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GEOTECHNICAL INVESTIGATION REPORT

HORNSBY KU-RING-GAI HOSPITAL REDEVELOPMENT

Prepared for

HEALTH INFRASTRUCTURE C/- THINC HEALTH

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EXECUTIVE SUMMARY

Aargus Engineering Pty Ltd (óAargusô) was requested to conduct a geotechnical investigation within Hornsby Ku-ring-gai Hospital NSW (óHospitalô).

The purpose of the geotechnical investigation was to assess the existing site and its subsurface conditions in order to provide recommendations from a geotechnical viewpoint for the conceptual scheme design and potential construction issues of a proposed development.

It is understood that the proposed development is currently at a concept phase and involves demolition of all existing structures and construction of up to five-storey buildings with no basements planned at this stage.

The findings of this investigation indicate that the subsurface stratum in general comprises a mixture of stiff to hard silty clays, overlying sandstone bedrock. Based on the results of this investigation, it is considered that the proposed concepts for the development is feasible in this site, subjected to the recommendations provided in this report.

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REFERENCES

1. Australian Standard ï Geotechnical Site Investigation, AS1726-1993.
2. Australian Standard ï Piling ï Design and Installation, AS2159-2009.
3. Australian Standards - Residential Slabs and Footings, AS2870-1996.
4. Australian Standards- Structural design actions, Part 4: Earthquake actions in Australia, AS1170.4-2007.
5. Australian Standards ï Guidelines on Earthworks for Commercial and Residential Developments, AS3798-2007.
6. Acid Sulphate Soils Management Advisory Committee (ASSMAC) (1998) ï Acid Sulphate Soils Assessment Guidelines
7. Aust roads Pavement Design Guide ï A Guide to the Structural Design of Road Pavements by Aust roads ï 2004.
8. Pells, P.J.N, Mostyn, G. & Walker B.F., ñFoundations on sandstone and shale in the Sydney regionò, Australian Geomechanics Journal, 1998

1.0 INTRODUCTION

This report presents and interprets the findings of the geotechnical investigation carried out on between the 24th and 28th of October at Hornsby Ku-ring-gai Hospital, NSW. The purpose of this investigation was to assess the existing site and subsurface conditions in order to provide geotechnical input on the proposed conceptual scheme design of the proposed development. The followings are addressed in this report:

- Method of investigation,
- Site description, including surface and sub-surface conditions,
- Site plan indicating borehole locations and footprint of the proposed development,
- Groundwater conditions and management, if encountered,
- Excavation conditions and recommendations including possible excavation equipment, trafficability with relation to natural materials, anticipated vibration levels, etc.
- Recommendations of permanent and temporary batter slopes for an excavation.
- Recommendations on retention/shoring systems and provision of substrata coefficients of lateral pressure,
- Recommendations relating to fill materials including compaction procedures and levels of appropriate compaction,
- Preliminary Acid Sulphate Soil Assessment.
- Recommendations on foundation systems, bearing capacities at different founding levels and anticipated settlements (where applicable),
- Provision of site subclass hazard factor in accordance with the Australian Standard AS1170.4-2007,
- Recommendations on pavement design and construction including subgrade preparation, compaction conditions, thickness and type of pavement layers and suitable drainage.

2.0 SITE DESCRIPTION

The Hornsby Ku-ring-gai Hospital is located about 25 kilometres north-west of the Sydney CBD. The hospital is within the local government area of Hornsby Shire and is bounded by Palmerston Road to the west, Burdett Street to the south, Derby Road to the east and Lowe Road to the north.

The proposed site was located in the south-east section of the hospital bounded by Burdett Street to the south and Derby Road to east. Existing one to three-storey buildings within the hospital area bounded the proposed development site to the north and west. At the time of the investigation existing one to three storey buildings utilised for hospital activities were located over the site. Supplied information indicated these structures were to be demolished at a later time as part of the development phase. The designated investigation area was irregular in shape and covered approximately 12000m^2. The hospital and site locations have been shown on Figure 1, ñSite Locality Mapò, and Figure 2, ñSite Plan.ò

3.0 REGIONAL GEOLOGY

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 Edition 1, 1983, indicated the site was likely to be underlain by Ashfield Shale (Rwa) overlaying Hawkesbury Sandstone (Rh) within Wianamatta Group. Ashfield Shale generally comprises black to dark-grey shale and laminite. Hawkesbury Sandstone generally comprises medium to coarse-grained quartz sandstone, with minor shale and laminate lenses.

4.0 AVAILABLE INFORMATION

At the time of preparation of this report, the following drawings were made available to Aargus:

- Borehole and CBR locations shown on a survey plan prepared by Craig & Rhodes Surveyors Engineers Planners referenced as 343/02 dated 12.08.2005,
- Hornsby Ku-ring-gai Hospital Redevelopment Stage 1 Brief provided to us by Thinc Health.

- Email correspondence from Mr Matthew von Bertouch of 18th October 2011 regarding changes to the borehole locations and scope of the proposed development.

Based on the available information, it was understood:

- The proposed development was currently at a conceptual stage and would involve the demolition of all existing structures within the development area with construction of buildings up to five storeys. No basements within the buildings were planned.
- Structural systems are anticipated to be reinforced concrete or prestressed concrete frame.
- Expected column loads could vary from 750kN to 4000kN.

5.0 FIELDWORK

Fieldwork for the geotechnical investigation was carried out by an engineering team from Aargus, and comprised the following works:

- A detailed walk-over inspection of the site to determine the overall surface conditions and to confirm geotechnical consistency with the surrounding landform.
- Drilling of six (6) boreholes using a tungsten carbide (TC) bit with solid flight augers to TC-bit refusal. The boreholes were subsequently advanced into the bedrock using NMLC diamond rock coring techniques.
- Standard Penetration Tests (SPT) were undertaken at regular intervals within the boreholes to assess the in-situ strength of subsurface soil layers.
- Dynamic Cone Penetrometer (DCP) tests were conducted adjacent to the boreholes to assess in-situ strength of the subsurface soil layers within shallower depths.
- Collection of test samples for Acid Sulphate Assessment at specific borehole locations.
- Collection of bulk samples for California Bearing Ratio (CBR) tests from

approximate subgrade levels at two locations as specified by the Client for evaluation of pavement design considerations.

- Collection of bulk samples for determination of plasticity characteristics by Atterberg limits tests.
- Reinstatement of the boreholes.

The approximate locations of the boreholes are presented in Figure 2.

6.0 INVESTIGATION RESULTS

6.1 Sub-surface conditions

Subsurface conditions encountered within the boreholes are detailed on the attached Engineering Borehole Logs, presented in Appendix C, and can be summarised as follows:

Fill/ Pavement

In overall the site is paved by asphalt (BH1, BH3& BH4). The pavement layers comprise 30-50mm asphalt/concrete underlain by 150-450mm road base materials (Gravel). As other locations (BH2, BH5 and BH6), topsoils comprise medium dense/dense sand, overlaying,

Natural strata

Mixtures of silty clay and clay, with low plasticity and of stiff/very stiff consistency which is grading to hard consistency within depths of 0.3m to 1.5m at BH1 to BH5. Occasionally some iron-strained bands were encountered within the layer from a depth of about 1.5m Below the existing Ground Level (BGL) downwards. This layer is underlain by

Sandstone bedrock, mostly of medium to high strength encountered in varying depths of less than 2.0m BGL (BH5 and BH6) at eastern portion of the subject site and over 5.0m (BH1 and BH4) within the western portion of the site.

The generalised subsurface strata have also been presented in the enclosed Geotechnical Cross Sections (Appendix B).

6.2 Groundwater condition

Groundwater or seepage was not encountered in the boreholes during drilling. However, it should be noted groundwater levels may be subject to seasonal fluctuations, rainfall, prevailing weather conditions and also future development of the surrounding lands.

7.0 LABORATORY TEST RESULTS

7.1 Atterberg Limit Testing

A representative sample of the natural clay profile was collected from the subgrade level at BH4 between the depths of 0.5m to 2.0m for laboratory testing, including Atterberg limits testing in accordance with AS 1289.3.1.1 ï 2009. Tests were conducted in Aargusôs NATA registered laboratory. The purpose of the testing was to determine the plasticity characteristics of the clay and estimate its reactivity with respect to potential total surface movements under seasonal moisture variations.

The results of the testing have been included in the laboratory test report in Appendix D and summarised in Table 1.

Table 1: Summary of Atterberg Limits Test

Location	Profile	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage
BH4	Brown orange clay	0.5-2.0	52	25	27	12

The laboratory test results indicated the natural clays to be of high plasticity.

7.2 Californian Bearing Ratio

Two representative samples of natural clay from approximate subgrade levels were collected for 4-day soaked CBR tests from BH4 and BH5 at depths varying between about 0.3m to 2.0m. The laboratory testing was undertaken in accordance with AS 1289.6.1.1 - 1998. The results are summarised in Table 2 with the sample locations shown in Figure 2.

Table 2: Summary of CBR tests

Sample ID	Location	Soil Description	Depth (m)	CBR
MT1	BH4	Brown orange clay	0.5-2.0	2.5%
MT2	BH5	Pink orange clay	0.3-1.8	1.5%

7.3 Point Load Strength Test Results

Recovered rock cores were returned to the Aargus Sydney laboratory for rock strength testing. This testing involved diametral and axial Point Load Strength Index tests. The Point Load Strength Indices for the rock cores and the assessed rock strengths, in accordance with Australian Standards (AS4133.4.1-2007), are summarised in the following table, Table 3.

TABLE 3 – Point Load Strength Test Results

Borehole	Depth (m)	Diametral $I_{s(50)}$ (MPa)	Axial $I_{s(50)}$ (MPa)	Assessed Strength
BH1	5.70	0.111	1.11	**Low to High**
	6.10	1.852	1.97	**High**
BH2	5.30	0.363	0.89	**Medium**
	6.30	0.808	1.84	**Medium to High**
BH3	3.70	0.769	0.92	**Medium**
	4.50	0.227	0.30	**Low to Medium**
BH4	6.40	0.866	0.88	**Medium**
	6.80	1.175	1.06	**High**
BH5	3.10	0.401	0.45	**Medium**
	3.80	0.788	0.89	**Medium**
BH6	3.20	0.285	0.34	**Low to Medium**
	3.90	0.343	0.35	**Medium**

8.0 ACID SULPHATE SOIL ASSESSMENT

8.1 Introduction

Soil samples were collected within boreholes BH2, BH3 and BH6 from different layers, representing the various subsurface strata, to perform a preliminary Acid Sulphate Soils

(ASS) assessment. The assessment is required as disturbance to acidic soils, during construction works, can result in the formation of acid. This acid, once formed, could damage infrastructures through deteriorating the reinforced concrete and structural elements and also could harm ecological systems.

The preliminary assessment may indicate if the soil layers are actually and/or potentially acidic and whether further investigation would be required.

8.2 Assessment Criteria

The decision to classify certain areas as ASS is based on a number of geomorphic conditions and site criteria. The following points are used to determine if ASS is likely to exist (ASSMAC (1998) Acid Sulphate Soils Assessment Guidelines):

- Sediments of recent geological age (Holocene) ~ 10 000 y.o.
- Soil horizons less than 5m AHD (Australian Height Datum).
- Marine or estuarine sediments and tidal lakes.
- In coastal wetlands or back swamp areas; waterlogged or scalded areas; interdune swales or coastal sand dunes.
- In areas where the dominant vegetation is mangroves, reeds, rushes and other swamp tolerant and marine vegetation.
- In areas identified in geological descriptions or in maps bearing sulphide minerals, coal deposits or former marine shales/sediments.
- Deeper older estuarine sediments >10m below the ground surface, Holocene or Pleistocene age.

The following soil indicators are used to determine if ASS are actually present on a site:

- field pH Ò4 in soils
- presence of shell
- any jarosite horizons or substantial iron oxide mottling in auger holes, in surface encrustations or in any material dredged or excavated and left exposed. Jarosite is not always found, however, in actual acid sulphate soils.

Beside the above indicators, the followings could also indicate whether PASS[1] is actually present on a site:

- waterlogged soils, unripe muds (soft, buttery, blue grey or dark greenish grey) or estuarine silty sands or sands (mid to dark grey) or bottom sediments of estuaries or tidal lakes (dark grey to black)
- presence of shell
- soil pH usually neutral but may be acid -positive Peroxide Test.

A positive peroxide test may include one but preferably more of the following:

- change in colour of the soil from grey tones to brown tones
- effervescence
- the release of sulfur smelling gases such as sulfur dioxide or hydrogen sulfide
- a lowering of the soil pH by at least one unit
- a final pH < 3.5 and preferably pH <3.

8.3 Laboratory Test Results

Field analyses were performed on the collected samples for pHf and pHfox in accordance with the required sampling techniques of the ASSMAC (1998) Assessment Guidelines (see Appendix F ï ASSMAC (1998) Field pH and peroxide test protocol. Standard sampling and analysing procedures were in accordance with the NSW ASSMAC (1998) ñAcid Sulphate Soils Assessment Guidelinesò. The results of pH tests are presented in Table 4.

To investigate the presence of Actual ASS (acid sulphate soils) of the soils, water was added to the soil samples. As shown in Table 1, pH_f for all the samples is well above 4, indicating the samples do not contain Actual ASS.

[1] Potential Acid Sulphate Soil

Table 4- Summary of ASS Test Analysis

Sample	Depth (m)	pH		pH		Change in pH ($pH_f - pH_{fox}$)
		H_2O	Soil pH_f	H_2O_2	Soil pH_{fox}	
BH2	0.60	7.52	6.19	4.64	4.08	2.11
BH2	1.00	7.52	5.60	4.64	3.91	1.69
BH2	2.00	7.52	5.65	4.64	4.17	1.48
BH3	0.50	7.52	5.55	4.64	3.46	2.09
BH3	1.00	7.52	6.05	4.64	5.40	0.65
BH3	1.50	7.52	5.80	4.64	4.21	1.59
BH6	0.30	7.52	6.90	4.64	4.86	2.04
BH6	1.00	7.52	5.44	4.64	4.89	0.55
BH6	2.20	7.52	5.53	4.64	3.88	1.65
BH6	3.00	7.52	5.47	4.64	4.01	1.46

Notes:

- pH_f refers to pH field (soil and distilled H_2O).
- pH_{fox} refers to pH field oxidised (soil and peroxide).
- Change in pH refers to pH field minus pH field oxidised.

To investigate the presence of PASS, 30% peroxide (H_2O_2) was also added to soil samples and the pH of the solutions was measured. The pH of the soil peroxide solutions (pH_{fox}) did not decrease below 3 pH units in any of the samples; however values for pH_{fox} of the sample collected at BH5 from a depth of 0.5m was close to 3.5. Therefore, further testing is recommended to be conducted on the sample to confirm whether PASS is present in the samples or not.

9.0 DISCUSSION & RECOMMENDATIONS

9.1 General

Based on the results of this investigation, it is considered the site is generally underlain by very stiff to hard silty clay and clay, with some iron-cemented bands overlaying sandstone bedrock.

Based on the estimated rock strengths (Table 3) and the rock discontinuities (shown in the borehole logs), the sandstone bedrock within the proposed development site may be classified as given in Table 5 in accordance with Pells et al (Reference 8).

TABLE 5 – Rock Classification

Borehole	Foundation Material	Depth (m)	Rock Classification (Reference 8)
BH1	Sandstone Bedrock	5.2-5.7	Class IV
BH1	Sandstone Bedrock	5.7-6.36	Class III
BH2	Sandstone Bedrock	4.5-5.2	Class III
BH2	Sandstone Bedrock	5.2-6.3	Class III
BH3	Sandstone Bedrock	3.4-4.15	Class IV/III
BH3	Sandstone Bedrock	4.15-5.23	Class III
BH4	Sandstone Bedrock	6.1-6.75	Class III
BH4	Sandstone Bedrock	6.75-8.23	Class IV/III
BH5	Sandstone Bedrock	1.8-3.1	Class V
BH5	Sandstone Bedrock	3.1-5.57	Class III
BH6	Sandstone Bedrock	1.5-3.1	Class V
BH6	Sandstone Bedrock	3.1-4.33	Class III

Although ground water was not encountered within the investigated depths at the time of the investigation, variations in groundwater/perched water levels may be possible due to influences mentioned above.

Based on the findings and observations made, the following comments and recommendations are provided with regards to the proposed development.

9.2 Site Classification

Based on visual field assessments and results of laboratory testing relating to plasticity determinations, the high plasticity clays may be estimated to be highly reactive with the possibility of total surface movements within the H1 or H2 ranges with respect to shrink/swell potential resulting from moisture condition variations within the soils.

It should be noted ranges associated with H1 or H2 only apply to single dwelling houses, townhouses or some light buildings similar to houses in size, loading and superstructure flexibility in accordance with AS2870-2011 ï Residential Slabs and Footings.

It should be noted that the classification is applicable to the site at the date of conducting the investigation, being 28th October 2011.

9.3 Excavation Conditions

Based on currently provided information, basements are not intended in the proposed development. However, should any basement excavation be considered or planned in the final stages of the development design, the following should be considered.

It is expected materials encountered during excavation are likely to comprise stiff to hard clays and silty clays and underlying weathered ranging to fresh sandstone.

Excavation of soil-based materials and extremely to highly weathered sandstone may be achieved using conventional earthmoving equipment such as backhoes or tracked excavators. Heavy ripping and/or vibratory rock breaking techniques are not likely to be required.

It is likely heavy ripping and/or vibratory rock breaking techniques may be required within the more competent, less weathered sandstones of medium to high strength. Should vibratory rock breaking equipment be required for excavations in bedrock, it is recommended it be complemented with saw cutting using an appropriate excavator mounted rock saw or approved alternative measure prior to excavation so as to minimise transmission of vibrations to adjoining structures. Hammering should be carried out horizontally along bedding planes where possible to minimise transmission of vibrations to adjoining structures.

Induced vibrations in structures adjacent to the excavation should not exceed a peak particle velocity (PPV) of 10mm/sec for structures in good condition or 2mm/sec for heritage or poor-conditioned structures. If vibrations in adjacent structures exceed these values or appear excessive, excavation should cease and Aargus should be contacted immediately for appropriate reviews.

Should the development of induced vibrations be considered possible during construction, it is recommended a structural assessment of adjoining structures be undertaken prior to project excavation proceeding.

The investigation indicated the presence of topsoils, vegetation and existing structures and pavements over the site. All topsoil materials, vegetation, including root systems, and deleterious materials, including old footings, services and bituminous materials should be stripped and removed from development areas to spoil.

Site earthworks should be properly drained to minimise the effects of wetting up and softening of exposed, natural subgrade soils, which may be caused by extraneous water sources and climatic variations. Trafficability across the site may be restricted to tracked plant during and following periods of wet weather and the trafficking of wet subgrades with any plant would be expected to result in significant subgrade damage. Should possible bulk excavation be terminated within the silty clay or clay layers, it is considered the natural materials at the base of such excavations may be trafficable under favourable climatic conditions and lack of groundwater presence. However, similar trafficability problems, as outlined for site subgrades, may be anticipated where ñwettingò may occur.

It is therefore suggested that consideration be given to the placement of a granular layers to provide convenient working platforms and improve site trafficability. Such a layer would also significantly assist in reducing potential drying out of reactive soil subgrades. Where such platforms are to be utilised for the support of heavy machinery or plant, such as piling rigs, it may be appropriate to design these platforms to such loads and if necessary have these confirmed and inspected by a geotechnical engineer.

9.4 Groundwater Management

Ground water was not observed within the investigated depths at the time of the investigation. However, it should be noted groundwater levels may vary subject to seasonal fluctuations, rainfall, prevailing weather conditions and also future development of the surrounding lands.

It is recommended possible groundwater presence or levels be confirmed if construction is undertaken during or following adverse weather or if a significant time period elapses between this investigation and construction.

Should groundwater or surface seepage be encountered during excavation, it is possible foundations excavations may be dewatered using appropriate drains and sump pits with a suitable pumping system.

A groundwater monitoring programme may be adopted prior to construction to confirm the groundwater regime and determine the design of appropriate drainage measures should groundwater presence be identified as problematic to construction or ongoing performance of structures.

It is recommended the final construction drawings be provided to Aargus for further assessment and confirmation of a suitable dewatering system, if required.

9.5 Temporary Batter Slopes

Temporary batter slopes may be appropriate for possible excavations or cut slopes provided basement excavations or cut slopes are set back sufficiently from common site boundaries to facilitate the formation of the recommended safe temporary batters outlined in Table 6.

Table 6- Minimum Temporary Batter Slopes

Materials	Temporary (Horizontal: Vertical)
Stiff CLAY	3.0:1.0
Very Stiff/ Hard Silty Clay	2.0:1.0
Distinctly Weathered Sandstone	1.0:1.0
Slightly weathered Sandstone or better	0.5:1.0

Temporary surface protection against erosion may be provided by covering the batter with plastic sheets or other applicable method. It is considered that plastic sheeting, if adopted, should extend at least 1.5m behind the crest of the cut face or at least up to the common site boundaries. Plastic sheeting should be positioned and fastened to prevent water infiltration into or onto the batter which may lead to softening and possible instability. All stormwater run-offs should be directed away from all temporary and permanent slopes.

9.6 Retaining Structures

In the long term, the excavation faces must be retained by engineered retaining structure. These structures should be designed to withstand the applied lateral pressures of the soil/rock layers, the existing surcharges in their zone of influence; including existing structures, and construction related activities, and also hydrostatic pressures (if it is appropriate). Depending on actual depth of bulk excavation on site, contiguous pile wall, reinforced concrete walls or Continuous Flight Augur (CFA) walls are among the feasible options for this purpose.

The pressure distribution on cantilever retaining structures, only due to the earth pressures and surcharges behind the wall, may be assumed to be triangular and estimated as follows(ignoring cohesion effect):

$$p_h = \gamma kH + qk$$

Where,

p_h = Horizontal pressure (kN/m^2)
γ = Wet density (kN/m^3)
k = Coefficient of earth pressure (k_a or k_o)
H = Retained height (m)
q = Surcharge pressure behind retaining wall (kN/m^2)

For the design of flexible retaining structures, where some lateral movement is acceptable, an active earth pressure coefficient is recommended. Should it be critical to limit the horizontal deformation of a retaining structure, use of an earth pressure coefficient at rest should be considered. Recommended parameters for the design of retaining structures are presented in the following Table 7.

Table 7: Geotechnical Design Parameters

Materials	Unit Weight (kN/m^3)	Active Earth Pressure coefficient (K_a)	At Rest Earth Pressure Coefficient (K_o)	Passive Earth Pressure coefficient (K_p)
Stiff/very stiff silty clay and clay	18	0.40	0.57	2.46
Hard silty clay	20	0.33	0.50	3
Extremely weathered sandstone (Class V or IV)	20	0.25	0.4	200kPa*
Distinctly weathered sandstone (Class IV or III)	22	0.15	0.25	400 kPa
Slightly weathered to fresh sandstone (Class III/II)	23	NA	NA	750kPa

* Passive lateral earth pressure.

The above coefficients assume that ground level behind the retaining structures is horizontal and the retained material is effectively drained.

9.7 Foundation System

Depending on proposed structures, associated structural loadings, tolerable settlements and cost-benefit considerations, foundation systems founded on very stiff to hard clays or silty clay or sandstone bedrock strata may be applicable. Possible foundation systems for various structures founded within the soil profile may consist of shallow pad and strip footings and piled rafts. For substantial structures likely to be founded within the bedrock profile, end bearing or socketed piles may be considered appropriate.

Shallow foundation systems or piles, with minimum length of 3.0m, founded within the very stiff clay may be designed adopting an allowable end bearing pressure of 250 kPa with this value being increased to 400 kPa for systems founded within the hard clay-based materials.

End bearing piles founded within low strength, Class IV sandstone may be designed with a maximum allowable end bearing pressure of 1000 kPa. End bearing or socketed piles founded within medium strength or higher, Class III sandstone may be designed with a maximum allowable end bearing pressure of 3500 kPa. A minimum socket length of 0.5m is considered appropriate.

In case piles are to be founded on clay layers, potential total and differential settlements should be evaluated under service loadings and be considered in the structural design. Long-term creep/consolidation settlements should also be taken into account.

Shaft adhesions for rock socket design may be adopted as presented in Table 8. Shaft adhesion should not be applied to the upper 0.5m length of socket located within the bedrock sequence. Adoption of appropriate values of shaft adhesion not only depends on the surrounding materials but also on the roughness of the footing excavation faces, cleanliness of the side walls and presence of any water.

TABLE 8 – Allowable shaft adhesion for deep footings

Material	Expected Depth Range Below Existing Ground Surface Levels	Serviceability Shaft Adhesion
Very low to low strength sandstone (Class IV)	5.20m ï 5.70m (BH1) 3.50m-4.15m (BH3)	100kPa
Medium strength sandstone (Class III)	5.70m-6.36m (BH1) 4.50m-6.30m (BH2) 4.15m-5.23m (BH3) 6.10m-8.23m (BH4) 3.10m-5.57m (BH5) 3.10m-4.33m (BH6)	350kPa

Ground slabs founded on stiff clays or medium dense sands may be designed using an allowable bearing capacity of 150 kPa.

Foundation systems associated to independent structures should be founded on similar foundation materials to minimise possible differential settlements.

Should groundwater flow or surface runoff be encountered within excavated footings, footing excavations should be dewatered and be clean and free of loose debris and wet soils prior to concrete placement or correct underwater placement techniques should be adopted.

An experienced geotechnical engineer or engineering geologist should inspect foundation excavations at the time of excavation and prior to reinforcement placement and construction to ensure suitable bearing materials satisfying design criteria have been achieved.

ñGeotechnical Strength Reduction Factorò of piled foundations can be determined in accordance with AS2159-2009 Cl.4.3.1. In absence of loading test of the piles, the factor can be determined based on risk ratings associated to Site, Design and Pile Installation; based on available information it could vary between ϕ_g=0.45-0.60 for low redundancy systems and ϕ_g=0.53-0.70 for high redundancy systems.

9.8 Pile Exposure Classification

Based on the results of laboratory testing and observations made, under AS2159-2009 the exposure classification for piles is anticipated to be **Non Aggressive**.

9.9 Subsoil Class for Earthquake Design

Under AS1170.4- 2007, site specific parameters are as follows:

- Importance level of hospital structures = 4,
- Earthquake design category of II or III, for Structures height, h<12m and hÓ12, respectively,
- Hazard Factor, Z=0.08 (ground acceleration coefficient),
- Site subclass is considered to be:
  - **Class B_e** for buildings founded on sandstone bedrock.
  - **Class C_e** for buildings founded on silty clay layers.

9.10 Managing Acid Sulfate Soils

Field pH tests indicated soil samples collected at depth from the site were not acidic and well above the ASSMAC (1998) guideline of pHÒ4. Also, pyrite and jarosite was not observed during the investigation and no sulphurous odours were recorded. During field investigations no unripe muds, mid to dark grey estuarine sands or shell were detected. The soils at the site down to the investigated levels, therefore, did not contain *Actual Acid Sulphate Soils*.

The pH of peroxide treated soil was found to be less than the field pH of the soil. The sample collected from BH3 at the depth of 0.5m showed pH_{fox} values of less than the ASSMAC (1998) guideline (pHÒ3.5) and pH reduction is more than 1 unit for the sample.

Therefore, chemical analysis of the samples is recommended to ascertain if sulfidic material is present and the oxidisable sulfur concentrations; and whether a ASS management plan is required for that specific area.

10.0 PAVEMENT DESIGN & RECOMMENDATIONS

10.1 Existing Pavement Profile and Subgrade Conditions

Boreholes located within areas encompassing existing pavements indicated profiles generally comprising AC seals approximately 30mm thick overlying base layers consisting of fill materials to an average depth of approximately 0.3m, further underlain by a subgrade profile of clays and silty clays.

Results of DCP tests indicated the base layer materials to be generally moderately to well compacted. The results also indicated the consistencies of the subgrade clay layers varied from stiff to very stiff.

10.3 Pavement Design CBR Values

Existing subgrade conditions over the site were assessed with two 4-day soaked CBR tests and results from six DCP tests carried in conjunction with the boreholes. Laboratory test results from samples obtained from BH4 and BH5 indicated soaked CBR values of 2.5% and 1.5% respectively, a median CBR value of 2%. Penetration rates within the clay subgrades from DCP tests indicated values ranging between approximately 12 to 25 mm/blow. CBR estimates from DCP blow counts may be approximated from Figure 5.3 of Austroads Pavement Design Guide and for the above penetration rate range estimated CBR values between approximately 8% and 15% may be applicable for stiff to very stiff clay-based subgrades.

10.5 Pavement Preparation

Existing asphalt surfacing should be stripped from pavement areas and disposed off-site.

Existing fill materials within proposed pavements containing any and all forms of deleterious materials together with any topsoils which may be encountered within future pavement areas should also be stripped and removed to spoil.

Subgrade materials within the top 500mm of final subgrade level should be compacted to a minimum of 100% of the Standard Maximum Dry Density in layers not exceeding 200mm compacted layer thickness and verified by in-situ field density compaction control testing in accordance with AS 1289, ñTesting Soils for Engineering Purposesò. Fill materials at depths greater than 500mm below final subgrade level should be compacted to a minimum of 95% of the Standard Maximum Dry Density, also in layers not exceeding 200mm compacted layer thickness and verified by appropriate testing as above. All fill materials placed within pavement areas and surrounds should be placed in a ñcontrolledò manner and certified by means of in-situ field density testing.

Exposed natural subgrade surfaces may be compacted with a minimum of 8 passes of an 8 to 10 tonne static weight smooth drum roller and should be proof rolled to detect potentially weak or softened spots or ground heave. Such defect areas should have affected materials removed, replaced with suitable, non-reactive, well graded, granular materials with maximum particle size not exceeding 75mm and compacted to a minimum of 100% of the Standard Maximum Dry Density.

Proof rolling should be supervised and approved by suitably qualified geotechnical personnel.

Upon certification of satisfactory compaction test results and approval of proof rolling, placement of base and sub-base course materials may be undertaken. Base and sub-base course materials, satisfying requirements of DGB20 and DGS40 respectively, should be compacted to a minimum of 98% of the Modified Maximum Dry Density and verified by in-situ field density compaction control testing.

Appropriate surface and subsurface drainage should be provided within the pavement and surrounds. Subsoil drains should be provided along all pavement edges. All surface and sub-surface waters should be collected and channelled by the use of approved collection methods and discharged into appropriate public utility drainage systems via approved discharge points. Drainage measures should be designed to prevent water from entering the pavement profile and materials.

11.0 CONCLUSION

This report presents and interprets the findings of the geotechnical investigation carried out between the 24th and 28th of October at Hornsby Ku-ring-gai Hospital, Hornsby, NSW with the view to assessing subsurface conditions within areas of the existing hospital environs and providing geotechnical recommendations for proposed development of the nominated area.

Based on supplied conceptual information and the findings of this investigation, it is considered proposed development of this site is feasible provided recommendations given in this report are taken into account.

For and on behalf of

Aargus Engineering Pty Ltd

Saman Zargarbashi

Dr.Saman Zargarbashi
BSc, MSc, PhD, MIEAust
Senior Geotechnical Engineer

Reviewed by

NMak

Noriman Mak, BSc, MSc, MBA, MIEAust,
RPE (Civ, Geo) NPER (Civ, Geo)
National Engineering Manager

(MF:SZ)

12.0 LIMITATIONS

The assessment of the sub-surface profile and geotechnical conditions within the proposed development area and the conclusions and recommendations presented in this report have been based on available information obtained from the drilling and associated site works carried out at provided locations over the period of 24th to 28th of October 2011.

Any site inspections and certifications should be performed by experienced Geotechnical Engineers, Engineering Geologists and field testing personnel.

Although the information provided by an ñAcid Sulphate Soils Assessment and Management Planò can reduce exposure to risks, no assessment, however diligently carried out, can eliminate them. It must be noted that these findings are professional findings and have limitations. Even a rigorous professional assessment may fail to detect all ASS and/or PASS on a site. Sulphates may be present in areas that were not surveyed or sampled

It is recommended that should ground conditions encountered during construction vary substantially from those anticipated within this report, Aargus be contacted immediately for further advice and any necessary review of recommendations or if surface and groundwater conditions encountered during excavation and construction vary from those presented in this report.

The conclusions and recommendations of this report should be read in conjunction with the entire report.

APPENDIX A

IMPORTANT INFORMATION ABOUT YOUR ENGINEERING REPORT

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IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, due in large measure to programs and publications of ASFE/ The Association of Engineering Firms Practicing in the Geosciences.

The following suggestions and observations are offered to help you reduce the geotechnical-related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include the general nature of the structure involved, its size and configuration, the location of the structure on the site and its orientation, physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program.

To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of the report may affect its recommendations.

Unless your consulting geotechnical engineer indicates otherwise, *your geotechnical engineering report should NOT be used:*

- when the nature of the proposed structure is changed: for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an un-refrigerated one,
- when the size or configuration of the proposed structure is altered,
- when the location or orientation of the proposed structure is modified,
- when there is a change of ownership, or

for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their report's development have changed.

Geotechnical reports present the results of investigations carried out for a specific project and usually for a specific phase of the project. The report may not be relevant for other phases of the project, or where project details change.

The advice herein relates only to this project and the scope of works provided by the Client.

Soil and Rock Descriptions are based on AS1726-1993, using visual and tactile assessment except at discrete locations where field and/or laboratory tests have been carried out. Refer to the attached terms and symbols sheets for definitions.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing are extrapolated by geotechnical engineers who then render an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those inferred to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than a report indicates. Actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact. For this reason, most experienced owners retain their geotechnical consultants through the construction stage*, to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time.* Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes or groundwater fluctuations may also affect subsurface conditions, and thus, the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

Subsurface conditions can change with time and can vary between test locations. Construction activities at or adjacent to the site and natural events such as flood, earthquake or groundwater fluctuations can also affect the subsurface conditions.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

Geotechnical engineers' reports are prepared to meet the specific needs of specific individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor, or even some other consulting civil engineer. Unless indicated otherwise, this report was prepared expressly for the client involved and expressly for purposes indicated by the client. Use by any other persons for any purpose, or by the client for a different purpose, may result in problems.

No individual other than the client should apply this report for its intended purpose without first conferring with the geotechnical engineer. No person should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professional develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid these problems, the geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy of their plans and specifications relative to geotechnical issues.

The interpretation of the discussion and recommendations contained in this report are based on extrapolation/interpretation from data obtained at discrete locations. Actual conditions in areas not sampled or investigated may differ from those predicted

BORING LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final boring logs are developed by geotechnical engineers based upon their interpretation of field logs (assembled by site personnel) and laboratory evaluation of field samples. Only final boring logs customarily are included in geotechnical engineering reports. These logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes and unanticipated costs are the all-too-frequent result.

To minimise the likelihood of boring log misinterpretation, give contractors ready access in the complete geotechnical engineering report prepared or authorized for their use. Those who do not provide such access may proceed under mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical consultants. To help prevent this problem, geotechnical engineers have developed model clauses for use in written transmittals. These are not exculpatory clauses designed to foist geotechnical engineers' liabilities onto someone else. Rather, they are definitive clauses which identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report, and you are encouraged to read them closely. Your geotechnical engineer will be pleased to give full and frank answers to your questions.

OTHER STEPS YOU CAN TAKE TO REDUCE RISK

Your consulting geotechnical engineer will be pleased to discuss other techniques which can be employed to mitigate risk. In addition, ASFE has developed a variety of materials which may be beneficial. Contact ASFE for a complimentary copy of its publications directory.

FURTHER GENERAL NOTES

Groundwater levels indicated on the logs are taken at the time of measurement and may not reflect the actual groundwater levels at those specific locations. It should be noted that groundwater levels can fluctuate due to seasonal and tidal activities.

This report is subject to copyright and shall not be reproduced either totally or in part without the express permission of the Company. Where information from this report is to be included in contract documents or engineering specifications for the project, the entire report should be included in order to minimise the likelihood of misinterpretation.

APPENDIX B

LOCALITY MAP (FIGURE 1) SITE PLAN (FIGURE 2) & GEOTECHNICAL CROSS SECTION

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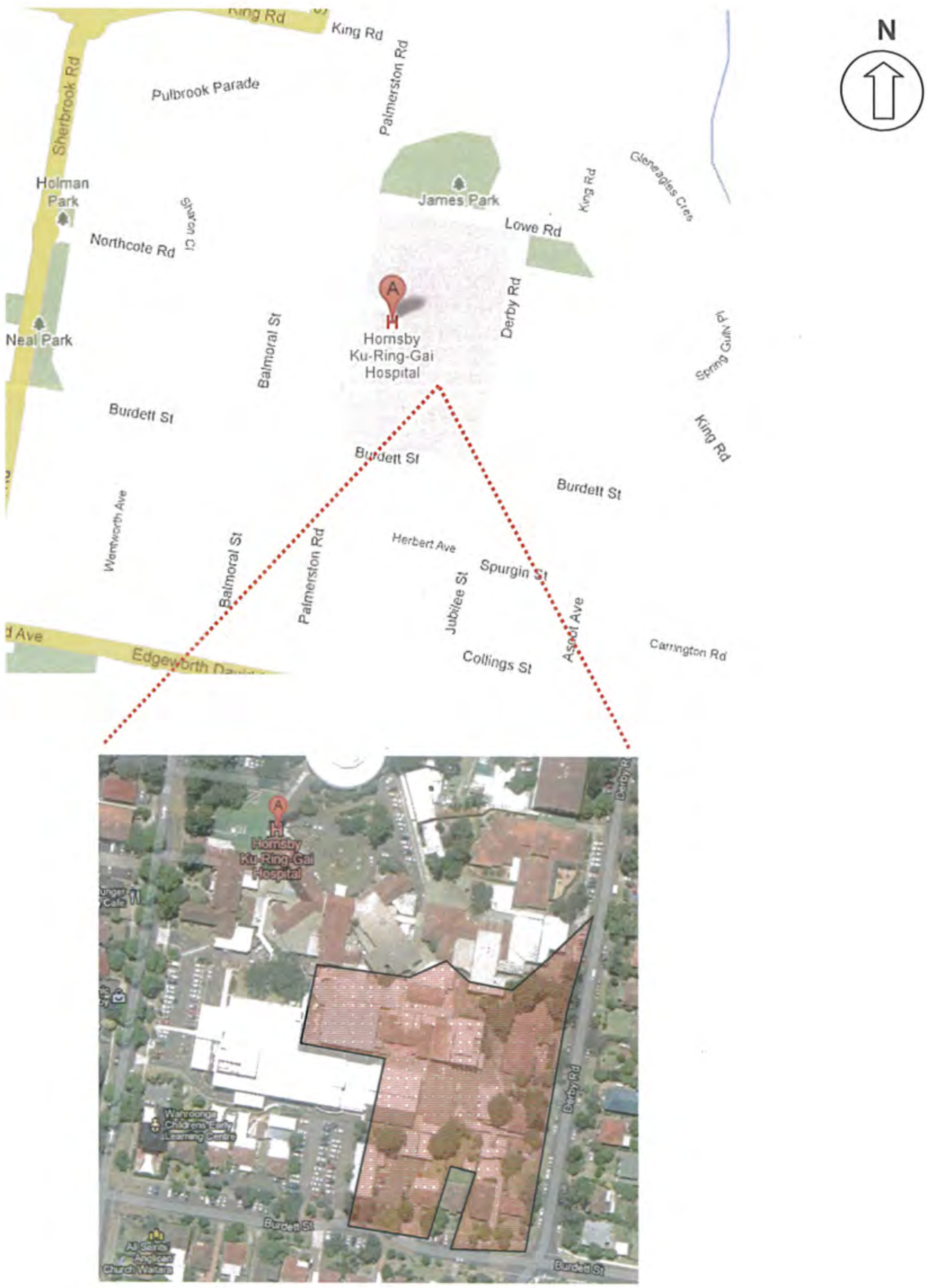
Locality Map

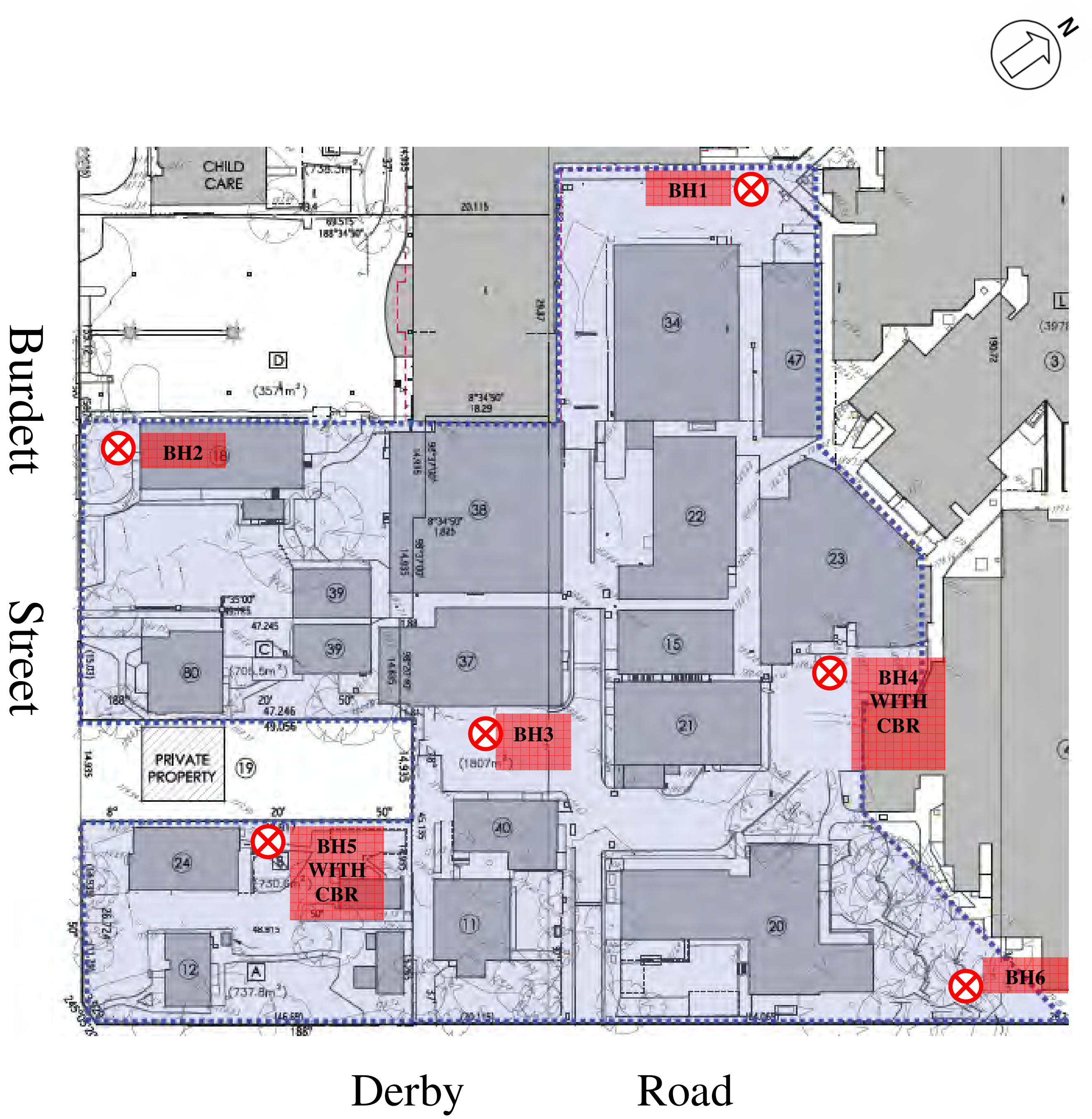


Reference Google Maps (2011)

Aargus Environmental- Remediation- Engineering- Drilling - Laboratories

Drawn	MT	*Health Infrastructure C/- Thinc Health* *Hornsby Ku-ring-gai Hospital Redevelopment* *Derby Road & Burdett Street,* *Hornsby NSW*	Aargus AUSTRALIA	Figure 1
Approved	SZ			
Date	11/11/2011			Job No: GS4661
Scale	NTS			

Site Plan



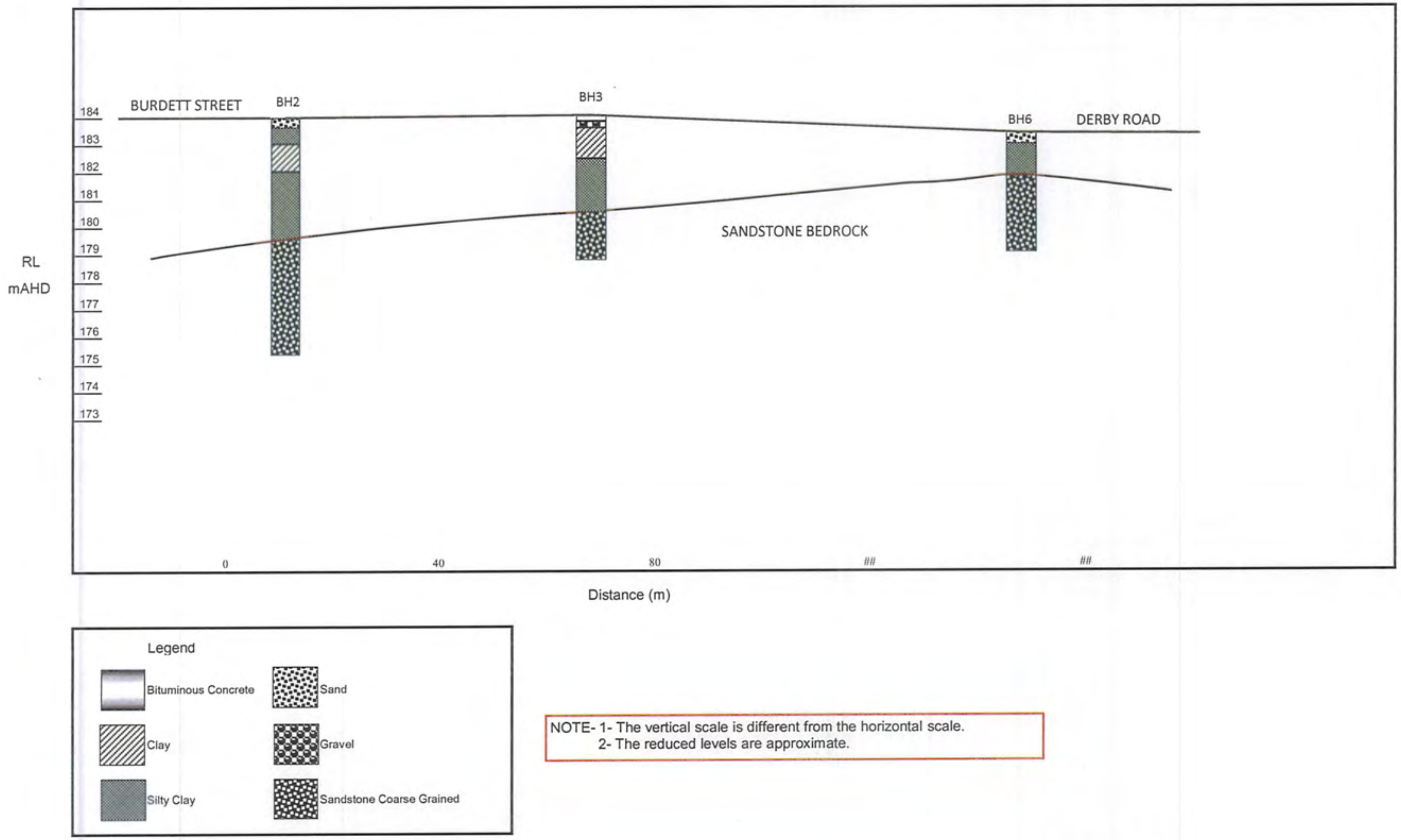
Aargus Environmental- Remediation- Engineering- Drilling - Laboratories

Drawn	MT	*Health Infrastructure C/- Thinc Health*	Figure 2
Approved	SZ	*Hornsby Ku-ring-gai Hospital Redevelopment*	
Date	11/11/2011	*Derby Road & Burdett Street, Hornsby*	
Scale	NTS		Job No: GS4661

Aargus AUSTRALIA

Geotechnical Cross Section

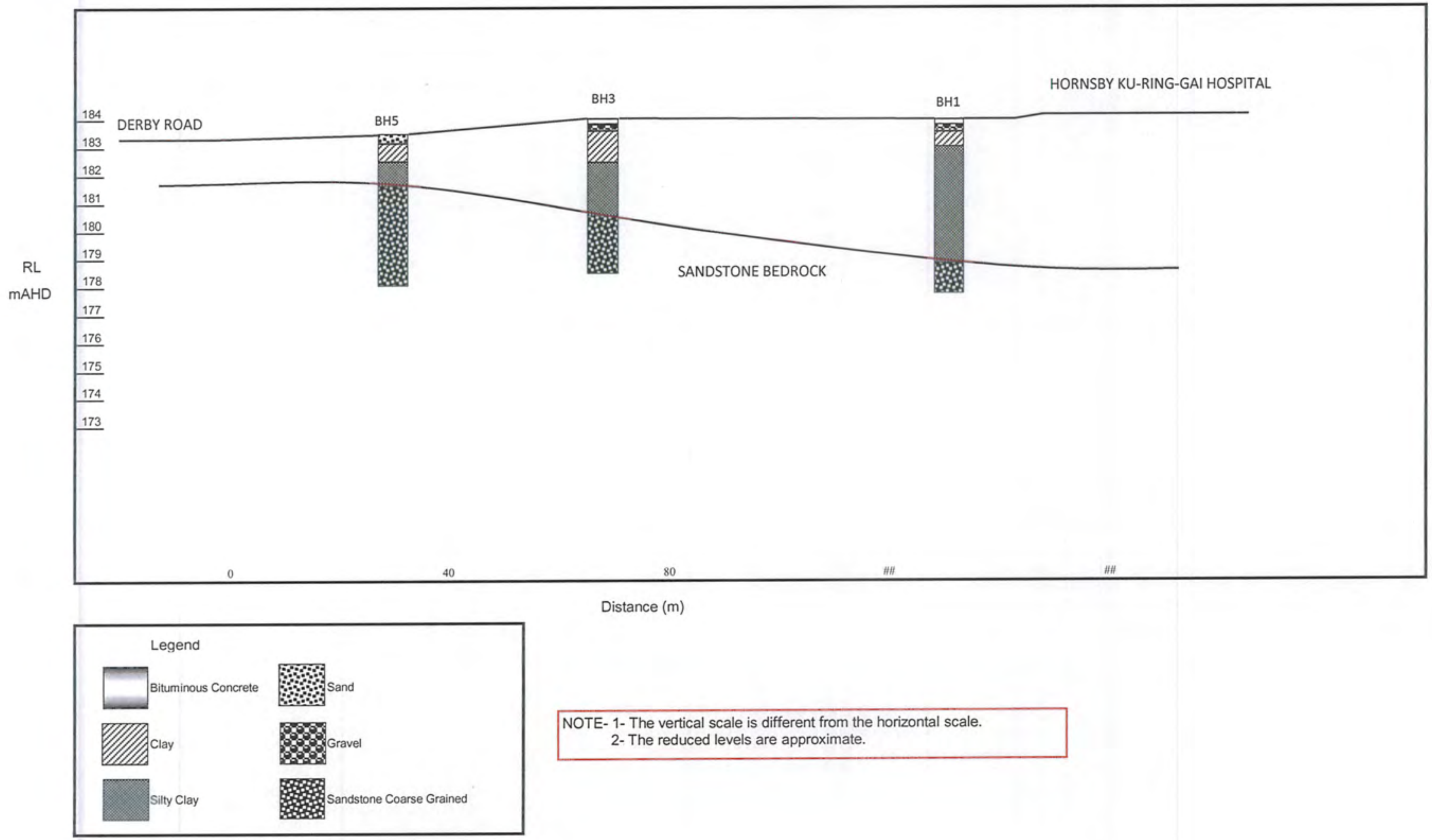
Date:	8/11/2011	Client:	Health Infrastructure C/- Thinc Health
Job No:	GS4661	Project:	Hornsby Ku-ring-gai Hospital Redevelopment



NOTE- 1- The vertical scale is different from the horizontal scale.
2- The reduced levels are approximate.

Geotechnical Cross Section

Date:	8/11/2011	Client:	Health Infrastructure C/-Thinc Health
Job No:	GS4661	Project:	Hornsby Ku-ring-gai Hospital Redevelopment



NOTE- 1- The vertical scale is different from the horizontal scale.
2- The reduced levels are approximate.

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GRAPHIC SYMBOLS FOR SOIL AND ROCK

Soil

- Bituminous Concrete
- Concrete
- Topsoil
- Fill
- Peat
- Clay
- Silty Clay
- Silt
- Sandy Clay
- Gravelly Clay
- Shaley Clay
- Clayey Silt
- Sandy Silt
- Basalt
- Sand
- Clayey Sand
- Silty Sand
- Gravel
- Sandy Gravel

Sedimentary Rock

- Sandstone Coarse Grained
- Sandstone Fine Grained
- Siltstone
- Laminite
- Shale
- Coal
- Limestone

APPENDIX C

ENGINEERING BOREHOLE LOGS & DYNAMIC CONE PENETROMETER TEST RESULTS

Aargus

Job No:	GS4661
Hole No:	BH1
Sheet	1 of 2

ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2	
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig	
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 27/10/2011	Logged by: MT
		Surface Level: 180.5	

Groundwater	Samples/ Field Tests	Depth (m)	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
		0.1		Asphalt pavement 30mm				0.1
	DCP		GW	GRAVEL, fine to medium grained, light brown with some aggregates	MD	D	Road Base Material	
		0.5	CL	CLAY, low plasticity, mottled orange to brown	M	Vst		0.5
		1.0	CL	Silty CLAY, low plasticity, light pink	M	Vst		1.0
		1.5	CL	Silty CLAY, low plasticity, light pink to brown with iron cemented bands	M	H		1.5
		2.0						2.0
		2.5						2.5
		3.0						3.0
		3.5						3.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample (50mm diam.)	**W**	Wet
St	Stiff	**D**	Dense			**Wp**	Plastic Limit
VSt	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**Wl**	Liquid Limit
H	Hard						

Job No:	GS4661
Hole No:	BH1
Sheet	2 of 2

ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2	
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig	
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 27/10/2011	Logged by: MT
		Surface Level: 180.5	

Groundwater	Samples/ Field Tests	Depth (m)	Graphic Log	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
		4.0		CL	Silty CLAY, low plasticity, light pink to brown with iron cemented bands	M	H		4.0
		4.5							4.5
		5.0							5.0
								TC Bit refusal	
					Refer to Corelog for additional information			Diamond rock coring started	
		5.5							5.5
		6.0							6.0
		6.5							6.5
		7.0							7.0
		7.5							7.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample (50mm diam.)	**W**	Wet
St	Stiff	**D**	Dense			**Wp**	Plastic Limit
VSt	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**Wl**	Liquid Limit
H	Hard						

Job No:	GS4661
Hole No:	BH2
Sheet	1 of 2

ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 26/10/2011 Logged by: MT
		Surface Level: 181.3

Groundwater	Samples/ Field Tests	Depth (m)	Graphic Log	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
	DCP	0.1		SW	SAND, fine to medium grained, dark brown with some grass roots	M	D	Topsoil	0.1
		0.5		CL	Silty CLAY, low plasticity, mottled orange, with a trace of iron cemented bands	M	H		0.5
		1.0		CL	CLAY, low plasticity, light pink	M	H		1.0
		1.5							1.5
		2.0		CL	Silty CLAY, low plasticity, light pink to brown with iron cemented bands	M	H		2.0
		2.5							2.5
	SPT 17,30, Re N>50	3.0		CL	Silty CLAY, low to medium plasticity, grey with distinctly to extremely weathered sandstone fragments	M	H		3.0
		3.5							3.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample (50mm diam.)	**W**	Wet
St	Stiff	**D**	Dense			**Wp**	Plastic Limit
VSt	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**Wl**	Liquid Limit
H	Hard						

Job No:	GS4661
Hole No:	BH2
Sheet	2 of 2

ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2	
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig	
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 27/10/2011	Logged by: MT
		Surface Level: 181.3	

Groundwater	Samples/ Field Tests	Depth (m)	Graphic Log	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
		4.0		CL	Silty CLAY, low plasticity, light grey, distinctly to extremely weathered sandstone fragments	M	H		4.0
		4.5						TC Bit refusal	4.5
					Refer to Corelog for additional information			Diamond rock coring started	
		5.0							5.0
		5.5							5.5
		6.0							6.0
		6.5							6.5
		7.0							7.0
		7.5							7.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample (50mm diam.)	**W**	Wet
St	Stiff	**D**	Dense			**Wp**	Plastic Limit
VSt	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**Wl**	Liquid Limit
H	Hard						

Job No:	GS4661
Hole No:	BH3
Sheet	1 of 1

ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2	
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig	
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 26/10/2011	Logged by: MT
		Surface Level: 180.0	

Groundwater	Samples/ Field Tests	Depth (m)	Graphic Log	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
		0.1			Asphalt pavement 30mm				0.1
	DCP			GW	GRAVEL, fine to medium grained, grey	M	MD	Road Base Material	
				CL	CLAY, low plasticity, mottled orange	M	H		
		0.5							0.5
		1.0							1.0
		1.5							1.5
				CL	Silty CLAY, low to medium plasticity, orange to grey with iron cemented bands	M	H		
		2.0							2.0
		2.5							2.5
	SPT 20, Re N>50	3.0							3.0
					Silty CLAY, low plasticity, light grey with iron cemented layers	M	H		
		3.5			Refer to Corelog for additional information			TC Bit refusal	3.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample (50mm diam.)	**W**	Wet
St	Stiff	**D**	Dense			**Wp**	Plastic Limit
VSt	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**Wl**	Liquid Limit
H	Hard						

Job No:	GS4661
Hole No:	BH4
Sheet	1 of 2

ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 28/10/2011 Logged by: MT
		Surface Level: 180.2

Groundwater	Samples/ Field Tests	Depth (m)	Graphic Log	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
		0.1			Asphalt/Concrete Pavement 50mm				0.1
	DCP			GW	GRAVEL, fine to medium grained dark brown	M	MD	Road Base Material	
				CL	CLAY, low plasticity, mottled orange to brown	M	Vst		
		0.5							0.5
		1.0							1.0
				CL	Silty CLAY, low plasticity, pink to white	M	H		
		1.5							1.5
		2.0							2.0
				CL	Silty CLAY, low plasticity, pink	M	H		
		2.5							2.5
	SPT 15,25 Ref	3.0							3.0
	N>50			CL	Silty CLAY, low plasticity, pink to red with some iron cemented layers	M	H	Getting hard to drill auger grinding into iron stained bands	
		3.5							3.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample (50mm diam.)	**W**	Wet
St	Stiff	**D**	Dense			**We**	Plastic Limit
Vest	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**We**	Liquid Limit
H	Hard						

Job No:	GS4661
Hole No:	BH4
Sheet	2 of 2

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ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 28/10/2011 — Logged by: MT
		Surface Level: 180.2

Groundwater	Samples/ Field Tests	Depth (m)	Graphic Log	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
		4.0		CL	Silty CLAY, low plasticity, pink to red with some iron cemented layers	M	H		4.0
		4.5							4.5
								Getting hard to drill	
		5.0							5.0
		5.5							5.5
		6.0							6.0
								TC Bit refusal	
					Refer to Corelog for additional information			Diamond rock coring started	
		6.5							6.5
		7.0							7.0
		7.5							7.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample (50mm diam.)	**W**	Wet
St	Stiff	**D**	Dense			**Wp**	Plastic Limit
VSt	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**Wl**	Liquid Limit
H	Hard						

Job No:	GS4661
Hole No:	BH5
Sheet	1 of 1

ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 26/10/2011 — Logged by: MT
		Surface Level: 179.7

Groundwater	Samples/ Field Tests	Depth (m)	Graphic Log	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
	DCP	0.1		SW	SAND, fine to medium grained, dark brown with some grass roots	M	MD	Topsoil	0.1
				CL	CLAY, low plasticity, mottled orange	M	Vst		
		0.5							0.5
		1.0							1.0
				CL	Silty CLAY, low plasticity, pink to grey, with iron cemented layers and some distinctly to extremely weathered sandstone fragments	M	H		
		1.5						TC Bit refusal	1.5
					Refer to Corelog for additional information			Diamond rock coring started	
		2.0							2.0
		2.5							2.5
		3.0							3.0
		3.5							3.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample (50mm diam.)	**W**	Wet
St	Stiff	**D**	Dense			**Wp**	Plastic Limit
VSt	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**Wl**	Liquid Limit
H	Hard						

Job No:	GS4661
Hole No:	BH6
Sheet	1 of 1

ENGINEERING LOG OF DRILLED BOREHOLE

Client:	Health Infrastructure C/-Thinc Health	Test Location: Refer to Fig. 2	
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	Test Method: Drill Rig	
Project Location:	Derby Road & Burdett Street, Hornsby	Date: 25/10/2011	Logged by: MT
		Surface Level: 179.1	

Groundwater	Samples/ Field Tests	Depth (m)	Graphic Log	Unified Classification	Description	Moisture Condition	Consistency/ Rel. Density	Additional Comments	Depth (m)
	DCP	0.1		SW	SAND, fine to medium grained, dark brown with some gravel and aggregates	M	MD	Topsoil	0.1
				CL	Silty CLAY, low plasticity, mottled orange	M	St		
		0.5							0.5
		1.0							1.0
				CL	Silty CLAY, low plasticity , pink to grey, with iron cemented layers and some distinctly to extremely weathered sandstone fragments	M	Vst		
		1.5						TC Bit refusal	1.5
					Refer to Corelog for additional information			Diamond rock coring started	
		2.0							2.0
		2.5							2.5
		3.0							3.0
		3.5							3.5

Explanatory Notes:

Consistency		Density Index		Samples		Moisture	
VS	Very Soft	**VL**	Very Loose	**B**	Bulk Sample	**D**	Dry
S	Soft	**L**	Loose	**D**	Disturbed Sample	**M**	Moist
F	Firm	**MD**	Medium Dense	**U50**	Undisturbed Sample	**W**	Wet
St	Stiff	**D**	Dense		(50mm diam.)	**Wp**	Plastic Limit
VSt	Very Stiff	**VD**	Very Dense	**N**	S.P.T. Value	**Wl**	Liquid Limit
H	Hard						

GRAPHIC SYMBOLS FOR SOIL AND ROCK

Surface Level: 180.5

Soil

Bituminous Concrete

Concrete

Topsoil

Fill

Peat

Clay

Silty Clay

Silt

Sandy Clay

Gravelly Clay

Shaley Clay

Clayey Silt

Sandy Silt

Sand

Clayey Sand

Silty Sand

Gravel

Sandy Gravel

Sedimentary Rock

Sandstone Coarse Grained

Sandstone Fine Grained

Siltstone

Laminite

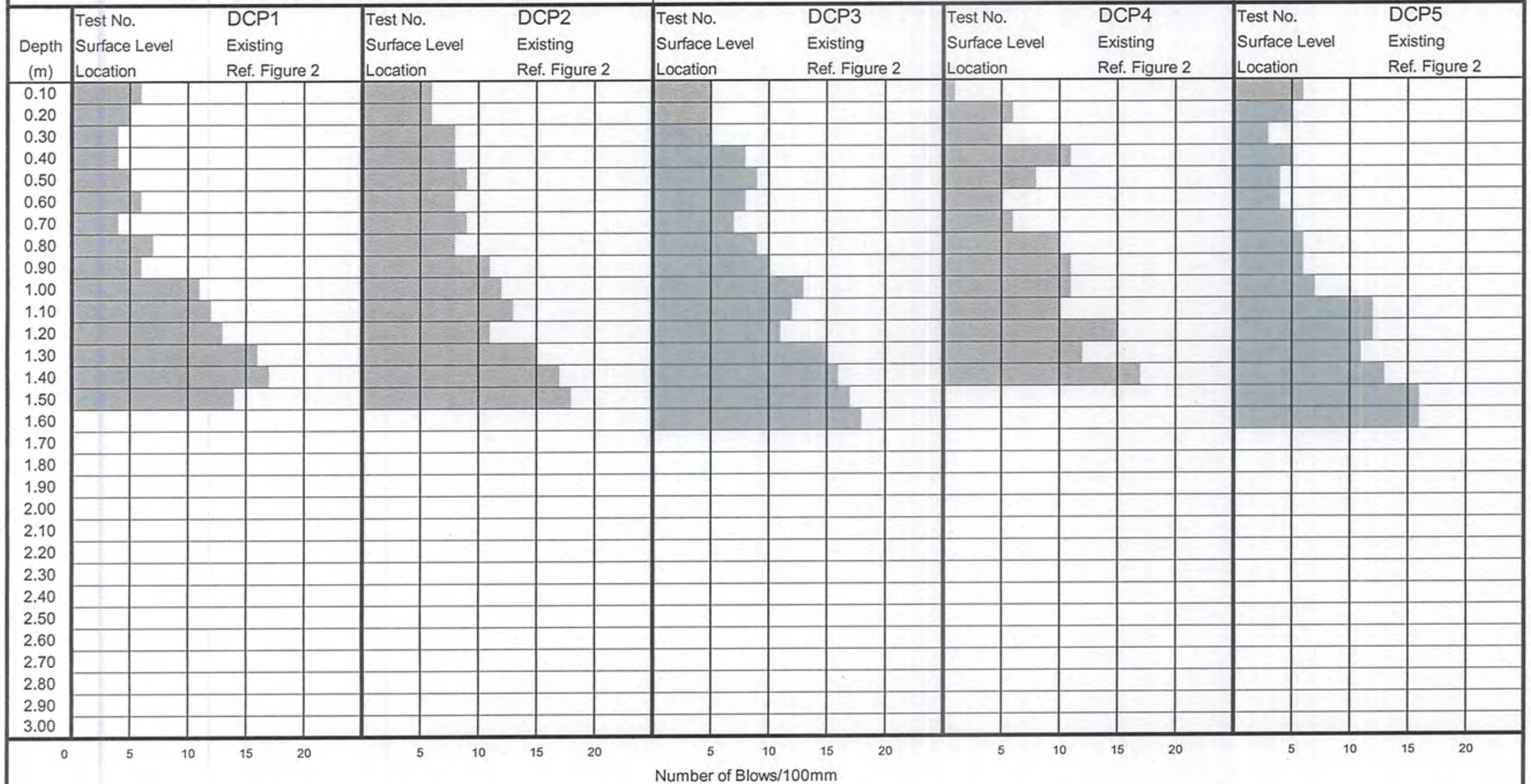
Shale

Coal

Limestone

PENETRATION RESISTANCE OF SOIL TEST REPORT - GRAPHIC

Client	Health Infrastructure C/- Thinc Health	AS1289 6.3.2	(Dynamic Cone Penetrometer)	x	Job No.	GS4661
Project	Hornsby Ku-ring-gai Hospital Redevelopment	AS1289 6.3.3	(Perth Sand Penetrometer)		Date	24/10/2011
Location	Derby Road & Burdett Street, Hornsby	RTA T161	(Dynamic Cone Penetrometer)		Page	1 of 2



Approved Signatory
Mrigesh Tamang
Date: 4/11/2011

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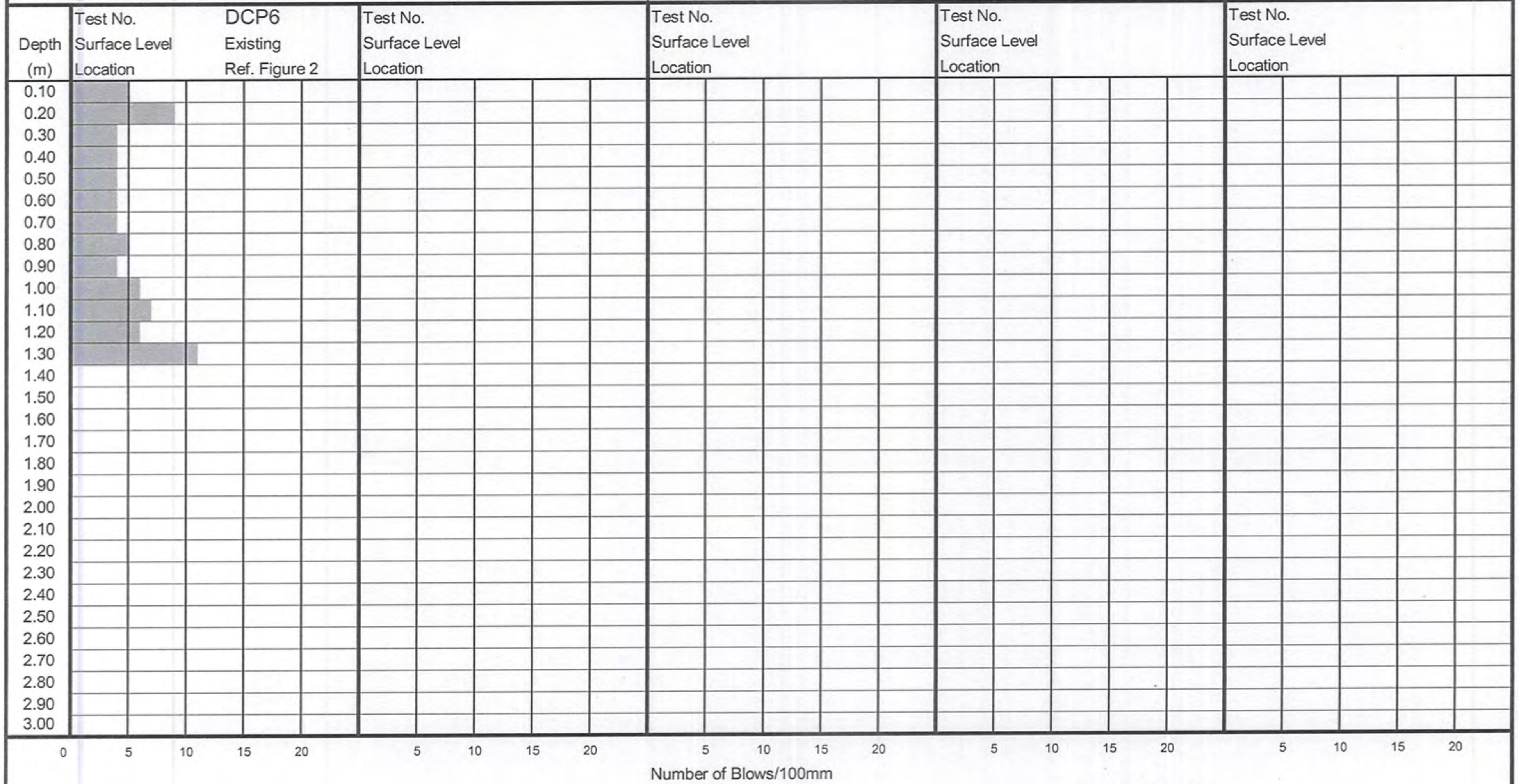
PENETRATION RESISTANCE OF SOIL TEST REPORT - GRAPHIC

Client	Health Infrastructure C/- Thinc Health	AS1289 6.3.2	(Dynamic Cone Penetrometer)	x	Job No.	GS4661
Project	Hornsby Ku-ring-gai Hospital Redevelopment	AS1289 6.3.3	(Perth Sand Penetrometer)		Date	24/10/2011
Location	Derby Road & Burdett Street, Hornsby	RTA T161	(Dynamic Cone Penetrometer)		Page	2 of 2



Approved Signatory
Mrigesh Tamang
Date: 4/11/2011

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AUSTRALIA

PENETRATION RESISTANCE OF SOIL TEST FIELD SHEET

Client:	Health Infrastructure C/- Thinc Health	DCP/DSP No.		Job No:	GS4661
Project:	Hornsby Ku-ring-gai Hospital Redevelopment	DCP	X	Date:	24/10/2011
Location:	Derby Road & Burdett Street, Hornsby	DSP		Sheet:	1 of 1

	DCP / DSP No.				DCP / DSP No.			
Depths (mm)	1	2	3	Depths (mm)	4	5	6	7
0-100	6	6	5	0-100	1	6	5	
100-200	5	6	5	100-200	6	5	9	
200-300	4	8	5	200-300	5	3	4	
300-400	4	8	8	300-400	11	5	4	
400-500	5	9	9	400-500	8	4	4	
500-600	6	8	8	500-600	5	4	4	
600-700	4	9	7	600-700	6	5	4	
700-800	7	8	9	700-800	10	6	5	
800-900	6	11	10	800-900	11	6	4	
900-1000	11	12	13	900-1000	11	7	6	
1000-1100	12	13	12	1000-1100	10	12	7	
1100-1200	13	11	11	1100-1200	15	12	6	
1200-1300	16	15	15	1200-1300	12	11	11	
1300-1400	17	17	16	1300-1400	17-Refusal	13	Refusal	
1400-1500	14-Refusal	18	17	1400-1500		16		
1500-1600		Refusal	18-Refusal	1500-1600		16		
1600-1700				1600-1700		Refusal		
1700-1800				1700-1800				
1800-1900				1800-1900				
1900-2000				1900-2000				
2000-2100				2000-2100				
2100-2200				2100-2200				
2200-2300				2200-2300				
2300-2400				2300-2400				
2400-2500				2400-2500				
2500-2600				2500-2600				
2600-2700				2600-2700				
2700-2800				2700-2800				
2800-2900				2800-2900				
2900-3000				2900-3000				

APPENDIX D

LABORATORY TEST RESULTS

Aargus Laboratories Pty Ltd ACN: 086 993 937
Environmental - Remediation - Engineering - Laboratories – Drilling
446 Parramatta Road, Petersham NSW 2049
Ph: 1300 137 038 Fax: 1300 136 038

REPORT OF THE SOAKED C.B.R. OF A SOIL

CLIENT: Aargus Engineering Pty Ltd
PROJECT: Hornsby Kuringai Hospital
TEST LOCATION: Burdett Street & Derby Road Hornsby

Job No. LS4661
Report No. LS4661/1
Sample No. BH4
Date Sampled: 28/10/2011

Sampling Location: Burdett Street & Derby Road Hornsby
Sampling Methods: AS1289.1.2.1.6.4b
Sample Description: Orange Brown CLAY

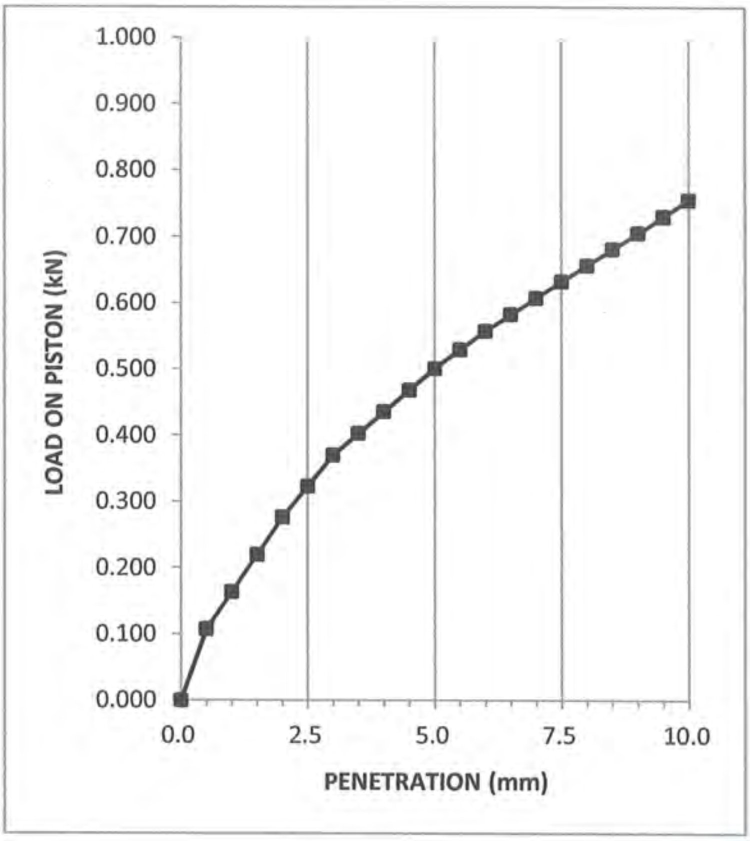
TEST METHOD: AS1289.6.1.1

Compaction Type: **Standard**
Maximum Dry Density (t/m3): **1.66**
Optimum Moisture Content (%): **19.2**

Remarks:

TEST CONDITIONS

CONDITION OF SPECIMEN	**SOAKED (4 DAYS)**
SURCHARGE (g)	**4500**
PERCENTAGE RETAINED 19mm (Not Included in Sample)	**0.0**
MOISTURE CONTENT - TOP 30mm (%)	**26.5**
MOISTURE CONTENT - Remainder (%)	**22.3**
SWELL/CONSOLIDATION (%)	**2.5**
LABORATORY MOISTURE RATIO %	**112**
LABORATORY DENSITY RATIO:	**100**
CBR PENETRATION IN mm	**5.0**
CBR VALUE%	**2.5**



NATA Accredited Laboratory Number: 12318

TESTED BY: KH DATE: 8/11/2011

APPROVED BY: MH ISSUED: 14/11/2011

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TEST METHODS: AS1289 1.2.1, 6.1.1, 5.1.1 (STANDARD) & 2.1.1

Authorised Signature: M Hoveling .....Mark Hoveling.

NATA ACCREDITED FOR TECHNICAL COMPETENCE

Aargus Laboratories Pty Ltd ACN: 086 993 937
Environmental - Remediation - Engineering - Laboratories – Drilling
446 Parramatta Road, Petersham NSW 2049
Ph: 1300 137 038 Fax: 1300 136 038

REPORT OF THE SOAKED C.B.R. OF A SOIL

CLIENT: Aargus Engineering Pty Ltd
PROJECT: Hornsby Kuringai Hospital
TEST LOCATION: Burdett Street & Derby Road Hornsby

Job No. LS4661
Report No. LS4661/2
Sample No. BH5
Date Sampled: 28.10.2011

Sampling Location: Burdett Street & Derby Road Hornsby
Sampling Methods: 1289 1.2.1.6.4b
Sample Description: Orange Brown CLAY

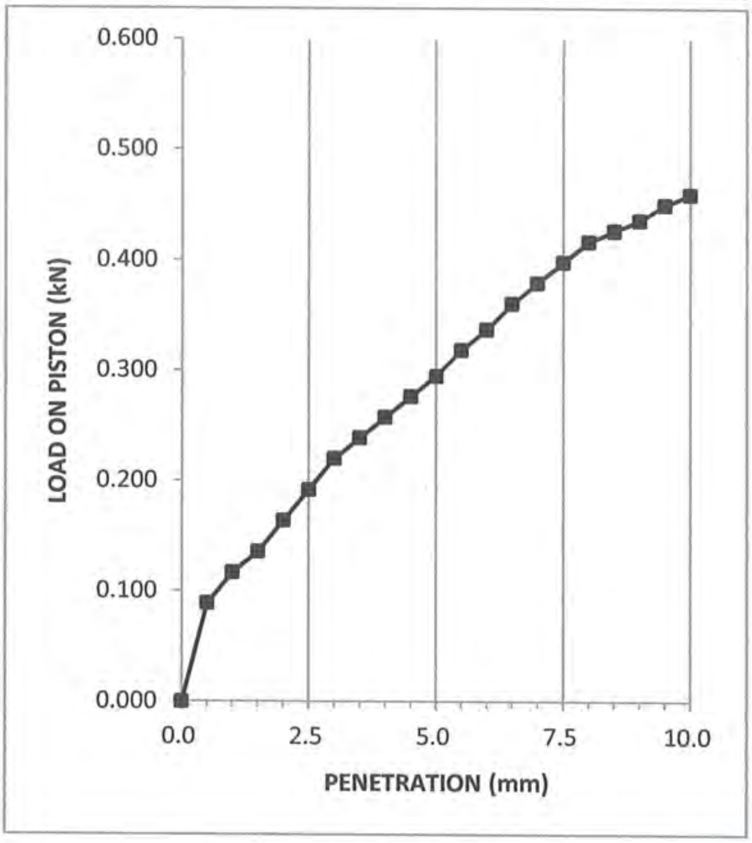
TEST METHOD: AS1289.6.1.1

Compaction Type: **Standard**
Maximum Dry Density (t/m3): **1.66**
Optimum Moisture Content (%): **20.5**

Remarks:

TEST CONDITIONS

CONDITION OF SPECIMEN	SOAKED (4 DAYS)
SURCHARGE (g)	4500
PERCENTAGE RETAINED 19mm (Not Included in Sample)	0.0
MOISTURE CONTENT - TOP 30mm (%)	23.5
MOISTURE CONTENT - Remainder (%)	22.7
SWELL/CONSOLIDATION (%)	4.9
LABORATORY MOISTURE RATIO %	95
LABORATORY DENSITY RATIO:	100
CBR PENETRATION IN mm	5.0
CBR VALUE%	**1.5**



TESTED BY: KH
DATE: 8/11/2011

APPROVED BY: MH
ISSUED: 15/11/2011

NATA Accredited Laboratory Number: 12318

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ACCREDITED FOR TECHNICAL COMPETENCE

TEST METHODS: AS1289 1.2.1, 6.1.1, 5.1.1 (STANDARD) & 2.1.1

Authorised Signature: M Hoveling .Mark Hoveling.

APPENDIX E

SITE PHOTOGRAPHS

Aargus

SITE PHOTOGRAPHS

Client	Health Infrastructure C/- Thinc Health
Project	Hornsby Ku-ring-gai Hospital Redevelopment
Location	Derby Road & Burdett Street, Hornsby
Job No.	GS4661
Checked By	PJ

Aargus AUSTRALIA

Photograph 1

View facing west within Hornsby Ku-ring-gai Hospital

Photograph 2

View facing east within Hornsby Ku-ring-gai Hospital

Photograph 3

View facing north-west within Hornsby Ku-ring-gai Hospital
Location of borehole 3

Photograph 4

Drilling equipment in operation at BH 5

Photograph 5

View facing Derby Road towards the north

Photograph 6

Drilling equipment in operation at BH 6

Photograph 7

Iron staining silty clay encountered at BH2

Photograph 8

View facing towards the north west from BH 2 location

APPENDIX F

ASSMAC (1998) FIELD PH AND PEROXIDE TEST PROTOCOLS

APPENDIX 1. Field pH and the Peroxide Test

1. Field pH Test

The field pH (pH_F) of actual acid sulfate soils tends to be ≤ 4 while the field pH of potential acid sulfate soils tends to be neutral. Field pH provides a useful quick indication of the likely presence and severity of "actual" acid sulfate soils. The field pH is a qualitative method only that cannot be used as a substitute for laboratory analysis in the identification of acid sulfate soils for assessment purposes.

Field pH readings should be taken at regular intervals down the soil profile. It is recommended this test be done every 0.25 m down the profile but at least every 0.5 m interval or horizon whichever is the lesser.

- pH readings of pH ≤4, indicates that actual acid sulfate soil are present with the sulfides having been oxidised in the past, resulting in acid soil (and soil pore water) conditions.
- pH values >4 and <5.5 are extremely acid and may be the result of some previous or limited oxidation of sulfides, but is not confirmatory of actual ASS. Substantial exchangeable/soluble aluminium and hydrogen ions usually exist at these pH values. Other factors such as excessive fertiliser use, organic acids or strong leaching can cause pH >4 - <5.5. Field pH alone cannot indicate potential ASS as they may be neutral to slightly alkaline when unoxidised.

In order to test for potential acid sulfate soils that contain unoxidised sulfides, peroxide is used to rapidly oxidise the iron sulfides (usually pyrite), resulting in the production of acid with a corresponding drop in pH.

Notes on pH equipment

Preferably a battery powered, field pH meter with a robust, spear point, double reference pH electrode should be used. The probe can be inserted directly into soft wet soils or soil mixed up into a paste with deionised water. Care must be exercised not to scratch the electrode on sandy or gravely soils. The probe should be standardised prior to use and regularly during use against standard solutions according to the manufacturers instructions.

Alternatively, an approximate 1:5 soil:deionised water suspension can be made up in small tubes, hand shaken and pH of the solution measured. pH test strips can be used to give an approximate value (pH +/- 0.25). Raupach soil pH test kits should be used with caution as they can give erroneous results. Both these latter methods are based on mixed indicator solutions that give a pH dependant colour and are subject to interferences.

2. Field Peroxide pH Test

To test for the presence of unoxidised sulfides and therefore potential acid sulfate soils, the oxidation of the soil with 30% (100 volume) hydrogen peroxide can be performed in the field. The most common method is:

- a small sample of soil is placed in a small glass container (eg short clear centrifuge tubes or clear tissue culture clusters) and a small volume of peroxide is dropped onto the soil.

Note: Allow the digested solution to cool after the reaction.
A pH probe will only measure to 60°C.

The reaction should be observed and rated. In some cases, the reaction may be instantaneous; in others, it may take 10 minutes or more. Heating over hot water or in the sun may be necessary to start the reaction on cool days, particularly if the peroxide is cold.

Potentially positive reactions includes one or more of the following:

- change in colour of the soil from grey tones to brown tones
- effervescence
- the release of sulfurous odours
- a substantial depression in pH below pH_F
- pH < 3

The strength of the reaction is a useful indicator. The peroxide test is most useful and reliable with clays and loams containing low levels of organic matter. It is least useful on coffee rock, sands or gravels, particularly dredged sands with low levels of sulfidic material (eg <0.05 % S). With soils containing high organic matter (such as surface soils, peats, mangrove/estuarine muds and marine clays), care must be exercised when interpreting the reaction as high levels of organic matter and other soil constituents particularly manganese oxides can also cause a reaction.

Note of caution with the use of peroxide

30 % hydrogen peroxide is a strong oxidising agent and should be handled carefully with appropriate eye and skin protection. This test should be only undertaken by trained operators.

The pH of analytical grade peroxide may be as low as 3 as manufacturers stabilise technical grade peroxide with acid, The peroxide pH should be checked on every new container and regularly before taking to the field and adjusted to 4.5 - 5.5 with a few drops of 0.1M NaOH if necessary. False field pH_{FOX} readings could result if this step is not undertaken.

3. pH after oxidation

The measurement of the change in the pH $_{FOX}$ following oxidation can give a useful indication of the presence of sulfidic material and can give an early indication of the distribution of sulfide down a core/ profile or across the site. The pH after oxidation test is not a substitute for analytical test results.

If the pH $_{FOX}$ value is at least one unit below field pH $_F$, it may indicate potential acid sulfate soils. The greater the difference between the two measurements, the more indicative the value is of a potential acid sulfate soils. The lower the final pH $_{FOX}$ value is, the better the indication of a positive result.

- If the pH $_{FOX}$ < 3 and there was a strong reaction to the peroxide, there is a high level of certainty of a potential acid sulfate soils. The more the pH $_{FOX}$ drops below 3, the more positive the presence of sulfides.
- A pH $_{FOX}$ 3-4 is less positive and laboratory analyses are needed to confirm if sulfides are present. Sands particularly may give confusing field test results and must be confirmed by laboratory analysis.
- For pH $_{FOX}$ 4-5 the test is neither positive nor negative. Sulfides may be present either in small quantities and be poorly reactive under quick test field conditions. In some cases, the sample may contain shell/carbonate that neutralises some or all acid produced by oxidation. In other cases, the pH $_{FOX}$ value may be due to the production of organic acids and there may be no sulfides present. In these cases, analysis for sulfur using the POCAS method would be the best to check for the presence of oxidisable sulfides.
- For pH $_{FOX}$ >5 and little or no drop in pH from the field value, little net acid generating ability is indicated. Again, the sulfur trail of the POCAS method should be used to check some samples to confirm the absence of oxidisable sulfides.

Care is needed with interpretation of the result on highly reactive soils. Some soil minerals other than pyrite react vigorously with peroxide, particularly manganese but may only show small pH changes. When selecting soil for testing it is advisable to avoid material high in organic matter as the oxidation of organic matter can lead to the generation of acid. However, pH of soils containing organic matter and no pyrite do not generally stay below 4 on extended oxidation. In general, positive tests on 'apparently well drained' surface soils should always be treated with caution and followed up with laboratory confirmation.

The field peroxide tests can be made more consistent if a fixed volume of soil (using a small scoop) is used, a consistent volume of peroxide is added and left to react for an hour, and the sample is made up to a fixed volume with deionised water before reading. However, such procedures take time in the field and are more suited to a 'field shed' situation. When effervescence (sometimes violent) has ceased, a few additional mL of peroxide should be added until the reaction appears complete. If the reaction is violent, it is recommended that deionised water be added to cool and dilute the reaction. The test may have to be repeated with a small amount of water added to the soil prior to peroxide addition. The pH ^{FOX} of the resultant mixture is then measured.

4. Reporting the results

All pH $_F$ and pH$_{FOX}$ results along with the strength of reaction should be tabulated by site and depth and reported in the ASS report. An example of a recording sheet is attached.