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SUPPLEMENTARY REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED REDEVELOPMENT OF THE FORMER RACHEL FORSTER HOSPITAL SITE
REDFERN

Prepared for
REDFERN WATERLOO AUTHORITY

Project 44742 Revision 1
May 2007

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ACEA
The Association of Consulting Engineers Australia

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MJT:jlb
Project 44742 Revision 1
18 May, 2007

SUPPLEMENTARY REPORT ON GEOTECHNICAL INVESTIGATION
FORMER RACHEL FORSTER HOSPITAL REDEVELOPMENT
134 – 150 PITT STREET, REDFERN

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1. INTRODUCTION

This supplementary report details the results of a geotechnical investigation carried out for the proposed redevelopment of the former Rachel Forster Hospital site in Pitt Street, Redfern. The work was requested by the Redfern Waterloo Authority.

It is anticipated that two of the existing buildings on the site will be demolished to enable construction of three 3 – 4 storey structures with basement car parking. Further, it is understood that the building near the southern boundary will be refurbished and have one extra floor added. The geotechnical assessment of the site was undertaken to provide advice on appropriate design and construction measures for excavation, retaining wall foundations and pavements.

The geotechnical assessment has been based upon data obtained during a geotechnical investigation of the site in November 2003 for a proposed community centre. No additional field testing has been performed as part of this assessment. The geotechnical appraisal has simply involved assessing the requirements for construction of the development which is now proposed for the site. As part of the current geotechnical assessment, a visit was made to the site by a Principal Geotechnical Engineer on 4 April 2007. There were no discernible changes in the physical characteristics of the site since the previous assessment was conducted in November 2003.

The investigation undertaken in November 2003 comprised test bores followed by laboratory testing. Details of the field and laboratory work done at that time are given in this report, together with comments relating to design and construction practice for the current redevelopment.

2. SITE DESCRIPTION

The site is a rectangular shaped allotment covering an area of about 6900 m^2 and is located on the corner of Albert and Pitt Streets, Redfern. At the time of the field investigation in November 2003 the site was occupied by the Rachel Forster Hospital comprising a five – six storey brick building (including a basement level) on the southern boundary, a two storey brick building on the centre of the site and a four storey building (including one basement level) on the northern (Albert St) boundary. The majority of buildings were vacant at the time of the field work with the exception of the northern wing of the building fronting Albert Street. The site was covered by either concrete or bitumen with minor landscaped areas. At the time of the current assessment the site conditions were essentially unchanged

The site is bounded by the following:

- North – Albert Street and residential units;
- South – residential units;
- West – commercial/industrial packing factory to the northwest and residential units to the southwest;
- East – Pitt Street and terrace houses used for commercial purposes (early childhood centre and offices).

Two disused wells covered with overgrown vegetation were noted in the south western corner of the site in November 2003. Anecdotal information suggests that there was also an additional well in the central portion of the hospital.

The general topography of the site slopes towards the south-west with a fall in level of approximately 3 m. The western portion of the site was at a lower level compared to Pitt and Albert Streets.

3. GEOLOGY

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by Quaternary sands, comprising medium to fine grained sand with some included podsols. In this area the sand deposits are typically underlain by Ashfield Shale which comprises siltstone, shale and laminite beds. Reference to previous investigations undertaken in the area indicates the presence of clays and shaly clays, which are probably weathered products of the Ashfield Shale.

Field work for the geotechnical investigation confirmed the presence of filling, sands, clays and siltstone.

4. FIELD WORK

4.1 Field Work Methods

The field work for the geotechnical investigation of November 2003 comprised ten bores drilled with a truck-mounted auger/rotary drilling rig to total depths ranging from 4.4 m to 11.55 m. The bores were augered through the soils to depths ranging from 2.7 m to 5.95 m with 110 mm diameter continuous spiral flight augers. Sampling and identification of subsurface strata were from the cuttings returned from the auger blade and standard penetration tests carried out at 1.5 m depth intervals in the soils. Four bores (Bores 1 to 4) were terminated on reaching underlying rock. Bores 5 to 10 were cored for a penetration of 4.6 m to 6.05 m into the rock using NMLC (50 mm diameter core) diamond drilling equipment.

The locations of the bores are given in Drawing 1 in Appendix A. The surface levels shown on the bore logs were determined by using a survey level on site and referenced to State Survey Mark SS 46150 (30 m AHD).

4.2 Field Work Results

The results of the test bores are given in detail in Appendix B, together with notes defining classification methods and descriptive terms. A geotechnical model of the site is presented in the form of three interpreted geotechnical cross sections (Sections A-A, B-B and C-C) in Drawings 2, 3 and 4 in Appendix A.

The bores encountered variable conditions over the site which can be briefly described as follows:

Concrete/Asphalt

A 140 – 180 mm thick concrete slab was encountered across the majority of the site. Asphalt surfacing up to 40 – 50 mm thick, was noted overlying the concrete slab in Bores 6, 8 and 10.

Filling

Fill materials comprising sand, clay, sandy clay, sandstone gravel, brick, coal, slag and ash were encountered in all bores to depths of generally between 0.3 m to 0.7 m. The filling layer was thicker in Bores 8 and 9 where the depths were 1.5 m and 1.9 m respectively.

Sand

Sand was encountered below the layer of filling to depths ranging from 1.4 m to 5.0 m in most of the bores, except Bores 5 and 9 where no sand was encountered. The sand was mostly loose and fine grained, but included some medium dense layers and some cemented bands. The depth of sand increased in a south westerly direction to a maximum of 5.0 m in Bore 1.

Clay

Clay and sandy clay was encountered beneath the sand layer to depths of 2.3 m to more than 6.0 m in all bores. The clay was generally stiff to hard and red brown and grey in colour.

The clay graded into shaly clay in the deeper bores.

Siltstone

Extremely low strength siltstone bedrock was first encountered in the deeper bores (Bores 5 to 10) below levels ranging from RL 22.9 to RL 26.9 with the strength of the rock generally increasing with depth. The thickness of the weathered siltstone bedrock varied significantly between bores. In Bore 6 the very low strength rock was about 0.8 m thick with low to medium strength rock below RL 22.3, while in Bore 7 the extremely low to low strength rock was about 5.4 m thick with medium strength rock intersected below RL 17.5.

The surface of the bedrock dips in a south westerly direction resulting in a progressively increasing depth of sand towards the south western corner of the site.

No free groundwater was encountered in any of the bores during the field work. The use of water as a drilling fluid during coring precluded the observation of groundwater in the deeper bores.

5. LABORATORY TESTING

A California bearing ratio (CBR) test was carried out on a clay sample retrieved from Bore 5 at a depth of 0.0 to 1.0 m. The laboratory test results are given in detail in Appendix C. The sample was compacted to 100% dry density ratio (standard compaction) at the estimated optimum moisture content. The sample was soaked for four days under a surcharge loading of 4.5 kg. There was 5.6% swell during soaking and the measured CBR was 2%. A test at the “bottom” of the sample (which may not be fully soaked) gave a CBR of 6%.

6. PROPOSED DEVELOPMENT

It is understood that the preferred design option will include four residential apartment buildings with public open space along Pitt Street and communal open space to the west.

The key elements are:

- The existing building adjacent to the southern boundary (referred to as Building 1) will be retained but with one extra floor added consisting of light weight steel construction;
- The existing buildings within the centre of the site will be demolished, with the exception of the existing colonnade fronting Pitt Street. A new three storey building (Building 2) will be erected behind this colonnade. Another three storey building (Building 4) will be erected between Building 2 and the western boundary;
- The existing building fronting Albert Street will be demolished and a new four storey building (including a lower basement level) will be erected in this location (Building 3);
- Car parking is proposed to be located at basement level underneath Buildings 2 and 4 and the areas separating these buildings. It is anticipated that two levels of basement car parking will be provided;
- Publicly accessible open space is to be provided between Pitt Street and Building 2. The remainder of the open space will be for the private use of development.

At this stage the concept design is tentative only and may need to be varied. Consequently, no design loads are available and the comments given below are of a generic nature only. When details of the development are finalised it will be necessary to obtain further advice to determine whether the comments are suitable for the specific development proposed.

7. COMMENTS

7.1 Excavation

At this stage a two level basement car park is proposed, probably resulting in excavation to about 6 m below the general site level. Relatively straightforward conditions are expected for the bulk excavation across the northern section of the site. Most of the cut material will comprise concrete slabs, filling, sand, clay and extremely low to low strength siltstone which can be readily removed using conventional earthmoving equipment. The base of the excavation may be in the low strength siltstone with some bands of low to medium strength siltstone. If these bands are encountered heavier bulldozers or rock hammers attached to excavators may be required.

Groundwater was not observed whilst augering the bores. However, it is expected that there will be some groundwater inflow along the sand/clay interface and through the siltstone. It is also expected that this flow will be readily removed by pumps located in sumps below the bulk excavation level. Groundwater inflow may also increase after periods of wet weather.

At this stage it is understood that no excavation is proposed in the southern portion of the site. However, if any excavation is to be undertaken in this section of the site it should be noted that groundwater inflow may be more significant here than in the northern portion of the site. Two disused wells are located in the south western corner and it is believed these wells sourced water from the thick sand layers present in this section of the site. Consequently any excavation in this area may experience groundwater inflow from the sand. Without further investigation it is difficult to predict the rate of any groundwater flow through the sand layers.

It should be noted that under the Protection of the Environment Operator Act (1997), the burden of proof that materials received by a waste fill site meet the environmental criteria for proposed land use rests on the waste/fill receiving site. This includes virgin excavated natural materials and filling, such as will be removed from this site. Reference should be made to the previous Contamination Assessment for this site which has been carried out by Douglas Partners Pty Ltd (dated October 2003, Reference No. 36299A).

7.2 Site Preparation

Site preparation (earth works) will need to be varied depending upon the proposed use of the area being prepared. The suggested site preparation measures are outlined below.

7.2.1 Proposed Pavement Areas

- Removal of the concrete slab and upper 1 m of filling and/or sand;
- Proof roll the exposed subgrade with a minimum of six passes of a 10 tonne deadweight vibrating roller. A further test rolling pass should then be carried out, without vibration, with visual inspection to allow detection and treatment of any compressible zones. If the subgrade at a depth of 1 m comprises loose sands then it would be advisable to place a 100 mm thick layer of road base prior to rolling;
- Replace the excavated material and compacted in layers of 300 mm (loose thickness), after removal of any large objects (greater than 300 mm long). Compaction should be to minimum 98% dry density ratio, standard compaction (AS 1289 5.1.1) for the lower 500 mm and 100% for the upper 500 mm.

7.2.2 Proposed Shallow Footings and Floor Slabs

The site preparation for areas where shallow foundations and/or floor slabs will be constructed is essentially the same as for pavement areas except that:

- All the filling and sand should be removed down to the surface of the stiff to hard clay;
- The excavated material should then be placed in 300 mm layers and compacted to 100% dry density ratio (AS1289.5.1.1).

If the work is undertaken after a period of heavy rainfall it is possible that the sandy materials will be wet and will therefore need to be dried before being used as bulk filling.

7.3 Safe Batter Slopes

There may be sufficient space to batter the sides of the basement excavation across some of the site during the construction stage. Slopes may also be required in landscaping areas as part of the final development. If this is the case it is suggested that the following safe batter slopes are used up to a maximum slope height of 4 m.

Table 1 - Safe Batter Slopes

Material Type	Safe Batter Slope (H:V)	
	Short term	**Long term**
Filling	2:1	2:1
Sand	1.5:1	2:1
Stiff to hard clay	1.5:1	2:1
Extremely low to very low strength siltstone	0.75:1	1:1
Low to medium strength siltstone	0.5:1	1:1

Flatter slopes will be required if they are to be vegetated with the maximum recommended to be 3H : 1V.

No groundwater was observed whilst augering during the field work. However, there may be some seepage, particularly at the soil to rock surface, and along the surface of the clayey soils. This may effect the surface stability of the slopes and it is therefore recommended that measures be taken to divert any runoff away from the excavation. In addition the stability of any proposed slope cut into the sand layer in the southern section of the site may be affected by groundwater seepage. It is recommended that a geotechnical engineer or engineering geologist inspects any slopes excavated across the site.

7.4 Excavation Support

Due to the depth of excavation (approximately 6 m) and the amount of space available, there may be insufficient space to batter the sides of the excavation as recommended in Section 7.3. In these areas retaining structures will be required, both during construction and as part of the final structure.

The following shoring options may be considered for the support of the excavation:

- Contiguous Pile Wall – Considered to be the most suitable option due to the presence of sand layers needing support. This type of wall consists of closely spaced, or touching, small diameter bored (or Continuous Flight Auger) piles. The wall may form part of the final structure, sealed by a shotcrete panel facing that is constructed as the bulk excavation progresses. A row of ground anchors tied into waling beams is likely to be required unless the piles are designed to cantilever from the base of the excavation.

- Soldier Pile/Infill Panel Wall System – Considered to be less suitable as there may be some problems supporting the sand layer. It consists of bored or CFA piles installed at 2 – 3 m centres in advance of excavation. As the excavation proceeds, structurally reinforced infill panels, or similar, are constructed in between the piles. The piles are often designed to also provide foundation support for the perimeter of the structure. Piles are normally drilled with a minimum "toe in" design to provide lateral restraint at the base of the excavation based on the passive resistance of the rock in which the pile is socketed. Again, a row of ground anchors tied into waling beams may be required unless the piles are designed to cantilever from the base of the excavation.

Anchors need only be of temporary construction (i.e. without specific corrosion protection) if permanent support is to be provided by the building itself.

It may be necessary to obtain permission from the local Council or adjacent residents for installing temporary anchors around the perimeter of the site as installation may encroach into their property. In addition, care should be taken to avoid damaging buried services or foundations of adjoining structures during anchor installation.

It is recommended that soldier piles, or other perimeter wall piles, are founded a minimum of two pile diameters below the bulk excavation level, except when they are required to carry structural loadings from the proposed building, in which case longer rock socket lengths may be required.

Drainage is normally provided behind soldier pile/infill panel wall systems using one of a number of proprietary strip drains combining a filter fabric and a cellular plastic matrix. A width of between 100mm and 300mm is usually adequate for strip drains, with one or two strips installed behind the face of each panel.

The shoring system may be required to support the footings of the existing five storey brick building which exists in the southern portion of the site and is to be retained as part of the final development. A soldier pile/infill panel wall system is not recommended in this section of the site due to the risk of a loss of material occurring beneath the existing foundations. However, a contiguous pile wall would reduce the risk of this situation occurring. In addition, if the excavation is to be immediately adjacent to the existing building then underpinning of the footings may be required. There is also a possibility that vibrations generated by the excavation equipment may cause cracking of this building.

As an alternative to a shoring system incorporating temporary or permanent tie-backs or bracing, a top-down construction method may be considered, where the internal basement floor slabs are used to permanently prop the perimeter wall.

7.5 Retaining Wall Design

It is suggested that the design of any cantilevered retaining structures (i.e. no ground anchors) be based on a triangular earth pressure distribution and an anchored retaining structure be based on a trapezoidal earth pressure distribution. An average bulk unit weight for the retained material of 20 kN/m^2, and the active earth pressure coefficients given in Table 2 are considered appropriate for both design methods.

Table 2 - Earth Pressure Coefficients

	Earth Pressure Coefficient (K)	
Retained Material	**Temporary Support**	**Permanent Support**
Filling and sand	0.35	0.35
Stiff to hard clay	0.25	0.3
Extremely low to low/medium strength siltstone	0.2	0.25

Unless basement drainage can be provided the wall should be designed for full hydrostatic head.

Additional pressures should be allowed for where the wall is adjacent to existing buildings and where construction traffic is expected to operate behind the wall.

Where appropriate, lateral restraint may also be developed by embedding the piles below the base of the excavation and developing passive pressure. The ultimate passive resistance available by embedding the piles into different materials and thus the required minimum "toe-in" can be estimated using the following values:

extremely low to very low strength siltstone	500 kPa
low strength siltstone	2000 kPa
medium strength siltstone	6000 kPa

The above values may be considered to be appropriate below one pile diameter beneath the bulk excavation level. It is noted that these values are "ultimate" values and will need to incorporate a factor of safety to limit wall movement.

7.6 Foundations

The type of foundations to be considered will depend on the type of structures to be constructed. However, regardless of the foundation type to be used, it is recommended that all footing and pile excavations be inspected by a geotechnical engineer.

7.6.1 Building 1

Building 1 is an existing five storey structure that is to be renovated and to have an additional floor added using a lightweight steel frame. This will result in an increase in the building foundation loads by an unknown amount but possibly as much as 15%. The increased load may cause some settlement of the existing structure, depending upon the type of foundations used to support the building. During the final design phase it will be necessary to determine the type of footings used beneath the existing structure and if they are shallow footings to investigate the strength and/or density of the supporting strata. This will enable an estimate to be made of the likely settlement due to the increased structural loads.

7.6.2 Buildings 2 and 4

After excavation of the basement to about RL 21.0, hard clay or extremely weathered rock will be exposed in the base. It is suggested that all footings be founded in the hard clay or better. This will result in shallow foundations below the whole of the basement car park.

The following allowable bearing capacities can be used in the design of the pad footings.

Table 3 - Allowable Bearing Capacities

Founding Material	Allowable Bearing Capacity (kPa)
Hard clay	400
Extremely low strength siltstone	700
Low to medium strength siltstone	1500

7.6.3 Building 3

Building 3 will be constructed on the northern boundary of the site where a lower basement is planned. Consequently, it is expected that shallow footings may be used below the basement level within either hard shaly clay or extremely low strength shale. Allowable bearing capacities for shallow footings supported by these materials would be the same as given in Table 3 above.

For more heavily loaded areas of the building it may be more economical for the foundation loads to be supported by bored piles taken down to siltstone. Allowable end bearing pressures of 1.5 MPa and 3.5 MPa can be used for design of piles founded in the low and medium strength siltstone respectively. Allowable shaft adhesions for bored piles socketed in these materials are 150 kPa and 350 kPa for the low and medium strength siltstone respectively.

At this stage only two bores have been drilled along the southern side of the proposed Building 3. In the final design phase when the existing structure has been demolished, it will be necessary to undertake additional geotechnical investigation along the Albert Street frontage to confirm the foundation advice given above.

7.7 Pavements

CBR values of 2% and 6% at the “top” and “bottom” of the clay sample retrieved from Bore 5 were recorded during the laboratory testing. Based on previous experience it is expected that the average CBR value for this material would be 3%. Therefore where residual clay soils form the pavement subgrade (e.g. surface car parks and roads or within access ramps), a design CBR of 3% can be used. However, most of the bores intersect sandy clay filling and sand near the surface. For these materials a CBR of 5% may be used.

Where the subgrade comprises reconditioned filling, clay or sand, as described in Section 7.2, a CBR value of 5% is acceptable for design.

It is recommended that all new roads be designed using the conservative CBR value of 3% unless the site can be delineated into areas where better subgrade will be present.

It is recommended that all pavements are protected by adequate surface and subsoil drains which are regularly maintained. The subgrade material should be compacted to a minimum 100% Standard Dry Density Ratio at a moisture content within 2% of the optimum moisture content for Standard Compaction.

8. ADDITIONAL INVESTIGATION

The investigation so far is sufficient for concept design but will need to be supplemented during the design phase to:

- Confirm the existing foundation system used beneath Building 1;
- Provide ground information along the Albert Street frontage.

The extra work should only be carried out after the concept design has been finalised and the existing structures demolished.

DOUGLAS PARTNERS PTY LTD

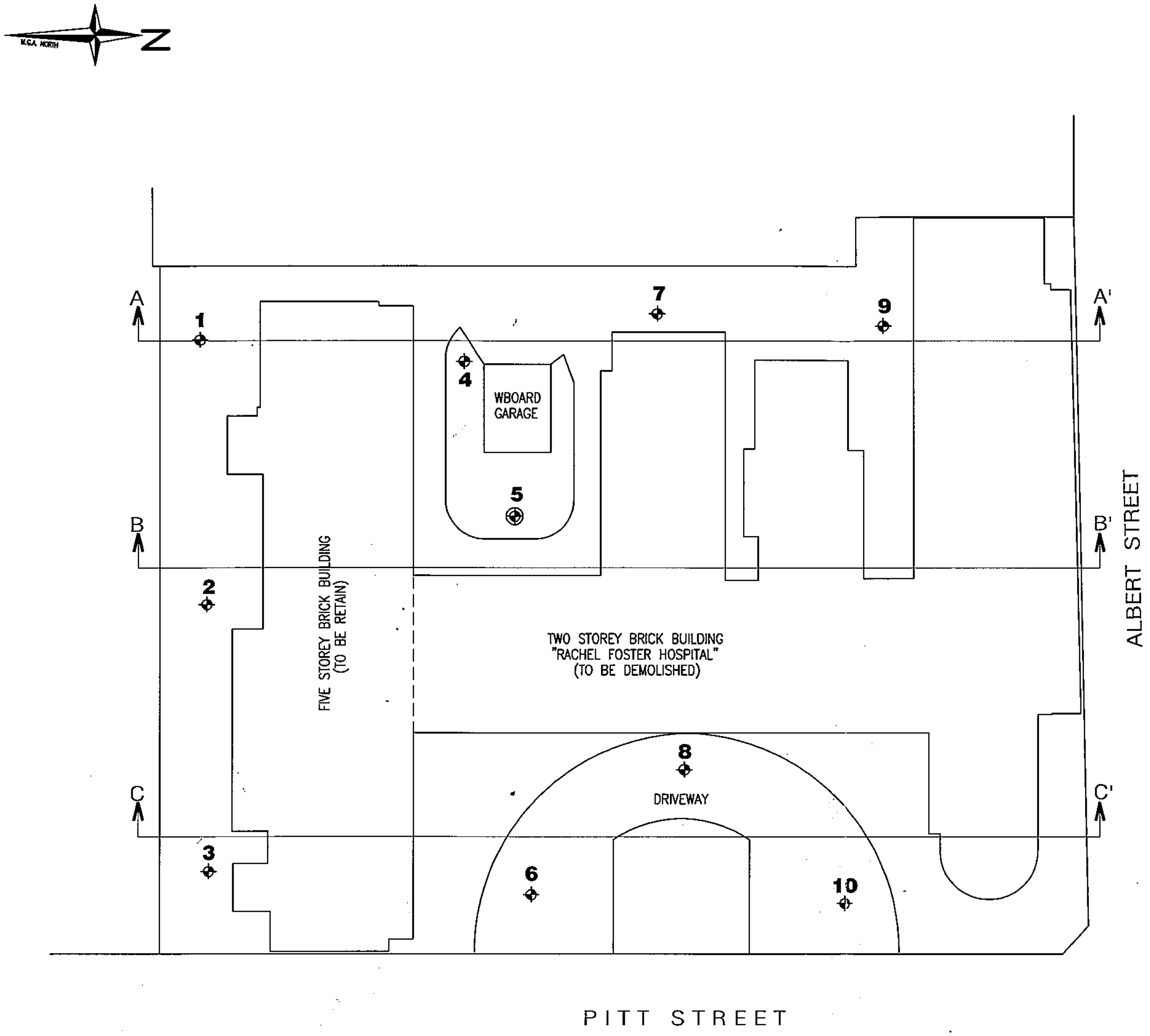
Reviewed by:

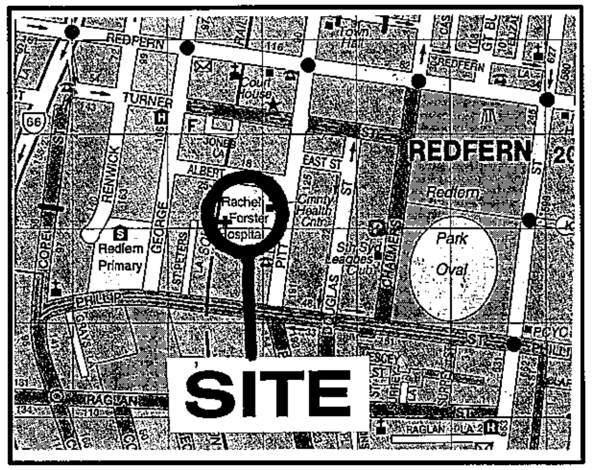
Michael J Thom
Principal

Fiona MacGregor
Principal

APPENDIX A

Site Plan and Geological Sections





LOCALITY PLAN

LEGEND

⊕ TEST BORE LOCATION

⊛ TEST BORE & CBR

SCALE 0 10 20 30 40 50m

1:500

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Geotechnics, Environment, Groundwater

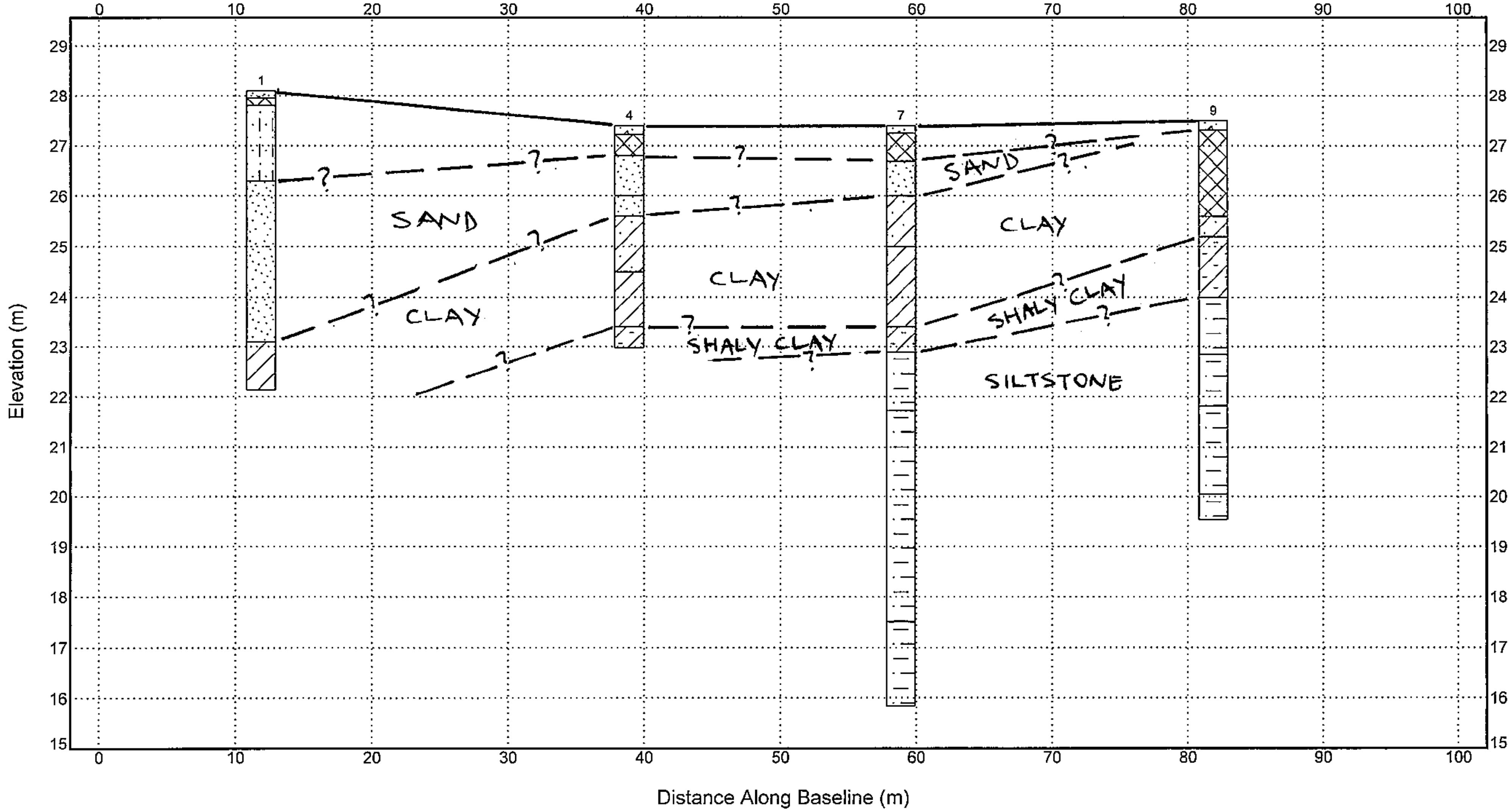
Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville Cairns, Wollongong

TITLE: Location of Test Bores
Rachel Forster Hospital
134-150 Pitt Street
REDFERN

CLIENT: Central Sydney Area Health Service			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 36299	OFFICE: SYDNEY
APPROVED BY: KJB.		DATE: 29.10.2003	DRAWING No: 1

C:\DRAWINGS2003\OTOBER2003\36299.dwg, 06/11/2003 08:53:35 AM

CROSS SECTION A-A'



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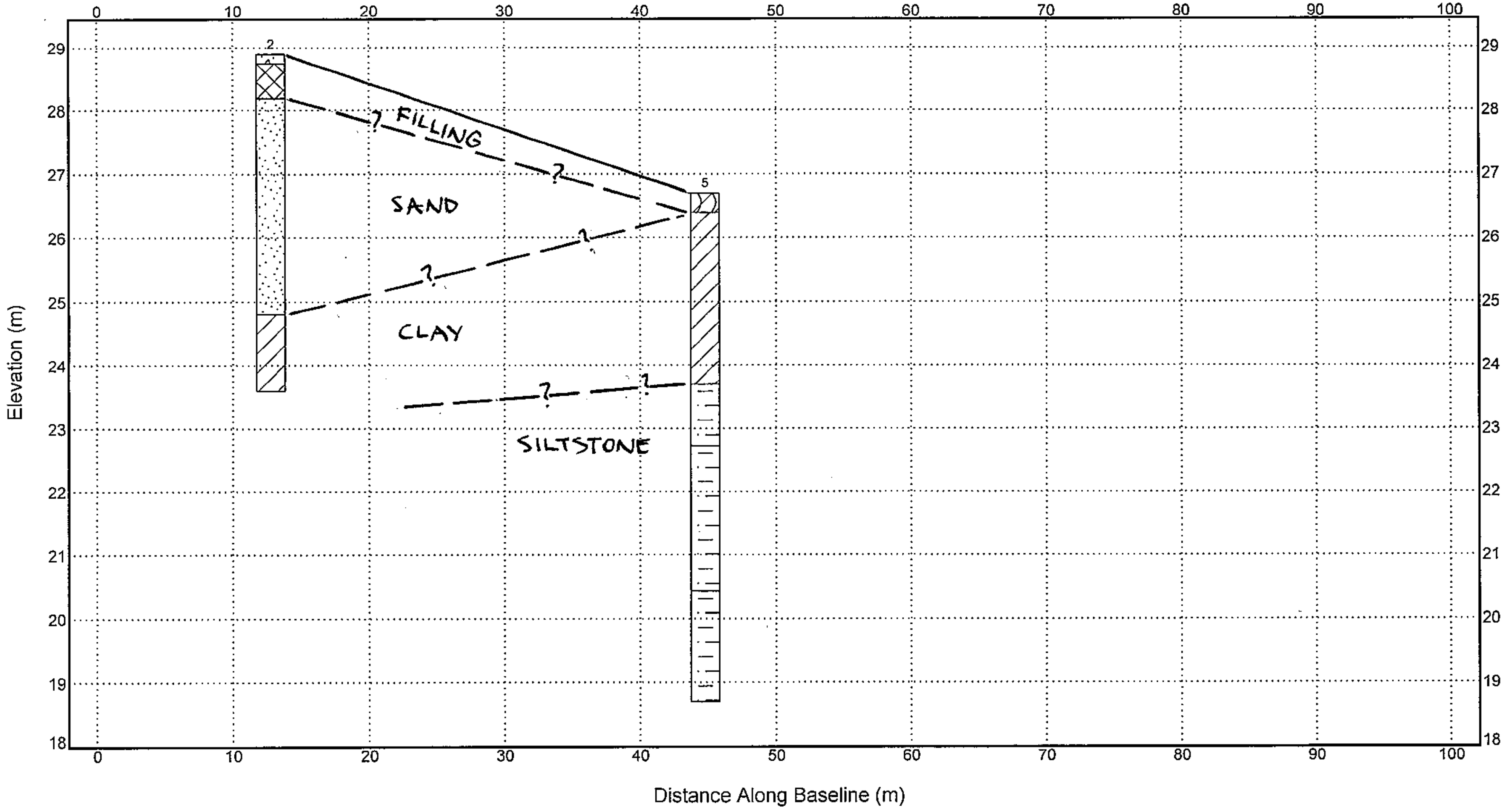
RACHEL FORSTER HOSPITAL
134-150 PITT ST, REDFERN

DRAWING No: 2

CROSS SECTION B-B'



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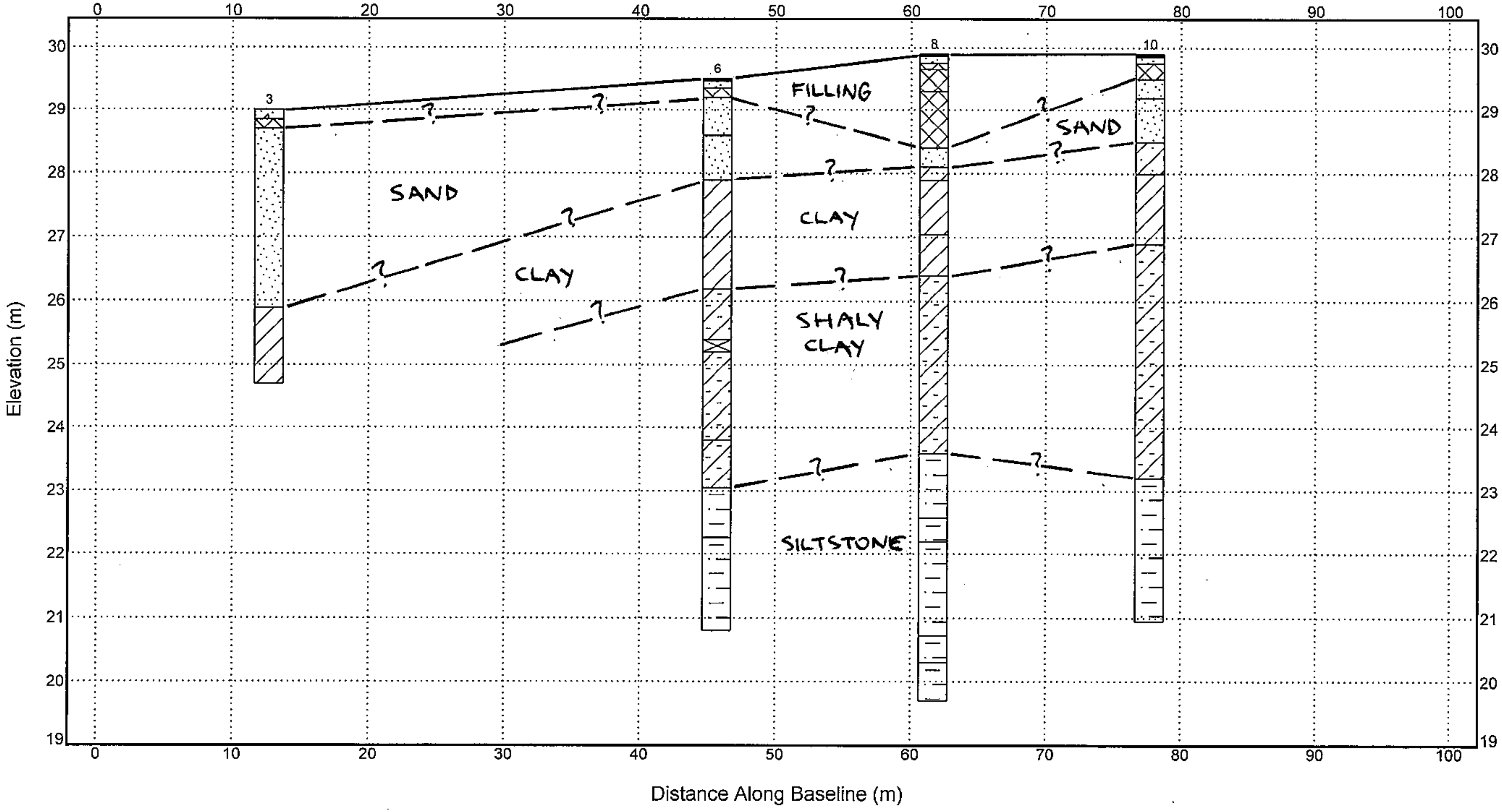
CENTRAL SYDNEY AREA HEALTH SERVICES
RACHEL FORSTER HOSPITAL
134-150 PITT ST, REDFERN

DRAWING No: 3

STRATIGRAPHY - A4 SIZE 36299.GPJ DOUGLAS PARTNERS.GDT 31/10/03

CROSS SECTION C-C'



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CENTRAL SYDNEY AREA HEALTH SERVICES
RACHEL FORSTER HOSPITAL
134-150 PITT ST, REDFERN

DRAWING No: 4

STRATIGRAPHY - A4 SIZE 36299.GPJ DOUGLAS PARTNERS.GDT 31/10/03

APPENDIX B

Test Bore Results and Notes Relating to this Report

TEST BORE REPORT

BORE No: 1
DATE: 25 Sep 03
SHEET 1 **OF** 1
AZIMUTH: --

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT: RACHEL FORSTER HOSPITAL
LOCATION: 134-150 PITT ST, REDFERN

PROJECT No: 36299/36299A
SURFACE LEVEL: 28.1 AHD
DIP OF HOLE: 90°

Depth (m)	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Results	Headspace PID (ppm)
0.15	CONCRETE	A	0.17		
0.3	FILLING - crushed sandstone filling				
	SILTY SAND - very loose, brown fine grained silty sand	A/E	0.5		1
		A/E*	1.0		1
		S		2,2,2 N=4	
			1.45		
		E	1.5		1
1.8	SAND - loose, yellow brown fine grained sand with some brown, slightly cemented bands (coffee rock)				
		E	2.5		1
		S		2,4,5 N=9	
			2.95		
		E	4.0		2
5.0	CLAY - hard, light grey mottled yellow and orange brown clay				
			5.5		
		S		5,11,25 N = 36	
5.95	TEST BORE DISCONTINUED AT 5.95m		5.95		

RIG: SCOUT **DRILLER:** WARD **LOGGED:** JARDINE **CASING:** UNCASED

TYPE OF BORING: SPIRAL FLIGHT AUGER TO 5.5m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED

REMARKS: A* INDICATES FIELD REPLICATE SAMPLE Z1 TAKEN. E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	PID	Photo Ionisation Detector

CHECKED
Initials: WJB
Date: 29/10/03

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TEST BORE REPORT

BORE No: 2

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT No: 36299/36299A
DATE: 25 Sep 03

PROJECT: RACHEL FORSTER HOSPITAL
SURFACE LEVEL: 28.9 AHD
SHEET 1 OF 1

LOCATION: 134-150 PITT ST, REDFERN
DIP OF HOLE: 90°
AZIMUTH: --

Depth (m)	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Results	Headspace PID (ppm)
0.15	CONCRETE	A	0.2		
	FILLING - brown sand filling with bricks and gravel	A/E	0.5		1
0.7	SAND - very loose, light brown/grey fine grained sand	A	1.0		
		S	1.45	1,2,1 N=3	
		E	1.5		1
	Below 2.5m: medium dense	E	2.5		1
		S	2.95	4,8,5 N=13	
4.1	CLAY - very stiff, light red brown and grey clay with a trace of sand	E	4.0		1
		S	4.45	6,9,9 N=18	
5.3	TEST BORE DISCONTINUED AT 5.3m	A	5.3		1

RIG: SCOUT
DRILLER: WARD
LOGGED: JARDINE
CASING: UNCASED

TYPE OF BORING: SPIRAL FLIGHT AUGER TO 5.3m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED

REMARKS: MOVED 30cm TO LEFT DUE TO PRESENCE OF METAL PIPE. E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U,	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	PID	Photo Ionisation Detector

CHECKED
Initials: WJB
Date: 29/10/03

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TEST BORE REPORT

BORE No: 3

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES **PROJECT No:** 36299/36299A **DATE:** 25 Sep 03
PROJECT: RACHEL FORSTER HOSPITAL **SURFACE LEVEL:** 29.0 AHD **SHEET** 1 OF 1
LOCATION: 134-150 PITT ST, REDFERN **DIP OF HOLE:** 90° **AZIMUTH:** --

Depth (m)	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Results	Headspace PID (ppm)
0.15	CONCRETE	A	0.2		
0.3	FILLING - brown sand filling with brick and sandstone gravel	A/E*	0.5		4
	SAND - loose, grey brown fine grained sand	A	1.0		
		S	1.0 1.45	1,3,4 N=7	
		E	1.5		4
	Below 2.5m: medium dense, yellow brown	E	2.5		3
		S	2.5 2.95	3,9,6 N=15	
3.1	CLAY - hard, light red brown and grey clay	E	4.0		2
		S	4.0 4.3	10,17,25/130mm refusal	
4.3	TEST BORE DISCONTINUED AT 4.3m				

RIG: SCOUT **DRILLER:** WARD **LOGGED:** JARDINE **CASING:** UNCASED

TYPE OF BORING: SPIRAL FLIGHT AUGER TO 4.3m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED

REMARKS: A* INDICATES FIELD REPLICATE SAMPLE Z3 TAKEN. E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	PID	Photo Ionisation Detector

CHECKED
Initials:
Date: 27/10/03

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TEST BORE REPORT

BORE No: 4
CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES **PROJECT No:** 36299/36299A **DATE:** 29 Sep 03
PROJECT: RACHEL FORSTER HOSPITAL **SURFACE LEVEL:** 27.4 AHD **SHEET** 1 **OF** 1
LOCATION: 134-150 PITT ST, REDFERN **DIP OF HOLE:** 90° **AZIMUTH:** --

Depth (m)	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Test Results & Comments	Core Rec. %
	CONCRETE	A	0.2		
0.18	FILLING - dark grey coarse grained sand filling, clayey with some roadbase and sandstone gravel	A E	0.4 0.5	PID=3ppm	
0.6	SAND - loose, grey fine grained sand	A S	0.9 1.0 1.45	1,4,6 N = 10	
1.4	SAND - yellow brown and dark brown sand, slightly cemented (coffee rock)	E	1.5	PID=1ppm	
1.8	SANDY CLAY - stiff, yellow brown sandy clay	E S	2.4 2.5 2.95	PID=1ppm 2,4,5 N = 9	
2.9	CLAY - light grey and red brown clay with ironstone	E	3.9	PID=3ppm	
4.0	SHALY CLAY - hard, light grey shaly clay with ironstone bands	S	4.0 4.42	10,17,25/120mm refusal	
4.42	TEST BORE DISCONTINUED AT 4.42m				

RIG: SCOUT **DRILLER:** J WARD **LOGGED:** JARDINE **CASING:** UNCASED
TYPE OF BORING: SPIRAL FLIGHT AUGER TO 4.4m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING
REMARKS: E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: KSB
Date: 29/10/03

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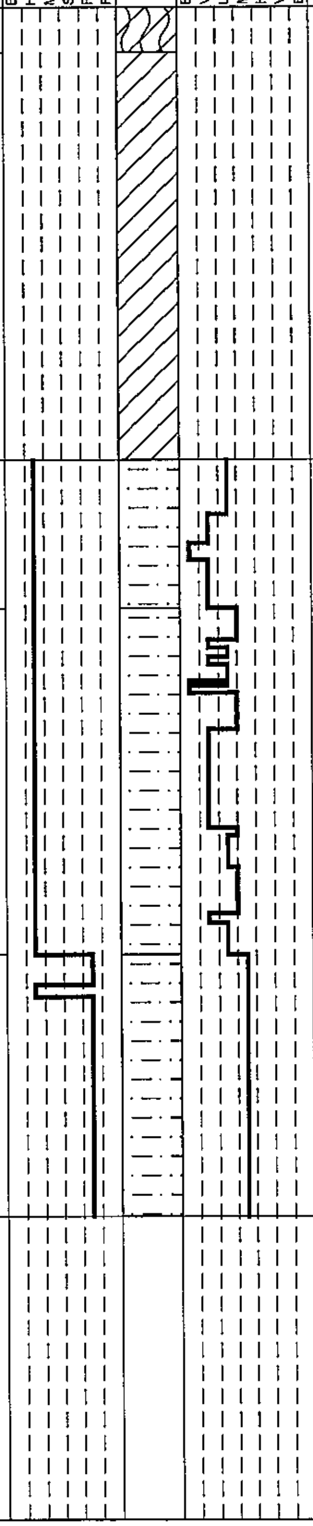
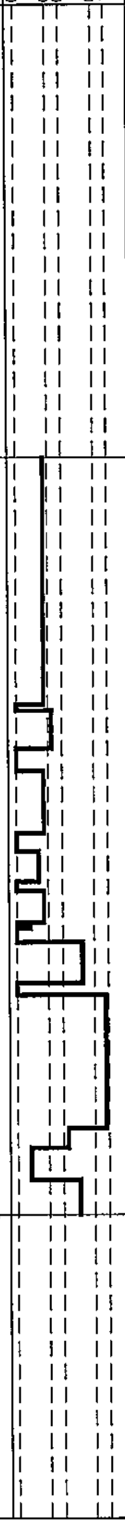
TEST BORE REPORT

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT: RACHEL FORSTER HOSPITAL
LOCATION: 134-150 PITT ST, REDFERN

PROJECT No: 36299/36299A
SURFACE LEVEL: 26.7 AHD
DIP OF HOLE: 90°

BORE No: 5
DATE: 26-29 SEPT 03
SHEET 1 **OF** 1
AZIMUTH: --





Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	TOPSOIL - brown peaty silty clay topsoil with some roots		B A			
0.3	CLAY - hard, light red brown and grey clay with a trace of sand		A/E			PID=2ppm
			A			
1			U$_{50}$			pp=>500kPa
			E			PID=1ppm
2			E			PID=2ppm
			S			9,17,12/140mm refusal
3.0	SILTSTONE - extremely low, very low and low strength, highly weathered, highly fractured, orange brown and light and dark grey siltstone	rock is fractured along bedding planes dipping 0°- 10° & Joints dipping 35°- 45°	C	100	0	
3.98	SILTSTONE - very low, low and low to medium strength, highly weathered, highly fractured and fragmented, orange brown grey and dark grey/black siltstone. Some extremely low strength bands		C	100	0	
5		5.48-5.60m: numerous 5mm to 10mm clay bands				
6.27	SILTSTONE - medium strength, fresh, slightly fractured, dark grey/black siltstone 6.47-6.55m: highly weathered, fragmented band		C	100	56	PL(A) = 0.9MPa
7		7.43m: J35°smooth planar 7.56-7.77m: rock fragmented & fractured along irregular planar joints dipping 90°				PL(A) = 0.3MPa
8.0	TEST BORE DISCONTINUED AT 8.0m					
9						

RIG: SCOUT **DRILLER:** J WARD **LOGGED:** JARDINE **CASING:** TO 3.5m

TYPE OF BORING: SPIRAL FLIGHT AUGER TO 3.0m; NMLC-CORING TO 8.0m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING

REMARKS: E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U$_x$	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: KJB
Date: 29/10/03

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TEST BORE REPORT

BORE No: 6
DATE: 02 Oct 03
SHEET 1 **OF** 1
AZIMUTH: --

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT: RACHEL FORSTER HOSPITAL
LOCATION: 134-150 PITT ST, REDFERN
PROJECT No: 36299/36299A
SURFACE LEVEL: 29.5 AHD
DIP OF HOLE: 90°

Discontinuities: B - Bedding, J - Joint, S - Shear, D - Drill Break

Depth (m)	Description of Strata	Discontinuities	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0.04	BITUMEN		A			
0.14	CONCRETE					
0.3	FILLING - grey sand and orange brown clay filling with some gravel		A E*			
0.9	SAND - light grey/yellow brown fine grained sand with a trace of silt		A			
	SAND - loose, orange brown fine grained sand, slightly cemented		S			2,3,3 N = 6
			E			
1.6	CLAY - very stiff, light grey and red brown clay with ironstone		E			
			S			4,9,10 N = 19
3.3	SHALY CLAY - light grey and red brown shaly clay with ironstone		E			pp = 300kPa
4.1		4.1m: CORE LOSS: 200mm				
4.3	SHALY CLAY - very stiff, light grey mottled yellow and red brown shaly clay (extremely weathered siltstone)		C	88	0	pp = 280kPa pp = 200kPa pp = 300kPa
5.7	SHALY CLAY - hard, light grey and red brown shaly clay with ironstone bands and nodules (extremely weathered siltstone)					pp>400kPa
6.45	SILTSTONE - very low and low to medium strength, highly and moderately weathered, highly fractured, light and dark grey siltstone with some sandstone laminae	rock is fractured along J dipping 30°- 55° & 75°-80° & numerous bedding planes dipping 0°- 5°	C	100	0	
7.24	SILTSTONE - low to medium strength, moderately weathered, slightly fractured then fractured, dark grey and orange brown siltstone with 10% sandstone laminae		C	100	35	PL(A) = 0.3MPa
8.7	TEST BORE DISCONTINUED AT 8.7m					

RIG: SCOUT **DRILLER:** J WARD **LOGGED:** JARDINE **CASING:** TO 4.1m

TYPE OF BORING: DIATUBE TO 0.14m; SPIRAL FLIGHT AUGER TO 4.1m; NMLC-CORING TO 8.7m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING

REMARKS: E* INDICATES FIELD REPLICATE SAMPLE Z5 TAKEN. E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: WSB
Date: 29/10/03

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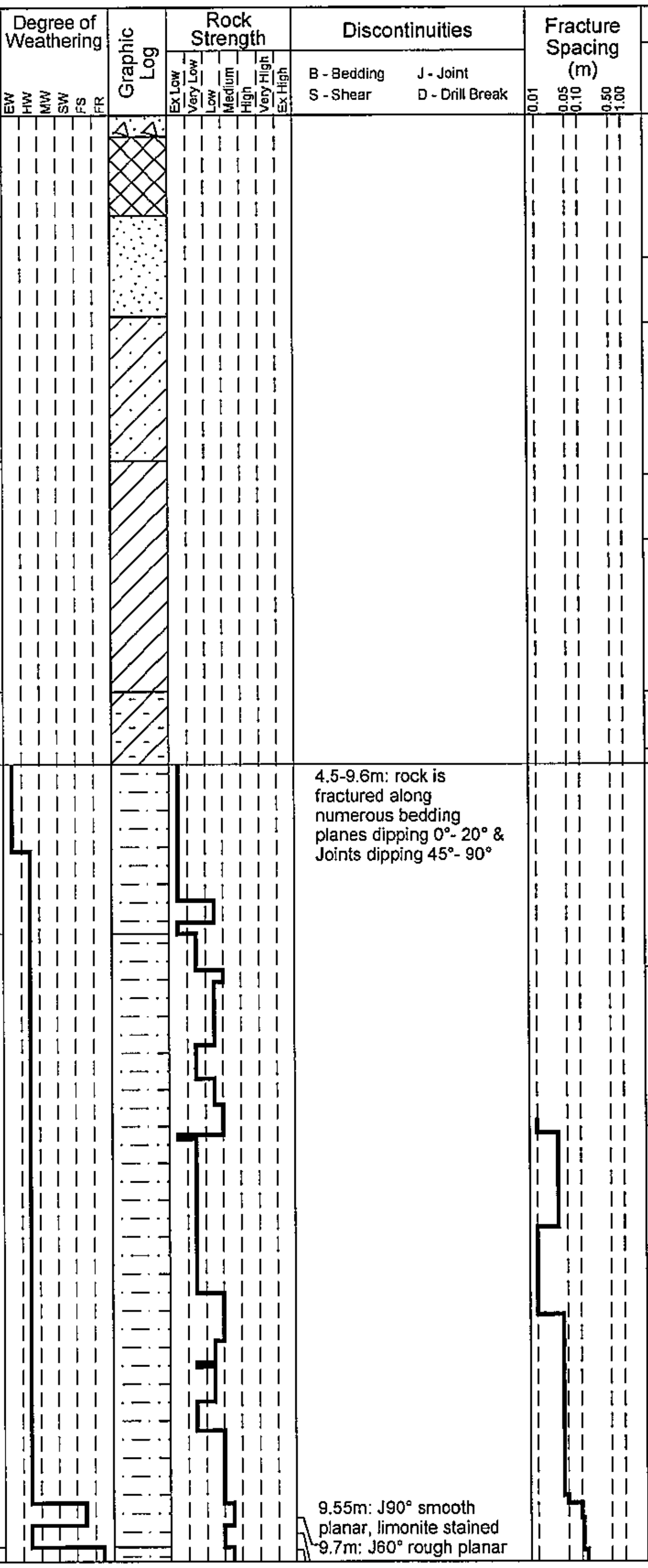
TEST BORE REPORT

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT: RACHEL FORSTER HOSPITAL
LOCATION: 134-150 PITT ST, REDFERN

PROJECT No: 36299/36299A
SURFACE LEVEL: 27.4 AHD
DIP OF HOLE: 90°

BORE No: 7
DATE: 29 Sep 03
SHEET 1 **OF** 2
AZIMUTH: --



Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	CONCRETE		A			
0.15	FILLING - dark brown black clayey sand filling, slightly peaty with a trace of glass and gravel		A E*			PID=1ppm
0.7	SAND - loose, grey brown fine grained sand with some silt		A			
			S			2,4,5 N = 9
1.4	SANDY CLAY - grey and yellow brown mottled sandy clay, slightly cemented		E			PID=3ppm
2.4	CLAY - stiff, light grey and red and yellow brown mottled clay with ironstone		E			PID=2ppm
			S			2,4,7 N = 11
			E			PID=2ppm
4.0	SHALY CLAY - hard, red brown and light grey shaly clay with ironstone bands		S			10,17,25/140mm refusal
4.5	SILTSTONE - extremely low strength, extremely weathered, light grey and orange and yellow brown siltstone. Numerous 5-10mm medium strength ironstone bands Below 5.10m: highly weathered	4.5-9.6m: rock is fractured along numerous bedding planes dipping 0°- 20° & Joints dipping 45°- 90°	C	100	0	
5.67	SILTSTONE - very low, low and low to medium strength, highly weathered, fragmented and fractured, orange brown and dark grey siltstone		C	100	0	
			C	100	0	
			C	100	58	
	9.6 to 9.75m: medium strength band	9.55m: J90° smooth planar, limonite stained 9.7m: J60° rough planar				
9.9						

RIG: SCOUT **DRILLER:** J WARD **LOGGED:** JARDINE **CASING:** TO 5.0m

TYPE OF BORING: SPIRAL FIGHT AUGER TO 4.5m; NMLC-CORING TO 11.55m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING

REMARKS: E* INDICATES FIELD REPLICATE SAMPLE Z4 TAKEN. E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: WJB
Date: 29/10/03

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TEST BORE REPORT

BORE No: 7

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES **PROJECT No:** 36299/36299A **DATE:** 29 Sep 03
PROJECT: RACHEL FORSTER HOSPITAL **SURFACE LEVEL:** 27.4 AHD **SHEET** 2 **OF** 2
LOCATION: 134-150 PITT ST, REDFERN **DIP OF HOLE:** 90° **AZIMUTH:** --

Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Sampling & In Situ Testing: Sample Type	Core Rec. %	RQD %	Test Results & Comments
11	SILTSTONE - medium strength, fresh, fractured and slightly fractured, dark grey siltstone with 5% sandstone laminae *(continued)*				9.81m: J75° rough planar, limonite stained 9.92m: J45° smooth planar, limonite stained 10.36m: J45°smooth planar 10.58m: J35°smooth planar 10.69m: J40°smooth planar 10.73m: B0° 5mm clay		C	100	58	PL(A) = 0.5MPa PL(A) = 0.7MPa
11.55 12 13 14 15 16 17 18 19	TEST BORE DISCONTINUED AT 11.55m									

RIG: SCOUT **DRILLER:** J WARD **LOGGED:** JARDINE **CASING:** TO 5.0m

TYPE OF BORING: SPIRAL FIGHT AUGER TO 4.5m; NMLC-CORING TO 11.55m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING

REMARKS: E* INDICATES FIELD REPLICATE SAMPLE Z4 TAKEN. E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U,	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: KJB
Date: 29/10/03

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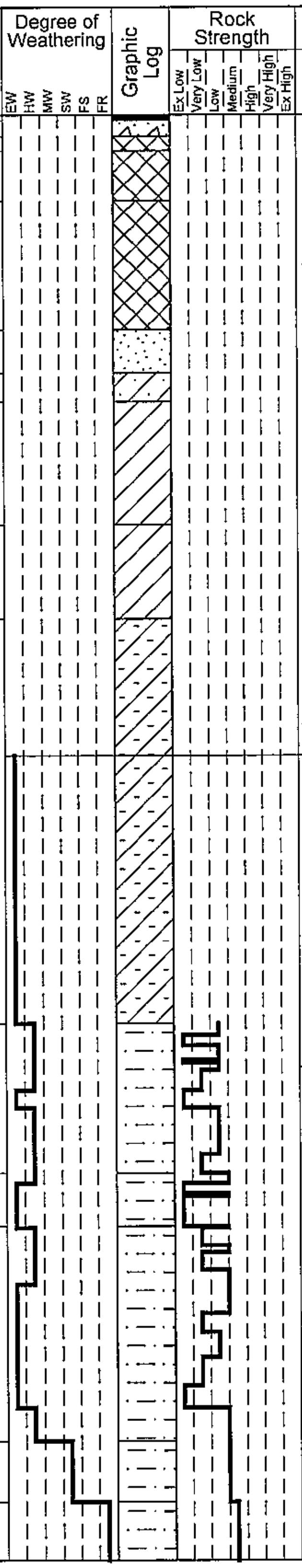
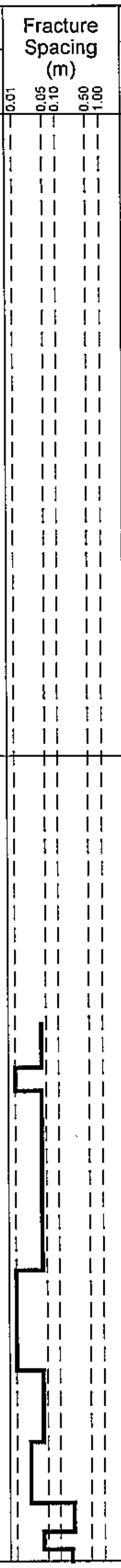
TEST BORE REPORT

BORE No: 8
DATE: 01 Oct 03
SHEET 1 **OF** 2
AZIMUTH: --

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT: RACHEL FORSTER HOSPITAL
LOCATION: 134-150 PITT ST, REDFERN
PROJECT No: 36299/36299A
SURFACE LEVEL: 29.9 AHD
DIP OF HOLE: 90°

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0.04	BITUMEN		A			
0.15	CONCRETE					
0.25	FILLING - brown sand filling with roots and sandstone gravel		A E			PID=2ppm
0.6	FILLING - orange brown clayey sand filling		A E			
	FILLING - light orange brown sand and clay nodule filling with brick and sandstone gravel		S			3,3,6 N=9
1.5	Below 1.0m: grey sand and crushed sandstone		E			PID=2ppm
1.8	SAND - brown sand (driller's description)					
2.0	SANDY CLAY - grey and brown sandy clay (driller's description)					
	CLAY - hard, light grey and red brown clay		E U_{50}			PID=10ppm pp=380->500kPa
2.85	CLAY - light grey and red brown clay with ironstone gravel					
3.5	SHALY CLAY - very stiff, light grey shaly clay with a trace of decomposed roots (extremely weathered siltstone)		E S			PID=2ppm 2,6,14 N = 20 ~~pp = 280kPa~~ pp = 500kPa
	Below 4.5m: hard Below 5.4m: some medium strength red brown ironstone bands and nodules		C	100	0	
6.3	SILTSTONE - very low and low strength, highly then extremely weathered, light grey, grey and orange brown siltstone . Some extremely low strength bands	6.3m: rock is fractured along bedding planes dipping 0°- 5° & joints dipping 45°- 90° 6.6-6.76m: brecciated zone	C	100	0	
7.33	SILTSTONE - extremely low and low to medium strength, extremely and highly weathered, light grey and orange brown siltstone					
7.7	SILTSTONE - very low and low to medium strength, extremely and highly weathered, light grey, dark grey and orange brown siltstone. Occasional extremely low and low strength bands	8.0-8.3m: fragmented, discontinuities obscured 8.3-8.69m: brecciated zone	C	100	0	
9.18	SILTSTONE - low to medium strength, slightly weathered, dark grey/black and orange brown siltstone		C	100	38	
9.6						





RIG: SCOUT
DRILLER: J WARD
LOGGED: JARDINE
CASING: TO 4.45m

TYPE OF BORING: DIATUBE TO 0.15m; SPIRAL FLIGHT AUGER TO 4.45m; NMLC-CORING TO 10.20m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING

REMARKS: E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: KJB
Date: 29/10/03

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TEST BORE REPORT

BORE No: 8
CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT No: 36299/36299A
DATE: 01 Oct 03
PROJECT: RACHEL FORSTER HOSPITAL
SURFACE LEVEL: 29.9 AHD
SHEET 2 **OF** 2
LOCATION: 134-150 PITT ST, REDFERN
DIP OF HOLE: 90°
AZIMUTH: --

Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low Very Low Low Medium High Very High Ex High	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Sample Type	Core Rec. %	RQD %	Test Results & Comments
10.2	SILTSTONE - medium strength, fresh, fractured and slightly fractured, dark grey/black siltstone *(continued)*				10.1m: J75° irregular, planar, limonite stained		C	100	38	
	TEST BORE DISCONTINUED AT 10.2m									
11										
12										
13										
14										
15										
16										
17										
18										
19										

RIG: SCOUT
DRILLER: J WARD
LOGGED: JARDINE
CASING: TO 4.45m
TYPE OF BORING: DIATUBE TO 0.15m; SPIRAL FLIGHT AUGER TO 4.45m; NMLC-CORING TO 10.20m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING
REMARKS: E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: LSB
Date: 29/10/03

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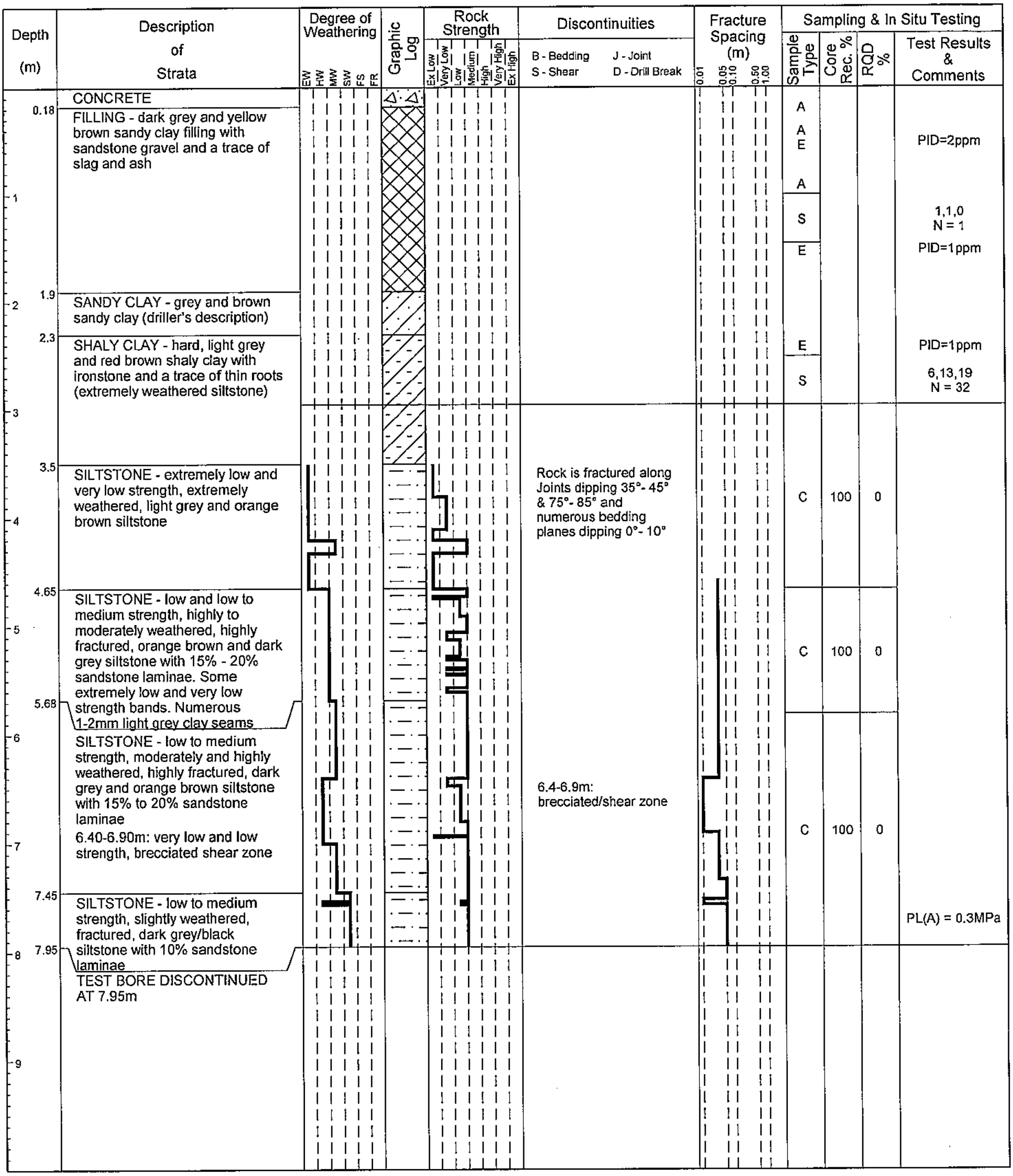
TEST BORE REPORT

CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT: RACHEL FORSTER HOSPITAL
LOCATION: 134-150 PITT ST, REDFERN

PROJECT No: 36299/36299A
SURFACE LEVEL: 27.5 AHD
DIP OF HOLE: 90°

BORE No: 9
DATE: 03 Oct 03
SHEET 1 OF 1
AZIMUTH: --



Discontinuities: B - Bedding, J - Joint, S - Shear, D - Drill Break

Depth (m)	Description of Strata	Discontinuities	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	CONCRETE		A			
0.18	FILLING - dark grey and yellow brown sandy clay filling with sandstone gravel and a trace of slag and ash		A E			PID=2ppm
			A			
			S			1,1,0 N = 1
			E			PID=1ppm
1.9	SANDY CLAY - grey and brown sandy clay (driller's description)					
2.3	SHALY CLAY - hard, light grey and red brown shaly clay with ironstone and a trace of thin roots (extremely weathered siltstone)		E			PID=1ppm
			S			6,13,19 N = 32
3.5	SILTSTONE - extremely low and very low strength, extremely weathered, light grey and orange brown siltstone	Rock is fractured along Joints dipping 35°- 45° & 75°- 85° and numerous bedding planes dipping 0°- 10°	C	100	0	
4.65	SILTSTONE - low and low to medium strength, highly to moderately weathered, highly fractured, orange brown and dark grey siltstone with 15% - 20% sandstone laminae. Some extremely low and very low strength bands. Numerous 1-2mm light grey clay seams		C	100	0	
5.68	SILTSTONE - low to medium strength, moderately and highly weathered, highly fractured, dark grey and orange brown siltstone with 15% to 20% sandstone laminae 6.40-6.90m: very low and low strength, brecciated shear zone	6.4-6.9m: brecciated/shear zone	C	100	0	
7.45	SILTSTONE - low to medium strength, slightly weathered, fractured, dark grey/black siltstone with 10% sandstone laminae					PL(A) = 0.3MPa
7.95	TEST BORE DISCONTINUED AT 7.95m					

RIG: SCOUT **DRILLER:** J WARD **LOGGED:** JARDINE **CASING:** TO 2.7m

TYPE OF BORING: DIATUBE TO 0.18m; SPIRAL FLIGHT AUGER TO 2.7m; NMLC-CORING TO 7.95m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING

REMARKS: E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U,	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: KDB
Date: 29/10/03

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TEST BORE REPORT

BORE No: 10
CLIENT: CENTRAL SYDNEY AREA HEALTH SERVICES
PROJECT No: 36299/36299A
DATE: 02 Oct 03
PROJECT: RACHEL FORSTER HOSPITAL
SURFACE LEVEL: 29.9 AHD
SHEET 1 **OF** 1
LOCATION: 134-150 PITT ST, REDFERN
DIP OF HOLE: 90°
AZIMUTH: --

Depth (m)	Description of Strata
0.05	BITUMEN
0.15	CONCRETE
0.4	FILLING - grey coarse grained sand filling with sandstone, brick and coal gravel
0.7	SAND - brown fine grained sand, slightly peaty with a trace of roots
	SAND - loose, orange brown fine grained sand. Slightly cemented in places
1.4	SANDY CLAY - grey and brown sandy clay (driller's description)
1.9	CLAY - stiff, red and yellow brown mottled clay
3.0	SHALY CLAY - hard, light grey shaly clay with ironstone bands (extremely weathered siltstone)
6.7	SILTSTONE - extremely low, very low and low strength, extremely and highly weathered, light grey, grey, red and orange brown siltstone with some sandstone laminae. Numerous low to medium strength, thin ironstone bands
8.95	TEST BORE DISCONTINUED AT 8.95m

Degree of Weathering: EW HW MW SW FS FR
Rock Strength: Ex Low, Very Low, Low, Medium, High, Very High, Ex High
Discontinuities: B - Bedding J - Joint S - Shear D - Drill Break
Fracture Spacing (m): 0.01 0.05 0.10 0.50 1.00

Discontinuities (6.7–8.95 m): Rock is fractured along numerous bedding planes dipping 0° - 10°

Sample Type	Core Rec. %	RQD %	Test Results & Comments
A			
B			
A			
E			PID=2ppm
A			
S			1,2,2 N = 4
E			PID=1ppm
E			PID=1ppm
S			2,3,7 N = 10
E			
C	100	0	pp>400kPa pp>400kPa pp>400kPa
C	100	0	
C	100	0	

RIG: SCOUT **DRILLER:** J WARD **LOGGED:** JARDINE **CASING:** TO 4.1m
TYPE OF BORING: DIATUBE TO 0.15m; SPIRAL FLIGHT AUGER TO 4.1m; NMLC-CORING TO 8.95m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED
REMARKS: E = ENVIRONMENTAL SAMPLE

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U,	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: KJB
Date: 29/10/03

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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

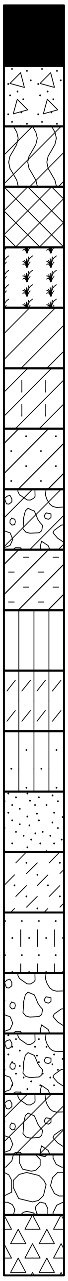
Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including 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.

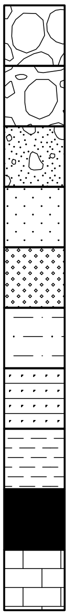
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL



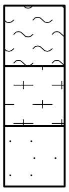
BITUMINOUS CONCRETE
CONCRETE
TOPSOIL
FILLING
PEAT
CLAY
SILTY CLAY
SANDY CLAY
GRAVELLY CLAY
SHALY CLAY
SILT
CLAYEY SILT
SANDY SILT
SAND
CLAYEY SAND
SILTY SAND
GRAVEL
SANDY GRAVEL
CLAYEY GRAVEL
COBBLES/BOULDERS
TALUS

SEDIMENTARY ROCK



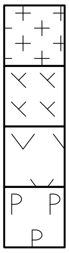
BOULDER CONGLOMERATE
CONGLOMERATE
CONGLOMERATIC SANDSTONE
SANDSTONE FINE GRAINED
SANDSTONE COARSE GRAINED
SILTSTONE
LAMINITE
MUDSTONE, CLAYSTONE, SHALE
COAL
LIMESTONE

METAMORPHIC ROCK



SLATE, PHYLITTE, SCHIST
GNEISS
QUARTZITE

IGNEOUS ROCK



GRANITE
DOLERITE, BASALT
TUFF
PORPHYRY

APPENDIX C

Laboratory Test Results

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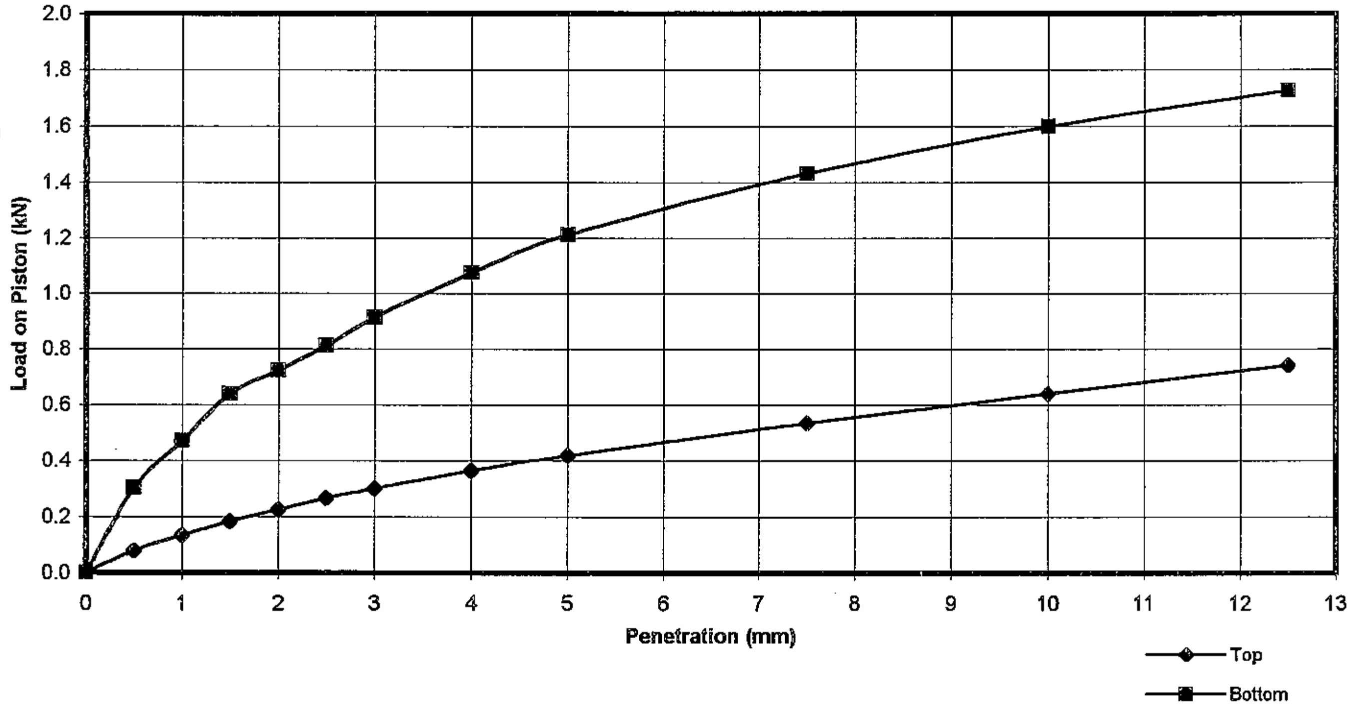
RESULT OF CALIFORNIA BEARING RATIO TEST

Client :	CENTRAL SYDNEY AREA HEALTH SERVICE	**Project No. :**	36299
		Report No. :	S03-376B
Project :	GEOTECHNICAL INVESTIGATION	**Report Date :**	08-10-03
		Date Sampled :	29-09-03
Location :	REDFERN	**Date of Test:**	03-10-03
Test Location :	BH5		
Depth / Layer :	0 - 1.0m	**Page:**	1 of 1



Description: SILTY CLAY - Red brown silty clay

Test Method(s): AS1289.6.1.1-1998, AS1289.2.1.1-1992

Sampling Method(s): AS 1289.1.2.1-1998, AS 1289.1.1-2001

Percentage > 19mm: 0.0%

LEVEL OF COMPACTION: 99% of STD MDD
MOISTURE RATIO: 102% of STD OMC
SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
SWELL: 5.6%

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		18.3	1.74
After soaking		24.3	1.64
After test	Top 30mm of sample	25.6	-
	Remainder of sample	21.8	-
Field values		22.4	-
Standard compaction		18.0	1.75

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**2.5 mm**	**2.0**
	5.0 mm	**2.0**
BOTTOM	**2.5 mm**	**6**
	5.0 mm	**6**

Tested: AY/SL
Checked: NW

Approved Signatory:

N Weimann

Norman Weimann
Laboratory Manager

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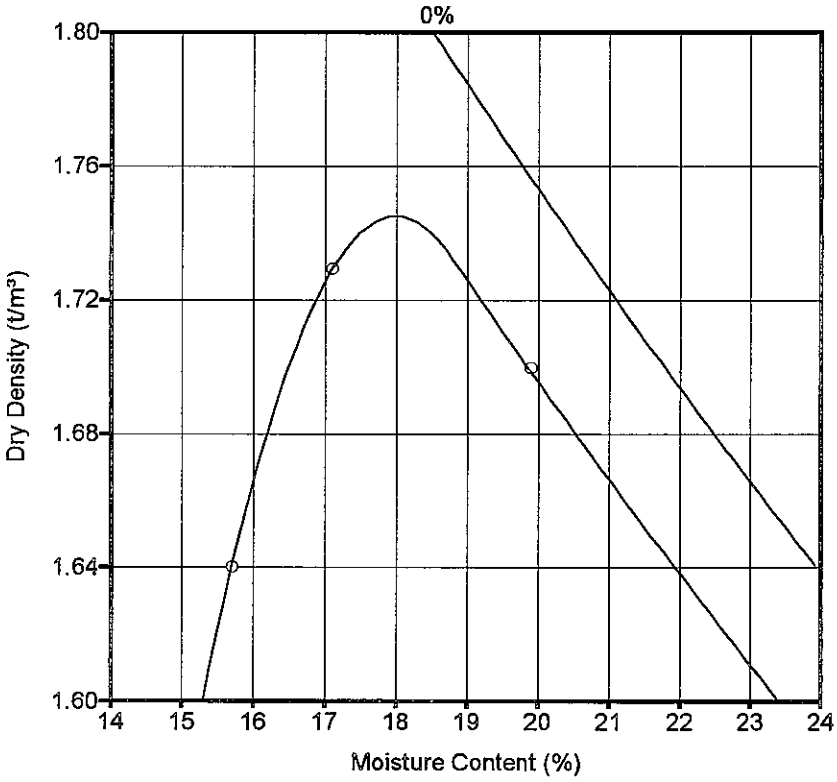
RESULTS OF COMPACTION TEST

Client:	CENTRAL SYDNEY AREA HEALTH SERVICE	**Project No:** **Report No:** **Report Date:**	36299 S03-376A 08-Oct-2003
Project:	GEOTECHNICAL INVESTIGATION	**Date Sampled:** **Date of Test:**	29/09/03 02/10/03
Location:	REDFERN	**Page:**	1 of 1



SAMPLE DETAILS:

Location:	BH5
Depth:	0 - 1.0m

Description:
SILTY CLAY - Red brown silty clay

Specific Gravity:	2.7
Particles >19mm:	
Field Moisture Content:	22.4 %
Optimum Moisture Content:	18.0 %
Maximum Dry Density:	1.75 t/m^3

Test Methods: AS1289.5.1.1-1993, AS1289.2.1.1-1992

Sampling Method(s): AS1289.1.2.1-1998, AS1289.1.1-2001

Remarks:

NATA Accredited Laboratory No: 828 Sydney
This Laboratory is accredited by the National Association of Testing Authorities, Australia. The test(s) reported herein have been performed in accordance with its terms of accreditation. This document shall not be reproduced except in full.

Approved Signatory:

Tested: CTM
Checked: NW

Norman Weimann
Laboratory Manager