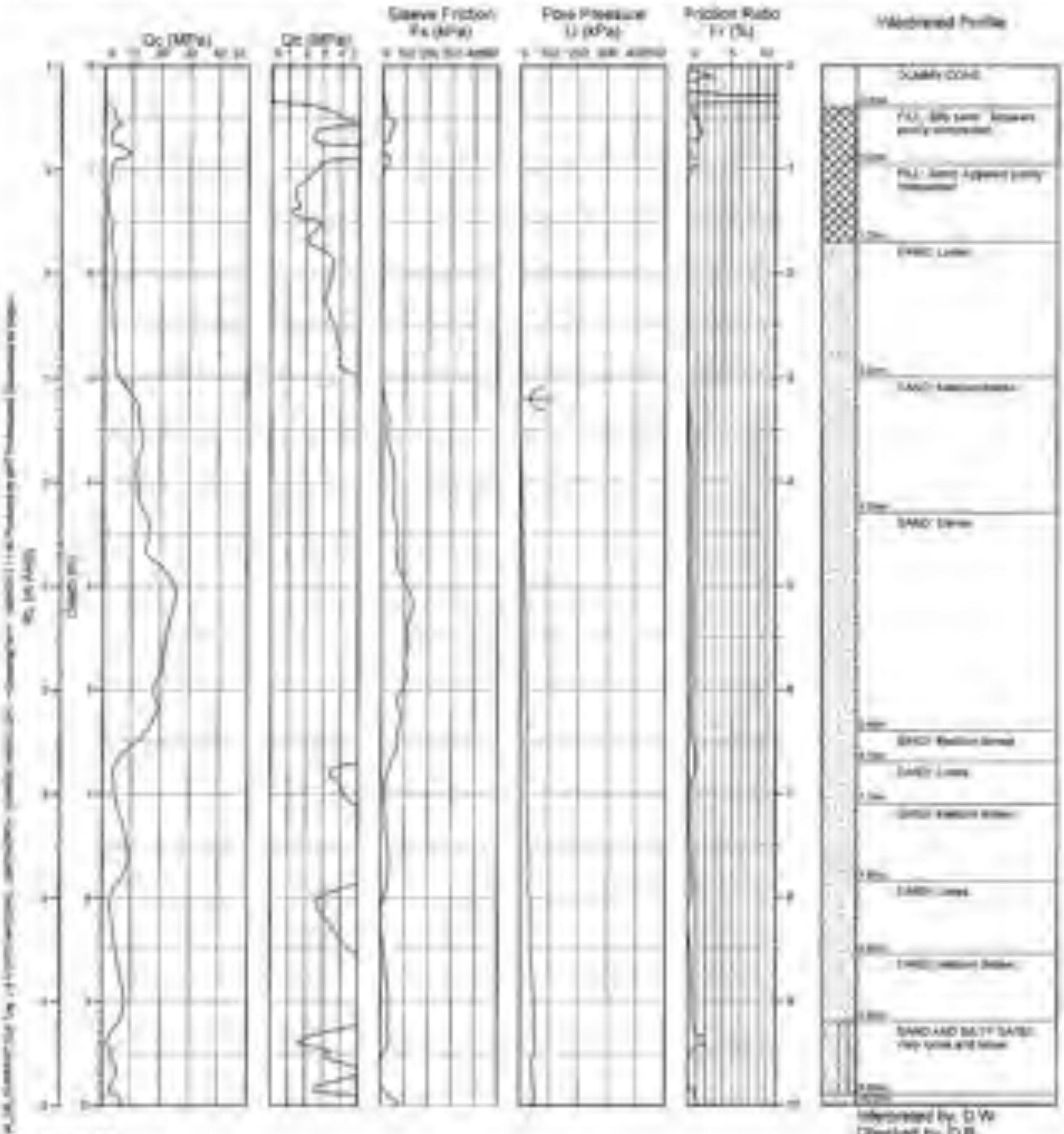
ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB2	**R.L. Surface:** ~4 m	**Data File:** 21496SB2_202.GEF
Date: 21/8/12	**Datum:** AHD	**Operator:** D.W.



Interpreted by: D.W.
Checked by: D.B.

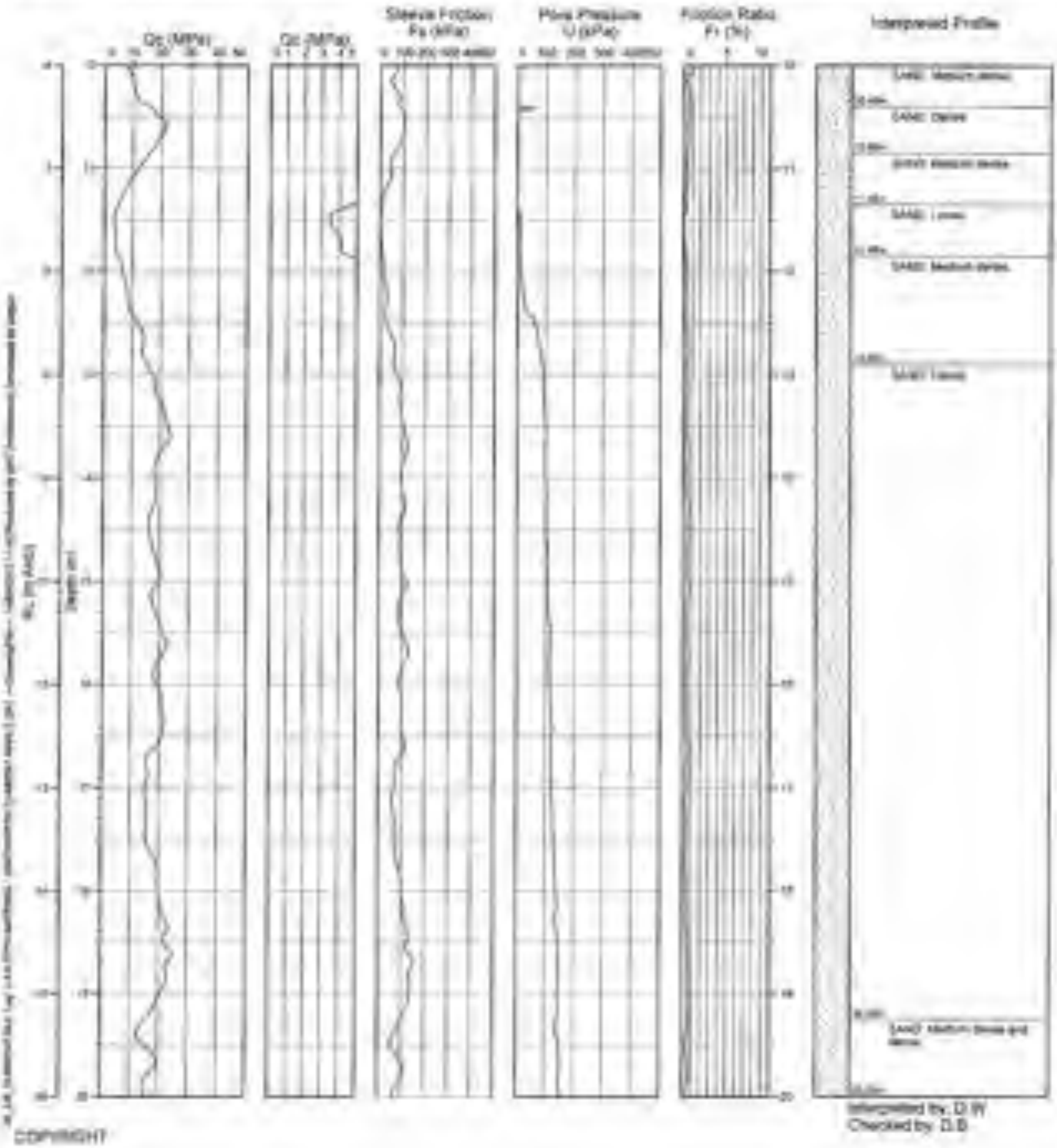
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
202
2 / 3

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB2	**R.L. Surface:** ~4 m	**Data File:** 21496SB2_202.GEF
Date: 21/6/12	**Datum:** AHD	**Operator:** D.W



Interpreted by: D.W
Checked by: D.B

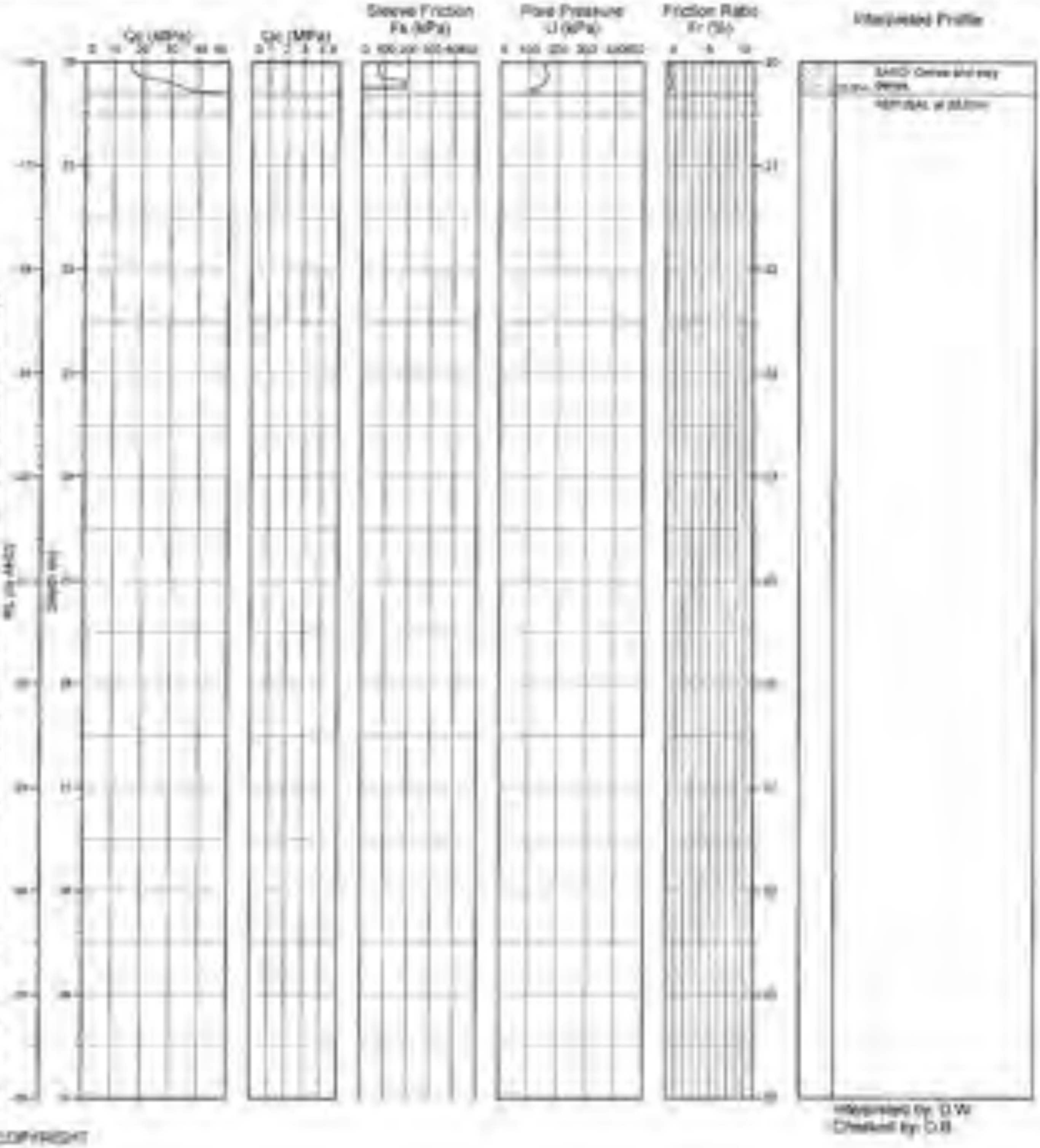
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

EFCP No.
202
3 / 3

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No.: 21496SB2	**R.L. Surface:** ~4 m	**Data File:** 21496SB2_202.GEF
Date: 21/8/12	**Datum:** AHD	**Operator:** D.W



Interpreted by: D.W
Checked by: D.B

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **3**
1/2

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21486SB
Date: 31-8-07
Method: SPIRAL AUGER JK300
Logged/Checked by: N.E.S./jk
R.L. Surface: ≈ 3.8m
Datum: AHD

Groundwater Record	Samples	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
			0			CONCRETE: 110mm.t				8mm DIAMETER REINFORCEMENT, 45mm TOP COVER
		N = 7 3,3,4				FILL: Gravelly sand, fine to coarse grained, dark grey, fine to medium grained igneous gravel.	M			APPEARS MODERATELY COMPACTED
						FILL: Clayey sand, fine to medium grained, orange grey brown, with fine to medium grained sandstone gravel.				
			1		SP	SAND: fine to medium grained, grey.	M	L	-	
		N = 5 2,3,2				as above, but light grey.				
			2							
AFTER 15 MINS		N = 7 2,2,5	3			SAND: fine to medium grained, grey brown, with a trace of silt fines.				
							W			SLIGHT ORGANIC ODOUR
			4							
		N > 30 17,19 15/100mm END						D		
			5							
						SAND: fine to coarse grained, grey, with a trace of fine grained quartz gravel.				
		N = 39 7,17,22	6							
			7							

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BOREHOLE LOG

Borehole No. **3**

2/2

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB

Date: 31-8-07

Method: SPIRAL AUGER JK300

R.L. Surface: ≈ 3.9m

Datum: AHD

Logged/Checked by: N.E.S./

Groundwater Record	Samples	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
		N = 10 2,4,6	8		SM	SILTY SAND: fine to medium grained, dark grey brown	W	L-MD		ORGANIC ODOUR NO SAMPLE RETURN IN SPT SPLIT SPOON SAMPLER
			9			END OF BOREHOLE AT 9.0m				PVC STANDPIPE PIEZOMETER INSTALLED TO 9m. 6m SCREEN, 3m CASING. CAST IRON GATIC COVER AT SURFACE
			10							
			11							
			12							
			13							
			14							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
101
1/5

BOREHOLE LOG

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21686SB1
Date: 9-9-10
Method: SPIRAL AUGER JK500
Logged/Checked by: J.C./
R.L. Surface: ≈ 3.8m
Datum: AHD

Groundwater Record	Samples	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
			0			CONCRETE: 140mm t				5&10mm DIA. REINFORCEMENT 35mm & 130mm TOP COVER
						VOID: 100mm t	-	-	-	
						CONCRETE: 170mm t	-	-	-	NO OBSERVED REINFORCEMENT
			1		-	FILL: Gravelly sand, fine to medium grained, light grey brown, fine to medium grained igneous gravel, with a trace of sandstone cobbles and brick.	M	-	-	SAMPLES RECOVERED FROM AUGERS
			2		SP	SAND: fine to medium grained, light grey brown.	M			REFER TO DCP101 TEST RESULTS FOR RELATIVE DENSITIES
▶										
			3			as above, but dark grey brown.	W			
			4							
			5							
			6							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
101
2/8

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB1
Date: 9-9-10
Method: SPIRAL AUGER JK500
R.L. Surface: ≈ 3.8m
Datum: AHD
Logged/Checked by: J.C./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
								SP	SAND: fine to medium grained, dark grey brown.	W			
						8							
						9							
						10							
						11							
						12							
						13							
						14							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
101
3/5

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB1
Date: 9-8-10
Method: SPIRAL AUGER JK600
Logged/Checked by: J.C./
R.L. Surface: ≈ 3.0m
Datum: AHD

Groundwater Record	SAMPLES	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
					SP	SAND: fine to medium grained, dark grey brown.	W			
			15							
			16							
			17							
			18							
			19			as above, but light grey.				
			20							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **101**

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB1 **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 3.8m

Date: 9-9-10 **Datum:** AHD

Logged/Checked by: J.C./

Groundwater Record	Samples	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/Weathering	Strength/Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
					SP	SAND: fine to medium grained, light grey.				
			22							
					SC	CLAYEY SAND: fine to medium grained, light grey and orange brown.				RESIDUAL
			23							POSSIBLY XW SANDSTONE
					-	SANDSTONE: fine to coarse grained, light grey and orange brown.	DRY	[illegible]		
						REFER TO CORED BOREHOLE LOG				
			24							
			25							
			26							
			27							


JEFFERY & KATAUSKAS PTY LTD
JOB No. 21496SB1 BH101 START CORING AT 23.63m
23
CORE LOSS 0.09m
24
CORE LOSS 0.67m
25
CORE LOSS 1.18m
26
27
28
29
END OF BOREHOLE AT 29.34m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **101**
5/5

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB1 | **Core Size:** NMLC | **R.L. Surface:** ≈ 3.8m
Date: 9-9-10 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** J.C./A.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION (Rock Type, grain characteristics, colour, structure, minor components)	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION (Type, inclination, thickness, planarity, roughness, coating) Specific / General
		23		START CORING AT 23.83m					
				SANDSTONE: fine to coarse grained, light grey.	SW				
					DW	M			
		24		CORE LOSS 0.08m					
				SANDSTONE: fine to coarse grained, orange brown.					
				CORE LOSS 0.67m					
				SANDSTONE: fine to medium grained, light grey.	SW	M			[illegible]
		25		CORE LOSS 1.3m					
		26							
FULL RET URN				SANDSTONE: fine to medium grained, light grey, with occasional red brown.	SW	L-M			[illegible]
		27		SANDSTONE: fine to medium grained, orange brown and light grey.	DW-SW				
		28							
				SANDSTONE: fine to coarse grained, red brown, orange brown and light grey.	FR	M			[illegible]
		29							[illegible]
				END OF BOREHOLE AT 29.14m					

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
103
1/5

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB1
Date: 8-9-10
Method: SPIRAL AUGER JK600
Logged/Checked by: J.C./
R.L. Surface: ≈ 3.8m
Datum: AHD

Groundwater Record	SAMPLES	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
			0			CONCRETE: 115mm.t				10mm DIA. REINFORCEMENT, 45 & 55mm TOP COVER
						FILL: Gravelly sand, fine to medium grained, dark grey, fine to medium grained igneous gravel, slag and a trace of silt.	M			SAMPLES RECOVERED FROM AUGERS
			1		SP	SAND: fine to medium grained, light grey brown.	M			REFER TO DCP TEST RESULTS FOR RELATIVE DENSITIES
			2							
			3							
			4			as above, but dark grey brown.	W			
			5							
			6							
			7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **103**

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21498SB1

Date: 8-9-10

Method: SPIRAL AUGER JK500

R.L. Surface: ≈ 3.0m

Datum: AHD

Logged/Checked by: J.C./

Groundwater Record	SAMPLES	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					SP	SAND: fine to medium grained, dark grey brown.	W			
			8							
			9							
			10							
			11							
			12							
			13							
			14							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **103**

3/5

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB1

Date: 8-9-10

Method: SPIRAL AUGER JK500

R.L. Surface: ≈ 3.8m

Datum: AHD

Logged/Checked by: J.C./

Groundwater Record	Samples	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
					SP	SAND: fine to medium grained, dark grey brown.	W			
			15							
			16			REFER TO CORED BOREHOLE LOG				
			17							
			18							
			19							
			20							


JEFFERY & KATAUSKAS PTY LTD
JOB No. 21496SBI BH103 START CORING AT 16.0m
16
17
18
19
20
CORE LOSS 1.4m
21
22
23
END OF BH AT 23.62m

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **103**

4/5

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21498SBT	Core Size: NMLC	R.L. Surface: ≈ 3.8m
Date: 8-9-10	Inclination: VERTICAL	Datum: AHD
Drill Type: JK500	Bearing: -	Logged/Checked by: J.C./

Water Loss/Level	Barrel Lift	Depth (m)	CORE DESCRIPTION (Rock Type, grain characteristics, colour, structure, minor components)	Weathering	Strength	DEFECT DETAILS – DESCRIPTION (Type, inclination, thickness, planarity, roughness, coating)
		15				
			START CORING AT 16.0m			
		16	SANDSTONE: fine to coarse grained, light grey and orange brown.	XW	M	
		17		VW	EL	
			SANDSTONE: medium to coarse grained, dark grey, light grey and orange brown.	DW	M	[illegible]
		18				[illegible]
						[illegible]
						[illegible]
						[illegible]
90% RET-URN		19	SANDSTONE: fine to medium grained, light grey and orange brown.	SW	M	[illegible]
						[illegible]
			SANDSTONE: fine to medium grained, dark orange brown and light grey, with clay bands.	XW	EL	
		20	CORE LOSS 1.4m			
		21				
			SANDSTONE: fine to medium grained, light grey and orange brown.	DW-SW	M	[illegible]

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **103**
5/5

Client:	MANLY CIVIC CLUB LIMITED
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location:	2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB1	**Core Size:** NMLC	**R.L. Surface:** ≈ 3.8m
Date: 6-9-10	**Inclination:** VERTICAL	**Datum:** AHD
Drill Type: JK500	**Bearing:** -	**Logged/Checked by:** J.C.I.[illegible]

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain character, colour, structure, minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS – DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating (Specific / General)
90% RETURN		23		SANDSTONE: fine to medium grained, light grey and orange brown.	DW-SW	M			[illegible] [illegible] [illegible] [illegible] CLAY COATING [illegible] [illegible]
				SANDSTONE: fine to coarse grained, light grey, grey and orange brown.		H			
		24		END OF BOREHOLE AT 23.62m					50mm DIA. PVC CLASS 18 STANDPIPE INSTALLED TO 8.47m. SLOTTED BETWEEN [illegible]m AND 8.47m. 2mm SAND FILTER PACK BETWEEN [illegible]m AND 8.47m. BENTONITE SEAL FROM [illegible]m TO [illegible]m. COMPLETED WITH A LOCKABLE CAP AND GATIC COVER. DATA LOGGER INSTALLED TO 8.47m AT [illegible]
		25							
		26							
		27							
		28							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
203
1/5

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB2
Date: 23-8-12

Method: SPIRAL AUGER TO 7.5m
THEN WASHBORING
JK500

Logged/Checked by: D.W./[illegible]

R.L. Surface: ≈ 4.3m
Datum: AHD

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
		0		CONCRETE: 190mm t				
	N = 1 2,0,1		-	FILL: Gravelly silty sand, fine to coarse grained, dark grey, fine to medium grained sandstone gravel, trace of slag, ash and tile fragments.	M			APPEARS POORLY COMPACTED
		1		FILL: Silty sand, fine to medium grained, light brown, with fine to medium grained sandstone gravel.				
	N = 4 2,2,2		SP	SAND: medium grained, grey	D	L		
		2						REFER TO DCP TEST RESULTS FOR RELATIVE DENSITIES
ON COMPLETION OF CORING		3			M W			
		4						MONITORING WELL INSTALLED ON COMPLETION TO 5.05m DEPTH. CLASS 18 50mm DIAMETER PVC SLOTTED 5.05m TO 2.05m, CASING 2.05m TO SURFACE. SAND BACKFILLED 5.05m TO 0.2m. BENTONITE 0.2m TO SURFACE. SEALED WITH LOCKABLE CAP AND GATIC COVER
		5						
		6						
			SM	SILTY SAND: medium grained, dark grey brown, with roots, trace of organic clay	W/ MC>PL			ORGANIC ODOUR
		7						

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BOREHOLE LOG

Borehole No. **203**

2/5

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21498SB2
Date: 23-8-12

Method: SPIRAL AUGER TO 7.5m THEN WASHBORING JK500
Logged/Checked by: D.W./

R.L. Surface: ≈ 4.3m
Datum: AHD

Groundwater Record	SAMPLES	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
					SP	SAND: medium grained, dark grey.	W			
										WASHBORING
			8			SAND: fine to medium grained, grey.				
			9			as above, but dark grey.				
			10							
			11							
			12							
			13			as above, but grey brown.				
			14							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
203
3/5

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB2
Date: 23-8-12

Method: SPIRAL AUGER TO 7.5m THEN WASHBBORING JK500
Logged/Checked by: D.W / [illegible]

R.L. Surface: ~ 4.3m
Datum: AHD

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
								SP	SAND: fine to medium grained, grey brown.	W			
						15							
						16							
						17							
						18							
						19							
						20							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
203
4/5

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21436SB2
Date: 23-8-12

Method: SPIRAL AUGER TO 7.5m THEN WASHBBORING JK500
Logged/Checked by: D.W / [illegible]

R.L. Surface: ≈ 4.3m
Datum: AHD

Groundwater Record	SAMPLES	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					SP	SAND: fine to medium grained, grey brown	W			
			22							
			23							
					SC/CL	BANDED SILTY SAND and SILTY CLAY: fine to medium grained, grey and grey brown	W, MC>PL			POSSIBLE RESIDUAL SANDSTONE
			24							
			25							
					SC	BANDED CLAYEY SAND and SANDSTONE: fine to medium grained, light grey and orange brown	W (XW)	(L)		BANDED RESISTANCE
			26							
			27							
						REFER TO CORED BOREHOLE LOG				


JK Geotechnics
CLIENT: Manly Civic Club Limited
PROJECT: Proposed Redevelopment of Manly Civic Club
LOCATION: 2 West Promenade, Manly, NSW
SCALE (CM)
Job No 21496SB2
BH203
START CORING AT 27.51m
27
START
28
29
30
31
32
33
END
END OF BH AT 33.94m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
203
5/5

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21498SB2 **Core Size:** NMLC **R.L. Surface:** ≈ 4.3m
Date: 23-8-12 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** D.W.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating (Specific / General)
		27		START CORING AT 27.5m					
90% RETURN				SANDSTONE: medium grained, orange brown, cross bedded at 15-20°	DW	H			
		28		as above, but orange brown, pink, yellow brown and light grey					
		29		as above, but medium to coarse grained	SW				[illegible]
		30		as above, but medium grained, orange brown and light grey, cross bedded at 10-20°					
				as above, but light grey, with occasional orange brown, grey and dark grey laminae	FR				[illegible]
		31		as above, but light grey, with occasional dark grey laminae					[illegible]
		32							
FULL RETURN				LAMINITE: fine grained, dark grey and light grey	SW	L			[illegible]
		33				M			
				END OF BOREHOLE AT 33.94m					

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
204
1/4

Client: MANLY CIVIC CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB2
Date: 24-8-12
Method: SPIRAL AUGER TO 6.0m THEN WASHBORING JK500
Logged/Checked by: D.W./NP
R.L. Surface: ≈ 4.0m
Datum: AHD

Groundwater Record	Samples	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/Weathering	Strength/Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
			0		ASPHALTIC CONCRETE: 50mm.t				
		N = 5 2,3,2			FILL: Gravelly silty sand, fine to coarse grained, dark grey and light brown, fine to medium grained sandstone gravel, with slag and ash, trace of clay.	M			APPEARS POORLY COMPACTED
			1						
		N = 3 1,1,2		SP	SAND: fine to medium grained, grey.	D	VL		
			2			M			REFER TO DCP TEST RESULTS FOR RELATIVE DENSITIES
▸			3		as above, but dark grey brown.	W			
			4						MONITORING WELL INSTALLED ON COMPLETION TO 6.1m DEPTH. CLASS 18 50mm DIAMETER PVC, SLOTTED 6.1m TO 2.1m. PVC CASING 2.1m TO SURFACE. SAND BACKFILLED 6.1m TO 0.25m. BENTONITE 0.25m TO SURFACE. SEALED WITH LOCKABLE CAP AND GATIC COVER.
			5						
			6						WASHBORING
			7						

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **204**

2/4

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB2

Date: 24-8-12

Method: SPIRAL AUGER TO 6.0m THEN WASHBORING JK500

Logged/Checked by: D.W./ [illegible]

R.L. Surface: ≈ 4.0m

Datum: AHD

Groundwater Record	SAMPLES	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa)	Remarks
					SP	SAND: fine to medium grained, dark grey brown.	W			
						as above, but dark grey.				
			8							
			9			as above, dark grey brown				
			10							
			11							
			12							
						as above, but dark grey.				
			13							
						as above, but dark grey brown.				
			14							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
204
3/4

Client:	MANLY CIVIC CLUB LIMITED
Project:	PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB
Location:	2 WEST PROMENADE, MANLY, NSW

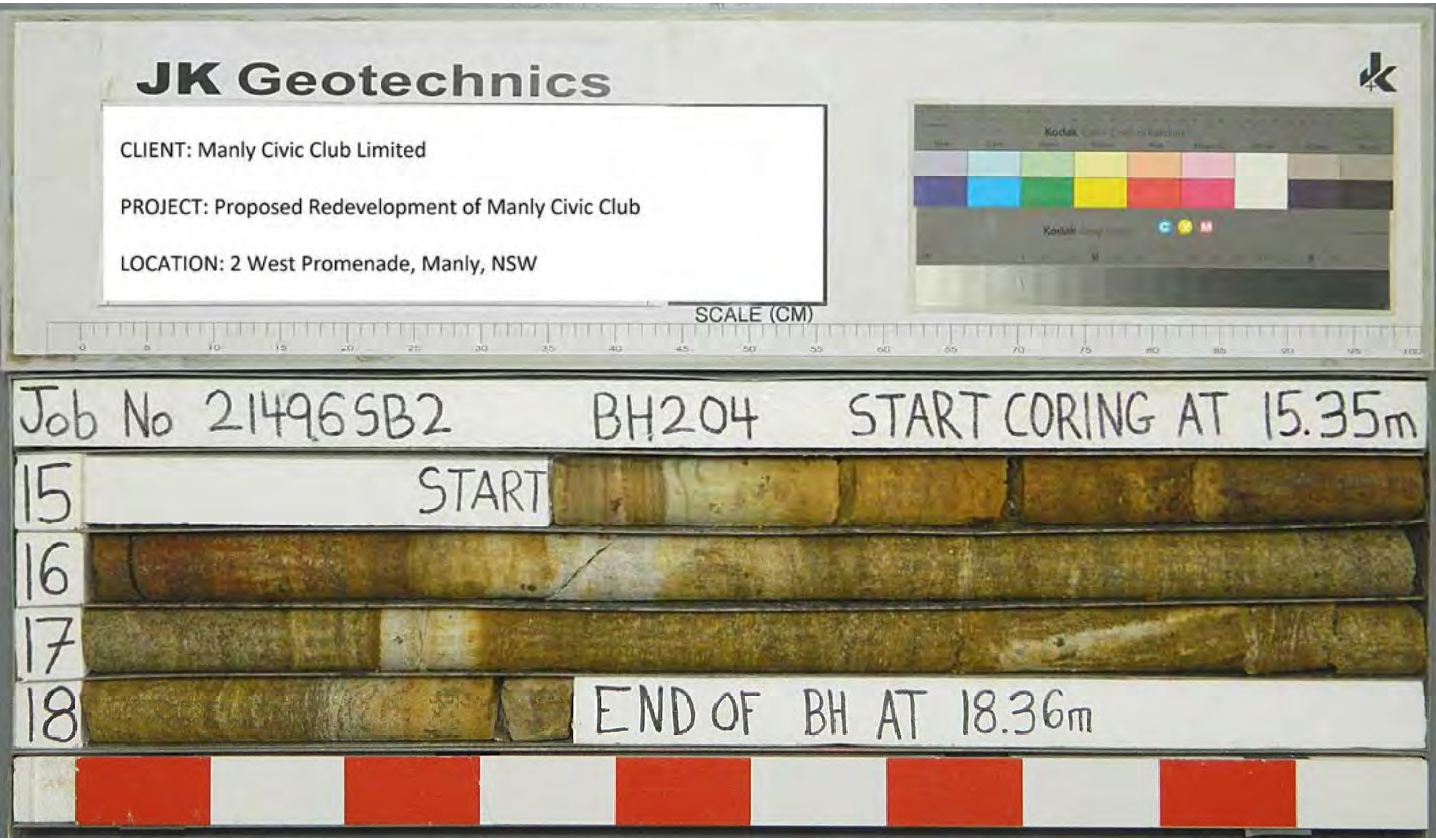
Job No. 21496SB2
Date: 24-8-12

Method: SPIRAL AUGER TO 6.0m THEN WASHBORING JK500

Logged/Checked by: D.W./

R.L. Surface: ≈ 4.0m
Datum: AHD

Groundwater Record	SAMPLES	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			15		SP	SAND: fine to medium grained, dark grey brown	W			
						SANDSTONE: medium grained, orange brown	DW	L-M		
						REFER TO CORED BOREHOLE LOG				
			16							
			17							
			18							
			19							
			20							


JK Geotechnics
CLIENT: Manly Civic Club Limited
PROJECT: Proposed Redevelopment of Manly Civic Club
LOCATION: 2 West Promenade, Manly, NSW
SCALE (CM)
Job No 21496SB2
BH204
START CORING AT 15.35m
15
START
16
17
18
END OF BH AT 18.36m

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **204**

4/4

Client: MANLY CIVIC CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF MANLY CIVIC CLUB

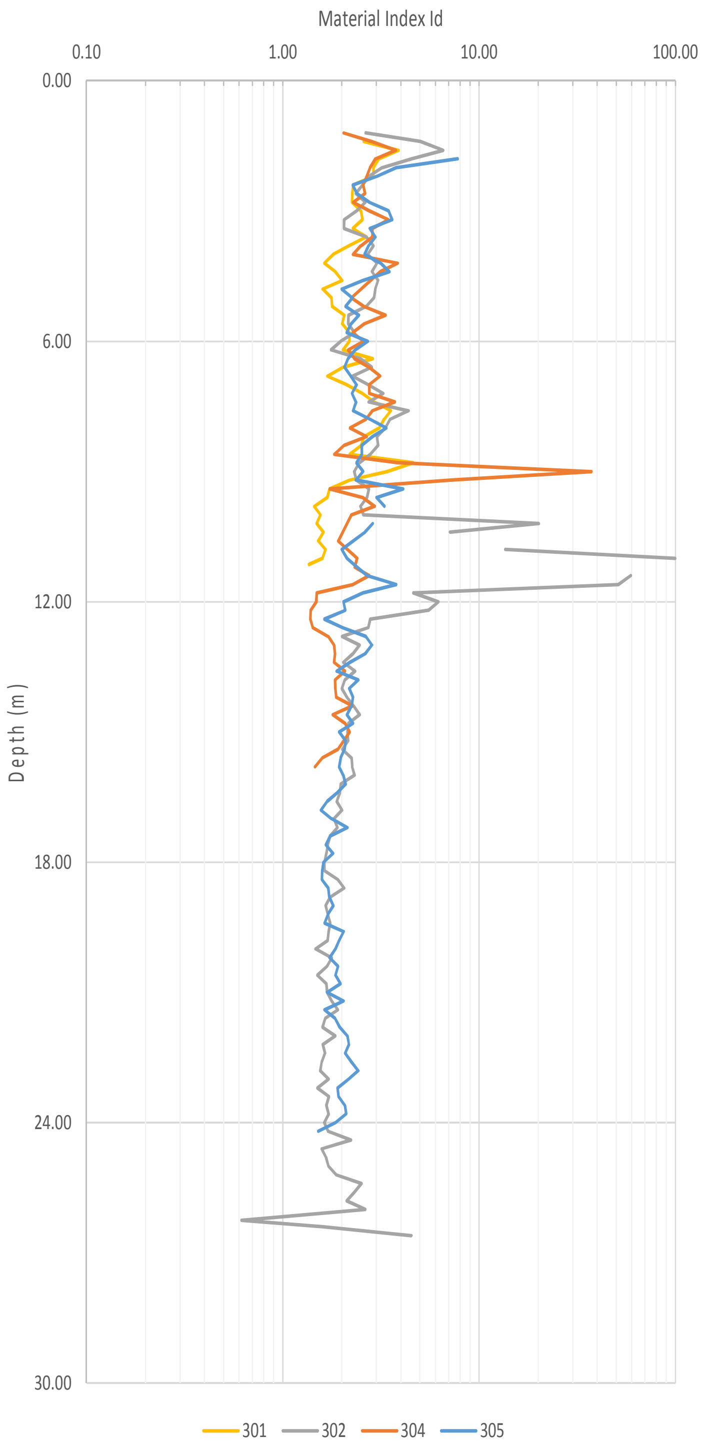
Location: 2 WEST PROMENADE, MANLY, NSW

Job No. 21496SB2	**Core Size:** NMLC	**R.L. Surface:** ≈ 4.0m
Date: 24-8-12	**Inclination:** VERTICAL	**Datum:** AHD
Drill Type: JK500	**Bearing:** -	**Logged/Checked by:** D.W / [illegible]

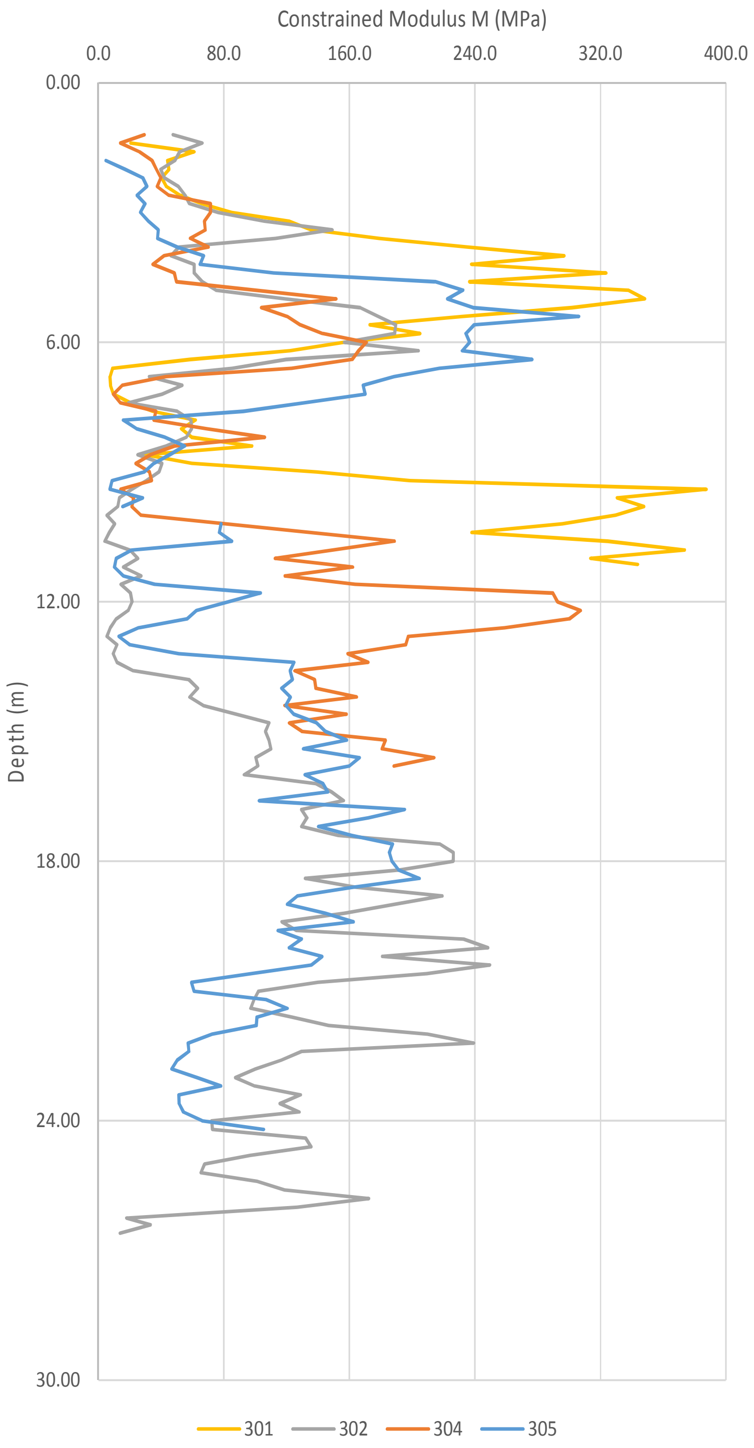
Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating — Specific / General
				START CORING AT 15.2m					
FULL RET-URN				SANDSTONE: medium grained, orange brown, light grey, dark red brown and dark brown.	DW	M			[illegible]
				as above, but medium to coarse grained.		L			
				as above, but medium grained.		M-H			[illegible]
				END OF BOREHOLE AT 18.5m					

COMBINED DMT INTERPRETED GEOTECHNICAL PARAMETERS

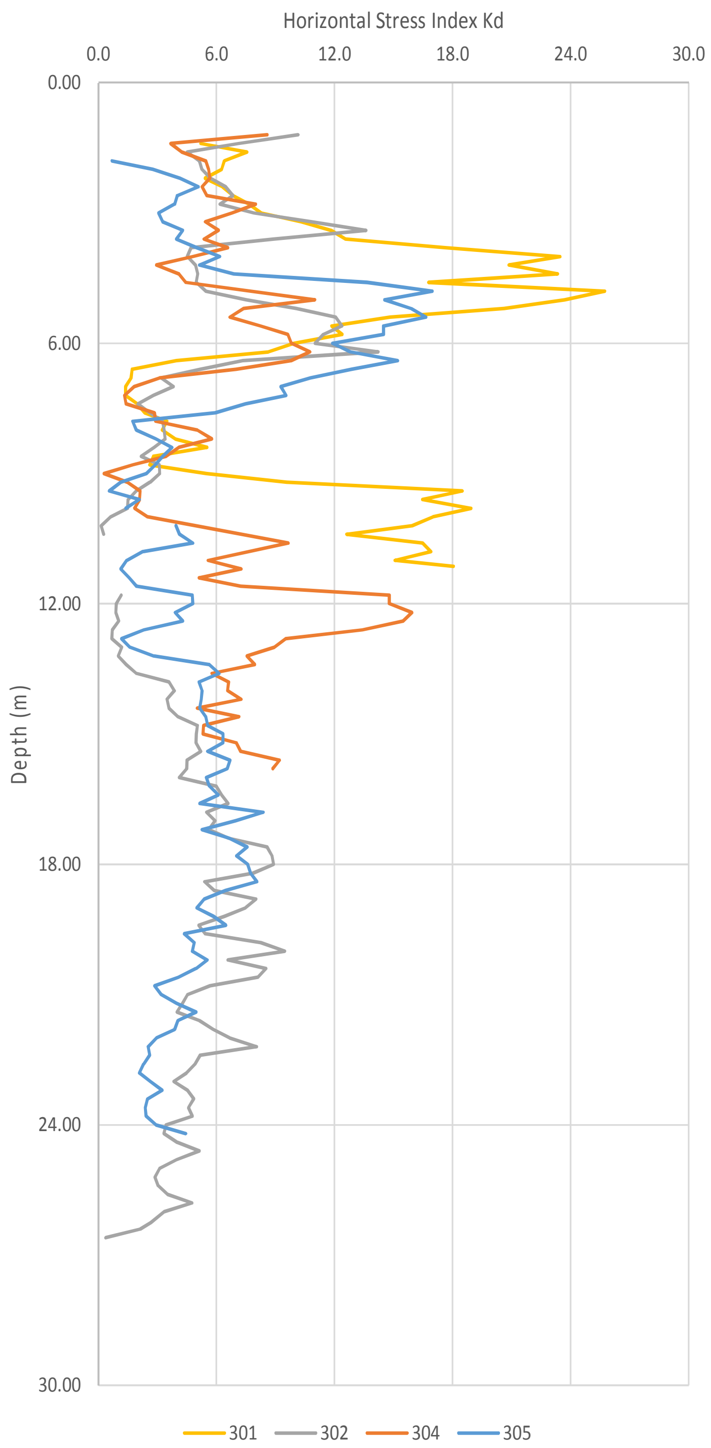
Material Index Id



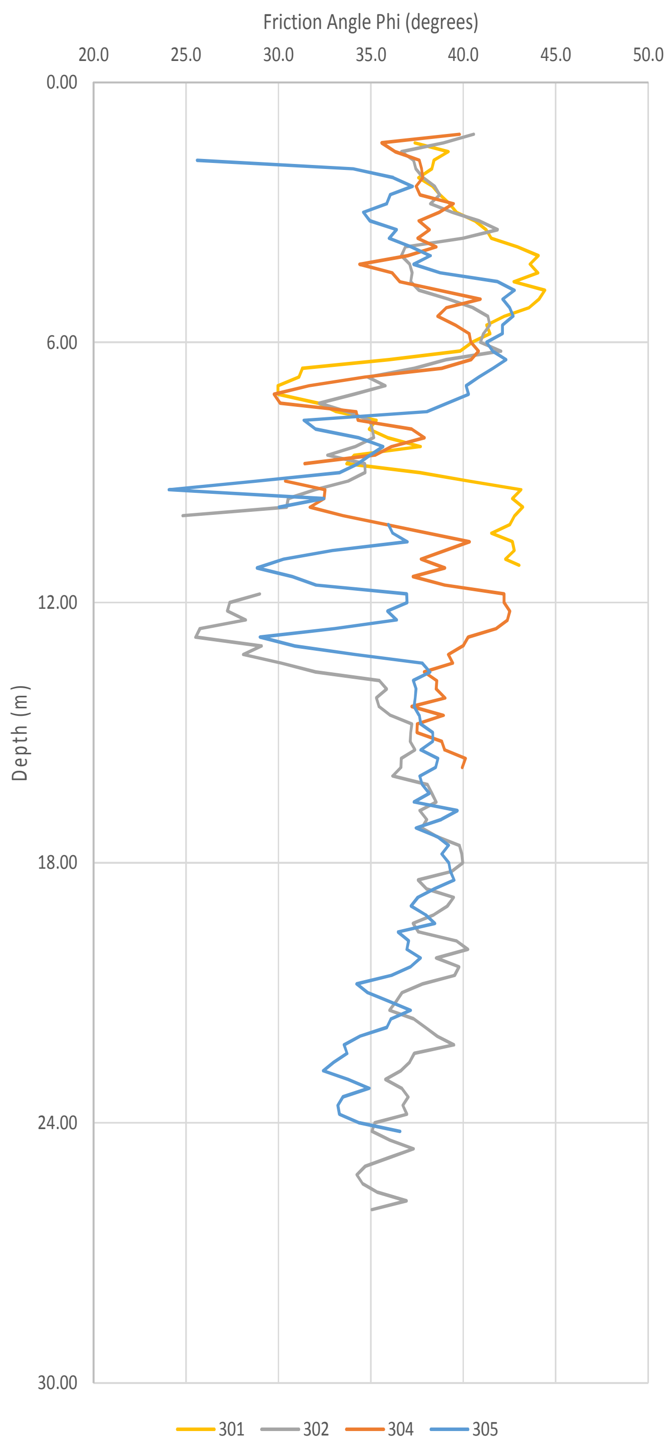
Constrained Modulus M (MPa)



Horizontal Stress Index Kd



Friction Angle Phi (degrees)



DMT301 SURFACE LEVEL ≅RL3.9m AHD
DMT302 SURFACE LEVEL ≅RL4.3m AHD
DNT304 SURFACE LEVEL ≅RL3.8m AHD
DNT305 SURFACE LEVEL ≅RL3.9m AHD

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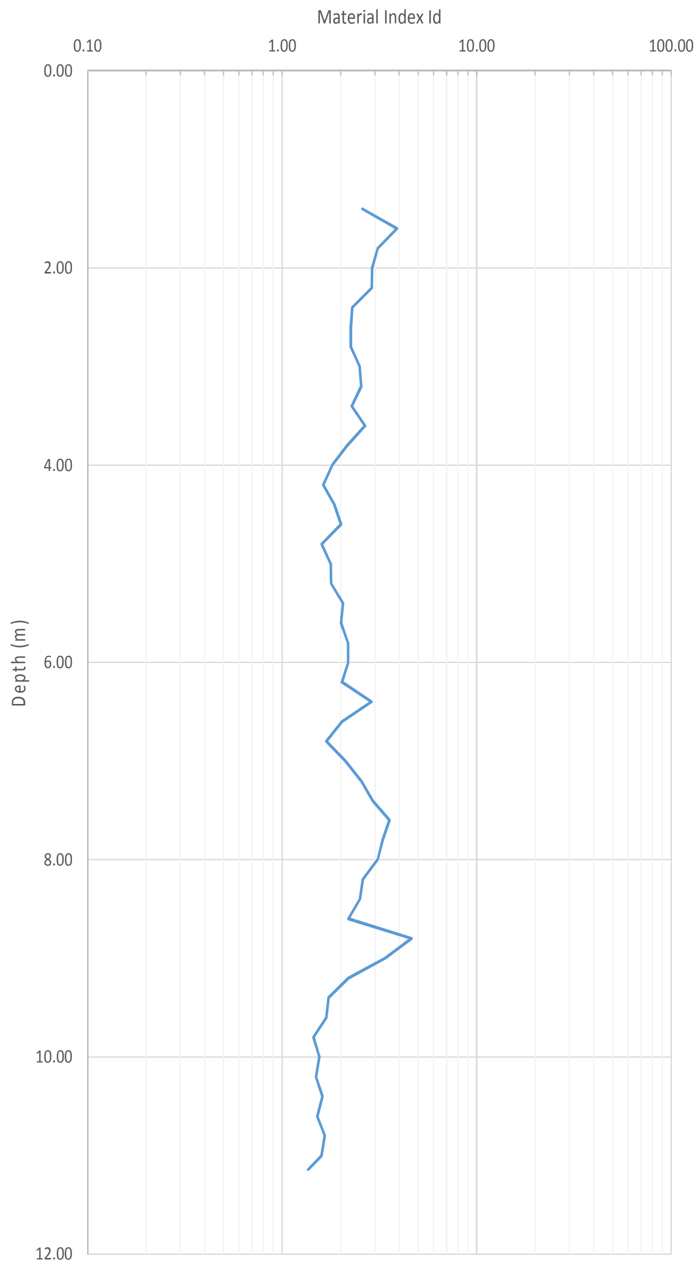
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
Report No. 21496LB

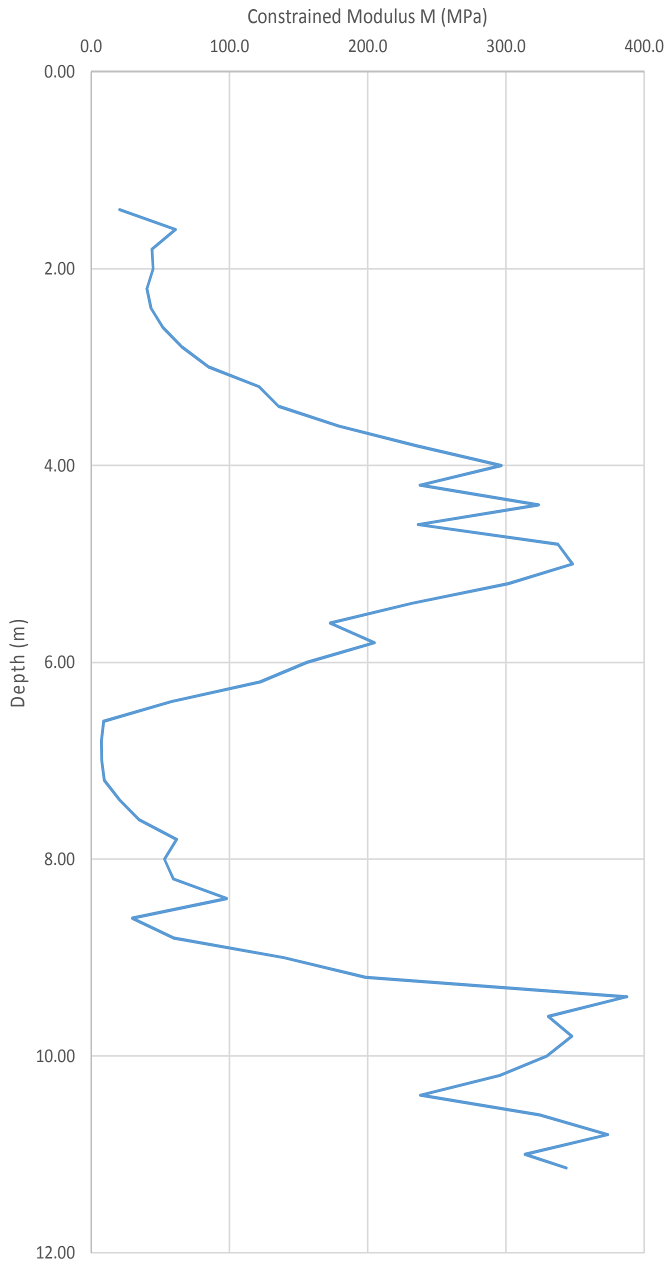
DMT301 INTERPRETED GEOTECHNICAL PARAMETERS

SURFACE LEVEL ≅RL3.9m AHD

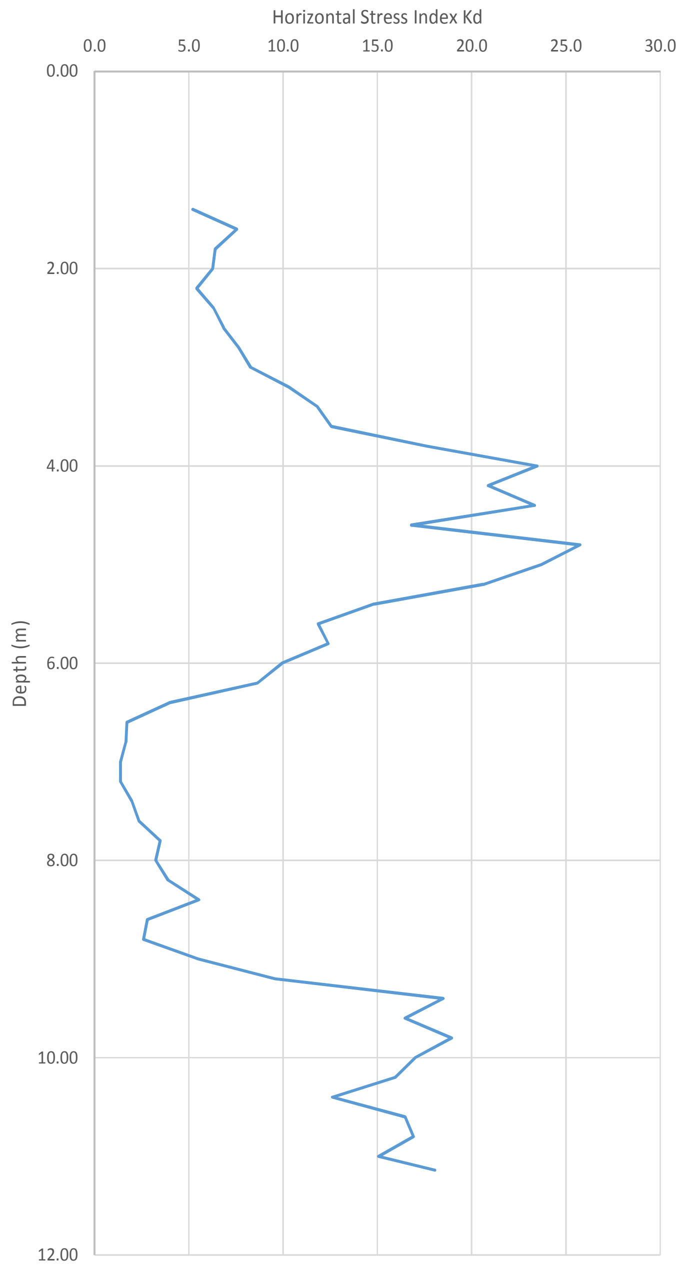
Material Index Id



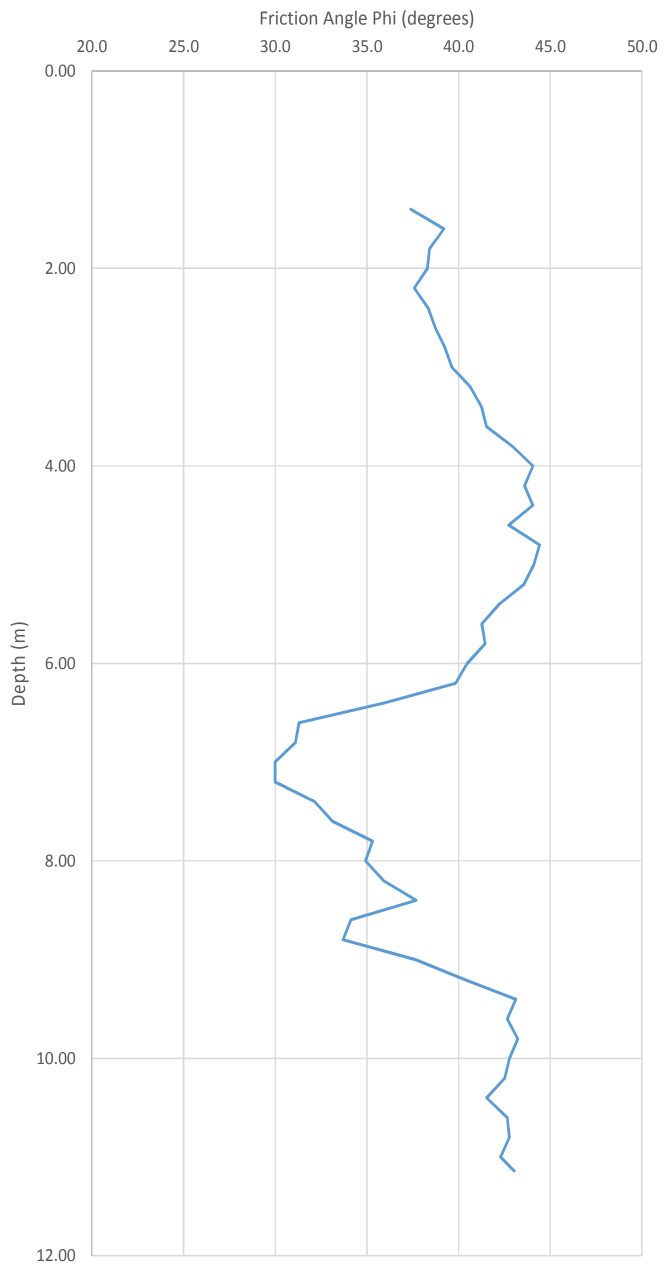
Constrained Modulus M (MPa)



Horizontal Stress Index Kd



Friction Angle Phi (degrees)



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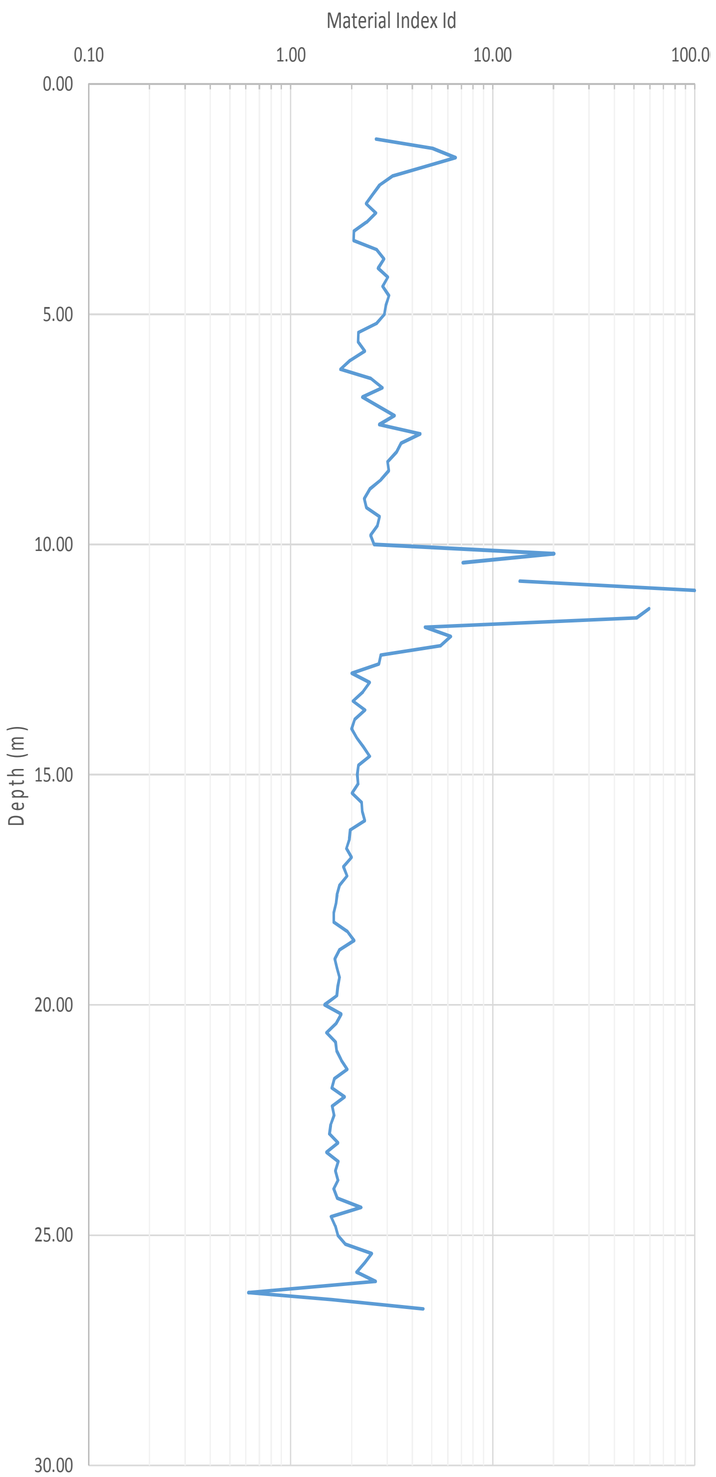
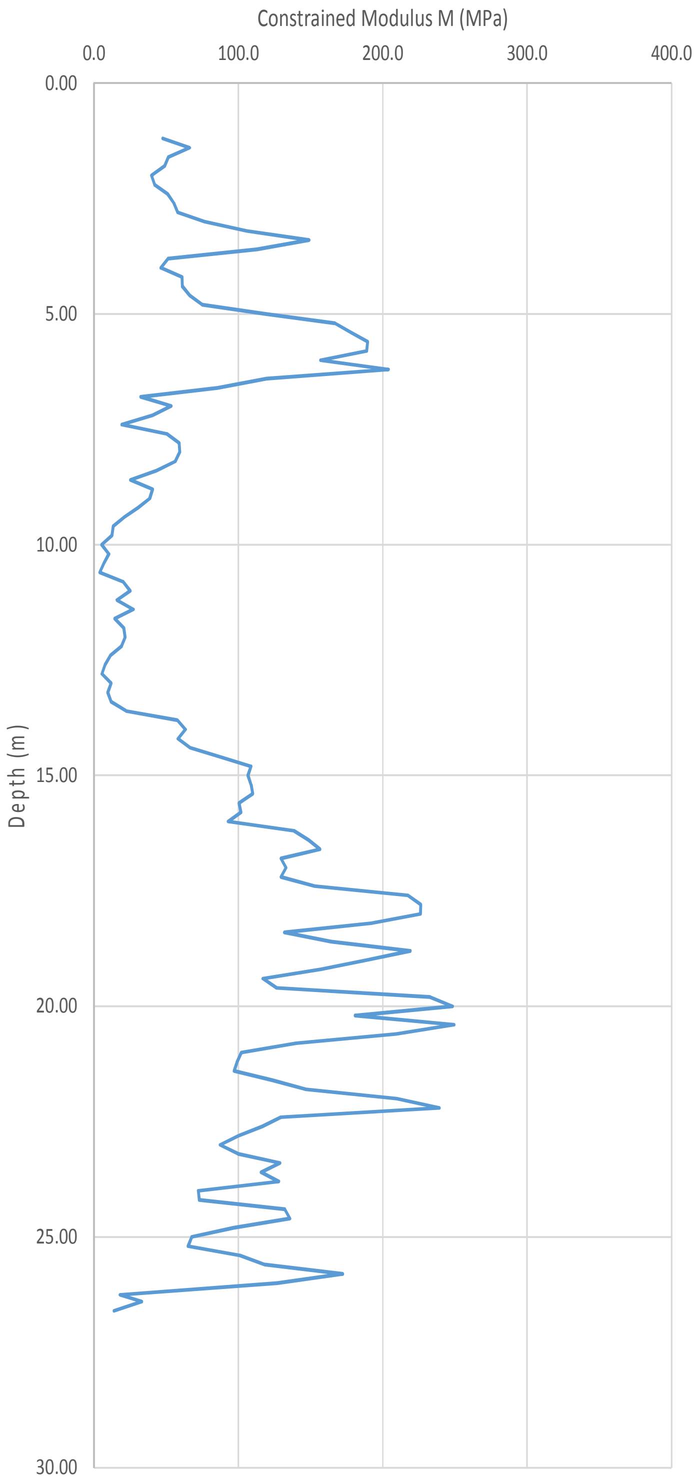
Report No. 21496LB

DMT302 INTERPRETED GEOTECHNICAL PARAMETERS
SURFACE LEVEL ≅RL4.3m AHD

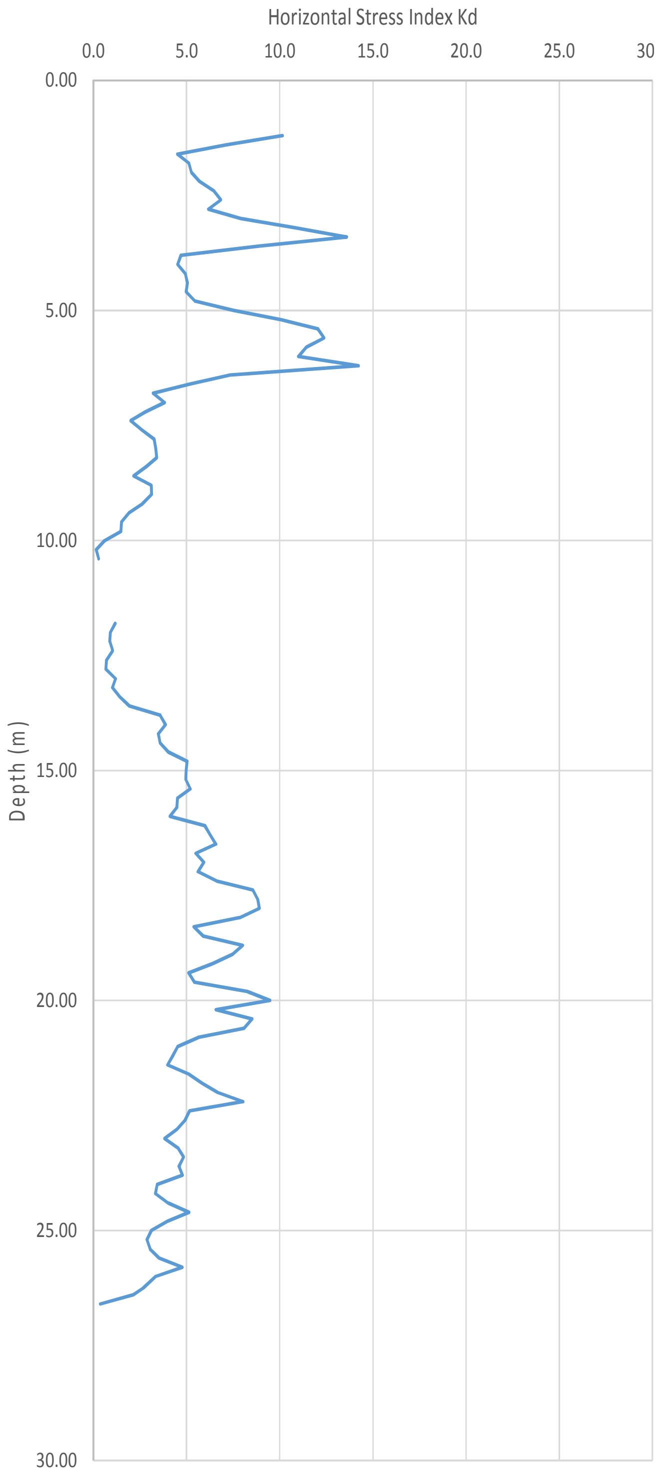
Material Index Id



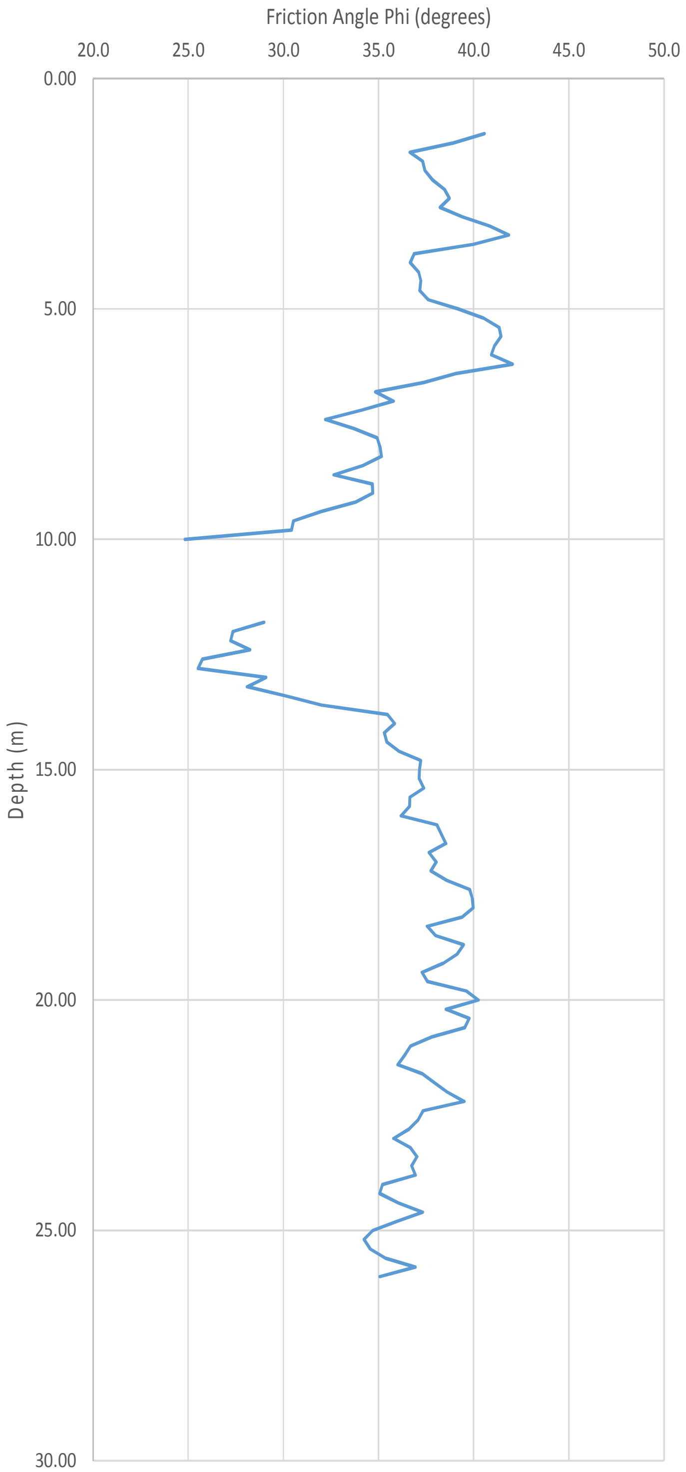
Constrained Modulus M (MPa)



Horizontal Stress Index Kd



Friction Angle Phi (degrees)



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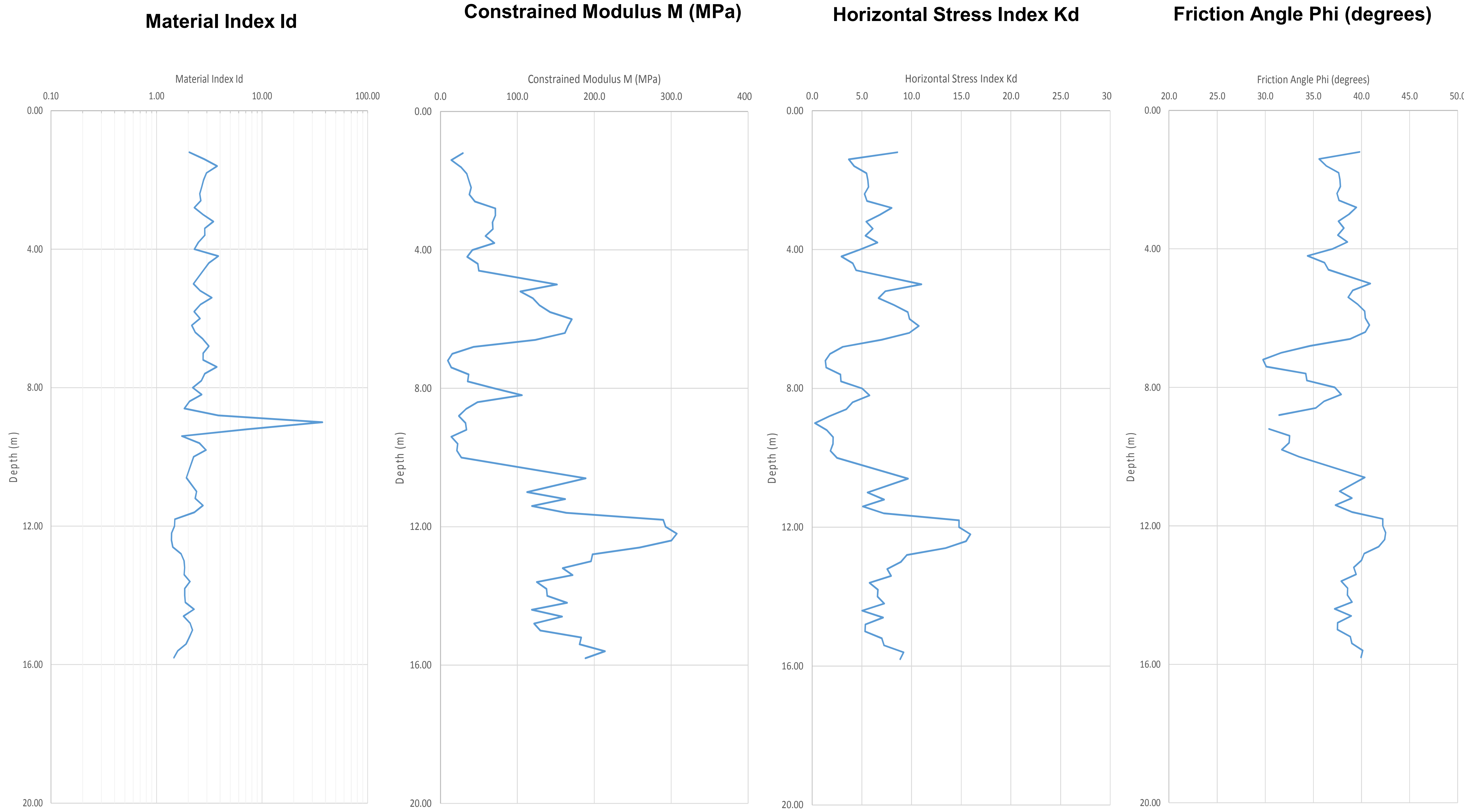
Report No. 21496LB

DMT304 INTERPRETED GEOTECHNICAL PARAMETERS

SURFACE LEVEL ≅RL3.8m AHD



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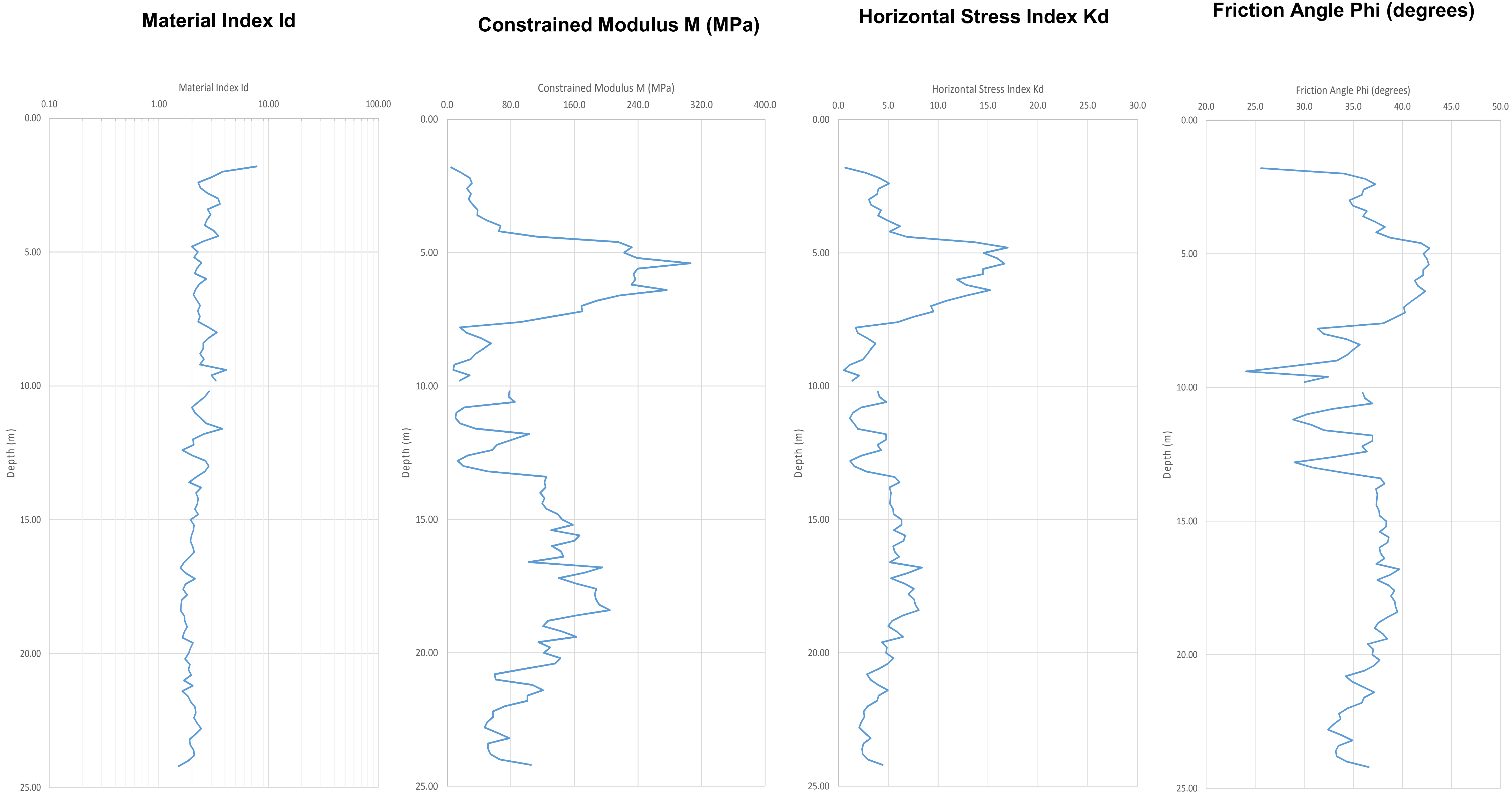
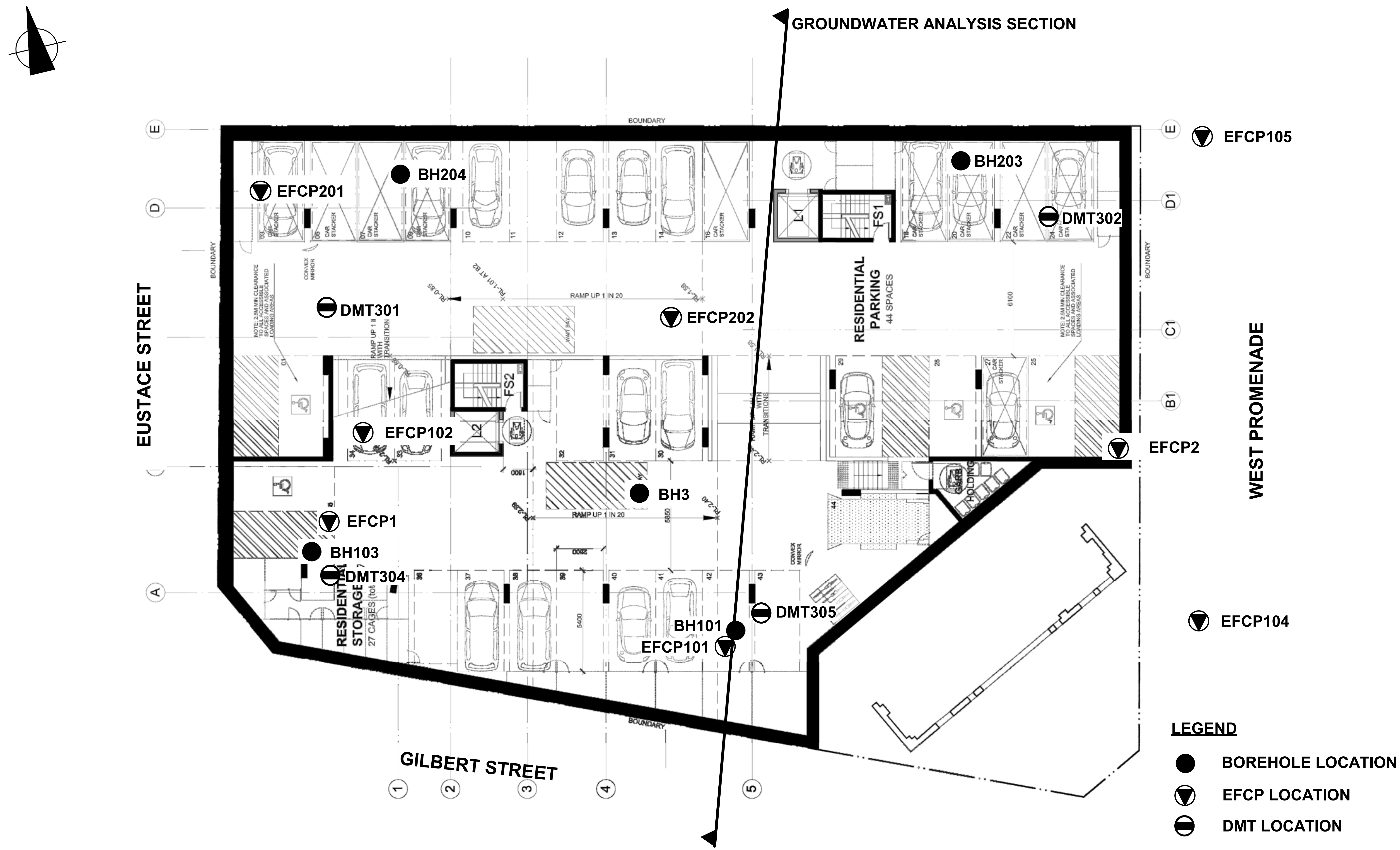
Report No. 21496LB

DMT305 INTERPRETED GEOTECHNICAL PARAMETERS

SURFACE LEVEL ≅RL3.9m AHD



JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
Report No. 21496LB



SCALE (m)

0 10

INVESTIGATION LOCATION PLAN

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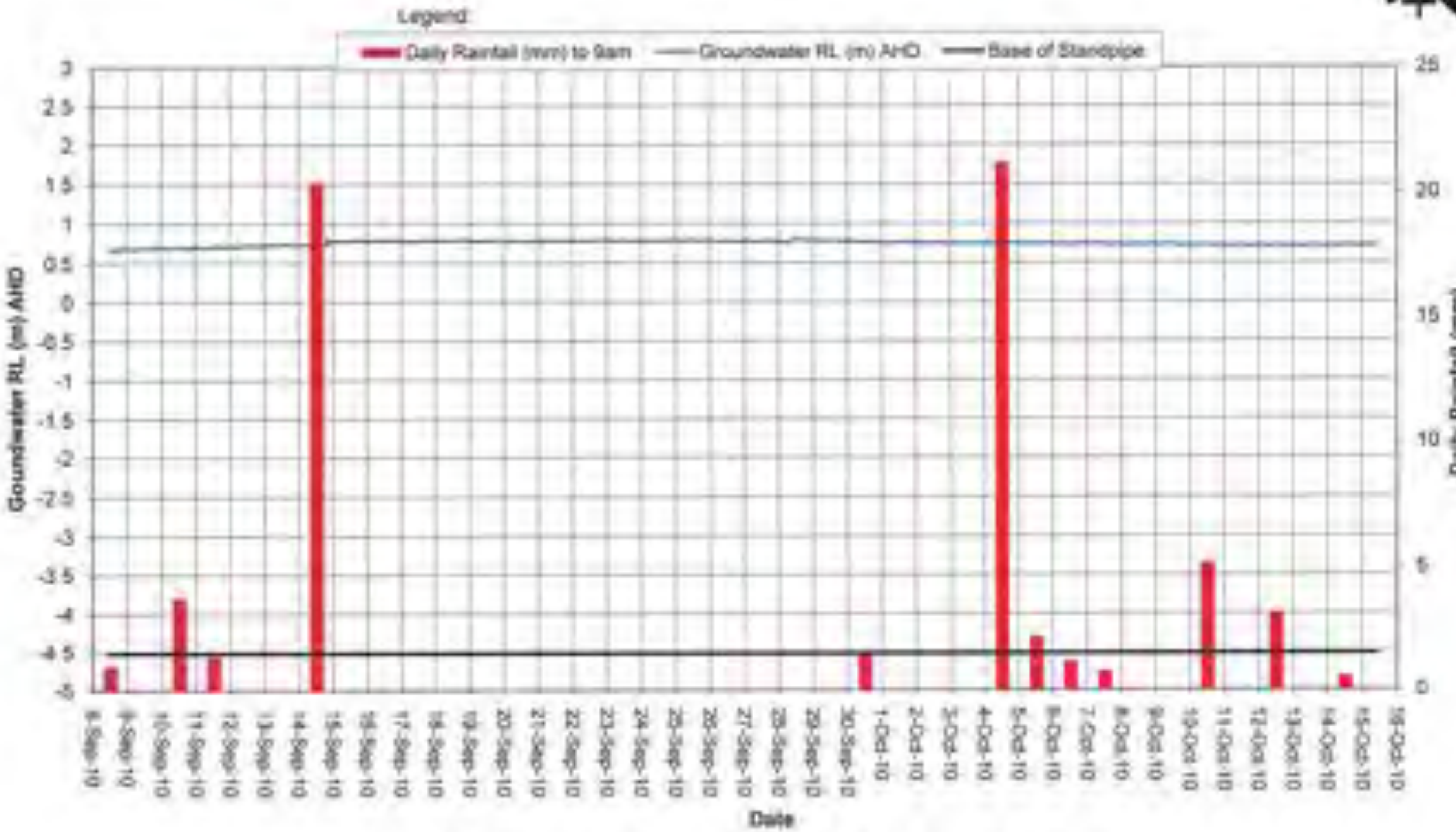
Report No. 21496LB Figure No. 1

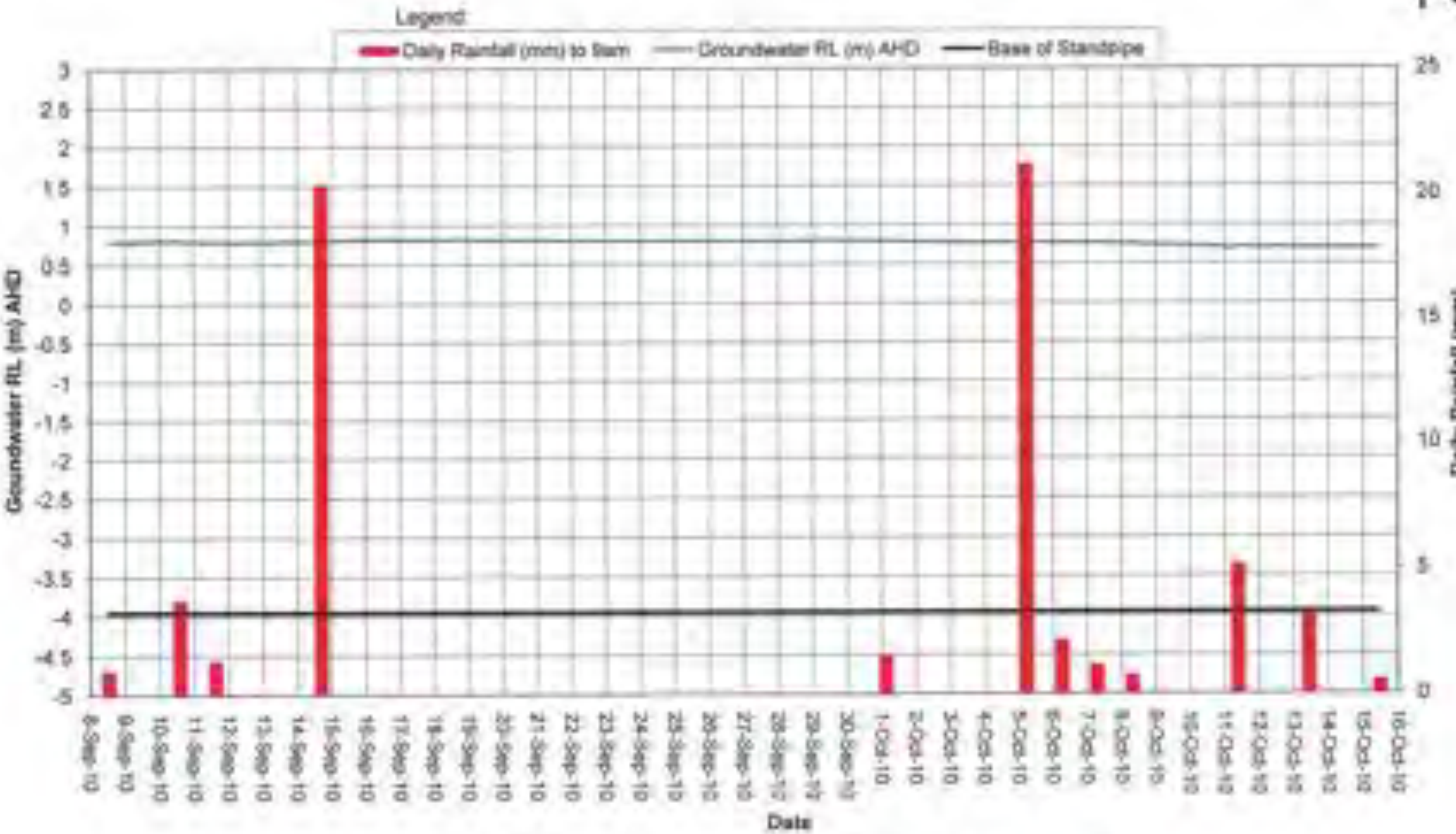


BH3 Groundwater Level and Daily Rainfall -v- Time Plot

Jeffery and Katauskas Pty Ltd

Report No. 21496SB1

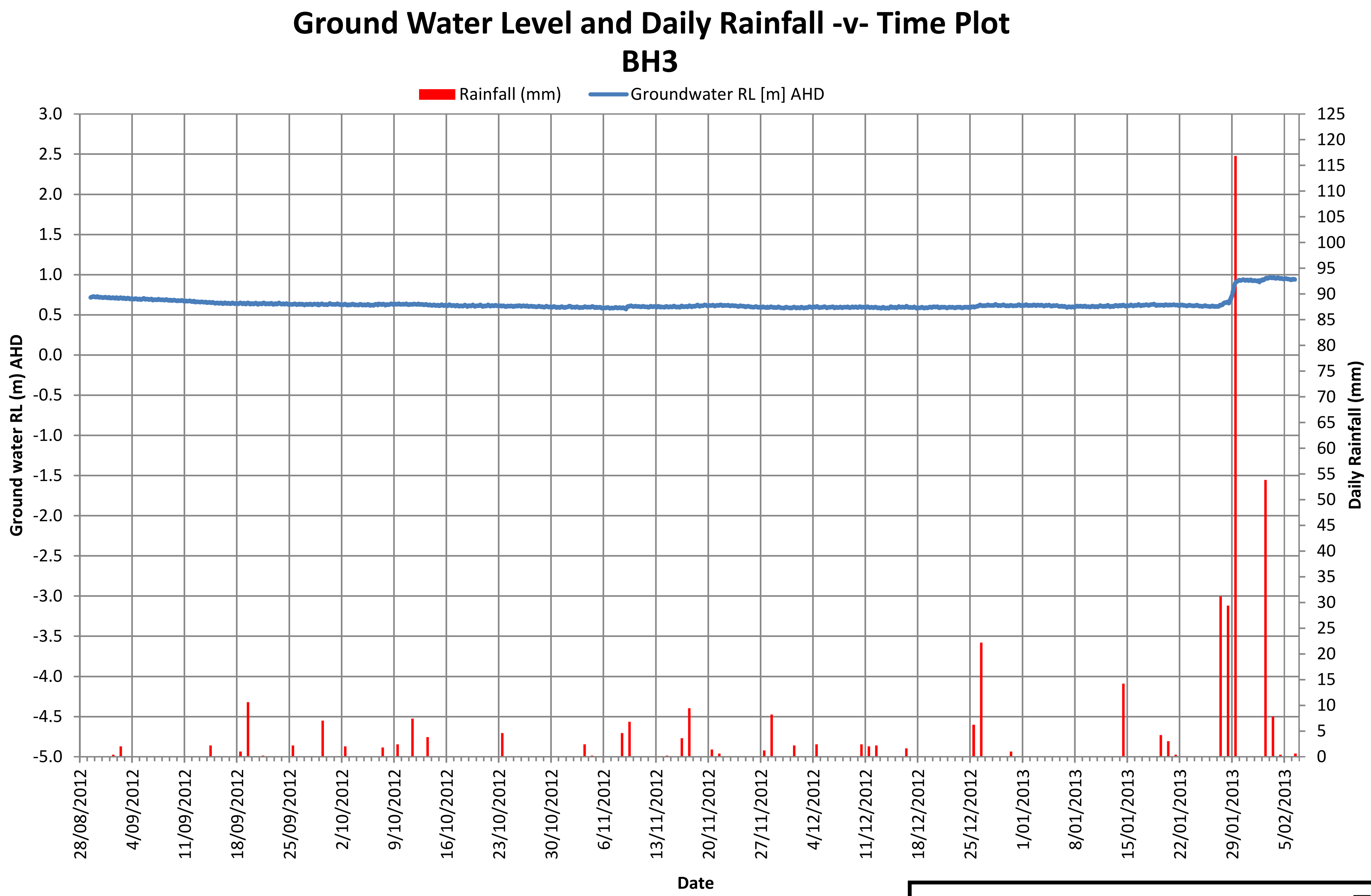
Figure No. 2



BH 103 Groundwater Level and Daily Rainfall -v- Time Plot

Jeffery and Katauskas Pty Ltd	
Report No. 21496SB1	Figure No. 3

File Name: 21496SB2 Plot BH3



Rainfall data from Collaroy (Long Reef Golf Club), Station No. 66126

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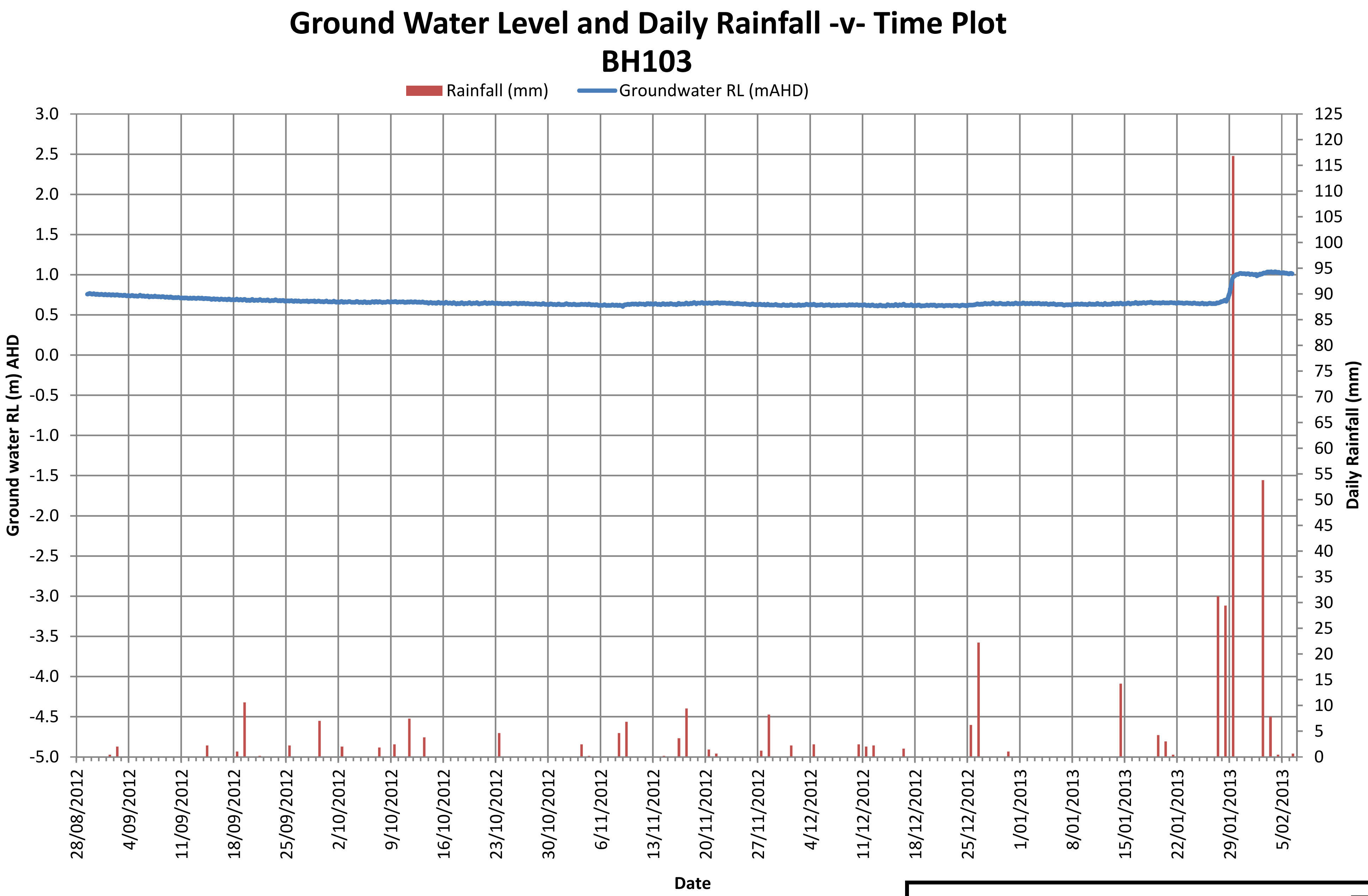
Report No. 21496SB2 **Figure No. 4**

File Name: 21496SB2 Plot BH103



Rainfall data from Collaroy (Long Reef Golf Club), Station No. 66126

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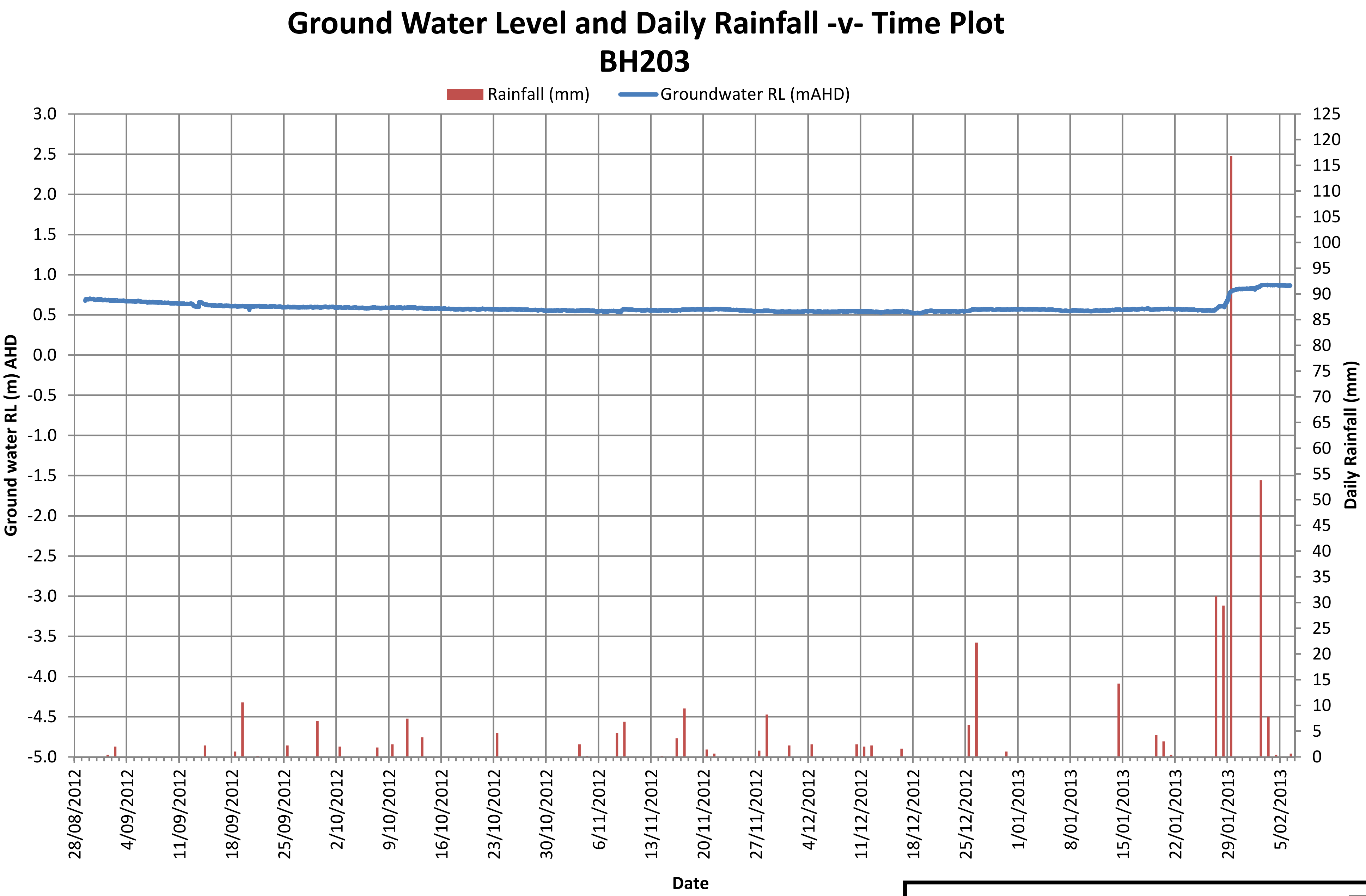
Report No. 21496SB2 Figure No. 5

File Name: 21496SB2 Plot BH203



Rainfall data from Collaroy (Long Reef Golf Club), Station No. 66126

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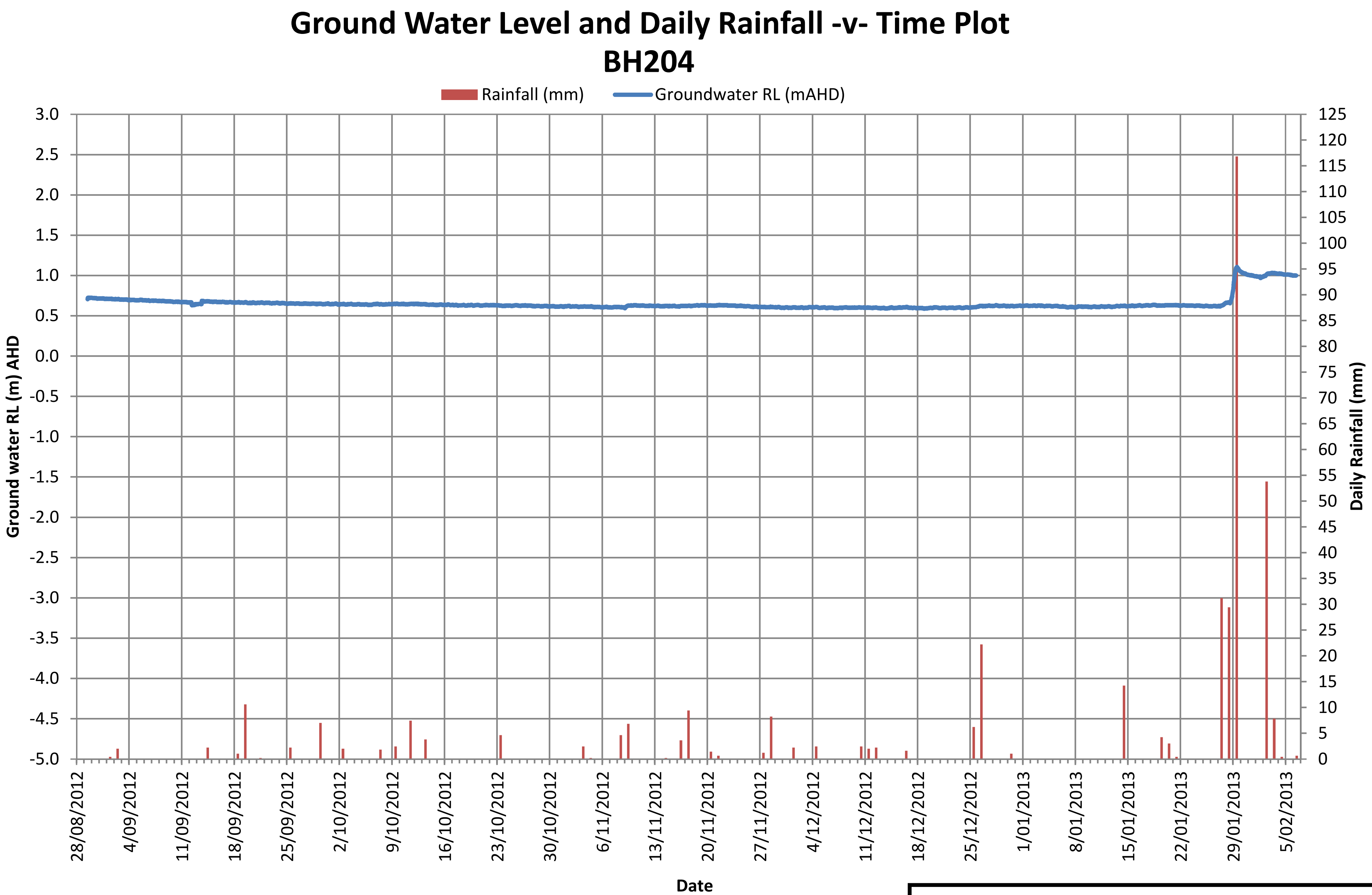
Report No. 21496SB2 Figure No. 6

File Name: 21496SB2 Plot BH204



Rainfall data from Collaroy (Long Reef Golf Club), Station No. 66126

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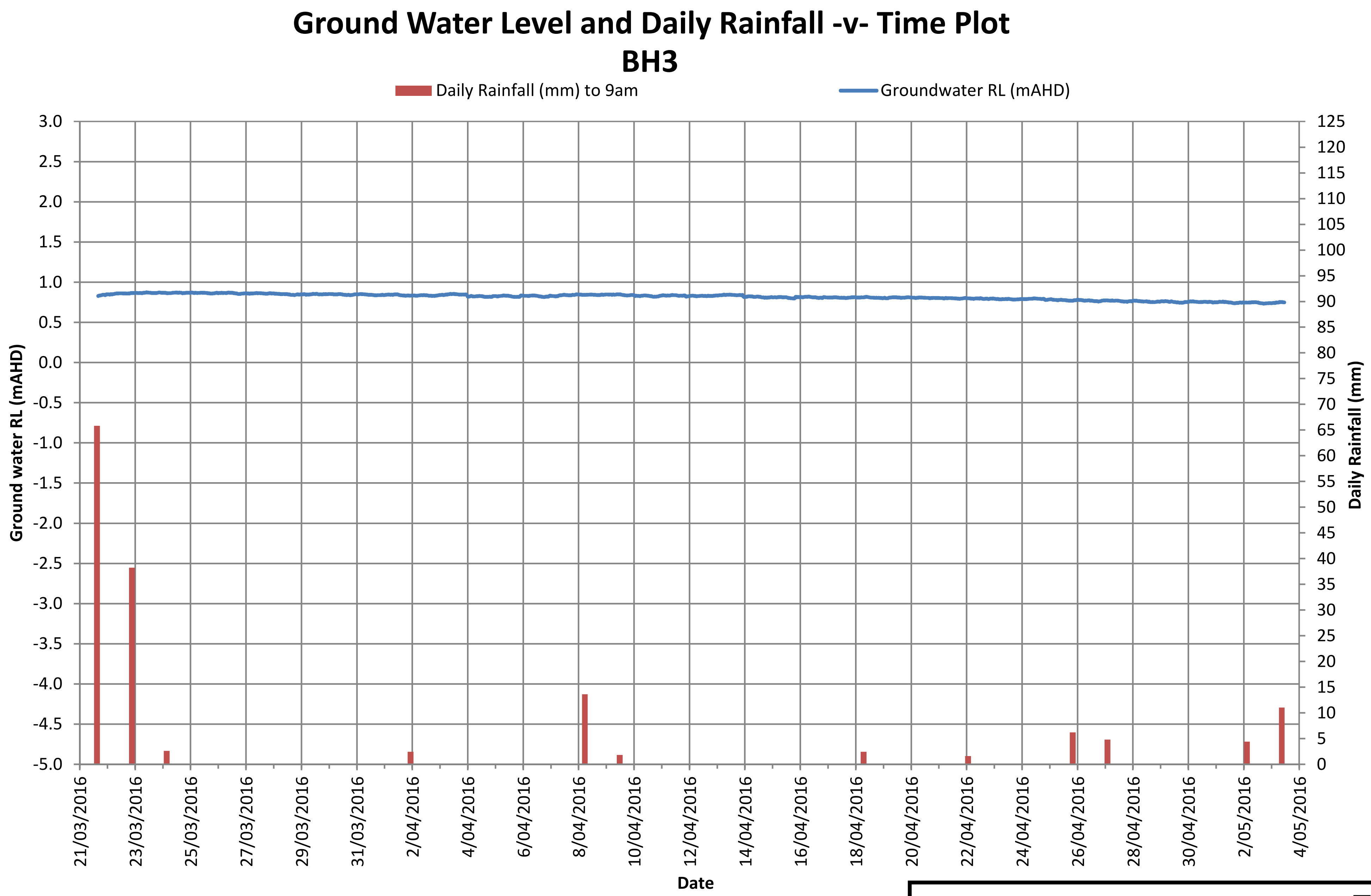
Report No. 21496SB2 **Figure No. 7**

File Name: 21496LB BH3 chart



Rainfall data from Collaroy (Long Reef Golf Club), Station No. 66126

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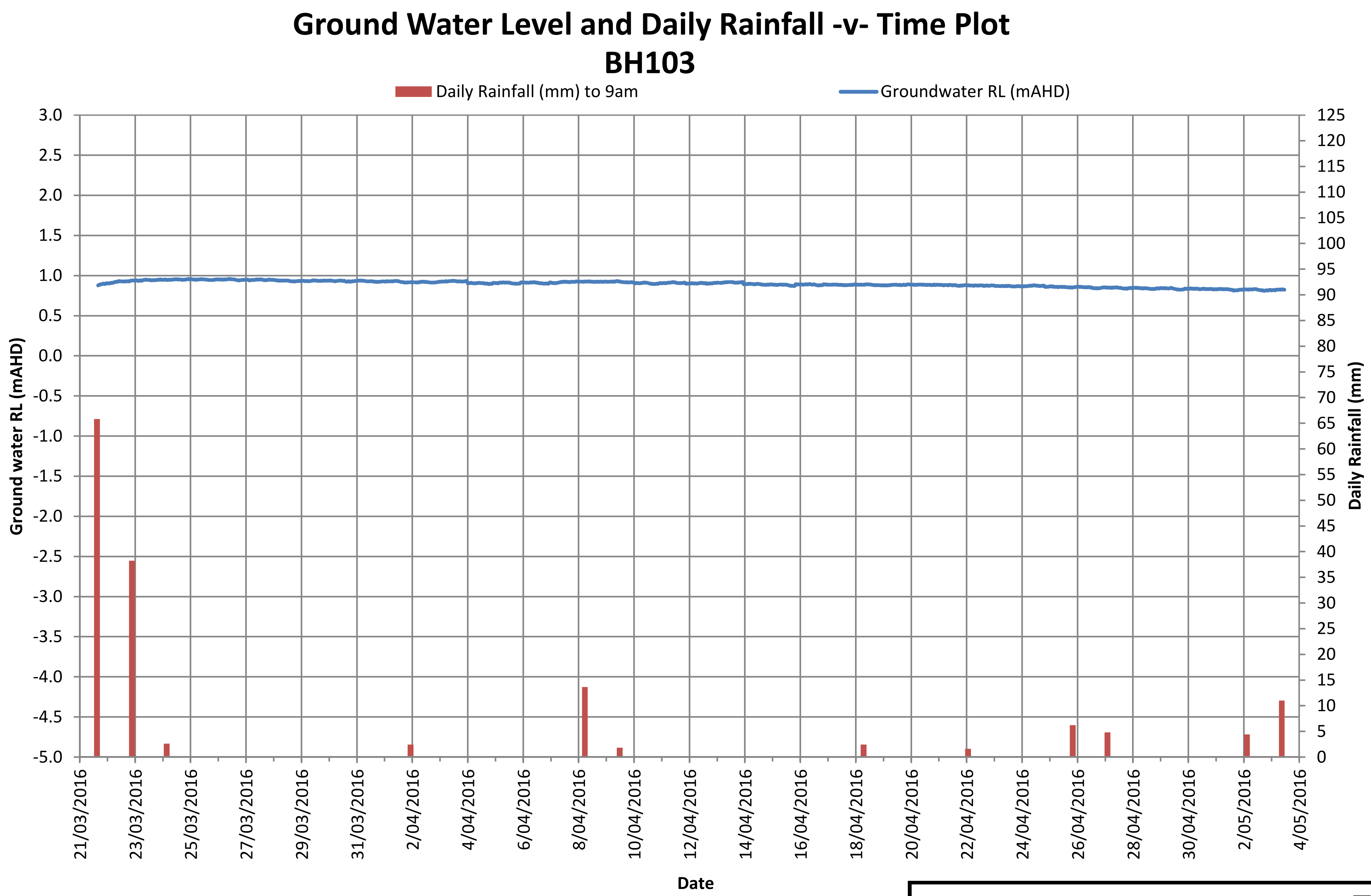
Report No. 21496LB **Figure No. 8**

File Name: 21496LB BH103 chart



Rainfall data from Collaroy (Long Reef Golf Club), Station No. 66126

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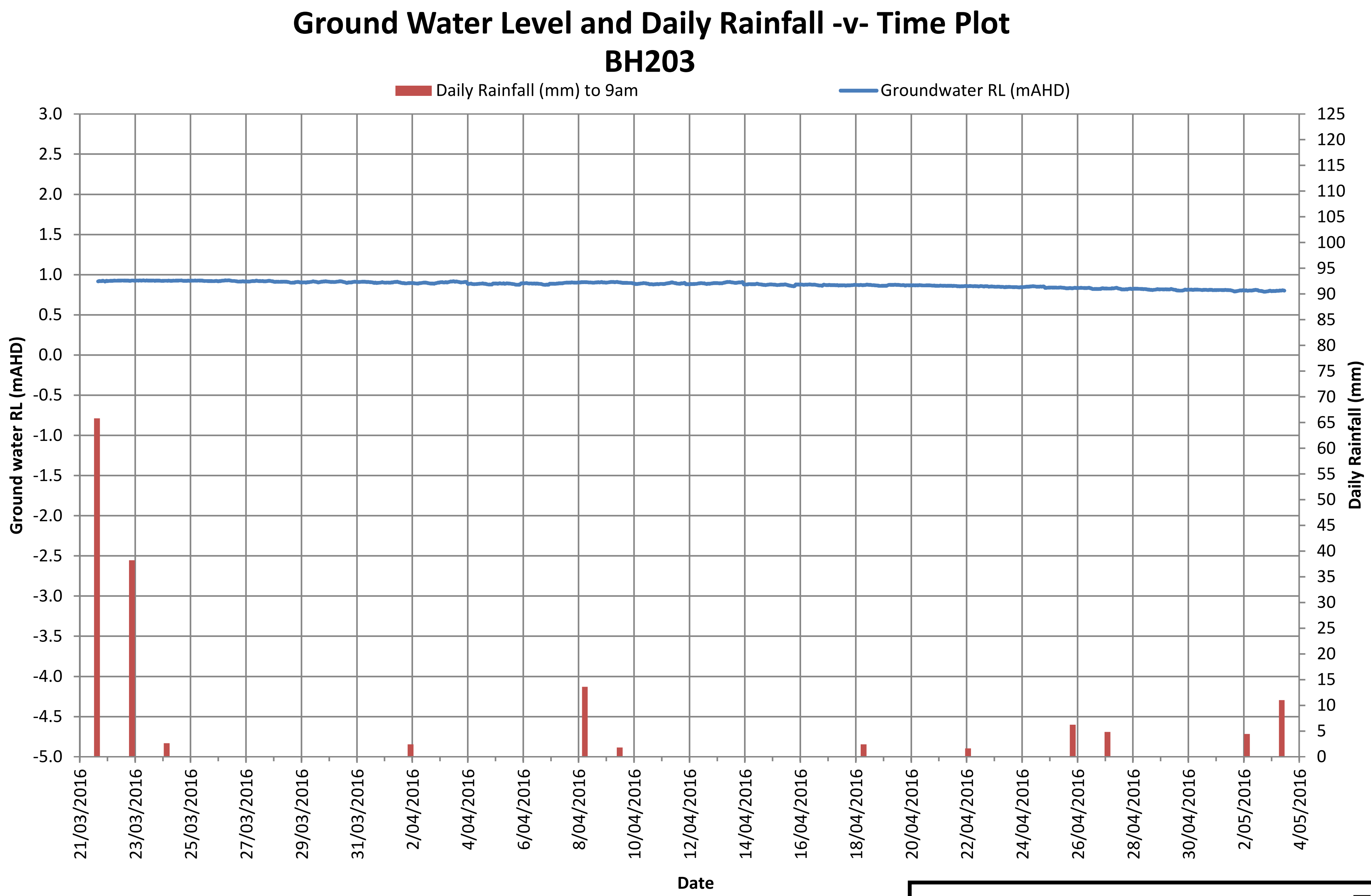
Report No. 21496LB **Figure No. 9**

File Name: 21496LB BH203 chart



Rainfall data from Collaroy (Long Reef Golf Club), Station No. 66126

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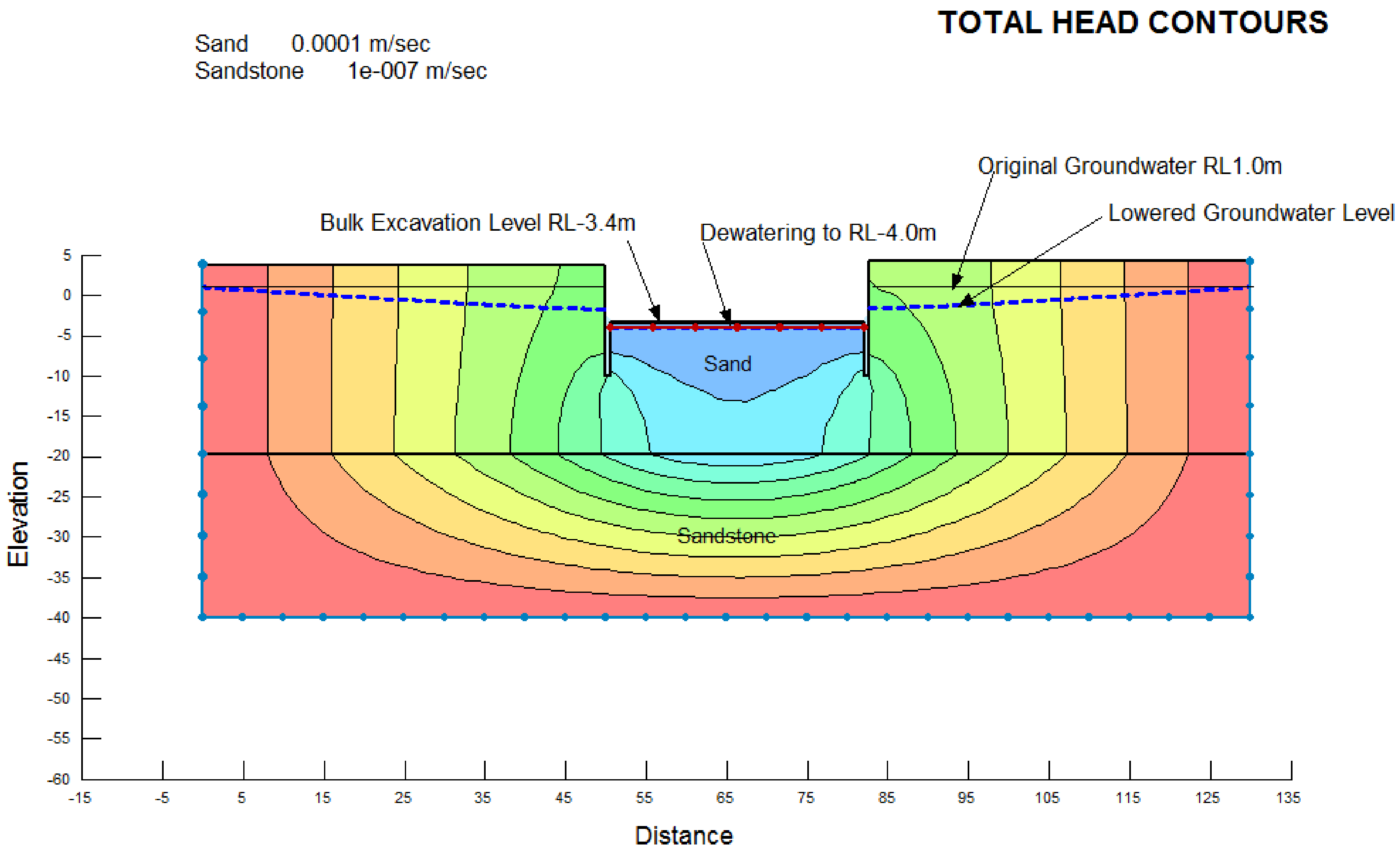
Report No. 21496LB **Figure No. 10**



GROUNDWATER SEEPAGE ANALYSIS TOE LEVEL RL-10m

Jeffery and Katauskas Pty Ltd
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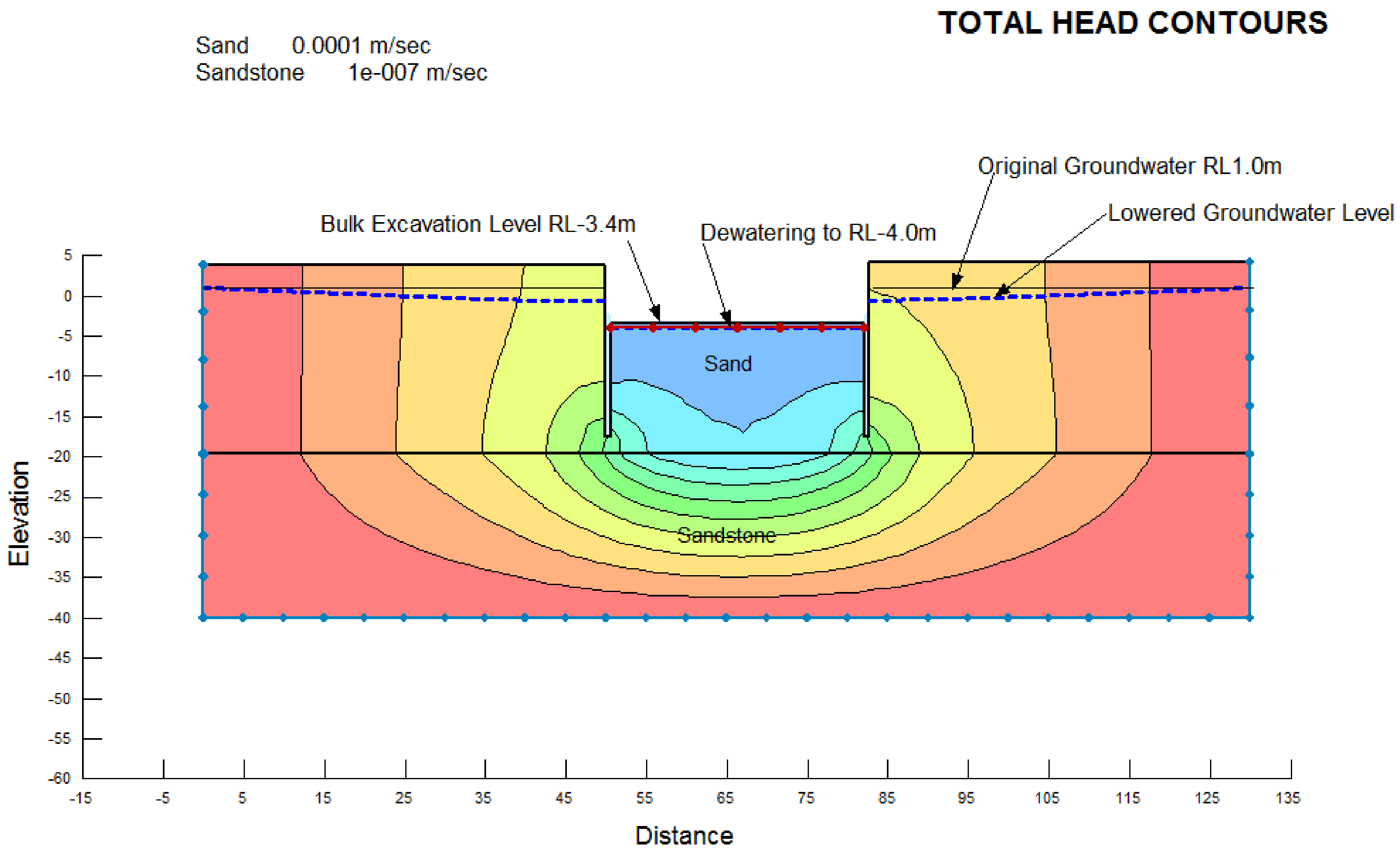
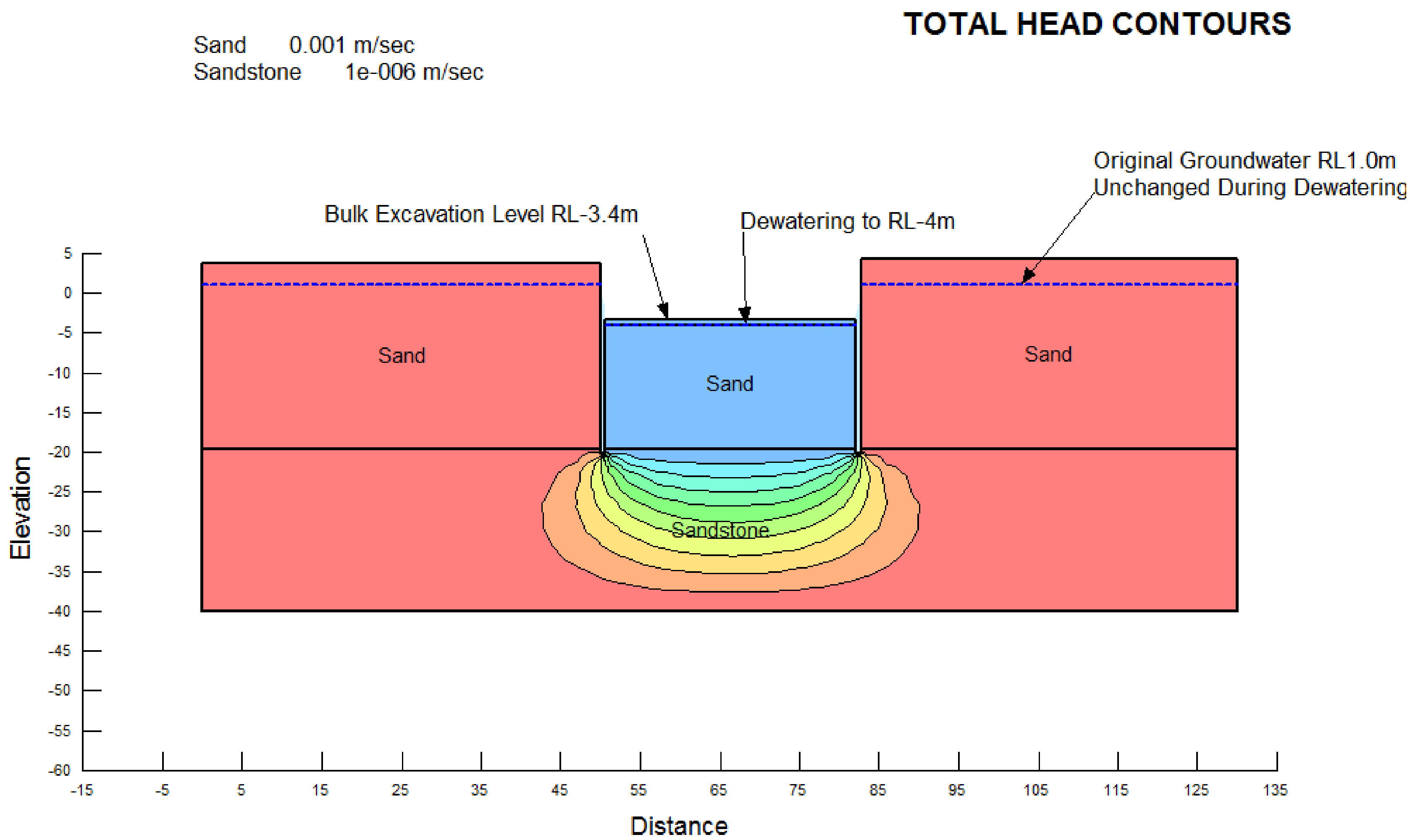
Report No. 21496LB **Figure No. 11**



GROUNDWATER SEEPAGE ANALYSIS TOE LEVEL RL-17.5m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

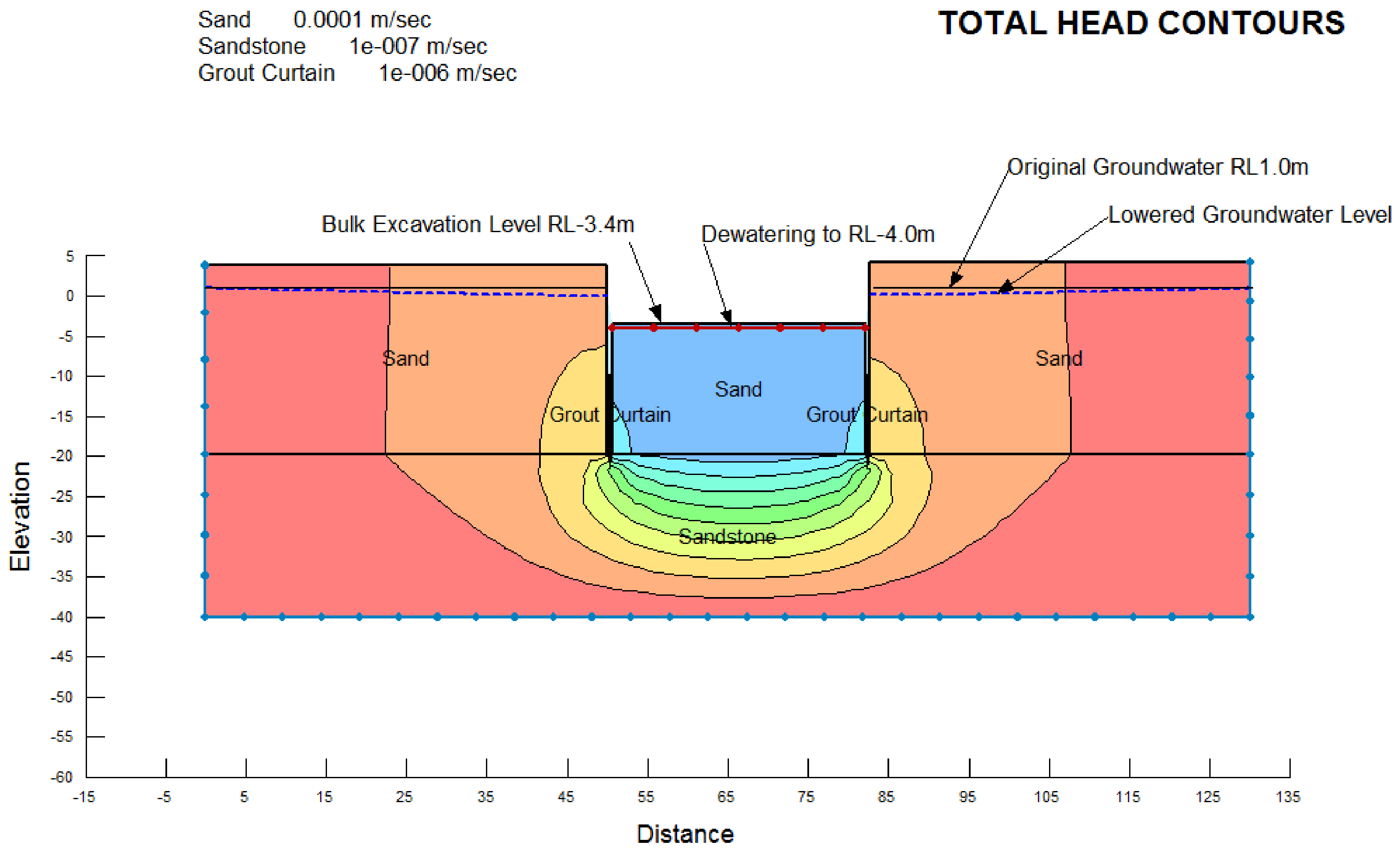
Report No. 21496LB **Figure No. 12**



GROUNDWATER SEEPAGE ANALYSIS TOE LEVEL RL-20.2m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

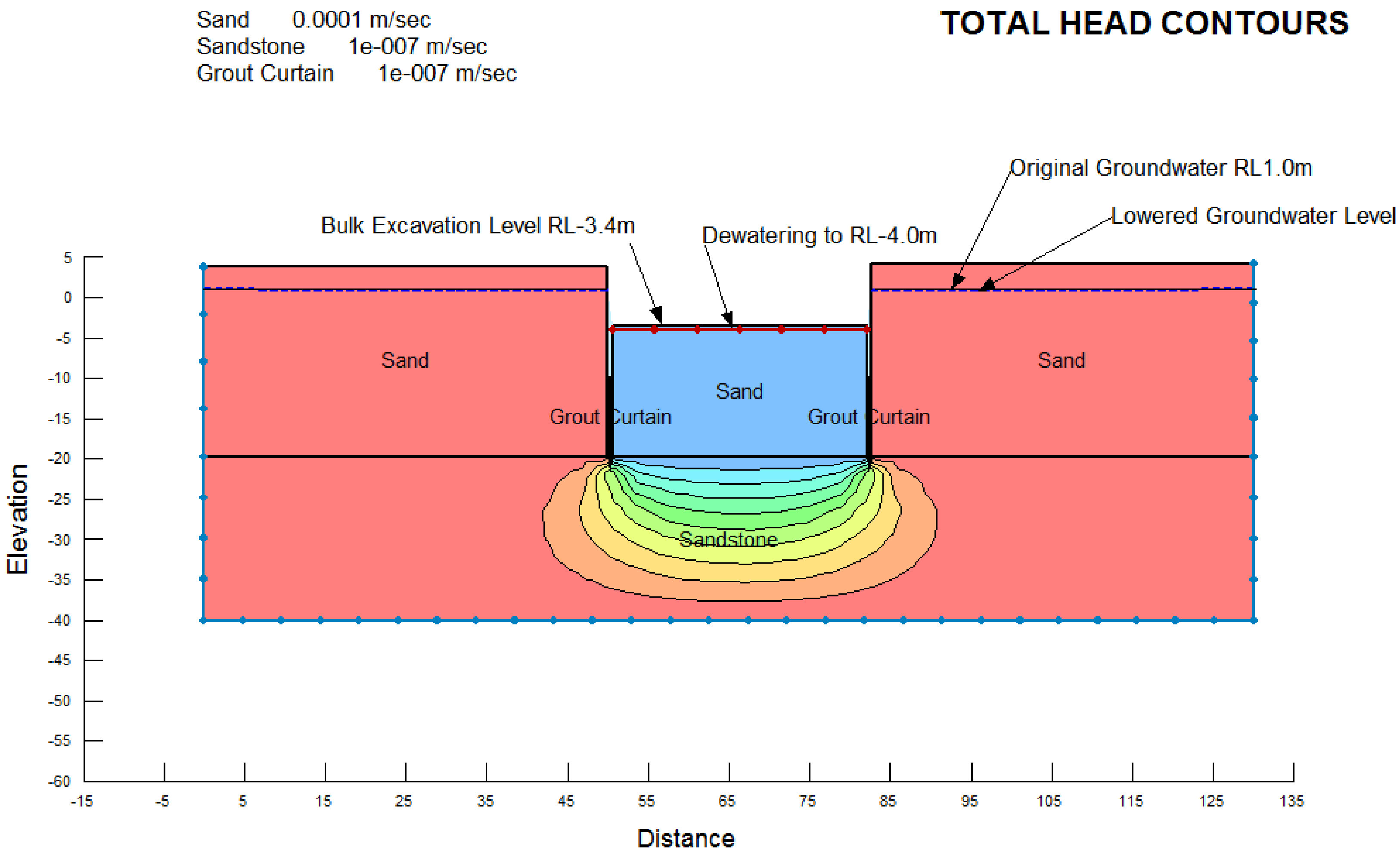
Report No. 21496LB **Figure No. 13**



GROUNDWATER SEEPAGE ANALYSIS TOE LEVEL RL-10m
WITH GROUT CURTAIN PERMEABILITY 1e-6m/sec

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 21496LB **Figure No. 14**



GROUNDWATER SEEPAGE ANALYSIS TOE LEVEL RL-10m WITH GROUT CURTAIN PERMEABILITY 1e-7m/sec

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 21496LB **Figure No. 15**

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

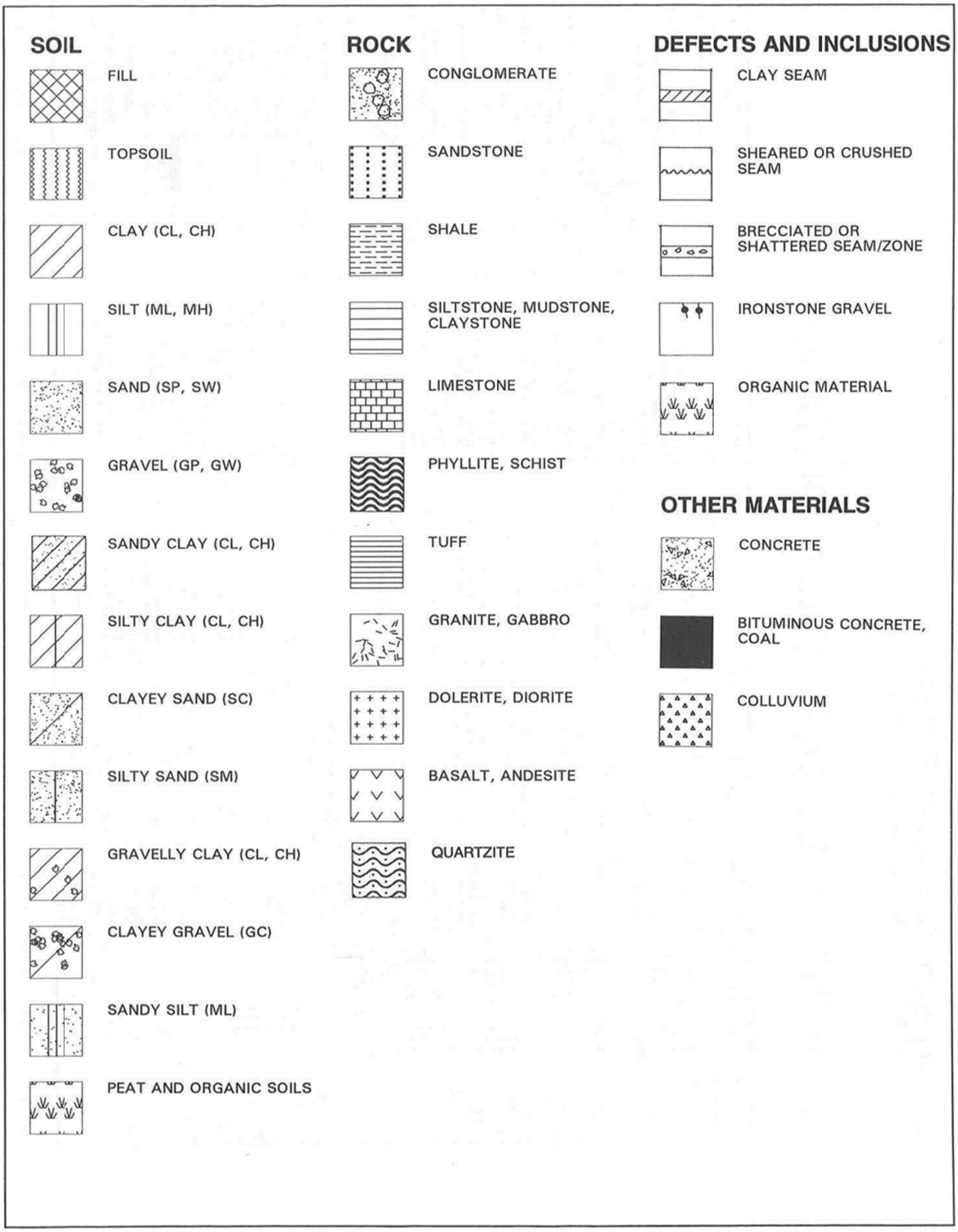
Requirements could range from:

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

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

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



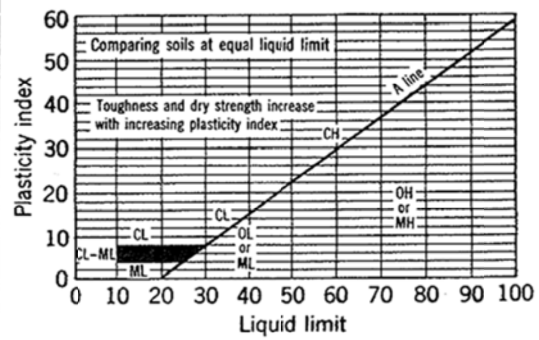
Major divisions			Field Identification Procedures (Excluding particles larger than 75 µm and basing fractions on estimated weights)	Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than 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	$C_U = \frac{D_{60}}{D_{10}}$ 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

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

Identification Procedures on Fraction Smaller than 380 µm Sieve Size

Major divisions		Dry Strength (crushing character-istics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Fine-grained soils More than half of material is *smaller* than 75 µm sieve size	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	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

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

LOG SYMBOLS

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

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

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

ROCK STRENGTH

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

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

ABBREVIATIONS USED IN DEFECT DESCRIPTION

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