Job No. 29845L  BH11  START CORING AT 5.76m

CORE LOSS: 0.2m

EOBH AT 14.76m
**BOREHOLE LOG**

**Client:** HEALTH INFRASTRUCTURE  
**Project:** NEPEAN HOSPITAL REDEVELOPMENT  
**Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

**Job No.:** 29845L  
**Date:** 9/1/17  
**Method:** SPIRAL AUGER  
**R.L. Surface:** ~49.9 m  
**Logged/Checked By:** A.B./L.S.

**Groundwater Record**

<table>
<thead>
<tr>
<th>RL (m AHD)</th>
<th>Strength/Rel Density</th>
<th>Hand Penetrometer Readings (kPa)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Graphic Log**

**Geotechnical & Environmental Engineers**

**Client:** HEALTH INFRASTRUCTURE  
**Job No.:** 29845L  
**Method:** SPIRAL AUGER  
**Date:** 9/1/17  
**Plant Type:** JK305  
**Logged/Checked By:** A.B./L.S.

**Groundwater Monitoring Well**

- Installed to 17.5m, slotted from 2.5m to 17.5m, 2mm sand filter pack 1.2m to 17.5m, bentonite plug to top of cap.
### CORED BOREHOLE LOG

**Client:** HEALTH INFRASTRUCTURE  
**Project:** NEPEAN HOSPITAL REDEVELOPMENT  
**Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

<table>
<thead>
<tr>
<th>Job No.: 29845L</th>
<th>Core Size: NMLC</th>
<th>R.L. Surface: ~49.9 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date: 9/1/17</td>
<td>Inclination: VERTICAL</td>
<td>Datum: AHD</td>
</tr>
<tr>
<td>Plant Type: JK305</td>
<td>Bearing: N/A</td>
<td>Logged/Checked By: A.B./L.S.</td>
</tr>
</tbody>
</table>

#### Water Loss  
**Level:**  
**Barrel Lift:**

---

#### Core Description

**Table:**

<table>
<thead>
<tr>
<th>Water Loss</th>
<th>Depth (m)</th>
<th>Graphic Log</th>
<th>Core Description</th>
<th>Waterlogging</th>
<th>Point Load Strength Index $f_c(50)$</th>
<th>Defect Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>RL m (AHD)</td>
<td></td>
<td></td>
<td>Rock Type, grain characteristics, colour, structure, minor components.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

#### Defect Details

**Table:**

<table>
<thead>
<tr>
<th>Defect Spacing (mm)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type, inclination, thickness, planarity, roughness, coating, specific, general</td>
</tr>
</tbody>
</table>

---

#### Graphic Log

- **Core Loss 1.50m**
- **Silty Clay:** high plasticity, light grey and brown.
- **Shale:** grey and brown, with fine grained, light brown sandstone and clay seams.
- **Shale:** grey and dark grey.
- **Core Loss 0.12m**
- **Shale:** dark grey, with fine grained, grey sandstone bands.
- **as above, but bedded at 20-30°** and some M strength bands.
- **Shale:** grey and dark grey.

---

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**Borehole No. 12**

**2 / 3**
**Core Description**

Rock Type, grain characteristics, colour, structure, minor components.

- **SHALE**: grey and dark grey.
- **SHALE**: dark grey, with carbonaceousshale seams.
- **SHALE**: dark grey, with fine grained, grey sandstone bands.
- As above, but bedded at 20°.

**Log Details**

- **Job No.**: 29845L
- **Core Size**: NMLC
- **Date**: 9/1/17
- **Plant Type**: JK305
- **Datum**: AHD
- **Logged/Checked By**: A.B./L.S.
- **R.L. Surface**: ~49.9 m

**Defect Details**

- **Type, inclination, thickness, planarity, roughness, coating.**
- **Specific**
- **General**
- **Defect Spacing (mm)**
- **Strength**
- **Weathering**

**Graphic Log**

- Slides and inclinations
- Defects and their descriptions
- Core descriptions

**END OF BOREHOLE AT 18.00 m**
<table>
<thead>
<tr>
<th>No.</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Core Loss: 1.5m</td>
</tr>
<tr>
<td>7</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Core Loss: 0.2m</td>
</tr>
<tr>
<td>10</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
</tr>
</tbody>
</table>

Job No 29845L BH12 Start coring at 5.79m

End of BH12 at 18.00m
## BOREHOLE LOG

**Client:** HEALTH INFRASTRUCTURE  
**Project:** NEPEAN HOSPITAL REDEVELOPMENT  
**Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

### General Information
- **Job No.:** 29845L  
- **Method:** SPIRAL AUGER  
- **R.L. Surface:** ~51.4 m  
- **Datum:** AHD

### Graphical Log
- **Plant Type:** JK305  
- **Logged/Checked By:** A.B./L.S.

### Groundwater

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>SAMPLES</th>
<th>Field Tests</th>
<th>RL (m AHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>N = 8</td>
<td></td>
<td>51</td>
</tr>
<tr>
<td>2</td>
<td>N = 25</td>
<td></td>
<td>50</td>
</tr>
<tr>
<td>3</td>
<td>N = 5, 1, 5</td>
<td></td>
<td>49</td>
</tr>
<tr>
<td>4</td>
<td>N = 5, 8, 17</td>
<td></td>
<td>48</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td>47</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td>46</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td>45</td>
</tr>
</tbody>
</table>

### Core Description

- **ASPHALTIC CONCRETE:** 40mm. t.  
- **FILL:** Sandy gravel, fine to medium grained, angular igneous, grey, fine to medium grained brown sand.  
- **SILTY CLAY:** high plasticity, light grey.  
- **SANDSTONE:** fine grained, light brown and grey, with iron indurated bands.  
- **SANDSTONE:** fine grained, grey.

### Remarks
- **ROADBASE**
- **LOW ‘TC’ BIT RESISTANCE**
- **HIGH RESISTANCE**

### Additional Information
- **Client:** HEALTH INFRASTRUCTURE  
- **Project:** NEPEAN HOSPITAL REDEVELOPMENT  
- **Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Water Level</th>
<th>Sand Lift</th>
<th>RL (m AHD)</th>
<th>Defect Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td></td>
<td></td>
<td></td>
<td>START CORING AT 4.55m</td>
</tr>
<tr>
<td>4.6</td>
<td></td>
<td></td>
<td></td>
<td>SANDSTONE: fine grained, grey and grey brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>as above, but with dark grey shale bands, trace of dark grey laminae.</td>
</tr>
<tr>
<td>4.7</td>
<td></td>
<td></td>
<td></td>
<td>SHALE: dark grey and grey, with fine grained, grey sandstone bands.</td>
</tr>
</tbody>
</table>

**CORE DESCRIPTION**
Rock Type, grain characteristics, colour, structure, minor components.

**POINT LOAD STRENGTH INDEX** ($I_{(50)}$)
Strength

**DEFECT DETAILS**
Type, inclination, thickness, planarity, roughness, coating.

**Specific**
- (5.12m) CB, 0°, 10 mm t
- (5.22m) XWS, 0°, 20 mm t
- (5.35m) XWS, 0°, 3 mm t
- (6.13m) J, 90°, P, B
- (6.15m) XWS, 0°, 5 mm t
- (6.30m) J, 90°, P, S
- (6.38m) J, 90°, P, S
- (6.45m) XWS, 0°, 110 mm t
- (6.70m) J, 90°, P, R, IS
- (7.84m) HEALED J, 90°, P, S
- (7.98m) J, 90°, P, S
- (8.02m) XWS, 0°, 15 mm t
- (9.14m) J, 90°, P, S
- (9.20m) J, 70°, P, S, CLAY INFILL
- (9.78m) XWS, 0°, 120 mm t
- (10.21m) CB, 0°, 25 mm t
- (10.24m) CB, 0°, 13 mm t
- (10.29m) CB, 0°, 1 mm t
- (10.51m) XWS, 0°, 4 mm t
- (10.89m) Ba, 0°, P, R, XW INFILL

**General**

**R.L. Surface**: -51.4 m

**Datum**: AHD

**Logged/Checked By**: A.B./L.S.
**CORED BOREHOLE LOG**

**Client:** HEALTH INFRASTRUCTURE  
**Project:** NEPEAN HOSPITAL REDEVELOPMENT  
**Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

<table>
<thead>
<tr>
<th>Job No.: 29845L</th>
<th>Core Size: NMLC</th>
<th>R.L. Surface: ~51.4 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date: 22/12/16</td>
<td>Inclination: VERTICAL</td>
<td>Datum: AHD</td>
</tr>
<tr>
<td>Plant Type: JK305</td>
<td>Bearing: N/A</td>
<td>Logged/Checked By: A.B./L.S.</td>
</tr>
</tbody>
</table>

**Core Description**
- **SANDSTONE:** fine grained, grey, with dark grey laminae. (continued)
- **SHALE:** dark grey and grey, with fine grained, grey sandstone bands.

**Defect Details**
- (11.76m) J, 90°, P, R
- (11.94m) XWS, 0°, 5 mm t
- (12.30m) J, 40°, P, R
- (12.52m) FRAGMENTED ZONE, 0°, 25 mm t
- (12.76m) XWS, 0°, 80 mm t
- (13.04m) J, 90°, P, R

**End of Borehole at 13.21 m**
Job No. 29845L  BH13  START CORING AT 4.55m

END OF BH13  AT  13.21m
### BOREHOLE LOG

**Client:** HEALTH INFRASTRUCTURE  
**Project:** NEPEAN HOSPITAL REDEVELOPMENT  
**Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

#### Job No.: 29845L  
**Method:** SPIRAL AUGER  
**R.L. Surface:** ~51.1 m  
**Date:** 21/12/16  
**Plant Type:** JK305  
**Logged/Checked By:** A.B./L.S.

<table>
<thead>
<tr>
<th>SAMPLES</th>
<th>Field Tests</th>
<th>RL (m AHD)</th>
<th>Depth (m)</th>
<th>Graphic Log</th>
<th>Description</th>
<th>Moisture Condition/Weathering</th>
<th>Strength/Rel Density</th>
<th>Hand Penetrometer Readings (kPa)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>51</td>
<td>CH</td>
<td>ASPHALTIC CONCRETE: 50mm.1 FILL: Gravely sand, fine to medium grained, brown, fine to medium grained igneous gravel.</td>
<td>M</td>
<td>MC-PL</td>
<td>St</td>
<td>130 180 340</td>
</tr>
<tr>
<td>N = 10</td>
<td></td>
<td></td>
<td>50</td>
<td></td>
<td>SILTY CLAY: high plasticity, light grey mottled red brown.</td>
<td></td>
<td></td>
<td>VSt</td>
<td>460 350 &gt;600</td>
</tr>
<tr>
<td>1,4,6</td>
<td></td>
<td></td>
<td>49</td>
<td></td>
<td>as above, but with fine to medium grained ironstone gravel.</td>
<td></td>
<td></td>
<td>VSt - H</td>
<td></td>
</tr>
<tr>
<td>N = 17</td>
<td></td>
<td></td>
<td>48</td>
<td></td>
<td>SANDSTONE: fine grained, grey brown, with ironstone and shale bands.</td>
<td>DW</td>
<td>VL</td>
<td></td>
<td>LOW TC BIT RESISTANCE WITH MODERATE BANDS</td>
</tr>
<tr>
<td>5,7,10</td>
<td></td>
<td></td>
<td>47</td>
<td></td>
<td>SANDSTONE: fine grained, brown grey.</td>
<td></td>
<td>L - M</td>
<td></td>
<td>MODERATE RESISTANCE</td>
</tr>
</tbody>
</table>

**Groundwater Record**

<table>
<thead>
<tr>
<th>RL (m AHD)</th>
<th>Depth (m)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1-3</td>
<td>REFER TO CORED BOREHOLE LOG</td>
</tr>
</tbody>
</table>

**REFERENCES**

- **Groundwater**
  - **Sample Numbers:** 50, 51
  - **Description:** ASPHALTIC CONCRETE: 50mm.1 FILL: Gravely sand, fine to medium grained, brown, fine to medium grained igneous gravel.
  - **Moisture Condition/Weathering:** M
  - **Strength/Rel Density:** MC-PL
  - **Hand Penetrometer Readings (kPa):** St 130 180 340

- **Rock Description**
  - **Sample Numbers:** 50, 51
  - **Description:** SILTY CLAY: high plasticity, light grey mottled red brown.
  - **Moisture Condition/Weathering:** M
  - **Strength/Rel Density:** MC-PL
  - **Hand Penetrometer Readings (kPa):** St 130 180 340

- **Sample Numbers:** 49, 48
  - **Description:** as above, but with fine to medium grained ironstone gravel.
  - **Moisture Condition/Weathering:** M
  - **Strength/Rel Density:** MC-PL
  - **Hand Penetrometer Readings (kPa):** St 130 180 340

- **Sample Numbers:** 47, 46
  - **Description:** SANDSTONE: fine grained, grey brown, with ironstone and shale bands.
  - **Moisture Condition/Weathering:** M
  - **Strength/Rel Density:** DW
  - **Hand Penetrometer Readings (kPa):** VL 460 350 >600

- **Sample Numbers:** 45, 44
  - **Description:** SANDSTONE: fine grained, brown grey.
  - **Moisture Condition/Weathering:** M
  - **Strength/Rel Density:** L - M
  - **Hand Penetrometer Readings (kPa):** MODERATE RESISTANCE
**CORE DESCRIPTION**

Rock Type, grain characteristics, colour, structure, minor components.

**SANDSTONE:** fine grained, brown grey.

**SHALE:** grey and dark grey, with light brown fine grained sandstone and medium strength iron indurated bands.

**SANDSTONE:** fine grained, grey brown, with dark grey shale bands, trace of iron indurated bands.

as above, but sandstone, fine grained, grey.

**SANDSTONE:** fine grained, grey, with dark grey shale bands, trace of iron indurated bands.

as above, but sandstone, fine grained, grey.

**SANDSTONE:** fine grained, grey, with dark grey laminae and some very high strength bands.

as above, but with dark grey shale bands.

---

**DEFECT DETAILS**

Type, inclination, thickness, planarity, roughness, coating.

**Specific**

- Depth (m) 47.5
- **DEFECT SPACING (mm)**
  - 0

**General**

- Depth (m) 47.5
- **DEFECT SPACING (mm)**
  - 0

---

**DEFECTS**

- **Type:** CS, 0°, 20 mm
- **Inclination:** 0°
- **Thickness:** 20 mm
- **Coating:** P, S, XW, INFILL

---

**PROJECT DETAILS**

- **Client:** HEALTH INFRASTRUCTURE
- **Project:** NEPEAN HOSPITAL REDEVELOPMENT
- **Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

---

**LOG DETAILS**

- **Job No.:** 29845L
- **Core Size:** NMLC
- **Inclination:** VERTICAL
- **Datum:** AHD
- **Logged/Checked By:** A.B./L.S.
**CORE DESCRIPTION**

**SANDSTONE:** fine grained, grey, dark, with grey laminae.

**SHALE:** dark grey and grey, as above, but grey.

**POINT LOAD STRENGTH INDEX**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>DEFECT SPACING (mm)</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.00</td>
<td>0.3</td>
<td>HEALED JOINT, 90°, P</td>
</tr>
<tr>
<td>12.00</td>
<td>0.3</td>
<td>J, 90°, P, S</td>
</tr>
<tr>
<td>13.00</td>
<td>0.5</td>
<td>HEALED JOINT, 90°, P</td>
</tr>
<tr>
<td>13.20</td>
<td>0.5</td>
<td>J, 90°, P, S</td>
</tr>
<tr>
<td>13.40</td>
<td>0.5</td>
<td>XWS, 0°, 3 mm.t</td>
</tr>
<tr>
<td>13.60</td>
<td>0.5</td>
<td>XWS, 0°, 10 mm.t</td>
</tr>
<tr>
<td>14.10</td>
<td>0.5</td>
<td>XWS, 0°, 5 mm.t</td>
</tr>
<tr>
<td>14.50</td>
<td>0.5</td>
<td>XWS, 0°, 3 mm.t</td>
</tr>
<tr>
<td>14.90</td>
<td>0.5</td>
<td>J, 10°, P, S</td>
</tr>
<tr>
<td>15.00</td>
<td>0.5</td>
<td>J, 90°, P, S</td>
</tr>
</tbody>
</table>

**DEFECT DETAILS**

- **Type:** Healed joint, 90°, P
- **Inclination:** 90°
- **Thickness:** 0.3 mm
- **Planarity:** 0
- **Roughness:** 3
- **Coating:** 0

**END OF BOREHOLE AT 16.29 m**
BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE
Project: NEPEAN HOSPITAL REDEVELOPMENT
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 29845L
Method: SPIRAL AUGER
R.L. Surface: ~53.9 m
Datum: AHD

Logged/Checked By: A.B./L.S.

FIELD TESTS

<table>
<thead>
<tr>
<th>SAMPLES</th>
<th>Field Tests</th>
<th>RL (m AHD)</th>
<th>Depth (m)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAMPLES</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

DEPTH (m)

1 2 3 4 5 6

DESCRIPTION

- ASPHALTIC CONCRETE: 45mm
  FILL: Sandy gravel, fine to medium grained igneous, dark grey, fine to medium grained sand.
  SILTY CLAY: high plasticity, light grey mottled red brown, with fine to coarse grained ironstone gravel.

- SHALE: grey, with fine grained, light grey sandstone bands and iron indurated bands and clay seams.

- SANDSTONE: fine grained, light brown, with grey shale bands.

REMARKS

ROADBASE
REMOULDS TO A MATERIAL WITH SOIL PROPERTIES
BANDED LOW AND VERY LOW 'TC' BIT RESISTANCE
LOW RESISTANCE

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Borehole No. 15

Client: HEALTH INFRASTRUCTURE
Project: NEPEAN HOSPITAL REDEVELOPMENT
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 29845L
Method: SPIRAL AUGER
R.L. Surface: ~53.9 m
Datum: AHD

Logged/Checked By: A.B./L.S.

FIELD TESTS

<table>
<thead>
<tr>
<th>SAMPLES</th>
<th>Field Tests</th>
<th>RL (m AHD)</th>
<th>Depth (m)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAMPLES</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

DEPTH (m)

1 2 3 4 5 6

DESCRIPTION

- ASPHALTIC CONCRETE: 45mm
  FILL: Sandy gravel, fine to medium grained igneous, dark grey, fine to medium grained sand.
  SILTY CLAY: high plasticity, light grey mottled red brown, with fine to coarse grained ironstone gravel.

- SHALE: grey, with fine grained, light grey sandstone bands and iron indurated bands and clay seams.

- SANDSTONE: fine grained, light brown, with grey shale bands.

REMARKS

ROADBASE
REMOULDS TO A MATERIAL WITH SOIL PROPERTIES
BANDED LOW AND VERY LOW 'TC' BIT RESISTANCE
LOW RESISTANCE

COPYRIGHT
### CORE DESCRIPTION
- **SANDSTONE**: fine grained, brown, trace of dark grey shale laminae.
- **SHALE**: dark grey and grey, with fine grained, grey sandstone bands.

### DEFECT DETAILS
- **Core Loss**: 0.50m
- **Core Loss**: 0.42m
- **Core Loss**: 0.20m

### Point Load Strength Index ($f_y$)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>DW</th>
<th>SW</th>
<th>Core Loss</th>
<th>SHALE</th>
<th>SANDSTONE</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.70</td>
<td>M-H</td>
<td></td>
<td>0.50m</td>
<td></td>
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</tr>
<tr>
<td>5.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.80</td>
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### Defect Spacing (mm)

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<th>SHALE</th>
<th>SANDSTONE</th>
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</table>

### Core Properties
- **Core Size**: NMLC
- **Inclination**: VERTICAL
- **Datum**: AHD
- **Logged/Checked By**: A.B./L.S.
**CORED BOREHOLE LOG**

**Client:** HEALTH INFRASTRUCTURE  
**Project:** NEPEAN HOSPITAL REDEVELOPMENT  
**Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

<table>
<thead>
<tr>
<th>Job No.:</th>
<th>29845L</th>
<th>Core Size:</th>
<th>NMLC</th>
<th>R.L. Surface:</th>
<th>~53.9 m</th>
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<td>Inclination:</td>
<td>VERTICAL</td>
<td>Datum:</td>
<td>AHD</td>
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<td>Plant Type:</td>
<td>JK305</td>
<td>Bearing:</td>
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<thead>
<tr>
<th>Water Level</th>
<th>Sand Lift</th>
<th>RL (m AHD)</th>
<th>Depth (m)</th>
<th>Graphic Log</th>
<th>Core Description</th>
<th>Weakening</th>
<th>Weather</th>
<th>Point Load Strength Index I_p(50)</th>
<th>Defect Details</th>
<th>Defect Spacing (mm)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Rock Type, grain characteristics, colour, structure, minor components.</td>
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<td>SHALE: dark grey and grey.</td>
<td>SW M</td>
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<td>CORE LOSS 0.32m</td>
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<td>SHALE: dark grey and grey.</td>
<td>SW EL</td>
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<td>as above, but trace of sandstone, fine grained, grey laminae.</td>
<td>SW M-H</td>
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<td></td>
<td>END OF BOREHOLE AT 16.46 m</td>
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**DEFECT DETAILS**

- **Type, Inclination, Thickness, Planarity, Roughness, Coating.**
- **Specific**
- **General**

**DEFECT SPACING (mm):**

- (11.0m) XWS, 0°, 4 mm.t
- (11.16m) XWS, 0°, 2 mm.t
- (11.4m) XWS, 0°, 3 mm.t
- (11.86m) XWS, 0°, 2 mm.t
- (12.09m) CS, 0°, 1 mm.t
- (12.10m) J, 90°, P, R
- (12.47m) XWS, 0°, 5 mm.t
- (12.65m) J, 90°, P, R
- (13.14m) J, 90°, P, R
- (13.52m) J, 90°, P, R
- (14.10m) J, 90°, P, R
- (14.51m) XWS, 0°, 1 mm.t
- (14.74m) J, 90°, P, S
- (14.84m) XWS, 0°, 2 mm.t
- (14.89m) XWS, 0°, 3 mm.t
- (15.11m) XWS, 0°, 2 mm.t
- (15.84m) XWS, 0°, 3 mm.t
- (15.94m) J, 90°, P, S
- (15.79m) XWS, 0°, 3 mm.t
- (16.03m) XWS, 0°, 20 mm.t
- (16.08m) J, 90°, P, R
- (16.28m) J, 90°, P, R

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JK Geotechnics 
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
Job No. 29845L BH15 START CORING AT 4.70m

CORE LOSS: 0.50m

CORE LOSS: 0.42m

CORE LOSS: 0.20m

CORE LOSS: 0.32m

END OF BH AT 16.46m
**Client:** HEALTH INFRASTRUCTURE  
**Project:** NEPEAN HOSPITAL REDEVELOPMENT  
**Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

**Job No.:** 29845L  
**Method:** SPIRAL AUGER  
**R.L. Surface:** ~52.2 m  
**Datum:** AHD  
**Date:** 11/1/17  
**Logged/Checked By:** A.B./L.S.

**Plant Type:** JK305

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<th>Borehole No.</th>
<th>Depth (m)</th>
<th>Groundwater</th>
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**Groundwater:**
- **FILL:** Gravelly sand, fine to coarse grained, brown, fine to medium grained, dark grey igneous gravel.
- **SILTY CLAY:** High plasticity, light grey, as above, but with fine to medium grained ironstone gravel.
- **SANDSTONE:** Fine grained, grey brown, with grey shale bands.

**Moisture Condition/Weathering:**
- **M:** ROADBASE  
- **MC>PL:** APPEARS MODERATELY COMPACTED  
- **XW:** VERY LOW 'TC' BIT RESISTANCE  
- **DW:** LOW TO MODERATE RESISTANCE

**Remarks:**
- REFER TO CORED BOREHOLE LOG

**Graphic Log**:
- Refer to the graphic log for detailed classification and description.
**CORE LOSS 0.49m**

**SHALE: dark grey, with iron indurated bands.**

**SHALE: dark grey, with fine grained, grey sandstone bands.**

**SHALE: dark grey, with some medium and high strength bands.**

**SANDSTONE: fine grained, grey, with dark grey laminae bedded subhorizontally.**

### Core Description
- **CASCADE LOSS:** 0.49m
- **SHALE:** dark grey, with iron indurated bands.
- **SHALE:** dark grey, with fine grained, grey sandstone bands.
- **SHALE:** dark grey, with some medium and high strength bands.
- **SANDSTONE:** fine grained, grey, with dark grey laminae bedded subhorizontally.

### Defect Details
- **Graph Log**
  - Depth (m): 4.85
  - **Type:** SHALE, SANDSTONE
  - **Inclination:** VERTICAL
  - **Strength:** WEATHERING
  - **Weathering Index:** ++
  - **Defect Spacing:** 0.03
  - **Defect Details:**
    - **Type:** WEATHERING
    - **Inclination:** 0°
    - **Thickness:** 60 mm
    - **Planarity:** N/A
    - **Roughness:** N/A
    - **Coating:** N/A

### borehole no.

**Job No.:** 29845L  
**Core Size:** NMLC  
**R.L. Surface:** ~52.2 m  
**Datum:** AHD  
**Logged/Checked By:** A.B./L.S.
### CORED BOREHOLE LOG

**Client:** HEALTH INFRASTRUCTURE  
**Project:** NEPEAN HOSPITAL REDEVELOPMENT  
**Location:** NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

**Job No.:** 29845L  
**Date:** 11/1/17  
**Plant Type:** JK305  
**Core Size:** NMLC  
**Inclination:** VERTICAL  
**Datum:** AHD  
**Logged/Checked By:** A.B./L.S.

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<tr>
<td>16.54</td>
<td>XWS, 0°, P, S</td>
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</table>

**Core Loss:** 0.26m  
**End of Borehole:** 16.63 m
Job No. 29845L  BH16  START CORING AT 4.85m

CORE LOSS: 0.49m

CORE LOSS: 0.26m

EOBH AT 16 63m
REPORT TO HEALTH INFRASTRUCTURE ON GEOTECHNICAL INVESTIGATION FOR PROPOSED MAIN WORKS, COMMERCIAL AND BUNKER DEVELOPMENTS AT NEPEAN HOSPITAL DERBY STREET, KINGSWOOD, NSW

24 February 2017
Ref: 29845L1rpt MWCDB
Date: 24 February 2017
Report No: 29845L1rpt MWCDB
Revision No: 0

Report prepared by: Linton Speechley
Principal I Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670

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   4.2 Subsurface Conditions .............................. 5
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ENVIROLAB CERTIFICATE OF ANALYSIS NO. 160819

BOREHOLE LOGS 7 TO 16 INCLUSIVE (WITH CORE PHOTOGRAPHS)

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REPORT EXPLANATION NOTES

APPENDIX A – PREVIOUS RELEVANT BOREHOLE LOGS BY GOLDER ASSOCIATES
1 INTRODUCTION

This report presents the results of a geotechnical investigation for proposed redevelopment works at Nepean Hospital, Derby Street, Kingswood, NSW. This report will cover the proposed main works area, the proposed commercial development area, and the proposed bunker. The Nepean Hospital site location and the location of each of these development areas are shown on the attached Figures 1 to 4 inclusive. The investigation was commissioned by Health Infrastructure and was carried out in accordance with the Health Infrastructure Consultancy Agreement (Contract No. H16465, dated 10 October 2016) and in accordance with our fee proposal P43459L dated 30 September 2016 and our email to Aurora projects dated 15 November 2017.

We have only been provided with limited information on the proposed developments in these areas. Based on advice from Bonacci Group and Aurora Projects we understand that;

- The main works area will comprise a new multi-storey hospital building in the central portions of the site. Working column loads up to about 12,000kN have been indicated and basements may or may not be required.
- The proposed commercial buildings will be located at the northern end of the site and will likely comprise four to six storey buildings with working column loads in the order of 5500kN. Basement levels would be ‘normal’ under such developments, however we do not know whether these will be included in the project at this stage.
- We have no information on the proposed bunkers, apart from advice from Bonacci Group that indicates they will be located toward the south western corner of the site and that the structures will have maximum working pile loads in the order of 2000kN.

The purpose of the investigation for these proposed areas was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, retention, bearing pressures for footings, and potential settlements.

This geotechnical report has been prepared for the various proposed works areas noted above. We note that the investigation for these proposed development works at the Nepean Hospital was also carried out in conjunction with geotechnical investigations for the proposed car park structure in the north western corner of the Hospital site. This report will not address the proposed car park structure and for further advice on that portion of the development, reference should be made to our previous report (Reference 29845Lrpt Car Park, dated 20 February 2017).
2  PREVIOUS INVESTIGATIONS
We have been provided with a number of previous geotechnical reports produced by Golder Associates for the Nepean Hospital site. These include;

- Geotechnical Investigation Report Number 097622055_002Rev1, dated 30 July 2009. This report was for a new four storey ‘East Block’ building.
- Geotechnical Investigation Report Number 107622058_002_R_Rev0, dated 9 June 2010. This report was an updated report for the new four storey ‘East Block’ building.
- Geotechnical Investigation Report Number 107622059_002_R_Rev1, dated 1 July 2010. This report was for a new Mental Health Patient Unit, a new Oral Health Building and a new Maintenance Depot

Relevant boreholes have been extracted from these reports to supplement our current investigations for the main building area. The relevant borehole logs have been attached as Appendix A, while the location of the relevant boreholes have also been shown on the attached Figure 5.

3  INVESTIGATION PROCEDURE
Prior to commencement of the fieldwork, the investigation locations were electromagnetically scanned by a specialist subcontractor so that all borehole locations could be located clear of buried services. Reference to ‘Dial Before You Dig’ plans was also carried out. Safe work measures and procedures were implemented during the course of the fieldwork.

The fieldwork for the investigation comprised the drilling of ten boreholes, (BH7 to BH16 inclusive) to total depths ranging from 13.09m to 18.00m below existing surface levels using our truck mounted JK350 rig and our track mounted JK305 rig. All boreholes were initially advanced using spiral auger drilling techniques and a Tungsten Carbide (TC) bit until reasonably competent bedrock was encountered. The boreholes were then continued to their final depth by rotary diamond coring techniques, using an NMLC triple tube core barrel and water flush.

The borehole locations are shown on the attached Figure 5, (which is based on survey plans by Cardno - Drawing No. 118117502 Rev 2 dated 22 November 2016), and these were set out by taped measurements from existing surface features shown on the survey plan. The approximate Reduced Level (RL) at each borehole location, as shown on the borehole logs, was interpolated from spot heights and contours from the survey plan. The height datum is Australian Height Datum.
The apparent compaction of the fill and strength of the cohesive soils were assessed from Standard Penetration Test (SPT) ‘N’ values, augmented by hand penetrometer tests carried out on cohesive samples recovered by the SPT split tube sampler. The strength of the bedrock in the augered portion was assessed from observation of the drilling resistance of the TC drill bit attached to the augers, tactile examination of rock cuttings, and correlation with the results of subsequent laboratory moisture content tests. It should be noted that strengths assessed in this way are approximate and variances of at least one strength order should not be unexpected.

Where the bedrock was cored, the recovered core was returned to Soil Test Services (STS), a NATA accredited laboratory, for photographing and Point Load Strength Index (Is50) testing. Using established correlations the Unconfined Compressive Strength (UCS) of the bedrock was then calculated from the Is50 results. These Point Load Strength test results are summarised in the attached Table C and on the borehole logs.

Selected soil samples were also returned to STS Pty Ltd, a NATA accredited laboratory, for Moisture Content, Atterberg Limit and Four-Day soaked CBR tests. The results of the laboratory testing are provided in the attached STS Tables A and B.

Selected samples were sent for soil aggression testing to Envirolab Services Pty Ltd, a NATA registered laboratory, the results of this testing are provided in the attached Envirolab Certificate of Analysis No. 160819.

We note that Tables A, B and C from STS, as well as the Envirolab test results, are from all boreholes completed across the entire hospital site during this current investigation. The test results from BH7 to BH16 inclusive are relevant to this report.

Groundwater observations were recorded in all boreholes during and on completion of auger drilling. A standpipe piezometer with data logger was installed in BH7, BH12 and BH13 to allow for long-term groundwater monitoring. No further groundwater monitoring has been carried out since the fieldwork was completed.

Our geotechnical engineer, Mr Arthur Billingham, was present on a full-time basis during the fieldwork, to direct the electromagnetic scanning, set out the borehole locations, nominate testing and sampling and prepare the borehole logs. The borehole logs, which include field test results and groundwater observations, are attached, together with a set of explanatory notes which
describe the investigation techniques, and their limitations and define the logging terms and symbols used.

During the fieldwork, samples were obtained for environmental testing and assessment purposes. Reference should be made to the report by EIS (Reference E29845K) for specific details.

4 RESULTS OF INVESTIGATION

4.1 Site Descriptions

Proposed Main Works Area (Refer to Figure 2)
This area consists of the existing open air carpark located centrally in the eastern half of the site. The car park is an asphaltic concrete (AC) paved surface that appeared in good condition upon a cursory visual inspection. The area slopes down to the north east at gradients ranging from about 1° to 2° from a local high point near the junction between North Block and East Block (in the vicinity of BH15). The car park is accessed off Somerset Street to the east.

The area is surrounded on all sides by hospital buildings which are generally on-grade with the adjacent paved areas. The exception is along the northern edge of the area where some excavation appears to have been undertaken for the construction of the Drugs and Alcohol Services building and Gateway Clinic. A concrete block retaining wall runs along the length of the northern edge of the car park, with surface levels in the car park generally between 1.8m and 2.0m above the ground floor levels of the buildings below.

Proposed Commercial Building Area (Refer to Figure 3)
The proposed commercial building area is located at the northern end of the site adjacent to the Great Western Highway and at the general location of the existing Tresillian and Cancer Care Units. The surface levels generally slope down towards the east at about 1° to 2° with a batter slope leading down to the carpark around the Tresillian Unit. In the eastern section of the area the ground surface flattens out between the Cancer Care Unit and Somerset Street. Medium to large trees are interspersed throughout the area.

Proposed Bunker Area (Refer to Figure 4)
The proposed bunker area consists of the decommissioned helipad and surrounds in the south-western corner of the hospital. Located centrally within the area is a concrete helipad at the apex of a circular mound that slopes away from the pad at approximately 5°. Around the western half of
the helipad, the sloping ground is retained behind a concrete block wall up to 2.5m high. At the base of the retaining wall is a swale drain that has been excavated to depths between 2m to 3m below the surface levels of Parker Street and Derby Street which bound the zone to the west and south respectively.

East of the helipad is an AC paved car park that slopes down to the south at approximately $3^\circ$ towards Derby Street. To the north-east, the surface levels continue to slope down from the helipad at approximately $7^\circ$ through a grassed embankment to the ground floor level of West Block.

### 4.2 Subsurface Conditions

The 1:000,000 Geological Series Sheet 9030 ‘Penrith’ indicates that the site is underlain by Bringelly Shale of the Wianamatta Group consisting of shale, carbonaceous claystone, claystone, laminite, fine- to medium-grained lithic sandstone, rare coal and tuff. This profile does not take into account in-situ weathering or any earthworks that have taken place on the site.

The investigation encountered a generalised profile consisting of relatively shallow fill overlying residual silty clay which transitioned to weathered shale or weathered sandstone bedrock at depths ranging from 1.3m to 5.5m. The weathered rock is quite variable in weathering, strength and defects across the various sites. In some of the boreholes there was an upper capping layer of Class II or III sandstone bedrock which comprised sandstone of high and very high strength. Some of the more pertinent subsurface observations are discussed below, however for specific details reference should be made to the attached borehole logs and graphical borehole summaries (Figures 6, 7 and 8).

### Pavements

Asphaltic concrete pavements were encountered in all boreholes with the exception of our BH8 and BH9. The asphalt pavements were 35mm to 60mm thick and were underlain by gravelly sand road base fill to a depth of up to approximately 0.5m. In BH7, the AC was underlain by a cement stabilised sandy gravel.

### Fill

Fill below the pavement layers was encountered in BH7, BH8, BH9, BH12 and BH16 and extended to depths generally ranging from 0.5m to 1.3m below existing surface levels. However deeper fill was encountered in BH7, where it extended to a depth of 3.5m. It is possible that the deeper fill in BH7 is associated with a nearby services trench or similar. The fill generally comprised silty clays of medium or high plasticity, although there was some silty sand fill in BH8 and some low plasticity.
silty clay in BH9. The fill was assessed as being moderately compacted in all boreholes, with the exception of BH7 where the deeper fill was assessed as being poorly compacted. The fill contained various gravel inclusions.

**Residual Silty Clay**
Residual silty clay was encountered in each borehole. The silty clay was assessed as being of medium and high plasticity and generally of very stiff to hard strength with some upper stiff silty clay in some of the boreholes. Traces of fine-grained ironstone gravel and ash were observed within the silty clay at some locations.

**Weathered Bedrock**
Weathered bedrock was first encountered at depths ranging from 1.3m to 5.5m below existing surface levels, with the general trend of rock levels reducing toward the north east. The bedrock on first rock contact comprised either weathered shale or weathered sandstone. The bedrock encountered in the boreholes was quite variable, with the typical profile comprising extremely low to very low strength rock within the upper profile, grading to low and medium strength with depth. BH10, BH11, BH13, and to a lesser extent BH14 and BH15, contained an upper layer of high or even very high strength sandstone immediately below the initial weathered sandstone or shale. Within the low to medium strength rock there were also numerous bands or zones of core loss and extremely weathered rock encountered.

**Groundwater**
All boreholes were dry on completion of auger drilling. We note that during the coring process, water is introduced into the borehole and therefore water levels immediately after completion of coring have not been recorded as they would be artificially high. Piezometer standpipes with data loggers have been installed in BH7, BH12 and BH13 for future groundwater measurements. The remainder of the boreholes were backfilled on completion for safety reasons. Further groundwater measurements via downloading of the data loggers is recommended to check groundwater levels with time and changing weather conditions.

**4.3 Laboratory Test Results**
The Atterberg limit tests have confirmed the underlying residual silty clays are of medium to high plasticity, while the moisture content tests on the recovered rock chips obtained during auger proving of the rock are generally consistent with our field assessment of rock strength. Previous Atterberg limit testing by Golder associates on samples taken at the southern end of the main works area also confirm that the underlying residual silty clays are of medium and high plasticity.
Four Day Soaked CBR tests have been carried out on samples of residual silty clay and one sample of silty clay fill. The residual silty clays gave soaked CBR values ranging from 1.5% to 2.5%, while the silty clay fill gave a soaked CBR of 3.5%. These soaked CBR values are quite low, but typical for residual soils derived from the Bringelly Shales. Values as low as 1% were encountered in the previous Golder Associates investigations.

The following table provides a summary of the soil aggression tests completed by Envirolab Pty Ltd. We note that the results in the table below present the results for all our testing across the site. This has been presented as we believe it provides a better understanding of the uniformity of materials and the potential for variability across the site. For specific details reference should be made to the Envirolab Certificate of Analysis No. 160819.

**Summary Table of Envirolab Aggression Testing**

<table>
<thead>
<tr>
<th>Borehole Number</th>
<th>Soil Type</th>
<th>Soil pH (pH Units)</th>
<th>Chloride Content (mg/kg)</th>
<th>Sulphate Content (mg/kg)</th>
<th>Resistivity (ohm.cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH2 (0.7m to 0.95m)</td>
<td>Residual Silty Clay</td>
<td>5.3</td>
<td>630</td>
<td>320</td>
<td>1500</td>
</tr>
<tr>
<td>BH4 (0.5m to 0.95m)</td>
<td>Silty Clay Fill</td>
<td>5.0</td>
<td>580</td>
<td>330</td>
<td>1600</td>
</tr>
<tr>
<td>BH5 (0.5m to 0.8m)</td>
<td>Silty Sand Fill</td>
<td>5.6</td>
<td>370</td>
<td>200</td>
<td>2200</td>
</tr>
<tr>
<td>BH7 (1.5m to 1.95m)</td>
<td>Silty Clay Fill</td>
<td>8.5</td>
<td>10</td>
<td>62</td>
<td>4000</td>
</tr>
<tr>
<td>BH8 (0.5m to 0.95m)</td>
<td>Residual Silty Clay</td>
<td>7.5</td>
<td>440</td>
<td>330</td>
<td>1600</td>
</tr>
<tr>
<td>BH10 (3.0m to 3.3m)</td>
<td>Residual Silty Clay</td>
<td>4.8</td>
<td>1400</td>
<td>400</td>
<td>900</td>
</tr>
<tr>
<td>BH12 (0.5m to 0.95m)</td>
<td>Silty Clay Fill</td>
<td>8.2</td>
<td>480</td>
<td>280</td>
<td>1400</td>
</tr>
<tr>
<td>BH15 (1.5m to 1.95m)</td>
<td>Extremely Weathered Rock</td>
<td>4.8</td>
<td>700</td>
<td>310</td>
<td>1400</td>
</tr>
<tr>
<td>BH16 (1.5m to 1.9m)</td>
<td>Residual Silty Clay</td>
<td>4.8</td>
<td>700</td>
<td>540</td>
<td>1300</td>
</tr>
</tbody>
</table>
The soil aggression tests indicate that all the residual silty clays, the fill and weathered rock samples tested will have a Mild’ classification for both concrete piles and steel piles when assessed in accordance with AS2159-2009 Tables 6.4.2(C) and 6.5.2(C). These results are consistent with results obtained by Golder Associates.

5 COMMENTS AND RECOMMENDATIONS
The following comments and recommendations are of a general nature only, since we have not been provided with specific details of any of the proposed development works. Therefore once further development details are provided, we recommend that we be requested to review these comments and recommendations to confirm that they are consistent and representative for the proposed works.

5.1 Site Classification
Due to the depth of the fill and the likely abnormal moisture conditions as a result of buildings and pavements, we consider that the proposed building areas within the Hospital site will classify as Class ‘P’ in accordance with AS2870-2011 ‘Residential Slabs and Footings’. Therefore all footings will need to be designed by engineering principles.

The use of AS2870-2011 will only be relevant to lightly loaded structures within the scope defined by the code. For such structures, the laboratory testing of the residual silty clay soils indicates that they will likely have characteristic surface movements up to the equivalent of a Class ‘H1’ site under ‘normal’ conditions. Where footings are designed on the basis of AS2870-2011, consideration will also need to be given to the adverse effect on shrink-swell movements from trees.

There is also a moderate depth of fill at some of the borehole locations. It appears from our visual appraisal that the fill is more than likely derived from on-site earthworks activities, as it has generally been assessed to be of medium and/or high plasticity. However during design of any footings systems within areas of fill, the potential for heave of the fill materials (as well as settlement) must be considered, and appropriate allowances made.

If the residual silty clay soils are used as an engineered fill, or if excavations into the residual silty clays are carried out, then it is possible that characteristic surface movements will be greater, and may be closer to Class ‘H2’ type movements. As such further advice from the geotechnical engineers is recommended when design details and levels are known.
Reference should also be made to Appendix B of AS2870-2011, for guidance on appropriate site maintenance, including site drainage and planting of trees and shrubs.

5.2 Excavation Conditions

The following recommendations should be read in conjunction with the ‘Excavation Work – Code of Practise’ by Safe Work Australia (July 2015). At this stage we do not know the extent of any excavation works on the site, and therefore these recommendations have been provided for general guidance on excavation works. Specific advice will be required once details of the proposed works are provided.

Excavation of the fill and residual soils, as well as any extremely weathered shale or sandstone will be readily achievable using the buckets of conventional hydraulic excavators. Where very low to low strength shale or sandstone is encountered it will be able to be ripped using a Dozer (say D7 size) with ripping tyne or by using a ripping tyne fitted to a large hydraulic excavator.

As the excavation depth increases, low to medium strength shale or sandstone bedrock will be encountered, and it is possible that for any deeper excavations, high or very high strength sandstone may also be encountered. Low to medium and medium strength shale and sandstone bedrock may be able to be ripped with a large dozer (say D9 or larger), where it has at least some defects to assist with the ripping process. High and very high strength sandstone will present ‘very hard’ rock excavation conditions and such rock may be effectively “unrippable”. Therefore we expect that excavation through any high or very high strength sandstone or any medium strength sandstone with few defects, will require the use of hydraulic impact hammers.

During the use of hydraulic impact hammers, precautions must be made to reduce the risk of vibrational damage to adjoining structures. At the commencement of the use of hydraulic impact hammers we recommend that some quantitative vibration monitoring be carried out on the adjoining structures or at the boundaries by an experienced vibration consultant or geotechnical engineer to check that vibrations are within acceptable limits.

If during excavation with the hydraulic impact hammers, vibrations are found to be excessive or there is concern, then alternative lower vibration emitting equipment, such as rock saws, rock grinders or smaller hammers may need to be used. The use of a rotary grinder or rock sawing in conjunction with ripping presents an alternative low vibration excavation technique, however, productivity is likely to be slower. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.
We recommend that only excavation contractors with appropriate insurances and experience on similar projects be used. Excavation contractors should be provided with a copy of this geotechnical report, including the borehole logs and point load strength test results, so that they can make their own assessment of suitable excavation equipment.

Groundwater was not encountered during auger drilling of any of the boreholes and therefore during bulk excavation significant groundwater inflows are not expected, provided excavation depths are within 5m or 6m of the existing ground surface levels. Some localised seepage will likely occur at the soil rock interface and through defects within the rock during and immediately following wet weather. During construction we expect that any groundwater seepage will be able to be controlled by conventional sump and pump techniques.

The excavated material will need to be disposed off site and therefore will need to be suitably classified for waste disposal.

5.2.1 Excavation Batters
The excavation of temporary batter slopes may be feasible provided there is suitable space around the perimeter of the excavation.

Temporary batter slopes may be excavated as per the recommendations below and are contingent on the batter slopes being not greater than 6m high and the batter slopes being inspected by a geotechnical engineer at not greater than 1.5m depth intervals. Higher batter slopes would require more specific geotechnical appraisal and advice.

- Temporary batters through the residual clays and all bedrock up to and including very low strength should be battered at not steeper than 1 Vertical (V) in 1 Horizontal (H). Seepage may occur at the soil/rock interface, from defects within the cut face or at the toe of the batter. Where the geotechnical engineers consider that the seepage is causing a higher risk of instability, it may be necessary to flatten batters or to provide some other local support.

- Where low or low to medium strength bedrock is encountered it may be temporarily battered at not steeper than 1V in 0.5H.

- Vertical excavation would be feasible through at least medium strength bedrock,
- Where adverse defects are encountered within temporary batter slopes they would need to be stabilised with rock bolts, shotcrete or other measures approved by the geotechnical engineers.

- Surcharge loads, including adjoining buildings, construction loads etc must be kept well clear of the crest of temporary batters (at least 2H from the crest, where H is the vertical height of the batter slope in metres).

- The underlying Bringelly Shales are particularly prone to erosion. The upper residual soils are also likely to be quite dispersive. Therefore we expect that temporary batter slopes will begin to show some signs of weathering if they are left exposed for extended periods of time. As such even after temporary batter slopes are fully formed, we recommend ongoing monitoring and inspections by the geotechnical engineers to check for any adverse weathering that may affect stability. Additional stabilisation may be required if adverse weathering occurs. We suggest some allowance in the construction budget be made for some rectification or stabilisation of temporary batter slopes with time. As a minimum surface drainage should not be allowed to flow over the crest of temporary batters, and should be directed and discharged in a manner which avoids concentrated flows and erosion.

Where temporary batters are formed, consideration needs to be given to the type of backfill to be used against the permanent basement walls. Uncompacted backfill placed up against basement walls will result in large settlements which can have adverse effects on structures, paving or landscaping supported above. The backfill placed against the permanent basement retaining walls should preferably comprise a uniform sized durable granular material which is surrounded in a geotextile fabric. A capping layer of at least 0.5m thickness of clayey site won material should be placed above the geofabric, to reduce water infiltration. A subsoil ‘agg’ drain surrounded by a geofabric filter sock should also be placed at the base and rear of the basement wall to collect seepage and discharge it to the stormwater system. This type of backfill has the advantage that only nominal compaction is required (such as by the use of a plate attached to the excavator). The alternative (although less preferred) is to use the site won material as backfill, however it will require careful control of moisture content, placement and compaction of material in thin layers, and density testing of each layer to ensure it is placed in a controlled manner as an engineered fill material. Placement and compaction of site won material at the rear of basement walls is difficult and time consuming due to the space limitations. Care should also be taken when compacting fill behind retaining walls, to ensure that compaction stresses do not exceed the design earth pressures. Advice during construction is recommended when the type of equipment proposed is known.
There are cost implications of excavating and disposing of the additional soil from the batters, and importing large amounts of drainage material to backfill the permanent basement walls. The space required to form the temporary batters may also be problematic due to limited storage and construction space. Therefore it may be preferable to install a shoring system to avoid the excavation of the material in the batters and replacement with high quality material.

Where permanent batter slopes are being proposed, the formation will be dependent on the height of the cut and the materials exposed. As a guide we suggest the following general recommendations;

- Permanent batters through the residual clays and all bedrock up to and including very low strength should be battered at not steeper than 1 Vertical (V) in 2 Horizontal (H).
- Permanent batters through low or low to medium strength bedrock should be battered at not steeper than 1V in 1H.

Any permanent batters will need to be fully protected from erosion in the long term, by a suitable and approved erosion protection measure. Suitable measures would include revegetation or shotcrete. Where revegetation is being proposed, consideration should be given to flattening the permanent batters even further than recommended above to assist with initial vegetation and topsoil establishment and provide for ease of maintenance.

### 5.3 Retaining Walls

Where temporary batter slopes are not preferred or cannot fit within the boundary constraints, we recommend that properly designed insitu shoring systems be constructed and installed prior to commencement of excavation. Such a shoring system may also be used as a permanent basement wall if required.

Given the subsurface conditions encountered, we consider that anchored soldier pile walls with shotcrete infill panels are suitable for this site.

Bored piles will be suitable for the soldier piles, however some seepage may occur into bored piles if they are left open for any extended periods of time and this will require pumping of water and thorough cleaning of the base (including removal of any softened material) prior to pouring or more likely the need to pour using tremie techniques.

Piles for the shoring system should be socketed at least 1.0m below the bulk excavation level, including allowances for nearby lift pits, footing and services excavations. Greater embedment may
be required for lateral stability of the shoring system. Deeper shoring systems may need to penetrate high or even very high strength sandstone bedrock which will require the use of large capacity piling rigs. Even with large capacity piling rigs, productivity may be very slow. We recommend that further advice from piling contractors be obtained on the suitability of their equipment to cost effectively penetrate through the required strength of rock.

Temporary lateral support of the shoring system will need to be provided by anchors or internal propping. During excavation, reinforced shotcrete panels should be sprayed progressively with the excavation to support the soil and weathered rock between the piles, such that there is no more than 1.5m of vertical face of material exposed at any one time. It will be necessary to install strip drains with a non-woven geotextile filter fabric behind each panel of shotcrete to dissipate the pore pressures behind the shotcrete. We recommend strip drain be placed at minimum 1.5m centres. We have assumed that the permanent support of the shoring system will be provided by bracing or propping from the floor slabs in the long term.

Where temporary batter slopes are adopted, conventional concrete block retaining walls can be constructed.

An alternative shoring option may be the use of soil nailed walls. While further specific design and construction staging would need to be provided once details are known, soil nail walls are likely to include soil nails drilled at 1.5m horizontal and vertical spacings, with the soil nails installed to a similar length to the height of the excavation.

### 5.3.1 Insitu Shoring Systems – Design Parameters

The following characteristic parameters may be adopted for shoring wall design. The parameters outlined below are on the assumption that a soldier pile wall is constructed and that inspection of the rock face between soldier piles is completed by a geotechnical engineer at not greater than 1.5m depth intervals to check for significant adverse defects.

- Where minor movements of the shoring wall are tolerable, we recommend a rectangular lateral earth pressure distribution of 5H (where H is the depth of excavation in metres).
- Where adjoining structures or movement sensitive services are within a horizontal distance of 2H from the shoring wall we recommend that the magnitude of the rectangular lateral earth pressure be increased to 8H to reduce the risk of adverse deflections.
• Within shales there is always a risk that large continuous defects will be encountered. Therefore although geotechnical inspections at 1.5m depth intervals are recommended, in addition, we also recommend that the structural shoring design be checked for the presence of a 45° sliding wedge of rock with a friction angle of 20° and with soil surcharge above. If such defects are encountered during geotechnical inspections, then additional and or higher capacity anchors may need to be installed.

• Measures should be taken to provide permanent and effective drainage of the ground immediately behind the soldier pile walls. As discussed above, strip drain protected by non-woven geotextile fabric should be used behind the shotcrete panels of soldier pile walls and should be connected into the basement drainage. Although the shotcrete panels will be provided with rear drainage in the form of strip drains, this drainage will essentially only be effective in reducing water pressures from immediately behind the shotcrete facing. Hydrostatic pressures can build up behind wedges of rock some distance back from the wall. Therefore we recommend that hydrostatic pressures based on the groundwater level should still be assumed to apply to the shoring wall design. These hydrostatic pressures are additional to the earth pressure recommendations above. Out of balance hydrostatic pressures will occur during construction and these need to be considered as part of the shoring wall design.

• All surcharge loads affecting the walls (e.g. nearby footings, construction loads and traffic etc) are additional to the earth pressure recommendations above and should be included in the design.

• Anchors should be bonded a minimum of 3m into shale or sandstone bedrock of at least low strength for which we consider that a maximum allowable bond stress of 100kPa and 150kPa may be adopted respectively. The anchor bond length should commence beyond a line drawn up at 45° from the bulk excavation level.

• All anchors should be proof loaded to 1.3 times their design working load and then locked off at about 85% of the working load under the direction of an experienced engineer or construction superintendent, independent of the anchor contractor. Lift off tests should be completed on all anchors about 4 days after lock off to confirm that anchors are holding their load.

• Piles embedded below bulk excavation level into weathered shale or sandstone bedrock of very low strength or low strength may be designed for a uniform passive resistance of 150kPa and 250kPa respectively. The upper 0.5m of the rock socket should be ignored in the passive resistance calculations to account for some disturbance and jointing within the upper shale from the excavation processes.
Shoring wall designs should include an assessment of wall movements during all stages of the excavation and anchoring construction stages. The wall designer should review the wall movements and assess whether such movements will adversely affect any nearby adjoining structures and services.

5.3.2 Permanent Basement Walls and Landscaping Walls
Where temporary batter slopes are adopted and permanent basement walls constructed within the excavation, we recommend that the following characteristic parameters may be adopted for shoring wall design. The following parameters are on the basis of either a properly placed and compacted engineered backfill or backfill comprising a uniform sized durable granular material which is surrounded in a geotextile fabric as discussed in Section 5.2.1 above.

- For cantilever walls where some movement can be tolerated we recommend a triangular lateral earth pressure distribution using an ‘active’ earth pressure coefficient (Ka) of 0.35.
- For cantilever walls which will be propped by floor slabs or where movements are to be reduced, we recommend a triangular lateral earth pressure distribution using an ‘at rest’ earth pressure coefficient (Ko) of 0.6.
- A bulk unit weight of 20kN/m$^3$ may be used for the backfill.
- All surcharge loads affecting the walls (e.g. nearby footings, construction loads and traffic etc) are additional to the earth pressure recommendations above and should be included in the design.
- Measures must be taken to provide permanent and effective drainage of the ground immediately behind the basement walls. We recommend the use of a free draining durable aggregate (such as 20mm size blue metal) with ‘agg’ pipe surrounded by a geotextile at the base and connected to the stormwater drainage system.

5.4 Earthworks
At this stage we do not know the extent of any site earthworks and as such the following should be used as a guide only. The following subgrade preparation is recommended.

- Strip off the existing grass, topsoil, root affected material, and any obvious deleterious fill materials. The root balls of any trees or shrubs should also be fully removed. Stripped materials will not be suitable for re-use as engineered fill and should be stockpiled separately. Such materials may be suitable for re-use within landscaped areas.
• In areas of existing pavements, we recommend to strip off the existing asphaltic concrete surface and the underlying granular roadbase material to expose the underlying soil subgrade. The existing granular roadbase material may be suitable for re-use as a base-course below new pavements, however it will need to be sampled and tested to confirm its quality is consistent with a DGB20 or similar good quality fine crushed rock material. Based on our visual assessment, we consider it is probably unlikely to be suitable as a base-course below new pavements, however it would be suitable for use as an engineered fill to be placed in any areas of poor subgrade.

• The exposed subgrade should be proof rolled with 8 passes of a minimum 10 tonne smooth drum roller to detect any soft or heaving areas. The proof rolling should be carried out in the presence of a geotechnical engineer or experienced earthworks technician. The boreholes have generally indicated that the residual silty clays are of very stiff or hard strength and we do not expect significant areas of heaving subgrade within those areas, unless they are allowed to wet up. However there were some areas where poorly/moderately compacted clayey fill or stiff clays were encountered, and this is expected to require some subgrade stabilisation; such as localised removal and replacement with engineered fill or the use of bridging layers and geogrid reinforcement. The subgrade should be well graded to promote runoff and reduce the risk of water ponding on the surface. If the subgrade becomes wet it may be untraffickable.

• Any areas of heaving subgrade should be locally removed to a competent base and replaced with engineered fill. As discussed above, where poorly compacted clayey fill is encountered as the subgrade, further more specific subgrade improvement may be required and this is best determined in consultation with the geotechnical engineers at the time of construction.

• Engineered fill should comprise a good quality granular material, such as crushed sandstone or the existing granular road-base material, and should be compacted in horizontal layers with a maximum 200mm loose thickness to at least 98% of Standard Maximum Dry Density (SMDD).

• While not preferred, the existing residual soils may also be used as engineered fill, provided they are compacted to between 98% and 102% of Standard Maximum Dry Density (SMDD) and to within ±2% of Standard Optimum Moisture Content (SOMC). If the residual silty clay soils are to be adopted for use as an engineered fill the following needs to be carefully considered.

  (i) Some of the clays have moisture contents greater than the plastic limit and therefore they may require drying out prior to their use as engineered fill, and
Where reactive silty clays are used as an engineered fill, they will undergo greater shrink swell movements with changes in moisture content than the insitu reactive clays. Therefore consideration needs to be given to the affect that greater shrink-swell movements will have on the performance of structures founded above.

- Density testing should be regularly carried out on any engineered fill. Regular density testing in accordance with Level 1 requirements of AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments' are recommended.

- Any of the existing weathered rock excavated from the site would be suitable for use as an engineered fill. However the weathered shales will likely degrade quickly and may well become closer to a silty clay when placed and compacted. Therefore these materials would also then have a relatively low soaked CBR value for pavement design purposes.

Soil may need to be removed from site during earthworks operations or pile drilling. A contamination assessment has been carried out by Environmental Investigation Services (EIS). Reference should be made to their report (Reference E29845KP dated February 2017) for further advice.

5.5 Footings
As the nature of the individual developments has not yet been determined, it is possible that shallow footings or deep piled footings may be used to support some of the structures; although for the main buildings with the higher column loads, piled footings to rock will likely be necessary. If the proposed buildings have basements, then shallow pad footings founded on the underlying bedrock may also be feasible. The following sections discuss the various footing options and our recommendations.

5.5.1 Shallow Footings on Soils
Lightly loaded structures may found on the underlying residual soils. Shallow pad/strip footings or stiffened raft slabs would be feasible, provided the structures are within the scope of AS2870-2011 ‘Residential Slabs and Footings’. These footing systems should then be designed in accordance with that code. Other structures outside the scope of AS2870-2011, will need to be designed on the basis of engineering principles, taking into effect the reactivity of the soils and the site conditions.

Where shallow footings are founded on the residual silty clays of at least very stiff strength, we consider that a maximum allowable bearing pressure of 150kPa would be applicable. The
settlement of shallow pad/strip footings will be dependent on the size of the footing, the strength of the founding material and the depth to any underlying rigid material (such as rock). As a guide, settlements in the order of 1% of the footing width can be assumed to apply, however shrink-swell movements as a result of the reactive clays are likely to be most critical to the design of shallow footings. As discussed in Section 5.1 above soil movements in the range similar to a H1 site are likely to occur.

If adopting shallow footings founded on the residual silty clays, consideration will need to be given to the potential for differential movements between other structural elements that may be founded on the underlying bedrock. We strongly recommend that these structures include good articulation to allow relative movements to occur. Reference should also be made to Appendix B of AS2870-2011 which provides further guidance on foundation performance and maintenance for structures on reactive silty clay soils.

Footing excavations should be inspected by the geotechnical engineer to confirm that a suitable founding stratum is being achieved. Water should be prevented from ponding in the base of footing excavations as this will lead to softening of the base. Any water softened founding material as well as any ‘fall in’ must be removed from the base of footings prior to pouring concrete.

5.5.2 Shallow Footings on Rock

Where bulk excavations expose weathered rock, shallow pad/strip footings founded on the weathered rock would be feasible. Pad/strip footings may be designed on the basis of the recommended end bearing pressures for piled footings outlined in Section 5.5.3 below.

Where Class V and Class IV founding materials are adopted, footings inspections should include a visual appraisal by the geotechnical engineers. Where Class III or better rock is to be adopted as a founding material, then footing inspections should include a visual appraisal of all footings and spoon testing of at least one third of footings. As discussed further below, at this stage additional borehole investigations would be required in order to adopt a founding stratum of Class III or better quality rock.

As for shallow footings on silty clays, water should be prevented from ponding in the base of shallow footings on rock as this will also lead to softening of the founding materials. Footings should be excavated, inspected, cleaned and poured with minimal delay.
5.5.3 Piled Footings on Rock

Columns supporting the main works area, the commercial development, and the bunker have been indicated to have working loads in the order of 12000kN, 5500kN and 2000kN respectively. Therefore for uniformity of support, we recommend that all new footings be taken down to the underlying bedrock. Due to the depth to rock, piled footings will be required unless basement excavations expose rock, or rock is at a shallow depth below any basement excavations.

We consider that bored piles will be feasible, however groundwater seepage will occur into pile holes during drilling and therefore allowance will need to be made for pumping of seepage from the pier holes or pouring concrete by tremie methods. If this is not preferred then consideration could be given to using grout injected piles.

The bedrock generally ranges from extremely low to medium strength, although there are some high and even very high strength bands within the rock profile. Therefore considering the bedrock profile and the likely large diameter piles needed to carry the column loads, the piling will require moderate to large drilling rigs with rock drilling equipment. We recommend that any potential piling contractors be provided with a copy of this geotechnical report and they should be requested to confirm that their equipment is suitable to penetrate the rock and achieve the required depths.

The following table provides our recommendations in regards to the depth and reduced levels for the various classes of rock encountered at each of our boreholes. We have provided only a general rock classification assessment for the Golder Associates cored boreholes BH02 and BH06 (labelled as GBH02 and GBH06 on our Figure 5) at the southern end of the main building area. This is mainly because we have not been provided with the specific rock strength test results. Nevertheless, the results for GBH02 in particular (which is on a similar section line to our Figure 6) indicates reasonably consistent results to our nearby boreholes.

The rock classes presented in the table below are also shown schematically on the attached Graphical Borehole Summaries (Figures 6, 7 and 8). These rock classes are approximate only and will be dependent on footing/pile sizes. Once footing/pile sizes are know, we recommend that the geotechnical engineers be requested to review the designs to confirm that they are suitable from a geotechnical perspective and are consistent with these recommendations.
### Summary Table of Depths for each Rock Class at each Borehole Location

<table>
<thead>
<tr>
<th>Borehole Number</th>
<th>Depths (Reduced Levels) Class V Rock</th>
<th>Depths (Reduced Levels) Class IV Rock</th>
<th>Depths (Reduced Levels) Class III Rock</th>
<th>Depths (Reduced Levels) Class II or better Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH7</td>
<td>5.5m to 8.0m (RL50.2 to RL47.7) Shale</td>
<td>8.0m to 9.0m (RL47.7 to RL46.7) Shale</td>
<td>9.0m to 10.65m (RL46.7 to RL45.05) Shale</td>
<td>10.65m to 12.1m (RL45.05 to RL43.6) Shale + Sandstone</td>
</tr>
<tr>
<td></td>
<td>4.85m to 6.0m (RL50.85 to 49.7) Shale</td>
<td>(REFER NOTE 1)</td>
<td>12.1m to 13.1m (RL43.6 to RL42.6) Shale</td>
<td></td>
</tr>
<tr>
<td>BH8</td>
<td>1.5m to 3.5m (RL51.2 to RL49.2) Shale</td>
<td>3.5m to 4.85m (RL49.2 to RL47.85) Shale</td>
<td>6.0m to 7.4m (RL46.7 to RL45.3) Shale</td>
<td>Not Encountered</td>
</tr>
<tr>
<td></td>
<td>4.85m to 6.0m (RL47.85 to 46.7) Shale</td>
<td></td>
<td>7.75m to 10.4m (RL44.95 to RL42.3) Shale</td>
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<td></td>
<td></td>
<td></td>
<td>11.1m to 14.4m (RL41.6 to RL38.3) Shale (REFER NOTE 2)</td>
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</tr>
<tr>
<td>BH9</td>
<td>5.0m to 8.8m (RL42.9 to RL39.1) Shale</td>
<td>8.8m to 10.1m (RL39.1 to RL37.8) Shale</td>
<td>Not Encountered</td>
<td>10.1m to 13.3m (RL37.8 to RL34.6) Shale</td>
</tr>
<tr>
<td>BH10</td>
<td>3.3m to 5.5m (RL47.8 to RL45.6) Shale + Sandstone</td>
<td>12.7m to 15.5m (RL38.4 to RL35.6) Shale</td>
<td>7.15m to 10.85m (RL43.95 to RL40.25) Shale</td>
<td>5.5m to 7.15m (RL45.6 to RL43.95) Sandstone</td>
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<tr>
<td></td>
<td>10.85m to 12.7m (RL40.25 to RL38.4) Shale</td>
<td></td>
<td>15.5m to 16.25m (RL35.6 to RL34.85) Shale</td>
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</tr>
<tr>
<td>BH11</td>
<td>4.2m to 5.6m (RL46.2 to RL44.9) Sandstone</td>
<td>Not Encountered</td>
<td>Not Encountered</td>
<td>5.6m to 7.5m (RL44.9 to RL43.0) Sandstone</td>
</tr>
<tr>
<td></td>
<td>7.5m to 8.7m (RL43.0 to RL41.8) Sandstone</td>
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<td></td>
<td>8.7m to 14.75m (RL41.8 to RL35.75) (REFER TO NOTE 3)</td>
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<tr>
<td>BH12</td>
<td>2.8m to 10.3m (RL47.1 to RL39.6) Sandstone and Shale</td>
<td>10.3m to 11.5m (RL39.6 to RL38.4) Shale</td>
<td>12.85m to 18.0m (RL37.05 to RL31.9) Shale (REFER TO NOTE 5)</td>
<td>Not Encountered</td>
</tr>
<tr>
<td></td>
<td>11.5m to 12.85m (RL38.4 to RL37.05) Shale (REFER TO NOTE 4)</td>
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<td></td>
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<tr>
<td>BH13</td>
<td>2.8m to 4.0m (RL48.6 to RL47.4) Sandstone</td>
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<td>4.0m to 5.6m (RL47.4 to RL45.8) Sandstone</td>
</tr>
<tr>
<td></td>
<td>5.6m to 6.7m (RL45.8 to RL44.7)</td>
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<td></td>
<td>6.7m to 10.9m (RL44.7 to RL40.5) Shale and Sandstone</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>11.7m to 13.2m (RL39.7 to RL38.2)</td>
<td>10.9m to 11.7m (RL40.5 to RL39.7)</td>
</tr>
<tr>
<td></td>
<td>Sandstone</td>
<td>Shale</td>
<td>Sandstone</td>
<td></td>
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<tr>
<td>-----</td>
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<td>---------------------------</td>
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<td></td>
</tr>
<tr>
<td>BH14</td>
<td>2.5m to 3.5m</td>
<td>11.9m to 13.3m</td>
<td>3.5m to 4.8m</td>
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</tr>
<tr>
<td></td>
<td>(RL48.6 to RL47.6)</td>
<td>(RL39.2 to RL37.8)</td>
<td>(RL47.6 to RL46.3)</td>
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</tr>
<tr>
<td></td>
<td>Sandstone</td>
<td>Sandstone</td>
<td>Sandstone</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.8m to 5.9m</td>
<td>5.9m to 8.3m</td>
<td>9.7m to 11.9m</td>
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</tr>
<tr>
<td></td>
<td>(RL46.3 to RL45.2)</td>
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<td>(RL41.4 to RL39.2)</td>
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<tr>
<td></td>
<td>Shale</td>
<td>Sandstone</td>
<td>Sandstone</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.3m to 9.7m</td>
<td>3.5m to 4.8m</td>
<td>13.7m to 16.3m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(RL42.8 to RL41.4)</td>
<td>(RL47.6 to RL46.3)</td>
<td>(RL37.4 to RL34.8)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sandstone</td>
<td>Shale</td>
<td>Shale</td>
<td></td>
</tr>
<tr>
<td>BH15</td>
<td>1.3m to 5.2m</td>
<td>5.2m to 7.9m</td>
<td>Not Encountered</td>
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</tr>
<tr>
<td></td>
<td>(RL52.6 to RL48.7)</td>
<td>(RL48.7 to RL46)</td>
<td>12.0m to 16.45m</td>
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</tr>
<tr>
<td></td>
<td>Shale and Sandstone</td>
<td>Shale and Sandstone</td>
<td>(RL41.9 to RL37.45)Shale</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7.9m to 9.75m</td>
<td>9.75m to 11.35m</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>(RL46 to RL44.15)</td>
<td>(RL44.15 to RL42.55)</td>
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<tr>
<td></td>
<td>Shale</td>
<td>Shale</td>
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</tr>
<tr>
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<td>2.5m to 3.6m</td>
<td>3.6m to 4.85m</td>
<td>12.5m to 16.65m</td>
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<tr>
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<td>(RL49.7 to RL48.6)</td>
<td>(RL48.6 to RL47.35)</td>
<td>(RL39.7 to RL35.55)Shale</td>
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<td></td>
<td>Shale</td>
<td>Sandstone</td>
<td>Shale</td>
<td></td>
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<tr>
<td></td>
<td>4.85m to 5.75m</td>
<td>11.2m to 12.2m</td>
<td>5.75m to 7.8m</td>
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</tr>
<tr>
<td></td>
<td>(RL47.35 to RL46.45)</td>
<td>(RL41.0 to RL40.0)</td>
<td>(RL46.45 to RL44.4)Shale</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shale</td>
<td>Shale</td>
<td>9.9m to 11.2m</td>
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<tr>
<td></td>
<td>7.8m to 9.9m</td>
<td>(REFER TO NOTE 7)</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>(RL44.4 to RL42.3)</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>Shale</td>
<td>Shale</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GBH02</td>
<td>2.0m to 5.5m</td>
<td>5.5m to 8.0m</td>
<td>Not Encountered</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(RL51.5 to RL48.0)</td>
<td>(RL48.0 to RL45.5)</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Shale and Siltstone</td>
<td>Shale and Siltstone</td>
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<td></td>
</tr>
<tr>
<td>GBH06</td>
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<td></td>
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<tr>
<td></td>
<td>(RL50.7 to RL48.8)</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Shale and Siltstone</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES**

1. Class IV rock from 8.0m to 9.0m in BH7 is based on an assessment of the augered borehole portion and is approximate only. The rock class may vary by one order.
2. Class II Rock in BH8 is separated by core loss and extremely low strength bands. Refer to borehole log.
3. Class II Rock from 8.7m to 14.75m is separated by an extremely weathered band between 12.1m and 12.5m.
4. The Class V rock in BH12 from 5.8m to 7.65m has 1.5m of core loss and a silty clay band. The core loss may also be clay material rather than Class V rock.
5. Class III rock from 3.5m to 4.4m has been largely based on assessment of the augered borehole portion and is approximate only. The rock class may vary by one order.
6. Class IV and II shale in BH14 is separated by an extremely weathered band between 13.3m and 13.7m.
7. Class IV and II shale is separated by core loss and extremely weathered shale from 11.35m to 12.0m.
8. Class IV sandstone in BH16 from 3.6m to 4.85m is based on an assessment of the augered borehole portion and is approximate only. The rock class may vary by one order.
9. Class IV and III shale is separated by a core loss seam from 12.2m to 12.5m.
10. The Class IV rock in GBH02 and the Class III rock in GBH06 are occasional separated by extremely weathered seams.
11. Rock Classifications based on Pells et al 1998 'Foundations on Shale and Sandstone in the Sydney Region'
12. Refer to borehole logs for more specific information.
Based on the above rock classifications, the following table presents our recommendations on maximum allowable end bearing pressures, ultimate end bearing pressures, maximum allowable skin friction values and ultimate skin friction values for the various classes of rock.

### Summary Table of Maximum Allowable and Ultimate End Bearing Pressures and Skin Friction Values

<table>
<thead>
<tr>
<th>Rock Class</th>
<th>Maximum Allowable End Bearing Pressure (kPa)</th>
<th>Ultimate End Bearing Pressure (kPa)</th>
<th>Maximum Allowable Skin Friction (kPa)</th>
<th>Ultimate Skin Friction (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class V Shale</td>
<td>700</td>
<td>1,500</td>
<td>50</td>
<td>70</td>
</tr>
<tr>
<td>Class IV Shale</td>
<td>1,000</td>
<td>3,000</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>Class III Shale</td>
<td>3,000</td>
<td>20,000</td>
<td>250</td>
<td>500</td>
</tr>
<tr>
<td>Class II Shale</td>
<td>4,000</td>
<td>30,000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>Class V Sandstone</td>
<td>1,000</td>
<td>3,000</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>Class IV Sandstone</td>
<td>1,500</td>
<td>4,000</td>
<td>150</td>
<td>300</td>
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<tr>
<td>Class III Sandstone</td>
<td>3,500</td>
<td>30,000</td>
<td>350</td>
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<td>6,000</td>
<td>60,000</td>
<td>600</td>
<td>1,500</td>
</tr>
</tbody>
</table>

### Bunker Area Piles

Only one borehole (BH7) has been carried out in this area. This borehole generally disclosed quite a reasonable founding stratum comprising Class III and II shale at depths below 9.0m. Above that Class IV and V shale was encountered. Based on the single borehole, piling in this area to a founding stratum of at least Class III shale would appear feasible if required. However additional boreholes are recommended once the proposed development is further defined to confirm that the Class III rock is a uniform founding layer below this area of the site.

### Commercial Development Area Piles

Only two boreholes (BH8 and BH9) were drilled in this area. These boreholes showed a drop in the rock level to the east. These boreholes also indicated a reasonably uniform layer of Class II and III rock at depths below 6.0m in BH8 and below 10m in BH9. Therefore it would appear that piling to a founding material comprising at least Class III shale would be feasible, subject to further geotechnical boreholes throughout the site once the specific development details are provided.
Main Works Area Piles

Class III and II rock was encountered in each borehole (BH10 to BH16 inclusive), but was encountered at quite variable depths. It was also found that often there were bands of Class IV and V rock within the class III and II units. At this stage due to the variability in the rock quality, particularly toward the southern end of the main building area we recommend piles (and shallow footings on rock) be designed on the basis of Class IV shale. We do not recommend design of piled footings be based on Class II or III Shale and Sandstone until further geotechnical investigations are carried out to define the depth and continuity of these materials with more certainty.

General Pile Footing Recommendations

We recommend that all piles be founded on and with a minimum embedment of 0.3m into the appropriate quality of rock. In addition to the maximum allowable and ultimate end bearing pressures, piles can also be designed for skin friction. The boreholes indicate bands of poorer quality rock within some of the better quality rock. For founding purposes a single pile must have a thickness of at least 1.5B (where B is the pile diameter) below the toe of the pile and within the required rock class, in order to adopt such a rock class for the founding material. Where pile groups are necessary, a similar 1.5 factor would apply, however this would apply to the minimum width of the pile group. Pile groups would need to be further assessed on a case by case basis.

Where ultimate end bearing and skin friction values are adopted, then the ultimate values recommended in the table above must be reduced by an appropriate geotechnical reduction factor. The geotechnical reduction factor should be based on the risk assessment procedure set out in Table 4.3.2 (A) of AS2159-2009, but should not be greater than 0.5, unless the risk factors producing a higher geotechnical reduction factor can be fully justified. Consideration should also be given to the pile testing requirements when determining a suitable geotechnical strength reduction factor.

In order to achieve the recommended skin friction values nominated in the table above, it is essential that the rock sockets be cleaned of any clay smear and suitably roughened using a side wall grooving tool, and that they be at least as rough as Roughness Class R2. We note that an R2 roughness is equivalent to grooves 1mm to 4mm deep and grooves 2mm wide, which are spaced at 50mm to 200mm down the socket length. It will be the responsibility of the piling contractor to ensure that he has the appropriate equipment and methodology to satisfy this roughness criteria.
Where allowable bearing pressures and skin friction values are adopted, settlement of piles will typically be less than 1% of the pile diameter at the toe of the pile. However where ultimate end bearing and skin friction values are adopted, settlements will be greater and therefore once column loads are known, some detailed settlement analysis of piles is recommended to check that predicted settlements are within acceptable limits.

We recommend that the geotechnical engineers inspect piles during drilling to confirm the above recommended bearing pressures and skin frictions are being achieved. Where the lower quality rock (equivalent to Class IV Shale) is adopted as the founding material, we consider that only a selection of piles will need to be inspected by the geotechnical engineers. However if further investigation allows the use of the higher quality rock (equivalent to Class III or II) for a founding material then all piles should be inspected by the geotechnical engineers. Inspection of piles will require the geotechnical engineer to be on site during the drilling process so that they can inspect both the material being drilled and check it’s consistency with nearby borehole logs. It is important to note that the geotechnical engineers can only ‘sign off’ on piles which they have inspected.

Prior to pouring concrete, piles will need to be dewatered, cleaned of all loose debris from the base, inspected and approved by the geotechnical engineers. Piles will need to be poured as soon as possible after drilling, but at least on the day of drilling. If piles are left open overnight they must be redrilled prior to pouring concrete to remove any softened or other debris from the base of the pile.

5.6 Pavements

Following satisfactory preparation of the subgrade (as detailed in Section 5.4 above), new pavements will need to be designed on the basis of the specific subgrade material. Where a granular fill material is used (such as an engineered crushed sandstone), then soaked CBR’s of at least 5% would be suitable subject to specific testing of the material to be used. Where the subgrade will comprise the existing residual clays, very low soaked CBR’s of 1.5% to 2.5% will apply and therefore we recommend that a select layer of granular material be used to supplement the pavement designs and reduce the pavement thickness.

Flexible pavements should be underlain by a good quality base-course layer comprising crushed rock to RTA QA specification 3051 (2010) unbound base material, or equivalent good quality and durable fine crushed rock compacted to at least 100% of Standard Maximum Dry Density (SMDD).

Concrete pavements should also be underlain by a subbase layer of at least 100mm thickness comprising DGB20 compacted to at least 100% of SMDD. This will reduce the risk of pumping of
fines where clayey subgrades are encountered. Concrete pavements at ground floor level of the car parking structure, should be isolated from the structural columns to allow relative movement.

We recommend that subsoil drains be placed around the perimeter of the new pavements. The subsoil drains should extend to a depth of at least 0.3m below the subgrade level and the drains should have adequate falls to reduce ponding in the drains.

5.7 Earthquake Design Parameters
The following parameters can be adopted for earthquake design in accordance with AS1170.4-2007 ‘Structural Design Actions, Part 4: Earthquake Actions in Australia’:
- Hazard factor (Z) = 0.08
- Site Subsoil Class = Class Ce

6 SALINITY
With reference to the Department of Natural Resource’s 1:100,000 Map of Salinity Potential in Western Sydney the site is located in an area where there is a moderate potential for soil and groundwater salinity to occur. Salinity can affect the longevity and appearance of structures as well as causing adverse horticultural and hydrogeological effects. The local council has guidelines relating to salinity issues which should be checked for relevance to this project.

7 GENERAL COMMENTS
The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, inspection of pile footings etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician.