NORTHERN SYDNEY HEALTH

PRELIMINARY GEOTECHNICAL INVESTIGATION

ROYAL NORTH SHORE HOSPITAL REDEVELOPMENT

ST LEONARDS

S21855/3-AC

3 August 2004

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3 August 2004

Northern Sydney Health
Redevelopment Project Office
Level 5 Vindin House
Royal North Shore Hospital
ST LEONARDS NSW 2065

Attention: Mr John Machon

Dear Sir,

RE: PRELIMINARY GEOTECHNICAL INVESTIGATION
ROYAL NORTH SHORE HOSPITAL REDEVELOPMENT
ST LEONARDS

Coffey Geosciences Pty Ltd is pleased to present the results of the preliminary geotechnical investigation for the proposed redevelopment of Royal North Shore Hospital at St Leonards. If you require further information on the report please contact the undersigned on 9911 1000.

For and on behalf of

COFFEY GEOSCIENCES PTY LTD

Peter Waddell

PETER WADDELL

Associate

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1. INTRODUCTION

Coffey Geosciences Pty Ltd (Coffey) has carried out a preliminary geotechnical investigation for a proposed redevelopment of Royal North Shore Hospital (RNSH), St Leonards. The investigation was commissioned by Northern Sydney Health (NSH) in a letter dated 12 May 2004. The investigation was generally carried out in accordance with our proposal, reference S21855/1-AB, dated 4 May 2004, with variations based on discussions and agreement with NSH.

We understand that a redevelopment is being considered for a substantial proportion of the RNSH Site, bounded by Herbert Street, Pacific Highway, Reserve Road, and the former Westbourne Street, St Leonards (see Figure 1 for site location). The investigation area generally contains older sections of the hospital complex, with buildings typically comprising multi storey brick construction. Details of the proposed redevelopment are not known at the time of the geotechnical investigation. However, we have assumed that redevelopment is likely to comprise demolition of existing buildings, excavation of basements for car parking, and construction of multi storey concrete frame buildings. Bridges may link new buildings with the main hospital buildings to the west of the older section.

The objectives of the geotechnical investigation were to assess subsurface conditions on a broad scale across the investigation area, provide a preliminary geotechnical model, and preliminary discussion and recommendations on geotechnical aspects such as excavation conditions, excavation support requirements, suitable footing types, design parameters for retaining walls, foundations and pavements.

Environmental assessments were carried out concurrently with the preliminary geotechnical investigation and are presented in separate reports.

2. METHOD OF INVESTIGATION

2.1 Fieldwork

A truck mounted drilling rig was used to drill 12 boreholes (BH1 to BH12) to depths ranging from 10.4m to 20m. The boreholes were drilled to refusal using solid flight augers fitted with a steel 'V' shaped bit or tungsten carbide bit. A triple tube NMLC core barrel was used to obtain cores of the rock from each of the boreholes. Standard penetration tests were carried out in soils to assess strength and to obtain samples for logging.

PVC standpipes were installed in three boreholes (BH4, BH5, and BH9) to allow water levels to be monitored and to obtain groundwater samples.

An engineer from Coffey set out the boreholes from existing site features, directed sampling and testing, logged and boxed the rock cores. Engineering logs of the boreholes are presented in Appendix A, together with Explanation Sheets defining the terms and symbols used in the preparation of the logs.

The project surveyors, Frank M Mason & Co Pty Ltd, surveyed the locations and ground elevations of the boreholes. These locations and levels are presented on the engineering logs. The locations of the boreholes are shown on Figure 2.

2.2 Laboratory Testing

Groundwater samples were tested for pH, Sulphate and Chloride Content to assess the potential for aggression to buried concrete and steel. The laboratory test results are presented in Appendix B.

Rock cores were colour photographed and point load strength index tests were carried out. The results of the point load strength index tests are shown on the borehole logs.

3. RESULTS OF INVESTIGATION

3.1 Geology

The Sydney 1:100,000 Geological Sheet indicates that the site locality is underlain by Ashfield Shale described as black to dark grey, shale and laminite. Hawkesbury Sandstone described as medium to coarse grained quartz sandstone with very minor shale and laminite lenses underlies the shale. There is also an intermediate unit known as the Mittagong Formation, which is found between the Hawkesbury Sandstone and Ashfield Shale in some locations, which is described as interbedded shale, laminite, and medium grained quartz sandstone.

The Ashfield Shale is likely to be of considerable thickness at higher elevations of St Leonards such as the northern part of the investigation area. At lower elevations, the shale is likely to be less thick until the underlying Mittagong Formation (if present) and Hawkesbury is encountered.

3.2 Subsurface Conditions and Preliminary Geotechnical Model

Based on the results of the fieldwork, the geology within the investigation area is consistent with the regional geology indicated by the Sydney 1:100,000 Geological Sheet. Across the investigation area there are relatively shallow and variable thicknesses of topsoil and fill, overlying residual soil, overlying bedrock. The bedrock comprises shales and sandstone of variable strength and weathering characteristics.

At the more elevated, northwest end of the investigation area, Ashfield Shale was encountered to the termination depth of the boreholes. As the site grade falls to the east and south, the boreholes located in these areas encountered Ashfield Shale overlying laminite and sandstone inferred to be Mittagong Formation/Hawkesbury Sandstone. At the most southern and eastern boreholes (BH3 and BH9) the shale was not present and sandstone belonging to either the Mittagong Formation or Hawkesbury Sandstone was cored. It should be noted that the boundaries between the various units can be gradational and the Mittagong Formation and Hawkesbury Sandstone can be difficult to differentiate.

The subsurface conditions and inferred geotechnical models are summarised on Sections A-A’, B-B’ and C-C’ in Figure 3 and Sections D-D’ and E-E’ in Figure 4 and the following table. It should be noted that the depths and thicknesses of various units presented is based on information at the borehole locations and variations outside of the ranges of depth and thickness could occur between borehole locations.

TABLE 1: SUMMARY OF SUBSURFACE CONDITIONS AT BOREHOLE LOCATIONS AND PRELIMINARY GEOTECHNICAL MODEL

Unit	General Description	Typical Depth to Unit Base (m)	Typical Thickness of Unit (m)
Topsoil and Fill	Silty Clay and Clayey Silt, some Gravely Clay	0.3 to 2	0.3 to 2
Residual Soil	Silty Clay: Medium and high plasticity, very stiff and hard	1.1 to 3.55	0.6 to 3.3
Ashfield Shale	Shale with Sandstone Laminations and Shale: • Extremely weathered to fresh; • Very low to high strength; • Class V (1), overlying generally Class IV and Class III Shale, some Class II Shale; • Not encountered in BH3 and BH9	7.36 to >20	4.76 to >16.2
Mittagong Formation/ Hawkesbury Sandstone	Interbedded Sandstone, Laminite and Shale, less laminated and predominantly Sandstone with increasing depth: • Highly weathered to fresh; • Fine and medium grained Sandstone; • Generally medium and High strength; • Class IV and Class III Sandstone in the upper few metres, becoming Class III or better Sandstone at depth; • Encountered in BH1, BH3, BH4, BH8, BH9, and BH11.	Drilled to the termination depth of BH1, BH3, BH4, BH8, BH9, BH11.	-

Notes: (1) Rock classified in accordance with Pells et al (1998) "Foundations on Sandstone and Shale in the Sydney Region" Aust. Geomech. Jnl. Dec 1998.

3.3 Groundwater

Groundwater was generally not encountered in the augered sections of the boreholes (soil and extremely weathered rock). Water was used as a drilling fluid in the cored sections of the boreholes, hence groundwater were not be monitored during the drilling. In the open standpipe piezometers that were installed in BH4, BH5, and BH9, the groundwater levels presented in the following table were monitored during the investigations.

TABLE 2: GROUNDWATER LEVELS MEASURED IN STANDPIPE PIEZOMETERS

Borehole Number	Measurement Date	Depth Below Ground Surface (m)
BH4	10/6/04	5.00
	17/6/04	4.39
	22/6/04	4.60
BH5	10/6/04	8.45
	17/6/04	8.65
	22/6/04	8.58
BH9	15/6/04	6.62
	17/6/04	6.68
	22/6/04	6.70

Drilling fluids may influence the earlier groundwater readings and the last reading is likely to be the more representative of a standing groundwater level at the borehole locations. Groundwater levels in Sydney are often associated with infiltration through fissured soils into the fractured rock mass and may vary seasonally. Perched groundwater is often encountered at the soil bedrock interface and within joints and bedding partings within the bedrock.

3.4 Laboratory Test Results

The results of the pH, Sulphate and Chloride tests are presented in Appendix B and are summarised in the following table.

TABLE 3: LABORATORY TEST RESULTS ON WATER SAMPLES

Sample	pH	Soluble Sulphate (SO_4 ppm)	Chloride (ppm)
BH4	5.1	149	154
BH5	5.8	124	466
BH9	5.5	1,130	122

4. PRELIMINARY DISCUSSION AND RECOMMENDATIONS

4.1 General

The subsurface conditions revealed by the boreholes are typical of the conditions that would be anticipated in this locality. There should generally not be significant geotechnical constraints to developments consisting of basement excavations and typical foundation loadings from multi storey buildings.

Rock quality, particularly in the Ashfield Shale unit, is relatively variable and significant depths of poor quality rock were identified in some boreholes. The presences of very low strength Class V and IV Shale could impact on support requirements for excavations, ground anchor and foundation design parameters.

Parameters for excavation support and foundation design are provided for the various geotechnical units for preliminary design purposes. However, given the assessed variability in rock quality within each geotechnical unit, additional site specific investigations will be required to further assess the quality and variability of rock, once building layouts and excavation depths are known.

4.2 Excavation Conditions

The fill and residual soil should be able to be excavated using hydraulic excavators and tracked loaders. Once the rock is encountered, dozers fitted with rippers are likely to be required for bulk excavation and impact hammers, rock saws and milling heads for detailed trimming. Potential contractors should be required to assess the rock cores, engineering logs and photographs to make their own assessment of equipment requirements and production rates.

The excavation methods presented in the following table are suggested as a guide only for the geotechnical units that will be encountered. Excavation contractors should assess the borehole logs and inspect the rock core to make their own assessment of suitable equipment and production rates for excavations.

TABLE 4 – SUGGESTED EXCAVATION METHODS

Geotechnical Unit	Excavation Method
Fill and Residual Soil	Excavated using a hydraulic excavator or bulldozer blade and bucket.
Class V and IV Shale (Applicable to some Ashfield Shale)	Excavated using a hydraulic excavator or Class 200C Crawler Tractors* (Cat D9L or equivalent). Ripping may be required where higher strength bands occur.
Class III and II Shale and Class IV and III Sandstone (Applicable to some Ashfield Shale, Mittagong Formation and Hawkesbury Sandstone)	Excavated using a hydraulic excavator or Class 200C Crawler Tractors* (Cat D9L or equivalent), however, productivity may be low. Should be readily excavated using Class 300/400C Crawler Tractors (Cat D10 or equivalent)
Class II and I Sandstone (Applicable to Mittagong Formation and Hawkesbury Sandstone)	Ripped with Class 300/400C Crawler Tractors (Cat D10 or equivalent). Production rates may be low, if feasible, using Class 300/400C Crawler Tractors in confined spaces. Rock hammers may be required to cut trenches for rippers to work between and for trimming along excavation boundaries.

Note: * Based on the classification system in AS2868-1986 “Classification of Machinery for Earthmoving, Construction, Surface Mining and Agricultural purposes.”

4.3 Vibrations

Plant used for excavations could cause vibration damage to nearby structures. We recommend that the following steps be taken to assess and manage the risks posed by vibrations:

- Carry out an assessment of the proximity of vibration sensitive structures to excavations;
- Carry out dilapidation surveys on vibrations sensitive structures before excavation commences and on the completion of excavations; and
- Prepare a vibration management plan setting limits on Peak Particle Velocity (PPV) and install, where required, monitoring systems to assess vibrations.

4.4 Groundwater

Groundwater levels are likely to be variable across the investigation area depending on factors such as structure within the soils and rock, seasonal variations in recharge, and other factors such as proximity to nearby excavations. Groundwater inflows into basement excavations will be dependent on a number of factors, including groundwater level, size, location and depth of excavation, retaining wall depth and permeability and defects in the rock mass (e.g. fractures) intersected by the excavation.

We would expect seepage into excavations through fissures in the residual soil and from joints and bedding planes in the rock. The permeability of the rock mass should generally be relatively low, although localised inflows could be high. We consider that it should be possible to control such inflows by installing a collection system of sumps and pumps in the base of excavations.

4.5 Excavation Induced Ground Movements

Ground movements will occur due to basement excavations. In the shored sections of excavations, lateral and vertical ground movements will be dependent on the design and construction of the shoring retention system. Published data suggests that lateral movements of an adequately designed and installed retention system in stiff clay and weathered rock will be between 0.2% and 0.5% of the retained height. Where Class V and Class IV Shale is retained, movements are likely to be at the lower end of the range. Vertical movements could be expected to be of a similar order to lateral movements.

Vertical excavations in the better quality rock will also result in lateral movement of the rock faces into the excavation due to in-situ stress relief.

Based on our experience and published data we expect the following ground movements could occur:

- Between about 0.5mm to 2mm horizontal movement per metre depth of excavation
- The extent of the horizontal movement behind the excavation face of typically between 1.5 and 3 times the excavated height.

The amount and timing of movement will be dependent on the depth of excavation and the location and condition of bedding defects (such as weak bands or bedding partings). The recommended ranges of movement presented above are for typical shoring and support systems. Modifications to shoring systems such as increasing anchor loads above those typically adopted may result in some reduction of movement. However, the extent of the reduction will be highly dependent on the excavation and construction sequence and it may be difficult in practice to achieve significant reductions in movement.

We understand that movement sensitive buried services such as oil cooled power lines are located within the investigation area. Such services are likely to lie within the upper few metres of the ground surface and hence could be impacted upon by ground movements induced by nearby excavations. For a rectangular excavation, the maximum movements due to the excavation are likely to occur at the crest of the middle of the excavation face. Ground movements at the corners of the excavation and at a horizontal distance back from the crest of between 1.5 and 3 times the excavation height are likely to be negligible.

Based on the above guidelines of typical movements, we estimate that the lateral displacement at the crest of say a 6m deep excavation with shoring extending to 3m depth could be up to about 20mm. If the excavation were say 20m wide, the gradient of displacement between the mid point of the crest of the excavation and corner would be 1:500.

4.6 Excavation Support Requirements

4.6.1 Unsupported Cuts

Recommendations for unsupported cuts batters are presented in the following table:

TABLE 5: CUT BATTER RECOMMENDATIONS

Geotechnical Unit	Temporary Cuts (Horizontal to Vertical Ratio) (1)	Permanent Cuts (Horizontal to Vertical Ratio) (1)
Fill	2H:1V	2.5H:1V
Residual Soil	1.5H:1V	2H:1V
Class V Shale Class IV Shale or better (Generally applicable to Ashfield Shale)	1.5H:1V 1.5H:1V	2H:1V 1.5H:1V
Class IV Sandstone or better (Generally applicable to Mittagong Formation and Hawkesbury Sandstone)	Vertical (2)	Vertical (2)

Note:

1) The above recommendations assume that:
   - excavations in soil are above the groundwater table;
   - the ground surface at the crest of the excavation is horizontal;
   - surcharge loads are kept clear of the crest of the excavation for a distance equal to the depth of the excavation;
   - All cuts are protected from erosion.

   Batters should be reassessed if the above criteria are not met.
2) Vertical excavations should be feasible in Mittagong Formation and Hawkesbury Sandstone where the rock is Class IV or better Sandstone, provided support requirements as detailed below are assessed and installed.

In Ashfield Shale there is a risk that relatively shallow dipping joints or shear zones (30° to 40° from the horizontal) daylight in the excavation face forming unstable wedges. Such joints or shear zones are known to occur within the Ashfield Shale and rarely in the Mittagong Formation and Hawkesbury Sandstone. Jointing in the Mittagong Formation and Hawkesbury Sandstone is typically steeper (>70°) and hence less likely to form large wedges. Therefore, it may be necessary to install relatively heavy support such as shoring walls for deep vertical excavations in Ashfield Shale.

4.6.2 Vertical Excavations in Rock

Provisions for the installation of support will be required where vertical excavations are made in rock. Specific rock support requirements in the un-shored sections of excavations can only be assessed during excavation. An experienced geotechnical engineer/engineering geologist should carry out regular inspections as excavation progresses (at least every 2m depth of excavation). For preliminary design purposes the support guidelines presented in the following table are recommended:

TABLE 6: VERTICAL EXCAVATION FACE SUPPORT REQUIREMENTS

Geotechnical Unit	Material/Rock Class	Support
All Units	Fill, Residual Soil, Class V Shale and Sandstone	• Shoring wall
Ashfield Shale	Class IV Shale or better	• Shoring wall; or • Anchors, mesh supported by 0.5m long dowels and shotcrete (minimum 75 mm thick) or fibre reinforced shotcrete of fractured zones below shoring walls.
Mittagong Formation and Hawkesbury Sandstone	Class IV Shale and Sandstone	• Shoring wall: or • Pattern rock bolting, mesh supported by 0.5m long dowels and shotcrete (minimum 75 mm thick) or fibre reinforced shotcrete of fractured zones below shoring walls.
	Class III Sandstone	• Generally only isolated rock bolting to support potentially unstable wedges. • Localised pattern rock bolting, mesh supported by 0.5m long dowels and shotcrete (minimum 75 mm thick) or fibre reinforced shotcrete of fractured zones
	Class II Sandstone or better	• Generally only isolated bolting of potentially unstable wedges required.

Where long-term support is required below the site retention system, anchors and rock bolts must be provided with a high level of corrosion protection if they cannot be maintained (i.e. inspected and replaced, if necessary). Multiple layers of corrosion protection such as encapsulating bolts in both grout and PVC sheaths may be required.

4.6.3 Retaining Walls

Conventional shoring such as contiguous bored pile walls or anchored soldier piles with walers and timber panels, or shotcrete infill panels should be suitable. Ideally, the pile toes should be founded below the bulk excavation level, or support measures such as the installation of toe anchors or bolts are required to prevent the kick-out of the pile toe.

Drilling contractors tendering for the soldier pile drilling should be required to inspect the rock core, engineering logs and photograph and to make their own judgement as to likely drilling productivity and drilling rig requirements.

Cantilever walls are generally limited to a retained height of about 3m. Therefore, temporary anchors or internal props will be required where more than one basement level is planned.

The following table provides parameters for retaining wall design for the following cases:

- Case 1 = temporary retention, no adjacent footings.
- Case 2 = permanent retention, no adjacent footings.
- Case 3 = adjacent footings and hence need to limit movement.

TABLE 7: EARTH PRESSURE COEFFICIENTS

Geotechnical Unit	Value of Lateral Earth Pressure Coefficient, K (1)			Passive Earth Pressure Coefficient, Kp (1,2)	Bulk Density (kN/m^3)
	Case 1	Case 2	Case 3		
Fill and Residual Soil	0.3	0.35	0.5	2.5	20
Class V and IV Shale and Class V Sandstone	0.25	0.3	0.4	2.5	22
Class III or better Shale and Class IV or better Sandstone	-	-	-	3	24

Note:

1. These values are only applicable for a horizontal ground surface.
2. Passive earth pressure coefficients for rock have been reduced to allow for potential defects in rock mass

Design of cantilevered retaining walls should be based on a triangular pressure distribution adopting the earth pressure coefficients recommended in Table 7. Design of shoring and permanent retaining walls, which are anchored or strutted at several levels, can be based on a trapezoidal earth pressure distribution as presented in the following table. A trapezoidal earth pressure distribution may be appropriate for a one-material layer (layer thickness H) requiring retention. Where retention of a multi-layered material profile is required, modification of the distribution (including the definition of H) will be necessary.

TABLE 8: TRAPEZOIDAL PRESSURE DISTRIBUTION

Depth (m)	Horizontal Pressure (kPa)
0	$K.p_s$
0.25 H	$K (0.8.\gamma.H + p_s)$
0.75 H	$K (0.8.\gamma.H + p_s)$
H	$K.p_s$

Where:

K = earth pressure coefficient which depends upon material type; whether movement needs to be limited; whether temporary or permanent

p_s = design surcharge pressure (kPa)

H = thickness of layer being retained (m), assume maximum depth would typically be to the base of Class IV Shale or Class V Sandstone.

γ = density calculated based on the bulk density presented in Table 7.

If temporary anchors are required, anchor bond lengths should be within the rock. Preliminary design of anchors could be based on working bond stresses of 300kPa in Class III or better Shale and 600kPa in Class III or better Sandstone and Laminite. However, where the Ashfield Shale is very low strength, Class IV, to considerable depth (BH2, BH4, and BH6) the working bond stress may have to be limited to 100kPa.

Anchor designs should be based on allowing effective bonding to be developed behind an 'active zone' determined by drawing a line at 45° from the base of the soldier pile to intersect the ground surface behind the excavated face. Retaining structures should be designed against hydrostatic water pressures, unless effective drainage is provided or the ground dewatered behind the retaining structures. Applicable surcharge loads should be added to lateral earth pressures.

4.7 Foundations

Fill is unlikely to have been placed in a controlled manner and should considered to be unsuitable as a bearing stratum for footings. Strip and pad footings or bored piles will be suitable to support loads on the other geotechnical units. Open bored piles should be feasible, however, seepage could occur and provision for measures such as temporary liners, dewatering and cleaning of open bored piers will be required. Alternatively, continuous flight auger (CFA) piles could be adopted, which do not require cleaning and dewatering.

The following table presents parameters for the preliminary design of footings bearing on residual soil and the rock classes assessed as likely to be encountered within each unit. For detailed design, additional investigations may be required to assess subsurface conditions and provide parameters at the founding levels for specific structures.

TABLE 9: PRELIMINARY ALLOWABLE FOOTING DESIGN PARAMETERS

Geotechnical Unit	Rock Class	Allowable End Bearing Pressure (kPa) (1)	Allowable Shaft Adhesion (kPa) (2)
Residual Very Stiff and Hard Clay	-	175	Nil
Ashfield Shale	Class V Class IV Class III or better	700 1,000 2,000	70 100 200
Mittagong Formation and Hawkesbury Sandstone	Class IV Class III Class II or better	2,500 5,000 8,000	250 500 800

Notes:

1. Allowable bearing pressures assume a minimum embedment of 0.3m into the relevant material. The recommended end bearing pressures should result in settlement of <1% of minimum footing dimension.
2. Assumes a clean socket roughness category R2 or better. Shaft adhesion should only be assigned where the socket length is at least 3 pile diameters. The socket should be cleaned and roughened by a suitable scraper such as a tooth, orientated perpendicular to the auger shaft. In shorter sockets the majority of the load will be carried in the pile base.

Higher load capacities may be able to be assessed if a limit state approach was adopted and settlements calculated using higher end pressures and shaft adhesion values are found to be acceptable. More detailed investigations at specific building locations would be required to estimate ultimate design parameters and elastic modulus values.

It should be noted that the allowable bearing pressure of piles founded near the edge of an excavation, such as retention system piles, should be downgraded. The allowable bearing pressure of footings located within 2 footing widths from the edge of the pile to the crest of an excavation should be downgraded by multiplying the allowable bearing pressures by a factor of 0.6.

Prior to concreting, all footings should be inspected by a geotechnical engineer or engineering geologist to assess the exposed rock. Where the required allowable bearing pressure is greater than 1,000 kPa, footing assessment should also include spoon testing (or cored boreholes) to assess whether defects below the base of the footing are within tolerable limits for the respective rock class. We recommend initial allowance for spoon testing or coring in every footing. It may be possible to reduce the amount of spoon testing if consistent conditions are exposed in early testing or if rock below founding levels can be inspected in excavations such as for lift shafts.

Where there are localised excavations close to a major footing, the entire base of the footing should lie outside a line projected upwards at 45^{o} from the toe of the base of the excavation or the allowable bearing pressure should be re-assessed and downgraded, if necessary.

4.8 Earthquake Design Parameters

The subsurface profile generally comprises fill and residual soil overlying bedrock. Based on AS1170.4-1993 the following tabulated parameters are recommended for earthquake design:

TABLE 10: EARTHQUAKE DESIGN PARAMETERS

Parameter	Values
Acceleration Coefficient (a)	0.08
Site Factor (S)	0.67-1.0 *

Note: * An S of 1.0 should be adopted if structures were founded on residual soils or very low strength shale encountered in some boreholes. An S of 0.67 could be adopted where structures are supported on Class III or better Shale and Class IV or better Sandstone.

4.9 Groundwater Aggressiveness to Concrete and Steel

Comparison of the laboratory test results with the criteria in Table 6.1 of AS2159-1995 for concrete indicates that the groundwater samples from BH4, BH5, and BH9 would be classified as non-aggressive to concrete. No special protection measures would be required, other than to provide densely compacted concrete and normal cover to reinforcement steel in accordance with the relevant standards.

Comparison of the laboratory test results with the criteria in Table 6.3 of AS2159-1995 for steel indicates that the groundwater samples would classify as non-aggressive to steel.

4.10 Pavements

Specific sampling and testing for pavements has not been carried out. The residual soils across the investigation area are of medium and high plasticity and are likely to have a relatively low CBR value. We recommend that for preliminary design of pavements a CBR value of 3% be adopted. Once pavement areas are defined, further investigation should be carried out to assess design CBR values. Fill below pavement subgrade level should be excavated, assessed by a geotechnical engineer for suitability as a pavement subgrade and recompacted.

Fill to within 0.5m of subgrade level in pavement areas should be compacted to a minimum of 98% Standard Density Ratio at a moisture content within $\pm 2\%$ of Standard Optimum Moisture Content. Fill to a depth of 0.5m below subgrade level should be compacted to a minimum of 100% Standard Density Ratio.

5. FURTHER INVESTIGATIONS AND LIMITATIONS

The preliminary geotechnical model presented in this report is based on relatively widely spaced boreholes and at a stage where no specific details of proposed structures are available. The engineering logs describe subsurface conditions only at the specific borehole locations and inferred boundaries between geotechnical units may vary from those shown on the interpreted sections.

Ground conditions can vary over relatively short distances and it may be necessary to carry out additional investigations for specific excavation and building sites. Once specific development proposals are known a geotechnical review should be undertaken and, if necessary, additional investigations commissioned to provide the level of information required for assessing design parameters. A geotechnical engineer should be engaged to review subsurface conditions during construction stages and to confirm that subsurface conditions are consistent with design assumptions.

The attached document entitled "Important Information About Your Coffey Report" presents additional information on the uses and limitations of this report.

For and on behalf of

COFFEY GEOSCIENCES PTY LTD

Peter Waddell

PETER WADDELL

Associate

Information

Important information about your **Coffey** Report

Coffey

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of the subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Coffey

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they have incorporated the report findings.

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design toward construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

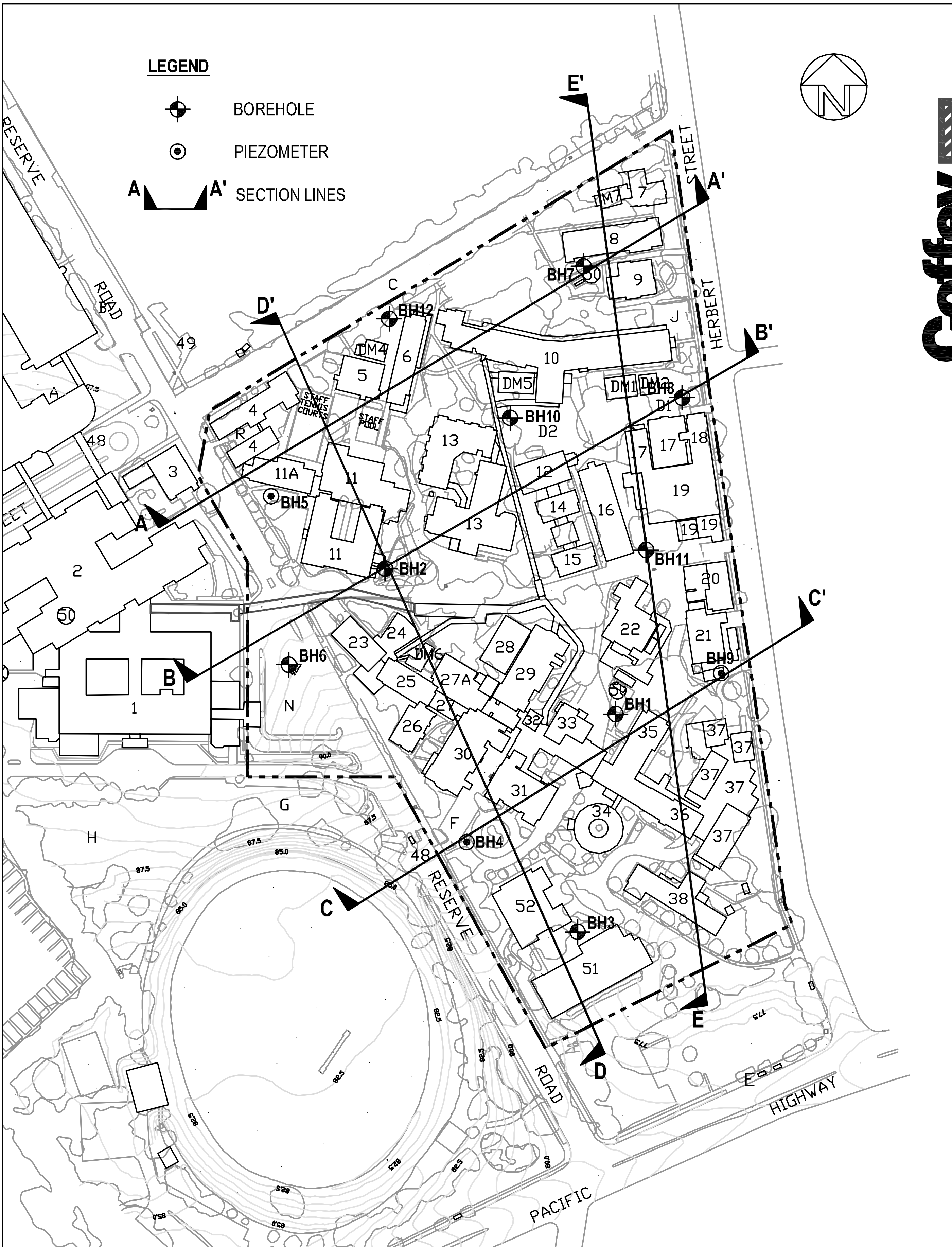
Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

** For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical Information in Construction Contracts" published by the Institution of Engineers Australia, National Headquarters, Canberra, 1987.*

Coffey Geosciences Pty Ltd ACN 056 335 516	Geotechnical I **Resources** I Environmental I **Technical** I Project Management

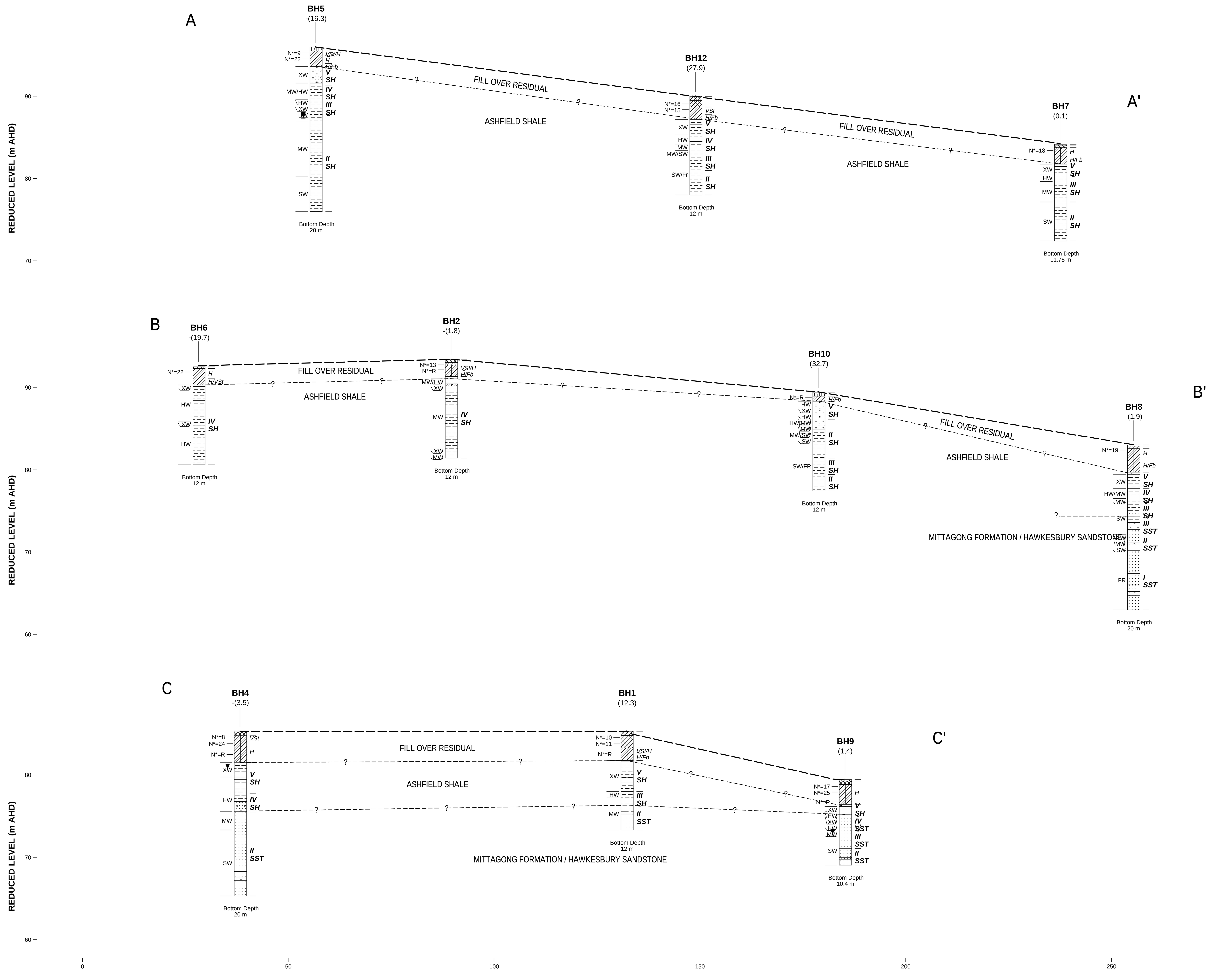
Drawn	EW/SW	NORTHERN SYDNEY HEALTH REDEVELOPMENT OF ROYAL NORTH SHORE HOSPITAL ST LEONARDS GEOTECHNICAL INVESTIGATION SITE LOCATION PLAN	FIGURE 1
Approved	PJW		
Date	3/8/04		
Scale	1:10,000		Job no: S21855/3-AC



Coffey Geosciences Pty Ltd ACN 056 335 516

Geotechnical | **Resources** | Environmental | **Technical** | Project Management

Drawn	EW/SW	NORTHERN SYDNEY HEALTH REDEVELOPMENT OF ROYAL NORTH SHORE HOSPITAL ST LEONARDS GEOTECHNICAL INVESTIGATION SITE PLAN SHOWING BOREHOLE LOCATIONS	**FIGURE 2**
Approved	PJW		
Date	3/8/04		
Scale	1:2500		Job no: S21855/3-AC



LEGEND

- ASPHALT
- TOPSOIL
- SILTY CLAY
- INTERBEDDED SHALE & SANDSTONE
- SHALE
- FILL
- NO CORE
- INTERLAMINATED SILTSTONE & SANDSTONE
- INTERLAMINATED CLAYSTONE & SILTSTONE
- SANDY SILTSTONE
- SILTSTONE
- SANDSTONE, FINE, FINE TO MEDIUM GRAINED
- SANDSTONE, MEDIUM, MEDIUM - COARSE GRAINED

IV ROCK CLASSIFICATION (PELLS ET AL, 1998)

DX WEATHERING (SEE EXPLANATION SHEETS)

WATER LEVEL

N*=17 STANDARD PENETRATION TEST RESULT

Coffey Geosciences Pty Ltd ACN 056 335 516		Geotechnical I Resources I Environmental I Technical I Project Management	
Drawn	PJW/SW	NORTHERN SYDNEY HEALTH REDEVELOPMENT OF ROYAL NORTH SHORE HOSPITAL ST LEONARDS GEOTECHNICAL INVESTIGATION GEOTECHNICAL SECTIONS A-A', B-B' & C-C'	FIGURE 3
Approved	PJW		
Date	3/8/04		
Scale	1:500H 1:250V @A1		Job No. S21855/3-AC

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Coffey Geosciences Pty Ltd ACN 056 335 516	Geotechnical I Resources I Environmental I Technical I Project Management		
Drawn	PJW/SW	NORTHERN SYDNEY HEALTH REDEVELOPMENT OF ROYAL NORTH SHORE HOSPITAL ST LEONARDS GEOTECHNICAL INVESTIGATION GEOTECHNICAL SECTIONS D-D' & E-E'	FIGURE 4
Approved	PJW		
Date	3/8/04		
Scale	1:500H 1:250V @A1		Job No. S21855/3-AC