Report on
Geotechnical Investigation

Proposed Commercial Development
11 Talavera Road, Macquarie Park

Prepared for
Dexus Funds Management Ltd

Project 85751.02
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<td>Huw Smith</td>
<td>Konrad Schultz</td>
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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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Appendix A: About This Report
Appendix B: Drawings
Appendix C: Current Field Results
Appendix D: Historical Field Results
1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed commercial development within the north-eastern portion of an existing business park at 11 Talavera Road, Macquarie Park. The investigation was commissioned in an email dated 17 January 2017 by Ms Denisse Dufey of Dexus Property Group, on behalf of Dexus Funds Management Ltd and was undertaken in accordance with Douglas Partners’ proposal SYD161520 dated 8 December 2016.

It is understood that the project involves demolition of the existing conference centre, tennis court and swimming pool, and excavations to between 15.7 m and 19.6 m (assumed final finished level (FFL) of RL33.35 m) below existing surface levels. It is understood that the current proposal is for the construction of a multi-storey commercial building with eleven above ground levels, one lower ground level and five levels of car park basement, with the excavation footprint extending to within a few metres of the property boundaries.

From the provided architectural drawings, the proposed vehicular entry point to the basement car park entry will be at RL52.85 m, from an internal access road on the southern side of the site, whilst the pedestrian entries to the building will be from the other three sides, including those with frontage to Lane Cove Road and Talavera Road.

A geotechnical investigation was undertaken within the northern and eastern sides of the site to provide information on the subsurface profile for the assessment of excavation and groundwater conditions, and for the design of the basement excavation, shoring systems and foundations. The investigation comprised three deep boreholes cored into the underlying rock and the installation of a standpipe piezometer (Borehole 201: “upper” portion of the site), and laboratory testing of selected rock samples. Details of the field work, together with comments relevant to design and construction, are given in this report.

2. Site Description

The proposed development is located within the north-eastern corner of an existing business park, with the site having frontage to both Talavera Road and Lane Cove Road (refer Drawing 1). The rectangular-shaped site has plan dimensions of approximately 90 m (parallel to Lane Cove Road) by 60 m.

The side property boundaries (parallel to Talavera Road) are oriented at approximately 130 degrees (east of north), relative to Grid North (refer survey plan prepared by RealServe Pty Ltd, reference 61622MN, dated 9/11/2016), and for the purposes of this report Project North has been assumed to be parallel to Talavera Road.
The site steps down to the south towards Lane Cove Road. A two-storey Conference Centre is built on the northern, upper side of the site, and an open-air, fenced tennis court, grassed area and swimming pool are located on the southern, lower side of the site, which is bounded by a high fence. The lower and upper areas of the site are internally linked with two flights of stairs. A disused, stepped, tiled water feature exists between the flight of steps and a fence on the eastern side of the site. The site includes areas of garden on both road frontages, external to the site fence.

Based upon the provided survey levels (RealServe Pty Ltd, reference 61622MN), the relatively flat, paved upper part of the site is at an elevation of RL52.8 m, and the lower, grassed area of the site (including tennis court and swimming pool) is at an elevation of RL49.0 m

Internal access roads and parking spaces are located on the northern and western sides of the site. Grated service pits, assumed to be inspection pits for a stormwater service, were observed in a grassed area within and west of the tennis court, in the southern part of the site near Borehole BH01 (refer Drawing 1).

3. Geology

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 (Geological Survey of NSW: Reference 5) indicates that the site is located near the boundary between the Triassic aged Ashfield Shale, the transitional Mittagong Formation, and Hawkesbury Sandstone. The extension / trace of a north-north-east trending regional structural feature, known as the Yarramalong Syncline, is inferred to extend near to the site, with the site inferred to be located on the eastern limb of the syncline. West-dipping faults and sub-vertical, north-north-east trending intrusive dykes have been mapped in nearby road cuttings (near the Lane Cove Road bridge over the M2 Motorway).

The Ashfield Shale typically comprises black to dark grey shale and laminites, the Mittagong Formation consists of interbedded shale, laminites and fine grained quartz sandstone, and the underlying Hawkesbury Sandstone typically comprises horizontally bedded and vertically jointed, massive and cross-bedded, medium grained quartz sandstone with a few shale interbeds.

The corresponding Sydney 1:100,000 Soils Landscape Series Sheet, prepared by the former NSW Department of Land and Water Conservation, indicates that the soils developed over most of the site belong to the Glenorie soil landscape, whilst the soils developed over the north-eastern corner of the site belong to the Lucas Heights soil landscape. Descriptions of each of these soil landscapes follow, however it is noted that due to past excavation and filling activities associated with prior developments, much of the residual soil profile (overlying the shale) has been extensively modified or removed.

The Glenorie soil landscape is described as shallow to moderately deep (<1.0 m) red podzolic soils on crests and moderately deep (0.7 m-1.5 m) red and brown podzolic soils on upper slopes, with a high soil erosion hazard, localised impermeable highly plastic and moderately reactive subsoil. The Lucas Heights soil landscape is described as moderately deep (0.5 m-1.5 m), hardsetting yellow podzolic soils and yellow soloths, with stony, low fertility soils with low available water capacity.

The 1:25,000 Acid Sulphate Soil Risk map for Prospect / Parramatta River indicates that the site does not lie within an area known for acid sulphate soils. Other references suggest that the site is not within an area known for soil salinity issues.
The drilling confirmed the presence of the geological formations described above.

4. Previous Investigations

Previous geotechnical investigations for the existing Conference Centre development were undertaken and reported by Douglas Partners Pty Ltd (DP) in February 2000 (DP report 28356). These previous investigations comprised four boreholes (with two of these cored into the underlying rock), drilled to depths of between 6.5 m and 9.8 m. These previous test locations are included on Drawing 1. Copies of the previous borehole logs are reproduced in Appendix D.

Additional intrusive investigations for a contamination assessment were also recently completed (refer Drawing 1: boreholes BH01 to BH05), and are reported under separate cover (DP report 85751.03.R.001, February 2017). A desktop geotechnical assessment for the site was also recently completed (DP report 85751.00, January 2017).

Subsurface conditions encountered during the current investigation are generally consistent with those encountered and previously reported by DP, with the more recent investigations being taken to deeper levels.

5. Current Field Work Methods

The current geotechnical field work comprised the drilling of three boreholes (Boreholes BH 201 to BH 203) with a bobcat-mounted drilling rig. The locations of the three boreholes, along with the five hand augered boreholes excavated as part of the environmental investigations, are shown on Drawing 1 in Appendix B. In conjunction with site measurements, a geo-referenced electronic survey drawing provided by the client (prepared by RealServe Pty Ltd, file name “North Ryde - 11 Talavera Rd - Level & Feature Rev A – MGA.dwg”) was utilised to obtain borehole co-ordinates and surface levels. The accuracy of these co-ordinates is considered to be about 100 mm in both plan and elevation.

The boreholes were initially progressed in soils using an 110 mm diameter spiral flight auger and rotary wash boring methods to depths of between 3.0 m and 6.4 m. The boreholes were subsequently cased and then extended into the underlying rock to depths of between 20.35 m and 23.0 m using NMLC (52 mm diameter) diamond coring equipment.

Standard Penetration Tests (SPTs) were generally carried out at regular depth intervals in the soils to assess soil consistency and composition. Diamond core drilling recovered 52 mm diameter samples of rock strata, with the strength of the cored rock assessed by examination of the recovered rock cores and laboratory Point Load Strength Index (ls\textsubscript{50}) tests. Further details of the methods and procedures employed in the investigation are presented in the standard Notes About This Report, included in Appendix A.
A standpipe piezometer was installed in BH 201, for monitoring of groundwater levels subsequent to the field work, and for the future collection of groundwater samples for contamination testing purposes. The standpipe was installed to a depth of 5.25 m, and screened between 1 m and 5.25 m.

6. Current Field Work Results

The subsurface conditions encountered in the boreholes are presented in the borehole logs in Appendix C, along with notes defining the descriptive terms and classification methods used.

The subsurface conditions encountered in the boreholes can be summarised as:

- **TOPSOIL** – sandy silt with some gravel, with thicknesses of between 0.2 m to 0.3 m (garden bed only);
- **FILLING** – clay and crushed shale filling with some sand and gravel, to depths of between 1.5 m and 2.0 m;
- **CLAY** – very stiff with some medium strength iron-cemented bands, grey / light grey mottled orange / red, 1.5 m to 2.5 m thick (elevations of between RL44.9 m to RL48 m); then
- **SHALE** – extremely low to very low strength in each borehole, to depths of between 3.76 m to 9 m (RL43 m to RL45.1 m);
- **('Upper') SANDSTONE** – extremely low strength, to high strength, with measured thickness of between 2.64 m and 4.3 m (1.24 m thick in BH 201, positioned between interlaminated siltstone / sandstone and laminite), with extremely low strength clay bands at the upper and lower interfaces of the unit;
- **INTERLAMINATED SILTSTONE AND SANDSTONE / LAMINITE** – low to high strength, with a fine grained sandstone at the base of the unit, with measured thickness of between 4.3 m and 5.4 m; and
- **('Lower') SANDSTONE** – medium to coarse grained, medium to high strength, encountered at elevations of between RL33.4 to RL38.2 m.

The elevations of the top of rock for geotechnical investigations within the site are presented on Drawing 2. Two geotechnical cross-sections have been prepared, running adjacent to and parallel with the southern property boundary (Section A-A') and the eastern property boundary (Section B-B'), and are presented as Drawings 3 and 4 in Appendix B – refer to Section 8. Core loss within the boreholes is interpreted to represent extremely low strength bedrock, ground up and washed away during the coring process.

A feature of the recovered rock cores is numerous smooth and clay coated joints (with dips between 30° and 45°), clay seams within the interlaminated siltstone / sandstone, laminite and ‘upper’ sandstone, and sheared seams 10 mm to 100 mm thick. Clay coated joints were also encountered in the underlying ('lower') sandstone. In addition, the presence of extremely low strength seams on the upper and lower sides of the ‘upper’ sandstone are inferred to be the upper and lower boundaries of a geological fault.

Some zones of steeply dipping to sub-vertical, planar or undulating joints were encountered within the ‘lower’ sandstone (e.g. BH 202 between 13.8 m to 16.0 m depth).
Based upon the three boreholes, the proposed bulk excavation level (RL36.35 m) will be partly within the interbedded materials and partly within the sandstone, inferred to be of medium and high strength.

Groundwater was not observed in the boreholes during auger drilling (including the boreholes drilled for the Conference Centre, Bores 101 to 104), and the use of water as a flushing medium for rotary drilling prevented further groundwater observations with depth.

A groundwater standpipe piezometer was installed in BH 201, comprising screened PVC pipe with gravel backfill, a bentonite pellet seal and ‘gatic’ cover at ground level (refer to Borehole Log for specific details). The standpipe was flushed upon completion and developed on 3 February 2017, with subsequent measurement of the groundwater level in the standpipe undertaken on 6 February 2017 (5 days after completion).

The groundwater measurement and standpipe construction details is summarised in Table 1. A summary of the soil and rock profiles is given in Table 2.

<p>| Table 1: Details of Installed Standpipe Piezometer and Groundwater Measurement |</p>
<table>
<thead>
<tr>
<th>Borehole</th>
<th>Screened interval (metres depth)</th>
<th>Screened interval (RL range)</th>
<th>Water level during drilling(^1) (RL)</th>
<th>Measured water level (RL), 8/02/2017</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH 201</td>
<td>1.0 – 5.25</td>
<td>51.0 – 46.8</td>
<td>&lt;48.0</td>
<td>47.4</td>
</tr>
</tbody>
</table>

Note: \(^1\)Drilling was completed on 1 February 2017.

<p>| Table 2: Summary of Soil and Rock Profiles in Boreholes |</p>
<table>
<thead>
<tr>
<th>Stratum</th>
<th>Level of top of stratum (m AHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>101</td>
</tr>
<tr>
<td>Filling / Topsoil</td>
<td>52.5</td>
</tr>
<tr>
<td>Clay</td>
<td>50.3</td>
</tr>
<tr>
<td>EL to VL Shale / Siltstone</td>
<td>46.9</td>
</tr>
<tr>
<td>L Shale / Siltstone</td>
<td>45.7</td>
</tr>
<tr>
<td>M Shale / Siltstone</td>
<td>ne</td>
</tr>
<tr>
<td>M to H Laminite(^{1,2})</td>
<td>44.9</td>
</tr>
<tr>
<td>M and H Sandstone (fine grained)</td>
<td>-</td>
</tr>
<tr>
<td>M and H Sandstone (coarse grained)</td>
<td>-</td>
</tr>
<tr>
<td>End of borehole</td>
<td>43.9</td>
</tr>
</tbody>
</table>

Notes: “ne” indicates this material was not encountered, except as thin bands / layers; “-“ indicates the borehole was not extended through this material; Strength descriptors: EL = extremely low, VL = very low, L = low, M = medium, H = high; \(^1\) includes rock described in the borehole logs as Interbedded Sandstone and Siltstone; \(^2\) includes an interval of extremely low to high strength, medium to coarse grained sandstone.
7. Laboratory Testing

Point load strength tests (axial only) were carried out on 49 samples selected from the better quality rock core for axial point load strength index ($I_{s0}$). The results are reported on the borehole logs and ranged between 0.2 MPa and 2.6 MPa, indicating rock ranging from low to high strength classification. To obtain inferred unconfined compressive strengths (UCS) from point load strength test results, a conversion factor of 15 to 20 is often used, indicating a UCS of up to about 50 MPa.

8. Geotechnical Model

The geotechnical model for the site is characterised by a shallow thickness of topsoil and filling (between 1.5 m and 2 m thick), over medium to high plasticity residual clay of very stiff to hard consistency (between 1.5 m and 2.5 m thick: extending to between 3 m and 4 m depth below existing surface levels), with or without some ironstone bands of higher strength.

These soils overlie 0.8 m to 4 m of extremely weathered, extremely low strength shale with an apparent dip to the north-west, over highly to moderately weathered, very low to medium strength rock (interlaminated siltstone / sandstone, and laminite). The interlaminated materials (including laminite), generally corresponding with the Mittagong Formation, appears to thicken to the north-west (encountered thickness 4.3 m to 5.4 m). It also has an apparent dip to the north-west. A fractured, medium to coarse grained sandstone unit (‘upper’ sandstone) of variable strength is stratigraphically above the laminite, with the extremely low strength clay bands at the upper and lower interfaces of the unit inferred to be fault planes and planes of weakness. Medium to coarse grained sandstone (inferred to be the Hawkesbury Sandstone), lies stratigraphically below the laminite.

The geotechnical model for the site is presented as two cross-sections in Appendix B: Section A-A’ - Drawing 3, and Section B-B’ - Drawing 4. The alignment of the sections has been selected to be parallel to the axes of the site, and to pass through the recently drilled cored boreholes. It is noted that the geological interpretation between the boreholes could vary from that shown on the cross-sections.

The rock materials encountered in the geotechnical boreholes have been classified in accordance with the procedures given in Pells et. al. (1998: Reference 7), and Bertuzzi and Pells (2002: Reference 4). The interpreted depth and reduced level at the upper surface of the various bedrock classes is shown in Table 3. It should be noted that the profiles are accurate at the borehole locations only, and that variations must be expected away from the boreholes. The strata units or layers have been shown on the cross-section as inferred strata boundaries only.
Table 3: Summary of Geotechnical Model

<table>
<thead>
<tr>
<th>Stratum</th>
<th>BH 101</th>
<th>BH 102</th>
<th>BH 103</th>
<th>BH 104</th>
<th>BH 201</th>
<th>BH 202</th>
<th>BH 203</th>
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<tr>
<td>Top of borehole</td>
<td>(52.5)</td>
<td>(52.2)</td>
<td>(51.8)</td>
<td>(51.6)</td>
<td>(52.0)</td>
<td>(48.9)</td>
<td>(48.9)</td>
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<tr>
<td>Shale – Class V</td>
<td>5.7</td>
<td>5.5</td>
<td>4.0</td>
<td>6.0</td>
<td>4.0</td>
<td>3.0</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>(46.9)</td>
<td>(46.7)</td>
<td>(47.8)</td>
<td>(45.6)</td>
<td>(48.0)</td>
<td>(45.9)</td>
<td>(44.9)</td>
</tr>
<tr>
<td>Shale – Class IV</td>
<td>6.8</td>
<td>-</td>
<td>6.8</td>
<td>-</td>
<td>5.3</td>
<td>3.8</td>
<td>5.8</td>
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<tr>
<td></td>
<td>(45.7)</td>
<td>-</td>
<td>(44.7)</td>
<td>-</td>
<td>(46.7)</td>
<td>(45.1)</td>
<td>(43.1)</td>
</tr>
<tr>
<td>Shale – Class III</td>
<td>7.66</td>
<td>-</td>
<td>8.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>6.4</td>
</tr>
<tr>
<td></td>
<td>(44.9)</td>
<td>-</td>
<td>(43.7)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>(42.5)</td>
</tr>
<tr>
<td>Laminate / Sandstone</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>14.4</td>
<td>8.2</td>
<td>9.8</td>
</tr>
<tr>
<td>– Class I/II</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(37.6)</td>
<td>(40.7)</td>
<td>(39.1)</td>
</tr>
</tbody>
</table>

Notes: (1) Depths are in metres (elevations are in m AHD), "-" = not encountered
(2) Rock Classification based on Pells et. al (1998), and Bertuzzi and Pells (2002).
(3) The intervals of Class III and Class IV Shale also includes the "interlaminated Siltstone and Sandstone”

In the process of preparing the rock classes and geotechnical model, some of the encountered rock classes have been downgraded due to significant weak seams, and that bands of higher strength rock do occur within rock of lower strength.

9. Comments

9.1 Proposed Development

Based upon the architectural drawings prepared by Rice Daubney Pty Ltd, dated 9 March 2017, it is understood that the proposed development will include the demolition of the Conference Centre, swimming pool and tennis court, and the construction of a new commercial building with eleven above ground levels, one lower ground level and five levels of basement car parking. The lowest basement has a finished floor level (FFL) of RL33.35 m, with the basement excavation extending to within a few metres of the property boundaries. This level is shown on Drawings 3 and 4 in Appendix B.

Based upon the available information, excavation for the basement will be required to depths of between 15.7 m and 19.6 m below existing surface levels.

9.2 Site Preparation and Trafficability

It is anticipated that the proposed bulk excavation will encounter filling and residual clay, extremely low to medium strength shale, interlaminated siltstone and sandstone and laminitite, and medium to high strength sandstone.
The insitu clayey filling materials may heave under the applied loading of construction vehicles with tyres, posing challenges to such plant and vehicles. It is anticipated that tracked machines will be able to safely traverse and work upon this material while it is exposed, though it would be prudent to incorporate a rockfill layer of at least 300 mm thickness over these materials to enable “all-weather” access for trucks. The thickness of (rockfill) working platforms for cranes and tracked piling rigs will generally require specific geotechnical assessment.

9.3 Excavation

Following demolition of the existing buildings and fences, and removal of concrete slabs (e.g. tennis court, disused water feature along eastern boundary), excavation for the building and basement level is expected to be required through up to 6 m of topsoil, filling and clay soil (with the potential to encounter some ironstone bands of locally higher strength), then rock of varying strength, including high strength sandstone. The buried utility search indicates that there could be conflicts with existing services passing beneath the southern part of the site.

The filling soils should be readily excavated using conventional earthmoving equipment, however, the use of heavy ripping equipment or rock hammers will be required to excavate medium strength and stronger rock. Based upon the cored boreholes, a few lower strength or fractured bands are present within the high strength rock units, which may aid extraction.

Rippability of the sandstone is critically dependent upon the spacing of bedding and vertical joints, as well as on strength. Effective removal of the medium or higher strength sandstone within the lower levels of the excavation should be achieved by heavy bulldozers or excavators with rippers and rock hammers to excavate to the required bulk excavation levels, however, excavation contractors should make their own assessment of likely productivity depending on their equipment capabilities and operator skills. Detailed footing excavations adjacent to boundary lines can be achieved by use of hydraulic rotary rock saws, or milling heads. Rock saws could also be used along the site boundaries to minimise over break.

The effect of the excavation on the foundation systems of nearby buildings within the inferred “zone of influence” (i.e. Building A) should also be considered, with the shoring design to minimise ground movements at the building’s foundation level. It is understood that the footings on the southern side of Building A are high level footings founded on extremely low strength shale (to be confirmed).

Any off-site disposal of material will require assessment for re-use or classification of the soil in accordance with Environmental Guidelines: Assessment, Classification and Management of Non-Liquid Wastes (NSW EPA, 2014: Reference 6), prior to disposal to an appropriately licensed landfill. Refer to DP report 85751.03 (February 2017) for further information.

9.4 Excavation Support

9.4.1 General

It is understood that excavation at this site will range to between 15.7 m and 19.6 m below existing surface levels.
As indicated on the supplied drawings, excavations for the basement levels will be required beneath the entire footprint of the proposed building, to setbacks of a few metres from each property boundary.

Inclined joints and an inferred inclined fault zone (apparent dip to the north-west) were observed in the rock cores, predominantly within the laminite / interlaminated siltstone and sandstone. In the Sydney region the orientation for the joints is usually north-south (±10° - 15°), or east-west (±10° - 15°), being oriented sub-parallel to both Lane Cove Road and Talavera Road. It is therefore expected that some narrow wedges will be formed where these near vertical joints intersect the excavation faces. If these features are exposed at unfavourable orientations during excavation, rock bolts or even rock anchors may be required to stabilise these feather edges.

Where space permits, it is usually most practical to batter the slopes of excavations, as vertical excavations in filling, soil and weathered shale will not remain stable for an extended period. In such circumstances, the sides of the excavation within stiff to very stiff clay (extending to around 6 m depth) would be expected to remain stable only with batters not exceeding 1.5H:1V during construction, and in the longer term with batters not exceeding 2H:1V. Note that with protection such as covering with Fortecon plastic or shotcrete, to prevent drying out and then rainwater ingress, it may be possible to steepen these short term slopes to 1H:1V. Based upon the proposed setback distances, however, and based on the proposed excavation depth, proximity to buried services and nearby major roads, it is considered impractical to batter the slopes of the entire excavation because these batters would cross the site boundaries. The sides of the excavation will therefore require lateral support during excavation and as part of the final construction.

In view of the depth of the proposed excavation, it is considered that temporary support would be required during construction in the form of a soldier pile shoring wall, spaced at approximately 2 m to 2.5 m centres, with the panels between the piles to be progressively shotcreted in lifts of approximately 2 m as excavation proceeds to reduce the risk of local slippages and collapse between piles. As the piles provide passive resistance only, it will also be necessary to install temporary anchors to minimise deflections. The piles should be taken to below bulk excavation levels.

Closer spacing of piles may be required to reduce wall movements, or prevent collapse of filling materials, particularly where pavements, structures or buried services are located in close proximity to the excavation.

With the consent of adjacent property owners and with due regard given to the locations of underground services, installation of temporary ground anchors will be required to prevent excessive lateral deformation of shoring/retaining walls, in conjunction with the passive resistance of the soldier piles. For the permanent situation, the basement structure usually provides the required lateral support to the perimeter excavation once the temporary anchors are de-stressed.

For an excavation of up to about 12 m below the top of rock, some inward horizontal movement due to stress relief effects could be expected. It is impracticable to provide restraint for any relatively high in-situ horizontal stresses present within the Hawkesbury Sandstone. Release of stresses due to the excavation may generally cause horizontal movement along the rock bedding surfaces and partings, however, due to previous faulting occurring in the area it is possible that stress relief will be minimal.

Stress-relief related movements can cause damage to adjacent buildings. It is recommended that appropriate allowance also be made for the repair of pavements and public utilities, where excavations are carried out close to such structures.
Regular monitoring of survey targets along the excavation perimeter during construction, such as following each successive ‘drop’ in excavation level, should be undertaken to monitor the effects of stress relief and any wall movements. The wall designer should predict the expected movements, and if monitoring suggests higher movements are occurring, a review of the design / construction methodology should be undertaken.

9.4.2 Design

Excavation faces retained either temporarily or permanently will be subjected to earth pressures from the ground surface down to the top of medium strength rock. The lateral pressure distribution on a multi-anchored or braced wall is complex and for preliminary design purposes a trapezoidal distribution with depth could be assumed, with the magnitude of total load estimated from:

\[ \delta_h = K_a \cdot \gamma \cdot Z \]

where
- \( K_a \) = Active coefficient of earth pressures, where some movement is acceptable (refer Table 4)
- \( K_0 \) = Coefficient of Earth Pressure at rest, where movement needs to be restrained (refer Table 4)
- \( \gamma \) = Unit weight of soil (kN/m³; refer Table 4)
- \( Z \) = Depth in metres below the top of the excavation

### Table 4: Recommended Design Parameters for Shoring Systems

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight (kN/m³)</th>
<th>Earth Pressure Coefficient</th>
<th>Ultimate Passive Earth Pressure (kPa)</th>
<th>Effective Cohesion ( c' ) (kPa)</th>
<th>Effective Friction Angle (Degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Active ( K_a )</td>
<td>At Rest ( K_0 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filling / Topsoil</td>
<td>20</td>
<td>0.4</td>
<td>0.6</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>Clay</td>
<td>20</td>
<td>0.3</td>
<td>0.5</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>EL to L Shale / Siltstone</td>
<td>22</td>
<td>0.2¹</td>
<td>0.25¹</td>
<td>400</td>
<td>10</td>
</tr>
<tr>
<td>M Shale / Siltstone</td>
<td>22</td>
<td>0.2¹</td>
<td>0.25</td>
<td>2000¹</td>
<td>20</td>
</tr>
<tr>
<td>M to H Laminite</td>
<td>24</td>
<td>0¹</td>
<td>0¹</td>
<td>2500¹</td>
<td>30</td>
</tr>
<tr>
<td>M and H Sandstone</td>
<td>24</td>
<td>0¹</td>
<td>0¹</td>
<td>3000¹</td>
<td>30</td>
</tr>
</tbody>
</table>

Note: Strength descriptors: EL = extremely low, VL = very low, L = low, M = medium, H = high; ¹ Providing adverse jointing is not encountered

The design of the shoring should allow for all surcharge loads, including building footings, inclined slopes behind the wall, traffic and construction related activities.

If two or more rows of anchors are used then the system should be designed on a trapezoidal pressure distribution using 4H kPa for an active pressure distribution, or 6H kPa to prevent excessive movement, where \( H \) is the depth of the shoring. The load could be assumed to act over the central 80% of the wall. For detailed design of excavation shoring, it is suggested that computer programs such as WALLAP, FLAC or PLAXIS are used to check movement and forces.
The design of temporary and permanent support will need to consider the possibility that 45° joints in the materials above the ‘lower’ sandstone will daylight within the excavation, leading to large wedges of rock requiring support by the temporary and permanent retaining structures. Sufficient anchoring of the shoring wall should be undertaken to prevent movements along 45° joints, even though there is a low probability that a joint would run the full length and height of the excavation.

The anchors should have their bond lengths behind the projected 45° line from the bulk excavation level and should provide sufficient force to resist the movement of a wedge of rock projected at 45° from just below the anchor to the ground surface. The frictional resistance of the wedge along the joint may be calculated assuming an angle of friction of 20°. It is suggested that preliminary design be carried out such that the support system has a factor of safety of 1.1 against the ultimate sliding force along the most unfavourable 45° joint.

Regular rock-face inspections will be required during excavation to determine whether the assumed factor of safety is adequate. Additional anchors may be required to increase the factor of safety if large wedges are observed during excavation.

Shoring walls should also be designed for full hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided. Drainage could comprise 150 mm wide strip drains pinned to the face at 2 m centres behind shotcrete in-fill panels. Additional drainage measures will be required along the faulted zone of rock, due to the anticipated greater volume of seepage inflows along this zone. It is noted that the base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain connected to the stormwater drainage system.

To estimate the passive resistance of the piles, it is suggested that an ultimate passive pressure of 3000 kPa is adopted for medium to high strength rock over any “toe-in” length developed at the base of the piles, from about 1 m below the base level of the excavation, or other excavation adjacent to the wall. The ultimate passive pressures adopted should incorporate a suitable factor of safety of at least 2.0 to limit deflection.

### 9.4.3 Ground Anchors

Where necessary, lateral earth pressures acting on the rear of a pile shoring wall may be resisted by a combination of declined “tie-back” ground anchors and the passive resistance of the soldier piles. Anchoring of soldier piles can be accomplished by prestressed-type strand or bar anchors. It is suggested that anchors be declined as steeply as possible, but not exceeding 30° below the horizontal, to allow anchoring in the stronger rock at lower levels (i.e. medium and high strength sandstone at depth).

For estimating purposes, the ultimate bond stresses at the grout-rock interface provided in Table 5 could be used to design temporary ground anchors.
Table 5: Typical Ultimate Bond Stresses for Anchor Design

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Ultimate Bond Stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay (very stiff and hard)</td>
<td>60</td>
</tr>
<tr>
<td>Low strength Shale / Laminite¹</td>
<td>200</td>
</tr>
<tr>
<td>Medium strength Shale / Laminite¹</td>
<td>600</td>
</tr>
<tr>
<td>Medium and High strength Laminite¹</td>
<td>1,000</td>
</tr>
<tr>
<td>Medium and High strength Sandstone</td>
<td>1,000</td>
</tr>
</tbody>
</table>

Notes: (¹) includes rock described in the logs as Interlaminated Sandstone and Siltstone

Most anchoring contracts are, however, “performance contracts” in which the anchoring contractor designs and constructs the anchors to carry the design loads. Therefore, it is the contractor’s responsibility to ensure that the correct design values specific to the anchor system, rock type and strength, and method of installation are used, and that each anchor is properly constructed and tested.

Care should be exercised in construction to ensure that anchors are installed progressively during excavation, and stressed, with installation of shotcrete carried out at regular intervals prior to excavation of the next row of panels. Ground anchors should be designed to have a free length equal to their height above the base of the excavation (i.e. bonded behind the 45° line) and have a minimum 3 m bond length. After anchors have been installed, it is recommended that they be tested to 125% of the design working load and locked-off at no higher than 80% of the working load. Periodic checks should be carried out during the construction phase to ensure that the lock-off load is maintained and not lost due to creep effects. Note that stress relief-related movement may lead to an increase in anchor stresses.

The parameters given in Table 5 assume that the anchor holes are clean and adequately flushed, with grouting and other installation procedures carried out carefully and in accordance with good anchoring practice. Careful installation and close supervision by a geotechnical specialist may allow increased bond stresses to be adopted during construction, subject to testing.

In normal circumstances, the building will support the basement excavation over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors would require careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.

The installation of ground anchors would require the consent of adjacent property owners and due regard given to the location of underground services.

9.5 Groundwater

Groundwater was not observed during auger drilling within soil and extremely weathered shale, and the introduction of water for rotary drilling precluded further measurements in most boreholes. A water level of 4.6 m depth (RL 47.4 m) was subsequently measured in a standpipe installed in Borehole BH 201, screened in the soil and extremely weathered rock. It is also noted from the geotechnical desktop study (DP report 85751.00.R.001.Rev1, dated January 2017) that a groundwater level of
RL47.8 m was measured (during March 2001) within a standpipe piezometer installed adjacent to the site.

Seepage along the top of the rock, through the rock substance and from rock defects has been observed within these rock materials in Sydney. It is noted that the basement car parking levels are likely to lie below the indicated shallow water table, however this water level may be a perched condition.

More rapid rates of groundwater seepage have also been observed, within both road cuttings and basement excavations nearby, from the inferred faulted zone of rock which is interpreted to pass through the proposed excavation. Testing of the groundwater ‘level’ (i.e. piezometric head) within the faulted zone, and its hydraulic permeability has not been undertaken, and should be confirmed at a later stage of the project.

At this stage it is not possible to accurately estimate the likely extent and rate of seepage, although it is anticipated that seepage volumes will be relatively low (less than 3 ML/year) given the expected low permeability of the rock mass. The possible additional groundwater inflows from the faulted zone may increase these seepage volumes considerably.

Excluding the possible contribution from the faulted zone, seepage inflows through the rock mass and from defects are usually readily handled by sump and pump de-watering measures, with dewatering also likely to be required prior to placing concrete in foundation excavations and also after wet weather. Confirmation of whether additional dewatering measures are required for the faulted zone will be provided at a later stage of the project, following additional testing.

It will be necessary to provide for ample drainage of the excavation during construction, and at the same time to make provision for sub-floor drainage below the lowest basement floor level to effectively prevent hydrostatic pressures developing on the underside of the slab and on the walls of the basement, if the slab is designed for drained conditions. This could comprise a minimum 100 mm thick, durable open graded crushed rock with subsurface drains and sumps.

It is suggested that monitoring of flow during the early phases of excavation be undertaken to assess long term pumping requirements. Grouting of open joints and partings may be necessary to reduce the flow rate to 3 ML/year if excessive water ingress is an issue during excavation.

Previous experience indicates that the groundwater from both the rock units encountered at the site can have moderate concentrations of dissolved solids, including iron. Once groundwater comes into contact with the atmosphere, precipitation of iron oxides is likely to occur and provision should be made for the filtering and/or cleaning of this precipitate from subsoil drains, sumps, pumps and other fittings over the medium to longer term.

Based upon the groundwater observations and ground conditions encountered during the investigation, the groundwater drawdown effects on adjacent properties are likely to be negligible.

9.6 Vibration Control

Noise and vibration will be caused by excavation work on the site. The use of rock hammers will cause vibrations which, if not controlled, could possibly result in damage to nearby structures and
disturbance to occupants. It is suggested that vibrations be provisionally limited to a peak particle velocity (PPV) of 8 mm/s at the foundation level of nearby buildings to protect their architectural features and to reduce discomfort for the occupants. This level complies with AS/ISO 2631.2 – 2014 (Reference 2) and is well below the normal building damage threshold level, however, it can be disturbing to occupants. The owners of any in-ground utilities on and around the property should also be consulted in regards to vibration levels.

Vibration monitoring carried out by DP at various excavation sites within the Sydney area has indicated that to limit vibrations (PPV) to 8 mm/s, a 500 - 1000 kg or 1000 - 2000 kg hydraulic rock hammer should not be used within 8 m or 15 m (respectively) from the building foundation or utility in question.

If vibrations are a potential problem, consideration should be given to rock sawing and rock milling methods of excavation. A site specific vibration monitoring trial is suggested during initial excavation of rock, to verify vibration levels and the effectiveness of rock saw cutting in reducing vibration.

It is also recommended that dilapidation surveys be carried out on adjacent properties including structures, pathways, walls or roadways within about 30 m of the proposed excavation, prior to commencement of the works. The dilapidation survey should document existing conditions and the presence of defects and thereby allow appropriate responses should any claims arise from construction at this site. Buildings supported on shallow foundations are especially prone to the detrimental effects of settlement and vibration.

9.7 Foundations

The consistency of the residual clays above the weathered shale was found to be generally very stiff to hard during the current investigation. A maximum allowable bearing pressure of 150 kPa is recommended for the design of shallow foundations in the very stiff to hard clay. Foundations for structures bearing on the residual clay could be designed on the basis of AS 2870 – 2011 (Reference 3) for a “Class H1” site.

Given the intended depth of the basement car park, it is considered unlikely that anything other than ancillary structures such as outbuildings for transformers or similar would be required to be founded on the residual clays. Soil movements due to moisture variations should also be considered for footings founded in clays.

The lowest confirmed basement level is RL33.35 m. Based upon the geotechnical model, and as shown on Drawings 3 and 4, the exposed rock at basement level in the south-eastern corner of the site is expected to be medium to high strength, Class I/II sandstone, whilst the rest of the exposed rock is expected to be medium to high strength, Class I/II laminite.

Foundations for the structure should be taken to a uniform founding stratum, such as Class I/II sandstone. On the basis of the materials anticipated at the bulk levels, spread footings (i.e. pad or strip footings) should be suitable for supporting the proposed building loads within the excavation footprint.
Recommended maximum allowable (and ultimate) bearing pressures, shaft adhesions and modulus values for the range of rock encountered in boreholes at the site are presented in Table 6. The effect of these footings on the pile shoring walls should be considered as part of the design process.

These parameters apply to the design of spread foundations, such as pads or strip footings, or for socketed bored piles, for the support of axial compression loadings. They can be adopted on the assumption that the excavations are clean and free of loose debris, with pile sockets (if constructed) free of smear and adequately roughened immediately prior to concrete placement. Foundations proportioned on the basis of the allowable parameters would be expected to experience total settlements of less than 1% of the footing width (or pile diameter) under the applied working (i.e. serviceability) load, with differential settlements between adjacent columns expected to be less than half of this value. An experienced geotechnical professional should inspect all spread footings (e.g. pads) and pile excavations prior to the placement of concrete and steel, to check the adequacy of the foundation material and to undertake spoon testing as appropriate.

Footings taken down into consistent Class I/II sandstone could potentially be designed for an allowable bearing pressure of 6,000 kPa and possibly up to 8,000 kPa, subject to spoon testing during construction. If higher bearing pressures are used in design, however, then significant additional testing will be required in the form of cored boreholes and/or spoon testing of footings, to ensure there are no defects beneath footings. Alternatively, if a lower allowable bearing pressure of 3,500 kPa is adopted then testing during construction could be limited to inspection of foundations.

To use a bearing pressure value for design of 6,000 kPa, 33% of the footings should be spoon tested to a depth equivalent to 1.5 times the footing width. (Note that further drilling should be carried out to confirm the rock strength before a bearing pressure of 6,000 kPa can be adopted.)

### Table 6: Recommended Design Parameters and Moduli for Foundation Design

<table>
<thead>
<tr>
<th>Foundation Stratum&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Allowable End Bearing (MPa)</th>
<th>Ultimate End Bearing (MPa)</th>
<th>Allowable Shaft Adhesion (kPa)</th>
<th>Ultimate Shaft Adhesion (kPa)</th>
<th>Field Elastic Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shale – Class V</td>
<td>0.7</td>
<td>3.0</td>
<td>50</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>Shale – Class IV</td>
<td>1.0</td>
<td>3.0</td>
<td>150</td>
<td>300</td>
<td>100</td>
</tr>
<tr>
<td>Shale – Class III</td>
<td>3.5</td>
<td>10.0</td>
<td>200</td>
<td>350</td>
<td>500</td>
</tr>
<tr>
<td>Laminite/Sandstone – Class I/II</td>
<td>8.0</td>
<td>60</td>
<td>600</td>
<td>1000</td>
<td>2000</td>
</tr>
</tbody>
</table>

Notes: (1) Rock Classification based on Pells et. al (1998).
(2) Shaft adhesion applicable to the design of bored piles, uncased over the rock socket length, where adequate sidewall cleanliness and roughness are achieved.

Where footings are located within the zone of influence of adjacent excavations, drawn upward at 45 degrees from the toe of the excavation (such as lift shafts or tanks), the allowable bearing pressure should be reduced by 50% and the excavation floor carefully inspected for adversely oriented joints. Alternatively, the footings may be taken deeper, below the zone of influence.
Prospective piling contractors should be aware that, in the process of preparing the rock classes and geotechnical model, some of the encountered rock classes have been downgraded due to significant weak seams and core losses, and that higher strength bands of rock do occur in classes of lower strength.

The floors at basement level can be designed as slabs on ground. The final rock surface should be trimmed and scraped clean of debris. As the floor will be excavated within rock it is suggested that slab design be based on a design CBR for the subgrade material not exceeding 10%.

9.8 Seismic Design

In accordance with the Earthquake Loading Standard, AS 1170.4 – 2007 (Reference 1), the site’s hazard factor (Sydney) is 0.08, and the sub-soil class for earthquake loading is “Class Ce” due to the depth of soils encountered on the site.

10. References

8. The Department of Land and Water Conservation, 1995. 1:25 000 Acid Sulphate Soil Risk map for Parramatta-Prospect.

11. Limitations

Douglas Partners (DP) has prepared this geotechnical investigation report for this project at 11 Talavera Road, Macquarie Park, in accordance with DP’s proposal SYD161520 (Rev1), dated 13 January 2017 and acceptance received from Ms Denisse Dufey of Dexus Property Group on behalf of Dexus Funds Management Limited, dated 17 January 2017. The work was carried out under the standard Dexus Property Group Consultancy Agreement. This report is provided for the exclusive use of Dexus Funds Management Limited for this project only and for the purposes as described in the
report. It should not be used for other projects or purposes 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 notes 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 report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. This has been addressed under separate cover. 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.

Asbestos has not been detected by observation or by laboratory analysis, either on the surface of the site, or in filling materials at the test locations sampled and analysed. Building demolition materials, such as concrete rubble, were, however, located in previous below-ground filling, and these are considered as indicative of the possible presence of hazardous building materials (HBM), including asbestos.

Although the sampling plan adopted for this investigation is considered appropriate to achieve the stated project objectives, there are necessarily parts of the site that have not been sampled and analysed. This is either due to undetected variations in ground conditions or to budget constraints (as discussed above), or to parts of the site being inaccessible and not available for sampling. It is therefore considered possible that HBM, including asbestos, may be present in unobserved or untested parts of the site, between and beyond sampling locations, and hence no warranty can be given that asbestos is not present.

Douglas Partners Pty Ltd
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.

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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.
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 drawing from Real Serve Pty Ltd (Ref 61935MN, dated 30.11.2016)
2: Test locations are approximate only and are shown with reference to existing features.

LEGEND
揀 Current cored borehole location
揉 Hand augered borehole (DP report 85751.03, February 2017)
揌 Historical borehole (February 2000, DP report 28356)
P Standpipe piezometer

Geotechnical Cross Section A-A'
Geotechnical Plan - Elevations of Top of Weathered Rock

Proposed Commercial Development

11 Talavera Road, MACQUARIE PARK

NOTE:
1: Base drawing from Real Serve Pty Ltd
(Ref 61935MN, dated 30.11.2016)
2: Test locations are approximate only and are shown with reference to existing features.

LEGEND
- Current cored borehole location
- Hand augered borehole (DP report 85751.03, February 2017)
- Historical borehole (February 2000, DP report 28356)

- Standpipe piezometer

45.9
Elevation of top of weathered rock

Geotechnical Cross Section A-A’
LEGEND

Rock classes are as per Pells et al. (1998) and Bertuzzi and Pells (2002)

<table>
<thead>
<tr>
<th>Rock Class</th>
<th>UCS (MPa)</th>
<th>Direct Spacing (mm)</th>
<th>Allowable Shears (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shale / Laminite - Class V</td>
<td>&gt; 1</td>
<td>&lt; 10</td>
<td>&gt; 20</td>
</tr>
<tr>
<td>Shale / Laminite - Class IV</td>
<td>&gt; 1</td>
<td>&lt; 25</td>
<td>&lt; 20</td>
</tr>
<tr>
<td>Shale / Laminite - Class III</td>
<td>&gt; 2</td>
<td>&lt; 40</td>
<td>&lt; 8</td>
</tr>
<tr>
<td>Shale / Laminite - Class II</td>
<td>&gt; 4</td>
<td>&lt; 60</td>
<td>&lt; 4</td>
</tr>
<tr>
<td>Clays / Laminite - Class I</td>
<td>&gt; 7</td>
<td>&lt; 90</td>
<td>&lt; 2</td>
</tr>
<tr>
<td>Sandstone - Class III</td>
<td>&gt; 7</td>
<td>&gt; 200</td>
<td>5</td>
</tr>
<tr>
<td>Sandstone - Class II</td>
<td>&gt; 14</td>
<td>&gt; 400</td>
<td>3</td>
</tr>
<tr>
<td>Sandstone - Class I</td>
<td>&gt; 24</td>
<td>&gt; 800</td>
<td>1.3</td>
</tr>
</tbody>
</table>

Potential planes of weakness

Boundary

Inferred rock class boundary

Soil/rock interface

DISTANCE ALONG PROFILE (m)

ELEVATION (AHD*)

Notes:
1. Rock classification is in accordance with Pells et al. (1998) and Bertuzzi and Pells (2002)
2. Standpipe piezometer installed in Borehole 201

NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. 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.
3. Summary logs only. Should be read in conjunction with detailed logs.
4. Vertical and horizontal scales are equal.

Rock classes are as per Pells et al. (1998) and Bertuzzi and Pells (2002)
Sampling Methods

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>

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>

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</td>
<td>0 - 5%</td>
<td>Clay with a trace of sand</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 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 I_{s(50)}, MPa</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. 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 longer 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>
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
- ▲ Water seep
- ▼ 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.
- h horizontal
- v vertical
- sh sub-horizontal
- sv sub-vertical

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

#### General
- Asphalt
- Road base
- Concrete
- Filling

#### Soils
- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

#### 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
<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</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>PAVERS and SAND</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.7</td>
<td>FILLING - brown, sandy clay filling with some fine gravel, MC&lt;PL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>FILLING - grey-brown, silty clay filling with some fine sand, MC&lt;PL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>CLAY - very stiff to hard, orange mottled grey, clay with some shaly bands, MC&lt;PL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>3.0m: becoming orange-brown</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>SHALE: extremely low strength, grey shale with ironstone bands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.2</td>
<td>SHALE - extremely low to very low strength, extremely and highly weathered, fractured, light grey and red-brown shale with low and medium strength iron-cemented bands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.27</td>
<td>5.2m: CORE LOSS: 70mm</td>
<td></td>
<td></td>
<td>5.27-5.86m: J (x3) 35°-45° (relict) un, ro, cly</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.0</td>
<td>INTERCALATED SILTSTONE &amp; SANDSTONE (60:20) - low to high strength, highly and moderately to slightly weathered, fractured, light grey-brown fine grained sandstone inter laminated with siltstone.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5</td>
<td></td>
<td></td>
<td></td>
<td>9.0-9.15m: Cs, 9.15m: J45°, pl, ro, fe, 9.3-9.42m: J (x3) 35°-45°, he, 9.5-9.55m: Cs, 9.55m: CORE LOSS: 70mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.62</td>
<td></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°-5°, some with iron staining.

---

**BOREHOLE LOG**

**CLIENT:** Dexus Funds Management Limited  
**PROJECT:** Proposed Commercial Development  
**LOCATION:** 11 Talavera Road, Macquarie Park  
**SURFACE LEVEL:** 52.0 AHD*  
**EASTING:** 326895.4  
**NORTING:** 6260406.8  
**BORE No:** BH201  
**PROJECT No:** 85751.02  
**DATE:** 31/1 - 1/2/2017  
**DIP/AZIMUTH:** 90°/--

**DRILLER:** GM  
**LOGGED:** SI  
**CASING:** HW to 4.0m  
**WATER OBSERVATIONS:** No free groundwater observed whilst augering  

**REMARKS:** Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016). Well installed to 5.25m (backfill 0.5-2.5m; bentonite plug 2.25-6.5m; screen 1-5.25m; gravel 0.3-5.25m; bentonite plug 0.4-0.9m; gable cover at surface)

---

**SAMPLING & IN SITU TESTING LEGEND**

- **A** Auger sample  
- **B** Bulk sample  
- **BK** Blocks sample  
- **C** Core drilling  
- **D** Disturbed sample  
- **E** Environmental sample  
- **G** Gas sample  
- **P** Piston sample  
- **T** Tube sample (x mm dia.)  
- **W** Water sample  
- **W** Water level  
- **D** Density  
- **I** Index  
- **E** Elasticity  
- **P** Penetration  
- **U** Unit weight  
- **Q** Volume  
- **T** Test  
- **G** Geotechnical  
- **P** Physical  
- **A** Analytical  
- **I** In Situ  

---

**BORE No: BH201**  
**DATE:** 31/1 - 1/2/2017  
**SHEET 1 OF 3**
<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</td>
<td>INTERLAMINATED SILTSTONE &amp; SANDSTONE (80:20) - low to high strength, highly to moderately then slightly weathered, fractured, light grey-brown fine grained sandstone interlaminated with siltstone. (continued)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11.95m</td>
<td>INTERLAMINATED SILTSTONE &amp; SANDSTONE (70:30), medium strength, slightly weathered, slightly fractured, light grey to grey, fine to medium grained sandstone.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.74m</td>
<td>SANDSTONE - very low strength with medium strength bands, slightly weathered, fractured, light grey, fine to medium grained sandstone, with sheared zones.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14.4m</td>
<td>LAMINITE - medium then high strength, fresh, slightly fractured then unbroken, light grey to dark grey laminite with approximately 20% fine sandstone laminations</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Borehole Log**

**Client:** Dexus Funds Management Limited

**Project:** Proposed Commercial Development

**Location:** 11 Talavera Road, Macquarie Park

**Surface Level:** 52.0 AHD

**Easting:** 326895.4

**Northing:** 6260406.8

**Date:** 31/1 - 1/2/2017

**Rig:** Bobcat

**Driller:** GM

**Logged:** SI

**Type of Boring:** Solid flight auger (TC-bit) to 4.0m; Rotary to 5.2m; NMLC-Coring to 23.0m

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

**Remarks:** Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016). Well installed to 5.25m (backfill 6.5-23m; Bentonite plug 2.25-6.5m; Screen 1.5-2.25m; gravel 0.3-2.25m; Bentonite plug 0.4-0.9m; Galc cover at surface).

**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
- PL: Point load axial test (MPa)
- PL(D): Point load diametral test (MPa)
- P: Piston sample
- S: Standard penetration test
- SI: Photo ionisation detector (ppm)
- T: Tube sample (x mm dia.)
- W: Water sample
- X: Water level
- Y: Shear vane (kPa)

**Test Results & Comments:**
- PL(A) = 1
- PL(A) = 1.5
- PL(A) = 0.5
- PL(A) = 2.4
- PL(A) = 2.6
- PL(A) = 2.7
- PL(A) = 1.1
- PL(A) = 0.5
- PL(A) = 2.5
- PL(A) = 1
## BOREHOLE LOG

**CLIENT:** Dexus Funds Management Limited  
**PROJECT:** Proposed Commercial Development  
**LOCATION:** 11 Talavera Road, Macquarie Park  
**SURFACE LEVEL:** 52.0 AHD*  
**EASTING:** 326895.4  
**NORTHING:** 6260406.8  
**DATE:** 31/1 - 1/2/2017  
**SHEET** 3  OF 3

### Description of Strata

<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>21.13m</td>
<td>SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium to coarse grained sandstone (continued)</td>
<td></td>
<td></td>
<td>21.13m: B25°, pl, ro, cln</td>
<td></td>
<td>PL(A) = 1.3</td>
</tr>
<tr>
<td>21.88m</td>
<td></td>
<td></td>
<td></td>
<td>21.88m: J30°, pl, ro, cln</td>
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<td>PL(A) = 1.3</td>
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<tr>
<td>22.42-22.75m</td>
<td></td>
<td></td>
<td></td>
<td>22.42-22.75m: B (x3) 0°-5°, cly co, 1-2mm &amp; J80°, un, ro, cln</td>
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<td></td>
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<tr>
<td>22.8-23.0m</td>
<td></td>
<td></td>
<td></td>
<td>22.8-23.0m: J80°-90°, pl, ro, cln</td>
<td></td>
<td>PL(A) = 2.3</td>
</tr>
</tbody>
</table>

Bore discontinued at 23.0m - target depth reached

---

**CLIENT:** Dexus Funds Management Limited  
**PROJECT:** Proposed Commercial Development  
**LOCATION:** 11 Talavera Road, Macquarie Park  
**SURFACE LEVEL:** 52.0 AHD*  
**EASTING:** 326895.4  
**NORTHING:** 6260406.8  
**DATE:** 31/1 - 1/2/2017  
**SHEET** 3  OF 3

**RIG:** Bobcat  
**DRILLER:** GM  
**LOGGED:** SI  
**CASING:** HW to 4.0m

**TYPE OF BORING:** Solid flight auger (TC-bit) to 4.0m; Rotary to 5.2m; NMLC-Coring to 23.0m

**WATER OBSERVATIONS:** No free groundwater observed whilst augering

**REMARKS:** Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016). Well installed to 5.25m (backfill 6.5-23m; bentonite plug 5.25-6.5m; screen 1.5-2.5m; gravel 0.3-5.20m; bentonite plug 0.4-0.9m; gatic cover at surface)
BORE: 201          PROJECT: MACQUARIE PARK          FEBRUARY 2017

15.0 – 20.0m

BORE: 201          PROJECT: MACQUARIE PARK          FEBRUARY 2017

20.0 – 23.0m
### BOREHOLE LOG

**CLIENT:** Dexus Funds Management Limited  
**PROJECT:** Proposed Commercial Development  
**LOCATION:** 11 Talavera Road, Macquarie Park  
**SAMPLING & IN SITU TESTING LEGEND**

<table>
<thead>
<tr>
<th>Type</th>
<th>Symbol</th>
<th>Description</th>
<th>Test Results &amp; Comments</th>
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</thead>
<tbody>
<tr>
<td>Auger sample</td>
<td>A</td>
<td>Gas sample</td>
<td></td>
</tr>
<tr>
<td>Bulk sample</td>
<td>B</td>
<td>Piston sample</td>
<td></td>
</tr>
<tr>
<td>BLK Block sample</td>
<td>BLK</td>
<td>Tube sample (x mm dia.)</td>
<td></td>
</tr>
<tr>
<td>Core drilling</td>
<td>C</td>
<td>Water sample</td>
<td></td>
</tr>
<tr>
<td>Disturbed sample</td>
<td>D</td>
<td>Water sample</td>
<td></td>
</tr>
<tr>
<td>Environmental sample</td>
<td>E</td>
<td>Water level</td>
<td></td>
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<tr>
<td>Gad sample</td>
<td>G</td>
<td>PLD (MPa)</td>
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<tr>
<td>Gas sample</td>
<td>U</td>
<td>PLG (MPa)</td>
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<tr>
<td>Photo ionisation detector</td>
<td>D</td>
<td>Pocket penetrometer (kPa)</td>
<td></td>
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<tr>
<td>Laser (ppm)</td>
<td>P</td>
<td>Standard penetration test</td>
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<tr>
<td>Piezometer</td>
<td>PW</td>
<td>Shear vane (kPa)</td>
<td></td>
</tr>
<tr>
<td>Photo ionisation detector</td>
<td>PDI</td>
<td>Vane permeability test (psi)</td>
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</tr>
<tr>
<td>Piston sample</td>
<td>P</td>
<td>Water level</td>
<td></td>
</tr>
<tr>
<td>Pocket penetrometer (kPa)</td>
<td>PP</td>
<td>Water level</td>
<td></td>
</tr>
<tr>
<td>Water sample</td>
<td>W</td>
<td>Water level</td>
<td></td>
</tr>
<tr>
<td>Water sample</td>
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<tr>
<td>Water level</td>
<td>W</td>
<td>Water level</td>
<td></td>
</tr>
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</table>

**SURFACE LEVEL:** 48.9 AHD
**BORE No:** BH202
**EASTING:** 326928.6
**NORTHING:** 6260383.4
**DATE:** 2 - 3/2/2017
**REMARKS:** Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016)

---

**CLIENT:** Dexus Funds Management Limited  
**PROJECT:** Proposed Commercial Development  
**LOCATION:** 11 Talavera Road, Macquarie Park  
**RIG:** Bobcat  
**DRILLER:** GM  
**LOGGED:** JN  
**CASING:** HW to 2.5m; HQ to 3.0m  
**TYPE OF BORING:** Solid flight auger to 2.5m; Rotary (water) to 3.0m; NMLC-Coring to 20.57m  
**WATER OBSERVATIONS:** No free groundwater observed whilst augering

---

### ROCK DESCRIPTION

<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</td>
<td>FILLING - brown sandy silt filling (topsoil) with some sandstone gravel and grass rootlets, humid</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>B - Bedding</td>
<td>A</td>
</tr>
<tr>
<td>1.50</td>
<td>CLAY - very stiff, light grey mottled red, clay with some medium strength iron-cemented bands, humid</td>
<td>Very Low</td>
<td>Medium</td>
<td>0</td>
<td>S - Shear</td>
<td>B</td>
</tr>
<tr>
<td>3.00</td>
<td>SHALE - extremely low strength, extremely weathered, light grey shale, some 20mm to 50mm iron cemented bands</td>
<td>Very Low</td>
<td>Very Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>3.76</td>
<td>SANDSTONE - extremely low strength, extremely weathered, light grey</td>
<td>Very Low</td>
<td>Medium</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>4.84</td>
<td>SANDSTONE - low to high strength, highly weathered, fractured, grey and orange-brown, medium to coarse grained sandstone with some high strength iron-cemented bands</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>5.48</td>
<td>LAMINITE - low then medium strength, moderately to slightly weathered then fresh stained, fractured, dark grey laminate</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>6.00</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>6.46</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>6.56</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>7.66</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>7.76</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>8.00</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>8.50</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>9.00</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
<tr>
<td>9.56</td>
<td>SANDSTONE - description next page</td>
<td>Very Low</td>
<td>Low</td>
<td>0</td>
<td>F - Fault</td>
<td>C</td>
</tr>
</tbody>
</table>

---

**CLAY - very stiff, light grey mottled red, clay with some medium strength iron-cemented bands, humid**

**SANDSTONE - extremely weathered, fractured, grey and orange-brown, medium to coarse grained sandstone with some high strength iron-cemented bands**

**LAMINITE - low then medium strength, moderately to slightly weathered then fresh stained, fractured, dark grey laminate**

---

** appended notes:**

- **Note:** Unless otherwise stated, rock is fractured along rough planar bedding dipping 0° - 10°, some iron stained
- **2.5m:** J45°, pl, ro, cly, 5mm
- **3.75-3.85m:** J45°, pl, ro, cly, 5mm
- **4.84m:** Cs, 10mm
- **5.48m:** J45°, un, ro, cly co
- **5.75m:** Cs, 10mm
- **5.99m:** Cs, 10mm
- **6.0 - 6.3m:** J70°, pl, ro, cly co
- **6.3m:** B10°, pl, ro, cly co
- **6.4m:** CORE LOSS: 100mm
- **7.10, 7.20, 7.25m:** J45°, pl, fe sfn, he
- **7.66m:** J30°, cu, ro, fe sfn
- **7.76m:** J45°, pl, fe sfn, he
- **7.77m:** J20°, pl, ro, fe sfn
- **7.86, 7.98, 8.03m:** J45°, pl, fe sfn, he
- **8.15m:** J0°, st, ro, fe sfn
- **9.56:** B5° - 10°, pl, sm, cly co
- **10.0:** B5° - 10°, pl, sm, cly co
<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>10.74</td>
<td>SANDSTONE - medium and high strength, fresh, fractured, light grey, fine grained sandstone</td>
<td>EWHWMWSWFSFR</td>
<td>cly</td>
<td>0.01</td>
<td>J - Joint, F - Fault</td>
<td>C 100 99 PL(A) = 2.1</td>
</tr>
<tr>
<td>11.24</td>
<td>SANDSTONE - medium and high strength, fresh, slightly fractured then unbroken, light grey, medium to coarse grained sandstone</td>
<td>EWHWMWSWFSFR</td>
<td>cly</td>
<td>0.01</td>
<td>J - Joint, F - Fault</td>
<td>C 100 99 PL(A) = 0.7</td>
</tr>
<tr>
<td>12.47</td>
<td>13.79-16.00m: fractured</td>
<td>C</td>
<td>B10°, pl, ro, cly</td>
<td>1mm</td>
<td></td>
<td>C 100 96 PL(A) = 0.6</td>
</tr>
<tr>
<td>13.79</td>
<td>13.91: Cs, 40mm</td>
<td>S</td>
<td>J45°, un, ro, cln</td>
<td>1mm</td>
<td></td>
<td>C 100 96 PL(A) = 1.8</td>
</tr>
<tr>
<td>14.1-14.5</td>
<td>14.25-14.5m and</td>
<td>S</td>
<td>J45°- 90°, un, ro, cln</td>
<td>1mm</td>
<td></td>
<td>C 100 96 PL(A) = 1.7</td>
</tr>
<tr>
<td>14.55-14.8m: J45°- 90°, un, sandy clay</td>
<td></td>
<td>S</td>
<td>J45°- 90°, un, ro, cln</td>
<td>2mm</td>
<td></td>
<td>C 100 96 PL(A) = 0.7</td>
</tr>
<tr>
<td>15.66-16.0m: J45°- 90°, un, ro, cln</td>
<td></td>
<td>S</td>
<td>J45°- 90°, un, ro, cln</td>
<td>2mm</td>
<td></td>
<td>C 100 96 PL(A) = 1.1</td>
</tr>
<tr>
<td>17.96m: B5°, pl, ro, cly</td>
<td></td>
<td>E</td>
<td>J45°- 90°, un, ro, cln</td>
<td>5mm</td>
<td></td>
<td>C 100 100 PL(A) = 0.9</td>
</tr>
<tr>
<td>19.25m: J45°, cu, ro, cly</td>
<td></td>
<td>E</td>
<td>J45°- 90°, un, ro, cln</td>
<td>10mm</td>
<td></td>
<td>C 100 100 PL(A) = 0.9</td>
</tr>
</tbody>
</table>
### BOREHOLE LOG

**CLIENT:** Dexus Funds Management Limited  
**PROJECT:** Proposed Commercial Development  
**LOCATION:** 11 Talavera Road, Macquarie Park  
**SURFACE LEVEL:** 48.9 AHD*  
**EASTING:** 326928.6  
**NORTHING:** 6260383.4  
**DATE:** 2 - 3/2/2017  
**REMARKS:** Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016)

<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>20.57</td>
<td>SANDSTONE - as previous</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Bore discontinued at 20.57m</td>
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<tr>
<td>0.01</td>
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<td>0.05</td>
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<td>0.10</td>
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</tr>
</tbody>
</table>

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

**Test Results & Comments:**
- PL(A) = 0.4

**SAMPLE & 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
- H: Gas hydrate
- P: Piston sample
- PL(D): Point load diametral test (50) (MPa)
- PLG: Point load axial test (50) (MPa)
- PD: Photo ionisation detector (ppm)
- PPI: Pocket penetrometer (kPa)
- PWP: Pocket water pressure (kPa)
- V: Shear vane (kPa)

**BORE No:** BH202  
**PROJECT No:** 85751.02  
**DATE:** 2 - 3/2/2017  
**SHEET:** 3 of 3

**RIG:** Bobcat  
**DRILLER:** GM  
**LOGGED:** JN  
**CASING:** HW to 2.5m; HQ to 3.0m

**TYPE OF BORING:** Solid flight auger to 2.5m; Rotary (water) to 3.0m; NMLC-Coring to 20.57m

**WATER OBSERVATIONS:** No free groundwater observed whilst augering

**SURFACE LEVEL:** 48.9 AHD*

**EASTING:** 326928.6  
**NORTHING:** 6260383.4  
**DIP/AZIMUTH:** 90°/--

**WATER OBSERVATIONS:**
- No free groundwater observed whilst augering

**REMARKS:** Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016)
BORE: 202          PROJECT: MACQUARIE PARK          FEBRUARY 2017

3.0 – 7.0m

BORE: 202          PROJECT: MACQUARIE PARK          FEBRUARY 2017

7.0 – 12.0m
### Borehole Log

**Client:** Dexus Funds Management Limited  
**Project:** Proposed Commercial Development  
**Location:** 11 Talavera Road, Macquarie Park

**Surface Level:** 48.9 AHD  
**Bore No.:** BH203  
**Easting:** 326898.4  
**NORTING:** 6260348  
**Date:** 1-2/2/2017  
**Sheet 1 of 3**

#### Description of Strata

- **Depth (m):** 0.2  
  - **Description:** Filling - brown, silty sand filling (topsoil), humid (garden bed)
- **Depth (m):** 0.8  
  - **Description:** Filling - brown, clay filling with some sand and fine gravel and rippled shaly fragments, humid
- **Depth (m):** 1.5  
  - **Description:** Clay - very stiff, grey mottled orange, clay with some extremely low strength ironstone bands, MC<PL
- **Depth (m):** 2.0  
  - **Description:**becoming orange-brown mottled grey, with fine subrounded gravel
- **Depth (m):** 3.5  
  - **Description:** becoming grey with ironstone bands
- **Depth (m):** 4.0  
  - **Description:** Shale - extremely low strength, grey shale with ironstone bands, trace fine roots
- **Depth (m):** 5.5  
  - **Description:** Sandstone - medium strength, moderately to highly weathered, fractured, light grey and orange-brown, medium grained sandstone
- **Depth (m):** 6.4  
  - **Description:** 9.73m: extremely weathered band of laminite

#### Degree of Weathering

- **EWH:** Extremely Weathered  
- **WM:** Weathered  
- **WS:** Weakly Weathered  
- **FS:** Fresh  
- **FR:** Fractured  
- **S:** Shear  
- **B:** Bedding

#### Rock Strength

- **Ex Low:** Extremely Low  
- **Very Low:** Very Low  
- **Low:** Low  
- **Medium:** Medium  
- **High:** High  
- **Very High:** Very High  
- **Ex High:** Extremely High

#### Sampling & In Situ Testing

- **Test Results & Comments:**
- **Discontinuities:**
- **Type of Bore:** Solid flight auger (TC-bit) to 4.0m; Rotary to 6.4m; NMLC-Coring to 20.35m
- **WATER OBSERVATIONS:** No free groundwater observed whilst augering

#### Remarks:

- Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016)

---

**DRILLER:** GM  
**LOGGED:** SI  
**CASING:** HW to 4.0m

---

**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.)

---

**Douglas Partners**

Geotechnics | Environment | Groundwater
<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>9.73-9.83</td>
<td>LAMINITE - medium then high strength, fresh, slightly fractured, dark grey laminite with some fine grained sandstone laminations</td>
<td>EWHWMWSWFSFR</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.12</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>11.9</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>12.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13.35</td>
<td>SANDSTONE - high strength, fresh, slightly fractured, light grey, fine grained sandstone with some carbonaceous laminations</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13.68</td>
<td>14.05-14.25 J80°, pl, ro, cln</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14.5</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>14.68</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>16.05</td>
<td>16.05m: B10°, clv co, 2mm</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>17.15-17.17</td>
<td>17.15-17.17m: Sz, 20mm</td>
<td></td>
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<tr>
<td>17.23</td>
<td>17.65m: B20°, clv, 3mm</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>18.45</td>
<td>18.45m: J35°, pl, ro, cln</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>18.87</td>
<td>18.87m: B10°, clv, 2mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19.18</td>
<td>19.18m: J35°, pl, ro, cln</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>19.23</td>
<td>19.23m: B9°, clv, 10mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**RIG:** Bobcat  
**DRILLER:** GM  
**LOGGED:** SI  
**CASING:** HW to 4.0m  
**TYPE OF BORING:** Solid flight auger (TC-bit) to 4.0m; Rotary to 6.4m; NMLC-Coring to 20.35m  
**WATER OBSERVATIONS:** No free groundwater observed whilst augering  
**REMARKS:** Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016)
### BOREHOLE LOG

**CLIENT:** Dexus Funds Management Limited  
**PROJECT:** Proposed Commercial Development  
**LOCATION:** 11 Talavera Road, Macquarie Park  
**SURFACE LEVEL:** 48.9 AHD  
**EASTING:** 326898.4  
**NORTHING:** 6260348  
**DATE:** 1 - 2/2/2017  
**DIP/AZIMUTH:** 90°/--  
**BORE No:** BH203  
**PROJECT No:** 85751.02  
**SHEET:** 3 OF 3

---

**Description of Strata**

<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>20.35</td>
<td>SANDSTONE - high strength, fresh, slightly fractured and unbroken, pale grey, medium grained sandstone (continued)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20.35</td>
<td>Bore discontinued at 20.35m - target depth reached</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Test Results & Comments**

- 20.1m: B5°, cly co, 1mm  
- C 100 100  PL(A) = 1.6

---

**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  
- **PD** Photo ionisation detector (ppm)  
- **PL** Point load axial test (50 (MPa)  
- **PL(D)** Point load diametral test (50 (MPa)  
- **pp** Pocket penetrometer (kPa)  
- **S** Standard penetration test  
- **V** Shear vane (kPa)

---

**RIG:** Bobcat  
**DRILLER:** GM  
**LOGGED:** SI  
**CASING:** HW to 4.0m  
**TYPE OF BORING:** Solid flight auger (TC-bit) to 4.0m; Rotary to 6.4m; NMLC-Coring to 20.35m  
**WATER OBSERVATIONS:** No free groundwater observed whilst augering  
**REMARKS:** Coordinates and surface level interpolated from Real Serve Pty Ltd Drawing 61935MN (dated 31.11.2016)
BORE: 203          PROJECT: MACQUARIE PARK          FEBRUARY 2017

16.0 – 20.35m
Appendix D

Historical Field Results
# Test Bore Report

**Client:** ICA Property  
**Project:** Proposed Conference Centre  
**Location:** 7-13 Talavera Road, North Ryde  
**Date:** 24/1/00

### Depth (m)  
<table>
<thead>
<tr>
<th>Description of Strata</th>
<th>Geotechnical Log</th>
<th>Strength</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>FILLING - light grey and yellow grey silty clay, crushed siltstone/shale and gravel filling</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SILTY CLAY - very stiff and hard, light brown grey mottled red and yellow silty clay with ironstone gravel and traces of sand</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SANDY SILTY CLAY - hard and very stiff, light grey sandy silty clay with traces of ironstone gravel (extremely weathered siltstone)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- 6.2m - core loss 300mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SILTSTONE - low strength, moderately weathered, fractured, light brown grey and dark grey siltstone with clayey bands along bedding planes and joints</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&quot;J 40&quot; at 6.84m and &quot;J 55&quot; at 7.43m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SANDSTONE - medium and high strength, moderately weathered, fractured to slightly fractured, light brown yellow, fine to medium grained sandstone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&quot;J 50&quot; at 7.7m and &quot;J 80-85&quot; at 8.1m and 8.28m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SILTSTONE - medium strength, slightly weathered, fractured, dark grey siltstone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TEST BORE DISCONTINUED AT 8.6 METRES</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Sampling & In Situ Testing Legend
- A: auger sample  
- B: bulk sample  
- C: core drilling  
- PP: pocket penetrometer (kPa)  
- PL: point load strength (50MPa)  
- S: standard penetration test  
- Ux: mm dia. tube  
- V: shear vane (kPa)

### Remarks
- **Rig:** Scout  
- **Driller:** Wilson  
- **Logged:** Parmar  
- **Casing:** GL to 6.2m  
- **Type of Boring:** Spiral Flight Auger to 6.2m, NML Coring to 8.8m  
- **Water Observations:** No free groundwater observed whilst augering  
- **REMARKS:** Relative to floor slab of adjacent building RL 54.36

### Test Results
- S: 7.7,8 N=15  
- S: 4.5,9 N=14  
- 25/ref
- 7,16,25/50mm ref
- PP=300kPa  
- PP=350kPa  
- PP=400kPa  
- PP=75kPa  
- PL(A)=0.6MPa  
- PL(A)=1.4MPa

### Checked
- Initials:  
- Date: 2/00

---

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## TEST BORE REPORT

**CLIENT:** ICA PROPERTY  
**PROJECT:** PROPOSED CONFERENCE CENTRE  
**LOCATION:** 7-13 TALAVERA ROAD, NORTH RYDE  
**PROJECT No:** 28356  
**SURFACE LEVEL:** 52.15m  
**DIP OF HOLE:** 90°  
**BORE No:** 102  
**DATE:** 25/1/00  
**SHEET 1 OF 1**  
**AZIMUTH:**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Sol Strength</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>FILLING - dark brown clayey silty filling</td>
<td></td>
<td>5.86 N=12</td>
</tr>
<tr>
<td>0.06</td>
<td>FILLING - apparently well compacted to compacted, light grey and red brown silty clay filling with shale and ironstone fragments</td>
<td></td>
<td>3.43 N=7</td>
</tr>
<tr>
<td>3.2</td>
<td>SILTY CLAY - stiff, light grey and yellow grey mottled red brown silty clay with ironstone gravel and traces of sand</td>
<td></td>
<td>4.55 N=10</td>
</tr>
<tr>
<td>4.2</td>
<td>SILTY CLAY - very stiff, light grey mottled yellow brown silty clay with ironstone gravel</td>
<td></td>
<td>3.712 N=19</td>
</tr>
<tr>
<td>4.65</td>
<td>CLAY - very stiff, grey clay with minor ironstone gravel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.5</td>
<td>SHALY CLAY - grey shaly clay with minor ironstone nodules</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.5</td>
<td>TEST BORE DISCONTINUED AT 6.5 METRES</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**RIG:** SCOUT  
**DRILLER:** WILSON  
**LOGGED:** PARMAR  
**CASING:** UNCASED  

**TYPE OF BORING:** SPIRAL FLIGHT AUGER TO 6.5m  
**WATER OBSERVATIONS:** NO FREE GROUNDWATER OBSERVED  
**REMARKS:** RELATIVE TO FLOOR SLAB OF ADJACENT BUILDING AT RL 54.3m

**SAMPLING & IN SITU TESTING LEGEND**

- A auger sample  
- B bulk sample  
- C core drilling  
- D pocket penetrometer (kPa)  
- E point load strength (kPa)  
- F standard penetration test  
- G shear vane (kPa)

**CHECKED:**

![Signature]

_Douglas Partners_  
Geotechnics - Environment - Groundwater

**Date:**
## TEST BORE REPORT

**CLIENT:** ICA PROPERTY  
**PROJECT:** PROPOSED CONFERENCE CENTRE  
**LOCATION:** 7-13 TALAVERA ROAD, NORTH RYDE

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>FILLING - light grey and yellow grey brown silty clay, crushed silstone/shale and gravel filling</td>
<td>Test Results &amp; Comments</td>
</tr>
</tbody>
</table>
| 0.8 to 4.0 | apparently compacted filing | 3.2,8  
|           |                        | N=8 |
| 4.0       | SANDY SILTY CLAY - hard, light yellow grey mottled red brown sandy silty clay (extremely weathered silstone) with ironstone gravel | 3.3,4  
|           |                        | N=7 |
| 4.7       | SHALY CLAY - hard, light grey shaly clay with ironstone bands | 3.4,5  
|           |                        | N=8 |
| 6.0       | SILTSTONE - very low and low strength, highly and moderately weathered, fractured, light brown grey and dark grey silstone with extremely weathered and clayey bands along bedding planes and joints | 9,28/ref |
| 7.2 to 7.7 | - medium strength | |
| 1.8       | SILTSTONE - low then medium strength, moderately weathered, fractured, light brown grey and dark grey silstone with extremely low and very low strength bands along bedding planes and joints | pp=125kPa |
| J's at 80' and 80' at 8.25m and 9.4m | PL (A)=0.4MPa  
| - 9.32m to 9.43m | - sandstone band | PL (A)=0.6MPa  
|           |                        | PL (A)=0.5MPa  
|           |                        | PL (A)=0.4MPa  
|           |                        | PL (A)=0.9MPa  
| 9.8       | TEST BORE DISCONTINUED AT 9.8 | |

**RIG:** SCOUT  
**DRILLER:** WILSON  
**LOGGED:** PARMAR  
**CASING:** GL TO 0.5m

**TYPE OF BORING:** SPIRAL FLIGHT AUGER TO 0.8m, NMLC CORING TO 0.8m

**WATER OBSERVATIONS:** NO FREE GROUNDWATER OBSERVED WHILST AUGERING

**REMARKS:** RELATIVE TO FLOOR SLAB OF ADJACENT BUILDING AT RL 54.36

**SAMPLING & IN SITU TESTING LEGEND**
- A auger sample  
- B bulk sample  
- C core drilling  
- pp pocket penetrometer (kPa)  
- V Shear Vane (kPa)  

**CHECKED:**

![Signature]

**DATE:** 25/1/00

**Douglas Partners**
Geotechnics · Environment · Groundwater
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Soils</th>
<th>Rock Strength</th>
<th>Sampling &amp; In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>FILLING - brown silty clay filling with shale fragments</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.8</td>
<td>FILLING - apparently well compacted, light grey and yellow brown silty clay, crushed</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>shale/siltstone and ironstone gravel filling</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.4</td>
<td>SILTY CLAY - very stiff, light yellow grey silty clay with traces of ironstone gravel</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>SILTY CLAY - very stiff, light grey mottled red silty clay with ironstone gravel</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.0</td>
<td>S-HALY CLAY - grey shaly clay with some ironstone nodules</td>
<td></td>
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</tr>
<tr>
<td>7.0</td>
<td>TEST BORE DISCONTINUED AT 7.0 METRES</td>
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</tr>
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</table>

RIG: SCOUT  
DRILLER: WILSON  
LOGGED: PARMAR  
CASING: UNCASED

TYPE OF BORING: SPIRAL FLIGHT AUGER TO 7.0M  
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED  
REMARKS: *RELATIVE TO FLOOR SLAB OF ADJACENT BUILDING AT RL 54.3E

**SAMPLING & IN SITU TESTING LEGEND**
- A auger sample  
- B bulk sample  
- C core drilling  
- D pocket penetrometer (kPa)  
- E PL point load strength \( I_p \) (50kPa)  
- F standard penetration test  
- G UX x dia. tube  
- H V shear vane (kPa)

**CHECKED:**