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REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED SUBSTATION
ROYAL NORTH SHORE HOSPITAL
ST LEONARDS

Prepared for
ENERGY AUSTRALIA PTY LTD

Project 45268
February 2008

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REPORT ON GEOTECHNICAL INVESTIGATION
PROPOSED SUBSTATION
ROYAL NORTH SHORE HOSPITAL, ST LEONARDS

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1. INTRODUCTION

This report presents the results of a geotechnical investigation undertaken for a proposed substation at Royal North Shore Hospital, St Leonards. The investigation was commissioned by Energy Australia.

It is understood that the proposed substation will include two large transformers with associated cable basements and access roads. The project is in the preliminary planning stages and therefore the transformer footprints and basement floor levels have not been finalised. It is however understood that the basements may require excavation to depths of 3 m to 4 m.

The field work for the investigation comprised the drilling of five boreholes within in-situ testing and sampling of the soils and coring of the underlying bedrock. Laboratory testing of selected soil and rock core samples was undertaken, followed by engineering analysis and reporting. This investigation also included a review of previous geotechnical investigations and assessments carried out by Douglas Partners Pty Ltd (DP) in close proximity to the site. The previous work includes a geotechnical investigation for proposed extensions to the North Shore Private Hospital (to the south of the site) and geotechnical assessments of the excavation faces around the former quarry to the north of the site.

2. PREVIOUS INVESTIGATIONS

2.1 General

The Royal North Shore Hospital has been built on a local topographic high and the original ground surface would have sloped gently down from this high point in all directions. There have, however, been some major changes to the original topography as a result of a number of quarries which have operated in the area. The quarries mined the Ashfield Shale for manufacture of bricks and have generally been taken down to depths of 15 m to 25 m to the base of the shale and the top of the underlying sandstone bedrock (either Mittagong Sandstone or Hawkesbury Sandstone). The quarries closed many years ago and the area has since been mostly occupied by industrial or warehouse buildings.

Of particular relevance to the proposed substation is an old quarry which occupied a large rectangular shaped area between Reserve Road and Clarendon Street, immediately to the north of the hospital and south of Campbell Street. Lanceley Place bisects the old quarry site which is currently occupied by a number of large buildings, including a Waste Transfer Station in the south-western corner of the former quarry.

The western, southern and eastern boundaries of the quarry are near-vertical, excavation faces which have mostly been covered with shotcrete. In the south-western corner of the old quarry site the excavation face is approximately 23 m high.

2.2 Previous Assessment of Quarry Faces

DP have undertaken geotechnical assessments of the excavation faces around the old quarry to the north of the proposed substation, which included mapping of the excavation faces and providing advice on stabilisation measures. In particular, minor excavation at the toe of the southern face in 1987 caused a major slide through the weathered shale and DP provided advice on stabilisation measures for the failed area as well as adjacent areas. A plan showing the location of the slide is provided in Appendix D and also indicates the location of the proposed substation.

The quarry faces have been excavated through the clays and weathered and jointed Ashfield Shale down to the underlying sandstone. The clays and shales deteriorate as a result of exposure and there are often steeply dipping joints within the shale which combine to form large wedge failures. The worst cases are wedges formed by joints or faults dipping at about 45 degrees below horizontal. These wedge failures can occur relatively suddenly without warning.

The south-western corner of the old quarry, along the western and southern boundaries of the Waste Transfer Station, includes excavation faces up to 23 m high. As indicated above, a large section of the southern face failed suddenly during minor excavation at the toe of this face. A number of investigations were undertaken in this area after the failure, including the drilling of several deep bores (the borehole location plan and borehole logs are included in Appendix D). This whole corner of the site has subsequently been stabilised by installation of soldier piles through the upper soils, numerous ground anchors and shotcrete facing. Sketches showing the proposed stabilisation measures are also provided in Appendix D, although it is understood that some anchors in addition to those shown were installed during construction. The stabilisation measures were designed using a factor of safety against further slope failure of 1.5, although these designs did not allow for any surcharge loads above the face. The ground anchors have been installed with double corrosion protection and hence should have a design life of at least 50 years and probably 100 years from the date of installation which was about 1987.

The eastern half of the southern face, to the north of the proposed substation and existing multi-storey car park, is up to about 20 m high. The upper section includes soil nails through the clay soils with a shotcrete facing and the lower section through the shale is covered by shotcrete only. The mapping did not identify any adverse joints through the shale.

2.3 Previous Geotechnical Investigation for Extensions to the North Shore Private Hospital

DP previously carried out a geotechnical investigation for proposed extensions to the North Private Hospital (Project 44433, dated January 2007). The investigation included four cored boreholes (BH1 to BH4) drilled to depths of approximately 18 m. The borehole locations and approximate footprint of the proposed extension are shown on Drawing 1 in Appendix B and the logs of the nearest boreholes to the substation site (BH1A and BH3) are provided in Appendix D.

2.4 Previous Geotechnical Asssessment for Substation Sites

DP carried out a geotechnical assessment for four possible substation sites within the hospital grounds (Project 44681, dated 18/4/07). This work was carried out for Enerserve in consultation with Taylor Thomson Whitting. This assessment included the current site of the proposed substation and comprised a desktop review of previous work carried out by DP in the area.

3. SITE DESCRIPTION

The site of the proposed substation (refered to as 'the site' in this report) is located on the northern side of the Royal North Shore Hospital grounds, to the north of the North Shore Private Hospital. The southern edge of the former quarry is located approximately 30 m to the north of the site.

The site is a rectangular-shaped lot which covers an area of approximately 3000 m^2. At the time of the investigation the site was occupied by single-storey brick buildings (Breast Screening Clinic) and surrounding asphaltic concrete paved access roads and carparks.

The site is located on a gentle north-facing slope near the top of a local topographic high. Within the site, the surface generally falls to the north from approximately RL 98.0 to RL 93.0, relative to Australian Height Datum (AHD), at an average slope of approximately 5 degrees. As outlined in Section 2.1, significant excavation has previously been carried out on the former quarry to the north of the site with near-vertical excavations to depths of approximately 15 m to 25 m (to about RL 72).

The Northern Sydney College (TAFE) is located on the property to the west of the site.

The area to the south of the site (the site of the proposed extension to the North Shore Private Hospital) is an open asphaltic concrete paved carpark. The area to the north of the site (south of the former quarry) is also an aspaltic concrete paved carpark.

There is a multi-level carpark on the property to the east of the site. The carpark is generally set back approximately 10 m from the common boundary, with the exception of the carpark ramp which extends up to the assumed common boundary.

4. GEOLOGY

Reference to the Sydney 1:100 000 Series Geological Sheet indicates the site is underlain by Ashfield Shale which typically comprises black to dark grey shale and laminite (interbedded shale, siltstone and fine grained sandstone). Near the ground surface these rocks often weather to form moderate to highly reactive clays. The geological mapping was confirmed by the field work which identified residual soils and underlying shale and laminite.

5. FIELD WORK METHODS

The field work comprised five boreholes (BH 101 to BH 105) drilled to depths of 4.0 m to 11.2 m using a bobcat-mounted drilling rig.

The boreholes were initially drilled using spiral augers and rotary washboring within soil and extremely weathered rock to depths of 3.0 m to 4.0 m. Boreholes 101 and 104 were then cased and continued into the underlying rock using diamond core drilling techniques to obtain continuous core samples of the bedrock.

Standard Penetration Tests (SPT's) were carried out below depths of 1.0 m to sample the soil and extremely weathered rock and assess the in-situ strength of the materials. Disturbed soil samples were also retrieved from the boreholes during drilling for identification and classification purposes.

The boreholes were logged on site by a geotechnical engineer. The rock cores were returned to the DP office where the rock cores were photographed and Point Load Strength Index (Is_{50}) tests carried out on selected samples of the rock core.

The borehole locations (including previous investigations) are shown on Drawing 1 in Appendix B.

The ground surface level at each of the test locations was interpolated from spot heights (relative to AHD) shown on the survey plan by Hard and Forrester Pty Ltd (Drawing 111521023, dated 30/6/07).

6. FIELD WORK RESULTS

Details of the conditions encountered are given in the borehole logs in Appendix A, together with colour photographs of the rock core samples and notes defining classification methods and descriptive terms. Borehole logs from the previous investigations are included in Appendix D.

The boreholes penetrated a subsurface profile typically comprising pavements and/or filling to depths of 0.15 m to 0.8 m, then residual clay and shaly clay to depths of 2.0 m to 3.5 m overlying bedrock comprising interbedded shale and laminite to the maximum investigation depth of 11.2 m. The various strata are summarised below and interpreted geotechnical sections (Section A-A' and B-B') through the site are given on Drawings 2 to 3 in Appendix B.

PAVEMENTS:	comprised asphaltic concrete (AC) 0.03 m thick over sandy gravel filling (roadbase) 0.12 m thick.
FILLING:	encountered in BH 104 and BH 105 to depths of 0.5 and 0.8 m, respectively. The filling generally comprised sandy clay with some gravel, glass and building rubble in BH 105.

RESIDUAL CLAY:	generally comprised stiff to very stiff sandy clay and clay to depths of 1.0 m to 2.0 m then hard shaly clay to depths of 2.0 m to 3.5 m.
SHALE/LAMINITE:	generally comprised highly weathered, very low to low strength rock to depths of 7.5 m to 9.0 m (RL 85.5 to RL 86.5) over slightly weathered to fresh, medium strength rock. Extremely low to very low strength and medium to high strength bands were encountered within the rock profile.

The rock included moderately and steeply dipping joints with dips ranging from 30^o to 60^o below the horizontal plane, together with some sub-vertical and low angle joints. Zones of crushed rock (possible shear zones) were identified in the rock core from BH 101 between a depth of 7.0 m to 7.4 m.

No free groundwater was observed during augering of the boreholes (i.e. within depths of 3.0 m to 4.0 m). The use of water during wash boring and coring within the bedrock prevented the measurement of groundwater below this depth.

7. LABORATORY TESTING

7.1 Soil

Selected samples of soil were tested in the DP laboratory to assess Atterberg Limits and Linear Shrinkage. Selected samples were also tested at an external laboratory to assess aggressivity (pH, chloride and sulphate content). The results of the laboratory testing are included in Appendix D and summarised in Table 1.

Table 1 : Summary of Laboratory Test Results (Soil)

Bore	Depth (m)	Material	Atterberg Limits			LS (%)	pH	SO_4^{2-} (mg/L)	Cl^- (mg/L)
			W_l (%)	W_p (%)	PI (%)				
BH101	1.0	Clay	36	22	14	6.5			
BH 105	2.5	Shaly Clay	37	21	16	7			
BH102	1.0	Shaly Clay					4.4	120	<100
BH102	2.5	Shale					5.3	140	<100
BH102	4.0	Laminite					5.3	110	<100

W_f = Field Moisture Content
W_l = Liquid Limit
W_p = Plastic Limit
PI = Plasticity Index
LS = Linear Shrinkage
SO_4^{2-} = Sulphate
Cl^- = Chloride

The results of the Atterberg Limits and Linear Shrinkage tests indicate the clay is of medium plasticity.

The results of the chemical analysis indicate the soil and rock samples were within a mild to non-aggressive exposure classification in accordance with AS2159 - 1995 (Piling - Design and Installation).

7.2 Rock Cores

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values. The results of the testing are shown on the borehole logs at the appropriate depth.

It is noted that Is_{50} tests are not readily carried out on extremely low to very low strength rock and hence strength classification for the weaker rock is primarily based on visual/tactile assessments of the rock core. The Is_{50} values for the various rock strata are described below together with the estimated unconfined compressive strength (UCS) which is based on a UCS:Is_{50} ratio of 1:15 for very low to low strength rock and 1:20 for medium to high strength rock.

The Is_{50} values for the rock cores ranged from 0.2 MPa to 1.5 MPa, corresponding to rock strengths ranging from low to high strength classification (estimated UCS ranging from 3 MPa to 30 MPa).

8. COMMENTS

8.1 Proposed Development

The project is in the preliminary planning stages and therefore the substation footprint and bulk excavation levels have not been finalised. It is understood that the proposed substation will probably include two large transformers (at the approximate locations shown on Drawing 1) with associated cable basements and access roads. The basements may require excavation to depths of approximately 3 m to 4 m.

8.2 Effect of Adjacent Excavations

8.2.1 Former Quarry

The northern site boundary is set back approximately 30 m from the former quarry excavation face which is approximately 20 m high.

The section of the quarry face to the north of the site has soil nails installed within the upper clay soils and an unreinforced shotcrete covering over the rest of the face. If there are any adverse defects in the face then the factor of safety against failure is likely to be close to 1.0, without any allowance for surcharge loading above the top of the face.

The section of the quarry face to the west of the site (previous slip zone) has been extensively stabilised with a factor of safety of 1.5 for the known failure surface, without any allowance for surcharge loading.

For the site of the proposed substation the worst case would be failure into the former quarry of a wedge extending up from the toe of the face at about 45 degrees above horizontal. The northern site boundary is set back approximately 10 m behind this potential failure line, and therefore it is considered that there would be a low risk of instability and no special precautions need to be taken for the foundations.

If access roads are located to the north of the site then it is possible that some of the access roads may run parallel to the top of the quarry excavation faces and that very heavy transformers may be transported occasionally along these roads. For these temporary load conditions it is assessed that the installed stabilisation in the south-western corner of the quarry will provide adequate support. However, for the south-eastern corner of the quarry (directly to the north of the site) it is assessed that the factor of safety would not be sufficient for these heavy loads. In these areas it is recommended that the access roads be located at least 3 m back from the top of the quarry face to minimise the risk of failure of the upper clay soils. Alternatively the roads could be supported on piers taken down into the shale.

8.2.2 Proposed Extension to North Shore Private

At the time of the previous investigation, the proposed extension to the North Shore Private Hospital included a six-storey building with three levels of basement carparking. It is understood the lowest basement floor level (RL 90) will require excavation to depths of approximately 8 m below existing surface levels. As shown on Drawing 1, the basement is set back approximately 9 m from the southern boundary of the substation site.

It is anticipated that the basement excavation will be supported by a shoring system comprising soldier piles with temporary rock anchors installed to provide lateral restraint during excavation. The basement walls would presumably be supported by the building structure in the long term.

In relation to the substation site, the worst case would be failure into the hospital excavation of a wedge extending up from the toe of the excavation face at about 45 degrees above horizontal. The southern site boundary is set back approximately 1 m behind this potential failure line, and therefore it is considered that there would be a low risk of instability and no special precautions need to be taken for the foundations.

8.2.3 Soil/Rock Anchors from Adjacent Excavations

As outlined in Section 2.2 there are existing permanent rock anchors and soil nails within the southern quarry face. The rock anchors comprise tensioned steel strands within grouted drill holes and the soil nails comprise passive (untensioned) steel bars within grouted drill holes.

On the eastern side of the southern quarry face (to the north of the site) there are soil nails within the clay profile. These soil nails are unlikely to extend much further than about 5 m from the quarry face and should be set back at least 25 m from the site.

As shown in Appendix D, the longest rock anchors for stabilisation of the previous slip area (on the western part of the southern quarry face) were proposed to be 24 m long. The slip area is located to the west of the site and the site is set back approximately 30 m from the quarry face. Specific details of the as-constructed anchors are not available, however, based on available information it is considered unlikely that these anchors extend below the site.

As outlined in Section 8.2.2, it is anticipated that temporary rock anchors will be used to support the excavation for the proposed extension to the North Shore Private Hospital. The length of the rock anchors will be determined by the actual basement design and required anchor capacity, however, it is likely that some of the rock anchors may need to extend below the substation site. It will, however, be necessary for the developer/builder to obtain permission from the adjacent property owners (i.e. Royal North Shore Hospital and Energy Australia) prior to installing rock anchors below their land. Temporary rock anchors are usually detensioned after permanent lateral support is provided by the completed basement structure. Typically the de-tensioned anchors are left in the ground. They can be removed, if necessary, however this may require specialist anchor construction. Details of the proposed anchors may be reviewed by a geotechnical engineer once the design has been finalised.

An electrical engineer should review the possible impact of steel anchors below or close to the site. There may be issues with stray currents from the proposed substation that could require further precautionary measures.

8.3 Site Preparation and Earthworks

8.3.1 Excavation Conditions

It is anticipated that excavations will be mostly through filling, clay and extremely to very low strength shale/laminite which should be achievable using conventional earthmoving equipment. Low to medium strength shale/laminite may be encountered within the deeper parts of the excavation below about 3 m depth and may require some light to moderate ripping with an excavator.

8.3.2 Disposal of Excavated Material

Under the Waste Avoidance and Resource Recovery Act (NSW EPA, 2001) a waste/fill receiving site must be satisfied that materials received meet the environmental criteria for proposed land use. This includes filling and virgin excavated natural materials (VENM), such as may be removed from this site. Accordingly, environmental testing will need to be carried out to classify spoil. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the receiving site.

8.3.3 Groundwater Seepage

Groundwater was not observed during auger drilling of the boreholes to maximum depths of 3.0 m to 4.0 m, however groundwater seepage may occur along the top of the rock or through fractures and defects in the rock mass, particularly following periods of extended wet weather. During construction, it is anticipated that groundwater seepage should be readily controlled by perimeter drains connected to a "sump-and-pump" dewatering system.

8.3.4 Dilapidation Surveys

Dilapidation surveys are often carried out on surrounding buildings, pavements and structures before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to excavation or construction related activities can be accurately assessed. The requirement for dilapidation surveys will depend on the actual depth of excavation and proximity to site boundaries. If excavations are limited to depths of 3 m to 4 m and set back 5 m or more from site boundaries then dilapidation surveys may not be necessary.

8.3.5 Vibrations

It is anticipated that the proposed excavation through the soils and underlying weathered rock will generally result in relatively minor vibrations however the site is located within the hospital grounds and therefore it is possible that sensitive structures may be located on surrounding properties. At the time of the investigation, the nearest building (with the exception of the carpark to the east) was located approximately 45 m to the south of the site.

There are no current Australian Standards for vibrations generated by construction plant. There are recommended maximum Peak Particle Velocities (PPV) in AS2187 (Explosives Code) for various structures subject to vibration. However the explosive code notes that these values are not applicable to specialist structures such as dams, reservoirs, hospitals or buildings housing scientific equipment which is sensitive to vibrations. Therefore there is no guidance as to the level of vibrations that would be acceptable.

The guidelines for vibration during excavation will need to be developed in consultation with the hospital. Any vibration level adopted should apply at the foundation level of adjacent structures with some attenuation occurring within the structures themselves to lower PPV below this value. For general structures, other than the hospital buildings, it is suggested that a maximum PPV of 5mm/ sec be adopted.

The current Department of Conservation (Environmental Protection Authority) guidelines on vibration indicate that PPV for critical working areas such as hospital operating theatres and scientific laboratories should not exceed 0.28 mm/sec. The transmission of vibration from the building foundations through the structure will depend upon the degree of damping that takes place and will also be a function of the frequency of the vibration. For this reason, it is considered essential that a vibration trial be conducted during the initial stages of construction in order to establish a site specific vibration attenuation relationship and safe operating distances for various earthmoving and excavating equipment.

8.4 Excavation Support

The excavation support requirements will depend on the actual depth of excavation and proximity to site boundaries and adjacent structures.

8.4.1 Batter Slopes

During bulk excavation, temporary batter slopes should be battered at no steeper than 1.5 Horizontal (H) : 1 Vertical (V) within filling and clay and 1H:1V within hard shaly clay and weathered rock. Permanent batters, if required, should be battered at no steeper than 2H:1V within filling and clay and 1H:1V within shaly clay and weathered rock. Permanent batters should be protected from erosion and deterioration by shotcrete cover (or similar). A minimum shotcrete thickness of 80 mm should be adopted unless stability issues dictate a greater thickness is required.

8.4.2 Shoring/Retaining Walls

Where batter slopes cannot be accommodated the excavation will require temporary shoring support and permanent retaining walls as part of the final construction. Shoring may comprise a bored soldier pile wall with shotcrete or timber infill panels between piles. Typically, soldier piles are spaced at approximately 2 m to 3 m centres however closer spaced piles may be required to limit wall movements or collapse of infill materials where structures or services are located in close proximity to the excavation. Generally shotcrete panels should be constructed in 2 m depth intervals within the soil and highly weathered rock. Shoring piles should be founded at least 0.3 m below the bulk excavation level and may be used to carry vertical structural loads.

Shoring/retaining walls may be designed on the basis of an average unit weight of 20 kN/m^3 and 22 kN/m^3 for soil and rock respectively, and a triangular earth pressure distribution based on lateral earth pressure coefficients as given in Table 2. Active earth pressure coefficients (Ka) may be used where some wall movement is acceptable. At rest earth pressure coefficients (Ko) should be used where wall movement is to be minimised such as close to structures or buried services and where the wall is propped or braced.

Table 2 - Suggested Lateral Earth Pressures Coefficients

Material	Lateral Earth Pressure Coefficients	
	Active (Ka)	**At Rest (Ko)**
Soil (filling, clay and shaly clay)	0.35	0.5
Extremely Low to Very Low strength rock	0.3	0.45

Drainage of the ground behind impermeable walls should be provided otherwise the wall should be designed for full hydrostatic pressures. Drainage could comprise 150 mm wide strip drains pinned to the face at 2 m centres behind shotcrete in-fill panels. The base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain which is connected to a sump dewatering system.

All surcharge loads should be allowed for in the retaining wall design including new building footings and slabs, traffic and construction related activities.

Passive resistance for piles founded below the base of the excavation may be based on an allowable passive resistance of 200 kPa for extremely low strength rock and 300 kPa for very low to low strength rock. Passive resistance should be assumed to start at least 0.5 m below bulk excavation level.

8.4.3 Ground Anchors

The design of temporary and permanent ground anchors, if required, for the support of excavations and/or shoring systems may be carried out on the basis of a maximum allowable bond stress of 80 kPa in extremely low strength rock and 100 kPa in very low to low strength rock.

Anchor holes should be clean and adequately flushed and the anchor should be bonded behind a line drawn up at 45° from the base of the bulk excavation. Higher bond stress values may be adopted if trial anchors are used to prove higher capacities. Care should be taken to avoid damaging buried services or pipes during anchor installation. In addition, possible interaction with temporary anchors used during construction of the extension to the North Shore Private Hospital may need to be considered.

8.5 Foundations

Following bulk excavation to depths of 3 m to 4 m it is anticipated that extremely low to very low strength shale will be exposed over most of the basement footprint. In areas where relatively minor excavation is carried out it is likely that residual clay and shaly clay may be exposed or at shallow depth.

Preferably all structural loads should be uniformly supported on underlying bedrock for which pad footings should be appropriate. Deepened pad footings or bored piles may be required in areas where relatively minor excavation is carried out. It is expected that uncased bored piles could be used for the site.

If some isolated structures are located in areas where residual clay is exposed then it may be appropriate to adopt pad footings within stiff clay or better. It will, however, be important to ensure that structures founded on clay are isolated from structures founded on rock due to potential differential settlement/movement between the soil and rock foundations.

Depending on the final design and basement layout it is possible that some footings may be required behind the basement excavation. In this case, the footings should be founded below a line extending upwards at an angle of 45 degrees from the base of adjacent excavations.

Recommended maximum allowable pressures for the various foundation materials encountered within the boreholes are presented in Table 3. These parameters apply to the design of spread foundations, such as pads or strip footings, or for rock socketed bored piles.

Table 3 – Recommended Design Parameters and Modulus Values for Foundation Design

Foundation Stratum	**Classification**	**Maximum Allowable Pressure**		**Field Elastic Modulus (MPa)**
		End Bearing (kPa)	**Shaft Adhesion (kPa)**	
Residual Clay	Stiff or better	150	-	30
Laminite or Shale	Extremely Low Strength	700	70	100
	Very Low to Low Strength	1000	100	200
	Low to Medium Strength	2000	200	350

Foundations proportioned on the basis of the above parameters would be expected to experience total settlements of less than 1% of the footing width (or pile diameter) under the applied working load, with differential settlements between adjacent columns/footings expected to be less than half of this value.

All footings should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters.

8.6 Pavements & Floor Slabs

During construction of pavements and access roads outside the basement area it is recommended that all topsoil, organic and deleterious material should be stripped and stockpiled separately for disposal or use in landscaping areas. Proof rolling of the exposed subgrade should be carried out under the supervision of a geotechnical engineer to detect any soft or heaving areas. Any soft spots detected during proof rolling would need to be stripped to a stiff base and replaced with engineered filling.

Engineered filling should be placed in maximum 200 mm thick loose layers and compacted to a minimum dry density ratio (DDR) of 98% Standard compaction with moisture contents within 2% of optimum moisture content (OMC). The compaction should be increased to a dry density ratio of 100% Standard compaction within 0.3 m of the subgrade surface. The existing filling, clay and excavated rock on site should generally be suitable for re-use as engineered filling provided it has a maximum particle size of 70 mm and moisture content within 2% of OMC (where possible, preference should be given to the use of granular material).

Subject to the subgrade preparation outlined above, the design of pavements on clay subgrade may be based on a CBR value of 3%. Design of pavements on weathered rock may be based on a CBR value of 5% for extremely low strength rock and 10% for very low to low strength rock. These CBR values assume all pavements are protected by adequate surface and subsoil drainage to minimise the risk of water infiltration and softening of pavement materials.

As outlined in Section 8.2.1, the south-eastern corner of the quarry (directly to the north of the site) may not have a sufficient factor of safety for the heavy loads associated with transporting large transformers. In this area it is recommended that the access roads be located at least 3 m back from the top of the quarry face to minimise the risk of failure of the upper clay soils. Alternatively the roads could be supported on piers taken down into the shale.

8.7 Seismic Design

In accordance with the AS1170.4-1993 (Earthquake Loading) an acceleration coefficient (a) of 0.08 and a site factor of 1.0 are applicable for the site, assuming that all major structural loads are carried to below bulk excavation level and founded on rock of at least extremely low to very low strength.

DOUGLAS PARTNERS PTY LTD

Reviewed by

Scott Easton
Associate

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Principal

APPENDIX A

Notes Relating to this Report
Results of Field Work

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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.

AN ENGINEERING CLASSIFICATION OF SEDIMENTARY ROCKS IN THE SYDNEY AREA

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

Under this system rocks are classified by Rock Type, Degree of Weathering, Strength, Stratification Spacing, and Degree of Fracturing. These terms do not cover the full range of engineering properties. Descriptions of rock may also need to refer to other properties (e.g. durability, abrasiveness, etc.) where these are relevant.

ROCK TYPE DEFINITIONS

Rock Type	Definition
Conglomerate:	More than 50% of the rock consists of gravel sized (greater than 2mm) fragments
Sandstone:	More than 50% of the rock consists of sand sized (.06 to 2mm) fragments
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06mm) 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, e.g. clayey sandstone, sandy shale.

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 and texture 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	Fs	Rock substance unaffected by weathering, limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

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

ROCK STRENGTH

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

Strength Term	Is(50) MPa	Field Guide	Approx. qu MPa*
Extremely Low:	0.03	Easily remoulded by hand to a material with soil properties	0.7
Very Low:	0.1	May be crumbled in the hand. Sandstone is "sugary" and friable.	2.4
Low:	0.3	A piece of core 150 mm long x 50 mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.	7
Medium:	1	A piece of core 150 mm long x 50 mm dia. can be broken by hand with considerable difficulty. Readily scored with knife.	24
High:	3	A piece of core 150 mm long x 50 mm dia. cannot be broken by unaided hands, can be slightly scratched or scored with knife.	70
Very High:	10	A piece of core 150 mm long x 50 mm dia. may be broken readily with hand held hammer. Cannot be scratched with pen knife.	240
Extremely High:		A piece of core 150 mm long x 50 mm dia. is difficult to break with hand held hammer. Rings when struck with a hammer.	

* The approximate unconfined compressive strength (qu) shownin the table is based on an assumed ratio to the point load index of 24:1. This ratio may vary widely.

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

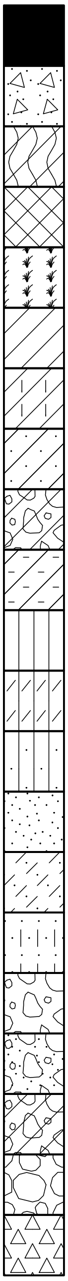
Term	Description
Fragmented:	The core is comprised primarily of fragments of length less than 20 mm, and mostly of width less than the core diameter.
Highly Fractured:	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured:	Core lengths are mainly 30 mm - 100 mm with occasional shorter and longer sections.
Slightly Fractured:	Core lengths are generally 300 mm - 1000 mm with occasional longer sections and occasional sections of 100 mm - 300 mm.
Unbroken:	The core does not contain any fracture.

REFERENCE

International Society of Rock Mechanics, Commission on Standardisation of Laboratory and Field Tests, Suggested Methods for Determining the Uniaxial Compressive Strength of Rock Materials and the Point Load Strength Index, Committee on Laboratory Tests Document No. 1 Final Draft October 1972

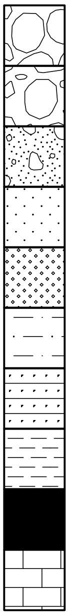
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL



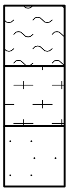
BITUMINOUS CONCRETE
CONCRETE
TOPSOIL
FILLING
PEAT
CLAY
SILTY CLAY
SANDY CLAY
GRAVELLY CLAY
SHALY CLAY
SILT
CLAYEY SILT
SANDY SILT
SAND
CLAYEY SAND
SILTY SAND
GRAVEL
SANDY GRAVEL
CLAYEY GRAVEL
COBBLES/BOULDERS
TALUS

SEDIMENTARY ROCK



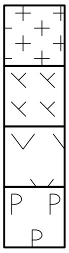
BOULDER CONGLOMERATE
CONGLOMERATE
CONGLOMERATIC SANDSTONE
SANDSTONE FINE GRAINED
SANDSTONE COARSE GRAINED
SILTSTONE
LAMINITE
MUDSTONE, CLAYSTONE, SHALE
COAL
LIMESTONE

METAMORPHIC ROCK

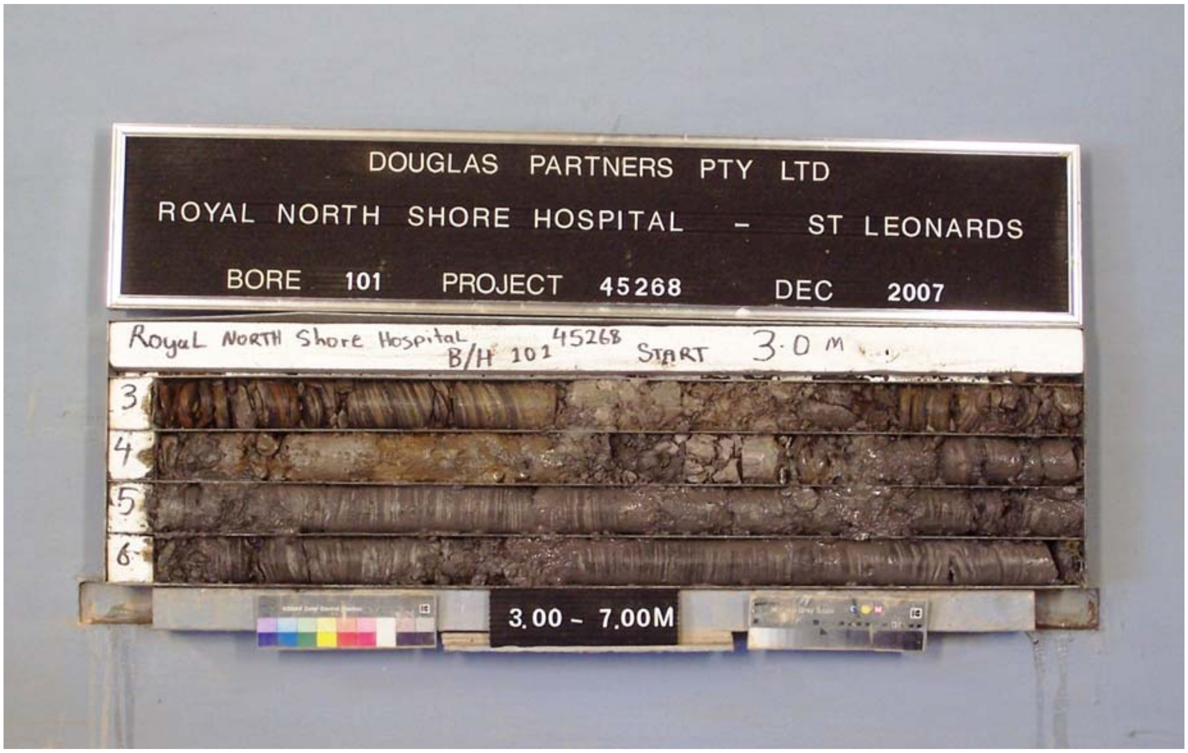


SLATE, PHYLITTE, SCHIST
GNEISS
QUARTZITE

IGNEOUS ROCK



GRANITE
DOLERITE, BASALT
TUFF
PORPHYRY


DOUGLAS PARTNERS PTY LTD
ROYAL NORTH SHORE HOSPITAL - ST LEONARDS
BORE 101 PROJECT 45268 DEC 2007
Royal NORTH Shore Hospital 45268 B/H 101 START 3.0 M
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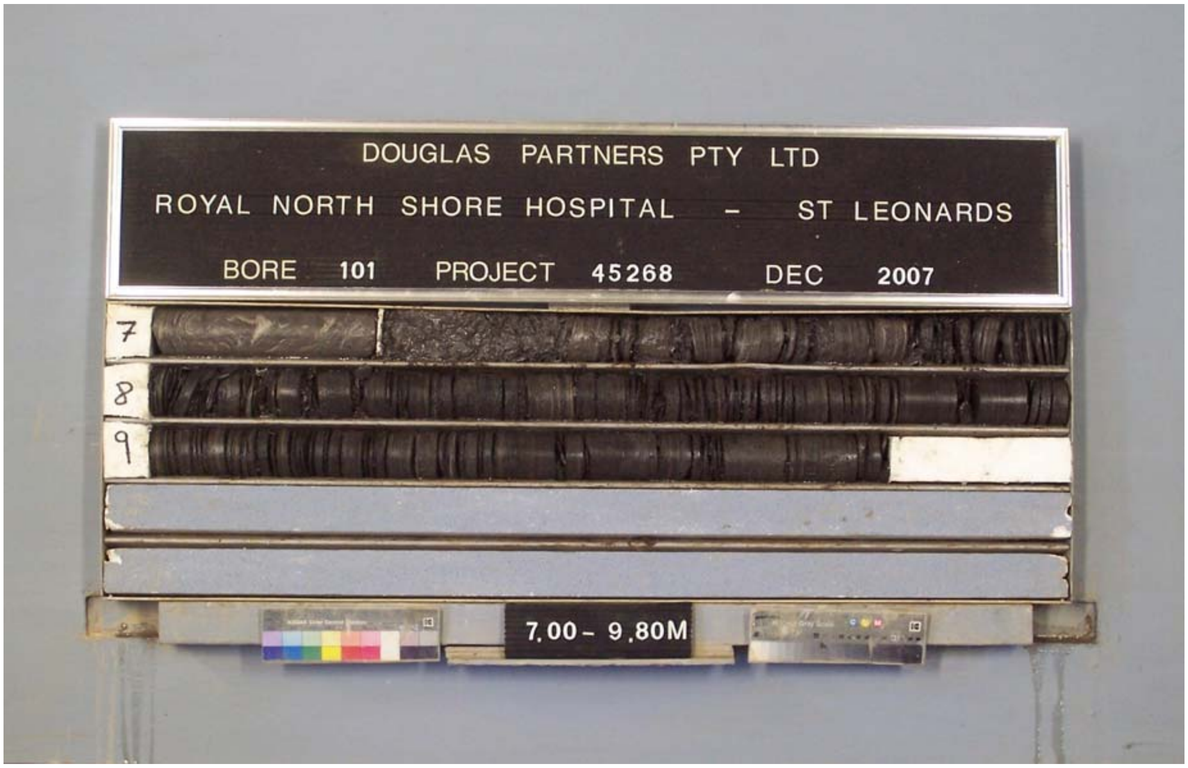
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ROYAL NORTH SHORE HOSPITAL - ST LEONARDS
BORE 101 PROJECT 45268 DEC 2007
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7.00 - 9.80M

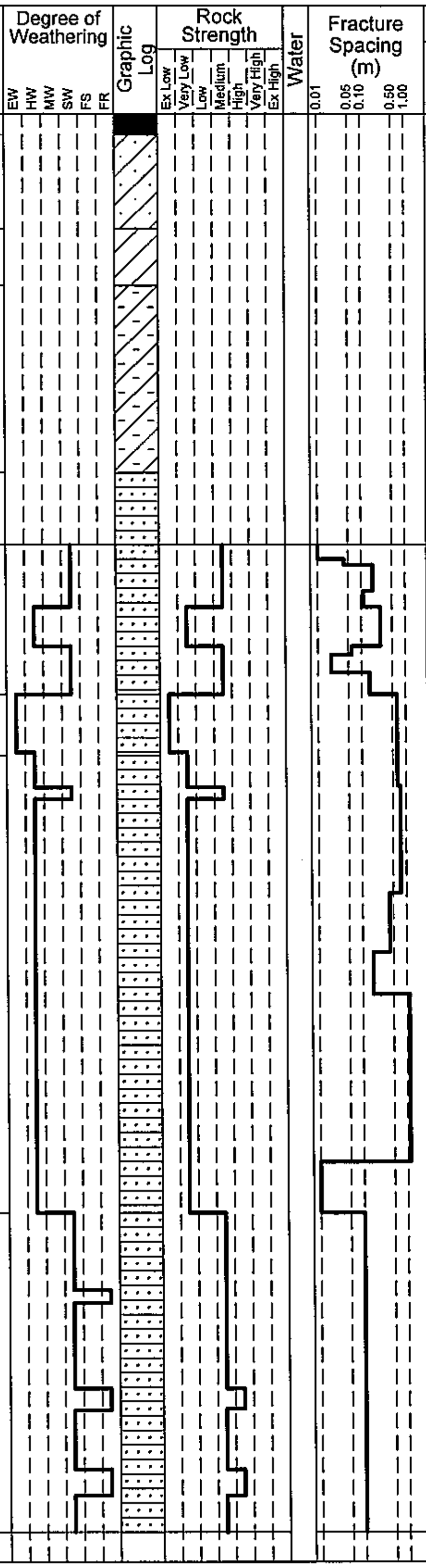
BOREHOLE LOG

CLIENT: Energy Australia
PROJECT: Proposed Substation
LOCATION: Royal North Shore Hospital, St Leonards

SURFACE LEVEL: 93.2 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 45268
DATE: 29 Nov 07
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
	0.15	ASPHALTIC CONCRETE - 30mm thick over roadbase		A			
		SANDY CLAY - brown sandy clay. Damp		A			
	0.8	CLAY - very stiff, orange-brown and light grey clay with some ironstone		A S			8,13,23 N = 36
	1.2	SHALY CLAY - hard, brown grey shaly clay	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping 0°- 10°				
	2.5	LAMINITE - low strength, grey laminite with some ironstone bands		S			26,25/50mm refusal
	3.0	LAMINITE - medium strength, slightly weathered, fragmented to slightly fractured, grey brown laminite with some sandstone laminations 3.43-3.7m: very low strength band	3.7m: J70°	C	100	21	PL(A) = 0.4MPa
	4.03	LAMINITE - extremely low strength, extremely weathered, light grey and grey laminite					
	4.45	LAMINITE - very low strength, highly weathered, fractured to slightly fractured, grey laminite	7.0-7.25m: abnormal bedding due to possible shear zone 7.2-7.4m: fragmented zone, possibly crushed zone	C	100	0	
	7.6	LAMINITE - medium strength, slightly weathered and fresh, fractured to slightly fractured, grey and dark grey laminite with some sandstone laminations and high strength bands		C	100	17	PL(A) = 0.8MPa PL(A) = 0.4MPa PL(A) = 0.4MPa PL(A) = 1.5MPa
	9.8	Bore discontinued at 9.8m					



RIG: Bobcat **DRILLER:** Eric/Steven **LOGGED:** PH/SI **CASING:** Uncased
TYPE OF BORING: 110mm auger to 3.0m; NMLC-Coring to 9.8m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ⫮ Water level

CHECKED
Initials: STE
Date: 13/2/08

Douglas Partners
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BOREHOLE LOG

CLIENT: Energy Australia
PROJECT: Proposed Substation
LOCATION: Royal North Shore Hospital, St Leonards

SURFACE LEVEL: 96.0 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 45268
DATE: 29 Nov 07
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
96		ASPHALTIC CONCRETE - 30mm thick over roadbase	■	A	0.1				
	0.15	SANDY CLAY - brown sandy clay with some gravel. Dry	▨						
	0.5	CLAY - stiff, orange-red and light grey clay with some ironstone	▨	A	0.5				
95	1.0	SHALY CLAY - hard, light grey shaly clay, with some ironstone bands	▨	A S	1.0 1.12		25/120mm refusal		1
94	2.0	SHALE - extremely low strength, light grey shale	▤	S	2.5 2.6		25/100mm refusal		2
93	3.0	LAMINITE - low strength, dark grey laminite	▦						3
92	4.0	Bore discontinued at 4.0m - limit of investigation		S	4.0 4.04		25/40mm refusal		4
91	5								5
90	6								6
89	7								7
88	8								8
87	9								9

RIG: Bobcat
DRILLER: Eric/Steven
LOGGED: PH
CASING: Uncased
TYPE OF BORING: 110mm auger
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 13/2/08

Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Energy Australia
PROJECT: Proposed Substation
LOCATION: Royal North Shore Hospital, St Leonards

SURFACE LEVEL: 97.0 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 45268
DATE: 29 Nov 07
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
97	0.15	ASPHALTIC CONCRETE - 30mm thick over roadbase		A	0.1				
		SANDY CLAY - brown sandy clay		A A	0.5 0.8				
96	0.9	SHALY CLAY - hard, orange and light grey shaly clay with ironstone bands		S	1.0 1.21		14,25/60mm refusal		1
95	2.0	SHALE - extremely low strength, light grey shale		S	2.5 2.6		25/100mm refusal		2
94									3
	3.5	LAMINITE - low strength, dark grey and light grey laminite							
93	4.0	Bore discontinued at 4.0m - limit of investigation		S	4.0		16,25/100mm refusal		4
92	5								5
91	6								6
90	7								7
89	8								8
88	9								9

RIG: Bobcat **DRILLER:** Eric/Steven **LOGGED:** PH **CASING:** Uncased
TYPE OF BORING: 110mm auger
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 13/2/08

Douglas Partners
Geotechnics · Environment · Groundwater


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ROYAL NORTH SHORE HOSPITAL - ST LEONARDS
BORE 104 PROJECT 45268 DEC 2007
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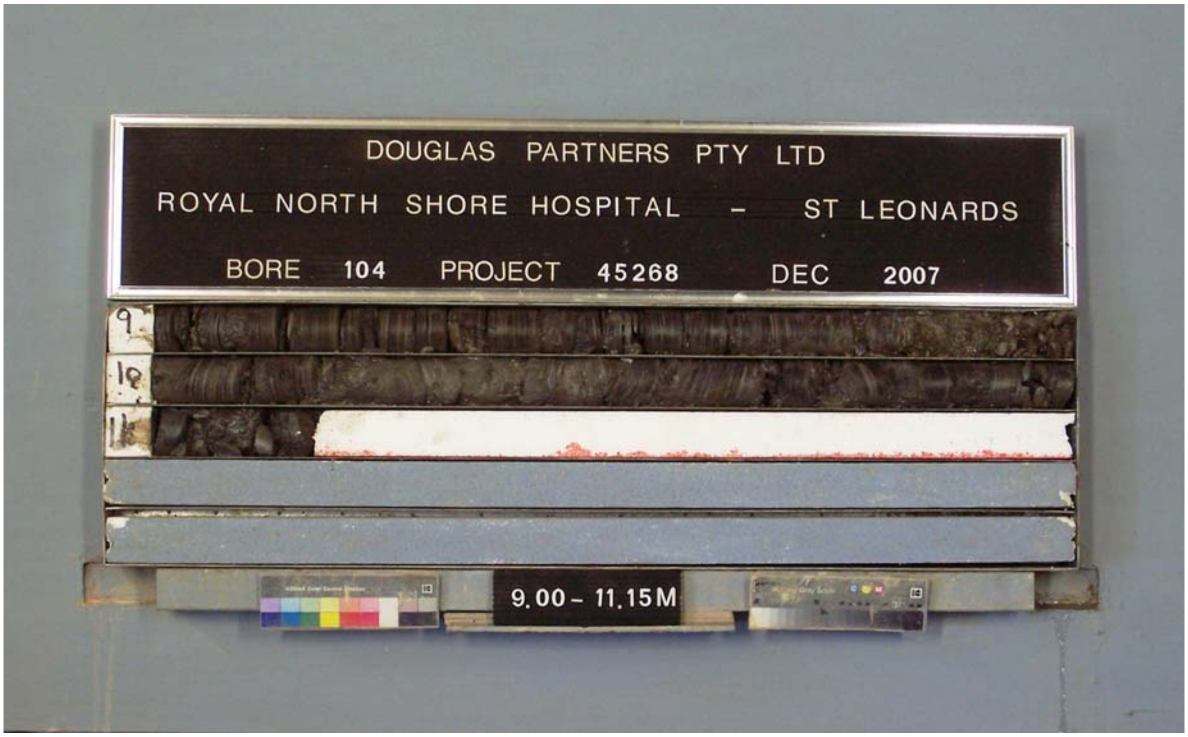
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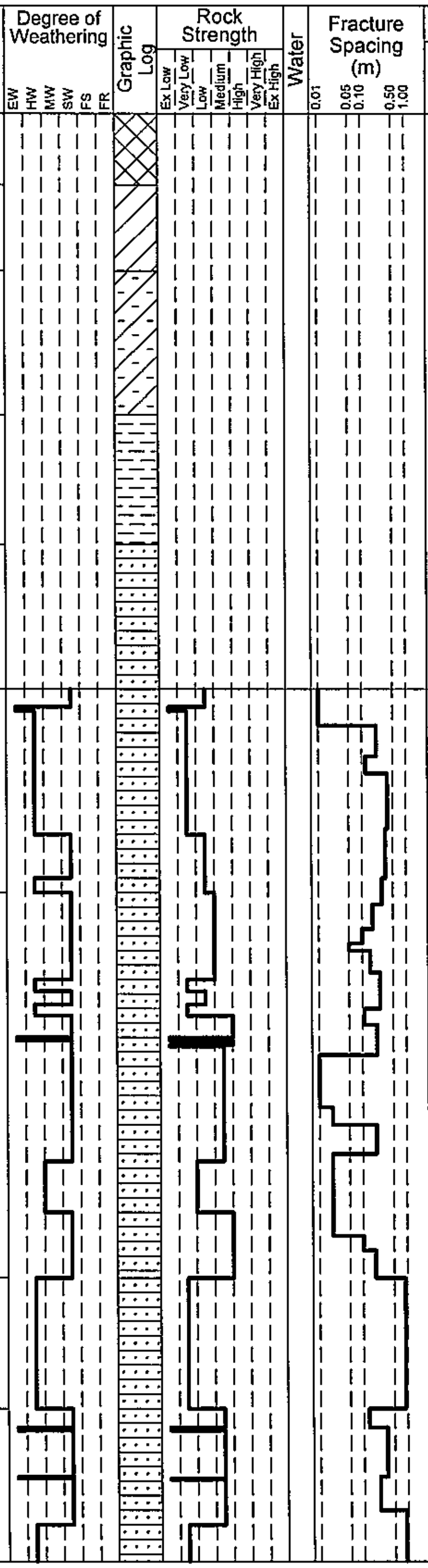
ROYAL NORTH SHORE HOSPITAL - ST LEONARDS
BORE 104 PROJECT 45268 DEC 2007
9
10
11
9.00 - 11.15M

BOREHOLE LOG

CLIENT: Energy Australia
PROJECT: Proposed Substation
LOCATION: Royal North Shore Hospital, St Leonards

SURFACE LEVEL: 95.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 45268
DATE: 29 Nov 07
SHEET 1 **OF** 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
95		FILLING - dark brown sandy clay filling, with some organic material. Damp		A A			
	0.5	CLAY - very stiff, orange and light grey clay. Damp					
94	1.1	SHALY CLAY - hard, light grey shaly clay, with some sand and ironstone gravel. Dry		S			4,8,12 N = 20
93	2.1	SHALE - extremely low strength, light grey shale with some ironstone bands		S			19,20,24 N = 44
92	3.0	LAMINITE - low strength, dark grey laminite. Dry	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping 0°- 10°				
91	4.0	LAMINITE - very low and low strength, highly and slightly weathered, fragmented then slightly fractured, grey and brown laminite		C	100	73	PL(A) = 0.2MPa
90 89 88	5.4	LAMINITE - low to medium then medium strength, slightly weathered, fragmented to slightly fractured, grey and brown laminite with some extremely low and very low strength bands	5.65m: J60° 5.75m: J75° 6.2m: J75°				PL(A) = 0.3MPa PL(A) = 0.6MPa
		7.6-8.05m: medium to high strength	7.86m: J30°	C	100	45	PL(A) = 1MPa
87	8.05	LAMINITE - very low strength, highly weathered, grey laminite	8.05m: J45°				
86	8.95	LAMINITE - medium strength, slightly weathered, slightly fractured, grey and dark grey laminite with some extremely low and very low strength bands	9.08m: B0°, 10mm clay 9.42m: B0°, 10mm clay	C	100	78	PL(A) = 0.5MPa

RIG: Bobcat **DRILLER:** Eric/Steven **LOGGED:** PH/SI **CASING:** Uncased
TYPE OF BORING: 110mm auger to 4.0m; NMLC-Coring to 11.2m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 13/2/08

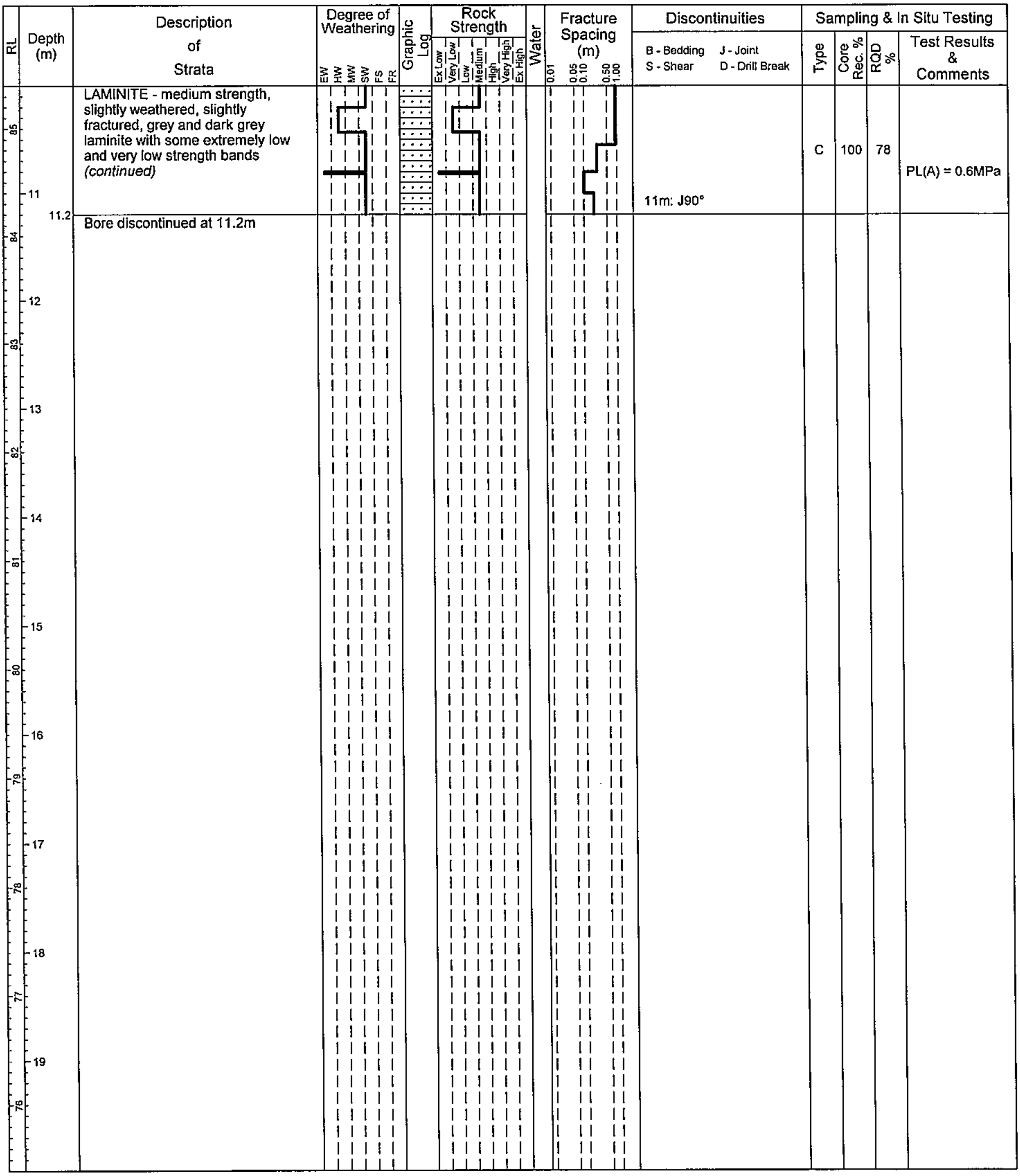
Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Energy Australia
PROJECT: Proposed Substation
LOCATION: Royal North Shore Hospital, St Leonards

SURFACE LEVEL: 95.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 45268
DATE: 29 Nov 07
SHEET 2 **OF** 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
		LAMINITE - medium strength, slightly weathered, slightly fractured, grey and dark grey laminite with some extremely low and very low strength bands *(continued)*	11m: J90°	C	100	78	PL(A) = 0.6MPa
	11.2	Bore discontinued at 11.2m					

RIG: Bobcat **DRILLER:** Eric/Steven **LOGGED:** PH/SI **CASING:** Uncased

TYPE OF BORING: 110mm auger to 4.0m; NMLC-Coring to 11.2m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ⊻ Water level

CHECKED
Initials: STE
Date: 13/2/08

Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT:	Energy Australia	**SURFACE LEVEL:** 95.0 AHD	**BORE No:** 105
PROJECT:	Proposed Substation	**EASTING:**	**PROJECT No:** 45268
LOCATION:	Royal North Shore Hospital, St Leonards	**NORTHING:**	**DATE:** 29 Nov 07
		DIP/AZIMUTH: 90°/--	**SHEET** 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
95	0.15	ASPHALTIC CONCRETE - 30mm thick over roadbase	■	A	0.1				
		FILLING - dark grey sandy clay filling with some gravel, glass and building rubble. Damp	▩	A	0.5				
94	0.8 1	CLAY - stiff, orange and light grey clay with some ironstone	▨	S	1.0 1.45		3,5,5 N = 10		1
93	2 2.0	SHALY CLAY - hard, light grey shaly clay with ironstone bands	▨	S	2.5 2.78		13,25/100mm refusal		2
92	3								3
	3.5	LAMINITE - low strength, dark grey laminite	▤						
91	4 4.0	Bore discontinued at 4.0m - limit of investigation		S	4.0 4.12		25/120mm refusal		4
90	5								5
89	6								6
88	7								7
87	8								8
86	9								9

RIG: Bobcat **DRILLER:** Eric/Steven **LOGGED:** PH **CASING:** Uncased

TYPE OF BORING: 110mm auger

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▿ Water level

CHECKED
Initials: STE
Date: 13/2/08

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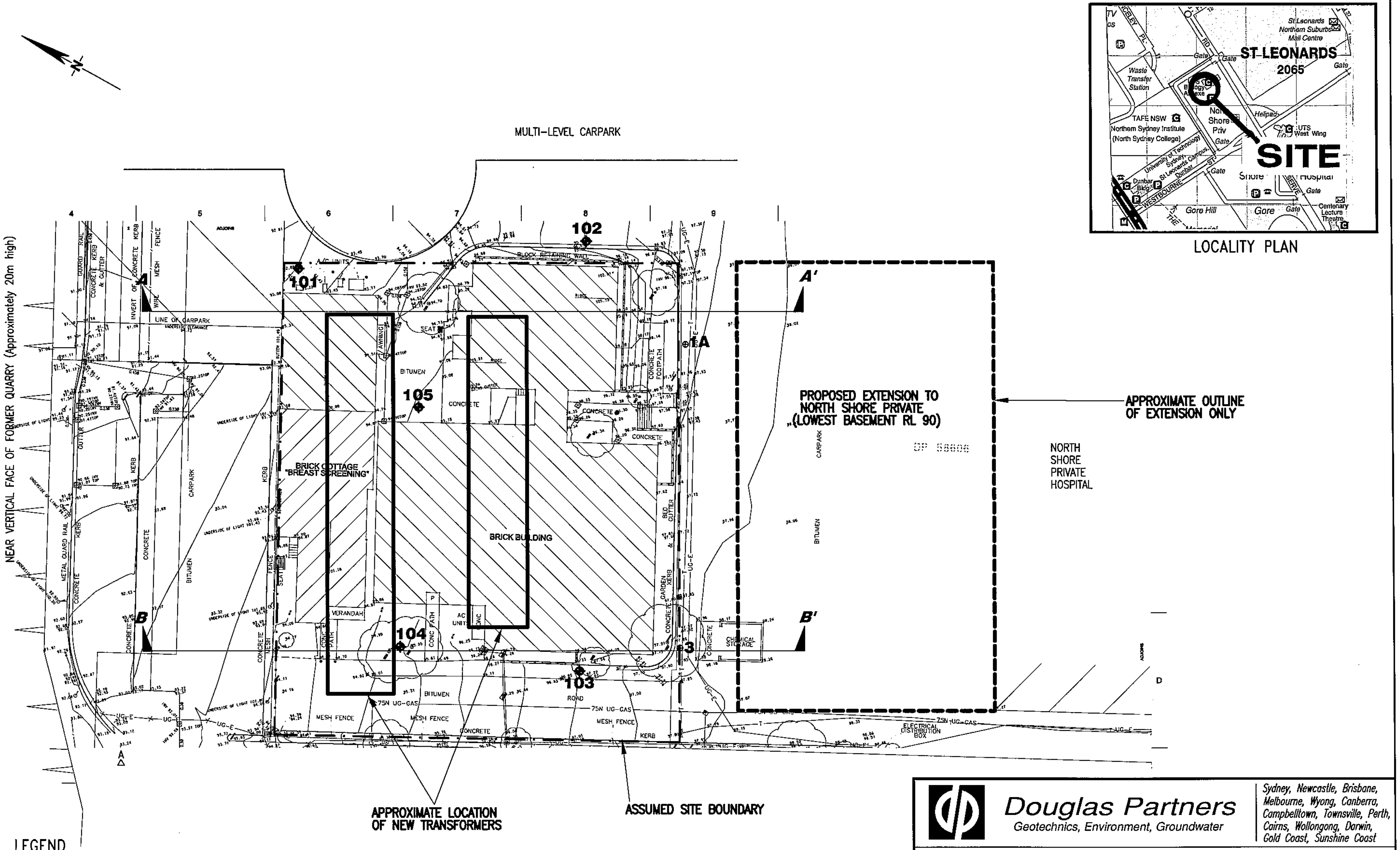
Geotechnics · Environment · Groundwater

APPENDIX B

Drawing 1 - Location of Tests

Drawing 2 - Geotechnical Cross Section (A-A')

Drawing 3 - Geotechnical Cross Section (B-B')



LEGEND

- ◆ CURRENT BOREHOLE LOCATION
- ⊕ PREVIOUS BOREHOLES (Project 44433-1 June 2007)
- △ APPROXIMATE LOCATION OF PREVIOUS BOREHOLE (Project 9730/1 Sept 1986)

SCALE 0 10 20 30 40 50m
1:500

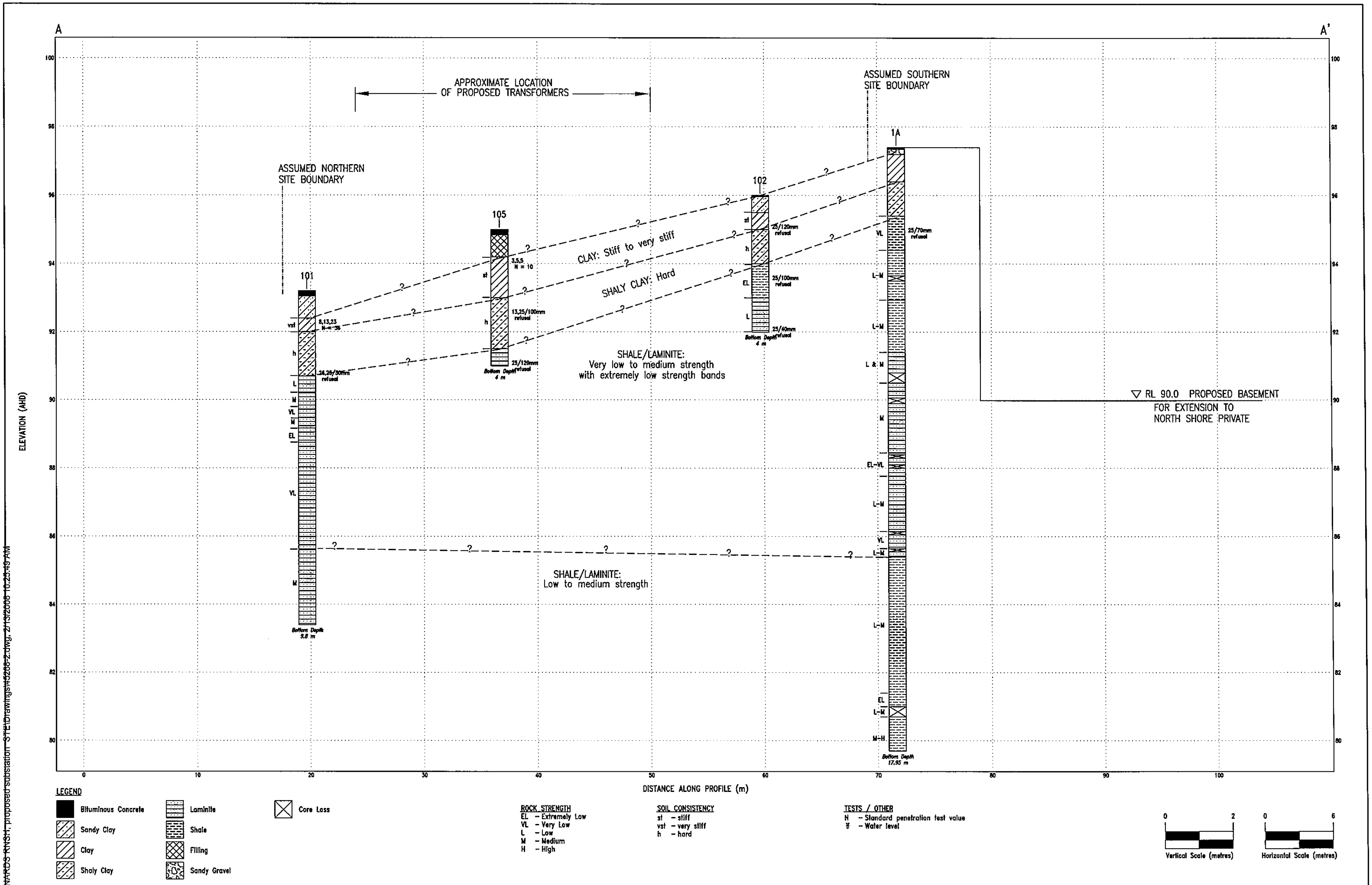
Refer to Drawing 2 and 3 for Sections A–A' and B–B'

Douglas Partners
Geotechnics, Environment, Groundwater

Sydney, Newcastle, Brisbane, Melbourne, Wyong, Canberra, Campbelltown, Townsville, Perth, Cairns, Wollongong, Darwin, Gold Coast, Sunshine Coast

TITLE:
Location of Tests
Proposed Substation
Royal North Shore Hospital
ST LEONARDS

CLIENT: Energy Australia			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 45268	OFFICE: SYDNEY
APPROVED BY: STE		DATE: 18.12.2007	DRAWING No: 1



Douglas Partners
Geotechnics · Environment · Groundwater

CLIENT: Energy Australia

PROJECT: Proposed Substation

APPROVED BY: STE

DATE: 8 February 08

REVISION:

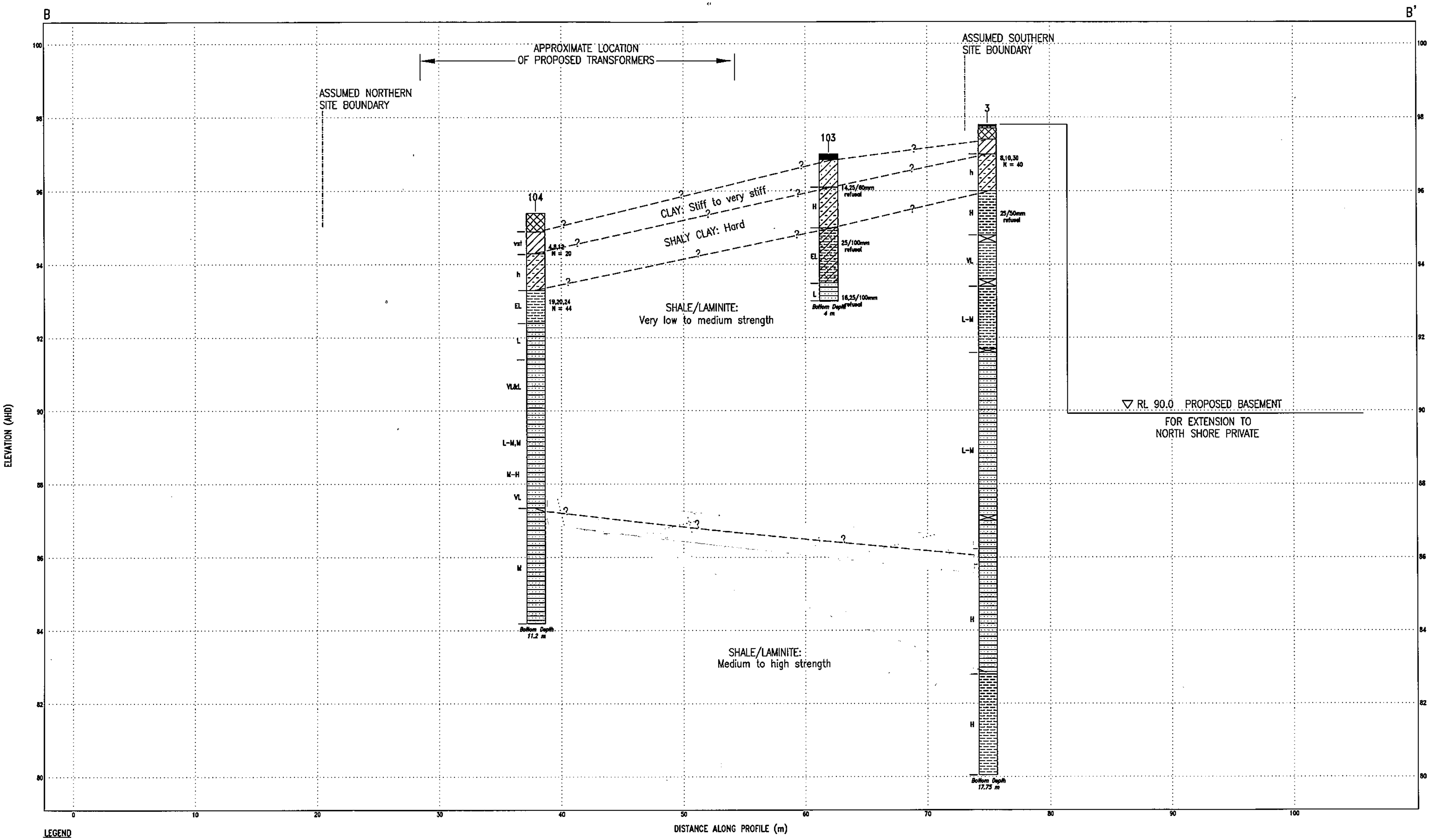
TITLE: Section A-A'
Royal North Shore Hospital

SCALE: AS SHOWN

PROJECT No: 45268

DRAWING No: 2

P:\45268 ST LEONARDS RNSH, proposed substation - STE\Drawings\45268-2.dwg, 2/13/2008 10:23:49 AM



LEGEND

■ Bituminous Concrete
▨ Sandy Gravel
▩ Filling
▨ Clay
▨ Shaly Clay
▤ Shale
☒ Core Loss
▤ Laminite
▨ Sandy Clay

ROCK STRENGTH
EL – Extremely Low
VL – Very Low
L – Low
M – Medium
H – High
VH – Very High

SOIL CONSISTENCY
st – stiff
vst – very stiff
h – hard

TESTS / OTHER
N – Standard penetration test value
≡ – Water level

0 2
Vertical Scale (metres)

0 6
Horizontal Scale (metres)

Douglas Partners
Geotechnics · Environment · Groundwater

CLIENT: Energy Australia
PROJECT: Proposed Substation
APPROVED DP: STE
DATE: 8 February 08
REVISION:

TITLE: Section B-B'
Westbourne Street, St Leonards
SCALE: AS SHOWN
PROJECT No: 45268
DRAWING No: 3

P:\45268 ST LEONARDS RNSH, proposed substation - STE\Drawings\45268-3.dwg, 2/13/2008 10:36:47 AM

APPENDIX C

Results of Laboratory Tests

Douglas Partners
Geotechnics · Environment · Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117

96 Hermitage Road
West Ryde NSW 2114
Australia

PO Box 472
West Ryde NSW 1685

Phone ***(02) 9809 0666***
Fax: *(02) 9809 4095*
sydney@douglaspartners.com.au

RESULTS OF MOISTURE CONTENT, PLASTICITY AND LINEAR SHRINKAGE TESTS

Client: ENERGY AUSTRALIA
Project: ST LEONARDS
Location: ST LEONARDS

Project No: 45268
Report No: S08-026
Report Date: 19/02/08

Date Sampled: NA
Date of Test: 14/02/08
Page: 1 of 1

TEST LOCATION	DEPTH (m)	DESCRIPTION	CODE	W_F %	W_L %	W_P %	PI %	*LS %
101	1.0	CLAY - Orange and light grey clay	2,5		36	22	14	6.5
105	2.5	SHALY CLAY - Light grey shaly clay	2,5		37	21	16	7

Legend:

W_F	Field Moisture Content
W_L	Liquid limit
W_P	Plastic limit
PI	Plasticity index
LS	Linear shrinkage from liquid limit condition (Mould length 150mm)

Test Methods:

Moisture Content:	AS 1289 2.1.1 - 2005
Liquid Limit:	AS 1289 3.1.2 - 1995, 3.1.1 - 1995
Plastic Limit:	AS 1289 3.2.1 - 1995
Plasticity Index:	AS 1289 3.3.1 - 1995
Linear Shrinkage:	AS 1289 3.4.1 - 1995
Cone Liquid Limit:	AS 1289 3.9.1 - 2002 AS 1289.1.3.1 - 1999

Code
Sample history for plasticity tests
1. Air dried
2. Low temperature (<50°C) oven dried
3. Oven (105°C) dried
4. Unknown

Method of preparation for plasticity tests
5. Dry sieved
6. Wet sieved
7. Natural

*Specify if sample crumbled CR or curled CU

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

Remarks:

Approved Signatory:

Norman Weimann
Laboratory Manager

Tested: AR
Checked: NW

NATA NATA Accredited Laboratory Number: 828
This Document is issued in accordance with NATA's accreditation requirements.
Accredited for compliance with ISO/IEC 17025
ACCREDITED FOR **TECHNICAL COMPETENCE**

Envirolab Services Pty Ltd
ABN 37 112 535 645
54 Frenchs Rd Willoughby NSW 2068
ph 02 9958 5801 fax 02 9958 5803
email: tnotaras@envirolabservices.com.au

CERTIFICATE OF ANALYSIS 16948

Client:
Douglas Partners
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Scott Eastern

Sample log in details:

Your Reference:	**45268, St Leonards**
No. of samples:	3 Soils
Date samples received:	12/02/08
Date completed instructions received:	12/02/08

Analysis Details:

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

Report Details:

Date results requested by:	19/02/08
Date of Preliminary Report:	Not Issued
Issue Date:	19/02/08

NATA accreditation number 2901. This document shall not be reproduced except in full.
This document is issued in accordance with NATA's accreditation requirements.
Accredited for compliance with ISO/IEC 17025.
Tests not covered by NATA are denoted with *.

Results Approved By:

Jacinta Hurst
Operations Manager

NATA
ACCREDITED FOR
TECHNICAL
COMPETENCE

Miscellaneous Inorg - soil Our Reference: Your Reference Type of sample	UNITS ------------- ------------	16948-1 102-1.0 Soil	16948-2 102-2.5 Soil	16948-3 102-4.0 Soil
Date analysed	-	15/02/2008	15/02/2008	15/02/2008
pH 1:5 soil:water	pH Units	4.4	5.3	5.3
Chloride 1:5 soil:water	mg/kg	<100	<100	<100
Sulphate, SO4 1:5 soil:water	mg/kg	120	140	110

Method ID	Methodology Summary
LAB.1	pH - Measured using pH meter and electrode in accordance with APHA 20th ED, 4500-H+.
LAB.11	Chloride determined by argentometric titration.
LAB.9	Sulphate determined turbidimetrically.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
pH 1:5 soil:water	pH Units		LAB.1	[NT]	[NT]	[NT]	LCS-W1	100%
Chloride 1:5 soil:water	mg/kg	100	LAB.11	<100	[NT]	[NT]	LCS-W1	91%
Sulphate, SO4 1:5 soil:water	mg/kg	25	LAB.9	<25	[NT]	[NT]	LCS-W1	114%

Report Comments:

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

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

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike: A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample): This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

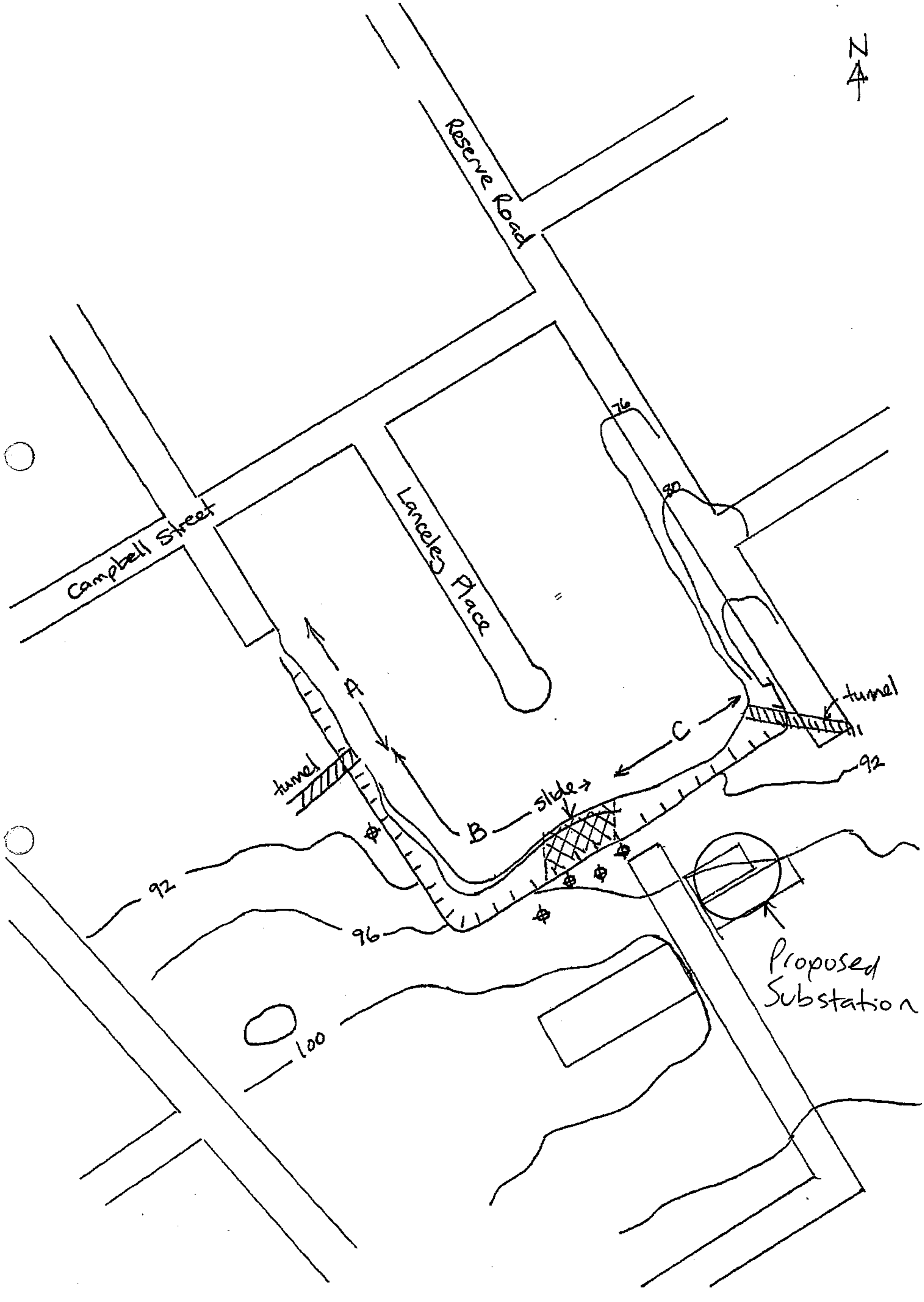
Laboratory Acceptance Criteria:
Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes and LCS: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable. Surrogates: 60-140% is acceptable for general organics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

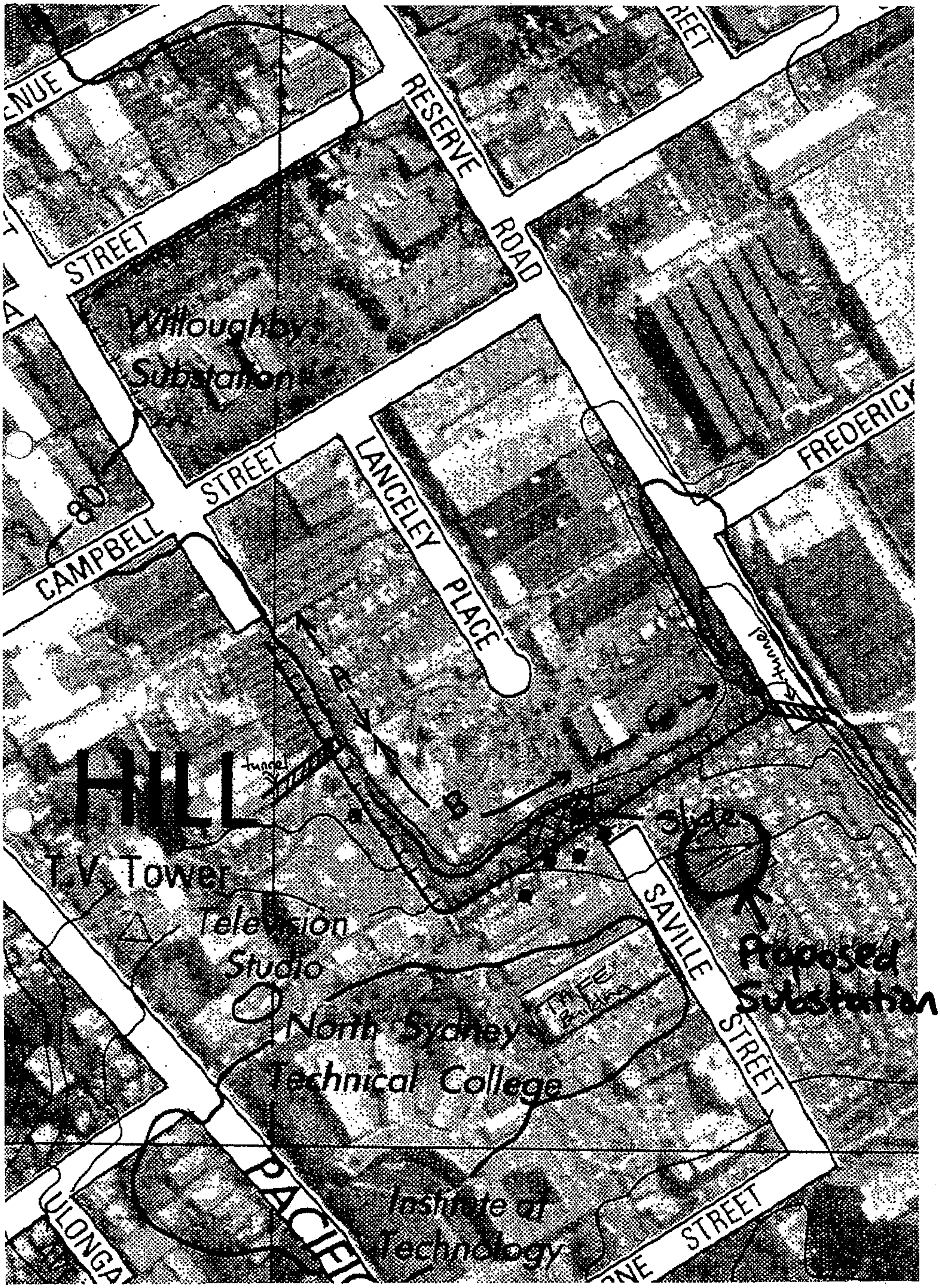
APPENDIX D

Results of Previous Investigations

Location of Quarry and Slip Area

Sketch Of Proposed Stabilisation of the Quarry Slip Area

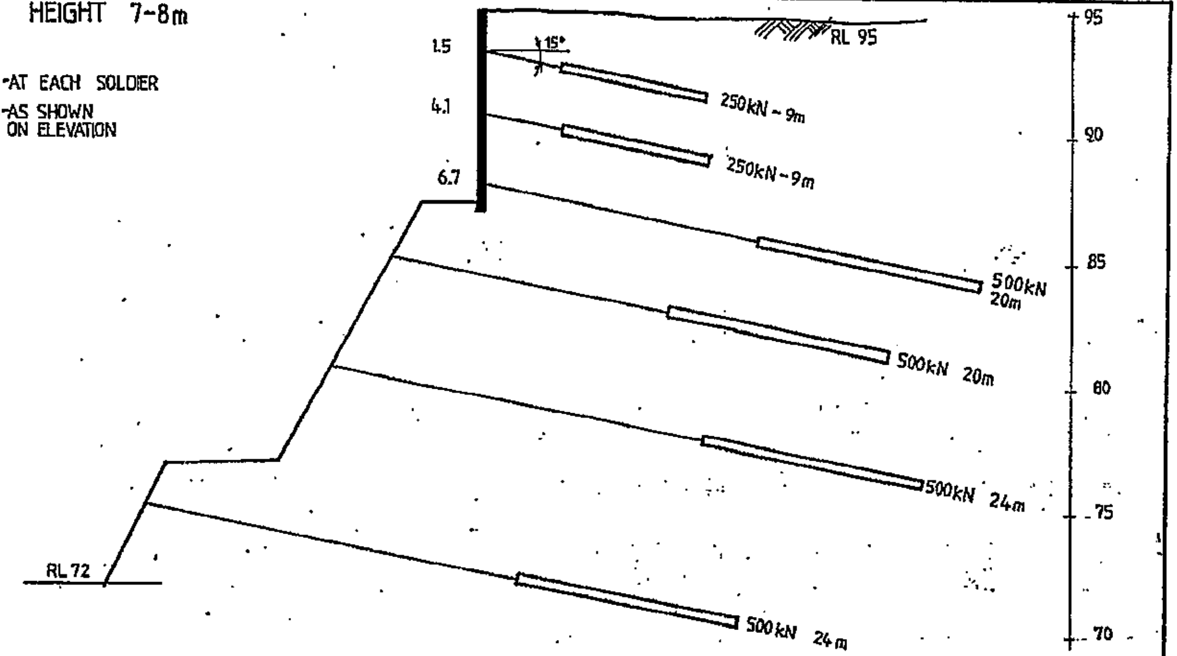

N
Reserve Road
Campbell Street
Lanceley Place
76
80
A
B
C
slide
tunnel
tunnel
92
92
96
100
Proposed Substation



ENUE
STREET
RESERVE
ROAD
REET
STREET
Willoughby
Substation
STREET
CAMPBELL
LANCELEY
PLACE
FREDERICK
80
tunnel
tunnel
A
B
HILL
T.V. Tower
Television
Studio
Slide
Proposed
Substation
SAVILLE
STREET
North Sydney
Technical College
Institute of
Technology
PACIFIC
ULONGA
STREET
ONE

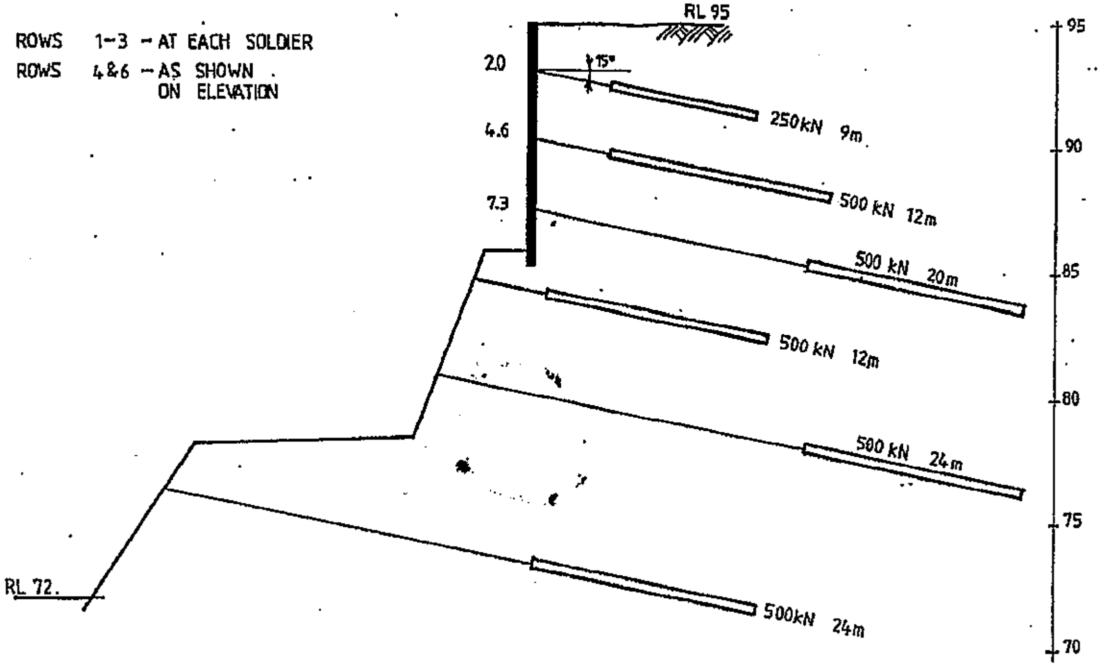
WALL HEIGHT 7-8m

ROWS 1-3 -AT EACH SOLDIER
ROWS 4&6-AS SHOWN ON ELEVATION



WALL HEIGHT 9-10m

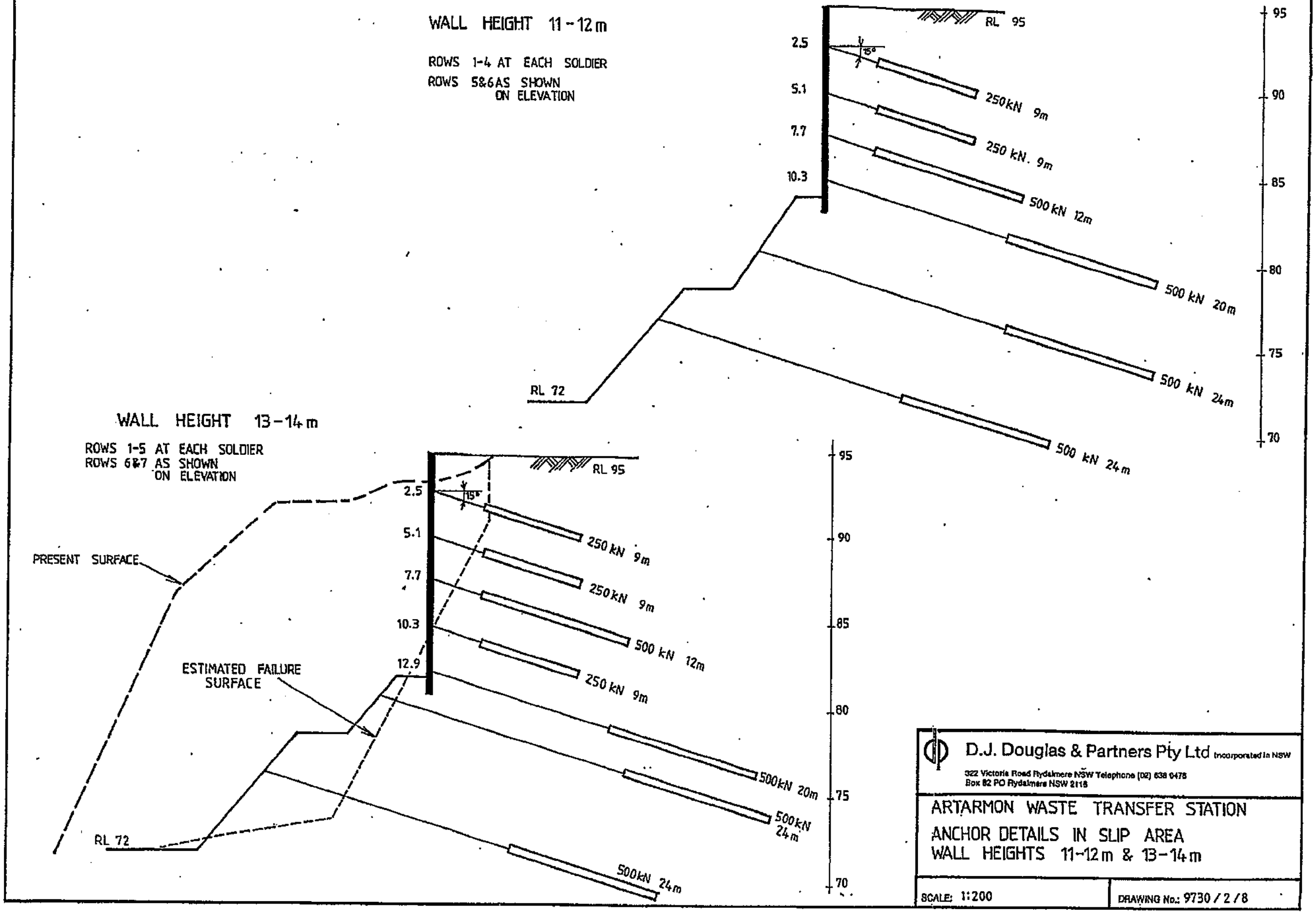
ROWS 1-3 - AT EACH SOLDIER
ROWS 4&6 - AS SHOWN ON ELEVATION



D.J. Douglas & Partners Pty Ltd Incorporated in NSW
322 Victoria Road Rydalmere NSW Telephone (02) 638 0478
Box 92 PO Rydalmere NSW 2116

ARTARMON WASTE TRANSFER STATION
ANCHOR DETAILS IN SLIP AREA
WALL HEIGHTS 7-8m & 9-10m

SCALE: 1:200
DRAWING No.: 9730/2/7



D.J. Douglas & Partners Pty Ltd Incorporated in NSW

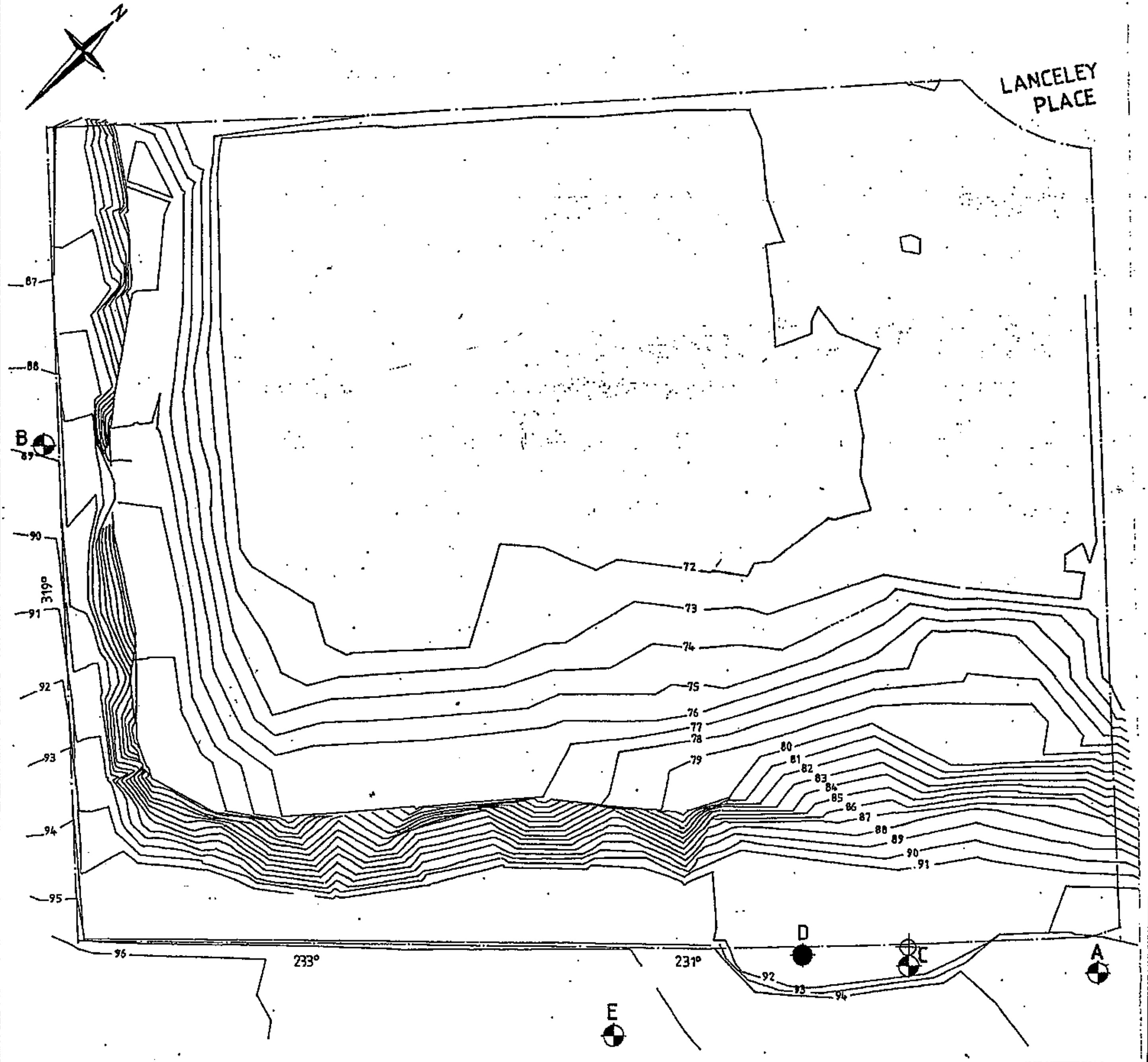
322 Victoria Road Rydalmere NSW Telephone (02) 638 0478
Box 82 PO Rydalmere NSW 2116

ARTARMON WASTE TRANSFER STATION

ANCHOR DETAILS IN SLIP AREA
WALL HEIGHTS 11-12 m & 13-14 m

SCALE: 1:200

DRAWING No.: 9730 / 2 / 8



LEGEND:

- ◒ CORED BORE HOLE
- ● CORED BORE HOLE AND INCLINOMETER CASING
- ⊕ INCLINOMETER CASING

D.J. Douglas & Partners Pty Ltd Incorporated in NSW

222 Victoria Road Rydalmere NSW Telephone (02) [illegible]
Box 82 PO Rydalmere NSW 2116

ARTARMON WASTE TRANSFER STATION
LOCATION OF CORED BORE HOLES
AND INCLINOMETER CASING
(SURVEY BY MWDA)

SCALE: 1:400

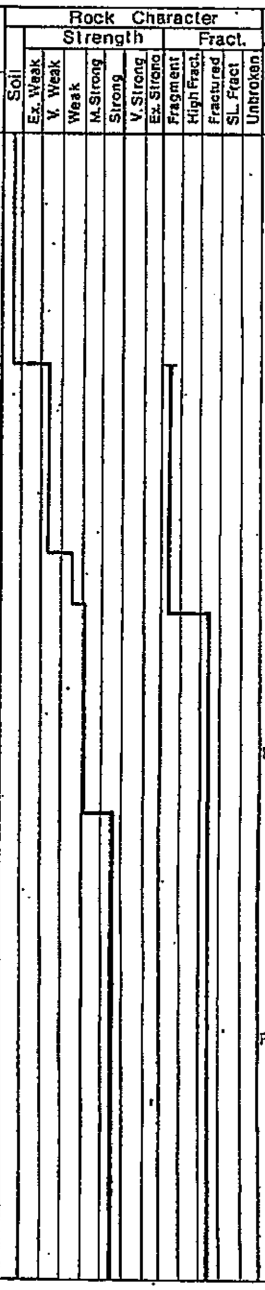
DRAWING No.: 9730/2/1

TEST BORE REPORT

CLIENT METROPOLITAN WASTE DISPOSAL AUTHORITY
PROJECT WASTE TRANSFER STATION
LOCATION LANCELEY PLACE, ARTARMON

Drilled from adjacent
Technical College

DATE 23-24.9.86 **BORE No** A
PROJECT No. SSI/9730/1 **SHEET** 1 **OF** 3
SURFACE LEVEL. APPROX RL 93.5
DIP OF HOLE 90° **AZIMUTH**



Depth m	Class	Description of Strata	RL	Discontinuities (B-Bedding J-Joint S-Shear D-Drill Brk.)	Sample Type	Core Rec. %	RQD %	Depth	Test Results
		FILLING - dark brown clay filling							
			93.0		S			0.5	4,5,5 N =10
1.0		CLAY - hard brown and grey clay							
2.0			91.4		S			2.0	7,12,13 N = 25
		CLAYEY SHALE - very weak, fragmented light grey and brown clayey shale with strong ironstone layers to 30 mm							
3.0						54 54	0 0		
			89.8						
4.0		LAMINITE - weak, fragmented grey and brown laminite							
			89.4						
		LAMINITE - weak to medium strong fractured grey and brown laminite with occasional clay seams to 10 mm				95	7		
5.0				joint dipping at 60° irregular	PLS			5.5	$Is_{(50)}$=0.74 MPa
6.0			87.5	joint dipping at 20°					
		LAMINITE - strong fractured light grey and dark grey laminite				100	20		
7.0									
8.0					PLS			8.0	$Is_{(50)}$=2.9 MPa
9.0									
10.0						99	33		

RIG B 40 DRILLER Cooper LOGGED Thompson CASING NX to 2.50m

TYPE OF BORING Flight auger to 2.50m, then NMLC core drilling

WATER OBSERVATIONS No free ground water observed

REMARKS

SAMPLING & IN SITU TESTING

A	auger sample	V	vane shear test
DP	dry plug	P	pressuremeter test
C	diamond cone	W	water pressure test
S	standard penetration	PP	pocket penetrometer
U( )	( )mm tube	PLS	point load strength

D.J.Douglas & Partners

TEST BORE REPORT

CLIENT	METROPOLITAN WASTE DISPOSAL AUTHORITY	**DATE** 23-24.9.86	**BORE No** A
PROJECT	WASTE TRANSFER STATION	**PROJECT No.** SSI/9730/1	**SHEET** 2 **OF** 3
LOCATION	LANCELEY PLACE, ARTARMON	**SURFACE LEVEL.** APPROX RL 93.5	
		DIP OF HOLE 90^0	**AZIMUTH**

Depth m	Class	Description of Strata	Graphic Log	RL	Discontinuities. D- Bedding J- Joint S- Shear D-Drill Brk.	Rock Character													Sampling & In Situ Testing				
							Strength							Fract.									
						Soil	Ex. Weak	V. Weak	Weak	M.Strong	Strong	V. Strong	Ex. Strong	Fragment	High Fract.	Fractured	SL. Fract	Unbroken	Sample Type	Core Rec. %	RQD %	Depth	Test Results
11.0		LAMINITE - strong fractured light grey and dark grey laminite																					
12.0																				99	43		
13.0																			PLS			12.9	$Is_{(50)}$=2.6 MPa
		SHALE - strong fractured to slightly fractured dark grey shale		80.0																			
14.0																							
15.0					joint dipping at 45^o joint dipping at 45^o															100	88		
16.0					joint dipping at 70^o joint dipping at 45^o																		
17.0					joint dipping at 80^o																		
18.0																				98	72		
19.0					joint dipping at 45^o joint dipping at 45^o joint dipping at 40^o joint dipping at 60^o														PLS			18.8	$Is_{(50)}$=1.8 MPa
		SANDSTONE & SHALE - inter-bedded strong fractured to slightly fractured light grey fine grained sandstone and grey shale		74.0																			

RIG B 40 DRILLER Cooper LOGGED Thompson CASING NX to 2.50 m

TYPE OF BORING Flight auger to 2.50 m, then NMLC core drilling

WATER OBSERVATIONS No free ground water observed

REMARKS

SAMPLING & IN SITU TESTING

A	auger sample	V	vane shear test
DP.	dry plug	P	pressuremeter test
C	diamond cone	W	water pressure test
S	standard penetration	PP	pocket penetrometer
U( )( )mm tube		PLS	point load strength

D.J. Douglas & Partners

TEST BORE REPORT

CLIENT METROPOLITAN WASTE DISPOSAL AUTHORITY
PROJECT WASTE TRANSFER STATION
LOCATION LANCELEY PLACE, ARTARMON

DATE 23-24.9.86 **BORE No** A
PROJECT No. SSI/9730/1 **SHEET** 3 **OF** 3
SURFACE LEVEL APPROX RL 93.5
DIP OF HOLE 90° **AZIMUTH**

Depth m	Class	Description of Strata	Graphic Log	RL	Discontinuities. B- Bedding J- Joint S- Shear D- Drill Brk.	Rock Character: Soil	Strength: Ex. Weak	V. Weak	Weak	M. Strong	Strong	V. Strong	Ex. Strong	Fract.: Fragment	High Fract.	Fractured	SL. Fract	Unbroken	Sampling & In Situ Testing: Sample Type	Core Rec. %	RQD %	Depth	Test Results
21.0		SANDSTONE & SHALE - inter-bedded strong fractured to slightly fractured light grey fine grained sandstone and grey shale		72.5							▮					▮	▮			100	98		
22.0 23.0 24.0 25.0		SANDSTONE - strong slightly fractured light grey fine to medium grained sandstone		68.5							▮						▮			100	100		

BORE DISCONTINUED AT 25.00 METRES.

RIG B 40 DRILLER Cooper LOGGED Thompson CASING NX to 2.50 m

TYPE OF BORING Flight auger to 2.50 m, then NMLC core drilling

WATER OBSERVATIONS No free ground water observed

REMARKS

SAMPLING & IN SITU TESTING

A	auger sample	V	vane shear test
DP	dry plug	P	pressuremeter test
C	diamond cone	W	water pressure test
S	standard penetration	PP	pocket penetrometer
U( )	( )mm tube	PLS	point load strength

D.J. Douglas & Partners

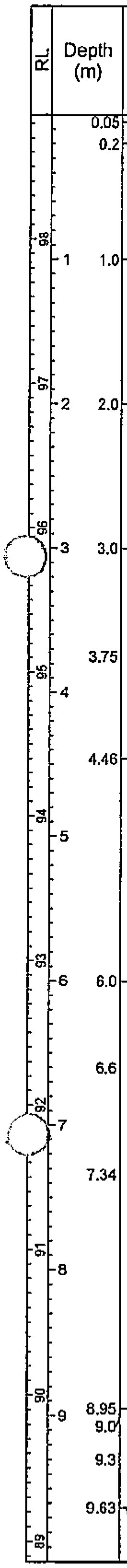
BOREHOLE LOG

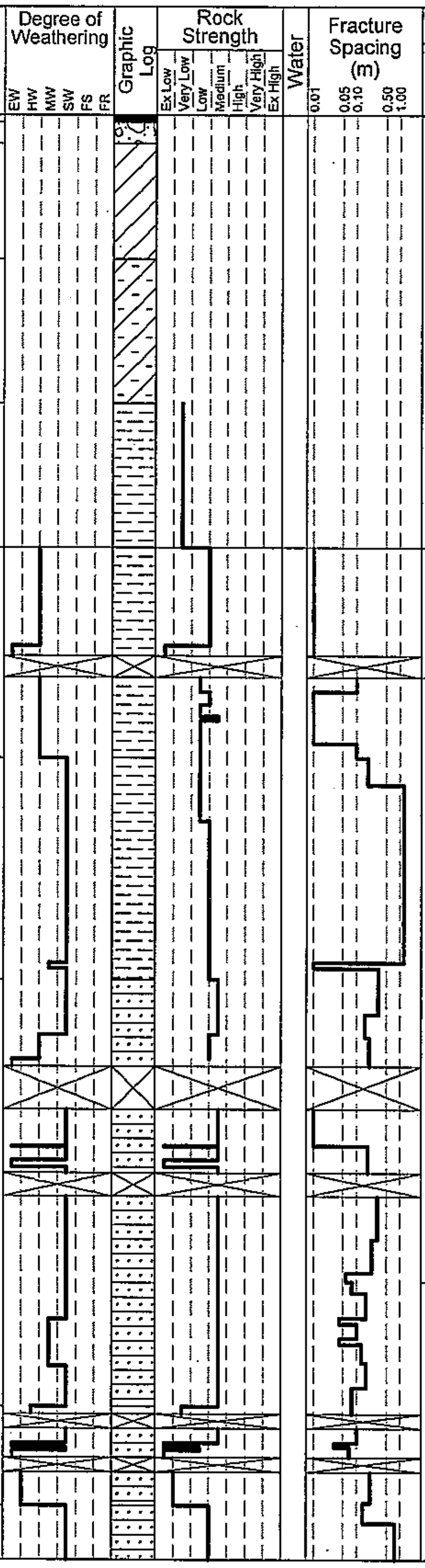
97.5 AHD

CLIENT: Bovis Lend Lease
PROJECT: Extension to North Shore Private Hospital
LOCATION: Westbourne Street, St Leonards

SURFACE LEVEL: ~~98.86~~
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1A
PROJECT No: 44433
DATE: 28 Nov 06
SHEET 1 **OF** 2





Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE					
0.2	ROADBASE GRAVEL					
	CLAY - mottled red brown clay with trace of ironstone, damp		A/E			
1.0	SHALY CLAY - mottled red brown shaly clay with ironstone bands, damp		A/E U E			pp>400kPa
2.0	SHALE - very low strength, grey brown shale	Note: Unless otherwise stated, rock is fractured along rough, planar bedding planes or joints dipping 0°- 10°	S			25/70mm refusal
3.0	SHALE - low and low to medium strength, highly to moderately weathered, fragmented to slightly fractured, light grey shale with a few medium strength ironstone bands		C	83	0	
3.75		3.75m: CORE LOSS: 150mm 3.9m: J85° smooth 4.36m: J60° rough	C	100	72	PL(A) = 0.2MPa
4.46	SHALE - low to medium strength, slightly weathered, slightly fractured, unbroken, grey shale with approximately <20% sandstone laminae	5.88m: B0° ironstained				PL(A) = 0.3MPa
6.0	LAMINITE - low and medium strength, moderately to slightly weathered, fractured to slightly fractured, light grey laminite (interbedded siltstone and sandstone) with approximately 40-50% sandstone laminae. Includes several extremely weathered, extremely low strength bands		C	82	77	PL(A) = 0.4MPa
6.6		6.6m: CORE LOSS: 300mm				
		7.15m: B0° 10mm clay	C	75	33	
7.34		7.34m: CORE LOSS: 160mm				PL(A) = 0.3MPa
		8.03m: J30° ironstained 8.09m: J45° smooth 8.38m: J45° ironstained, healed 8.82m: J45° rough	C	94	75	PL(A) = 0.2MPa
8.95	LAMINITE - low to medium strength, highly to slightly weathered, fractured to slightly fractured, light grey laminite with laminae and extremely low strength bands					
9.0		9m: CORE LOSS: 100mm	C	75	25	
9.3		9.3m: CORE LOSS: 100mm				
9.63		9.75m: J30° rough	C	95	77	PL(A) = 0.3MPa

RIG: Bobcat **DRILLER:** Lloyd **LOGGED:** Shafiq **CASING:** HW to 3.0m
TYPE OF BORING: Solid flight auger to 2.7m; Rotary to 3.0m; NMLC-Coring to 17.95m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: TBM: Metal rod and red triangle in TOK near boom gate: Assumed RL 100.0

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▽ Water level

CHECKED
Initials:
Date: 11/1/07

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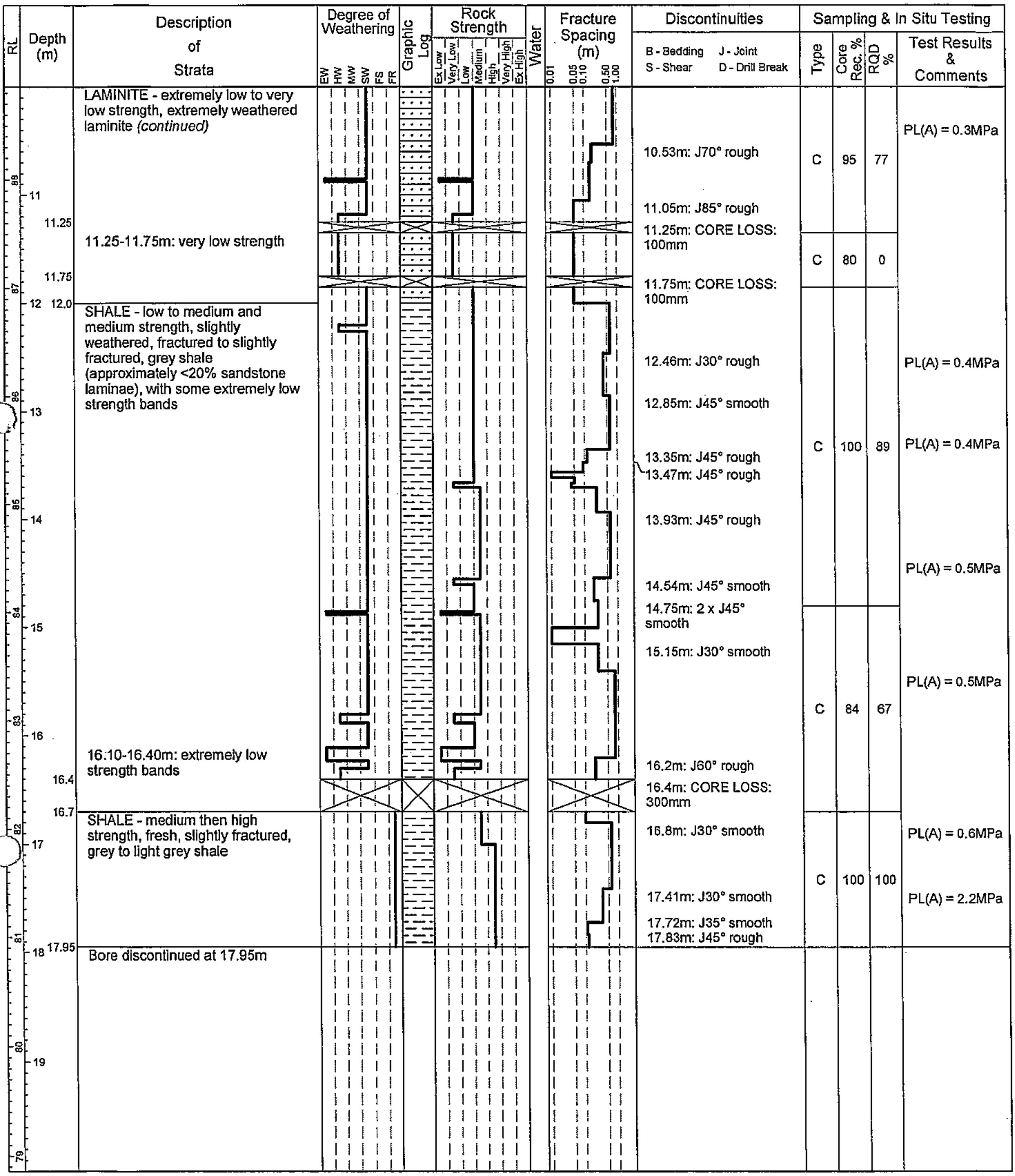
BOREHOLE LOG 97·5 AHD

CLIENT: Bovis Lend Lease
PROJECT: Extension to North Shore Private Hospital
LOCATION: Westbourne Street, St Leonards

SURFACE LEVEL: ~~98.86~~
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1A
PROJECT No: 44433
DATE: 28 Nov 06
SHEET 2 **OF** 2



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	LAMINITE - extremely low to very low strength, extremely weathered laminite *(continued)*					PL(A) = 0.3MPa
		10.53m: J70° rough	C	95	77	
		11.05m: J85° rough				
11.25		11.25m: CORE LOSS: 100mm				
	11.25-11.75m: very low strength		C	80	0	
11.75		11.75m: CORE LOSS: 100mm				
12.0	SHALE - low to medium and medium strength, slightly weathered, fractured to slightly fractured, grey shale (approximately <20% sandstone laminae), with some extremely low strength bands	12.46m: J30° rough				PL(A) = 0.4MPa
		12.85m: J45° smooth				
		13.35m: J45° rough	C	100	89	PL(A) = 0.4MPa
		13.47m: J45° rough				
		13.93m: J45° rough				
		14.54m: J45° smooth				PL(A) = 0.5MPa
		14.75m: 2 x J45° smooth				
		15.15m: J30° smooth				PL(A) = 0.5MPa
			C	84	67	
	16.10-16.40m: extremely low strength bands	16.2m: J60° rough				
16.4		16.4m: CORE LOSS: 300mm				
16.7	SHALE - medium then high strength, fresh, slightly fractured, grey to light grey shale	16.8m: J30° smooth				PL(A) = 0.6MPa
			C	100	100	
		17.41m: J30° smooth				PL(A) = 2.2MPa
		17.72m: J35° smooth 17.83m: J45° rough				
17.95	Bore discontinued at 17.95m					

RIG: Bobcat **DRILLER:** Lloyd **LOGGED:** Shafiq **CASING:** HW to 3.0m
TYPE OF BORING: Solid flight auger to 2.7m; Rotary to 3.0m; NMLC-Coring to 17.95m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: TBM: Metal rod and red triangle in TOK near boom gate: Assumed RL 100.0

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date: 11/1/07

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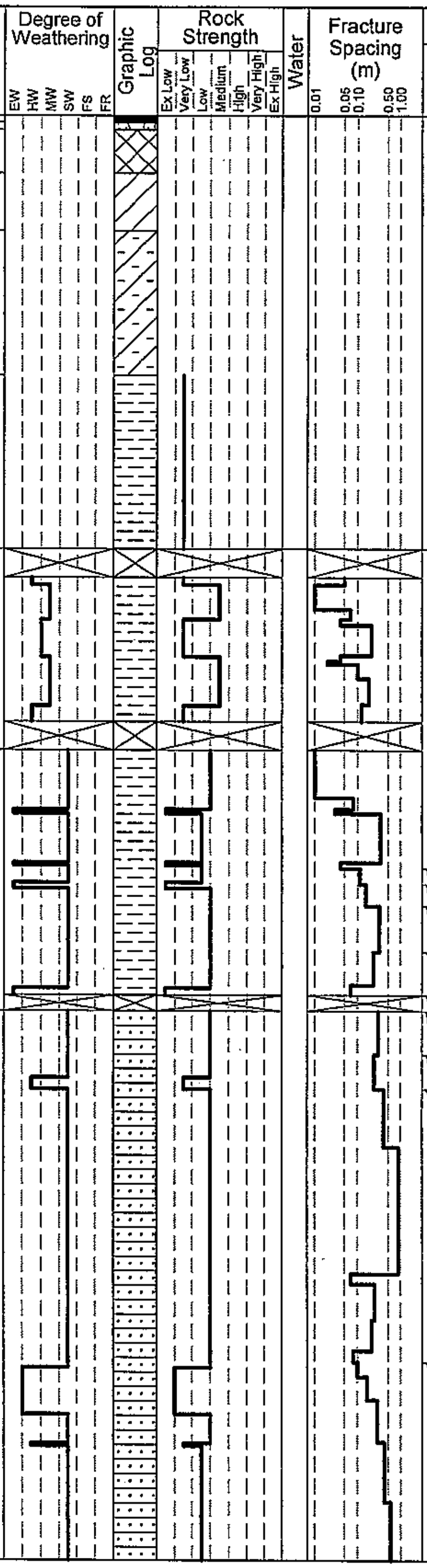
BOREHOLE LOG

97.4 AHD

CLIENT: Bovis Lend Lease
PROJECT: Extension to North Shore Private Hospital
LOCATION: Westbourne Street, St Leonards

SURFACE LEVEL: ~~99.33~~
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 44433
DATE: 29 Nov 06
SHEET 1 **OF** 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE		A			
0.1	ROADBASE - grey gravelly silty sand roadbase					
0.4	FILLING - crushed sandstone filling, with trace of clay and gravel		A/E			
0.8	CLAY - red brown clay with some ironstone bands, damp		A/E			
	SHALY CLAY - hard, light grey shaly clay, damp		S E			8,10,30 N = 40
1.8	SHALE - very low strength, grey brown shale with some ironstone bands	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping at 0°- 10°	S E			25/50mm refusal
3.0	SHALE - very low strength, highly to moderately weathered, fragmented to slightly fractured, light grey and brown shale with medium strength ironstone bands	3m: CORE LOSS: 200mm 3.9m: J40° rough	C	86	29	
4.2		4.2m: CORE LOSS: 200mm				
4.4	SHALE - low and low to medium strength, slightly weathered, fragmented to slightly fractured, light grey to grey shale (approximately <20% sandstone laminae), with some extremely low strength bands	5.18m: B0°, 10mm clay 5.22m: J45° rough 5.33m: B0° 20mm clay 5.48-5.70m: J85°- 90° 5.8m: J60° rough 6.03m: J45° rough	C	88	51	PL(A) = 0.3MPa PL(A) = 0.2MPa
6.1		6.1m: CORE LOSS: 110mm				
6.21	LAMINITE - low to medium and low strength, slightly weathered, fractured to slightly fractured, light grey to grey laminite (interbedded sandstone and siltstone) (approximately 40% to 50% sandstone laminae), with some very low strength bands	6.21m: J85° smooth 6.51m: J90° rough 6.75m: J55° rough 8.03m: J30° rough 8.35m: J45° rough 8.56m: J45° rough 8.64m: J60° rough 8.9m: J45° smooth	C C	100 92	80 82	PL(A) = 0.2MPa PL(A) = 0.3MPa PL(A) = 0.2MPa

RIG: Bobcat **DRILLER:** Lloyd **LOGGED:** Shafiq **CASING:** HW to 3.0m

TYPE OF BORING: Hand auger to 0.5m; Solid flight auger to 2.5m; Rotary to 3.0m; NMLC-Coring to 18.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: TBM: Metal rod and red triangle in TOK near boom gate: Assumed RL 100.0

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▽ Water level

CHECKED
Initials:
Date: 11/1/07

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BOREHOLE LOG

97·4 AHD

CLIENT: Bovis Lend Lease
PROJECT: Extension to North Shore Private Hospital
LOCATION: Westbourne Street, St Leonards

SURFACE LEVEL: ~~99.33~~
EASTING:
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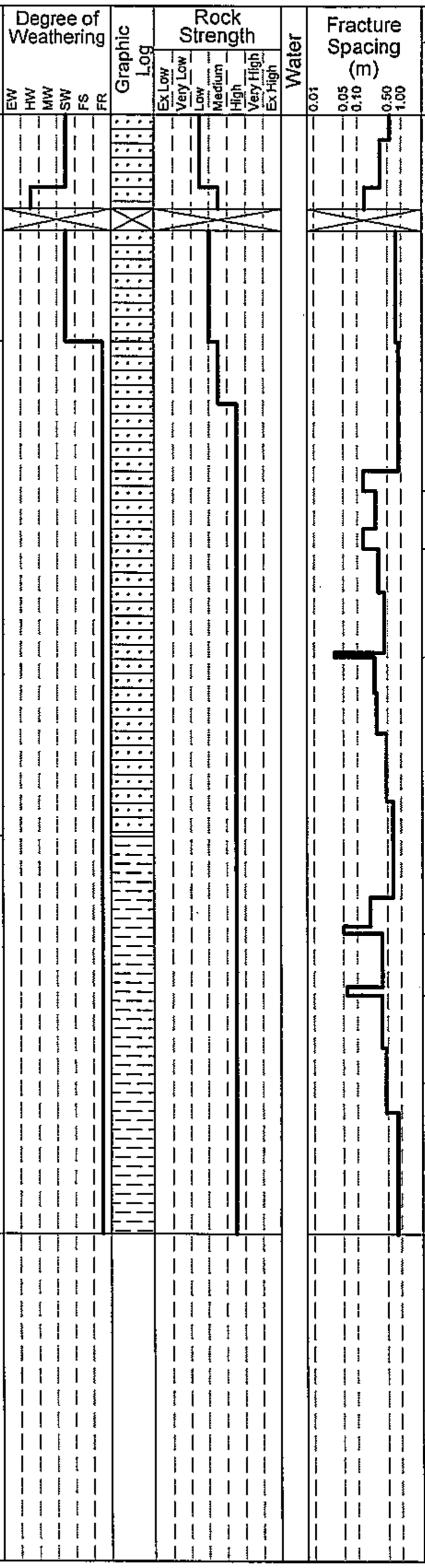
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 44433
DATE: 29 Nov 06
SHEET 2 OF 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
89		LAMINITE - low to medium and low strength, slightly weathered, fractured to slightly fractured, light grey to grey laminite (approximately 40% to 50% sandstone laminae), with very low strength bands *(continued)*		C	92	82	PL(A) = 0.2MPa
	10.65		10.65m: CORE LOSS: 150mm				
88	11			C	100	100	PL(A) = 0.3MPa
	11.57	LAMINITE - medium then high strength, fresh, slightly fractured, grey laminite (approximately 20-30% sandstone laminae)					PL(A) = 0.7MPa
87	12		12.46m: J90° rough 12.6m: J45° rough				PL(A) = 1.1MPa PL(A) = 1.3MPa
86	13		12.86m: J20° smooth 13m: J75°- 85° rough				
85	14		13.72m: J45° 13.75-14.20m: J90° rough 14.28m: J30° smooth 14.75m: J70° smooth	C	100	95	PL(A) = 1.3MPa
84	15	15.0 SHALE - high strength, fresh, slightly fractured, grey shale (approximately <20% sandstone laminae)	15.42m: J20° 15.62m: J30° smooth 15.67m: J60° smooth				PL(A) = 1.4MPa
83	16		16.04m: J20° smooth 16.1m: J30° smooth 16.46m: J25° smooth				
82	17			C	100	100	PL(A) = 1.1MPa PL(A) = 1.1MPa
	17.75	Bore discontinued at 17.75m					
81	18						
80	19						

RIG: Bobcat **DRILLER:** Lloyd **LOGGED:** Shafiq **CASING:** HW to 3.0m
TYPE OF BORING: Hand auger to 0.5m; Solid flight auger to 2.5m; Rotary to 3.0m; NMLC-Coring to 18.0m
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SAMPLING & IN SITU TESTING LEGEND

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W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ⊻ Water level

CHECKED
Initials:
Date: 1/1/07

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