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REPORT
ON
GEOTECHNICAL INVESTIGATION

PROPOSED DEVELOPMENT OF HAKOAH CLUB
61 - 67 HALL STREET, BONDI

Prepared for
HAKOAH CLUB

PROJECT 36387
OCTOBER 2003

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PRELIMINARY REPORT ON GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT OF HAKOAH CLUB
61 – 67 HALL STREET, BONDI

1. INTRODUCTION

This report presents the results of a preliminary geotechnical investigation at 61 - 67 Hall Street, Bondi for the proposed development of Hakoah Club. The investigation was commissioned by M+G Consulting Engineers Pty Ltd on behalf of the Club.

The proposed development will entail the following:

- Demolition of the existing Club building except for the walls of the two-level underground carpark.
- Excavation for and construction of an additional two basement levels.
- Construction a new 8-storey building (Club, retail and residential units)

As no information about the structural loads was available, it has been assumed that column loads will be in the order of 5500kN – 7500kN. The investigation was commissioned to provide preliminary information on:

- subsurface conditions for excavation and retention support for the proposed basements
- excavation techniques for the proposed additional two basement levels
- the allowable bearing capacity for footings.

The site was previously investigated by Groundtest Pty Ltd (now a subsidiary of the Douglas Partners Pty Ltd) in 1974 (report No 4371). The results of this earlier investigation are incorporated within this report.
The current investigation comprised two cone penetration tests (CPTs), one beyond either end of the proposed site. The test results are given in this report together with comments relating to design and construction practice.

Site survey plans were provided by Michael Lockley & Associates and architectural drawings were provided by JPR Architects Pty Ltd.

2. SITE DESCRIPTION AND GEOLOGY

The site is located within the floor of a broad valley that slopes towards Bondi Beach at between 1° and 3°. The site itself slopes toward the northeast at less than 3°, and is bounded by O'Brien Street to the north and Hall Street to the south.

The southern portion of the site is rectangular in shape with a triangular shaped portion abuting O'Brien Street, with a total area of about 2000 square metres.

At the time of the investigation, the site was occupied by a 5-storey concrete framed building with 2 basement levels. The existing lower basement floor slab is at approximately RL 14.0 AHD (about 7m below Hall Street level). The building generally appeared to be in good condition when inspected from the outside.

At the Hall Street end of the site there are three storey brick units to the west and east, located about 3 m from the common boundaries but abuting the Club building for the first 7m in from Hall Street on the west side. To the northwest of the site there is a single-storey brick house located between 1m and 2 metres from the common boundary.

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by aeolian sand overlying Triassic aged Hawkesbury Sandstone. The sandstone comprises medium to coarse grained quartz sandstone with minor siltstone lenses.

The cone tests indicated the presence deep sands with refusal possibly on weathered sandstone bedrock.

3. FIELD WORK

As the existing Club building covers the entire site, the investigation comprised two CPT tests taken to refusal at locations on Hall and O'Brien Streets adjacent to the subject site. The locations of the tests are given in Drawing 1.

The interpreted CPT graphical traces are attached. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

The approximate surface levels of the test locations, as shown on the test sheets, were estimated by interpolation between spot levels shown on the supplied architectural plans. The datum for the levels is the Australian Height Datum (AHD).

Assessment of possible contamination of the soils and groundwater was beyond the scope of the investigation.

Previous investigations in 1974 comprised the drilling of two bores (locations are shown on the attached Drawing 1) using an 85mm flight auger to depths of about 9.6m (31'6") from existing ground levels at the time. Bores incorporated Standard Penetration Tests (SPT) at regular intervals as shown on the attached bore logs.

4. FIELDWORK RESULTS

The subsurface conditions encountered comprised sand overlying a thin layer of silty clay which in turn overlies what was inferred as sandstone bedrock (CPT refusal). Sand deposits in CPT1 (O'Brien Street side) were assessed as being loose increasing to medium dense below 4.0 m depth. Medium dense and dense sand was encountered in CPT2 (Hall Street side) from about 0.5m depth. Refusal occurred at depths of 9.06 m (RL 9.64) and 11.8 m (RL 9.70) beneath

O'Brien and Hall Streets respectively. Although not positively confirmed it appears likely that refusal was on weathered sandstone. No groundwater was observed down to 8m at the end of testing CPT2. For further details of the strata encountered at each test location, reference should be made to the attached CPT traces.

SPT results from the 1974 drilling indicate sands of loose condition near the surface becoming medium dense below depths of 3 m to 4.5 m (10 ft to 15 ft). Groundwater was encountered in the two bores at depths of about 8.2m (27 ft) and 9.1 m (30 ft).

5. COMMENTS AND RECOMMENDATIONS

5.1 Pre- Construction Phase Geotechnical Investigation

Due to the limited scope of the current and previous geotechnical investigations and considering the complexity of the proposed development, it is considered essential that the following further investigations be carried out prior to making final decisions on the preferred method of construction and on the founding stratum and its corresponding design parameters:

- Obtain as-constructed information for the existing building foundation system. It is noted that it was originally recommended that the building be founded on pad footings below the basement designed for bearing pressures of 250 kPa (Reference Ground Test Report No 4371 of 1974). The technique of excavating an additional two basements will greatly depend on this information. Information on the foundations types and depths for the buildings on the adjoining sites will be required prior to making final recommendations on excavation methods and excavation support.
- Prior to excavation commencing, detailed dilapidation reports should be compiled for each of the adjoining buildings and their respective owners be asked to confirm that they represent a fair record of actual conditions. These reports should be carefully reviewed prior to excavation commencing to ensure that appropriate equipment and techniques are used.

- At least three additional bores (with associated water pressure testing) should be drilled at least 3m into the underlying medium strength rock, to confirm the depth to good quality bedrock, to assess rock strength and to assess rock mass permeability.
- Current information on groundwater needs to be obtained by the installation and monitoring of groundwater standpipes around the perimeter of the excavation, prior to the start of excavation. Information on groundwater is required to assess the magnitude of hydrostatic uplift pressure on the lower basement slab once the building's dead weight is removed and to evaluate the likely rate of ground water seepage into the excavation and the dewatering needs during and following construction; it is understood that it is preferred that a fully tanked, watertight basement be avoided. This will only be possible if pumping can accommodate the rate of seepage inflow to the basement over the long-term following hydraulic conductivity tests.

5.2 Excavation and Retention

Excavation recommendations provided in this report should be complemented by reference to the Code of Practice of Excavation Work, Cat. No. 312, by Work Cover NSW. dated 31 March 2000.

5.2.1 Geotechnical Issues to Consider

The following issues need to be considered carefully prior to proceeding with excavation as some of these issues are critical and could significantly affect the construction methodology or possibly even negate the feasibility of the proposed development.

- To maintain the existing basement walls and to allow retention works for the proposed new basements to be carried out through the existing floor of Basement 2 will require demolition of the ground floor and Basement slabs. The existing basement walls will need to be progressively anchored or internally propped as the floor slab demolition proceeds. Where anchors are to run below adjoining properties, permission of the owners should be obtained before installation.
- If the groundwater level is above the level of the Basement 2 floor slab, dewatering will need to be carried out to draw the groundwater down to below the slab level.

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5.2.2 Excavation Support

- **Retention Systems**

To enable excavation beneath the existing basement floor a watertight retention system will need to be installed beneath these existing walls prior to proceeding with excavation. The amount of rock toe-in for the retention system will greatly depend on the finding of the geotechnical investigations outlined in Sec. 5.1.

The aim of the proposed retention system is to provide sufficient lateral support for the sands and to hydraulically isolate the excavation from the aquifer. A hard-soft secant pile wall using continuous flight auger (CFA) piles installed from the basement floor slab, hard up against the existing external basement walls and keyed into competent (Class III or better) sandstone is probably the only feasible option.

The secant piles should be adequately socketed into the sandstone bedrock in order to provide the required passive lateral toe restraint and to provide adequate cut-off to groundwater inflow. Other lateral restraints in the form of anchors or internal props will be required and must be installed progressively as the excavation proceeds (ie. the lateral restraints must be installed once the restraining point has been uncovered). Alternatively, if the floor slab above the retention system is poured and restrains the pile heads prior to excavation, then a top down construction solution could be used. Details of the structural connection between the old basement wall and the new secant wall will need to be carefully thought through.

Construction of the secant pile walls should be of high quality, taking care of pile verticality and location to prevent soil loss and water inflow through gaps. Any gaps should be rectified progressively during the excavation, such as by mass concrete infill or shotcrete. The hard piles of the secant wall can be used as load bearing piles for the proposed building if designed for that purpose

High pressure jet grouting is not considered suitable as the jet-grout system does not enable sufficient wall cutoff into bedrock.

Conventional driven sheet pile walls will also not be suitable due to excessive vibration and because they cannot be driven sufficiently into the underlying competent bedrock.

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- **Retention Design Parameters**

The retention systems will need to be designed to limit deformation outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of temporary or permanent retention systems:

1. All retaining walls should be uniformly founded on the sandstone bedrock. Reference should be made to Section 5.3 for bearing pressure recommendations.

2. Based on the supplied architectural drawings, progressively anchored or propped walls will probably be required for the entire proposed basements where neighbouring buildings and streets either abut or are in close proximity to the excavation. As these areas will be sensitive to lateral movement of the retention system, progressively anchored or propped walls should be designed using a trapezoidal lateral earth pressure distribution of magnitude **7H** kPa (where H equals the height of the soil and extremely weathered rock excavation in metres). The full **7H** pressure should be applicable over the central 50% height of the trapezoidal distribution, reducing to zero at the upper and lower surfaces of the wall. A bulk unit weight of $20kN/m^3$ should be adopted for the soil and extremely weathered sandstone, above the water table with a buoyant density plus hydrostatic pressure used below the water table.

3. For retention, piles should be socketed into Class III or better quality sandstone below bulk excavation level and below nearby footings and service trenches. The upper 0.3m depth of the socket should not be taken into account to allow for disturbance and tolerance effects during excavation. For Class III sandstone an allowable lateral toe resistance of 3000kPa may be adopted.

4. Any surcharge (including nearby footings, traffic, etc.) affecting the walls should be allowed for in the design.

5. The basement retaining walls should be designed to withstand full hydrostatic pressures.

- **Anchors**

Sand or rock anchors with bond lengths in medium dense sands or low strength sandstone may be designed for an allowable bond stress of 75kPa and 350kPa respectively. Increased bond strengths may be achieved in the sand by pressure grouting. Rock anchors bonded in medium strength sandstone may be designed for an allowable bond stress of 750kPa. All anchors should be proof tested to 1.3 times the working load.

5.2.3 Excavation Methods

For the proposed new basement levels, it is expected that bulk excavation will extend through medium dense sand into sandstone bedrock (inferred from the CPT refusal but to be confirmed by carrying out extra bores involving rock coring as stated earlier in the report). Excavation of the sand profile and extremely weathered sandstone can be readily carried out using hydraulic excavators and bulldozers.

Low and medium strength rock will require rock hammers or ripping. Depending on rock levels and rock strength, hydraulic rock hammers or rock saws may be required for trimming of the sides of the excavation and for detailed excavations such as for footings and service trenches in the bedrock.

5.2.4 Excavation Related Ground Movements

It is likely that the excavation will induce some movements of the adjacent ground within the area of influence of the excavation. In sand, precedent suggests that for propped or anchored walls which are designed on the basis of a uniform lateral earth pressure of 7H, lateral and adjacent vertical movements would probably be less than 0.1% of the excavation depth. Additional movements may occur if the internal dewatering induces a drawdown in groundwater levels outside the excavation.

The actual retention wall movements are highly dependent of the construction sequence, detailing and quality of installation, and should be closely monitored in critical areas. It is recommended that settlement monitoring surveys of the neighbouring buildings and boundary walls be carried out weekly during the period of basement construction. Settlement effects may extend a horizontal distance from the excavation perimeter equal to at least 2 times the excavation depth.

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5.2.5 Vibration

Vibration and noise associated with rock excavation, particularly the use of rock hammers may result in complaints from neighbours and restrictions to equipment operating on site. The recommended maximum peak particle velocity from AS 2187 Explosives Code for various structures subject to vibration is 10 mm/sec for houses and low rise residential buildings and 25 mm/sec for commercial and industrial buildings or structures of reinforced concrete or steel construction. However, previous experience by Douglas Partners Pty Ltd has indicated that settlement of loose sands may occur at lesser vibration levels. It is therefore recommended that peak particle velocity of no more than 5 mm/sec at foundation level of adjacent residential structures be employed at this site for both structural and human comfort considerations.

Vibration monitoring carried out by Douglas Partners at various excavation sites in Hawkesbury Sandstone around Sydney has indicated the following relationships (Table 1) of peak particle velocity versus distance for various hammer types, milling heads and rock saw attachments.

Table 1 - Measured Relationship Between Hammer Weight/Rock Saw/Milling Head and Distance from Monitor

Hammer Type	Distance (m)	Peak Particle Velocity (mm/s)
Krupp 300 or equivalent	1.0 1.5 2.0	15 10 7
Krupp 600 or equivalent	3.5 6 9	10 4 – 5 2
Krupp 900 or equivalent	6 9 20	20 10 – 11 4
3m diameter saw on excavator	0.5 - 1 1 – 2	5 3
Milling head of excavator	1 1 - 2	5 3

Neighbours will probably find vibration levels above about 3 mm/s as being strongly perceptible to disturbing. Hence complaints from neighbours are possible and some reassurance, possibly by vibration monitoring, may be necessary.

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Table 2 may be used for initial plant selection, however monitoring may be necessary to establish site-specific relationships if rock breaking equipment is used for extensive excavation close to the boundaries.

To minimise the potential for damage of property or annoyance of residents, the excavation process should include:

- notification of neighbouring residents and occupiers of the proposed timing of the excavation so that any vibration sensitive items can be secured.
- excavation of loose or rippable sandstone blocks by bucket or single tyne attachments prior to commencement of rock hammering.
- progressive breakage from open excavated faces.
- selective breakage along open joints where these are present.
- use of rock hammers in short bursts to prevent generation of resonant frequencies.
- orientation of the rock hammer pick away from property boundaries and into the existing open excavation.
- the movement of large blocks away from the structures prior to breaking up for transport from site.
- the use of rock sawing or grinding equipment at and adjacent to the site boundaries.

5.2.6 Seepage

Groundwater inflow into the basement excavation must be expected and will need to be controlled as most of the proposed excavation will be located below the water table.

Provided that the secant wall has been well installed there should be little lateral seepage into the excavations but there will be some vertical seepage along joints and bedding planes in the underlying rock.

Both construction and long term seepage flows into the basement are expected to be controllable by conventional sump and pump dewatering methods with a dewatering sump kept at a low point during excavation.

The structure will require appropriate waterproofing for permanent walls in contact with the excavated areas. For the long term the sump(s) should have an automatic level controlled pump to avoid flooding of the basement level with an under slab drainage system. Outlets into the stormwater system will require Council approval. Note, it is likely that iron oxide/hydroxide "sludge" will be precipitated from the groundwater. This should be allowed for in the design of the permanent drainage system. Alternatively, the other solution would be to tank the basement and design the basement floor slab against full hydrostatic uplift pressures.

5.3 Foundations

The CPT refusal depths were inferred to represent the top of the bedrock. If this is the case, then sandstone bedrock will be exposed over most of the base of the proposed basement excavation. Therefore conventional high-level footings such as pad and strip foundations will be the most feasible founding system for this building.

It is recommended the entire building be uniformly supported on sandstone bedrock. Where sandstone bedrock is not exposed within the basement excavation, a deep footing system such as augered grout-injected (CFA) piles could to be adopted.

Pile socketed at least 0.3m into low strength sandstone may be designed for a maximum allowable end bearing pressure of 1500kPa. Footings (pads or piles) founded at least 0.3 m into sandstone of medium strength (Class III) sandstone may be designed for 3500 kPa. The above parameters need to be proved by conducting further bores involving rock coring as recommended earlier.

The initial stages of any CFA pile drilling should be witnessed by a geotechnical engineer to confirm that a satisfactory bearing stratum has been achieved.

5.4 Basement & Floor Slabs

It is recommended that the following subgrade preparation be carried out for construction of the basement floor slab. If the exposed subgrade is sand, it should be compacted with at least ten passes of a non-vibratory roller of at least 2 tonnes deadweight. The vibration mode on the roller should not be used as this could lead to "pumping" of groundwater to the surface and

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result in "bogging" of the roller. For a slab on sandstone subgrade the subgrade should be over cut by at least 100 mm. For a drained basement floor slab, the slab should be constructed independent of the building footings and walls (ie. designed as a "floating slab") supported on at least a 100mm thick sub-base layer comprising fine crushed rock material such as RTA Specification 3051 unbound base or equivalent quality, and compacted to a minimum density of 100% of Standard Maximum Dry Density (SMDD).

A sump and pump dewatering system will be required for any surface water run-off into the basement (see 5.2.6).

Slabs founded on a combination of sand and sandstone rock subgrade should be provided with joints at, or close to, the change in founding conditions. If this is not possible, then additional reinforcement should be provided to the slabs to cater for the differential settlement.

5.5 Summary of Additional Geotechnical Input

The previously recommended additional works are summarised below:

1. Obtain information on the footing conditions for the existing Club building and the adjoining buildings.
2. At least six groundwater monitoring standpipes should be installed around the building perimeter.
3. At least three bores should be drilled from the existing basement floor to prove the depth to, strength and permeability of the underlying sandstone.
4. Settlement monitoring of the neighbouring building and boundary walls should be carried out starting prior to demolition of the internal basement floors.
5. Dilapidation surveys should be carried out on the neighbouring buildings.
6. Vibration monitoring during demolition and excavation.
7. All temporary anchors should be proof stressed.
8. The initial stages of augered grout-injected pile drilling (if employed) should be witnessed by an experienced geotechnical engineer.

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APPENDIX A
Notes relating to this Report
Results of Field Work

NOTES RELATING TO THIS REPORT

Introduction

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

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

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

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil 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 are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is 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.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining 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 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

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

  as 15, 30/40 mm.

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

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end 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 friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical 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 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- 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 in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\, c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is 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.

Bore Logs

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

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, 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. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or 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 a perched water table.

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 condition, 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 depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

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 than 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.

Site Inspection

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

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DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

DEGREE OF WEATHERING

Term	Symbol	Definition
Extremely Weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly Weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
Moderately Weathered	MW	Rock substance affected by weathering to the extent that staining or discolouration of the rock substance usually by limonite has taken place. The colour of the fresh rock is no longer recognisable.
Slightly Weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh Stained	Fs	Rock substance unaffected by weathering, but showing limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index ($I_{s(50)}$) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by Australian Standard 4133.4.1 - 1993.

Term	Symbol	Field Guide*	Point Load Index $I_{s(50)}$ MPa	Approx Unconfined Compressive Strength q_u ** MPa
Extremely low	EL	Easily remoulded by hand to a material with soil properties	<0.03	< 0.6
Very low	VL	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger pressure.	0.03-0.1	0.6-2
Low	L	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150 mm long 40 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.	0.1-0.3	2-6
Medium	M	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.	0.3-1.0	6-20
High	H	Can be slightly scratched with a knife. A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow, rock rings under hammer.	1 - 3	20-60
Very high	VH	Cannot be scratched with a knife. Hand specimen breaks with pick after more than one blow, rock rings under hammer.	3 - 10	60-200
Extremely high	EH	Specimen requires many blows with geological pick to break through intact material, rock rings under hammer.	>10	> 200

Note that these terms refer to strength of rock material and not to the strength of the rock mass, which may be considerably weaker due to rock defects.

* The field guide assessment of rock strength may be used for preliminary assessment or when point load testing is not able to be done.

** The approximate unconfined compressive strength (q_u) shown in the table is based on an assumed ratio to the point load index of 20:1. This ratio may vary widely.

STRATIFICATION SPACING

Term	Separation of Stratification Planes
Thinly laminated	<6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	>2 m

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks. The orientation of rock defects is measured as an angle relative to a plane perpendicular to the core axis. Note that where possible, recordings of the actual defect spacing or range of spacings is preferred to the general terms given below.

Term	Description
Fragmented	The core consists mainly of fragments with dimensions less than 20 mm.
Highly Fractured	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured	Core lengths are mainly 40 mm - 200 mm with occasional shorter and longer sections.
Slightly Fractured	Core lengths are generally 200 mm - 1000 mm with occasional shorter and longer sections.
Unbroken	The core does not contain any fracture.

ROCK QUALITY DESIGNATION (RQD)

This is defined as the ratio of sound (i.e. low strength or better) core in lengths of greater than 100 mm to the total length of the core, expressed in percent. If the core is broken by handling or by the drilling process (i.e. the fracture surfaces are fresh, irregular breaks rather than joint surfaces) the fresh broken pieces are fitted together and counted as one piece.

SEDIMENTARY ROCK TYPES

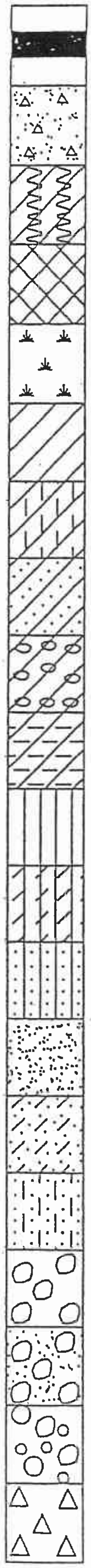
This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

Rock Type	Definition
Conglomerate	More than 50% of the rock consists of gravel-sized (greater than 2 mm) fragments
Sandstone:	More than 50% of the rock consists of sand-sized (0.06 to 2 mm) grains
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06 mm) granular particles and the rock is not laminated.
Claystone:	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated.
Shale:	More than 50% of the rock consists of silt or clay-sized particles and the rock is laminated.

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, eg. clayey sandstone, sandy shale.

GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL

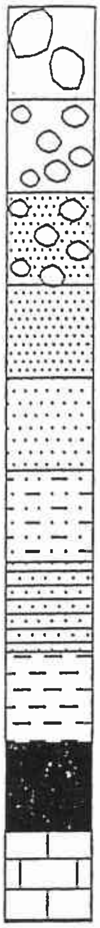


BITUMINOUS CONCRETE
CONCRETE
TOPSOIL
FILLING
PEAT
CLAY
SILTY CLAY
SANDY CLAY
GRAVELLY CLAY
SHALY CLAY
SILT
CLAYEY SILT
SANDY SILT
SAND
CLAYEY SAND
SILTY SAND
GRAVEL
SANDY GRAVEL
COBBLES/BOULDERS
TALUS

SEAMS

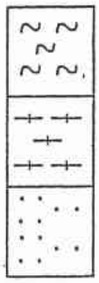
SEAM >10mm
SEAM <10mm

SEDIMENTARY ROCK



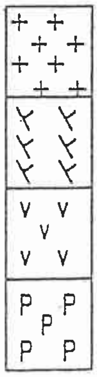
BOULDER CONGLOMERATE
CONGLOMERATE
CONGLOMERATIC SANDSTONE
SANDSTONE FINE GRAINED
SANDSTONE COARSE GRAINED
SILTSTONE
LAMINITE
MUDSTONE, CLAYSTONE, SHALE
COAL
LIMESTONE

METAMORPHIC ROCK



SLATE, PHYLLITE, SCHIST
GNEISS
QUARTZITE

IGNEOUS ROCK



GRANITE
DOLERITE, BASALT
TUFF
PORPHYRY

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CONE PENETRATION TEST

CLIENT: HAKOAH CLUB C/- M & G CONSULTING

PROJECT: HAKOAH CLUB

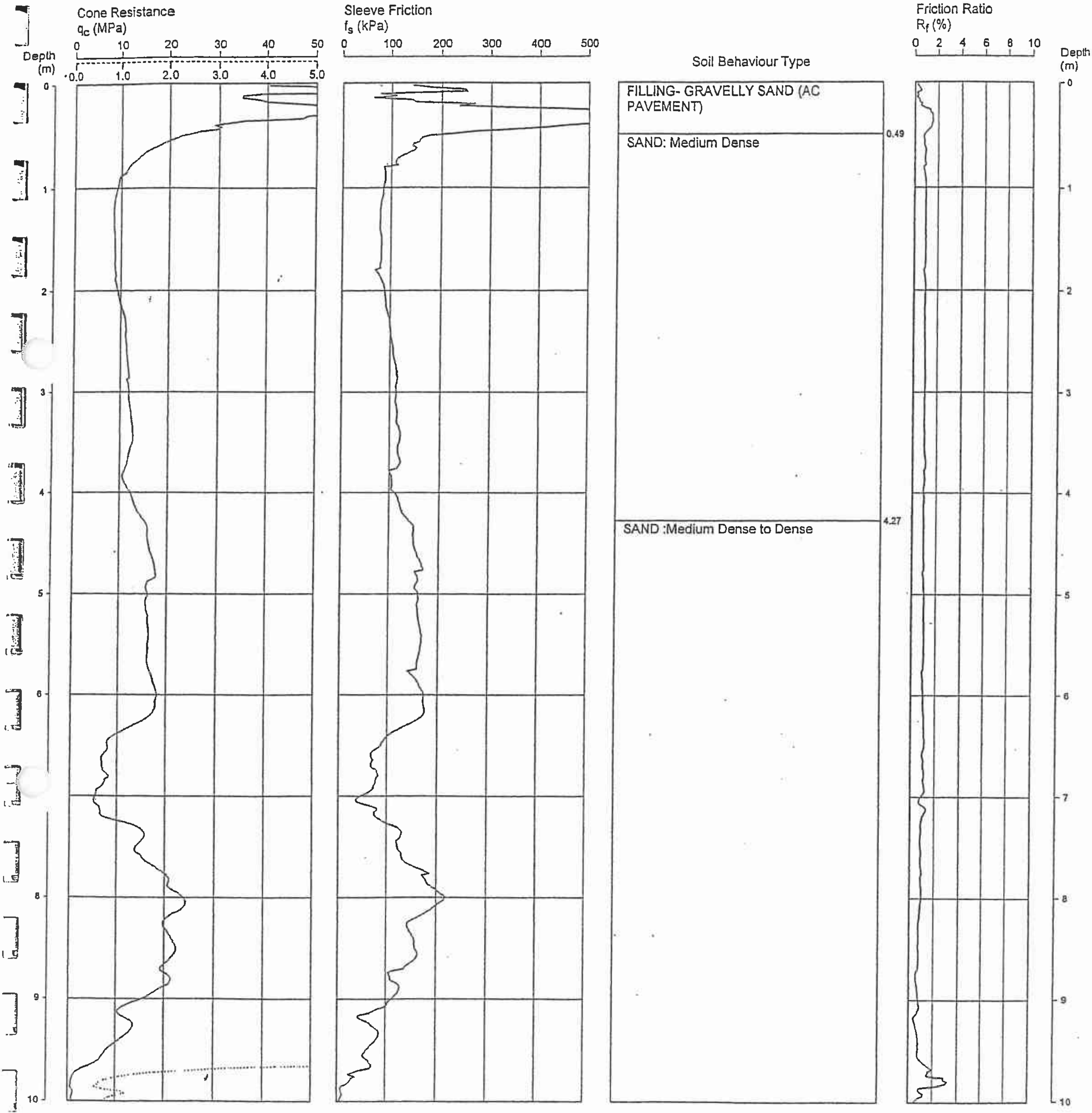
LOCATION: 61-67 HALL STREET, BONDI BEACH

PROJECT No: 36387

CPT 2

DATE 26 September 03

SURFACE RL: 21.5 m AHD



REMARKS: NO GROUNDWATER OBSERVED AT END OF TEST TO 8.0 m

Date	30/10/03
Plotted	YN
Checked	JCB

File: C:\Program Files\ConePlot\36387 CPT2.CP5
Cone ID: CONE-401 Type: 2 Standard

ConePlot Version 5.8.0

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CONE PENETRATION TEST

CLIENT: HAKOAH CLUB C/- M & G CONSULTING

PROJECT: HAKOAH CLUB

LOCATION: 61-67 HALL STREET, BONDI BEACH

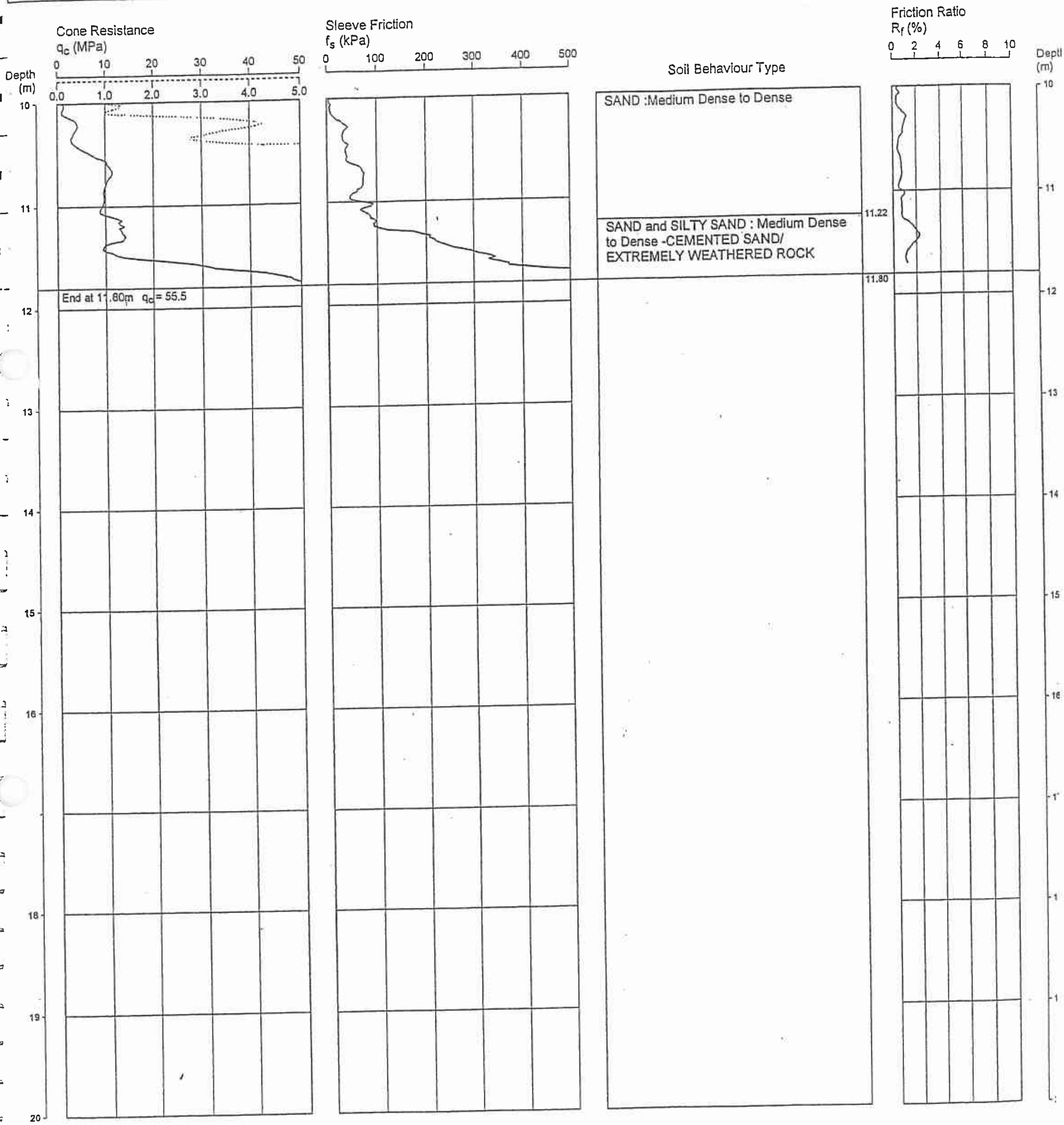
PROJECT No: 36387

CPT 2

Page 2 of 2

DATE 26 September 03

SURFACE RL: 21.5 m AHD



REMARKS: NO GROUNDWATER OBSERVED AT END OF TEST TO 8.0 m

Date 30/10/03
Plotted YN
Checked JCB

File: C:\Program Files\ConePlot\36387 CPT2.CP5
Cone ID: CONE-401 Type: 2 Standard

ConePlot Version 5.8.0

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CONE PENETRATION TEST

CLIENT: HAKOAH CLUB C/- M & G CONSULTING

PROJECT: HAKOAH CLUB

LOCATION: 61-67 HALL STREET, BONDI

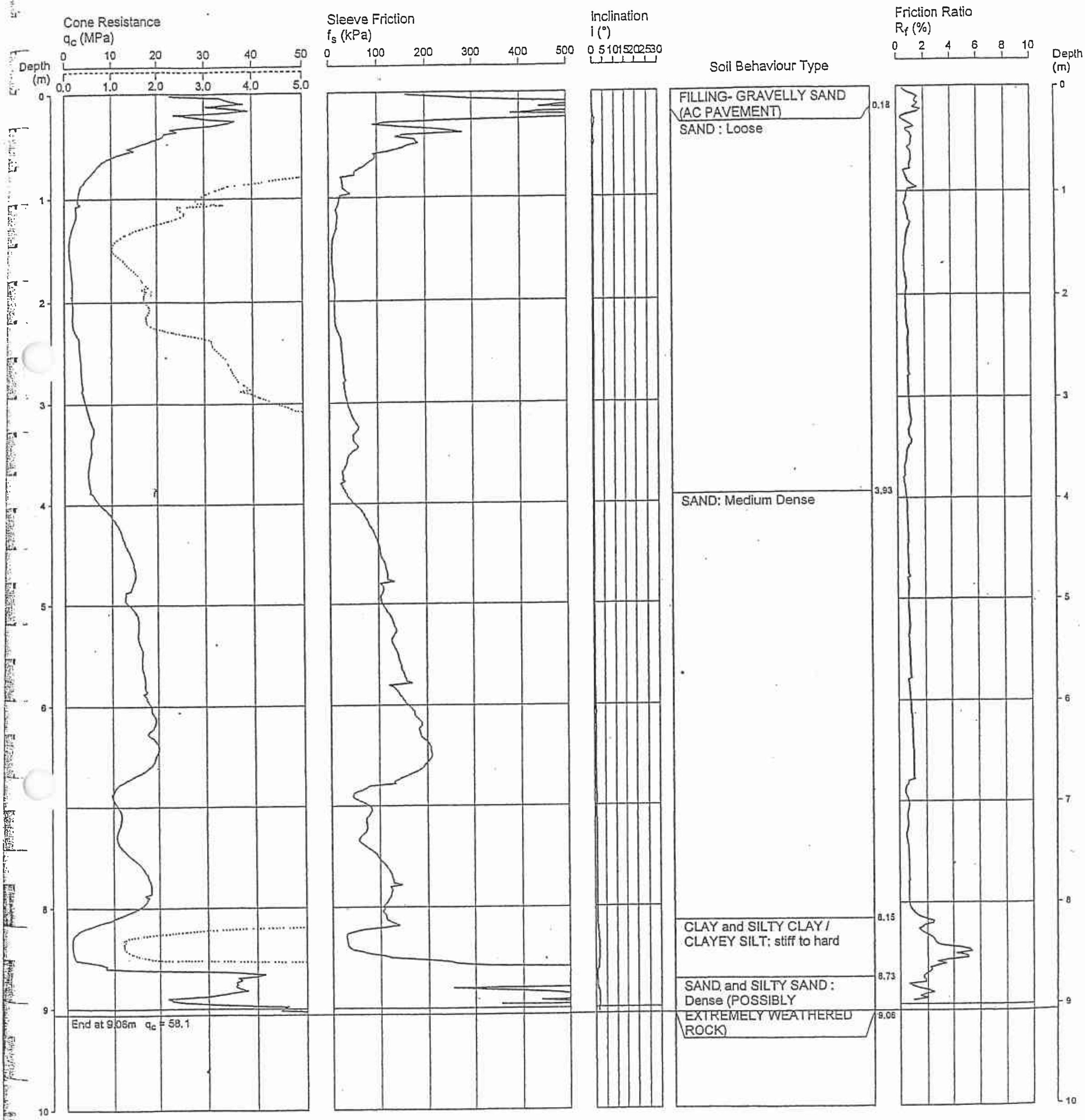
PROJECT No: 36387

CPT 1

Page 1 of 1

DATE 26/09/03

SURFACE RL: 18.70 m AHD



REMARKS: GROUNDWATER LEVEL AT COMPLETION OF TEST: m

Date	30/10/03
Plotted	YN
Checked	JCB

File: C:\Program Files\ConePlot\36387 CPT1.CP5
Cone ID: CONE-401 Type: 2 Standard

ConePlot Version 5.8.0

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TEST BORE REPORT

BORE No. 1

CLIENT MATEFFY-PERL-NAGY
SITE HAKOAH CLUB
LOCATION 61-67 HALL STREET, BONDI

DATE 25th January, 1974
CONTRACT No. SSI/1-4371
SURFACE LEVEL

Description of Strata	Depth	Sampling and in-situ Testing			
		Type	Depth	'N' value	Core recovery %
	S.L.				
SAND - Brown sand, slightly peaty between 1'6" - 2'0"					
	2'0"				
		S	3'0" - 4'6"	5	
SAND - Loose light yellow brown medium grained sand		S	6'0" - 7'6"	7	
	9'0"				
		S	10'0" - 11'6"	14	
		S	14'0" - 15'6"	22	
		S	18'0" - 19'6"	16	
SAND - Medium dense light yellow brown medium grained sand with occasional very small clay balls ($^1/_8$" diameter) below 20'0"		S	22'0" - 23'6"	20	
		S	26'0" - 27'6"	20	
		S	30'0" - 31'6"	32	
	31'6"				

BORE DISCONTINUED @ 31'6"

RIG GEMCO
DRILLER SEEEMAN
CASING

TYPE OF BORING FLIGHT AUGER

WATER LEVEL OBSERVATIONS FREE WATER @ 27'0"

REMARKS

TYPE

A — auger sample

'N' VALUE

blows of a 140lb hammer falling 30" to

TEST BORE REPORT

BORE No. 2

CLIENT MATEFFY-PERL-NAGY
SITE HAKOAH CLUB
LOCATION 61-67 HALL STREET, BONDI

DATE 25th January, 1974
CONTRACT No. SSI/1-4371
SURFACE LEVEL

Description of Strata	Depth	Sampling and in-situ Testing			
		Type	Depth	'N' value	Core recovery %
	S.L.				
FILLING - Bricks and sand					
	1'0"				
SAND - Brown sand		S	3'0" - 4'6"	2	
	3'6"				
SAND - Loose light grey fine to medium grained sand		S	6'0" - 7'6"	6	
	7'3"				
PEATY SAND - Loose brown slightly peaty sand					
	8'0"				
SAND - Medium dense light yellow brown and light brown medium gravel sand with some small lumps of carbonaceous material		S	10'0" - 11'6"	8	
		S	14'0" - 15'6"	15	
		S	18'0" - 19'6"	18	
		S	22'0" - 23'6"	18	
		S	26'0" - 27'6"	22	
		S	30'0" - 31'6"	17	
	31'6"				
BORE DISCONTINUED @ 31'6"					

RIG GEMCO
DRILLER SLEEMAN
CASING

TYPE OF BORING FLIGHT AUGER

WATER LEVEL OBSERVATIONS FREE WATER @ 30'0"

REMARKS

TYPE
A — auger sample
S — standard penetration test

'N' VALUE
blows of a 140lb hammer falling 30" to drive a standard 2"O.D. split pentrometer

APPENDIX B
Drawing 1 - Test Locations