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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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Report on Geotechnical Investigation
Concord Repatriation General Hospital Redevelopment
Concord

1. Introduction

This report presents the results of a geotechnical investigation undertaken for the proposed Concord Repatriation General Hospital (hereafter Concord Hospital) redevelopment at Concord. The investigation was commissioned in an email dated 11 February 2016 by Ms Deborah Flood of Sydney Local Health District c/- Capital Insight and was undertaken in accordance with Douglas Partners’ proposal SYD151675 dated 17 December 2015.

The work was undertaken in conjunction with a preliminary contamination investigation, which is reported separately in DP Report 85356.01.R.001.Rev0. That investigation did not identify significant contamination that would impact on the geotechnical recommendations within this report. The presence of acid sulphate soils (ASS) were also assessed as part of the work, and the results are contained within this report.

It is understood that a multi-phased redevelopment of the site is proposed, including the construction of a new multi-storey car park (Phase 1 and Phase 2) and new tower buildings with basement levels (Phase 1a and 1b). Within this report, these structures are referred to as the proposed “multi-storey car park” and “tower buildings”, respectively. Further redevelopment for a clinical services block is also proposed for Phase 2 works, but is excluded from the current scope.

The aim of the investigation was to assess the subsurface soil and groundwater conditions across the site in order to provide:

- an assessment of the geotechnical suitability of the site for the proposed development;
- recommendations on excavations and retaining structures;
- an appropriate foundation system for the proposed development, including an assessment of allowable bearing pressures and likely settlements;
- assessment of acid sulphate soils; and,
- suitable parameters for the design of new pavements

The investigation included the drilling of seventeen (17) boreholes and laboratory testing of selected samples. The details of the field work are presented in this report, together with comments and recommendations on the issues listed above.

Site survey information in the vicinity of the proposed tower blocks was provided by the Client to assist with reporting (Drawing B1846G-1 dated 27 March 2015, by Project Surveyors). Ground levels are discussed relative to the Australian Height Datum (AHD).
2. Site Description

The Concord Hospital site is located on a peninsula on Parramatta River, between Bray’s Bay and Yaralla Bay. Ground levels rise up from the river level to approximately RL 8 m to RL 12 m along the central, south-west to north-east oriented ridgeline of the peninsula.

The approximate locations of the subject sites with respect to the overall hospital site are shown in Drawing 1, and with respect to the local site features in Drawings 2 and 3, in Appendix B. Selected site photographs are included in Appendix C.

The site of the proposed multi-storey car park is in the existing car park area, north of Hospital Road. The proposed footprint of the building is shown in Drawing 1 and 2 in Appendix C. The following site features were noted at the time of the field work:

- Ground levels in this area typically slope down from Hospital Road to Bray’s Bay to the north. At the proposed car park footprint this corresponds to levels of approximately RL 8 to RL 9 m at Hospital Road down to approximately RL 4 m, and slopes of approximately 2° to 3°.

- The proposed footprint of the multi-storey car park is largely over existing car park areas, but over some grass verge areas at the north-western edge. These verge areas adjoin the hospital site boundary, with scrub and bushland areas and mangroves located beyond the boundary. Ground levels continue to fall towards the north, to mangroves around Bray’s Bay. A photograph of the car park, facing south from the northern corner of the site, is included as Photo 1 in Appendix C.

- The existing asphaltic-concrete paved car park appears to be in good conditions. Some very local cracking and movement of the car park kerb along the north-eastern boundary was observed towards the northern corner of the proposed car park, possibly associated with a local channel depression on the grass verge (see Photo 2 in Appendix C). The cracking did not appear to extend onto the asphaltic concrete pavement.

The site of the proposed towers building is located towards the western end of the hospital site, south of the existing Building 5 (also known as the “multi building”). The proposed footprint of the site is shown in Drawing 1 and 3 in Appendix C. The following site features were noted at the time of the field work, and with reference to the provided survey information:

- Ground levels in this area generally slope down from the north-east to the south-east, towards Yaralla Bay. Within the proposed footprint, levels generally slope consistently down from approximately RL 8.9 to RL 8.3 m at Building 5 down to approximately RL 4.0 to RL 3.5 m at the southern-eastern side of the proposed building, at approximately 2° to 4°.

- Higher ground levels are present locally at the western corner of the footprint, where the existing tennis courts are at approximately RL 9.5 m. Ground levels adjacent to the tennis courts rise steeply up grassed batters to the north-east, and fall down flagstone batters to the south and east. Based on these batters, it is considered likely that the tennis court bench was constructed by cut and fill.

- The site is largely occupied by existing one to two storey brick, weatherboard and fibreboard buildings, typically with brick pier foundations, and asphaltic concrete or concrete paved roads. Also present are small, landscaped garden areas, the artificially turfed tennis courts and one two-to-three storey brick building at the northern corner of the site. Typical buildings on the site are shown in Photo 3 in Appendix C.
• The proposed development footprint will extend up “Hospital Street” at its north-western side, as shown in Drawing 3, in Appendix C. In this area the works will directly adjoin the existing, 8 storey brick, concrete and glass Building 5 and the brick, 4 to 5 storey Building 3. It is understood that these buildings have existing basement floor levels for pharmacy and kitchen areas. Also present in this area are Building 4, a substation and the liquid oxygen storage tanks which are retained by an existing concrete retaining wall, adjoining Hospital Street. This part of the site is shown in Photo 4 in Appendix C. In other areas, the proposed building is adjacent to existing roads, or existing buildings that are expected to be demolished as part of the development.

• Extensive underground services are present in this area, particularly in the vicinity of Buildings 4 and 5, with manholes, hatches, underground services and confined spaces signage observed at the time of field work.

2.1 Regional Mapping

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the hospital site is generally underlain by Ashfield Shale, typically comprising black to dark grey shale and laminitre. Hawkesbury Sandstone is present towards the eastern side of the peninsula.

The Ashfield Shale in this area is usually underlain by the relatively thin Mittagong Formation (typically consisting of shale laminitre and medium grained sandstone), which is in turn underlain by Hawkesbury Sandstone. Hawkesbury Sandstone is typically a medium to coarse grained quartz sandstone with very minor shale and laminitre lenses.

The proximity of the mapped Hawkesbury Sandstone suggests that the Ashfield Shale is of limited depth in this area.

Quaternary Age stream alluvial and estuarine sediment is mapped towards Yaralla Bay.

The proposed multi-storey car park is located in an area mapped as Ashfield Shale, while the proposed towers building is in an area mapped as Quaternary Age sediment.

Past DP experience at the hospital site has generally been consistent with the presence of Ashfield Shale, and suggests that the Quaternary Age sediment is less extensive around Yaralla Bay than indicated by the mapping. The results of the current field work are generally consistent with the presence of Ashfield Shale underlain by the Mittagong Formation and Hawkesbury Sandstone. As with previous field work by DP at the hospital, the current results are also not consistent with the extensive Quaternary Age sediment mapping.

Reference to the Sydney 1:100 000 Soils Landscape Series Sheet indicates that the sites are generally underlain by residual soil of the Blacktown soil landscape. At the southern corner of the proposed tower buildings, however, a “disturbed” mapping is recorded. Given its location at the edge of a bay, it is considered likely that the mapping refers to an area of suspected land reclamation.

Reference to the 1:25 000 NSW Acid Sulfate Soil Risk Mapping indicates that the soils within Parramatta River are at high risk of acid sulphate soils. The high risk zoning extends onto land at the northern end of the proposed multi-storey car park. At the towers building, the area of disturbed
terrain (here mapped along the south-eastern side of the site) is at “unknown” risk, with investigation required to assess the likelihood of acid sulphate soils.

3. Previous Investigation

Douglas Partners Pty Ltd (DP) have previously undertaken investigations at the site including recent investigation for the Palliative Care Unit, located north-east of the proposed tower buildings, and historic investigation for the chapel building, north of the proposed tower buildings. These investigations encountered filling and clay to depths of between 0.2 and 1.6 m, underlain by shale.

4. Field Work Methods

The field work for the geotechnical investigation included 14 boreholes; with 6 test locations at the proposed multi-storey car park (BH 1 to BH 6), and 8 locations at the proposed tower building (BH 10 to BH 17). The test locations were necessarily restricted by the presence of existing buildings and below ground services, and by the need to limit disruption to site operations. The field work was undertaken in February and March 2016.

The boreholes were drilled with a truck-mounted geotechnical drilling rig using 110 mm diameter solid flight auger and rotary drilling to depths of between 1.0 m and 3.3 m. Regular standard penetrometer tests (SPTs) were undertaken within the soils in order to provide information on the engineering properties of the soils and to obtain soil samples for visual and tactile assessment and for laboratory testing. The boreholes were then continued into the underlying bedrock using diamond core drilling techniques to final depths of between 6.0 m and 12.1 m, except at BH 13 where the borehole was discontinued at 5.2 m after encountering a void from 3.9 m. Following the completion of drilling, the strength of the rock was assessed by examination of the recovered rock cores and subsequent correlations with laboratory Point Load Strength Index ($I_{50}$) tests. The ground surface at test locations was reinstated with cold-mix.

Following the completion of drilling, a 12 m standpipe was installed in one borehole (BH 17) and finished with a gatic cover, to allow future measurement of groundwater levels.

Further details on the methods and procedures employed in the investigation are presented in the notes in Appendix A of this report.

The plan location and ground surface level relative to Australian Height Datum (AHD) at each test location were determined by differential GPS accurate to about 0.1 m in plan and elevation. The test locations are shown in Drawing 2 and Drawing 3, in Appendix B of this report for the proposed multi-storey car park, and proposed tower building sites, respectively.
5. Field Work Results

The detailed results of the field work are included in Appendix D of this report, together with relevant notes on classification.

At the proposed multi-storey car park (BH 1 to BH 6) the test results generally indicate the following profile:

- **Asphaltic Concrete** – pavement surface, to depths of 0.03 m to 0.05 m; underlain by,
- **Filling** – typically including basaltic gravel to depths of up to 0.1 m to 0.15 m, absent in some locations, underlain by variable clay and sand filling to depths of up to 0.8 m; underlain by,
- **Clay and Silty Clay** – typically firm to very stiff, orange-brown clay with some ironstone gravel, becoming shaly clay from approximately 2.0 m depth, to 2.5 m; underlain by,
- **Shale** – extremely low and very low strength, with medium and high strength iron-cemented bands, (though these bands were absent at BH 2 and BH 3); at bores in higher ground level only (BH 4 to BH 6), the shale increased to very low to low strength from depths of 4.3 m to 4.5 m (RL 1.9 to RL 2.8 m) with some medium strength shale below depths of 4.5 m at BH 6 and 6.1 m at BH 5; underlain by,
- **Sandstone and Laminite** – at bores in lower ground only (BH 1 to BH 3) from depths of 4.0 m to 5.1 m (RL 0 to RL 0.5 m), typically medium and high strength rock, with some very low and low to medium strength bands

At the proposed tower buildings (BH 10 to BH 17) the test results generally indicate the following profile:

- **Filling** – including asphaltic concrete and concrete pavement surfaces at some locations, underlain by variable filling including gravelly sand, clayey silt and silty clay to typical depths of between 0.2 m and 0.5 m, but up to 1.4 m depth at BH 11; underlain by,
- **Clay** – stiff and very stiff clay, with some shaly clay to depths of up to 2.6 m, but absent in some locations (BH 10, BH 15, BH 17); underlain by,
- **Shale and Laminite** – extremely low and very low strength, fragmented to fractured, light grey and light grey brown shale and laminite, with some low to high strength iron-cemented bands, to depths of 0.3 m to 5.85 m, then generally low and medium strength to depths of 2.83 to 7.5 m; then generally medium strength and fractured to slightly fractured to depths of 5.6 m to 8.2 m; underlain by high strength shale and laminite at some locations; underlain by,
- **Sandstone** – medium then coarse grained, slightly fractured and unbroken sandstone from depths of 7 m to 9.2 m (though not encountered at BH 10, BH 11 and BH 13, and from shallower depth, 4.1 m, at BH 17), generally high strength with some very high strength bands, except at BH 17, where an upper layer of low to medium strength sandstone was present, before improving to high strength sandstone with a very high strength band, from 5.6 m depth

It is noted that BH 13 was discontinued at 5.2 m after intersecting a void within the shale from 3.9 m depth. Following further investigation of the void by Concord Hospital, it is understood that the void is not a service and is of unknown origin. Based on the ground conditions, the void is nonetheless considered likely to be man-made.
No free groundwater was observed at any borehole whilst augering. The use of drilling fluid precluded observation of groundwater during rotary and coring operations.

At BH 17, the standpipe was bailed and a groundwater depth of 4.2 m (0.0 m AHD) measured on 14 March 2016.

6. Laboratory Testing

Laboratory testing was undertaken on selected soil samples for the purpose of testing for the presence of acid sulphate soils, for soil aggressivity and for information for pavement design.

6.1 Acid Sulphate Soil

Soil samples were obtained at various depths in the boreholes on site and selected samples were screened for the presence of acid sulphate soils (ASS). Screening tests involved pH testing before and after the addition of hydrogen peroxide (pH_F and pH_FOX, respectively). The following methodology was adopted for the pH screening testing:

- **pH measurement:** Place approximately 5 g of soil material in a small glass container, add 25 mL demineralised water, mix, and measure pH;
- **Peroxide pH measurement:** Place approximately 5 g of soil material in a small glass container, add 2 mL of hydrogen peroxide which has a nominal pH of 4.5 to 5.5, observe sample for effervescence, colour change or odour, allow the sample to react for a minimum of 30 minutes, add 18 mL distilled water, mix, and measure pH.

Screening results are for indicative purposes only and no firm criteria are applicable. General comparative values for pH screening are provided by the ASS Management Advisory Committee (ASSMAC), however it is noted that these may provide a false indication due to the potential presence of inclusions in the soil (e.g. organic matter, shells), that may affect the pH values.

The results for pH screening are presented in Table 1. Results that meet the indicative screening values for acid sulphate soils (as given in the notes following the table) are shown highlighted.

<table>
<thead>
<tr>
<th>Area</th>
<th>Sample Location</th>
<th>Depth (m)</th>
<th>Sample RL (m AHD)</th>
<th>pH_F</th>
<th>pH_FOX</th>
<th>pH_F-pH_FOX</th>
<th>Strength of Reaction</th>
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<tr>
<td>proposed car park</td>
<td>BH1</td>
<td>1.0</td>
<td>3.5</td>
<td>4.73</td>
<td>3.32</td>
<td>1.41</td>
<td>4</td>
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<tr>
<td></td>
<td>BH2</td>
<td>0.5</td>
<td>4.2</td>
<td>4.9</td>
<td>3.48</td>
<td>1.42</td>
<td>4F</td>
</tr>
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<td></td>
<td></td>
<td>2.5</td>
<td>2.2</td>
<td>4.83</td>
<td>3.09</td>
<td>1.74</td>
<td>3</td>
</tr>
<tr>
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<td>BH3</td>
<td>0.5</td>
<td>4.0</td>
<td>7.2</td>
<td>6.4</td>
<td>0.8</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0</td>
<td>3.5</td>
<td>5.6</td>
<td>3.8</td>
<td>1.8</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>BH6</td>
<td>0.95</td>
<td>4.8</td>
<td>4.81</td>
<td>3.45</td>
<td>1.36</td>
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<td>BH14</td>
<td>0.5</td>
<td>5.5</td>
<td>5.26</td>
<td>4.07</td>
<td>1.19</td>
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### Area Sample Location

<table>
<thead>
<tr>
<th>Area</th>
<th>Sample Location</th>
<th>Depth (m)</th>
<th>Sample RL (m AHD)</th>
<th>pH</th>
<th>pHFOX</th>
<th>pH_F-pHFOX</th>
<th>Strength of Reaction</th>
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<tr>
<td>BH15</td>
<td></td>
<td>0.45</td>
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<td>5.5</td>
<td>3.81</td>
<td>1.69</td>
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<td>0.45</td>
<td>4.4</td>
<td>6.06</td>
<td>4.62</td>
<td>1.44</td>
<td>2</td>
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<tr>
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<td>3.85</td>
<td>0.73</td>
<td>2</td>
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<tr>
<td>BH17</td>
<td></td>
<td>0.45</td>
<td>3.8</td>
<td>6.87</td>
<td>6.14</td>
<td>0.73</td>
<td>2</td>
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<td></td>
<td></td>
<td>1.0</td>
<td>3.2</td>
<td>4.5</td>
<td>3.70</td>
<td>0.8</td>
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Notes: pH_F = non-oxidised pH (soil in distilled water), measures existing acidity; pHFOX = peroxide pH

Strength of reaction: 1 = no or slight reaction, 2 = moderate reaction, 3 = vigorous reaction, 4 = ‘volcanic’ reaction, F = bubbling/frothy reaction indicative of organics

Indicative values: pH_F ≤ 4 or pH_F = 4-5 may indicate actual acidity; pHFOX <3 may indicate potential acidity; pH_F-PHFOX ≥ 1 may indicate potential ASS (PASS). Results within the indicative values are highlighted.

On the basis of the pH screening, samples from BH 3 (1.0 m) and BH 17 (1.0 m) were selected for SPOCAS (suspended peroxide oxidation combined acidity and sulphate) testing. These samples were from the natural firm to stiff, brown silty clay at BH 3 and natural stiff, mottled red-brown and light grey clay at BH 17.

The results of the SPOCAS analysis are summarised in Table 2 together with the action criteria specified for fine textured soils in the ASS manual (ASSMAC, 1998). The detailed laboratory results are included in Appendix E.

### Table 2: Results of SPOCAS Testing

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>pH_KCl</th>
<th>pH_OX</th>
<th>Acid trail (mol H+/tonne)</th>
<th>Sulphur trail (% w/w)</th>
<th>Liming Rate (kg CaCO_3/t)</th>
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<tbody>
<tr>
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<td></td>
<td></td>
<td>TPA</td>
<td>TAA</td>
<td>TSA</td>
</tr>
<tr>
<td>BH 3/1.0 m</td>
<td>4.6</td>
<td>4.2</td>
<td>47</td>
<td>29</td>
<td>19</td>
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<tr>
<td>BH 17/1.0 m</td>
<td>4.9</td>
<td>3.9</td>
<td>72</td>
<td>25</td>
<td>47</td>
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<td>Action Criteria</td>
<td>&lt;1000 tonne, fine texture</td>
<td>62</td>
<td>62</td>
<td>62</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>&gt;1000 tonne, fine texture</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>-</td>
</tr>
</tbody>
</table>

The results indicate that both samples meet the action criteria if disturbance of more than 1000 tonnes of the soil is proposed. The sample at BH 17 could be considered to slightly exceed the acid trail action criteria if disturbance of less than 1000 tonnes of the soil is proposed. The probable extent of acid sulphate soils is discussed in further detail in Section 8.1.1.2 and Section 8.1.2.2, for the proposed multi-storey car park and towers building site, respectively.
6.2 Soil Aggressivity

Selected soil samples were laboratory tested for aggressivity to subsurface structures. The results are summarised in Table 3, below. Detailed results are included in Appendix E.

Table 3: Results of Aggressivity Testing

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Depth (m)</th>
<th>pH</th>
<th>Electrical Conductivity (µS/cm)</th>
<th>Resistivity (omh.m)</th>
<th>Chloride (mg/kg)</th>
<th>Sulphate (mg/kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH 2</td>
<td>1.0</td>
<td>4.5</td>
<td>130</td>
<td>76</td>
<td>69</td>
<td>130</td>
</tr>
<tr>
<td>BH 12</td>
<td>0.95</td>
<td>5.0</td>
<td>46</td>
<td>220</td>
<td>&lt;10</td>
<td>55</td>
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<tr>
<td>BH 14</td>
<td>0.95</td>
<td>5.2</td>
<td>51</td>
<td>200</td>
<td>&lt;10</td>
<td>71</td>
</tr>
</tbody>
</table>

The pH results are considered to be consistent with the results of existing acidity undertaken during acid sulphate soil screening. Discussion of aggressivity with respect to subsurface structures is included in Section 8.6.

6.3 California Bearing Ratio Tests

Four bulk soil samples were taken at the site for California Bearing Ratio (CBR) testing, to aid earthworks and pavement design. The compaction properties of the samples were determined, and the sample then prepared in a CBR mould at 100% of the Standard Maximum Dry Density (SMDD) and within 2% of the Standard Optimum Moisture Content (SOMC). Once prepared, the samples were immersed in a water tank for a minimum 4 day period. The results of the laboratory compaction and CBR tests are summarised in Table 4, below.

Table 4: Results of CBR Testing

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Depth (m)</th>
<th>Field Moisture Content (%)</th>
<th>SMDD (t/m³)</th>
<th>SOMC (%)</th>
<th>CBR, 4 day soak (%)</th>
<th>Comment</th>
</tr>
</thead>
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<tr>
<td>3</td>
<td>0.1-0.6</td>
<td>19.0</td>
<td>1.78</td>
<td>18.9</td>
<td>6</td>
<td>0.8% swell, sandy clay filling</td>
</tr>
<tr>
<td>4</td>
<td>0.3-0.7</td>
<td>19.4</td>
<td>1.68</td>
<td>19.2</td>
<td>4.5</td>
<td>1.7% swell, clay</td>
</tr>
<tr>
<td>15</td>
<td>0.1-0.6</td>
<td>15.4</td>
<td>1.60</td>
<td>19.5</td>
<td>6</td>
<td>0.9% swell, extremely weathered shale</td>
</tr>
<tr>
<td>17</td>
<td>0.1-0.6</td>
<td>20.2</td>
<td>1.70</td>
<td>19.9</td>
<td>5</td>
<td>0.4% swell, silty clay filling</td>
</tr>
</tbody>
</table>

The results indicate CBRs of 4.5% in the natural clay soils, with higher CBRs of 6% obtained in the (reworked) extremely weathered shale and 5% to 6% in the sandy clay and clay filling. Swell values are consistent with low to medium plasticity clay. Field moisture contents were generally higher than Standard optimum moisture content, except in the weathered shale at BH 15.
7. Proposed Development

The proposed developments are at Concept Design stage, but will include a multi-storey car park, and the proposed tower building. The footprints of the proposed developments are shown in Drawings 1 to 3, in Appendix B.

At the proposed multi-storey car park the concept designs provided indicate a 5 to 6 storey car park. A stepped footprint is proposed, with floor levels to be approximately 1.5 m higher in the area closest to Hospital Road. No floor levels have been provided. It is expected that the ground floor parking will bear on ground, while the remaining storeys will be separately supported on foundations below the columns. Assuming an 8 m column grid spacing and car park floor loads of 10kPa, working column loads in the order of 3000 kN to 4000 kN are expected for the structure.

At the proposed tower buildings, the following is understood:

- A two level basement is proposed under much of the building footprint, with the basement levels to include plant areas and undercroft areas. The basement floor level and bulk excavation levels are unknown at this stage.
- The new building will extend up Hospital Street, between the existing Building 3 and Building 5, connecting the existing and new buildings. This extension will include a one level basement, which will be at a similar level to existing basements in Building 3 and Building 5.
- The building will include three towers of three to five storeys.
- The building is to be constructed in two phases, Phase 1a and Phase 1b. The initial Phase 1a construction will include the north-eastern half of the proposed area and will include two towers, the two level basement under this footprint and the extension up Hospital Street. Phase 1b will comprise excavation of the remaining basement footprint at the south-western end of the footprint, and construction of the final tower building. The approximate footprints of the Phase 1a and Phase 1b works are shown in Drawing 3, in Appendix B. This final tower building may be extended by a further two storeys, to make a seven storey tower. Demolition of the existing structures will be likewise staged.
- Typical working column loads of 6000 kN have been advised for the towers.

8. Comments

8.1 Geotechnical Model

Due to differences in ground conditions, and the different geotechnical considerations at the two site, separate geotechnical models have been developed for the proposed multi-storey car park and for the proposed towers building areas. The models are summarised in the following sections. The relevant interpreted geotechnical cross-sections are included in Appendix F, showing simplified borehole logs. The plan locations of the cross-sections are shown in Drawings 2 and 3 in Appendix B.
8.1.1 Proposed Multi-Storey Car Park

The geotechnical model for ground conditions at the proposed multi-storey car park are summarised in Table 5, below.

Table 5: Geotechnical Model for Proposed Multi-Storey Car Park

<table>
<thead>
<tr>
<th>Unit</th>
<th>Simple Description</th>
<th>Detailed Description</th>
<th>Estimated Typical Depth Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Filling</td>
<td>Variable filling, including near-surface pavement and roadbase layers, underlain by more variable filling; apparently moderately compacted</td>
<td>Ground surface to up to 1 m depth</td>
</tr>
<tr>
<td>2</td>
<td>Clay</td>
<td>Typically firm to very stiff clay soils; consisting of alluvial and residual soil with ironstone gravel. The boundary between alluvial and residual clay is unclear.</td>
<td>To approximately 2m depth</td>
</tr>
<tr>
<td>3</td>
<td>Shaly Clay</td>
<td>Residual, stiff and very stiff shaly clay with ironstone gravel bands</td>
<td>To approximately 2.5m depth</td>
</tr>
<tr>
<td>4a</td>
<td>Shale</td>
<td>Extremely low and very low strength, fractured, shale, with some low strength bands, and including some high strength, iron-cemented bands. Includes some extremely weathered sandstone (e.g. at BH 3)</td>
<td>To depths of 4.5 m to 6.5 m or to top of sandstone (Unit 5), where higher</td>
</tr>
<tr>
<td>4b</td>
<td></td>
<td>Low and medium strength shale, fractured, with some high strength, iron-cemented bands</td>
<td>between Unit 4a and Unit 5</td>
</tr>
<tr>
<td>5</td>
<td>Sandstone</td>
<td>Medium and high strength sandstone, fractured, with some low and possibly some very high strength bands</td>
<td>Below approximately RL 0.5 and RL -0.5 m at the north-western side of the site, but falling towards the south-east</td>
</tr>
</tbody>
</table>

The units of the interpreted geotechnical model are shown superimposed on simplified boreholes in Drawing 4, in Appendix F. It is noted that Cross-Section A-A’ includes the boreholes and interpreted units at 33 m offset from this Section. Interpreted Cross-Section E-E’ is also included on that drawing, and illustrates the absence of Unit 4b in some parts of the site and the falling sandstone level.

The following notes are made on the geotechnical model:

- The evaluation of the existing filling as moderately-compacted is based on the existing, good condition of the pavements at the sites. This good condition may, however, be due to relatively infrequent heavy loads on the pavement. As there are no records of the filling placement, it is assumed to be uncontrolled filling. This is also consistent with the significant variation in filling materials at the site.
- The transition from alluvial to residual soils (in Unit 2) is unclear at this site from the small-diameter augered boreholes, as similar strength and colouring is present in both soils. It is considered likely that the presence of ironstone gravels will be the primary indicator of residual soils. It is also expected that the transition depth from alluvial to residual will vary across the site,
due to likely variations in the depth and plan extent of the shoreline at the time of the alluvial deposition.

- The shale at this site is susceptible to strength loss due to weathering, with low and medium strength shale (Unit 4b) present only below depths of approximately 4.5 m to 6.5 m. The top of sandstone is present above this depth at some location (i.e. in areas of lower ground levels), therefore Unit 4b is absent in these areas.

- The depth to consistent sandstone appears to be relatively consistent at BH 1, BH 2 and BH 3 (between RL 0.5 and RL 0 m) but is not encountered at these levels at BH 4, BH 5 and BH 6, suggesting that the top of sandstone generally falls towards the south or south-east. This is also consistent with the lower sandstone levels found at the proposed towers building.

- The sandstone is less susceptible to strength loss due to weathering, and is therefore expected to be relatively consistently medium and high strength across the site. Some low strength bands were present at BH 1, and very high strength sandstone bands may also be present, based on the boreholes at the proposed towers building.

8.1.1.1 Groundwater

No groundwater observations were noted during augering (i.e. within the filling and clays). In combination with the ground levels at the site, it is considered likely that the permanent groundwater level is within the rock, some seepage should be expected through the soils, particularly at interfaces between layers (e.g. at the base of Unit 1, 2 and 3 soils), particularly following periods of rainfall.

8.1.1.2 Acid Sulphate Soil

The presence of acid sulphate soils at the proposed multi-storey car park was suggested by the SPOCAS testing of a sample from BH 3 at 1.0 m. Similar responses to acid sulphate screening were also obtained from numerous other soil samples in this part of the site (refer to Table 1), and suggest that similar soils are present across much of this area. The extent of acid sulphate soils is likely to be limited by the typical upper level of acid sulphate soils (RL 5 m), the base of pavement filling and the top of residual clay (up to approximately 2 m depth). The variable filling below the asphaltic concrete and roadbase layers is of unknown original, and its status is unknown.

8.1.2 Proposed Towers Building

The ground conditions at the proposed towers building is more variable that the proposed car park. The resulting geotechnical model is summarised in Table 6, below.
### Table 6: Geotechnical Model for Proposed Towers Building

<table>
<thead>
<tr>
<th>Unit</th>
<th>Simple Description</th>
<th>Detailed Description</th>
<th>Estimated Typical Depth Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>Filling</td>
<td>Highly variable filling, including near-surface pavement and roadbase layers at existing roads. Variable compaction expected.</td>
<td>To up to 0.5 m depth, but deeper at some locations, particularly below the tennis court bench and near underground services</td>
</tr>
<tr>
<td>12</td>
<td>Clay</td>
<td>Firm to very stiff clay and shaly clay. Typically residual soils, but including some alluvial clay at BH 17.</td>
<td>To depths of 0.3 m to 1 m (and of a thickness of up to 1.5 m), but to depths of up to approximately 2.5 m in areas of bulk filling.</td>
</tr>
<tr>
<td>13a</td>
<td>Shale</td>
<td>Extremely low and very low strength, fragmented to fractured shale, with some low and medium strength bands, high and very high strength iron-cemented bands; includes some local laminite (e.g. BH 11)</td>
<td>To depths of approximately 3 m to 6 m below ground level</td>
</tr>
<tr>
<td>13b</td>
<td></td>
<td>Low and medium strength, fractured shale, with some high and very high strength iron-cemented bands; includes some local laminite (e.g. BH 11) and fine grained sandstone, with some extremely and very low strength bands</td>
<td>To depths of approximately 4.5 m to 7 m below ground level</td>
</tr>
<tr>
<td>14</td>
<td>Laminate</td>
<td>Consistently medium and high strength, slightly fractured and fractured, shale, laminite and sandstone</td>
<td>To depths of 5.5 m to 7.5 m below ground level</td>
</tr>
<tr>
<td>15a</td>
<td>Sandstone</td>
<td>High and very high strength, unbroken to slightly fractured, fine and fine to medium grained sandstone</td>
<td>To depths of 9.5 m to 11.5 m below ground level</td>
</tr>
<tr>
<td>15b</td>
<td></td>
<td>Medium to coarse grained, unbroken, high strength sandstone</td>
<td>To beyond the limit of investigation</td>
</tr>
</tbody>
</table>

The units of the interpreted geotechnical model are shown superimposed on simplified boreholes in Drawings 5-7, in Appendix F. While the proposed basement depths are not yet known, an illustrative basement line has been included in these cross-sections to show the lateral extent of the basement, and to assist with reporting. The shown basement level is at RL 5.5 m for the single basement level at Hospital Street and RL 2.5 m in the two level towers basement. These levels, however, do not correspond to known basement or excavation levels and should be considered illustrative only.

The following notes are made on the geotechnical model:

- The condition, depth and consistency of the existing filling is expected to be variable, as multiple phases of filling are expected to have occurred on the site. Filling is expected to be of greater depth below the tennis courts, and in the immediate vicinity of below ground services.
• The natural clay and shaly clay is present at most locations on the site. It is generally considered to be residual. Some alluvial soil is, however, present at BH 17, as evidenced by the acid sulphate soil test results.

• The rock at the site appears to generally progress through shale, then laminite, then sandstone. This is considered to be consistent with rock progressing from Ashfield Shale, then through the Mittagong Formation to Hawkesbury Sandstone. The variability in rock types is consistent with this transitional layer, and exhibits some variation across the site (see, for example, the laminite overlying shale at BH 11). For practical purposes, the geotechnical model is therefore primarily based on rock strength and fracturing, rather than rock type, and some variation in rock type is present within each unit. The predominant rock type is identified by the brief description.

• The shale is generally deeply weathered, with an upper extremely low to very low strength layer (Unit 13a) transitioning to low and medium strength (Unit 13b). Significant variation in rock strength and fracturing is observed within these units. These variations do not, however, reliably map across significant areas of the site.

• The strength of the shale and laminite do, however, tend to improve to a consistently medium strength with depth (Unit 14).

• The sandstone is typically of high and very high strength, with a very high strength band common at the upper boundary of the sandstone (Unit 15a). Towards the base of some boreholes, consistently high strength, medium to coarse grained sandstone has been identified (Unit 15b), which is considered likely to be Hawkesbury Sandstone.

• This area of the site has been affected by past cut and fill. This can be seen in the current site topography (e.g. batters around the tennis court), and from the results of boreholes (e.g. truncated upper units at BH 10. The void at BH 13 is considered to be a man-made local feature, and is not reflected in the geotechnical model. It does, however, suggest that past human disturbance at this site may be more extensive than on a similar, less developed site.

8.1.2.1 Groundwater

No groundwater observations were noted during augering. Groundwater at BH 17 was measured at 4.2 m depth (RL 0) i.e. within the shale.

It is considered likely that the permanent groundwater level is within the rock, some seepage should be expected at interfaces between layers (e.g. towards the base of each geotechnical unit) and through fractures and joints in the shale, particularly following periods of rainfall.

8.1.2.2 Acid Sulphate Soil

Comparison of the results of SPOCAS testing of soil from BH 17, at 1.0 m depth gave marginal results when compared to the action criteria for disturbance of less than 1000 tonnes, or meet the acid trail action criteria for disturbance of more than 1000 tonnes. It is further noted, that only BH 16 and BH 17 encountered natural soils below RL 5 m, which is generally the upper limit of coastal acid sulphate soils. In addition, the clay at BH 16, while of relatively low natural pH, has been identified as shaly clay, and is therefore considered to be of residual origin and not an acid sulphate soil.

The presence of ASS in the natural clays in this area (Unit 12) is therefore expected to be limited to the vicinity of BH 17. Additional acid sulphate soil may possibly be present within the filling (Unit 1), if
acid sulphate soil was used. Given the significant variability in filling at this site, however, any ASS within the filling, if present, is likely to be local.

8.2 Excavation

8.2.1 Excavation Conditions

At the proposed car park, excavation may be required for subgrade preparation and for cut and fill operations. The extent of such works is not known at this stage, as ground levels have not been provided. It is considered likely, however, that any such excavation will be largely within the filling and natural clay soils (Unit 1 and Unit 2). Some excavation into the shaly clay and deeper extremely low and very low strength shale (Unit 3 and 4a) may occur towards Hospital Road, if significant cut is proposed in this area.

At the proposed towers building (including Hospital Street) excavation will be required for the proposed basements. Excavation depths are not known, however depths of up to 3 m (Hospital Street basement) and up to 6 m (towers basements) are considered likely, and are shown for illustrative purposes in Drawings 5 to 7, in Appendix E. Some limited cut and fill may also be required around the buildings for proposed roads.

As can be seen in Drawing 7 in Appendix E, the towers building excavation is expected to be largely within the filling, clay and extremely low to very low strength shale (Units 11, 12 and 13a). Excavation into some low and medium strength shale (Unit 13b) and medium and high strength laminite (Unit 14) is expected towards the north-eastern side of the two-basement level area and for much of the Hospital Street basement.

At both sites, the excavation in the soils and extremely low to very low strength shale should generally be readily undertaken using conventional earthmoving equipment (e.g. bulldozers and hydraulic excavators). Iron-cemented bands, or other stronger bands within the rock may be present within these units and sawing and/or rock hammering may be required locally, if these bands are thick.

Excavation in medium and high strength shale and laminite, as expected at the towers building only (Units 13b and 14) will require excavator mounted rock hammers, rock saws or milling heads. The size of rock hammer that may be used is likely to be limited by the vibrations generated by the excavation process. Rock saws or milling heads generate much less vibration than rock hammers but generate substantially more dust. Measures for control of dust generated by rock saws, milling heads or other excavation techniques will be required.

No significant contamination was identified at the site by the preliminary contamination investigation. All excavated materials will, however, need to be disposed in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (DECCW, 2014). This includes filling and natural materials that may be removed from the site. Reference should be made to DP Report 85356.01.R.001.Rev0 for further information.

8.2.1.1 Groundwater and Dewatering

The regional groundwater table is expected to be within the rock and is considered likely to be largely below the bulk excavation level for the proposed two level basement, but depending on excavation
levels. Some seepage through the soil and rock from temporary flows will always be expected. As the shale on the site is likely to be of relatively low permeability, groundwater inflows are expected to be manageable. A suitably designed sump type drainage system should be installed in the basement to collect and discharge seepage if a gravity system cannot be used. Seepage may also need to be removed from bored pile and footing excavations prior to pouring concrete.

8.2.1.2 Vibrations

High vibration levels are not anticipated during works at the proposed multi-storey car park.

At the proposed tower building, while excavation of the soil and weak rock by excavator are expected to produce only minor vibrations, the excavation of medium and high strength rock (and bands within the rock) is likely to produce significant vibrations.

During excavation it will be necessary to use appropriate methods and equipment to keep ground vibration within acceptable limits. The standards listed below are considered appropriate documents on which to base the management of ground vibration:

- German Standard DIN4150-3-1999 “Structural vibration – effects of vibration on structures”; and
- Australian Standard AS2670.2-1990 “Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)”.

Given the presence of adjacent hospital buildings, it is likely that the structures and equipment adjacent to the site will be more sensitive to vibration than their occupants. A sensitive structural criterion is therefore indicated and the vector sum peak particle velocity (VSPPV) is proposed as the control parameter. It is recommended that an initial Provisional Allowed Vibration Limit of 5.0 mm/sec (VSPPV) be set, at foundation level of the potentially affected building/s. This is the lowest value of the Sensitive Structural Limit of DIN4150-3-1999 (requiring no assumptions about the frequency of particle vibration) and is lower than the AS2670.2-1990 guideline limit for human comfort. It is noted, however, that vibrations at this level would still be strongly perceptible to occupants of the building. The vibration limits at this site may need to be further limited by the requirements of equipment and functions within the adjacent buildings, and this should be further investigated by the Client to ensure an appropriate vibration limit is set.

As lower vibration limits usually result in longer time frames for the works, some balance is usually required between the allowed vibration limit and the duration of vibration-inducing site works.

DP maintains a database of vibration trial results which can provide guidance for the selection of plant. Trial data is dependent on site conditions, equipment and excavation method, hence actual vibration levels may differ from predictions. A specific trial is therefore recommended at the commencement of rock excavation, to assess vibration levels due to the plant proposed at the site. Table 7, below provides indicative buffer distances for selected plant, based on a Provisional Allowed Limit of 5 mm/s VSPPV and the DP database. These indicate distances which should be maintained between typical excavation plant and adjacent buildings to limit vibration levels at those buildings.
Table 7: Preliminary Buffer Distances for a Provisional Allowed Limit of 5 mm/s VSPPV

<table>
<thead>
<tr>
<th>Plant Items</th>
<th>Distance from plant by which vibration is likely to attenuate to a Provisional Allowed Limit of 5 mm/s VSPPV</th>
<th>From DP trial maxima</th>
<th>From DP trial averages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moving machinery (eg excavator)</td>
<td>9 m</td>
<td>0.9 m</td>
<td></td>
</tr>
<tr>
<td>Trimmers (grinders/milling heads)</td>
<td>2 m</td>
<td>1 m</td>
<td></td>
</tr>
<tr>
<td>Rock Saw on excavator (^1)</td>
<td>2 m</td>
<td>0.8 m</td>
<td></td>
</tr>
<tr>
<td>Jackhammers</td>
<td>5 m</td>
<td>1 m</td>
<td></td>
</tr>
<tr>
<td>Auger drilling rigs</td>
<td>2 m</td>
<td>0.6 m</td>
<td></td>
</tr>
<tr>
<td>Rippers on 6 – 36t excavators</td>
<td>5 m</td>
<td>1 m</td>
<td></td>
</tr>
<tr>
<td>Rock Hammers &lt; 501 kg</td>
<td>11 m</td>
<td>4 m</td>
<td></td>
</tr>
<tr>
<td>Rock Hammers 501 - 1000 kg</td>
<td>10 m</td>
<td>5 m</td>
<td></td>
</tr>
<tr>
<td>Rock Hammers 1001 - 2000 kg</td>
<td>19 m</td>
<td>7 m</td>
<td></td>
</tr>
<tr>
<td>Rock Hammers &gt; 2000 kg (^2)</td>
<td>9 m</td>
<td>6 m</td>
<td></td>
</tr>
</tbody>
</table>

Notes: *Smaller distances can generally be determined from individual trials, as indicated by those from trial averages;

1. Buffer distances for rock hammers may be reduced by prior saw cutting along, or parallel to, excavation boundaries, to reduce vibration transmitted in surface waves. These cuts should be progressively deepened and extended laterally, to maintain a barrier to direct surface waves between the hammer and the structures to be protected;

2. Buffer distances for rock hammers increase with increasing hammer operating weight up to approximately 2000 kg, however a trend reversal is apparent in currently available data for heavier hammers (on heavier carriers), which is attributed to more efficient application of force and less carrier vibration;

3. Loading effects from adjacent buildings may reduce vibration levels, to enable boundary saw cuts with few exceedances; and

4. The use of diamond saws (not represented in DP trial data), may produce negligible foundation vibration, even when cutting immediately adjacent to those foundations.

As the hospital site is likely to be vibration sensitive, it is recommended that vibration monitors are installed at the foundation level of adjacent, sensitive buildings. Additional monitors in vibration sensitive locations (e.g. near sensitive equipment within the building) should also be considered. Vibration levels may then be monitored throughout the works, and the monitors may be set to trigger warning lights or issue notifications to advise that the allowed limit has been breached. The vibration-inducing work may then be halted and the work method revised or plant changed to reduce vibrations.

8.2.1.3 Acid Sulphate Soils

At the proposed multi-storey car park site, excavation into possible acid sulphate soils is likely to be limited to piling excavation. The acid sulphate soil test results at this site are, however, below the action criteria for disturbance of less than 1000 tonnes soil. Based on the currently available information, the disturbance is expected to be limited to less than 1000 tonnes (and therefore no management plan is required), assuming:

- pile diameters of 0.6 m to 0.9 m diameter on an 8 m by 8 m grid; and,
- no significant excavation is required into the alluvial clays (or no excavation into clays below RL 5 m) for earthworks.
At the proposed towers building, excavation into acid sulphate soils may potentially occur during basement excavation works in the vicinity of BH 17. Based on the existing information on the site development, however, it is unclear whether more than 1000 tonnes of ASS soil will be disturbed. Furthermore, the test result at BH 17 indicates only a slight exceedance (for less than 1000 tonnes disturbance) on the acid trail. Further assessment to define the extent of acid sulphate soils around this borehole would be appropriate and may indicate that the results at BH 17 are not representative. The scope of further assessment should be informed by more detail on the works proposed in the area, including excavation depths.

Based on the above, it is likely that excavation into acid sulphate soil can be readily managed. Further assessment, and possibly further investigation, would be appropriate once more detailed information is available on the proposed works. The development of a management plan may be required, depending on the proposed works and results of any further investigation.

If required, management works are likely to include the following in the areas of acid sulphate soils:

- Adopting suitable construction measures to limit the amount of spoil produced on site in the areas of ASS;
- Placing ASS spoil into bins lined with plastic and lime in accordance with the recommendations of the NSW Acid Sulphate Soil Manual. The spoil should then be tested to determine whether PASS or ASS is present. If present, the spoil will need to be neutralised with lime.
- Keeping the spoil wet at all times.

### 8.2.2 Batters

Vertical excavations in filling, soil and weathered rock are unlikely to be stable for any significant period of time and will need to be battered or shored to support the adjacent ground. It is recommended that temporary and long term batter slopes for this site be no steeper than those given in Table 8. Ground conditions, geometry, available space or surcharges (e.g. due to existing foundations or proposed plant) may require flatter slopes or the provision of excavation support.

<table>
<thead>
<tr>
<th>Material</th>
<th>Relevant Units</th>
<th>Temporary Batter Slope</th>
<th>Long Term Batter Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Controlled Filling</td>
<td>new</td>
<td>1.5V:1H or flatter</td>
<td>2V:1H or flatter</td>
</tr>
<tr>
<td>Filling and natural clay</td>
<td>1, 2, 3, 11, 12</td>
<td>1.5V:1H or flatter</td>
<td>2V:1H or flatter</td>
</tr>
<tr>
<td>Extremely and very low strength rock</td>
<td>4a, 13a</td>
<td>1V:1H or flatter</td>
<td>2V:1H or flatter</td>
</tr>
<tr>
<td>Low and medium strength shale</td>
<td>4b*, 13b</td>
<td>0.5V:1H or flatter</td>
<td>1V:1H or flatter</td>
</tr>
<tr>
<td>Consistently medium and high strength shale, laminites and sandstone</td>
<td>5*, 14, 15a*, 15b*</td>
<td>Vertical or flatter</td>
<td>1V:1H or flatter</td>
</tr>
</tbody>
</table>

Note: These batter slopes are based on a level ground surface behind the batter and beyond the batter toe, no surcharges and a maximum batter height of 4 m. *No excavation into Units 4b, 5, 15a or 15b is anticipated.
Review of these batter slopes would be appropriate if weaker soils are encountered, if seepage emerged from the slope or where any surcharges are present behind the crest of the batter (e.g. plant, foundations). The stability of slopes in rock may also be dictated by the jointing and fracturing of the rock mass, and may require flatter slopes than recommended above. Regular geotechnical inspection of batter faces during excavation would be appropriate during the works to assess the risk of adverse defects.

Any long term slopes will generally need to be protected against erosion and weathering. Slopes that are to be vegetated will generally require flatter slopes for maintenance (3H:1V recommended). Alternatively, slope faces may be protected by a hard face e.g. by the use of mesh and shotcrete, pinned into the face.

If batters cannot be provided, or are otherwise unsuitable, then shoring support will be needed.

### 8.2.3 Excavation Support

Excavation support will generally be required at the proposed towers building for basement excavations adjacent to existing structures and below existing foundations. Support may not be required in Hospital Street, depending on the depth of excavation relative to the structure of the adjoining Buildings 3 and 5. New retaining walls may also be required at the proposed multi-storey car park (e.g. to retain the Hospital Road, or around the perimeter of any new controlled filling area), depending on final ground floor levels.

For the towers basement, a suitable shoring system would be a soldier pile wall. This type of wall is constructed by installing bored piles around the perimeter of the excavation at about 2 m centres. Infill panels of reinforced shotcrete are then installed to support the materials between the piles as the excavation proceeds. More closely spaced or contiguous piles may be appropriate locally, if the wall is required to provide confinement and support for existing shallow footings or movement-sensitive infrastructure. Rock anchors may be used to tie the wall back into the ground, to provide additional support and limit wall movement.

The soldier pile wall can form the permanent basement wall or a block wall can be constructed inside the excavation and the void backfilled.

### 8.2.3.1 Design

Simply supported shoring and retaining walls required on the site could be designed using the parameters shown in Table 9.
Table 9: Design Parameters for Retaining Walls

<table>
<thead>
<tr>
<th>Material</th>
<th>Relevant Units</th>
<th>Coefficient of Active Earth Pressure (Ka)</th>
<th>Ultimate Passive Earth pressure (kPa)</th>
<th>Bulk Unit Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Temporary</td>
<td>Permanent</td>
<td></td>
</tr>
<tr>
<td>Controlled filling</td>
<td>new</td>
<td>0.25</td>
<td>0.3</td>
<td>100</td>
</tr>
<tr>
<td>Filling and natural clay</td>
<td>1, 2, 3, 11, 12</td>
<td>0.3</td>
<td>0.35</td>
<td>100</td>
</tr>
<tr>
<td>Extremely and very low strength rock</td>
<td>4a, 13a</td>
<td>0.15</td>
<td>0.25</td>
<td>400</td>
</tr>
<tr>
<td>Low and medium strength shale</td>
<td>4b, 13b</td>
<td>0.15</td>
<td>0.25</td>
<td>1000</td>
</tr>
<tr>
<td>Consistently medium and high strength shale, laminite and sandstone</td>
<td>5, 14, 15a*, 15b*</td>
<td>0</td>
<td>0.1</td>
<td>4000</td>
</tr>
</tbody>
</table>

The above parameters are provided for a horizontal ground surface behind and beyond the toe of the wall. The development of active and passive earth pressures is dependent on some movement of the wall. Where movement of the wall is to be reduced the initial design may be based on an ‘at rest’ earth pressure on the active side, 50% higher than the active earth pressure coefficients given above.

Where movement of the wall is critical (e.g. due to the presence of foundations or movement-sensitive services) more detailed design is recommended using an appropriate analysis program that can also indicate potential movement for the proposed wall/soil system, such as Wallap. In these cases a multi-tied wall system is likely to be required, except for very low retaining walls. The preliminary design of multi-tied system may be based on a trapezoidal pressure distribution and a maximum earth pressure of 4.4H, where H is the height of the wall in metres and pressure is in kPa. The maximum earth pressure should be taken to act over the central half of the wall, reducing to zero at the top of the wall and the bottom of the excavation.

Consideration should also be given to the potential presence of 45° joints within the rock, which are common in Ashfield Shale, and which were identified within the rock core. These joints may lead to wedge failures in the rock and thence upwards into the underlying soils. Slope support should therefore also be checked to ensure it is adequate for the case of a 45° wedge failure extending from the base of the excavation up to surface level.

In all cases additional allowance should be made for surcharge loads from structures, sloping ground surfaces, road pavement and construction machinery. Adequate drainage should be provided behind the wall to reduce the risk of hydrostatic pressures acting on the wall, or alternatively, the wall designed with allowance for full hydrostatic loading.

Assessment of the existing structures on the site in the vicinity of the excavation will form an important part of assessing support requirements. The depth, type and foundation material of foundations and retaining walls adjacent to the excavation will necessarily inform the design of excavation support. Depending on these factors underpinning and/or additional lateral restraint may be required.
### 8.2.3.2 Anchors

Temporary lateral support in the form of anchors is likely to be required during excavation where retaining walls are deep (say, more than 3 m to 4 m), or where the area to be retained is movement sensitive and/or includes high surcharges (e.g. due to adjacent foundations). These conditions are only anticipated at the towers building, based on the concept design information.

Long term support for excavation retaining walls is usually provided by the final building. Given the sloping land at this site, in some areas this would involve the transmission of shear forces into the foundations. Alternatively the use of permanent anchors could be considered.

The detailing, testing, maintenance regime and factors of safety adopted for the anchors should reflect the required design life.

It is recommended that any anchors be angled downward in order to obtain a bond length within higher strength bedrock. The anchors should have a free length extending beyond a 45° line drawn up from the base of the excavation, with a minimum free length of 3 m.

Preliminary temporary anchor design could be based on the working bond stresses given in Table 10 for strand anchors installed in grouted, inclined boreholes.

#### Table 10: Preliminary working bond stresses

<table>
<thead>
<tr>
<th>Unit</th>
<th>Description</th>
<th>Preliminary Working Bond Stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13a</td>
<td>Extremely low and very low strength shale</td>
<td>70</td>
</tr>
<tr>
<td>13b</td>
<td>Low and medium strength shale</td>
<td>150</td>
</tr>
<tr>
<td>14</td>
<td>Consistently medium and high strength shale, laminite and sandstone</td>
<td>700</td>
</tr>
<tr>
<td>15a &amp; 15b</td>
<td>Sandstone – very high and high strength</td>
<td>2000</td>
</tr>
</tbody>
</table>

In practice, the available bond stresses will depend on the drilling and installation methodology of the anchoring contractor, and should therefore be confirmed by the contractor. It is recommended that the final design bond stresses are confirmed by the installation of test anchors by the contractor, followed by testing to failure in the appropriate materials. Such testing is particularly important if permanent anchors are proposed.

After installation, all anchors should be proof stressed to at least 1.25 or 1.5 times the working load for temporary or permanent anchors, respectively. Proof stressing of a proportion of anchors should also be carried out at intervals after installation to ensure that the load is maintained in the anchors and not lost due to creep effects. The appropriate proportion and intervals will depend on the proposed life of the anchors. For permanent anchors, the final detailing of walls around the anchors should be suitable to permit continued periodic proof-testing for the life of the anchors.
8.3 Foundations

Working column loads in the order of 3000 kN to 4000 kN are anticipated for the proposed multi-storey car park, and of 6000 kN at the proposed towers building. For these loads, foundations bearing on bedrock will be required. Pile foundations are therefore anticipated at the car park site, and in parts of the towers building site, with possible shallow foundations in the remaining areas.

8.3.1 Construction

Given the relatively shallow soil, and typically firm to very stiff clay soils, it is considered likely that open augered bored piles will be suitable for pile foundations on this site. Some upper casing may be required to protect against upper soils, and particularly granular filling, from falling into the pile holes. Wet augering conditions may be encountered at depth.

Given the high and very high rock encountered in some areas of the site, it would be prudent to provide the detailed borehole logs to the piling contractor to allow them to assess the appropriate plant for the site. Slow drilling conditions should be expected where foundations are taken down to or into high strength rock. Such foundation conditions are considered likely given the high column loads proposed.

8.3.2 Design

Recommended maximum ultimate and allowable pressures and modulus values for foundation materials encountered by the investigation are presented in Table 11, below, based on rock classes.

### Table 11: Recommended Parameters for Foundation Design

<table>
<thead>
<tr>
<th>Foundation Classification ****</th>
<th>Description</th>
<th>End Bearing Pressure (kPa)</th>
<th>Shaft Adhesion (kPa)*</th>
<th>Field Elastic Modulus, E (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Allowable</td>
<td>Ultimate**</td>
<td>Allowable</td>
</tr>
<tr>
<td>Shale Class V</td>
<td>Extremely low strength shale and laminite</td>
<td>700</td>
<td>3000</td>
<td>75</td>
</tr>
<tr>
<td>Shale Class IV</td>
<td>Very low strength shale and laminite</td>
<td>1000</td>
<td>4000</td>
<td>100</td>
</tr>
<tr>
<td>Shale Class III</td>
<td>Low strength shale and laminite</td>
<td>1500</td>
<td>6000</td>
<td>150</td>
</tr>
<tr>
<td>Shale Class II (lower bound)</td>
<td>Medium strength shale and laminite</td>
<td>3500</td>
<td>30000</td>
<td>350</td>
</tr>
<tr>
<td>Shale Class II (upper bound) ***</td>
<td>Medium to high strength shale and laminite</td>
<td>6000</td>
<td>120000</td>
<td>750</td>
</tr>
</tbody>
</table>
### Foundation Classification

<table>
<thead>
<tr>
<th>Foundation Classification</th>
<th>Description</th>
<th>End Bearing Pressure (kPa)</th>
<th>Shaft Adhesion (kPa)*</th>
<th>Field Elastic Modulus, E (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Allowable</td>
<td>Ultimate**</td>
<td>Allowable</td>
</tr>
<tr>
<td>Sandstone Class III</td>
<td>Medium strength sandstone</td>
<td>3500</td>
<td>30000</td>
<td>350</td>
</tr>
<tr>
<td>Sandstone Class II (lower bound) ***</td>
<td>High strength sandstone</td>
<td>6000</td>
<td>40000</td>
<td>600</td>
</tr>
<tr>
<td>Sandstone Class II (upper bound) ***</td>
<td>Medium to high strength sandstone</td>
<td>12000</td>
<td>120000</td>
<td>1200</td>
</tr>
</tbody>
</table>

Notes:

* Shaft adhesion provided for pile foundations in compression only; assumes adequately roughened and clean pile sockets and a minimum pile depth of 1.5 times the pile diameter; reduce by 50% to obtain values for design against uplift.

** Ultimate values occur at large settlements, typically >5% of the minimum footing dimension

*** Material consistent with Class II Shale and Sandstone has been encountered by the investigation at the towers site. The current density of testing does not, however, warrant the use of these (grey text) values due to the potential for the presence of weaker underlying rock or seams. Should these values be adopted for the design of pile foundations, then additional investigation would be required at the subject foundations (or a proportion thereof) to confirm the classifications. Such investigation may also indicate that higher values may be adopted. For shallow foundations (e.g. in the shale below much of Hospital Street), the use of spoon testing to 1.5 times the minimum plan dimension could assess the depth of shale, and permit these higher values to be adopted (if no seams are found by testing)

**** Foundation classification is based on strength and defect criteria, and may not match the geotechnical model units – refer Tables 11 and 12.

It is noted that the values given in Table 11 have been limited based on the density of investigation on the site and variability or potential variability of ground conditions (e.g. frequent presence of seams within the shale). Higher values may potentially be obtained. Depending on the proposed design, further investigation such as by cored boreholes at highly loaded column locations, or reassessment of the existing boreholes, where foundation and borehole locations coincide, may allow these parameters to be increased in the borehole vicinity.

The broad foundation classification for rock at specific borehole locations is summarised in Tables 12 and 13, below, based on the generalised classifications given above. As noted above, the classifications have been broadly downgraded based on the density of investigation and defects within the rock. Higher values and classifications may, however, be appropriate for foundations at specific borehole locations. Augering to the required foundation depth may therefore require augering through higher strength material, including iron-cemented bands, and potentially high and very high strength, massive rock. It is therefore recommended that the piling contractors be provided with the full borehole logs to ensure that appropriate piling equipment is adopted for the site.
Table 12: Broad Foundation Classification at Borehole Locations

<table>
<thead>
<tr>
<th>Foundation Classification</th>
<th>BH1</th>
<th>BH2</th>
<th>BH3</th>
<th>BH4</th>
<th>BH5</th>
<th>BH6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m) to top of:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shale Class V</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Shale Class IV</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Shale Class III</td>
<td>4.7</td>
<td>4.6</td>
<td>2.5</td>
<td>5.3</td>
<td>4.25</td>
<td>4.6</td>
</tr>
<tr>
<td>Shale Class II (lower bound)</td>
<td>5.0</td>
<td>5.0</td>
<td>4.0</td>
<td>5.8</td>
<td>6.1</td>
<td>4.6</td>
</tr>
<tr>
<td>Sandstone Class III</td>
<td>5.0</td>
<td>-</td>
<td>4.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 13: Broad Foundation Classification at Borehole Locations

<table>
<thead>
<tr>
<th>Foundation Classification</th>
<th>BH10</th>
<th>BH11</th>
<th>BH12</th>
<th>BH14</th>
<th>BH15</th>
<th>BH16</th>
<th>BH17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m) to top of:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shale Class V</td>
<td>0.3</td>
<td>2.6</td>
<td>0.8</td>
<td>0.8</td>
<td>0.25</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Shale Class IV</td>
<td>-</td>
<td>-</td>
<td>2.3</td>
<td>1.0</td>
<td>3.4</td>
<td>3.7</td>
<td>-</td>
</tr>
<tr>
<td>Shale Class III</td>
<td>1.0</td>
<td>3.3</td>
<td>3.8</td>
<td>5.9</td>
<td>-</td>
<td>-</td>
<td>4.1</td>
</tr>
<tr>
<td>Shale Class II (lower bound)</td>
<td>1.6</td>
<td>4.4</td>
<td>6.0</td>
<td>6.1</td>
<td>5.0</td>
<td>5.8</td>
<td>4.7</td>
</tr>
<tr>
<td>Shale Class II (upper bound)</td>
<td>2.8</td>
<td>5.2</td>
<td>7.5</td>
<td>6.1</td>
<td>-</td>
<td>5.8</td>
<td>4.7</td>
</tr>
<tr>
<td>Sandstone Class III</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>7.2</td>
<td>7.0</td>
<td>7.6</td>
<td>5.6</td>
</tr>
<tr>
<td>Sandstone Class II (lower bound)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>7.2</td>
<td>7.0</td>
<td>7.6</td>
<td>5.6</td>
</tr>
<tr>
<td>Sandstone Class II (upper bound)</td>
<td>-</td>
<td>-</td>
<td>9.2</td>
<td>7.2</td>
<td>7.0</td>
<td>7.6</td>
<td>5.6</td>
</tr>
</tbody>
</table>

Note: Classification not possible at BH 13 due to presence of void below cored section. Founding above a void is not advised. If the void is ignored, the rock from 1.0 m depth is generally consistent with Shale Class IV.

The values provided assume that the foundation excavation is suitably cleaned prior to pouring of concrete, that the sidewalls of piles are suitable roughened to ‘R2 roughness’ and clean of smear and that concrete is poured on the same day as drilling to reduce the risk of softening.

The following foundation design examples have been developed using the values given in the tables above:

- Based on the results at BH 1, for a car park column supporting a 3000 kN working load, a 900 mm diameter pile with a 2.5 m socket in Class V shale, and 2 m socket in Class III Sandstone could be considered.
Based on the results at BH 10, for a Hospital Street column supporting a 6000 kN working load, a 1 m by 1 m pad footing below the basement level bearing on Class II (upper bound) shale could be adopted, provided that spoon testing were carried out to 1.5 m (i.e. 1.5 times the minimum plan dimension) and results are consistent with the upper bound Class II Shale classification (by the absence of significant clay or fragmented shale seams below the foundation).

Based on the results at BH14, for a tower building column supporting a 6000 kN working load at this location, a 900 mm diameter pile footing drilled down to found on Class II (upper bound) sandstone could be adopted. Additional investigation would be appropriate before adopting these higher bearing capacities more broadly.

8.3.3 Settlement

The settlement of foundations proportioned on the basis of the above allowable parameters would be expected to not exceed 1% of the footing width/diameter.

Differential movement should be expected between structures or services founded on rock, and those founded on soils. Allowance should be made for suitable articulation between structures or infrastructure that are founded on the different strata.

8.4 Site Preparation

Site preparation is likely to be required for the ground floor of the new car park, and for roads associated with the tower building. The following process is recommended:

- Where present, the existing vegetation, organic-rich soil layers, buried tree roots or trucks should be stripped from the site;
- The existing pavement layers and filling should be stripped and removed. These materials may be assessed for re-use in engineered filling. Based on the filling encountered at test locations it is considered likely that the majority of the filling will be suitable for re-use.
- The exposed (natural) subgrade should be proof-rolled in the presence of an experience geotechnical professional using a minimum 12 tonne pad foot or steel smooth drum roller. Any areas on the site exhibiting excessive deflection should be excavated and replaced with suitable, select material compacted in layers. Specific remediation advice can be provided on site once the subgrade has been inspected.
- Any new filling should be placed in layers not exceeding a loose thickness of 300 mm and compacted to a dry density ratio of between 98% and 102% relative to Standard Maximum Dry Density, and within 2% of Standard Optimum Moisture Content.

Filling compacted in accordance with the above recommendations may experience long-term creep settlements of 0.5% to 1% of the filling depth, due to the self-weight of the material. This is in addition to settlement caused by the application of loads on the surface of the filling.
Australian Standard AS3798-2007 Guidelines on earthworks for commercial and residential developments provides guidance as to appropriate testing frequencies for the testing of filling. It is recommended that all compaction control testing in areas that will support pavements be undertaken with a Level 1 responsibility.

8.5 Pavements

The laboratory testing undertaken indicated CBR values of 4.5% in the natural clay, 6% in re-worked, extremely weathered shale. CBRs between these values are suggested by testing of site filling.

It is therefore recommended that preliminary design of future pavements around the site be based on a CBR of 4% for pavements on the natural clay and 5% for pavements on the extremely weathered shale, allowing for potential variability of the soils in the area.

In any case, appropriate cross-fall and subsurface drainage should be installed for new pavement areas to reduce the risk of clayey subgrade becoming saturated during periods of wet weather. Good construction practice, such as installing subsurface drains around the perimeter of any garden or grassed areas, will also help to reduce the change of subgrade deterioration caused by excessive irrigation or stormwater run off.

8.6 Subsurface Aggressivity

Ground conditions at this site generally consist of low permeability soils over rock. Groundwater levels are within the rock at the towers site, and within the low permeability soil and/or rock at the car park site. Comparison of the results of laboratory testing to Tables 6.4.2(C) and 6.5.2(C) in AS2159 indicate that the existing soil conditions are nonaggressive to subsurface steel, and mildly aggressive (at the towers building) to moderately aggressive (at the car park site).

It is noted that the results of screening suggest that the acid sulphate soils, if oxidised, may lead to a soil pH in the order of 3 to 4.5. This would correspond to a mildly aggressive soil environment for steel piles, or a moderate to severe aggressivity. Consideration should be given to use of these more aggressive conditions, if oxidation of the soil may occur during the project, or over the life of the building (e.g. due to dewatering by new excavations or basements).

8.7 Seismic Site

Using the site sub-soil classes defined in AS1170.4-2007 Structural design action – Part 4: Earthquake actions in Australia, the two sites are currently considered to be Class Be – Rock.

Should the car park be raised by more than 0.5 m for the ground floor level, the car park site will then be a Class Ce – Shallow Soil site.
8.8 Further Investigation

The results of the current investigation indicate that high strength rock is present under much of the site. As discussed in Section 8.3, higher bearing capacities could potentially be justified if further investigation is undertaken to ‘prove’ the strength and consistency of the rock. Given the relatively high column loads advised, further investigation in areas of high loads is likely to be result in more efficient foundation design on this site.

9. Limitations

Douglas Partners (DP) has prepared this report for this project at Concord Hospital in accordance with DP’s proposal dated and acceptance received from Sydney Local Health District dated 11 February 2016. The work was carried out under DP’s Conditions of Engagement. This report is provided for the exclusive use of for this project only and for the purposes as described in the report. It should not be used for other projects or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP’s field testing has been completed.

DP’s advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.
The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the (geotechnical / environmental / groundwater) components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

References

2. G Wilson, I D McDonald, P S Roy and C Herbert, Sydney 1:100,000 Geology Sheet Edition 1, 1983, Geological Survey of NSW.
Appendix A

About this Report
Introduction
These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright
This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs
The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater
Where groundwater levels are measured in boreholes there are several potential problems, namely:

- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports
The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.
About this Report

Site Anomalies
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection
The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.
Appendix B

Drawings
NOTE:
1. Base image from Nearmap.com
2. Test locations are approximate only and were located using differential GPS

LEGEND
- Borehole location
- Geotechnical Cross Section A-A'

PROPOSED MULTI-STOReY CAR PARK

PHASE 1
BH1
BH2
BH3
BH4
BH5
BH6

PHASE 2

0 10 20 30 40 50 60 70 80 90 100m
1:1000 @ A3

TITLe: Test Locations - Proposed Multi-Storey Carpark
Concord Hospital
Hospital Road, Concord West
NOTE:
1. Base image from Nearmap.com
2. Test locations are approximate only and were located using differential GPS

LEGEND
- Borehole location
- Geotechnical Cross Section B-B'

CLIENT: Sydney Local Health District
OFFICE: Sydney
SCALE: 1:1000 @A3
DATE: 9.3.2016

TITLE: Test Locations - Proposed Tower Building
Concord Hospital
Hospital Road, Concord West
Appendix C

Site Photographs
Photo 1 – Existing car park area, facing south from the northern corner of the proposed car park

Photo 2 – Existing car park, local movement of the concrete kerb near the northern corner of the proposed car park
Photo 3 – Facing north-west towards Building 5 across the proposed tower building footprint. Note the typical one and two storey weatherboard and fibreboard buildings in the area.

Photo 4 – Facing north-west up "Hospital Street". Tennis courts are above the batter at the left hand side. Liquid oxygen storage, substation and Building 4 are present near the centre of the photo, behind the concrete retaining wall. Building 3 is visible behind the drilling rig, and Building 5 is at right.
Appendix D

Results of Field Work
Sampling
Sampling is carried out during drilling or test pitting to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits
Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers
Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers
The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling
The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling
A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests
Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:
  4, 6, 7
  N=13
- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:
  15, 30/40 mm
Sampling Methods

The results of the SPT tests can be related empirically to the engineering properties of the soils.

**Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests**

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil, the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- **Perth sand penetrometer** - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.

- **Cone penetrometer** - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.
**Description and Classification Methods**

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

**Soil Types**

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

<table>
<thead>
<tr>
<th>Type</th>
<th>Particle size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulder</td>
<td>&gt;200</td>
</tr>
<tr>
<td>Cobble</td>
<td>63 - 200</td>
</tr>
<tr>
<td>Gravel</td>
<td>2.36 - 63</td>
</tr>
<tr>
<td>Sand</td>
<td>0.075 - 2.36</td>
</tr>
<tr>
<td>Silt</td>
<td>0.002 - 0.075</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;0.002</td>
</tr>
</tbody>
</table>

The sand and gravel sizes can be further subdivided as follows:

<table>
<thead>
<tr>
<th>Type</th>
<th>Particle size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse gravel</td>
<td>20 - 63</td>
</tr>
<tr>
<td>Medium gravel</td>
<td>6 - 20</td>
</tr>
<tr>
<td>Fine gravel</td>
<td>2.36 - 6</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>0.6 - 2.36</td>
</tr>
<tr>
<td>Medium sand</td>
<td>0.2 - 0.6</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.075 - 0.2</td>
</tr>
</tbody>
</table>

The proportions of secondary constituents of soils are described as:

<table>
<thead>
<tr>
<th>Term</th>
<th>Proportion</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>And</td>
<td>Specify</td>
<td>Clay (60%) and Sand (40%)</td>
</tr>
<tr>
<td>Adjective</td>
<td>20 - 35%</td>
<td>Sandy Clay</td>
</tr>
<tr>
<td>Slightly</td>
<td>12 - 20%</td>
<td>Slightly Sandy Clay</td>
</tr>
<tr>
<td>With some</td>
<td>5 - 12%</td>
<td>Clay with some sand</td>
</tr>
<tr>
<td>With a trace of</td>
<td>0 - 5%</td>
<td>Clay with a trace of sand</td>
</tr>
</tbody>
</table>

Definitions of grading terms used are:
- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

**Cohesive Soils**

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

<table>
<thead>
<tr>
<th>Description</th>
<th>Abbreviation</th>
<th>Undrained shear strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
<td>vs</td>
<td>&lt;12</td>
</tr>
<tr>
<td>Soft</td>
<td>s</td>
<td>12 - 25</td>
</tr>
<tr>
<td>Firm</td>
<td>f</td>
<td>25 - 50</td>
</tr>
<tr>
<td>Stiff</td>
<td>st</td>
<td>50 - 100</td>
</tr>
<tr>
<td>Very stiff</td>
<td>vst</td>
<td>100 - 200</td>
</tr>
<tr>
<td>Hard</td>
<td>h</td>
<td>&gt;200</td>
</tr>
</tbody>
</table>

**Cohesionless Soils**

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>Abbreviation</th>
<th>SPT N value</th>
<th>CPT qc value (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>vl</td>
<td>&lt;4</td>
<td>&lt;2</td>
</tr>
<tr>
<td>Loose</td>
<td>l</td>
<td>4 - 10</td>
<td>2 - 5</td>
</tr>
<tr>
<td>Medium dense</td>
<td>md</td>
<td>10 - 30</td>
<td>5 - 15</td>
</tr>
<tr>
<td>Dense</td>
<td>d</td>
<td>30 - 50</td>
<td>15 - 25</td>
</tr>
<tr>
<td>Very dense</td>
<td>vd</td>
<td>&gt;50</td>
<td>&gt;25</td>
</tr>
</tbody>
</table>
Soil Origin
It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.
**Rock Descriptions**

**Rock Strength**
Rock strength is defined by the Point Load Strength Index ($I_s(50)$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

<table>
<thead>
<tr>
<th>Term</th>
<th>Abbreviation</th>
<th>Point Load Index</th>
<th>Approx Unconfined Compressive Strength MPa*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely low</td>
<td>EL</td>
<td>&lt;0.03</td>
<td>&lt;0.6</td>
</tr>
<tr>
<td>Very low</td>
<td>VL</td>
<td>0.03 - 0.1</td>
<td>0.6 - 2</td>
</tr>
<tr>
<td>Low</td>
<td>L</td>
<td>0.1 - 0.3</td>
<td>2 - 6</td>
</tr>
<tr>
<td>Medium</td>
<td>M</td>
<td>0.3 - 1.0</td>
<td>6 - 20</td>
</tr>
<tr>
<td>High</td>
<td>H</td>
<td>1 - 3</td>
<td>20 - 60</td>
</tr>
<tr>
<td>Very high</td>
<td>VH</td>
<td>3 - 10</td>
<td>60 - 200</td>
</tr>
<tr>
<td>Extremely high</td>
<td>EH</td>
<td>&gt;10</td>
<td>&gt;200</td>
</tr>
</tbody>
</table>

* Assumes a ratio of 20:1 for UCS to $I_s(50)$

**Degree of Weathering**
The degree of weathering of rock is classified as follows:

<table>
<thead>
<tr>
<th>Term</th>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely weathered</td>
<td>EW</td>
<td>Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.</td>
</tr>
<tr>
<td>Highly weathered</td>
<td>HW</td>
<td>Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable</td>
</tr>
<tr>
<td>Moderately weathered</td>
<td>MW</td>
<td>Staining and discolouration of rock substance has taken place</td>
</tr>
<tr>
<td>Slightly weathered</td>
<td>SW</td>
<td>Rock substance is slightly discoloured but shows little or no change of strength from fresh rock</td>
</tr>
<tr>
<td>Fresh stained</td>
<td>Fs</td>
<td>Rock substance unaffected by weathering but staining visible along defects</td>
</tr>
<tr>
<td>Fresh</td>
<td>Fr</td>
<td>No signs of decomposition or staining</td>
</tr>
</tbody>
</table>

**Degree of Fracturing**
The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fragmented</td>
<td>Fragments of &lt;20 mm</td>
</tr>
<tr>
<td>Highly Fractured</td>
<td>Core lengths of 20-40 mm with some fragments</td>
</tr>
<tr>
<td>Fractured</td>
<td>Core lengths of 40-200 mm with some shorter and longer sections</td>
</tr>
<tr>
<td>Slightly Fractured</td>
<td>Core lengths of 200-1000 mm with some shorter and loner sections</td>
</tr>
<tr>
<td>Unbroken</td>
<td>Core lengths mostly &gt; 1000 mm</td>
</tr>
</tbody>
</table>
Rock Descriptions

Rock Quality Designation
The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

\[
\text{RQD} \% = \frac{\text{cumulative length of 'sound' core sections } \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}
\]

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing
For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

<table>
<thead>
<tr>
<th>Term</th>
<th>Separation of Stratification Planes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thinly laminated</td>
<td>&lt; 6 mm</td>
</tr>
<tr>
<td>Laminated</td>
<td>6 mm to 20 mm</td>
</tr>
<tr>
<td>Very thinly bedded</td>
<td>20 mm to 60 mm</td>
</tr>
<tr>
<td>Thinly bedded</td>
<td>60 mm to 0.2 m</td>
</tr>
<tr>
<td>Medium bedded</td>
<td>0.2 m to 0.6 m</td>
</tr>
<tr>
<td>Thickly bedded</td>
<td>0.6 m to 2 m</td>
</tr>
<tr>
<td>Very thickly bedded</td>
<td>&gt; 2 m</td>
</tr>
</tbody>
</table>
## Symbols & Abbreviations

### Introduction
These notes summarise abbreviations commonly used on borehole logs and test pit reports.

### Drilling or Excavation Methods
- **C**: Core Drilling
- **R**: Rotary drilling
- **SFA**: Spiral flight augers
- **NMLC**: Diamond core - 52 mm dia
- **NQ**: Diamond core - 47 mm dia
- **HQ**: Diamond core - 63 mm dia
- **PQ**: Diamond core - 81 mm dia

### Water
- **Z**: Water seep
- **V**: Water level

### Sampling and Testing
- **A**: Auger sample
- **B**: Bulk sample
- **D**: Disturbed sample
- **E**: Environmental sample
- **U50**: Undisturbed tube sample (50mm)
- **W**: Water sample
- **pp**: Pocket penetrometer (kPa)
- **PID**: Photo ionisation detector
- **PL**: Point load strength IS(50) MPa
- **S**: Standard Penetration Test
- **V**: Shear vane (kPa)

### Description of Defects in Rock
The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

#### Defect Type
- **B**: Bedding plane
- **Cs**: Clay seam
- **Cv**: Cleavage
- **Cz**: Crushed zone
- **Ds**: Decomposed seam
- **F**: Fault
- **J**: Joint
- **Lam**: Lamination
- **Pt**: Parting
- **Sz**: Sheared Zone
- **V**: Vein

#### Orientation
The inclination of defects is always measured from the perpendicular to the core axis.

<table>
<thead>
<tr>
<th>Orientation</th>
<th>Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>horizontal</td>
<td>h</td>
</tr>
<tr>
<td>vertical</td>
<td>v</td>
</tr>
<tr>
<td>sub-horizontal</td>
<td>sh</td>
</tr>
<tr>
<td>sub-vertical</td>
<td>sv</td>
</tr>
</tbody>
</table>

#### Coating or Infilling Term
- **cln**: clean
- **co**: coating
- **he**: healed
- **inf**: infilled
- **stn**: stained
- **ti**: tight
- **vn**: veneer

#### Coating Descriptor
- **ca**: calcite
- **cbs**: carbonaceous
- **cly**: clay
- **fe**: iron oxide
- **mn**: manganese
- **slt**: silty

#### Shape
- **cu**: curved
- **ir**: irregular
- **pl**: planar
- **st**: stepped
- **un**: undulating

#### Roughness
- **po**: polished
- **ro**: rough
- **sl**: slickensided
- **sm**: smooth
- **vr**: very rough

#### Other
- **fg**: fragmented
- **bnd**: band
- **qtz**: quartz
### Symbols & Abbreviations

#### Graphic Symbols for Soil and Rock

<table>
<thead>
<tr>
<th>General</th>
<th>Sedimentary Rocks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt</td>
<td>Boulder conglomerate</td>
</tr>
<tr>
<td>Road base</td>
<td>Conglomerate</td>
</tr>
<tr>
<td>Concrete</td>
<td>Conglomeratic sandstone</td>
</tr>
<tr>
<td>Filling</td>
<td>Sandstone</td>
</tr>
<tr>
<td>Soils</td>
<td>Siltstone</td>
</tr>
<tr>
<td>Topsoil</td>
<td>Laminate</td>
</tr>
<tr>
<td>Peat</td>
<td>Mudstone, claystone, shale</td>
</tr>
<tr>
<td>Clay</td>
<td>Coal</td>
</tr>
<tr>
<td>Silty clay</td>
<td>Limestone</td>
</tr>
<tr>
<td>Sandy clay</td>
<td></td>
</tr>
<tr>
<td>Gravelly clay</td>
<td></td>
</tr>
<tr>
<td>Shaly clay</td>
<td></td>
</tr>
<tr>
<td>Silt</td>
<td></td>
</tr>
<tr>
<td>Clayey silt</td>
<td></td>
</tr>
<tr>
<td>Sandy silt</td>
<td></td>
</tr>
<tr>
<td>Clayey sand</td>
<td></td>
</tr>
<tr>
<td>Silty sand</td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td></td>
</tr>
<tr>
<td>Sandy gravel</td>
<td></td>
</tr>
<tr>
<td>Cobbles, boulders</td>
<td></td>
</tr>
<tr>
<td>Talus</td>
<td></td>
</tr>
</tbody>
</table>

#### Sedimentary Rocks
- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminate
- Mudstone, claystone, shale
- Coal
- Limestone

#### Metamorphic Rocks
- Slate, phyllite, schist
- Gneiss
- Quartzite

#### Igneous Rocks
- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry

---

July 2010
2.5 – 6.0m
### Description of Strata

- **0.04 m:** Asphaltic concrete
- **0.3 m:** Filling - dark grey, gravelly sand filling with some silt
- **0.6 m:** Clay - stiff, brown clay with a trace of ironstone gravel
- **1 m:** Clay - stiff, grey mottled brown clay with a trace of ironstone gravel, damp
- **2.0 m:** Becoming shaly clay with some ironstone gravel
- **2.5 m:** Shale - extremely low to very low strength, extremely to highly weathered, fractured, light grey-brown, shale with medium and high strength iron-cemented bands
- **4.72 m:** Sandstone - low to medium then medium strength, moderately weathered, fractured, brown medium grained sandstone with approximately 25% siltstone laminae

### Degree of Weathering

- **Uniformly Weathered (UW)**
- **Uniformly Strong (US)**
- **Uniformly Weak (UW)**
- **Uniformly Strong (US)**
- **Uniformly Weak (UW)**
- **None**

### Rock Strength

- **J - Joint**
- **F - Fault**
- **S - Shear**

### Fracture Spacing (m)

- **0.01**
- **0.05**
- **0.10**
- **0.50**
- **1.00**

### Discontinuities

- **B - Bedding**
- **J - Joint**
- **S - Shear**
- **F - Fault**

### Sampling & In Situ Testing

- **Type**
- **Core Rec. %**
- **RQD %**
- **Comments**

### Test Results

- **PL(D) Point load diametral test Is(50) (MPa)**
- **pp Pocket penetrometer (kPa)**
- **SLA (Self-weighted load average) (kPa)**
- **PID Photo ionisation detector (ppm)**
- **Ipt load axial test Is(50) (MPa)**
- **Pocket penetrometer (kPa)**
- **Standard penetration test**
- **Shear vane (kPa)**

### Remarks

- Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°

- **Note:**
  - 2.9m: J70°, pl, ro, cly
  - 3.22-3.26m: Cs
  - 3.53m: J35°, pl, sm, cly
  - 3.88m: B0°, fe
  - 4.2m: J25°, pl, ro, cly
  - 4.46-4.52m: Cs
  - 4.57m: J30°, un, ro, fe
  - 4.69-5.12m: fg, fe
  - 5.0-5.12m: fg, fe
  - 5.5m: J45°, un, ro, fe
  - 5.86m: J30° & 70°, st, ro, cly

- **Bore discontinued at 6.0m**

---

### Borehole Log

- **Client:** NSW Health
- **Project:** Concord Hospital
- **Location:** Hospital Road, Concord West
- **Surface Level:** 4.5 AHD
- **Easting:** 323490
- **Northing:** 6254401
- **Date:** 2/3/2016
- **Rig:** Scout 2
- **Water Observations:** No free groundwater observed whilst augering
- **Remarks:** BD1-010316

---

### Sampling & In Situ Testing Legend

- **A:** Auger sample
- **B:** Bulk sample
- **BLK:** Block sample
- **C:** Core drilling
- **D:** Disturbed sample
- **E:** Environmental sample
- **G:** Gas sample
- **P:** Piston sample
- **U:** Tube sample (x mm dia.)
- **W:** Water sample
- **X:** Water level
- **Y:** Y-sieve sample
- **Z:** Z-sieve sample
2.5 – 6.0m
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength Type</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>ASPHALTIC CONCRETE</td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>0.05</td>
<td>E</td>
<td>PID&lt;1</td>
</tr>
<tr>
<td>0.10</td>
<td>FILLING - dark grey, sand filling with some basaltic gravel</td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>0.15</td>
<td>E</td>
<td>PID&lt;3</td>
</tr>
<tr>
<td>0.7</td>
<td>SILTY CLAY - brown, silty clay with a trace of ironstone gravel, moist</td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>0.70</td>
<td>E</td>
<td>PID&gt;2.5</td>
</tr>
<tr>
<td>1.5</td>
<td>CLAY - stiff, brown-orange clay with some silt, moist</td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>1.50</td>
<td>E</td>
<td>23.5,6</td>
</tr>
<tr>
<td>2.0</td>
<td>CLAY - stiff, brown-grey, clay with some ironstone gravel, humid</td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>2.00</td>
<td>E</td>
<td>N = 11</td>
</tr>
<tr>
<td>2.5</td>
<td>SHALE - extremely low then extremely low to very low strength, extremely to highly weathered, fractured, light grey-brown shale with some medium to high strength ironstone bands</td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>2.50</td>
<td>E</td>
<td>PID&gt;3.2</td>
</tr>
<tr>
<td>3.5</td>
<td>SANDSTONE - medium then high strength, moderately weathered, slightly fractured, brown, medium grained sandstone</td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>3.50</td>
<td>E</td>
<td>PL(A) = 1</td>
</tr>
<tr>
<td>4.0</td>
<td></td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>4.00</td>
<td>E</td>
<td>PL(A) = 0.5</td>
</tr>
<tr>
<td>4.57</td>
<td></td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>4.57</td>
<td>E</td>
<td>PL(A) = 1.2</td>
</tr>
<tr>
<td>5.8</td>
<td></td>
<td>Ex Low</td>
<td>J - Joint</td>
<td>5.80</td>
<td>E</td>
<td></td>
</tr>
</tbody>
</table>

Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°-10°.
2.5 – 6.0m
### Borehole Log

**Description of Strata**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00-0.15</td>
<td>ASPHALTIC CONCRETE, FILLING - sandy gravel filling</td>
</tr>
<tr>
<td>0.15-0.80</td>
<td>SILTY CLAY - firm to stiff, brown mottled red-orange, silty clay, moist</td>
</tr>
<tr>
<td>0.80-2.00</td>
<td>2.0m: becoming shaly clay with some ironstone gravel</td>
</tr>
<tr>
<td>2.00-2.70</td>
<td>SANDSTONE/SHALE - extremely low to very low strength, extremely to highly weathered, fractured, light grey-brown, fine grained sandstone/shale with high strength ironstone bands</td>
</tr>
<tr>
<td>2.70-4.00</td>
<td>SANDSTONE - high strength, moderately then slightly weathered, slightly fractured, brown and light grey, fine to medium grained sandstone</td>
</tr>
<tr>
<td>4.00-6.00</td>
<td>Bore discontinued at 6.0m</td>
</tr>
<tr>
<td>6.00-7.00</td>
<td></td>
</tr>
<tr>
<td>7.00-8.00</td>
<td></td>
</tr>
<tr>
<td>8.00-9.00</td>
<td></td>
</tr>
</tbody>
</table>

**Degree of Weathering**

- **EW**: Ex Low
- **HW**: Very Low
- **MW**: Low
- **SW**: Medium
- **FS**: High
- **FR**: Very High

**Rock Strength**

- **Ex Low**
- **Very Low**
- **Low**
- **Medium**
- **High**
- **Very High**

**Fracture Spacing (m)**

- **0.01**
- **0.10**
- **0.50**
- **1.00**

**Discontinuities**

- **B**: Bedding
- **J**: Joint
- **S**: Shear
- **F**: Fault

**Sampling & In Situ Testing**

- **PID**: Photo ionisation detector (ppm)
- **PL(A)**: Point load axial test (50kN) (MPa)
- **PL(D)**: Point load diametral test (50kN) (MPa)
- **Pocket penetrometer (kPa)**
- **Standard penetration test**
- **Shear vane (kPa)**

**Test Results & Comments**

- **0.05**
- **0.10**
- **0.50**
- **1.00**

---

**Sampling & In Situ Testing Legend**

- **A**: Auger sample
- **B**: Bulk sample
- **BLK**: Block sample
- **C**: Core drilling
- **D**: Disturbed sample
- **E**: Environmental sample
- **G**: Gas sample
- **P**: Piston sample
- **U**: Tube sample (x mm dia.)
- **W**: Water sample
- **S**: Standard penetration test
- **V**: Shear vane (kPa)

**Additional Information**

- **Surface Level**: 4.5 AHD
- **Easting**: 323546
- **Northing**: 6254439
- **Date**: 1/3/2016
- **RIG**: Scout 2
- **Driller**: WG
- **Logged**: AT/SI
- **Casing**: HW to 2.5m
- **Remarks**: No free groundwater observed whilst augering

---

**Notes**:

- Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°-10°

---

**Water Observations**:

- No free groundwater observed whilst augering

---

**Client**: NSW Health

**Project**: Concord Hospital

**Location**: Hospital Road, Concord West

---

**Other Information**

- **Core Loss**: 200mm
- **Joints**: relict, 40°-45°, pl, sm, cly
- **Base**: 40°, pl, ro, fe
- **3.55m**: J30°, pl, ro, cln
- **5.8 & 5.85m**: Cs
- **PL(A)** values:
  - 2.1
  - 1.5
  - 1.2
  - 2.1
- **Notes**: Bore discontinued at 6.0m
Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00-0.06</td>
<td>ASPHALTIC CONCRETE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.3</td>
<td>FILLING - dark grey, gravelly sand with some slag</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.8</td>
<td>CLAY - apparently firm to stiff, dark brown-brown clay with trace of sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>CLAY - stiff to very stiff, grey mottled orange-red, clay with trace of ironstone gravel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>2.0m: becoming shaly clay with some ironstone gravel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>SHALE - extremely low strength, extremely to highly weathered, fractured, light grey and red-brown shale with some high strength, iron-cemented bands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2.85m: J30°, pl, ro, fe, cly</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>3.0m: J70°, un, ro, cly</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.55-3.57</td>
<td>3.5m: J30°, un, ro, fe, cly</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.64</td>
<td>3.55: J35°, un, ro, fe, cly</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.77</td>
<td>3.66: J35°, pl, sm, cly</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.45</td>
<td>SHALE - very low then low strength, highly weathered, fractured, light grey-brown shale with low strength bands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.53</td>
<td>4.35: J80°, un, ro, fe, cly</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.68</td>
<td>4.53: J35°, pl, sm, cly</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.32</td>
<td>5.32: J45°, pl, ro, fe</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.6-5.75</td>
<td>5.32: J45°-5.53m: fg</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.08</td>
<td>6.08m: J30°, he, fe</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.38</td>
<td>6.38m: J35°, pl, sm, fe</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.6-6.86</td>
<td>6.6-6.86m: B (x10) 0°-5°, fe</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.0</td>
<td>Bore discontinued at 7.0m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sampling & In Situ Testing Legend:
- A: Auger sample
- B: Bulk sample
- BLK: Block sample
- C: Core drilling
- D: Disturbed sample
- E: Environmental sample
- G: Gas sample
- P: Piston sample
- U: Tube sample (mm dia.)
- W: Water sample
- Y: Water level
- PL: Point load axial test (kPa)
P: Point load diametral test (kPa)
- D: Pocket penetrometer (kPa)
- V: Shear vane (kPa)
2.5 – 6.4m
Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°-10°.

- Core Loss: 380mm
- Fracture Spacing: 2.5m
- Type: A
- Test Results: pp = 450
- Comments: 1, 5, 5

Depth (m) | Description of Strata | Degree of Weathering | Rock Strength | Fracture Spacing (m) | Discontinuities | Sampling & In Situ Testing
--- | --- | --- | --- | --- | --- | ---
0.00 | ASPHALTIC CONCRETE | \( B \) | \( J \) | 0.01 | 2.5 | CORE LOSS: 380mm
0.14 | FILLING - dark grey, gravelly sand filling with trace silt and clay | \( U \) | \( F \) | 0.10 | 0.6 | 
0.7 | SILTY CLAY - dark grey, silty clay,\( J \) (relict), un, ro, cly | \( U \) | \( F \) | 0.50 | 1.0 | 

<table>
<thead>
<tr>
<th>Rock</th>
<th>Sampling &amp; In Situ Testing</th>
<th>Type</th>
<th>Core Rec. %</th>
<th>Test Results &amp; Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Surface Level: 6.4 AHD
EASTING: 323535
NORTHING: 6254392
DATE: 2/3/2016

RIG: Scout 2
DRILLER: WG
LOGGED: AT/SI
CASING: HW to 2.5m

TYPE OF BORING: Solid flight auger to 2.5m; NMLC-Coring to 6.45m
WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:
DOUGLAS PARTNERS PTY LTD
CONCORD HOSPITAL CARPARK – CONCORD WEST
BORE 6       PROJECT  85356       MAR  2016

2.5 – 6.0m
<table>
<thead>
<tr>
<th>ID</th>
<th>Depth (m)</th>
<th>Description of Strata</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>0.01</td>
<td>ASPHALTIC CONCRETE</td>
</tr>
<tr>
<td>0.07</td>
<td>0.7</td>
<td>CLAY - apparently firm to stiff, brown clay with a trace of ironstone</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>CLAY - very stiff, brown-red clay with some ironstone gravel</td>
</tr>
<tr>
<td>2.0</td>
<td>2.0</td>
<td>becoming shaly clay with some ironstone gravel</td>
</tr>
<tr>
<td>2.5</td>
<td>2.5</td>
<td>SHALE - extremely low to very low strength, extremely to highly weathered, fractured, light grey and red-brown, shale with some medium strength iron-cemented bands</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>SHALE - low and medium strength, highly to moderately weathered, fragmented to fractured, dark grey-brown shale</td>
</tr>
<tr>
<td>3.6</td>
<td>3.6</td>
<td>SHALE - extremely low to very low strength, extremely to highly weathered, fractured, light grey and red-brown, shale with some medium strength iron-cemented bands</td>
</tr>
<tr>
<td>4.55</td>
<td>4.55</td>
<td>SHALE - low and medium strength, highly to moderately weathered, fragmented to fractured, dark grey-brown shale</td>
</tr>
<tr>
<td>6.0</td>
<td>6.0</td>
<td>Bore discontinued at 6.0m</td>
</tr>
</tbody>
</table>

**Discontinuities**

- B - Bedding
- J - Joint
- S - Shear
- F - Fault

**Sampling & In Situ Testing**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Type</th>
<th>Core Rec. %</th>
<th>RQD %</th>
<th>Test Results &amp; Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.55m</td>
<td>J55°</td>
<td>un, ro, fe</td>
<td></td>
<td>PID=3.5</td>
</tr>
<tr>
<td>2.6-2.25m</td>
<td>J's 0°- 5°, fe, cly</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.25m</td>
<td>CORE LOSS: 350 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.53m</td>
<td>J45°</td>
<td>pl, sm, cly</td>
<td></td>
<td>PID=3.3</td>
</tr>
<tr>
<td>4.62m</td>
<td>J, sv, pl, ro, fe</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.72-5.1m</td>
<td>fg, fe</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.18m</td>
<td>J45°</td>
<td>pl, ro, fe</td>
<td></td>
<td>PID=3.3</td>
</tr>
<tr>
<td>5.28m</td>
<td>J (x2) 35°, pl, ro, fe</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.43m</td>
<td>J45°</td>
<td>pl, ro, fe</td>
<td></td>
<td>PID=3.3</td>
</tr>
<tr>
<td>5.59m</td>
<td>CORE LOSS: 100 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note:** Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°

**Description of Strata**

- S - Clay - apparently firm to stiff, brown clay with a trace of ironstone
- C - Clay - very stiff, brown-red clay with some ironstone gravel
- W - Clay - low and medium strength, highly to moderately weathered, fragmented to fragmented, dark grey-brown shale
- H - SHALE - extremely low to very low strength, extremely to highly weathered, fractured, light grey and red-brown, shale with some medium strength iron-cemented bands
- R - SHALE - low and medium strength, highly to moderately weathered, fragmented to fractured, dark grey-brown shale

**Sampling & In Situ Testing Legend**

- A Auger sample
- B Bulk sample
- BLK Block sample
- C Core drilling
- D Disturbed sample
- E Environmental sample
- G Gas sample
- P Piston sample
- U Tube sample (x mm dia.)
- W Water sample
- V Shear vane (kPa)
- PP Pocket penetrometer (kPa)
- PL(D) Point load diametral test (50) (MPa)
- PL(A) Point load axial test (50) (MPa)
- PID Photo ionisation detector (ppm)
- S Standard penetration test

**Remarks:**

- Sampling & In Situ Testing
- Type of boring: Solid flight auger to 2.5m; NMLC-Coring to 6.0m
- Water observations: No free groundwater observed whilst augering
- Remarks:
1.0 – 5.0m

5.0 – 6.1m
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength Type</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>ASPHALTIC CONCRETE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.3</td>
<td>FILLING - grey, sand filling with some gravel and silt (crushed sandstone)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>SHALE - very low and low strength, light grey shale</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>SHALE - medium strength, highly to moderately then moderately weathered, highly fractured to fractured, grey-brown, shale with approximately 10% fine sandstone laminations and some clay bands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.4</td>
<td>SHALE - medium then high strength, fresh stained, fractured, grey to grey-brown shale</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.1</td>
<td>Bore discontinued at 6.1m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°-10°.

B - Bedding J - Joint
S - Shear F - Fault

Test Results & Comments:

- PL(A) = 0.7
- PL(A) = 0.6
- PL(A) = 0.8
- PL(A) = 0.9
- PL(A) = 0.8
- PL(A) = 1.2

Surface Level: 8.2 AHD
Easting: 323548
Nordling: 6254198
Date: 25/2/2016
Project No: 85356.00
Bore No: 10

Remarks:

- No free groundwater observed whilst augering
**BOREHOLE LOG**

**CLIENT:** NSW Health  
**PROJECT:** Concord Hospital  
**LOCATION:** Hospital Road, Concord West  
**SURFACE LEVEL:** 12.1 AHD  
**EASTING:** 323517  
**NORTHING:** 6254169  
**DATE:** 22/2/2016

**FILLING - dark brown, organic clayey silt filling with fine to medium sized sand and rootlets (topsoil)  
CLAY - stiff to very stiff, brown-grey clay with some shaly clay layers, moist  
LAMINITE - extremely low strength, brown-grey laminite  
LAMINITE - medium and low to medium strength, highly weathered, fragmented and highly fractured, light grey-brown, laminite with some very low to low strength bands  
LAMINITE - medium strength, moderately weathered, fragmented to fractured, grey-brown laminite with approximately 20% fine sandstone laminations  
SHALE - high strength, fresh, fractured to slightly fractured, grey shale  

**Discontinuities**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Fracturing Spacing (m)</th>
<th>Discontinuities</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>FILLING - dark brown, organic clayey silt filling with fine to medium sized sand and rootlets (topsoil)</td>
<td>RAL</td>
<td>B - Bedding</td>
<td>PID &lt; 1</td>
</tr>
<tr>
<td>0.3</td>
<td>FILLING - dark brown, organic clayey silt with trace gravel</td>
<td>RAL</td>
<td>S - Shear</td>
<td>PID &lt; 1</td>
</tr>
<tr>
<td>1.4</td>
<td>CLAY - stiff to very stiff, brown-grey clay with some shaly clay layers, moist</td>
<td>HW</td>
<td>F - Fault</td>
<td>S</td>
</tr>
<tr>
<td>2.6</td>
<td>LAMINITE - extremely low strength, brown-grey laminite</td>
<td>HW</td>
<td>B - Bedding</td>
<td>PID &lt; 1</td>
</tr>
<tr>
<td>3.3</td>
<td>LAMINITE - medium and low to medium strength, highly weathered, fragmented and highly fractured, light grey-brown, laminite with some very low to low strength bands</td>
<td>HW</td>
<td>S - Shear</td>
<td>2.2, 3</td>
</tr>
<tr>
<td>4.15</td>
<td>LAMINITE - medium strength, moderately weathered, fragmented to fractured, grey-brown laminite with approximately 20% fine sandstone laminations</td>
<td>HW</td>
<td>F - Fault</td>
<td>N = 5</td>
</tr>
<tr>
<td>5.2</td>
<td>LAMINITE - high strength, fresh, fractured to slightly fractured, grey shale</td>
<td>MW</td>
<td>J - Joint</td>
<td>PID &lt;/ 1</td>
</tr>
</tbody>
</table>

**Sampling & In Situ Testing**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Fracturing Spacing (m)</th>
<th>Discontinuities</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>FILLING - dark brown, organic clayey silt filling with fine to medium sized sand and rootlets (topsoil)</td>
<td>RAL</td>
<td>B - Bedding</td>
<td>PID &lt; 1</td>
</tr>
<tr>
<td>0.3</td>
<td>FILLING - dark brown, organic clayey silt with trace gravel</td>
<td>RAL</td>
<td>S - Shear</td>
<td>PID &lt; 1</td>
</tr>
<tr>
<td>1.4</td>
<td>CLAY - stiff to very stiff, brown-grey clay with some shaly clay layers, moist</td>
<td>HW</td>
<td>F - Fault</td>
<td>S</td>
</tr>
<tr>
<td>2.6</td>
<td>LAMINITE - extremely low strength, brown-grey laminite</td>
<td>HW</td>
<td>B - Bedding</td>
<td>PID &lt; 1</td>
</tr>
<tr>
<td>3.3</td>
<td>LAMINITE - medium and low to medium strength, highly weathered, fragmented and highly fractured, light grey-brown, laminite with some very low to low strength bands</td>
<td>HW</td>
<td>S - Shear</td>
<td>2.2, 3</td>
</tr>
<tr>
<td>4.15</td>
<td>LAMINITE - medium strength, moderately weathered, fragmented to fractured, grey-brown laminite with approximately 20% fine sandstone laminations</td>
<td>HW</td>
<td>F - Fault</td>
<td>N = 5</td>
</tr>
<tr>
<td>5.2</td>
<td>LAMINITE - medium strength, moderately weathered, fragmented to fractured, grey-brown laminite with approximately 20% fine sandstone laminations</td>
<td>HW</td>
<td>J - Joint</td>
<td>PID &lt;/ 1</td>
</tr>
<tr>
<td>7.6</td>
<td>SHALE - high strength, fresh, fractured to slightly fractured, grey shale</td>
<td>MW</td>
<td>J - Joint</td>
<td>PID &lt; 1</td>
</tr>
</tbody>
</table>

**Test Results & Comments**

Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0° - 10°

**samplIng & in situ testing Legend**

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>Auger sample</td>
<td>Bulk sample</td>
<td>Core sample</td>
<td>Disturbed sample</td>
<td>Environmental sample</td>
<td>Gas sample</td>
<td>Piston sample</td>
<td>Water sample</td>
</tr>
</tbody>
</table>

---

**Type of Boring:** Solid flight auger to 2.5m; Rotary to 3.3m; NMLC-Coring to 12.0m  
**Water Observations:** No free groundwater observed whilst augering

**Remarks:**

---

**Douglas Partners**

Geotechnics | Environment | Groundwater
**BOREHOLE LOG**

**CLIENT:** NSW Health  
**PROJECT:** Concord Hospital  
**LOCATION:** Hospital Road, Concord West  
**SURFACE LEVEL:** 12.1 AHD  
**BORE No:** 11  
**EASTING:** 323517  
**PROJECT No:** 85356.00  
**NORTHING:** 6254169  
**DATE:** 22/2/2016  
**DIP/AZIMUTH:** 90°/--  

### Sampling & In Situ Testing Legend

- **A** Auger sample  
- **B** Bulk sample  
- **BLK** Block sample  
- **C** Core drilling  
- **D** Disturbed sample  
- **E** Environmental sample  
- **G** Gas sample  
- **P** Piston sample  
- **PL** Pocket penetrometer (kPa)  
- **S** Shear vane (kPa)  
- **U** Tube sample (x mm dia.)  
- **W** Water sample  
- **X** Water seep  
- **Y** Water level  
- **Z** Water seep  
- **PID** Photo ionisation detector (ppm)  
- **PL(A)** Point load axial test (50) (MPa)  
- **PL(D)** Point load diametral test (50) (MPa)  

### Table

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.5</td>
<td>SHALE - high strength, fresh, fractured to slightly fractured, grey shale (continued)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>SHALE - medium strength, fresh, slightly fractured, grey shale</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.1</td>
<td>Bore discontinued at 12.1m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
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</tr>
<tr>
<td>15</td>
<td></td>
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<tr>
<td>16</td>
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</tr>
<tr>
<td>17</td>
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<tr>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**RIG:** Scout 2  
**DRILLER:** WG  
**LOGGED:** AT/SI  
**CASING:** HW to 2.5m  
**TYPE OF BORING:** Solid flight auger to 2.5m; Rotary to 3.3m; NMLC-Coring to 12.0m  
**WATER OBSERVATIONS:** No free groundwater observed whilst augering  
**REMARKS:**  

---

**BRICK - high strength, fresh, fractured to slightly fractured, grey shale (continued)**
DOUGLAS PARTNERS PTY LTD
CONCORD HOSPITAL CARPARK – CONCORD WEST
BORE 12         PROJECT 85356        FEB 2016

10.0 – 12.0m
<table>
<thead>
<tr>
<th>ID</th>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>10.25-11.38</td>
<td>SANDSTONE - high strength, fresh, unbroken, light grey, fine to medium grained sandstone with some siltstone beds (continued)</td>
<td>Ex Low</td>
<td>0.01</td>
<td></td>
<td></td>
<td>PL(A) = 2.2</td>
</tr>
<tr>
<td>11</td>
<td>12</td>
<td>Bore discontinued at 12.0m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>12.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>PL(A) = 2</td>
</tr>
</tbody>
</table>

**SAMPLING & IN SITU TESTING LEGEND**

A Auger sample  C Gas sample  PID Photo ionisation detector (ppm)
B Bulk sample  D Piston sample  PL(A) Point load axial test (50) (MPa)
BLK Block sample  PL(D) Point load diametral test (50) (MPa)
C Core drilling  W Tube sample (x mm dia.)  pp Pocket penetrometer (kPa)
D Disturbed sample  Water sample  gs Standard penetration test
E Environmental sample  Water level  V Shear vane (kPa)
Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°-10°.

1.0-1.65m: fg

1.8 & 1.9m: J20°, un, ro, cln

2.0-2.1m: J55°-60°, un, ro, fe

2.1m: CORE LOSS: 170mm

2.6m: B10°, cly

2.8m: B0°, fe

2.86m: J35°, pl, ro, fe

3.1m: J25°, pl, ro, fe

3.2m: B0°, fe

Possible void from 3.9m to 5.2m

Bore discontinued at 5.2m
Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°.

- CORE LOSS: 220mm
- 1.36m: J35°, pl, ro, fe
- 1.9m: B5°, fe
- 2.06, 2.24 & 2.46m: J (x3) 55°- 65°, un, ro, cly
- 2.65m: J50°, un, ro, cly
- 2.76-2.8m: Cs
- 3.15-5.0m: fg, he, cly
- 3.6, 3.8 & 4.35m: J (x3), 70°- 80°, he, cly
- 5.5m: J45°, un, ro, fe
- 5.68m: B0°, fe, cly
- 5.9m: B10°, fe
- 6.05m: J80°, un, ro, fe
- 6.35-6.65m: B (x3) 0°, fe
- 7.7-8.6m: J85°- 90°, un, ro, cln
- 9.12 & 9.6m: B0°, cbs co

Discontinuities:
- B - Bedding
- J - Joint
- S - Shear
- F - Fault

Sampling & In Situ Testing:
- Core Rec. %
- ROD %
- Test Results & Comments

PL(A) = 1.4
PL(A) = 0.8
PL(A) = 0.2
PL(A) = 1.3
PL(A) = 0.9
PL(A) = 2
PL(A) = 3.6
PL(A) = 2.5
PL(A) = 2.2

Description of Strata:
- ASPHALTIC CONCRETE
- FILLING - gravelly sand filling with trace silt
- CLAY - apparently stiff, red-brown clay, moist
- SHALE - extremely low strength, light grey shale
- SHALE - very low strength, highly weathered, fractured, light grey and brown, with some medium and high strength iron cemented bands
- SHALE - alternate bands of very low and low strength, highly to moderately weathered, fragmented to fractured, grey to grey-brown shale
- LAMINITE - medium and high strength, moderately weathered then fresh, slightly fractured, brown then light grey to grey, laminite with approximately 20% fine grained sandstone laminations
- SANDSTONE - very high then high strength, fresh, slightly fractured and unbroken, light grey, fine to medium grained sandstone with some siltstone laminations and beds

Water Level: 5.9 AHD
Easting: 323613
Northing: 6254199
Dip/Azimuth: 90°/--

Remarks:
- No free groundwater observed whilst augering

Type of Boring:
- Solid flight auger to 1.0m; NMLC-Coring to 12.0m
10.0 – 12.0m
## BOREHOLE LOG

**CLIENT:** NSW Health  
**PROJECT:** Concord Hospital  
**LOCATION:** Hospital Road, Concord West  
**SURFACE LEVEL:** 5.9 AHD  
**EASTING:** 323613  
**NORTHING:** 6254199  
**PROJECT No:** 85356.00  
**DATE:** 24/2/2016  
**BORE No:** 14  
**DIP/AZIMUTH:** 90°/--

### Depth (m)  
<table>
<thead>
<tr>
<th>ID</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, fine to medium grained sandstone with some siltstone laminations and beds</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11.3</td>
<td>SANDSTONE - high strength, fresh, slightly fractured, light grey, medium to coarse grained, cross bedded sandstone</td>
<td></td>
<td></td>
<td>11.35m: B10°, cbs co</td>
<td></td>
<td>PL(A) = 1.6</td>
</tr>
<tr>
<td>12</td>
<td>Bore discontinued at 12.0m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Discontinuities
- Type: B - Bedding, J - Joint, S - Shear, F - Fault
- Core Rec.: C
- RQD %: 100

### Test Results & Comments
- PL(A) = 2.7
- PL(A) = 1.6

### Remarks:
- RIG: Scout 2  
- DRILLER: WG  
- LOGGED: SI/AT  
- CASING: HW to 1.0m  
- WATER OBSERVATIONS: No free groundwater observed whilst augering
## Description of Strata

### Depth (m)

<table>
<thead>
<tr>
<th>ID</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.08</td>
<td>ASPHALTIC CONCRETE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>SHALE - extremely low strength, light grey-brown, shale with some low strength bands</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.6</td>
<td>SHALE - very low and low strength, highly to moderately weathered, light grey to grey shale with some clay bands and medium strength bands</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>SHALE - medium strength, moderately weathered, fractured, grey-brown, shale with approximately 15% - 20% fine sandstone laminations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.4</td>
<td>LAMINITE - high strength, slightly weathered, slightly fractured, light grey to grey laminite with approximately 30% siltstone laminations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.0</td>
<td>SANDSTONE - high and very high strength, fresh, slightly fractured and unbroken, light grey, fine grained sandstone</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Discontinuities

- **B - Bedding**
- **J - Joint**
- **S - Shear**
- **F - Fault**

### Sampling & In Situ Testing

<table>
<thead>
<tr>
<th>Test Results &amp; Comments</th>
<th>Type</th>
<th>Core Rec. %</th>
<th>RQD</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>PL(A) = 0.5</td>
<td>C</td>
<td>84</td>
<td>0</td>
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</tr>
<tr>
<td>PL(A) = 0.3</td>
<td>C</td>
<td>100</td>
<td>41</td>
<td></td>
</tr>
<tr>
<td>PL(A) = 0.6</td>
<td>C</td>
<td>100</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>PL(A) = 0.7</td>
<td>C</td>
<td>100</td>
<td>98</td>
<td></td>
</tr>
<tr>
<td>PL(A) = 1.7</td>
<td>C</td>
<td>100</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>PL(A) = 3.2</td>
<td>C</td>
<td>100</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

### Other Information

- **Note:** Unless otherwise stated, rock is fractured along rough planar bedding dipping 0° - 10°

---

**Sampling & In Situ Testing Legend**

- A: Auger sample
- B: Bulk sample
- BLK: Block sample
- C: Coned sample
- D: Disturbed sample
- E: Environmental sample
- G: Gas sample
- P: Piston sample
- U: Tube sample (x mm dia.)
- W: Water sample
- X: Water level
- PID: Photo ionisation detector (ppm)
- PL(D): Point load diametral test (MPa)
- PL(D): Point load axial test (50) (MPa)
- S: Standard penetration test
- T: Shear vane (kPa)

---

**Borescope Log**

- **CLIENT:** NSW Health
- **PROJECT:** Concord Hospital
- **LOCATION:** Hospital Road, Concord West
- **SURFACE LEVEL:** 3.4 AHD
- **EASTING:** 323596
- **NORTING:** 6254109
- **DATE:** 25/2/2016
- **BORE No:** 15

---

**Remarks:**

- Solid flight auger to 1.0m; NMLC-Coring to 12.0m
- No free groundwater observed whilst augering

---

**Water Observations:** No free groundwater observed whilst augering
10.0 – 12.0m
**BOREHOLE LOG**

**CLIENT:** NSW Health  
**PROJECT:** Concord Hospital  
**LOCATION:** Hospital Road, Concord West  
**SURFACE LEVEL:** 3.4 AHD  
**EASTING:** 323596  
**NORTHING:** 6254109  
**DIP/AZIMUTH:** 90°/--  
**BORE No:** 15  
**PROJECT No:** 85356.00  
**DATE:** 25/2/2016  

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.0</td>
<td>SANDSTONE - high strength, fresh, unbroken, light grey, fine grained sandstone (continued)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.0</td>
<td>Bore discontinued at 12.0m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Discontinuities**
- B - Bedding  
- J - Joint  
- S - Shear  
- F - Fault

**Sampling & In Situ Testing**
- PL(A) Point load axial test Is(50) (MPa)
- PL(D) Point load diametral test Is(50) (MPa)
- PP Pocket penetrometer (kPa)
- ST Standard penetration test
- SHA Shear vane (kPa)

**Test Results & Comments**
- PL(A) = 1.9
- PL(A) = 1.3

**Remarks:**
- No free groundwater observed whilst augering

**Type of Boring:** Solid flight auger to 1.0m; NMLC-Coring to 12.0m

**Water Observations:** No free groundwater observed whilst augering

**Driller:** SS  
**Logged:** SI  
**Casing:** HW to 1.0m

---

**Sampling & In Situ Testing Legend**

- **A** Auger sample  
- **B** Bulk sample  
- **BLK** Block sample  
- **C** Core drilling  
- **D** Disturbed sample  
- **E** Environmental sample  
- **G** Gas sample  
- **P** Piston sample  
- **U** Tube sample (x mm dia.)  
- **W** Water sample  
- **W** Water level  
- **PID** Photo ionisation detector (ppm)  
- **PL(A)** Point load axial test Is(50) (MPa)  
- **PL(D)** Point load diametral test Is(50) (MPa)  
- **PP** Pocket penetrometer (kPa)  
- **ST** Standard penetration test  
- **SHA** Shear vane (kPa)
DOUGLAS PARTNERS PTY LTD
CONCORD HOSPITAL CARPARK – CONCORD WEST
BORE 16     PROJECT 85356.00     FEB 2016

1.0 – 5.0 m

DOUGLAS PARTNERS PTY LTD
CONCORD HOSPITAL CARPARK – CONCORD WEST
BORE 16     PROJECT 85356.00     FEB 2016

5.0 – 10.0 m
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>FILLING - topsoil</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>Filling - brown, slightly clayey, silt filling</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>SHALY CLAY - very stiff to hard, brown, shaly clay, moist</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.33</td>
<td>SHALE - extremely low and very low strength, extremely to highly weathered, fragmented to fractured, light grey-brown, shale with low and low to medium strength bands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>SHALE - low and medium strength, moderately to slightly weathered, fractured, grey to grey-brown, shale with some very low strength bands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.55</td>
<td>SHALE - low and medium strength, moderately to slightly weathered, fractured, grey to grey-brown, shale with some very low strength bands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.5</td>
<td>LAMINITE - high strength, fresh, unbroken, light grey, laminite with approximately 60% fine sandstone and 40% siltstone laminations and beds</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.0</td>
<td>SANDSTONE - high and very high strength, fresh, unbroken, light grey, fine grained sandstone with some carbonaceous laminations and bands</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

**备注：**

- 除非另有说明，岩石沿粗糙平面层状断面倾斜0°-10°。
- 1m: 核芯损失：330mm
- 1.53-1.6m: fg, fe
- 1.66-1.73m: J (x3)
- 1.87 & 2.05m: J (x3)
- 2.15-2.35m: fg
- 2.7m: J60° & 80°, ST, RO, CLN
- 2.9m: J70°, pl, sm, fe
- 3.3m: B0°, cly, 1mm
- 3.4-3.7m: cly
- 7.78m: B0°, cly, 5mm

**测试结果：**

- PID = 2.1
- PL(A) = 0.2
- PL(A) = 0.3
- PL(A) = 0.5
- PL(A) = 0.4
- PL(A) = 0.4
- PL(A) = 0.4
- PL(A) = 0.4
- PL(A) = 0.4
- PL(A) = 6.4
- PL(A) = 2.3
- PL(A) = 2

**备注：**

- 核芯损失：330mm
- 1m

**备注：**

- 除非另有说明，岩石沿粗糙平面层状断面倾斜0°-10°。

**备注：**

- 除非另有说明，岩石沿粗糙平面层状断面倾斜0°-10°。
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SANDSTONE - high and very high strength, fresh, unbroken, light grey, fine grained sandstone with some carbonaceous laminations and bands (continued)</td>
<td>EW</td>
<td>C9</td>
<td>0.01</td>
<td>B - Bedding</td>
<td>PL(A) = 1.9</td>
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<tr>
<td>10.8</td>
<td>SANDSTONE - high strength, fresh, unbroken, light grey, medium grained sandstone</td>
<td>UW</td>
<td>C9</td>
<td>0.01</td>
<td>S - Shear</td>
<td>PL(A) = 1.9</td>
</tr>
<tr>
<td>12.0</td>
<td>Bore discontinued at 12.0 m</td>
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<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

**Remarks:**
- **RIG:** Scout 2
- **DRILLER:** SS
- **LOGGED:** SI
- **CASING:** HW to 1.0m
- **TYPE OF BORING:** Solid flight auger to 1.0m; NMLC-Coring to 12.0m
- **WATER OBSERVATIONS:** No free groundwater observed whilst augering

**Sampling & In Situ Testing Legend:**
- A: Auger sample
- B: Bulk sample
- BLK: Block sample
- C: Core drilling
- D: Disturbed sample
- E: Environmental sample
- G: Gas sample
- P: Piston sample
- U: Tube sample (x mm dia.)
- W: Water sample
- T: Water level
- PID: Photo ionisation detector (ppm)
- PL(A): Point load axial test (MPa)
- PL(D): Point load diametral test (MPa)
- PP: Pocket penetrometer (kPa)
- V: Shear vane (kPa)
DOUGLAS PARTNERS PTY LTD
CONCORD HOSPITAL CARPARK – CONCORD WEST
BORE 17 PROJECT 85356.00 FEB 2016

2.5 – 7.0m

DOUGLAS PARTNERS PTY LTD
CONCORD HOSPITAL CARPARK – CONCORD WEST
BORE 17 PROJECT 85356.00 FEB 2016

7.0 – 12.0m
Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°-10°

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Degree of Weathering</th>
<th>Rock Strength</th>
<th>Fracture Spacing (m)</th>
<th>Discontinuities</th>
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<td>0.18</td>
<td>CONCRETE</td>
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<tr>
<td>1.0</td>
<td>FILLING - grey-brown, silty clay filling with some gavel</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>2.0</td>
<td>CLAY - stiff, mottled red-brown, light grey clay, moist</td>
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<tr>
<td>2.5</td>
<td>SHALE - extremely low strength, light grey-brown, shale with ironstone gravel, moist</td>
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<td></td>
</tr>
<tr>
<td>3.05</td>
<td>SHALE - extremely low and very low strength, extremely to highly weathered, highly fractured to fractured, light grey-brown, shale with some low to medium strength ironstone bands</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>4.13</td>
<td>SANDSTONE - low to medium then medium strength, moderately weathered, fractured to slightly fractured, grey-brown fine grained sandstone with some siltstone laminations</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>5.55</td>
<td>SANDSTONE - very high then high strength, slightly weathered then fresh, slightly fractured and unbroken, light grey-brown to light grey, fine to medium grained sandstone with some carbonaceous laminations</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Sampling & In Situ Testing**

- **PL(A)**: Point load axial test
- **Is(50)**: Ultimate compressive strength

**Test Results & Comments**

- **PL(A)**: 0.5
- **PL(A)**: 0.3
- **PL(A)**: 3.5
- **PL(A)**: 2.8
- **PL(A)**: 2.8
- **PL(A)**: 2.7

**Notes:**

- Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°-10°
- Core loss: 200mm
- Fragmentation: 2.8-3.05m: B's 0°-10°, fe, cly
- Joints: 3.05-3.15m: fg
- 3.57m: J30°, pl, sm, cly
- 4.26-4.75m: B (x3) 0°, fe

**Water Observations:**

- No free groundwater observed whilst augering; Water level at 4.2m depth on 14/3/16 (after bailing)
- Depth: 43210-1-2-3-4-5

**Remarks:**

- Standpipe installed to 12.0m (screen 9.0-12.0m; gravel 8.5-12.0m; bentonite 8.0-8.5m; backfill to GL with gatic cover)

**RIG:** Scout 2

**DRILLER:** WG

**LOGGED:** SI

**CASING:** HW to 2.5m

**SURFACE LEVEL:** 4.2 AHD

**NORTHING:** 6254179

**EASTING:** 323639

**DATE:** 26/2/2016

**LOCATION:** Hospital Road, Concord West

**PROJECT:** Concord Hospital

**CLIENT:** NSW Health

**PROJECT No:** 85356.00

**BORE No:** 17

**SURVEY Details:**

- **EASTING:** 323639
- **NORTHING:** 6254179
- **DIP/AZIMUTH:** 90°/--

**Water level:** 4.2 AHD

**Surface Level:** 4.2 AHD

**Easting:** 323639

**Northing:** 6254179

**Remarks:**

- Standpipe installed to 12.0m (screen 9.0-12.0m; gravel 8.5-12.0m; bentonite 8.0-8.5m; backfill to GL with gatic cover)
### SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium to coarse grained sandstone with some siltstone clasts (continued)

**FRACKURE SPACING (m)**
- 10.15 & 10.35m: B (x2)  
  - D°, cbs co

**PL(A)**
- PL(A) = 1.8
- PL(A) = 2

**Discontinuities**

**Sampling & In Situ Testing**
- PL(A) = 1.8
- PL(A) = 2

**Description of Strata**
- SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium to coarse grained sandstone with some siltstone clasts

**Additional Information**
- Standpipe installed to 12.0m (screen 9.0-12.0m; gravel 8.5-12.0m; bentonite 8.0-8.5m; backfill to GL with gatic cover)

**Remarks**
- No free groundwater observed whilst augering; Water level at 4.2m depth on 14/3/16 (after bailing)

**Additional Details**
- Standpipe installed to 12.0m (screen 9.0-12.0m; gravel 8.5-12.0m; bentonite 8.0-8.5m; backfill to GL with gatic cover)
Appendix E

Results of Laboratory Testing
CERTIFICATE OF ANALYSIS 143113

Client: Douglas Partners Pty Ltd
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Sally Peacock

Sample log in details:
Your Reference: 85356.00, Concord Hospital
No. of samples: 5 Soils
Date samples received / completed instructions received 10/03/16 / 10/03/16

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

Report Details:
Date results requested by: / Issue Date: 17/03/16 / 16/03/16
Date of Preliminary Report: Not Issued
NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025. Tests not covered by NATA are denoted with *.

Results Approved By:

Jacinthe Hurst
Laboratory Manager
<table>
<thead>
<tr>
<th>Misc Inorg - Soil</th>
<th>Our Reference:</th>
<th>UNITS</th>
<th>143113-1</th>
<th>143113-2</th>
<th>143113-3</th>
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<td>Your Reference</td>
<td></td>
<td>BH2</td>
<td>BH12</td>
<td>BH14</td>
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<tr>
<td>Depth</td>
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<td>14/03/2016</td>
<td>14/03/2016</td>
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<td>pH 1:5 soil:water</td>
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<td>Electrical Conductivity 1:5</td>
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<td>Chloride, Cl 1:5 soil:water</td>
<td>mg/kg</td>
<td>69</td>
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<td>Sulphate, SO4 1:5 soil:water</td>
<td>mg/kg</td>
<td>130</td>
<td>55</td>
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<td>Resistivity in soil^</td>
<td>ohmm</td>
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<td>$pH_{ld}$</td>
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<td>TAA pH 6.5</td>
<td>moles</td>
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<td>$s$-TAA pH 6.5</td>
<td>%w/w S</td>
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<td>$pH_{ak}$</td>
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<td>TSA pH 6.5</td>
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<td>$ANC_E$</td>
<td>%</td>
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<tr>
<td>$a$-$ANCE$</td>
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<td>&lt;5</td>
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<tr>
<td>$s$-ANCE</td>
<td>%w/w S</td>
<td>&lt;0.05</td>
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<td>$S_{KCl}$</td>
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<td>$S_{P}$</td>
<td>%w/w</td>
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<td>$S_{POS}$</td>
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<td>$a$-$S_{POS}$</td>
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<td>$Ca_{KCl}$</td>
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<td>$a$-Net Acidity</td>
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<td>Liming rate</td>
<td>kg CaCO$_3$/ t</td>
<td>2.4</td>
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<td>$a$-Net Acidity without ANCE</td>
<td>moles</td>
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<td>Inorg-001</td>
<td>pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.</td>
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<td>Inorg-002</td>
<td>Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment &amp; Lyons.</td>
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<tr>
<td>Inorg-081</td>
<td>Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyzer.</td>
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<td>Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA 22nd ED 2510 and Rayment &amp; Lyons. Resistivity is calculated from Conductivity.</td>
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<td>pH Units</td>
<td>Inorg-001</td>
<td>[NT]</td>
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<td>Electrical Conductivity 1:5 soil:water</td>
<td>µS/cm</td>
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<td>&lt;1</td>
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<td>130</td>
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<td>Chloride, Cl 1:5 soil:water</td>
<td>mg/kg</td>
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<td>Sulphate, SO4 1:5 soil:water</td>
<td>mg/kg</td>
<td>Inorg-081</td>
<td>&lt;10</td>
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<td>130</td>
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<td>Resistivity in soil*</td>
<td>ohmm</td>
<td>Inorg-002</td>
<td>&lt;1.0</td>
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<th>METHOD</th>
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<td>11/03/2016</td>
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<td>Date prepared</td>
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<td></td>
<td>1/03/2 11/03/2016</td>
<td>11/03/2016</td>
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<tr>
<td>Date analysed</td>
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<td>11/03/2016</td>
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<td>LCS-1</td>
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<td>TAA pH 6.5</td>
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Report Comments:

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

INS: Insufficient sample for this test
NR: Test not required
<: Less than
>: Greater than

PQL: Practical Quantitation Limit
RPD: Relative Percent Difference
NT: Not tested
NA: Test not required
LCS: Laboratory Control Sample
**Quality Control Definitions**

**Blank**: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc., can be determined by processing solvents and reagents in exactly the same manner as for samples.

**Duplicate**: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

**Matrix Spike**: A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

**LCS (Laboratory Control Sample)**: This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

**Surrogate Spike**: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

**Laboratory Acceptance Criteria**

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicate: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (±50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.
### Project Information
- **Project:** Concord Hospital
- **Project No.:** 85356.00
- **DP Contact Person:** Sally Peacock (standard TAT)
- **Ph.:** 9910 6200
- **Prior Storage:** Esky ❑, Fridge ❑, Shelved ❑, Freeze ❑
- **Attn.:** Tania Notaros

### Sample Information

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### Notes

- **PQL (S):** mg/kg
- **PQL (W):** mg/L
- **PQL = practical quantitation limit,** "As per Laboratory Method Detection Limit"

### Sampling Information

- **Date Relinquished:** 10/3/16
- **Total Number of Samples in Container:** 5
- **Results Required by:** Standard TAT
- **Samples Received:** Please sign and date to acknowledge receipt of samples and return by email

### Signature

- **Signature:** (Signature)
- **Date:** 18/3/16
- **Lab Ref.:** 368

### Additional Information

- **Send Results to:** Douglas Partners Pty Ltd
- **Address:** 96 Hambury Rd, West Ryde
- **Email:** sally.peacock@douglaspartners.com.au
CERTIFICATE OF ANALYSIS

Client: Douglas Partners Pty Ltd
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Sally Peacock

Sample log in details:
Your Reference: 85356.00, Concord Hospital
No. of samples: 1 soil
Date samples received / completed instructions received 11/03/16 / 11/03/16

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

Report Details:
Date results requested by: / Issue Date: 18/03/16 / 17/03/16
Date of Preliminary Report: Not Issued
NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025. Tests not covered by NATA are denoted with *.

Results Approved By:

[Signature]
Jacinta Hurst
Laboratory Manager
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<tr>
<td>CaP</td>
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<td>CaA</td>
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<td>MgKCl</td>
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<tr>
<td>SNAS</td>
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<tr>
<td>a-SNAS</td>
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<td>s-SNAS</td>
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<td>a-Net Acidity</td>
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<td>Liming rate</td>
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<td>QUALITY CONTROL</td>
<td>UNITS</td>
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<tr>
<td>sPOCAS</td>
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<tr>
<td>Liming rate without ANCE</td>
<td>kg CaCO$_3$/t</td>
<td>&lt;0.75</td>
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</table>
Report Comments:

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

INS: Insufficient sample for this test
NR: Test not required
<: Less than
>: Greater than

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

Laboratory Acceptance Criteria
Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.
**Project:** Concord Hospital  
**Project No:** 85356.00  
**DP Contact Person:** Sally Peacock  
**Ph:** 9910 6200  
**Prior Storage:** Esky ☐ Fridge ☐ Shelved ☐ Freezer ☑  
**Attn:** Tania Notaras

**Do samples contain HBM?** Yes ☐ No ☑ (If YES, then handle, transport and store in accordance with FPM HAZID)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Type</th>
<th>Lab ID</th>
<th>Analytes</th>
<th>Notes</th>
</tr>
</thead>
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<tr>
<td>BH310</td>
<td>S</td>
<td>✔</td>
<td>SPOCAS</td>
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</table>

**Ref:** PO 125382 Issued 10/3/16

**PQL (S)** mg/kg  
**PQL (W)** mg/L

**PQL = practical quantitation limit, *As per Laboratory Method Detection Limit**

Date relinquished: 11/3/16  
Total number of samples in container: 1  
Results required by: Standard TAT

**SAMPLES RECEIVED**  
Please sign and date to acknowledge receipt of samples and return by email

Signature: PT  
Date: 11/3/16 Lab Ref: 143162

Send results to: Douglas Partners Pty Ltd  
Address:

Email:
Results of California Bearing Ratio Test

Client: Sydney Local Health District

Project: Concord Hospital Redevelopment

Location: Hospital Road, Concord West

Test Location: 3

Depth / Layer: 0.10 - 0.60m

Description: Filling - dark grey, sandy clay filling with some gravel

Test Method(s): AS1289 6.1.1, AS1289 5.1.1, AS1289 2.1.1

Sampling Method(s): Sampled by Engineering Department

LEVEL OF COMPACTION: 100% of STD MDD

MOISTURE RATIO: 99% of STD OMC

SURCHARGE: 4.5 kg

SOAKING PERIOD: 4 days

Percentage > 19mm: 0%

<table>
<thead>
<tr>
<th>CONDITION</th>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (t/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>At compaction</td>
<td>18.8</td>
<td>1.78</td>
</tr>
<tr>
<td>After soaking</td>
<td>20.4</td>
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<tr>
<td>After test Top 30mm of sample</td>
<td>22.0</td>
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</tr>
<tr>
<td>Remainder of sample</td>
<td>20.7</td>
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<td>Field values</td>
<td>19.0</td>
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<td>1.78</td>
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RESULTS

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<th>TYPE</th>
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<th>CBR (%)</th>
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<tbody>
<tr>
<td>TOP</td>
<td>2.5 mm</td>
<td>6</td>
</tr>
</tbody>
</table>

Michael Gref
Senior Technician

NATA Accredited Laboratory No 628
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian National standards
Accredited for compliance with ISO/IEC 17025
Results of California Bearing Ratio Test

Client : Sydney Local Health District
Project : Concord Hospital Redevelopment
Location : Hospital Road, Concord West
Test Location : 4
Depth / Layer : 0.30 - 0.70m

Description: Clay - dark brown to brown clay with trace of sand.
Test Method(s): AS1289 6.1.1, AS1289 5.1.1, AS1289 2.1.1
Sampling Method(s): Sampled by Engineering Department

Percentage > 19mm: 0%

LEVEL OF COMPACTION: 100% of STD MDD
MOISTURE RATIO: 100% of STD OMC

SOAKING PERIOD: 4 days

SURCHARGE: 4.5 kg
SWELL: 1.7%

<table>
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<th>CONDITION</th>
<th>MOISTURE CONTENT %</th>
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<tr>
<td>At compaction</td>
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<td>1.68</td>
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<td>After test</td>
<td>Top 30mm of sample</td>
<td>27.4</td>
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<td>Remainder of sample</td>
<td>21.4</td>
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<td>Field values</td>
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<td>19.4</td>
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<tr>
<td>Standard Compaction (OMC/MDD)</td>
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<td>19.2</td>
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RESULTS

<table>
<thead>
<tr>
<th>TYPE</th>
<th>PENETRATION</th>
<th>CBR (%)</th>
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</thead>
<tbody>
<tr>
<td>TOP</td>
<td>2.5 mm</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Michael Gref
Senior Technician
Results of California Bearing Ratio Test

Client: Sydney Local Health District
Project: Concord Hospital Redevelopment
Location: Hospital Road, Concord West
Test Location: 15
Depth / Layer: 0.10 - 0.60m

Project No.: 85356.00
Report No.: 3
Report Date: 15/03/2016
Date Sampled: 2/03/2016
Date of Test: 14/03/2016
Page: 1 of 1

Description: Shale - extremely low strength, light grey-brown shale
Test Method(s): AS1289 6.1.1, AS1289 5.1.1, AS1289 2.1.1
Sampling Method(s): Sampled by Engineering Department
Percentage > 19mm: 0%

LEVEL OF COMPACTION: 100% of STD MDD
MOISTURE RATIO: 100% of STD OMC

SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
SWELL: 0.9%

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<td>After test Top 30mm of sample</td>
<td>26.0</td>
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<td>Remainder of sample</td>
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RESULTS

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</table>

Michael Gref
Senior Technician
Results of California Bearing Ratio Test

Client: Sydney Local Health District
Project: Concord Hospital Redevelopment
Location: Hospital Road, Concord West
Test Location: 17
Depth / Layer: 0.10 - 0.60m

Project No.: 85356.00
Report No.: 4
Report Date: 15/03/2016
Date Sampled: 2/03/2016
Date of Test: 14/03/2016
Page: 1 of 1

Description: Filling - grey-brown, silty clay with some gravel
Test Method(s): AS1289 6.1.1, AS1289 5.1.1, AS1289 2.1.1
Sampling Method(s): Sampled by Engineering Department

LEVEL OF COMPACTION: 100% of STD MDD
MOISTURE RATIO: 100% of STD OMC
SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
Percentage > 19mm: 0%
SWELL: 0.4%

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<td>1.70</td>
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<tr>
<td>After soaking</td>
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RESULTS

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NATA Accredited Laboratory No 828
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Accredited for compliance with ISO/IEC 17025

Michael Gref
Senior Technician
Appendix F

Interpreted Geotechnical Cross-Sections
NOTE:
1. Vertical exaggeration (4:1)
2. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
3. Strata layers and rock classification shown is generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
4. Summary logs only. Should be read in conjunction with detailed logs.
5. Plan locations of cross-sections shown in Dwg 2.
NOTE:
1. Vertical exaggeration (4:1)
2. Where appropriate, interpreted unit boundaries have been extrapolated. Subsurface conditions are however accurate at the borehole locations only and variations may occur away from the borehole locations.
3. Strata layers and rock classification shown is generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
4. Summary logs only. Should be read in conjunction with detailed logs.
5. Plan locations of cross-section is shown in Dwg 3
6. Interpreted geotechnical units are summarised in Section 8.1.2 of the report.
7. Basement levels are not yet known. Illustrative basement shown only for indicative extent of basement.

**Legend**
- Core Loss
- Asphaltic Concrete
- Filling
- Clay
- Laminite
- Concrete
- Sandstone coarse grained
- Sandstone fine grained
- Void

**Tests / Other**
- N = Standard penetration test value
- VL = Water level
- PP = Pocket penetrometer test value

**Interpreted Geotechnical Cross-section B-B’**

**Ground Level**

**TOWER BUILDING BASEMENT** (ILLUSTRATIVE ONLY)

**Table: Substrata Layers and Rock Classification**
- **Sandstone (15a)**
- **Laminite (14)**
- **Shale (13a)**
- **Shale (13b)**
- **Clay (12)**
- **Concrete**
- **Filling (11)**

**CLIENT:** Sydney Local Health District  
**TITLE:** Concord Hospital  
**DATE:** 30.03.2016  
**SCALE:** 1:100 (V)
NOTE:
1. Vertical exaggeration (4:1)
2. Where appropriate, interpreted unit boundaries have been extrapolated. Subsurface conditions are however accurate at the borehole locations only and variations may occur away from the borehole locations.
3. Strata layers and rock classification shown is generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
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