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REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED COCHLEAR GLOBAL HEADQUARTERS
MACQUARIE UNIVERSITY SOUTH PRECINCT
MACQUARIE PARK

Prepared for
LACHLAN PROJECT DEVELOPMENT PTY LTD

Project 45298
March 2008

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REPORT ON GEOTECHNICAL INVESTIGATION
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1. INTRODUCTION

This report presents the results of a geotechnical investigation undertaken for the proposed Cochlear Global Headquarters project at Macquarie University South Precinct, Macquarie Park. The investigation was commissioned by CRI Australia Pty Ltd on behalf of Lachlan Project Development Pty Ltd.

It is understood that the proposed development will include construction of a six to seven-storey building with two levels of basement carparking. The lowest basement level (RL 58.3) may require excavation to depths of 10 m in the north-western corner of the site reducing to 3 m on the south-eastern corner of the site.

The field work for the investigation comprised the drilling of eight boreholes and installation of two groundwater monitoring wells for sampling and measurement of groundwater levels. Laboratory testing of selected soil, groundwater and rock core samples was undertaken, followed by engineering analysis and reporting. Details of the field work are given in the report, together with comments on design and construction practice.

Douglas Partners Pty Ltd (DP) carried out a Phase 1 contamination assessment of the site in conjunction with the geotechnical investigation, the results of which have been reported separately (Project No. 45298.01, dated March 2008). The contamination assessment included the drilling of nine additional shallow boreholes to depths of 0.5 m to 4.7 m.

2. SITE DESCRIPTION

The site of the proposed development (refered to as 'the site' in this report) is located on the eastern side of a large triangular lot which covers an area of approximately 34,000m^2 and includes part of Lots 181-182 in Deposited Plan 1112777. The site covers an area of approximately 19,000 m^2.

The site is bounded by University Avenue to the north, the 'Gumnut Cottage' childcare centre to the west, the "Waratah Occasional Care Centre" to the south and vacant grass covered land further to the south and east. An open water-course runs along the south-eastern boundary.

The site and surrounding area are located on a gentle south-east-facing hill which generally falls towards the water-course. Within the site, the surface generally falls to the south-east from approximately RL 69.0 to RL 61.0, relative to Australian Height Datum (AHD), at an average slope of approximately 2 to 3 degrees. Along the south-eastern side of the site the surface falls more steeply towards the water course at an average slope of approximately 10 degrees.

At the time of the investigation the site was generally covered with an asphaltic concrete (AC) paved carpark, operated by Macquarie University. Based on the borehole data and topography it appears that the carpark surface has been formed by previous filling on the down-slope (south-eastern) side of the site and possibly some excavation on the upslope (north-western) side of the site. A mound approximately 2 m to 3 m high is located along the northern boundary (adjacent to University Drive) and a mound approximately 2 m high is located along the south-eastern side of the site.

3. GEOLOGY

Reference to the Sydney 1:100 000 Series Geological Sheet indicates the site is underlain by Ashfield Shale and that the site is close to the boundary with Hawkesbury Sandstone to the north and east of site. Ashfield Shale typically comprises black to dark grey shale and laminite (interbedded shale, siltstone and fine grained sandstone) and typically weathers to form clays of medium to high plasticity. Hawkesbury Sandstone typically comprises medium to coarse grained quartz sandstone with some shale bands or lenses. The geological mapping was confirmed by the field work which identified residual soils then laminite overlying sandstone bedrock. The laminite may be part of the Mittagong Formation which is a transitional rock unit between the Ashfield Shale and Hawkesbury Sandstone.

4. FIELD WORK METHODS

The field work comprised eight boreholes (BH 1 to BH 8) drilled to depths of 11.5 m to 17.0 m using a truck-mounted drilling rig and installation of two groundwater monitoring wells.

The boreholes were initially drilled using spiral augers and rotary washboring within soil and extremely weathered rock to depths of 1.5 m to 5.1 m and 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.

Soil samples and rock cores were returned to the DP office where they were logged by a geologist, the cores photographed and Point Load Strength Index (Is_{50}) tests carried out on selected samples of the rock core.

Groundwater monitoring wells (50 mm diameter slotted PVC) were installed in BH 1 and BH 7 to depths of 14.2 m and 11.5 m respectively, to allow for sampling of the groundwater and measurement of the groundwater level during the investigation period. The wells were purged (i.e. water bailed out) and then samples taken approximately four days later for laboratory analysis. Measurements of the groundwater levels were taken at the time of purging and sampling. No long term monitoring of groundwater levels was carried out.

The borehole locations are shown on Drawing 1 in Appendix A.

The ground surface level at each of the test locations was interpolated from spot heights relative to Australian Height Datum (AHD) shown on the survey plan by Lockley Land Title Solutions Pty Ltd (Ref. 30431DT, dated 27/6/07).

5. FIELD WORK RESULTS

Details of the conditions encountered are given in the borehole logs in Appendix B, together with colour photographs of the rock core samples and notes defining classification methods and descriptive terms.

5.1 Soil and Rock Conditions

The boreholes penetrated a subsurface profile typically comprising topsoil or pavements over filling to depths of 0.1 m to 3.8 m, then residual clay to depths of 1.3 m to 5.1 m overlying bedrock. The bedrock comprised sandstone in BHs 2, 4, 6 and 8 and laminite to depths of 7.6 m to 9.8 m in BHs 1, 3, 5 and 7 overlying sandstone to the maximum investigation depth of 17.0 m. The various strata are summarised below and interpreted geotechnical sections (Section A-A', B-B' and C-C') through the site are given on Drawings 2 to 4 in Appendix A.

PAVEMENTS: comprised asphaltic concrete (AC) 10 mm to 20 mm thick over sandy gravel filling (roadbase) 190 mm to 230 mm thick. The total pavement thickness ranged from between 200 mm to 250 mm.

TOPSOIL: comprised sandy/silty soil with roots to depths of 0.1 m to 0.3 m.

FILLING: encountered in all boreholes, generally to depths of 0.1 m to 1.2 m but increasing to depths of 2.2 m to 3.8 m in BHs 4, 6 and 7 along the south-eastern side of the site. The filling generally comprised clay with shale, ironstone and sandstone gravel.

RESIDUAL CLAY: generally encountered to depths of 1.3 m to 2.1 m but increasing to depths of 3.9 m to 6.4 m in BHs 4, 6 and 7 along the south-eastern side of the site. The residual soils typically comprised stiff to hard clay and/or silty clay. The silty clay was firm in BH 4 between a depth of 2.2 m to 2.8 m (logged as possibly an old topsoil layer).

LAMINITE : encountered in BHs 1, 3, 5 and 7 to depths of 7.6 m to 9.8 m (RL 51.3 to RL 58.0) and comprised interbedded fine grained sandstone, siltstone and shale. The proportion of sandstone was estimated to vary from 20% to 50%. The laminite was generally fractured and extremely low to very low strength grading to fractured to slightly fractured and medium to high strength with depth. Very high strength laminite was encountered in BH 1 between depths of 8.6 m to 9.2 m and BH 3 between depths of 7.3 m to 9.7 m.

SANDSTONE : encountered directly below the clay in BHs 2, 4, 6 and 8 and underlying the laminite in BHs 1, 3, 5 and 7. The sandstone in BHs 2, 4, 6 and 8 was generally very low to low strength grading to medium to high strength below depths of 3 m (BH 2) to 6 m (BH 6). The sandstone was generally fractured to slightly fractured to depths of 6.2 m to 10.7 m (RL 56.5 to RL 51.0) overlying more uniform slightly fractured to unbroken sandstone.

The rock, particularly the laminite, included jointing and defects with numerous 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 cores at some locations.

Zones of core loss generally ranging from 0.2 m to 0.6 m thick occurred during coring of the rock. The thicker core loss zones generally occurred within the laminite profile (core loss is inferred to be extremely low strength and/or highly fractured rock).

5.2 Groundwater

No free groundwater was observed during augering of the boreholes (i.e. within depths of 1.5 m to 5.1 m). The use of water during wash boring and coring within the bedrock prevented the measurement of groundwater below this depth. The water level within the groundwater monitoring wells was measured prior to purging on 14/12/07 and again prior to sampling on 18/12/07. The monitoring well in BH 1 appeared to be filled with silt below a depth of 8.1 m and therefore observation/measurement of groundwater below this depth was not possible. A summary of the groundwater measurements is given in Table 1.

Table 1 – Summary of Groundwater Monitoring

Date	Measured Depth (m) to Groundwater	
	BH 1	BH 7
14/12/07	No groundwater to 8.1 m (RL59.1)*	4.8 m (RL 56.3) (prior to purging)
18/7/07	No groundwater to 8.1 m (RL59.1)*	5.2 m (RL 55.9) (prior to sampling)

Note * Groundwater well in BH 1 appeared to be blocked with silt below a depth of 8.1 m - no observation/measurement possible below this depth.

6. LABORATORY TESTING

6.1 Soil and Groundwater

Selected samples of soil were tested in the DP laboratory to assess Atterberg Limits, Linear Shrinkage, standard compaction and four-day soaked Californian Bearing Ratio (CBR) values.

A groundwater sample taken from the monitoring well in BH 7 was tested at an external laboratory to assess aggressivity (pH, chloride and sulphate content) and iron concentration. The results of the laboratory testing are included in Appendix C and summarised in Tables 2 and 3.

Table 2 : Summary of Laboratory Test Results (Soil)

Bore	Depth (m)	Material	W_f (%)	Atterberg Limits			LS (%)	OMC (%)	MDD (t/m^3)	Swell (%)	4-day soaked CBR (%)
				W_l (%)	W_p (%)	PI (%)					
BH1	1.0	Silty Clay	10.7	35	20	15	8				
BH3	0.3-1.0	Silty Clay	20.3					22.3	1.68	0.6	7

W_f = Field Moisture Content W_l = Liquid Limit W_p = Plastic Limit
PI = Plasticity Index LS = Linear Shrinkage OMC = Optimum Moisture Content
MDD = Maximum Dry Density CBR = Californian Bearing Ratio

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

Table 3 – Summary of Laboratory Chemical Analysis (Groundwater)

Bore No.	Depth (m)	pH	Chloride, Cl^- (mg/L)	Sulphate, SO_4^{2-} (mg/L)	Filtered Iron (mg/L)
BH7	1.3-1.75	4.5	50	290	4.2

The results of the chemical analysis indicate the water sample was generally alkaline and within a non-aggressive exposure classification in accordance with AS2159 - 1995 (Piling - Design and Installation).

6.2 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 bore 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 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 20.

Generally, the Is_{50} values for the rock cores ranged from approximately 0.5 MPa to 2.0 MPa, corresponding to a medium to high strength classification (estimated UCS ranging from 10 MPa to 40 MPa). Lower Is_{50} values of 0.2 MPa to 0.3 MPa (corresponding to a low to medium strength) were measured on some of the rock cores within the upper rock profile. Higher Is_{50} values of 2.1 MPa to 3.1 MPa (corresponding to high to very high strength) were measured for rock cores below depths of approximately 6 m to 10 m and Is_{50} values of 3.1 MPa to 3.7 MPa (estimated UCS of 62 MPa to 74 MP) were measured for very high strength laminite in BH 3 between depths of 7.3 m to 9.7 m.

7. GEOTECHNICAL MODEL

The interpreted geological model for the site comprises:

- surficial topsoil and pavements overlying filling to depths of approximately 0.5 m to 1 m on the north-western part of the site increasing to depths of approximately 2.0 m to 4.0 m along the south-western part of the site. The mound along the northern boundary is formed by filling over natural clay whilst the mound on the south-eastern side of the site is formed by filling only.
- mostly stiff to hard residual clays to depths of approximately 1 m to 2 m on the north-western part of the site increasing to depths of approximately 4 m to 6.5 m on the south-eastern part of the site.
- bedrock underlying the clay and comprising sandstone on the southern part of the site and laminite to depths of approximately 7.5 m to 10.0 m (RL 51.0 to RL 58.0) over sandstone on the northern part of the site. The laminite is probably part of the Mittagong Formation which is a transitional rock unit between the Ashfield Shale and Hawkesbury

Sandstone. The rock strength generally increases from extremely low to low strength to medium to high strength with depth, however there are medium to high strength bands at least 0.5 m to 1 m thick within the upper rock profile. The laminite is typically more fractured and jointed than the sandstone.

- a perched groundwater table near the interface of residual clay and rock surface on the lower south-eastern part of the site. It is anticipated that groundwater flows may also occur within fractured zones and joints within the rock, as evident from ironstained joints observed within the highly weathered to slightly weathered rock profile. Groundwater seepage flows are likely to increase following periods of extended wet weather.

Three geotechnical cross sections (Sections A-A', B-B' and C-C') showing the interpreted subsurface profile are shown on Drawings 2 to 4 inclusive. The orientations of the cross-sections are shown on Drawing 1. The sections show interpreted geotechnical divisions of underlying soil and rock together with the extent of the proposed basement excavations and structures.

The rock encountered in the boreholes has been classified in general accordance with the procedures given in Reference 1 (Pells et. al. - 1998) which use a combination of rock strength and fracture spacing to divide the rock into five classes ranging from Class I (high strength and very few defects) to Class V (extremely low to very low strength and/or highly fractured). The interpreted depth and Reduced Level (RL) at the top of the various rock classes is shown in Table 4. It should be noted that the profiles are accurate at borehole locations only and that variations must be expected away from the boreholes. It should also be noted that closely fractured zones can occur within higher strength rocks and the classification will reduce in these areas.

Table 4 – Summary of Geotechnical Model (Rock Classification)

Borehole No.	Surface RL (AHD)	Depth & Reduced Level to Top of Various Rock Classes ([1])			
		Class V	Class IV	Class III	Class II-I
BH1	67.2	-	1.3 (65.9)*	8.6 (58.6)	10.7 (56.5)
BH2	64.8	1.3 (63.5)	-	3.0 (61.8)	6.2 (58.6)
BH3	64.5	2.1 (62.4)	3.3 (61.2)	7.3 (57.2)*	9.7 (54.8)
BH4	61.8	-	3.9 (57.9)*	5.3 (56.5)	7.0 (54.8)
BH5	62.4	2.0 (60.4)	2.5 (59.9)	6.5 (55.9)	7.6 (54.8)
BH6	61.0	-	4.2 (56.8)*	8.1 (52.9)*	10.0 (51.0)
BH7	61.1	5.1 (56.0)	6.4 (54.7)	8.4 (52.7)*	9.8 (51.3)
BH8	64.0	-	2.0 (62.0)	5.8 (58.2)*	8.5 (55.5)

Notes:

Bracketed numbers are the Reduced Level (to AHD) for the top of the stratum.

(1) In-situ rock classified in accordance with Reference 1.

* Rock contains thick bands of medium to high strength however the classification is reduced due to fractures/defects

8. COMMENTS

8.1 Proposed Development

Based on architectural drawings by Toland Williams Pty Ltd (dated 10/2/08) it is understood that the proposed development will include construction of a six to seven-storey building with two levels of basement carparking and some external carparking to the east and west of the building. The outline of the basement and external carparking is shown on Drawing 1. The lowest basement level (RL 58.3) may require excavation to depths of 10 m in the north-western corner of the site reducing to 3 m on the south-eastern corner of the site.

It is understood the Parramatta Rail Link tunnel (under construction) is located to the north of the site. DP do not have details of the tunnel or the actual tunnel alignment, however, reference to the UBD Street Directory indicates the tunnel is approximately 200 m to the north of the site. It is considered that the tunnel, at a distance of approximately 200 m, would not pose a geotechnical risk to the proposed development. The actual location of the tunnel should be confirmed and if the tunnel is significantly closer than expected then more detailed review of possible impacts on the proposed development may be required.

8.2 Site Preparation and Earthworks

8.2.1 Excavation Conditions

As shown on Drawings 2, 3 and 4 excavation for the basement is generally expected to be through filling, clay and extremely low to low strength (Class IV-V) rock to depths of approximately 3 m to 6 m then low to medium strength (Class IV) rock to depths of approximately 5 m to 8 m followed by medium to high strength (Class III-I) rock. It is important to note that the upper layers of rock contains bands of medium to high strength rock at least 0.5 m to 1 m thick. Very high strength laminite was encountered in BH 1 and BH 3 between depths of 8.6 m to 9.2 m and 7.3 m to 9.7 m respectively. Slightly fractured to unbroken, medium to high strength (Class II-I) sandstone is expected below depths of approximately 6 m to 10.5 m (RL 58.5 to RL 51.0) with the top of the Class II-I rock typically falling toward the south-east. High strength sandstone was encountered at a relatively shallow depth of 3 m (RL 61.8) in BH 2.

Excavation of filling, residual soils and extremely low to low strength rock should be achievable using conventional earthmoving equipment, however the assistance of rock hammering or ripping will probably be required for effective removal of medium to high strength bands and/or ironstone bands within the weathered rock sequence. Excavation of low to medium strength (Class III-IV) rock may require moderate ripping with an excavator whilst excavation of medium and high strength rock will require heavy ripping with a large excavator or bulldozer. Productivity within medium to high strength rock may be low (even with large dozers) and therefore some pre-splitting or rock hammering may be necessary to improve efficiency. The underlying slightly fractured to unbroken (Class II-I) may be effectively unrippable in which case large hydraulic rock breakers in conjunction with heavy ripping will be required to remove this material. The Class II-I sandstone was generally encountered below the proposed basement level however it may be encountered within deeper detailed excavations (i.e. for lift pits and footings) particularly in the vicinity of BH 2.

The excavation rate that can be achieved within the medium to high strength rock varies considerably and is dependent upon the degree of jointing in the rock, the rock strength, the type of machinery being used and the skill of the operator. Some of these factors vary between individual contractors and it is therefore recommended that bulk excavation tenderers be required to make their own assessment of the equipment required to carry out the work. Contractors are also encouraged to inspect the rock core samples at the DP office in West Ryde prior to submitting final tenders (rock cores are generally kept for 6 months after drilling unless longer holding times are requested).

8.2.2 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. Reference should be made to the Phase 1 contamination assessment report by DP for comments on the suitability of the site soils for disposal and additional testing that may be required.

8.2.3 Groundwater Seepage

Groundwater was not observed during auger drilling of the boreholes to maximum depths of 5.1 m (BH 7) however groundwater was later measured within the groundwater monitoring well in BH 7 at depths of 4.8 m and 5.2 m (RL 55.9 and RL56.3). The measured groundwater level is probably associated with a perched groundwater table near the interface of residual clay and bedrock. It is expected, however, that there will be some seepage within fractured zones and joints in the underlying rock.

During construction, it is anticipated that groundwater seepage should be readily controlled by perimeter drains connected to a "sump-and-pump" dewatering system. The need for ongoing dewatering, after construction, will depend on whether the basement is designed as a drained basement or water tight (tanked) basement. A drained basement will require permanent subfloor drainage below the basement floor slab connected to a sump and pump dewatering system. A tanked basement will avoid the need for dewatering after construction, however the tanked basement may be considerably more expensive than the drained basement. A tanked basement would need to be designed to resist uplift forces associated with groundwater pressure, for which preliminary design could be based on a groundwater level at the clay/rock interface.

Water resulting from dewatering operations may be suitable for disposal by pumping to the stormwater system subject to additional testing of groundwater quality during excavation and approval from relevant authorities (i.e. Council and Macquarie University). The contamination report by DP indicated that contaminant concentrations were low and generally below laboratory detection limits.

8.2.4 Dilapidation Surveys

Dilapidation surveys may be 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 vibrations or construction related activities can be accurately assessed. However, given that the excavation is generally set back at least 5 m to 10 m from roads and 15 m from buildings (i.e. the childcare centre to the south and west) this requirement is probably not relevant for the proposed development.

8.2.5 Vibrations

It is anticipated that the proposed rock excavation will result in vibration of the surrounding ground, however, the excavation is generally set back more than 15 m from adjacent buildings and 5 m to 10 m from University Drive. Where impact breakers are required in the vicinity of the childcare centres it would be prudent to monitor and limit vibrations on the adjacent building. Generally, a maximum peak particle velocity of 8 mm/sec (in any component direction) at foundation level of adjacent structures should be adopted for both structural and human comfort considerations.

Based on vibration monitoring carried out by DP at various excavation sites in Sydney it is anticipated that vibrations resulting from a 2000 kg rock hammer would be less than 8 mm/sec at distances of more than 10 m from the excavation. However, as the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of excavation. The trial may indicate that smaller or different types of excavation equipment should be used.

8.3 Excavation Support

8.3.1 General

The basement excavation is generally set back 15 m or more from the site boundaries to the south and east, approximately 5 m to 10 m from University Drive and approximately 15 m from the childcare centres to the south and west.

Due to the set back distances from boundaries and structures it is anticipated that excavations may be battered in soil and Class IV to III rock followed by vertical excavation in Class II or better rock (if encountered), as described in Section 8.3.2. Alternatively shoring may be adopted within the soil and Class IV to III rock due to site specific requirements or to minimise the volume of material to be removed.

8.3.2 Batter Slopes and Excavation Faces

Where there is sufficient space available it may be possible to temporarily batter excavations in soil and Class III to V rock. The following maximum batter slopes are recommended for the design of temporary and permanent batters.

Table 5 – Recommended Maximum Batter Slopes

Material [1]	Temporary Batter Slope (H:V)	Permanent Batter Slope (H:V)
Filling or natural clay soils	1.5:1	2:1
Class IV-V rock	0.75:1	1:1
Class III rock	0.25:1	0.25:1
Class II-I rock	Vertical*	Vertical*

Notes:

1. In-situ rock classified in accordance with Reference 1.

* Subject to jointing and geotechnical inspection

The Class IV-V rock and possibly Class III laminite is expected to deteriorate and break down if left exposed to weather. It is therefore recommended that any exposed soil and Class III-V rock faces are covered with mesh reinforced shotcrete pinned to the face with dowels. A minimum shotcrete thickness of 80 mm should be adopted unless stability issues dictate a greater thickness is required. The need for shotcrete of rock faces may be reassessed by a geotechnical engineer at the time of excavation. Alternatively flatter batters could be adopted in these materials, preferably 1:1 or flatter.

Excavations in medium or greater strength (Class II-I) rock will generally be self-supporting (subject to joint orientation) and may be cut vertically. It is possible that some of the less fractured Class III sandstone may also be self-supporting and therefore able to be cut vertically, however, this will need to be assessed by a geotechnical engineer at the time of excavation. All vertical rock faces must be progressively inspected by a geotechnical engineer at 1.5 m depth intervals to check for adversely inclined joints and to assess whether additional stabilisation measures are required. Stabilisation of vertical rock faces may include shotcrete of fractured or highly weathered zones or rock bolts/anchors where adverse joints form potentially unstable wedges of rock. A contingency budget should be included for stabilisation of rock faces particularly given size of the site and length of excavated faces that will be exposed.

8.3.3 Retaining Walls/Shoring

Vertical excavations within the soils and Class IV-III rock, if required, will require both temporary and permanent lateral support during and after excavation. It is anticipated that a bored bored soldier pile wall with shotcrete or timber infill panels would be suitable. 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 soil and Class V rock and then 3 m depth intervals within Class IV or better rock.

Preferably, shoring piles should be founded below the base of the bulk excavation level in order to provide lateral restraint at the base of the excavation and avoid the risk of adversely inclined joints or wedges undermining the base of the piles. It may be possible to terminate the shoring piles within Class II-I sandstone above the bulk excavation level, however, this is probably not applicable for the proposed depth of excavation. Where piles are terminated above the bulk excavation level it will be important for a geotechnical engineer to assess the stability of the rock directly beneath each pile. The toe of the piles above bulk excavation should be restrained with rock bolts or anchors.

Shoring piles may be used to carry vertical structural loads and may be designed on the basis of the allowable foundation pressures given in Section 8.5. A reduction in bearing pressure will generally apply for piles founded close to, or on the edge of vertical (or steep) rock excavations.

Suitably sized drilling rigs fitted with rock augers will be required to penetrate medium and high strength rock and coring buckets may be required to penetrate high to very strength rock. Piling contractors are encouraged to inspect the rock cores at the DP office in West Ryde in order to assess requirements for drilling equipment.

8.3.4 Design

The design of the shoring will depend somewhat upon whether it is cantilevered or restrained by multiple rows of temporary rock anchors. It is anticipated that at least one or two rows of rock anchors will be required to provide lateral restraint to shoring piles above the top Class II-I rock.

It is suggested that design of cantilevered shoring systems (or shoring with a single row of anchors) be based on a triangular earth pressure distribution based on earth pressure coefficients provided in Table 6. Active earth pressures (Ka) may be used where some wall movement is acceptable, and at rest earth pressures (Ko) should be used where wall movement is to be minimised.

Table 6 – Recommended Earth Pressure Coefficients and Bulk Unit Weights

Material	Earth Pressure Coefficient		Bulk Unit Weight (kN/m^3)
	Active (Ka)	At Rest (Ko)	
Filling and Residual clay	0.35	0.5	20
Class V Rock	0.3	0.45	22
Class IV Rock	0.2	0.3	22
Class III Rock	0.1	0.2	22

All surcharge loads should be allowed for in the shoring design including building footings, inclined slopes behind the wall, traffic and construction related activities.

Design for lateral earth pressures for walls with more than one row of anchors may be based on a uniform rectangular earth pressure distribution. The additional lateral pressures due to surcharge loading behind the wall and hydrostatic pressures (if appropriate) must also be considered. For situations where movements are critical, a uniform pressure of between 6H may be adopted (where H is the depth to the top of the Class III rock). Where lateral movement is less critical (as expected for this site) a lower pressure distribution of 4H may be considered.

The design of temporary and permanent support will need to consider the possibility that 45° joints in the rock will daylight near the base of the shoring wall leading to wedges of rock which need to be supported by the temporary and permanent retaining structures. The support system would typically comprise rock bolts or anchors spaced at 2 m to 3 m centres over the rock face. These anchors should have their bond lengths formed in rock behind a line projected up at 45° from the base of the excavation. As a guide, the support system should be designed to withstand a horizontal force per unit width of $4.2H^2$ (kN) where H is the height of the excavation in metres. This approximation of the horizontal force required to support a 45° wedge is based on an anchor inclination of 10° below horizontal, an average bulk weight of 21 kN/m^3, and friction angle of 25° and cohesion of 0 kPa along the failure plane. Given that there is a low probability that a joint would run the full length and height of the excavation it suggested that this design may be carried out for a factor of safety of 1.1.

Passive resistance for piles founded below the base of the excavation may be estimated from the passive pressures provided in Table 7. Passive resistance should be assumed to start at least 0.5 m below bulk excavation level.

Table 7 – Allowable Passive Resistance for Piles

Material Description	Allowable Passive Resistance (kPa)
Class V Rock	200
Class IV Rock	300
Class III Rock	600
Class II Rock (or better)	2000

Shoring walls should be designed for full hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided. 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.

Inspection of the cut faces must be carried out by an experienced geotechnical engineer during the excavation phase to confirm assumptions and check the adequacy of design.

8.3.5 Ground Anchors

The design of temporary and permanent ground anchors for the support of excavations and/or shoring systems may be carried out on the basis of the maximum allowable bond stresses given in Table 8. These bond stresses may also be adopted for design of vertical ground anchors below the floor slab, if required.

Table 8 – Bond Stresses for Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)
Class V Rock	80
Class IV Rock	100
Class III Rock	300
Class II Rock (or better)	500

The parameters given above assume that anchor holes are clean and adequately flushed. The anchors should be bonded behind a line drawn up at 45° from the base of the shoring, and "lift-off" tests should be carried out to confirm the anchor capacities. Higher bond stress values may be adopted if trial anchors are used to prove higher capacities. It should be noted that permission will be required from adjacent property owners prior to installing bolts/anchors below their land. In addition, care should be taken to avoid damaging buried services or pipes during anchor installation.

It is anticipated that the building will restrain the basement excavation over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors, if required, would generally require careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.

8.4 Excavation Induced Ground Movements

For deep rock excavations, as proposed for the north-western part of the site, there is a possibility that there will be some horizontal movement due to stress relief effects. Release of these stresses due to the excavation will generally cause horizontal movements along the rock bedding surfaces and partings. Generally, it is not practicable to provide restraint for the relatively high in-situ horizontal stresses.

Based on monitoring experience for excavations in the Sydney region, excavations of over 70 m in length may give rise to lateral stress relief movements on the adjacent ground surface in the order of 1 mm to 2 mm per metre depth of excavation. On this basis, lateral movements in the order of 10 mm to 20 mm may be expected for excavations up to 10 m deep. It is noted that this estimate of ground movement generally relates to Hawkesbury Sandstone and stress relief movements are likely to be less within the more fractured laminite present on the site. Empirical data suggest that most of the movement occurs during or shortly after the bulk excavation phase.

8.5 Foundations

Following bulk excavation it is anticipated that variable foundation materials will be exposed , including medium to high strength (Class III) rock on the western part of the site, very low to medium strength (Class IV) rock on the central part of the site and filling and clay approximately 1 m to 2m thick overlying rock on the eastern part of the site.

All structural loads should be uniformly supported on underlying bedrock for which pad footings should be appropriate. Deepened pad footings or bored piles will be required in the eastern part of the site to reach rock and may also be used to reach the underlying Class III or better rock. It is expected that Class III rock will be exposed at bulk excavation level on the western part of the site with the depth to Class III rock gradually increasing to approximately 5 m below bulk excavation level on the eastern part of the site.

Depending on the final design and building layout it is possible that some columns/footings may be required behind the basement excavation. Where pad footings or pile shafts are within a line extending upwards at an angle of 45° from the base of adjacent excavations a reduction of the allowable bearing pressure or shaft adhesion parameters may be appropriate. Generally the design parameters provided in Table 9 should be halved for footings and piles close to adjacent excavations, however specific advice should be sought when the column/pile layout is confirmed.

It is expected that uncased bored piles could be used for the site. However, some provision is recommended for temporary casing support within the filling and soil and the use of submersible pumps to dewater pile holes immediately prior to concrete placement.

Recommended maximum allowable pressures and modulus values for the various foundation materials encountered within the boreholes are presented in Table 9. These parameters apply to the design of spread foundations, such as pads or strip footings, or for rock socketed bored piles.

Table 9 – Recommended Design Parameters and Modulus Values for Foundation Design

Foundation Stratum	Classification(1)	Maximum Allowable Pressure		Field Elastic Modulus (MPa)
		End Bearing (kPa)	Shaft Adhesion(2) (kPa)	
Residual Clay	Stiff or better	150	-	30
Laminite or Sandstone	V	700	70	100
	IV	1000	100	200
	III	3500	350	500
	II-I	6000	600	1000

Notes:
1) Rock classification based on Reference 1
2) Shaft adhesion applicable for the design of bored piers, uncased over rock socket length, where adequate sidewall cleanliness and roughness achieved.

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 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.

Spoon testing (or proof core drilling) should be undertaken in at least one-third of high level footings which are proportioned on the basis of allowable bearing pressures of between

3500 kPa and less than 6000 kPa. For spread footings designed using allowable bearing pressures of 6000 kPa, spoon testing should be undertaken in at least half of all footing locations. The purpose of spoon testing is to check that no significant weak seams exist below the base of the footing within a depth equal to 1.5 times the least footing dimension. If the testing identifies the presence of weak seams then the footing will either have to be deepened or widened to reduce the actual bearing pressure.

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).

If the exposed subgrade is unsuitable (i.e. heaving) then it will generally be necessary to construct a bridging layer. Such treatment may be required if pavements are to be constructed on the deeper filling encountered along the south-eastern side of the site. The extent of the bridging layer and most suitable form of construction should be determined on site by a geotechnical engineer. As a guide, a bridging layer could be constructed by excavation to a depth of 0.5 m followed by placement of a geofabric layer then compacted granular material (possibly including medium to high strength ripped sandstone from the site).

Subject to the subgrade preparation outlined above, the design of pavements on filling or clay subgrade may be based on a CBR value of 5%. Design of pavements on weathered rock may be based on a CBR value of 5% for Class V rock and 10% for Class IV rock or better. 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.

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 0.67 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 low to medium strength (i.e. Class III or better).

DOUGLAS PARTNERS PTY LTD

Reviewed by

Scott Easton
Associate

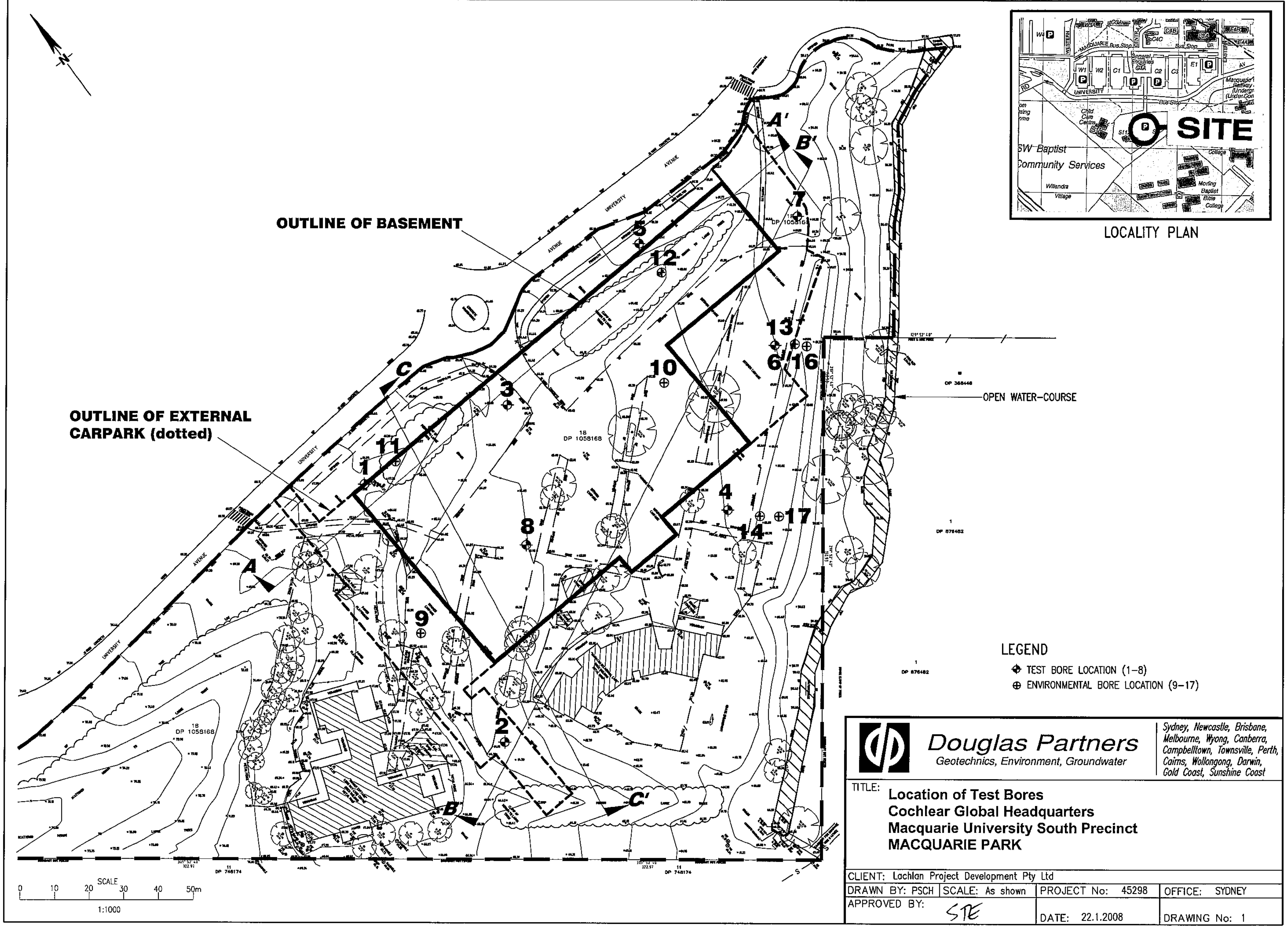
Fiona MacGregor
Principal

Reference

1. Pells, P.J., Mostyn, G., and Walker, B.F. *"Foundation on Sandstone and Shale in the Sydney Region"*. Australian Geomechanics Journal, Vol. No. 33 Part 3, Dec. 1998.

APPENDIX A

Drawing No. 1 to 4



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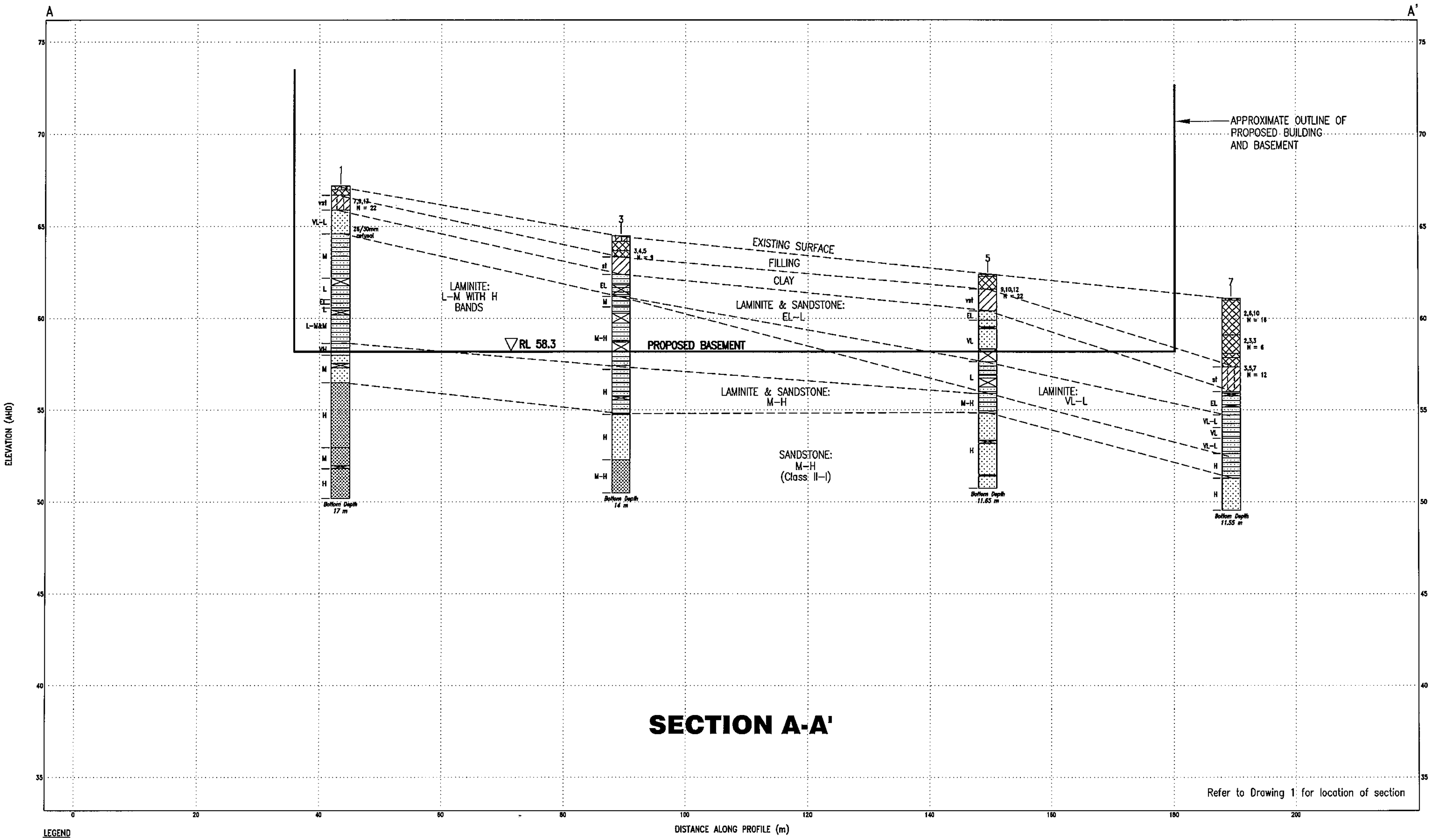
TITLE: **Location of Test Bores**
Cochlear Global Headquarters
Macquarie University South Precinct
MACQUARIE PARK

CLIENT: Lachlan Project Development Pty Ltd			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 45298	OFFICE: SYDNEY
APPROVED BY: STE		DATE: 22.1.2008	DRAWING No: 1



LEGEND

Topsoil	Laminite	Topsoil
Filling	Core Loss	Silty Clay
Silty Clay	Sandstone coarse grained	
Sandstone fine grained	Clay	

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

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

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

0 5 Vertical Scale (metres)
0 12 Horizontal Scale (metres)

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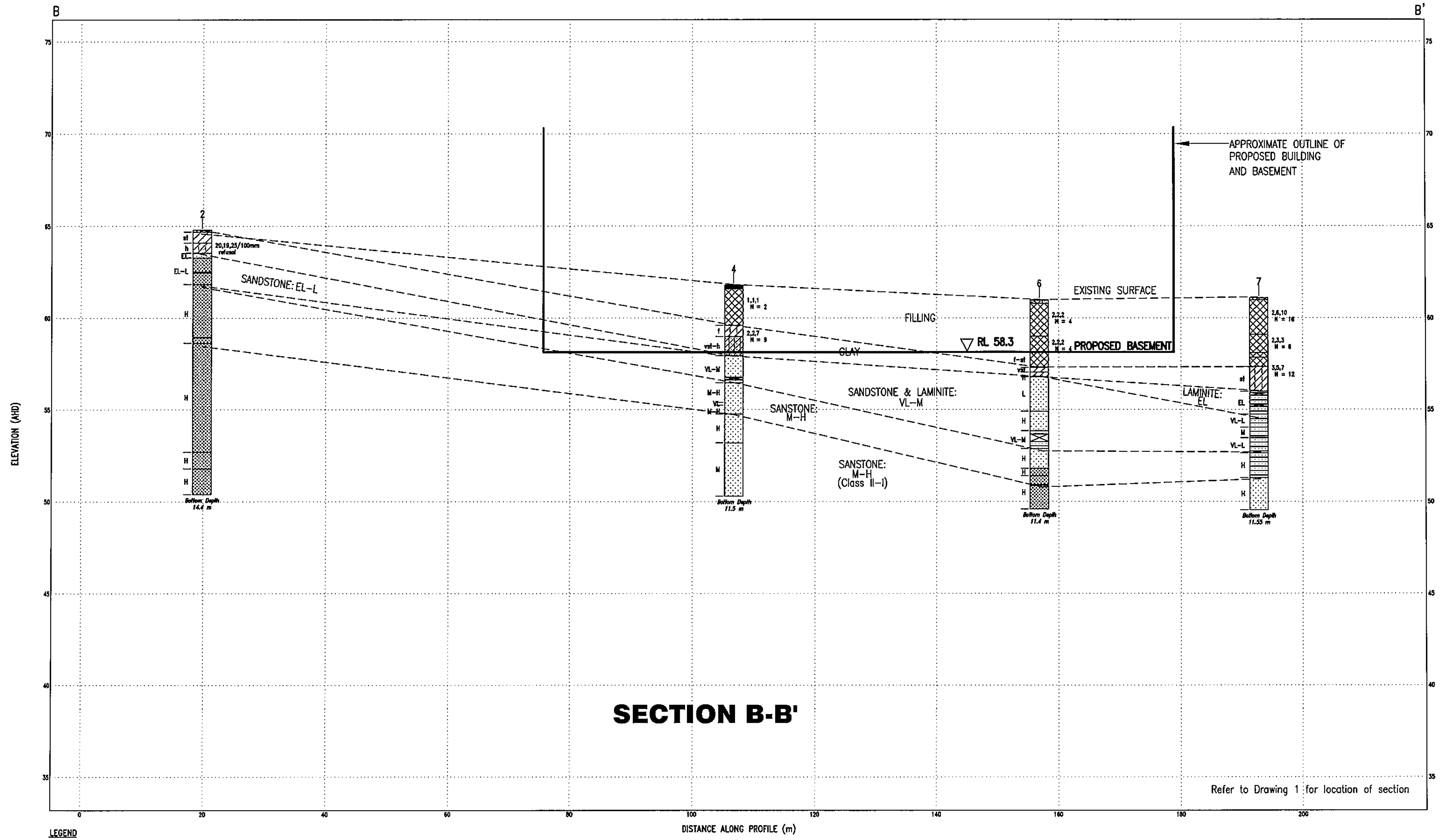
CLIENT: Lachlan Project Development Pty Ltd			TITLE: Section A–A' Macquarie University South Precinct
PROJECT: Cochlear Global Headquarters			
APPROVED DP: STE	DATE: 22 January 07	REVISION:	SCALE: As shown
PROJECT No: 45298	DRAWING No: 2		



SECTION B-B'

LEGEND

Topsoil	Sandstone coarse grained	Silty Sandy Clay
Clay	Core Loss	Roadbase
Silty Clay	Bituminous Concrete	Laminite
Sandstone fine grained	Filling	Topsoil

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

SOIL CONSISTENCY
- f – firm
- st – stiff
- vst – very stiff
- h – hard

TESTS / OTHER
- N – Standard penetration test value
- ∇ – Water level

Vertical Scale (metres) 0 5

Horizontal Scale (metres) 0 12

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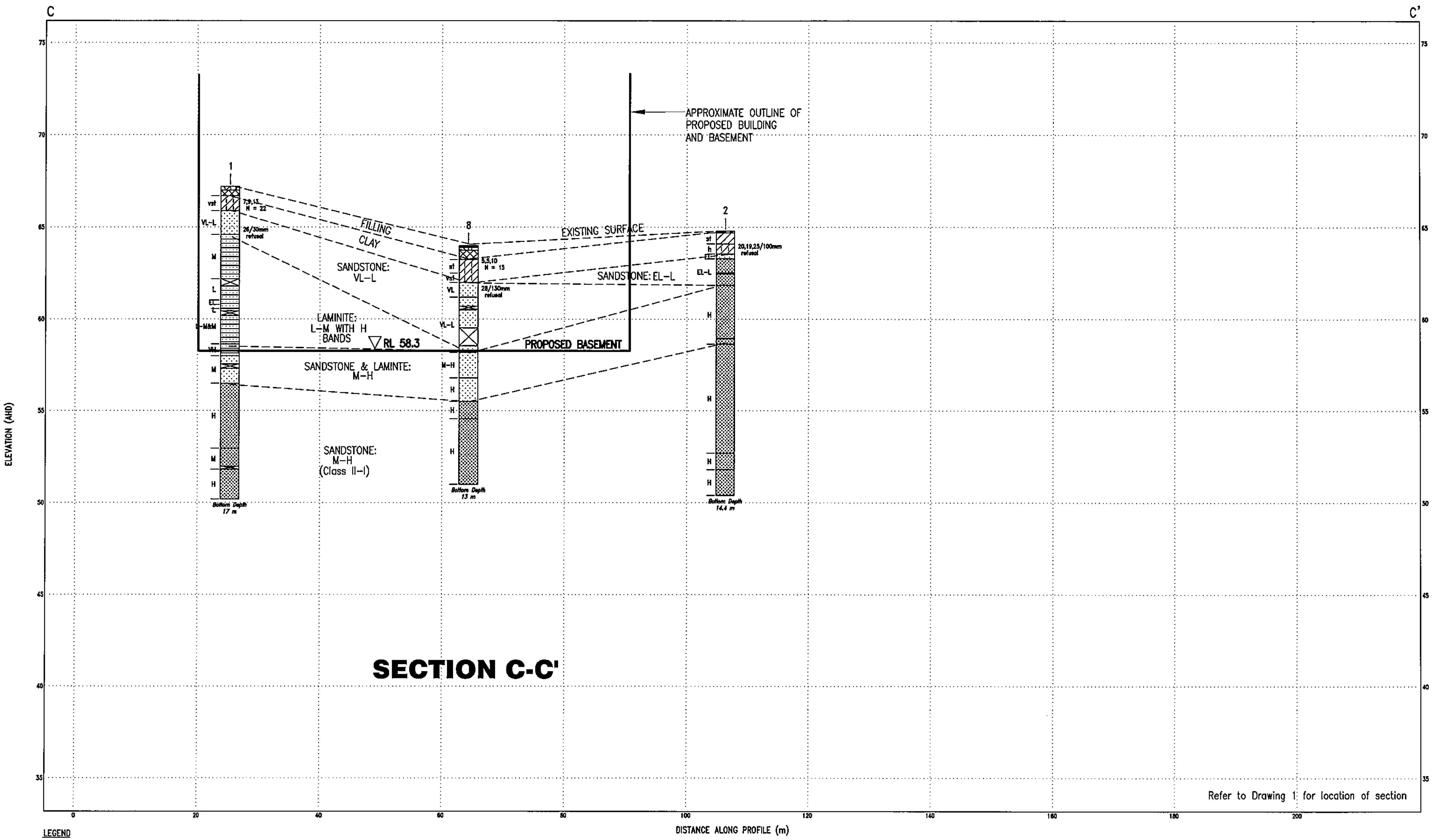
PROJECT: Cochlear Global Headquarters					
APPROVED DP: STE	DATE: 22 January 07	REVISION:	SCALE: As shown	PROJECT No: 45298	DRAWING No: 3



LEGEND

- Topsoil
- Filling
- Silty Clay
- Sandstone fine grained
- Laminite
- Core Loss
- Sandstone coarse grained
- Clay
- Bituminous Concrete
- Gravel

ROCK STRENGTH

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

SOIL CONSISTENCY

- f – firm
- st – stiff
- vst – very stiff
- h – hard

TESTS / OTHER

- N – Standard penetration test value
- ∇ – Water level

Vertical Scale (metres): 0 – 5
Horizontal Scale (metres): 0 – 12

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CLIENT:	Lachlan Project Development Pty Ltd
PROJECT:	Cochlear Global Headquarters
APPROVED DP:	STE
DATE:	22 January 07
REVISION:	
TITLE:	Section C–C' Macquarie University South Precinct
SCALE:	As shown
PROJECT No:	45298
DRAWING No:	4