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Report on
Desktop Geotechnical Assessment

Proposed Residential Development
Ivanhoe Estate, Macquarie Park

Prepared for
Frasers Property Australia Pty Ltd

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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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1. Introduction

This report presents the results of a desktop geotechnical assessment undertaken by Douglas Partners Pty Ltd (DP) for a proposed residential development at Ivanhoe Estate, Macquarie Park. The assessment was commissioned in an email dated 29 June 2017 by Joe Zannino of Citta Property Group Pty Limited on behalf of Frasers Property Australia Pty Ltd and was undertaken in accordance with DP's proposal (ref:MAC170179) dated 7 June 2017.

This report supports a Concept Development Application for the Ivanhoe Estate Masterplan, a State Significant Development (SSD) submitted to the Department of Planning and Environment (DPE) pursuant to Part 4 of the *Environmental Planning and Assessment Act 1979* (EP&A Act). It has been prepared for Aspire Consortium on behalf of NSW Land and Housing Corporation.

It is understood that the development of the site will include the demolition of existing structures at the site followed by the construction of new high rise buildings, largely for residential purposes. Basements of up to four levels are proposed to provide suitable parking for the development. The construction of a new bridge across Shrimptons Creek is also proposed.

This report relates to Ivanhoe Estate on the north-western side of Shrimptons Creek. A separate, supplementary letter report has been prepared to address the site on the south-eastern side of Shrimptons Creek, on the north-eastern side of 2-4 Lyon Park Road, and is enclosed in Appendix C.

The aim of this desktop assessment was to assess the subsurface soil and groundwater conditions at Ivanhoe Estate in order to provide

- a preliminary assessment of geotechnical constraints relating to basement excavation and construction;
- a preliminary assessment of likely subgrade conditions for new pavements; and,
- comments in relation to other relevant geotechnical issues at the site

The assessment included:

- the review of selected mapping for the area;
- review of DP's database of past investigations in the vicinity of the site; and,
- a brief site visit to assess site conditions and make observations.

2. Background

In September 2015 the Ivanhoe Estate was rezoned by the Department of Planning and Environment as part of the Macquarie University Station (Herring Road) Priority Precinct, to transform the area into a vibrant centre that benefits from the available transport infrastructure and the precinct's proximity to jobs, retail and education opportunities within the Macquarie Park corridor.

The Ivanhoe Estate is currently owned by NSW Land and Housing Corporation and comprises 259 social housing dwellings. The redevelopment of the Ivanhoe Estate is part of the NSW Government Communities Plus program, which seeks to deliver new communities where social housing blends with private and affordable housing, with good access to transport, employment, improved community facilities and open space.

The Communities Plus program seeks to leverage the expertise and capacity of the private and non-government sectors. As part of this program, Aspire Consortium, comprising Frasers Property Australia, Citta Property Group and Mission Australia Housing, was selected as the successful proponent to develop the site in July 2017.

The Masterplan DA is the first step of the planned redevelopment of the Ivanhoe Estate and will create an integrated neighbourhood including social housing mixed with affordable and private housing, as well as seniors housing, a new school, child care centres, community facilities and retail development.

3. Site Description

The Ivanhoe Estate site is located in Macquarie Park near the corner of Epping Road and Herring Road within the Ryde Local Government Area (LGA). The site is approximately 8.2 hectares and currently accommodates 259 social housing dwellings, comprising a mix of townhouse and four storey apartment buildings set around a cul-de-sac street layout. An aerial photo of the site is provided at Figure 1 below.

Immediately to the north of the site are a series of four storey residential apartment buildings. On the north-western boundary, the site fronts Herring Road and a lot which is currently occupied by four former student accommodation buildings and is likely to be subject to redevelopment. Epping Road runs along the south-western boundary of the site and Shrimptons Creek, an area of public open space, runs along the south-eastern boundary. Vehicle access to the site is via Herring Road.

The site is comprised of 17 individual lots and a part lot and are owned and managed by Land and Housing Corporation. The Masterplan site also incorporates adjoining land, being a portion of Shrimptons Creek and part of the commercial site at 2-4 Lyonpark Road. This land is included to facilitate a bridge crossing and road connection to Lyonpark Road.

Figure 1: Ivanhoe Estate site

Ground levels in the Ivanhoe Estate site slope down from approximately RL 71 towards the Herring Road/Epping Road intersection towards the east and south-east, to RL 42 at Shrimptons Creek, at the south-eastern site boundary.

3.1 Site Observations

A walkover of the site was undertaken by an experienced geotechnical engineer on 3 July 2017. Observations from the walkover are discussed below, and summarised in Drawing 1, in Appendix B.

The existing site usage is medium density residential, with local access roads, public open space and a childcare centre. The photo in Figure 2 shows common development at the site – 4 storey brick residential flats and two-storey townhouses on gently sloping ground.

Figure 2: Facing uphill from the cul-de-sac of Ivanhoe Place. Note townhouses at right. Medium-rise residential flats are obscured by trees at left hand side.

The existing structures were generally in poor to reasonable condition, although some cracks were present in the brickwork of buildings and retaining walls, possibly indicating differential movement of foundations.

Pavements were generally in poor to reasonable condition, with potholes, alligator cracking and rutting present at some locations through the site.

Construction of the existing residential buildings has included cut and fill activities, including cuts into bedrock in the western part of the site. Exposed rock was visible in several locations at the rear of residences west of Ivanhoe Place, at the locations shown in Drawing 1, in Appendix B. The rock faces were generally not accessible for close inspection, but appeared to generally comprise sandstone, sometimes thinly bedded. Shale and siltstone were also considered to be present, particularly towards the upper (north–western) end of the site, based on the apparent bedding thickness and weathering of the rock, although this may also be due to the presence of weaker sandstone.

Figure 3: Exposed rock face at rear of 10 Ivanhoe Place.

Gravelly clay, clayey sand and sand filling was also exposed in the faces of some batters at the site, likely due to past construction works for building platforms and road embankments, and are noted in Drawing 1 in Appendix B.

A medium strength sandstone outcrop was also visible at the southern corner of the site, near Shrimptons Creek (refer feature “G” in Drawing 1).

Figure 4: Epping Road Culvert for Shrimptons Creek, facing south. Note low sandstone outcrop (Feature “G”) at right hand side of path.

The banks of Shrimptons Creek slope were gently to moderately steep, and were generally vegetated or comprised exposed filling. Numerous sandstone boulders were present in the banks and creek bed (refer feature “F” in Drawing 1). It is considered probable that the boulders were used to stabilise the slope against erosion, although this appear largely ineffective in their current alignment. Some sandstone boulder headwalls are present near stormwater outlets, and appear to be in better condition.

Figure 5: Shrimptons Creek, facing south-east near centre of south-eastern boundary. Note presence of sandstone boulders in creek bank and floor of creek.

Figure 6: Shrimptons Creek, facing north-west near centre of south-eastern boundary. Possible outcrop or boulder at toe of slope.

Signage at Shrimptons Creek indicates that the creek is subject to flooding.

Groundwater seepage was not observed at cuts or creek banks. Throughout the site, however, mossy growth was observed between pavers and on walls, suggesting that the existing soils are poorly drained. Some of these areas are shown in Drawing 1 (Appendix B) and are denoted by the letter “D”.

Figure 7: Moss growth on pavers and walls; 3 Wilcannia Way

3.2 Regional Mapping

Reference to the Sydney Soils Landscape Series Sheet (see Figure 8) indicates that the site is largely underlain by the residual Lucas Heights soil landscape, though partly underlain by the erosional Glenorie soils towards Herring Road, over the upper (north-western) third of the site.

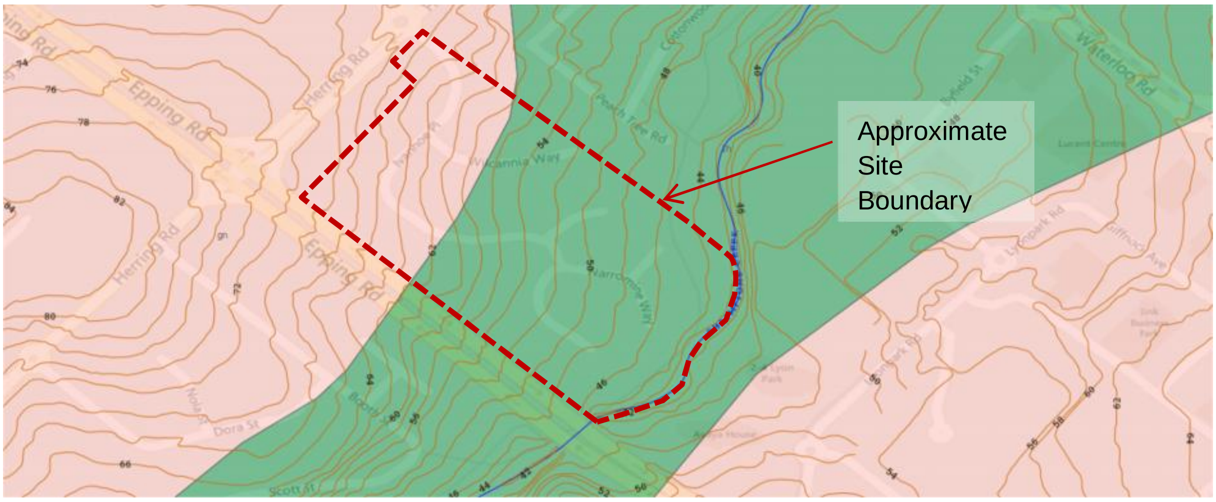


Figure 8: Extract of Soils Landscape Series Mapping (pink = Glenorie; green = Lucas Heights)

Based on previous DP experience in the area, the natural soils are likely to comprise sandy clay and clayey sand soils, possibly with some sandstone gravel or ironstone bands, and possibly some silty clay soils towards Herring Road.

The soils are likely to be relatively shallow, but may be deeper in areas of past filling. While filling is not shown within this area by the mapping, observations during the site walkover suggest that filling is likely to be present in many parts of the site due to past cut and fill activities. Such filling may variously include gravelly clay and sand soils, though other types of filling are also likely to be present.

Reference to the Sydney Geology Series Sheet, shown in Figure 9, suggests that the site is underlain by Ashfield Shale towards Herring Road (typically comprising shale and laminite); and by Hawkesbury Sandstone in the lower half of the site (typically comprising medium to coarse grained sandstone with very minor shale and laminite lenses). The Ashfield Shale, where present, would be underlain by Hawkesbury Sandstone, possibly with a transitional Mittagong Formation (interbedded shale, laminite and sandstone) between the two layers. DP experience varies somewhat from this mapping and is discussed in greater detail in Section 4.1.

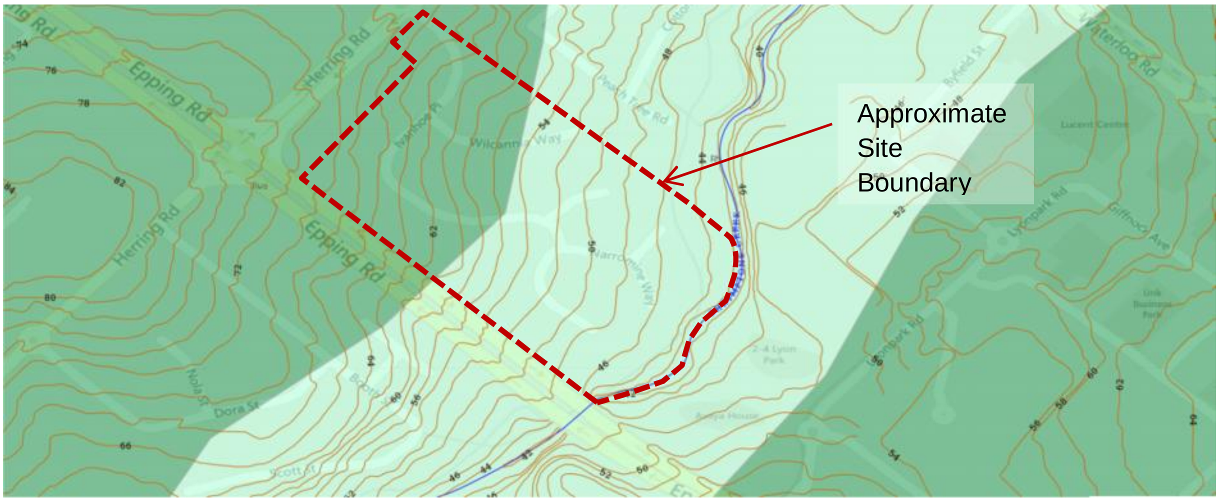


Figure 9: Edited Extract of Geological Series Mapping (dark green = Ashfield Shale, light green = Hawkesbury Sandstone)

A broader view of the geological mapping, as shown in Figure 10, indicates that a dyke has been identified approximately 2 km north-west of the site, oriented parallel to and near Epping Road. Dykes are often long, linear (in plan) features in the Sydney area, and, while not yet mapped at the subject site, may potentially extend to or near it.

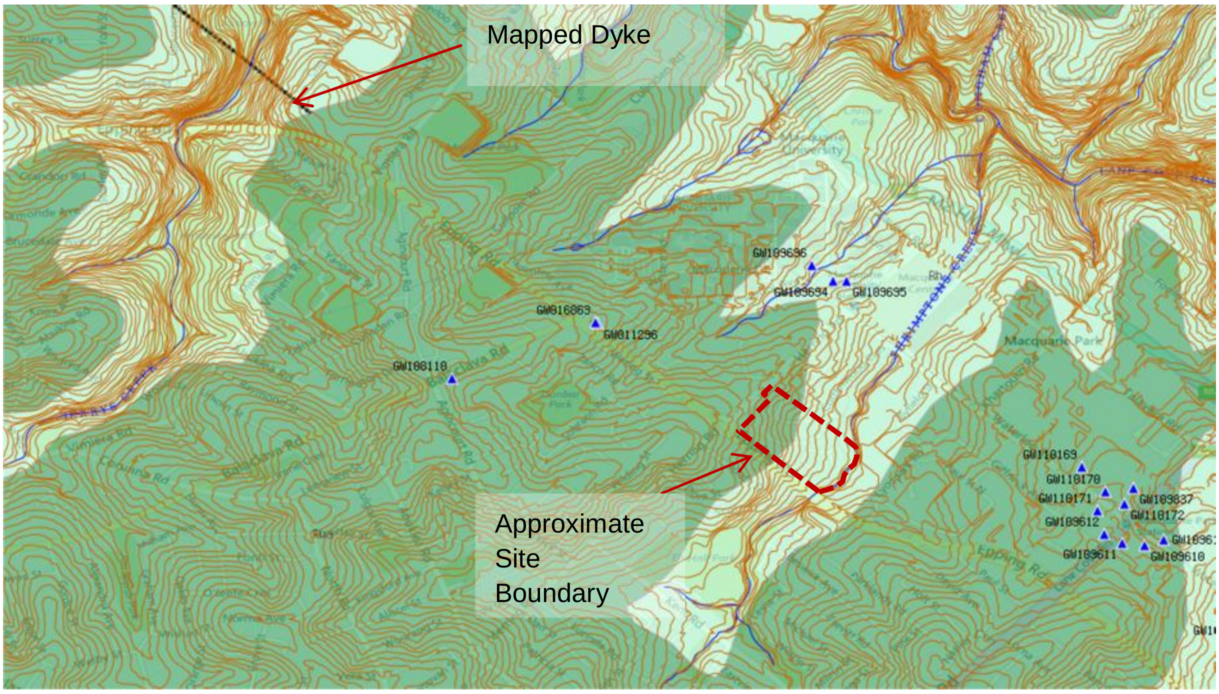


Figure 10: Registered Groundwater Bores (blue triangles) and Geological Series Mapping – Note diagonal black line at top left corner (mapped dyke)

Further discussion of dykes is included in Section 4.1 and 4.2.

3.3 Groundwater Bores

Also shown in Figure 10 are the registered groundwater bores in the site vicinity. These bores are clustered at the university, near the intersection of Lane Cove Road and Waterloo Road, and west of the subject site, and are approximately 300 m to 1 km from the subject site.

Groundwater levels at most of these bores are unknown based on the NSW Office of Water register (2010), with the exception of the following standing water level measurements:

GW109837 (East of site) – 18 m

GW108110 (West of site) – 7.3 m

GW016863 (West of site) – 6 m

GW011296 (West of site) – 4.5 m

Based on the drillers logs, these levels all correspond to levels within the underlying bedrock.

3.4 Salinity and Acid Sulfate Soils

The site is in an area of no known risk of coastal Acid Sulfate Soil, and is at an elevation above those associated with coastal Acid Sulfate Soils.

The site is outside of the Salinity Potential in Western Sydney mapping, and is not in an area generally associated with saline soils. Some layers within the Glenorie soil landscape are associated with localised salinity, but are not considered likely to be present in significant volumes on this site.

4. Proposed Development

4.1 Overview of Proposed Development

The proposed Masterplan is a Concept DA (in accordance with Section 83B of the EP&A Act), which sets out the concept proposal for the development of the site. The concept contained in the Masterplan DA establishes the planning and development framework, which will form the basis for the detailed design of the future buildings and against which the future detailed DAs will be assessed.

The Masterplan DA seeks approval for the maximum building envelopes for future stages of development, the maximum gross floor area (GFA) and land uses for the development. Specifically:

- A mixed use development involving a maximum of GFA of 281,905m^2, including:
  - residential flat buildings comprising private, social and affordable housing
  - seniors housing comprising residential care facilities and self-contained dwellings
  - a new high school
  - child care centres
  - minor retail development

  - community uses
- maximum building heights and GFA for each development block;
- public domain landscape concept, including parks, streets and pedestrian connections;
- provision of the Ivanhoe Estate Design Guidelines to guide the detailed design of the future buildings; and
- vehicular and intersection upgrades.

An image of the Masterplan DA is provided at Figure 11 below.

Figure 11: Ivanhoe Estate Materplan

4.2 Expected Works

The proposed development at the site will include the demolition of the existing buildings, followed by subdivision of the site into new blocks, followed by the construction of the new access roads and multi-storey buildings.

Excavation will be required for proposed two to four-level basements parking levels, below most buildings, and between some of the structures. Some more minor cut and fill earthwork will be required for earthworks for the proposed roads, where surface levels of up to approximately 1 m below and 2 m above existing levels are proposed.

At Shrimptons Creek, at the eastern corner of the site, a bridge is proposed to extend one of the roads over Shrimptons Creek and to connect with a new access road at 2-4 Lyon Park Road. The site at 2-4 Lyon Park Road has been considered in the separate report (86043.00.R.002), enclosed in Appendix C. Therefore, only the northern bridge abutment is discussed within this report.

5. Comments

5.1 Background Information

DP has undertaken numerous past investigations in the area with approximate locations shown in Figure 12, below.

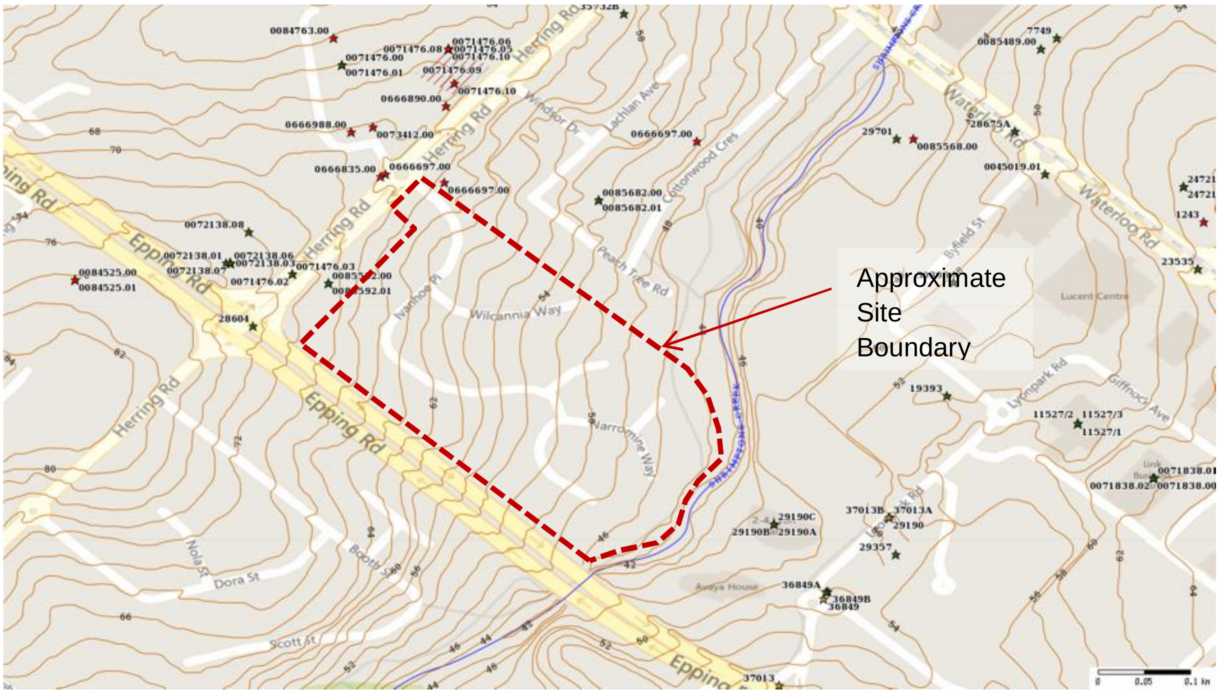


Figure 12: Previous DP project locations (stars)

A review of past DP projects in the site vicinity indicates that, in contrast to the geological mapping, Ashfield Shale is not likely to be present on the subject site, but that some limited depth of Mittagong Formation (presenting as up to approximately 3 m of siltstone, laminite or fine grained sandstone) is likely towards Herring Road, with Hawkesbury Sandstone underlying the Mittagong Formation and below the remainder of the site. This is consistent with the thinly bedded strata observed in the northern part of the site during the walkover.

The review also indicates that:

- In addition to the dyke noted in Figure 10, a second, unmapped, approximately north-south oriented dyke is present to the north-east of the site. If extended, this dyke would be present on the far side of Shrimptons Creek (i.e. east of the subject site), although it is noted that dykes may "step" or fork in plan, and thus may intersect the subject site.

- Thrust faulting, often associated with dykes, has been identified at a nearby site to the north-west of the subject site. A photograph showing the subsurface profile exposed by bulk excavation on this site is included in Figure 13.
- Groundwater monitoring at discrete time intervals upslope of the site identified water levels within the sandstone, at depths of between 2 m and 12 m, although only very minor seepage occurred into the resulting (3 basement level) excavation.

A geotechnical and hydrogeological model of the site, based on the published mapping and previous DP experience, is given in Section 4.2.

Figure 13: Back thrusts in an excavation wall at a nearby site

5.2 Geotechnical and Hydrogeological Model

A preliminary (and broadly generalised) geological model for the site, based on the above information, is summarised as follows:

- **Filling –** including pavement materials, past filling from onsite and possibly imported materials, to variable depth but typically less than 1 m depth, though possibly deeper, particularly towards the creek, underlain by,
- **Residual Soil** – likely to be generally stiff and very stiff silty clay, sandy clay and clayey sand, possibly with some ironstone or sandstone gravel fragments, to typical depths of 0.2 m to 1.5 m, though likely to be generally deeper (and possibly weaker, with overlying alluvial soils) towards Shrimptons Creek; underlain by,
- **Sandstone –** including the following strata:
    - Highly and extremely weathered sandstone, possibly including some laminite and siltstone bands, with inter-bedded bands of extremely low, medium and high strength rock and clay seams. Some very high strength, iron-cemented bands may also be present. The typical thickness of this highly and extremely weathered sandstone is likely to be in the range of 0.5 m to 2 m. Such a profile is likely to be deeper towards Herring Road, where the Mittagong Formation is more likely to be encountered, or in areas of thrust faulting (if present); underlain by,

- Medium and high strength sandstone, possibly with some bands of weaker rock or clay seams; underlain by,
- Consistently high strength sandstone, limited defects. Some very high strength bands may also be present.

While not confirmed at the subject site, based on the available information there is a relatively high risk of dykes and/or thrust faults being present at this site, within the bedrock. The following notes are made in this regard:

Dykes in Sydney are typically near-vertical, planar features that may change in thickness, become discontinuous and/or step in plan. Common dyke widths in Sydney range from less than 1 m to approximately 6 m. They are typically completely weathered basalt or dolerite (clay) near surface and are usually weathered and weaker than the surrounding rock to significant depth. The rock adjacent to the dyke can also be highly fractured, variable, or abnormally high strength due to the heat and pressure effects of the intrusion. Higher permeability and greater water seepage is also often observed within and on either side of the dyke material.

Thrust faults are often associated with dykes, though may affect a larger area. Thrust faults may vary significantly between sites, and their presence may relatively abruptly discontinue within a site. Their impact may include a band of fractured rock (e.g. the thrust fault, or back thrust), to thick zones (to several metres depth) of altered sandstone affected by iron-cementation and bleaching, as seen in Figure 13.

Permanent groundwater levels are expected to be at depths of greater than 5 m to 10 m within the sandstone, with seepage generally through weathered bands and defects within the rock mass. Temporary, perched water seepage is anticipated within the soils, particularly towards the base of filling and along the top surface of the bedrock, and usually associated with periods of rainfall.

5.3 Geotechnical Issues

The primary geotechnical issues for the proposed development at this site are expected to be:

- Excavation – excavatability, vibrations, excavation support (and movement), groundwater seepage;
- Footings – footing type and bearing capacity;
- Earthworks; and,
- Pavements.

5.4 Excavation

Soil, extremely low and very low strength rock are likely to be readily excavated using conventional earthmoving equipment. Higher strength rocks are likely to require excavation by ripping tynes mounted on large bulldozers, large rock hammers, rock saws and milling heads. The excavatability of the medium and high strength sandstone will be governed by the defects within the rock mass, which may be widely spaced within these materials, with decreasing defects with depth. In general, excavation of high strength sandstone is likely to be difficult and slow with low productivity and high hammer/tyne wear expected.

Significant vibrations are anticipated during excavation within low to high strength sandstone. Excavation methods may therefore be limited by acceptable vibration levels, particularly towards neighbouring sites or services, or previously constructed buildings if the works are phased.

Batters or excavation support will be required for excavations through soil and extremely low to very low strength sandstone, and into the upper, higher strength sandstone where significant weak bands or steeply oriented defects are more likely to be present.

Preliminary safe batter slopes are provided in Table 1, for batter slopes no greater than 3 m in height, with horizontal ground beyond the crest and below the toe, no deflection sensitive structures or services above the crest, no surcharges above the crest and no seepage from the face. All batter slopes are subject to inspection by a geotechnical engineer.

Table 1: Preliminary Safe Batter Slopes

Material	**Maximum Temporary Safe Batter Slope (H:V)**	**Maximum Long-Term Safe Batter Slope (H:V)**
Filling	1.5:1	2:1
Stiff to very stiff clay	1.5:1	2:1
Extremely low to low strength rock	1:1	2:1
Extremely low and medium strength rock	0.75:1, but dependent on jointing	1:1, but dependent on jointing
Medium or High Strength Rock	vertical, but dependent on jointing	vertical, but dependent on jointing

Steeper batters, or batters subject to surcharges, adjacent sloping ground, seepage or potentially subject to flooding, would generally require more detailed assessment.

Within the consistently medium and high strength rock, the rock is likely to be able to be cut vertically and stand unsupported, subject to defect and localised stability assessment. Defect assessment is recommended to identify if additional local support (e.g. bolts or anchors) and/or shotcrete is required.

Long-term batters in soil should be protected against erosion, such as by shotcrete or vegetation. Shotcrete is also likely to be required to protect against weathering of extremely low to low strength rock. For permanent batters protected by vegetation, flatter slopes of 3H:1V are recommended to allow for adequate slope maintenance. Separate scour protection assessment would generally be appropriate for any batters subject to flowing water.

All excavated material to be removed off site, will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the *Waste Classification Guidelines* (EPA, 2014). This includes filling and natural materials that may be removed from the site.

5.5 Excavation Support

Where there is insufficient space to accommodate the above batters, excavation support in the soils and extremely low to low strength rock could be by soldier pile shoring walls, with infill shotcrete panels. Typical soldier pile spacings at 2 m to 2.5 m are likely to be suitable for the support of natural clay soils and weathered rock above the groundwater table.

Bored piles are likely to be suitable for the drilling of shoring (soldier) piles at this site. Casing may be required where deep or granular filling is present to reduce the risk of side-wall material falling in to the hole.

For basements near Shrimptons Creek, if groundwater levels are well above rock level, then secant pile "cut-off" walls may possibly be required in the affected areas, extending below basement level. Continuous flight auger (CFA) piles may be required to protect against collapsing ground conditions (e.g. water saturated filling, or high rates of groundwater inflow) are present. Further investigation is warranted in these areas to assess whether these geotechnical conditions are present, as shallow sandstone would likely preclude such conditions.

Anchor support of shoring will generally be required if movement-sensitive structures (e.g. building foundations) are present behind the walls or, where the excavation is deep (e.g. more than about 3 m). Anchors or bolts may also be required at the toe of soldier piles, if the piles are not socketed below basement level, or if adverse jointing is present within the rock face.

For bored or CFA piles, a high torque drilling rig is likely to be required to obtain significant socket into medium and high strength sandstone, as may be present at this site.

Preliminary parameters for the design of shoring/retaining walls within these materials are summarised in Table 2. The parameters are based on 'simply-supported' retaining walls (with a triangular pressure distribution), where movement of the wall is permitted, and assuming no significant adverse jointing within the rock.

Table 2: Preliminary Design Parameters for Permanent Retaining Walls

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient/Value	
		Active	Passive
Filling	20	0.4	3
Stiff to Very stiff Clay	20	0.3	2
Extremely low to low strength rock	24	0.25	200 kPa (2)
Extremely low and medium strength rock (1)	24	0.2	400 kPa (2)
Medium or high strength rock (1)	24	0	4000 kPa (2)

Note: (1) Additional support may be required due to defects, subject to inspection of faces

(2) Ultimate passive pressure

Allowance should be made for the provision of drainage behind retaining walls, or alternatively the walls should be designed for full hydrostatic pressures (ie. water rising to the measured groundwater level plus an allowance for rise).

The presence of dykes or thrust faulting may result in locally poorer rock conditions, which may lead to addition support being required in some areas of the site (or over a short distance). Detailed investigation and/or careful monitoring and inspection of ground conditions during excavation (including for soldier piles) would generally be appropriate to ensure that support is taken down to an appropriate depth in any affected areas.

Survey monitoring of excavation would generally be appropriate to assess movement of any shoring walls during excavation. Where basements are proposed near RMS roads, then the installation of inclinometers and progressive monitoring with excavation depth may also be required by RMS.

In addition to retaining wall movements, major excavations into sandstone bedrock may result in lateral movement of the sandstone faces due to stress relief effects. Release of these stresses may cause horizontal movements along the rock bedding surfaces and defects, with estimated movements of approximately 0.5 mm to 2 mm per metre depth of excavation at the midpoint of the excavation. It is not practical to provide restraint against these movements, and appropriate allowance should instead be made for such movements in construction and planning.

5.6 Groundwater and Dewatering

From an engineering perspective, and assuming basement levels are kept above likely flood levels of Shrimptons Creek, groundwater seepage into basement excavations is likely to be readily managed using ‘sump-and-pump’ methods. This is consistent with DP experience with excavations near the subject site. Such drainage is expected to include strip drains behind shotcrete faces, connecting to perimeter and sub-floor basement drains directing to a sump, or simply downslope, for basements constructed into the existing slope. Such seepage is likely to be iron-rich and a precipitate (gelatinous ‘sludge’) may develop within drains over time. Allowance should be made for future maintenance to clear such material from the drainage lines and from pump fixtures.

Depending on ground conditions, cut-off walls may be required over the lower part of the site if basement levels are taken below the design flood levels within soil.

The Department of Primary Industries (DPI) has been increasingly requiring tanked basements to limit dewatering and disturbance of aquifers. Tanking requirements are unlikely to be appropriate where shallow bedrock is present, as is the case over much of the Ivanhoe Estate, although groundwater modelling may potentially be required by DPI to assess the potential impact. A regulatory requirement for tanking is considered likely to apply if proposed basement levels are taken below the relevant Shrimptons Creek design flood levels, particularly if ground conditions warrant the use of cut-off walls. DPI requirements may also include the need for subsurface drainage for diversion of subsurface water flows around basements.

5.7 Foundations

At this site it would generally be recommended that foundations for structures be taken down to uniformly bear on bedrock. Given the proposed basement levels, this is expected to typically involve shallow foundations at basement level. For associated structures, shallow or pile foundations may be required, depending on the proposed filling or excavation for the affected area, and desired founding material. Pile foundations are expected to be required for the proposed bridge abutments.

Maximum allowable bearing pressures for preliminary design of foundations supported within the bedrock likely to be encountered at the site are provided in Table 3.

Table 3: Preliminary Allowable Foundation Design Parameters

General Material Description	Allowable Bearing Pressure[2]	Recommended Minimum Testing/Requirements[1]
Very low strength sandstone or laminite, or higher strength sandstone with significant defects	1 MPa	See Note 1
Low strength sandstone or higher strength but with significant defects	2 MPa	Cored boreholes
Medium strength sandstone, limited defects	3.5 MPa	Cored boreholes, greater test density
Medium to High strength sandstone	6 MPa	Cored boreholes, greater test density, spoon testing of at least 1/3 of footings

Note: 1 Geotechnical investigation and inspection of footing excavations is recommended in all cases; minimum testing is to provide guidance on additional requirements for investigation and inspection, depending on desired bearing pressures.

2. All bearing pressures may be limited by defects, subject to inspection of the excavation and possible spoon testing, which may require the bearing pressure to be downgraded.

Higher bearing pressures (up to, say, 10 MPa allowable) may potentially be available, depending on the depth of basement excavation and local site conditions. Such bearing pressures would require a significant test density during the investigation phase and proving footings, and may be associated with a higher risk of inspection ‘failures’ (i.e. with rock indicating a lower bearing capacity, or further deepening of foundations required). The use of this bearing pressure is therefore not recommended at this time, but may be appropriate to consider after geotechnical investigation at the site, and subject to further investigation or proving.

If thrust faults or dykes are present within the site, then this is likely to result in lower allowable bearing pressures. The following solutions would generally need to be considered:

- For dykes, the likely foundation solution will involve downgrading the rock strength on either side of the dyke, and structurally bridging foundations across the dyke. As the dyke width and strength of rock on either side is likely to be variable, this would normally be assessed at construction stage;
- For thrust faults, the solution will depend on the area affected, but is more likely to include downgrading of the allowable bearing pressure, or deepening of footings to below the affected zone.

Where pile foundations are required, open, bored piles likely to be adequate for most situations, although continuous flight auger (CFA) piles may be required for piles near Shrimptons Creek (e.g. for bridge abutments). Powerful high-torque rigs may be necessary to form pile sockets in medium and high strength sandstone.

5.8 Earthworks

New building platforms, pavements and level areas (eg. Northern bridge abutment) are likely to be required as part of the development. It is generally recommended that new filling areas are constructed as controlled filling. It is suggested that the following general procedure be adopted:

- Excavate any topsoil, filling and any deleterious materials (and natural soil if required to achieve levels) at the site, retaining the excavated material in separate stockpiles for assessment for possible re-use;
- Test-roll the underlying natural site soils to identify any soft or yielding areas which require additional treatment (e.g. replacement), and treat such materials;
- Place filling in uniform layers not exceeding 250 mm (loose) layer thickness and compact to an appropriate density. Appropriate densities and compaction ratios will depend on the filling material adopted, and proposed use of the platform. Material should be placed and compacted beyond any proposed batters, to ensure suitable compaction, with excess material later trimmed from the batter face;
- Undertake inspection and testing of compaction activities at a suitable frequency. Guidance on test frequencies may be sought from AS3798 *Guidelines on Earthworks for Commercial or Residential Developments.*

From a geotechnical perspective, material reuse is likely to be practicable, noting the following:

- Excavated topsoil is likely to be suitable for re-use in landscaping purposes, only;
- Other than topsoil, excavated (existing) filling and natural soils are likely to be largely suitable for reuse in controlled filling, subject to assessment, removal of oversize and organic materials and probable moisture conditioning;
- Excavated sandstone may potentially be suitable for reuse in controlled filling, subject to crushing or ripping and sorting of the material to remove oversize rock.It is noted that space is required to allow for stockpiling, and time and favourable weather conditions are generally required for moisture conditioning of filling. Crushing of medium and high strength rock on site may not be cost-effective, and may present noise issues. Other project considerations may make these options unfavourable.

Where pavement construction is required on cuts into bedrock, allowance may need to be made for special drainage provisions (e.g. provision of a drainage blanket at the base of the excavation), to provide for seepage and prevent excessive moisture developing within the pavement layers.

5.9 Pavements

Moss growth was observed on roads and retaining walls in several areas on the site, suggesting that existing soil moisture levels are relatively high. Control of subsurface moisture by the use of appropriate drainage, will be critical to the performance of any pavements.

Based on past DP experience, CBR values of 3% to 10% are generally observed in natural soils developed from the Mittagong Formation and Hawkesbury Sandstone. A CBR of 5% is suggested for preliminary design purposes, until further laboratory testing may be undertaken. This value assumes that the pavement subgrade may be maintained near the equilibrium (or optimum) moisture condition.

For pavements subject to light traffic loads only (design ESA of less than 10^5) on a subgrade with CBR of 5%, a typical pavement thickness design may comprise:

- a thin asphaltic concrete layer (<40 mm); underlain by,
- 100 mm thickness of base quality material (CBR ≥ 80%); underlain by,
- 300 mm of granular subbase.

It is noted that the proposed staging suggests that construction will commence in areas near Herring Road, followed by staged development down the slope. As a result, construction traffic for the later stage developments may pass over roads constructed as part of the early stage development. Given that the construction traffic is likely to be a significant load on the pavements, it may be preferable to allow for the construction of temporary roads until most construction work is complete, followed by replacement with the permanent road formation. This would involve accepting more rapid deterioration of the temporary roads, while allowing the permanent roads to be designed for the lower, largely residential, long-term traffic load.

For both temporary and permanent roads, the performance of the pavements is likely to be governed by moisture levels within the subgrade, and therefore by the design of suitable surface and subsurface drainage for the site. Subsoil drainage should be installed to at least 500 mm below subgrade levels along the high side of all pavements and adjacent to lawn and garden areas. It is generally impractical to design roads for flooding conditions due to changes in soil behaviour with increased moisture content. Therefore, the use of rock fill and concrete pavements and approach slabs are common for bridge abutments. It is suggested that imported rockfill be assumed at this stage, although further detailed testing of the sandstone may be used to assess whether ripped sandstone rock fill may be practical.

If rock is intersected at the base of the road formation, drainage requirements may include a drainage blanket, or similar cross-sectional drains, to allow for seepage from the rock surface to be transported away from the road formation.

5.10 Further Assessment and Investigation

Geotechnical investigation will be required to assess actual on-site conditions for specific structures, and to confirm or revise the geological model developed for the site, and the corresponding geotechnical advice.

It is understood that due to expected access limitations in accessing the site, a staged investigation process is preferred. Based on the assessment to date, the suggested scope of next-phase investigation would include:

- **Selected cored boreholes in accessible areas** – to “ground-truth” the desk-top assessment and provide further information in areas where only limited data is available. These would preferably include:
    - Bores near the basement perimeter near Shrimptons Creek, to assess whether rock levels may preclude the need for cut-off walls for basement design and construction;
    - Bores at the proposed bridge abutment, to assess foundation design and construction conditions.

  - Bores at selected accessible locations through the site, likely in roadways and preferably in or near the proposed basement footprints, to provide greater confidence in the consistency of ground conditions across the site. A low test/borehole density is suggested due to current access issues, and further testing would be recommended to provide greater confidence for detailed design. If practical, greater density would be focussed in areas of earlier ("Stage 1") development.

- **Installation of standpipes and dataloggers** – at selected bores to allow for monitoring of groundwater levels. This will assist with responses to expected DPI requirements with regard to tanking and dewatering. It is noted that as groundwater levels do fluctuate over time, a longer period of monitoring with dataloggers is often useful.

In conjunction with the above, the following testing may also be considered as part of the preliminary test scope. These works may be undertaken at a later time, with staging is likely to depend on the priorities of the design and construction teams,:

- **Test pits** – to provide information on the depth of filling and obtain samples to assess CBR to provide increased confidence for pavement design. Such testing is relatively intrusive, and replacement of filling under roads and accessible areas will be more expensive at this stage than if undertaken when the site is unoccupied and the test pits may be more simply backfilled.
- **Laboratory testing for waste classification purposes** – may be undertaken on soil samples obtained from the boreholes and/or test pits, to provide a preliminary indication of waste classification, to assist in assessment of disposal costs.
- **Other laboratory testing** – including petrographic testing of sandstone (to assess durability for use in rockfill), soil properties near Shrimptons Creek (to inform future scour assessment), soil and groundwater aggressivity (for structural design), groundwater chemistry (for preliminary assessment for disposal).
- **Rising head tests** – at standpipe locations, for preliminary assessment of groundwater inflow.

The above outline is intended to facilitate discussion. The actual investigation methods, and number of test locations will depend on the priorities of the design and construction teams and expected accessibility of the site for testing. Those priorities may dictate that other methods of investigation are appropriate.

6. Limitations

Douglas Partners (DP) has prepared this report for this project at Ivanhoe Estate, Macquarie Park in accordance with DP's proposal dated 7 June 2017 and acceptance received from Joe Zannino dated 29 June 2017. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Frasers Property Group Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site 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 observed on the site and inferred from geological and other mapping and nearby DP investigations. 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 site work has been completed.

The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site. The advice may also be limited by scope 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. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- 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

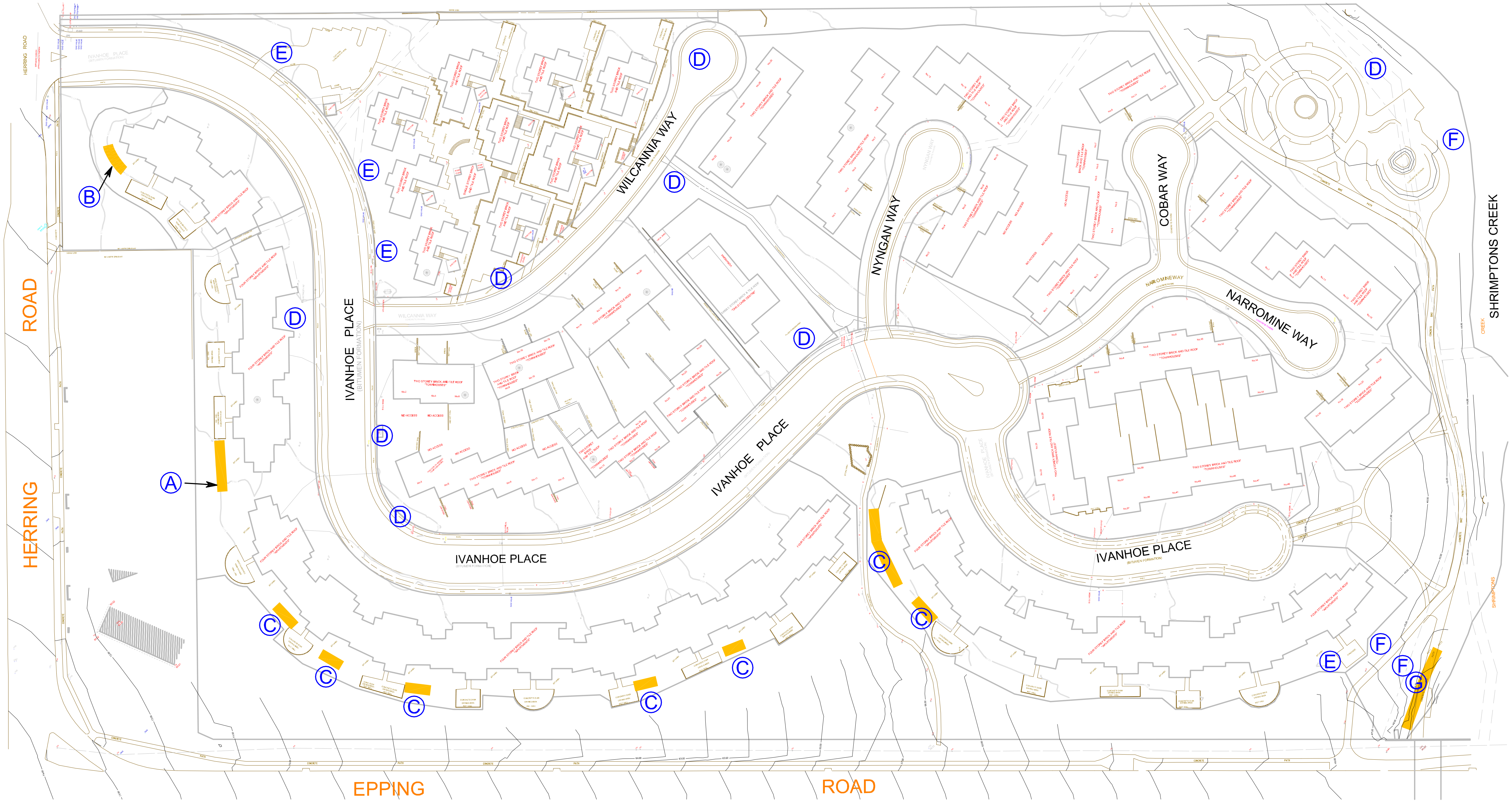
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate 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



Ⓐ Rock Cut. Following strata observed.
Soil to ~ R.L. 66.8
Very low to low strength shaleto ~ R.L. 66.5
Low to medium and high strength laminated shale and silstone to ~ R.L. 65.5
Medium strength, fine grained sandstone to ~ R.L. 65

Ⓑ Extremely weathered shale and medium strength sandstone
(Inaccessible for inspection)

Ⓒ Very low, low and medium strength sandstone
(Inaccessible for inspection)

Ⓓ Mossy growth/moist ground

Ⓔ Fill batter

Ⓕ Disturbed ground and sandstone boulders

Ⓖ Medium strength sandstone

🟧 Rock outcrop/exposure

Locality Plan

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Frasers Property Australia Pty Ltd
OFFICE: Sydney
DRAWN BY: Vojta
SCALE: N.T.S.
DATE: 14.7.2017

TITLE: **Site Features**
Proposed Residential Development
Ivanhoe Estate, Macquarie Park

PROJECT No: 86043.00
DRAWING No: 1
REVISION: 0

Appendix C

DP Report 86043.00.R.002.Rev0
Geotechnical Desktop Assessment at
2-4 Lyon Park Road, Macquarie Park

Douglas Partners
Geotechnics / Environment / Groundwater

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Frasers Property Australia Pty Ltd
c/- Citta Property Group Pty Limited
Level 23
6 O'Connell Street
SYDNEY NSW 2000

Project 86043.00
03 August 2017
R.002.Rev0
SCP:cjm

Attention: Joe Zannino

Email: JoeZannino@citta.com.au

Dear Sirs

Geotechnical Desktop Assessment
Proposed Residential Development
2-4 Lyon Park Road, Macquarie Park

1. Introduction

This letter report presents the results of a desktop geotechnical assessment undertaken by Douglas Partners Pty Ltd (DP) for the construction of a proposed road along the north-eastern side of 2-4 Lyon Park Road. The assessment was commissioned in an email dated 29 June 2017 by Joe Zannino of Citta Property Group Pty Limited on behalf of Frasers Property Australia Pty Ltd.

The proposed road will extend from Lyon Park Road along the north-eastern boundary of 2-4 Lyon Park Road, then onto a new bridge over Shrimptons Creek. Only the south-eastern abutment of the bridge is considered in this report. (Reference should be made to DP Report 86043.00.R.001 in relation to the abutment on the north-western side of Shrimptons Creek.) The development also includes new car parking and landscaping areas adjacent to the new road.

The aim of the desktop assessment is to consider the subsurface soil and groundwater conditions expected at the development areas in order to provide information on the proposed road and bridge construction.

The assessment included:

- the review of selected mapping for the area;
- review of DP's database of past investigations in the vicinity of the site; and,
- a brief site visit to assess site conditions and make observations.

2. Site Description

The greater site at 2-4 Lyon Park Road is occupied by a central, multi-storey commercial building with three basement parking levels. The ground around the site is generally paved, with some grassed and vegetated garden areas towards the rear (north-west) of the site. Ground levels at the site slope gently down from RL 52 towards the southern corner of the site, to approximately RL 41 at Shrimptons Creek at the rear of the site.

From discussions with the Facilities Manager at the site, it is understood that the rear portion of the site is susceptible to flooding from Shrimptons Creek, and that a recent flood event resulted in damage to a chain-link fence at the rear of the site, apparently by undermining of the shallow concrete foundations.

Of particular interest to this report is the subject area within 12–15 m of the north-eastern boundary, which is proposed for the road development. In this area, the site is currently generally paved, as shown in Figure 1, with some landscaped garden areas.

Figure 1: Subject new road alignment, facing west from Lyon Park Road

The existing pavement comprises interlocking segmental paving blocks and is generally in good condition. Pavement levels within the subject development area slope gently down from approximately RL 49 at Lyon Park Road towards Shrimptons Creek at approximately 1° to 2°.

Beyond the pavements, still within the subject development area, ground levels generally slope down at approximately 8° to 10° from RL 46 at the pavement edge to approximately RL 41 at Shrimptons Creek. The site is generally grassed in this area, with some trees, and the surface of some sandstone boulders is visible at ground surface indicating filling or disturbed soils. The ground surface was observed to be moist at the time of the site visit.

Higher ground levels on the adjoining site at 6 Lyon Park Road (for pavement areas, at approximately RL 49) are supported by crib walls on the common site boundary and, in some parts of the site, by batters within the proposed road corridor. The difference in level between the sites increases towards Shrimptons Creek and at the rear half of the site, mulched and vegetated batters rise along the proposed corridor towards the toe of the crib walls. The batters are up to 2 m high, and support crib walls of typical heights of 1.5 m to 2 m (see Figure 2)

Figure 2: Mulched batter slope and crib wall at rear boundary between 2-4 and 6 Lyon Park Road

At the rear of the proposed road corridor, a stormwater outlet drains to Shrimptons Creek. The outlet has steep sandstone boulder headwalls and concrete sidewalls, built into the steeper batter in this area. The concrete and sandstone apron may possibly be constructed on sandstone blocks or outcrop, but was not accessible for safe inspection during the site visit.

Figure 3: Stormwater outlet at Shrimptons Creek, with concrete sidewalls and sandstone boulder headwall, as viewed from north-western bank.

Sandstone boulders were visible within the soil creek banks, and the banks had apparently been subject to previous erosion in some areas. A chain-link fence generally separated the site from Shrimptons Creek, with the area of fencing undermined by recent erosion located south-west of the subject area.

3. Regional Mapping

Reference to the Sydney 1:100 000 Soil Landscape Series Sheet indicates that the site is underlain by the residual Lucas Heights soil landscape towards Shrimptons Creek, and by the erosional Glenorie soil landscape towards Lyon Park Road. These natural soils are likely to comprise sandy clay and clayey sand soils, possibly with some silty clay.

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is mapped as underlain by Hawkesbury Sandstone under most of the site, with a boundary with Ashfield Shale towards Lyon Park Road. The Ashfield Shale, where present, would be underlain by Hawkesbury Sandstone, possibly with a transitional Mittagong Formation (interbedded shale, laminite and sandstone) between the two layers. DP experience is consistent with the Hawkesbury Sandstone mapping.

There are no registered groundwater bores in the immediate area (ie. around the proposed road corridor and new bridge). The site is outside of areas of known salinity or acid sulphate soils, although some layers within the Glenorie soil landscape can be associated with local occurrence of salinity.

4. Past Field Work

DP has undertaken past investigation and inspections on the subject site. Five boreholes were undertaken near the proposed development corridor in August 2000, prior to the construction works for the existing building (refer to Bores 1 to 5, at the locations shown in Drawing 1, attached). The test bore reports for these locations are also attached, together with the relevant notes. The boreholes indicate that:

- Poorly compacted filling was present to depths of up to 1.8 m, (though noting that the earthworks involved in the construction of the existing building and pavements are likely to have altered this upper profile, potentially removing some or all of the unsuitable filling and/or the placement of new, possibly engineered filling).
- The natural soils underlying the filling generally comprised soft, firm and firm to stiff silty, sandy clay, sometimes with ironstone gravel. While a thick band of ironstone was noted at Bore 1,given the drilling methods, it is considered likely that this refers to a layer of weakly iron-cemented soil or rock, or ironstone gravel, rather than solid ironstone.
- Sandstone was identified underlying the natural soils at Bores 2 to 5, at levels falling from RL 45 at Bore 5 to RL 42.9 at Bore 2. The sandstone ranged from extremely low strength, improving to high strength, with strength generally improving with depth.
- No free groundwater was observed whilst augering at borehole locations, however it was noted that “considerable seepage and saturated soil conditions” were present, particularly over the rear third of the site during the investigation. These observations suggest that very high moisture levels were present within the soils, though seepage flows may be localised, temporary or slow-flowing (based on the absence of water inflow to boreholes whilst augering).

5. Comments

5.1 Proposed Development

The construction of a new pavement is proposed along the north-eastern side of the site, with a pavement width of approximately 7 m.

The works will include associated works to relocate existing services in the area, which will include the construction of additional parking bays, landscaping and moving electrical substation(s).

A new bridge is proposed at Shrimptons Creek, and the south-eastern bridge abutment will be located within the subject road corridor. The precise location of the south-eastern abutment is not currently known.

5.2 Geotechnical and Hydrogeological Model

The geotechnical and hydrogeological model for the subject area is as follows:

- **Filling** – comprising likely well-compacted, possibly partly lime-stabilised filling below existing pavements (only), and uncontrolled, poorly compacted filling below the grassed and landscaping areas, including an upper layer of topsoil; to typical depth of 0.5 m to 1.5 m below ground level, but likely deeper in some areas; underlain by,
- **Sandy and Silty Clay** – soft and firm to stiff, moist to wet, to depths of approximately 2 m to 3 m below ground level, possibly deeper towards Shrimptons Creek; underlain by,
- **Sandstone** – extremely low strength sandstone, improving with depth to high strength sandstone, possibly including bands of iron-cemented strong and weak (bleached) layers.

Groundwater levels are generally expected to be within the sandstone, or towards the base of the natural soils. Towards Shrimptons Creek, where deeper rock may be present, groundwater levels may be within the soils. Temporary groundwater levels are likely to be present towards the base of the filling and natural soil layers, particularly following rainfall, with rainfall events expected to result in elevated groundwater levels, particularlyin the vicinity of Shrimptons Creek. Based on previous observations at the site, these elevated groundwater levels may persist for some time following wet weather, although they may also be alleviated by drainage installed during the site development.

Even where groundwater levels are not elevated, soils in the vicinity of Shrimptons Creek are likely to have a relatively high moisture content.

5.3 Foundations

Foundations for the new bridge should be taken down to bear on sandstone bedrock. Pile foundations are likely to be required. Contiguous flight auger (CFA) piles may be required for construction if groundwater levels are above the bedrock surface to avoid the risk of sidewall collapse and major seepage inflows. Augered pile holes with temporary casing, to protect against sidewall collapse may be considered, but such temporary casing may be ineffective if groundwater inflow into the pile hole is significant.

Where pile foundations are taken down to bear on at least consistent, medium strength sandstone, an allowable bearing pressure of up to 3.5 MPa may be adopted for preliminary design purposes. Specific geotechnical investigation using cored boreholes at the abutment locations would generally be required in advance of the works to confirm the bearing capacity. Higher bearing capacities could potentially be achieved, subject to the material encountered by the investigation.

Shallow foundations may be appropriate for minor structures associated with the road extension, such as the substation(s). These relatively lightly loaded structures could be founded on at least stiff, natural clay soils, based on an allowable bearing capacity of 100 kPa. It would generally not be appropriate to support structures on filling that has not been placed and engineered for that purpose.

5.4 Earthworks

The proposed road formation levels have not been confirmed at this time, and are likely to depend on the desired road alignment, interface with adjacent buildings and roads, and construction practicalities. Such practicalities may include the following:

- Saturated soil conditions identified towards the rear of the site during previous investigations suggests that there may be difficulties undertaking earthworks during wet weather. Raised road levels may be an alternative approach, to limit the influence of any soft or saturated soils on the road formation, although batter stability will need to be appropriately addressed. Also, the construction of engineered fill platforms (eg. for abutments) over soft soils may requires either the improvement/treatment of such soils to reduce consolidation settlement to acceptable levels, or the removal and replacement of such soft soils. The use of rockfill with concrete pavements and approach slabs is a common solution for the design of pavement batters that will be subject to flooding.
- Care will need to be taken during any excavation for earthworks below the adjacent crib walls, as this may cause movement of the walls. Assessment and investigation will generally be required to assess the foundation level of the existing boundary retaining walls, foundation strata and any passive support requirements which may dictate the approach to excavation. The following options may be available for excavation:
    - Embedded piled shoring walls could be used to support the existing boundary retaining walls, and allow ready excavation in front of the wall. A piled wall is likely to be required to provide rigidity to the sides of the excavation. This will require some offset from the wall in order to install the piles, and piles may need to be relatively closely spaced or possibly contiguous to provide appropriate support;
    - Alternatively, where excavation is only required for the short-term and the soil below the foundation is of adequate strength, the use of a ‘hit-and-miss’ excavation and replacement sequence (ie. in alternate ‘panels’) could be considered. This can be a relatively time consuming process, but can avoid the need for pile installation. The width of panels would require further assessment;
    - It is possible in areas of limited excavation, if favourable ground conditions and wall design exist, that batters and benches may be used. This may not be compatible with the current road geometry, particularly towards the rear of the site, where steep batters are already present below the ‘high-side’ retaining wall along the site boundary.

Given the above, the proposed road would ideally limit excavation requirements below the crib wall.

5.5 Pavements

In order to provide a suitable subgrade for pavement and road construction, it is suggested that the following procedures be adopted:

- Excavate the existing pavement materials and filling, and stockpile for possible re-use. Some of the filling, particularly towards the rear of the site, may be unsuitable for re-use. Where re-use is practical, the filling is nonetheless expected to require removal of unsuitable material (e.g. organic material, over-wet or highly saturated soils and sandstone cobbles/boulders) and moisture treatment prior to reuse as engineered filling.

- Proof roll the underlying firm to stiff clay to identify any exceptionally soft and yielding areas which should be removed;
- Place filling in uniform layers and compact to an appropriate density/compaction level. The appropriate densities and compaction ratios will depend on the filling material adopted, proposed traffic loading and level of the fill layer relative to the top-of-subgrade.

It is recommended that any new filling is placed as controlled filling, with inspection and testing of compaction activities undertaken at a suitable frequency. Guidance on test frequencies may be sought from AS3798 *Guidelines on Earthworks for Commercial or Residential Developments.*

Appropriate surface and subsurface drainage is critical to pavement performance, and should be carefully considered in the design. Towards the rear of the site, where saturated soil conditions have been previously observed, this may require the use of a drainage blanket to limit the ingress of moisture into the road pavement layers. The efficacy of such drainage will depend on the pavement levels and vulnerability of the drainage system to flooding from Shrimptons Creek. Robust subsoil drainage should be installed to at least 500 mm below subgrade level along the high side of all pavement areas and adjacent to garden/grassed areas

In the absence of specific laboratory testing it is suggested that a CBR of 5% is adopted as a preliminary value for pavement design, given the previous earthworks at the site and reasonable condition of the existing pavement. This CBR assumes that equilibrium moisture content is maintained within the subgrade, and thus is reliant on appropriate subsurface drainage.

6. Limitations

Douglas Partners (DP) has prepared this report for this project at 2-4 Lyon Park, Macquarie Park in accordance with DP’s proposal dated 7 June 2017 and subsequent email of 30 June and acceptance received from Joe Zannino of Citta Property Group Pty Limited (on behalf of Frasers Property Group Pty Ltd.) dated 30 June 2017. The work was carried out under DP’s Conditions of Engagement . This report is provided for the exclusive use of Fraser Property Australia Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site 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.

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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. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Please contact the undersigned if you have any questions on this matter.

Yours faithfully
Douglas Partners Pty Ltd

Sally Peacock
Geotechnical Engineer/Associate

Reviewed by

Bruce McPherson
Principal

Attachments:	About this Report Drawing 101 Previous Test Bore Results and Related Notes

About this Report

Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- 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

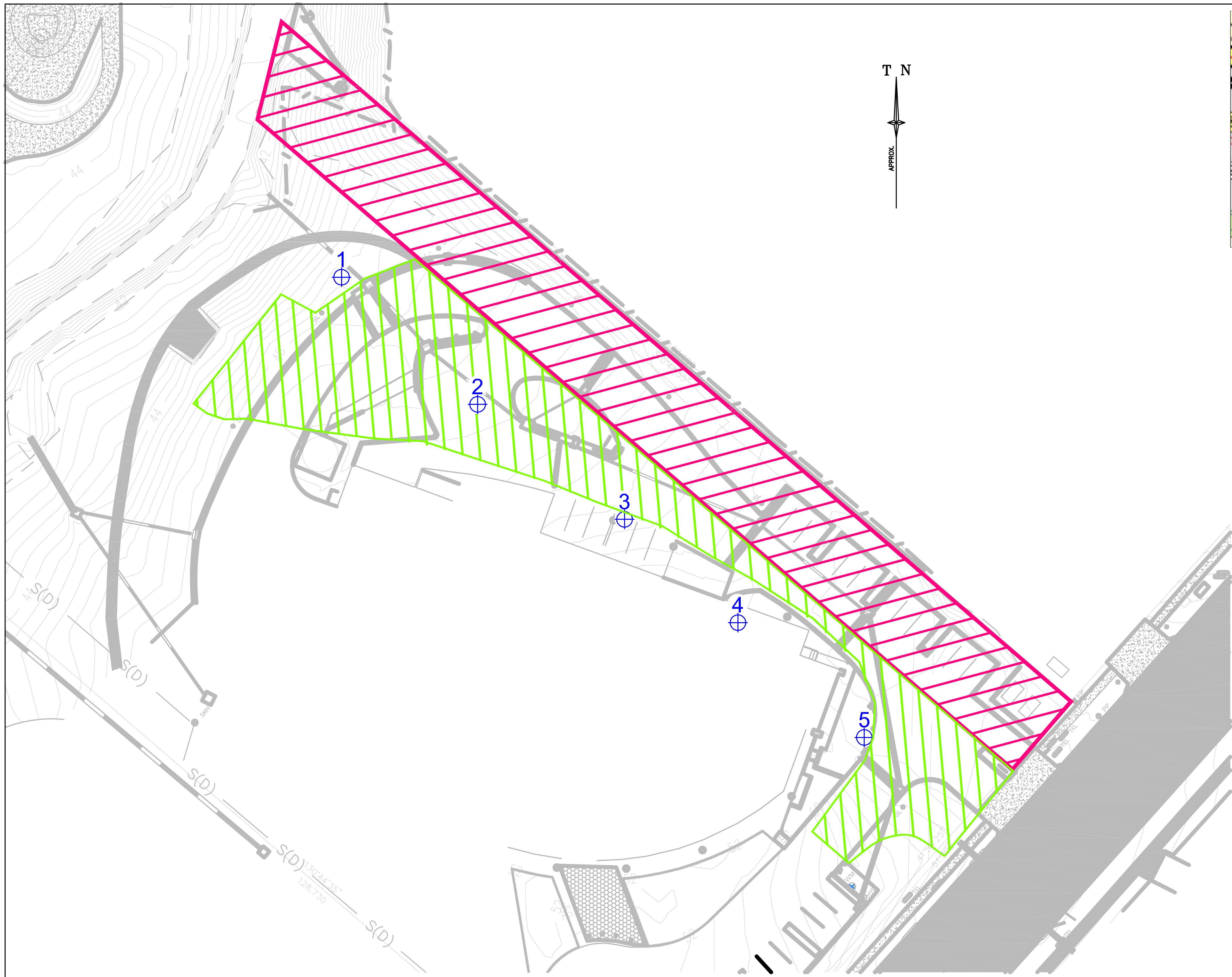
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later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

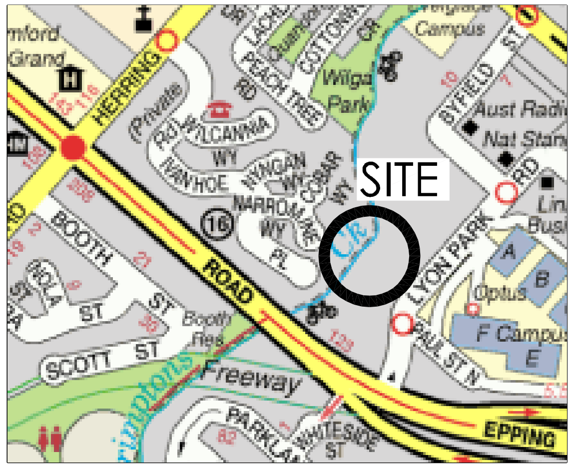
Information for Contractual Purposes

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.





Locality Plan

LEGEND

⊕ Previous Test Boreholes (1-5)

🟥 Proposed Road

🟩 Associated Development

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CLIENT: Frasers Property Australia Pty Ltd		
OFFICE: Sydney	DRAWN BY: Vojta	
SCALE: 1:500@A3	DATE: 24.7.2017	

TITLE: **Previous Test Locations**
Proposed Road - Ivanhoe Estate
2-4 Lyon Park Road, Macquarie Park

PROJECT No:	86043.00
DRAWING No:	101
REVISION:	0

TEST BORE REPORT

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTI STOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

DATE: 1 AUGUST 00
PROJECT No.: 29190
SURFACE LEVEL: 45.12

BORE No. 1
SHEET 1 **OF** 1

Depth m	Description of Strata		Sampling & In Situ Testing			
			Type	Depth (m)	Results	Headspace PID (ppm)
0	FILLING - poorly compacted, light brown to brown clay filling with a trace of silt and gravel	▨	A S	0.5 0.95	1,1,2 N=3	2
1.4	CLAY - firm, brown mottled red brown clay with a trace of ironstone gravel	▨				
1.8	IRONSTONE	○				
2.0	TEST BORE DISCONTINUED AT 2.0 METRES - auger refusal					

RIG: B40
DRILLER: DRIVER
LOGGED: CARLE
CASING:
TYPE OF BORING: 100mm DIAMETER SPIRAL FLIGHT AUGER
GROUND WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED
REMARKS: TBM GRATE IN LYON PARK ROAD RL 48.22

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp Pocket Penetration (kPa)	V shear vane (kPa)

CHECKED:
Initials: LD
Date: 10/8

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TEST BORE REPORT

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTISTOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

PROJECT No: 29190
SURFACE LEVEL: 45.91
DIP OF HOLE: 90°

BORE No: 2
DATE: 2/8/00
SHEET 1 **OF** 1
AZIMUTH:

Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0	FILLING - poorly compacted, dark brown silty sandy clay						S			1,2,4 N=6
0.75	FILLING - poorly compacted, dark grey and yellow brown sandy clay and gravel filling									
1.1	FILLING - crushed sandstone and gravel filling									
1.7	SANDY CLAY - firm to stiff, light grey and yellow brown sandy clay						S			3,4,4 N=8
3.0	SANDSTONE - extremely low to very low strength, light grey brown sandstone									
3.5	TEST BORE DISCONTINUED AT 3.5 METRES						A			

RIG: B40
DRILLER: DRIVER
LOGGED: PARMAR
CASING: UNCASED
TYPE OF BORING: SPIRAL FLIGHT AUGER TO 3.5m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp pocket penetrometer (kPa)	V Shear Vane (kPa)

CHECKED:
Initials:
Date: 10/8

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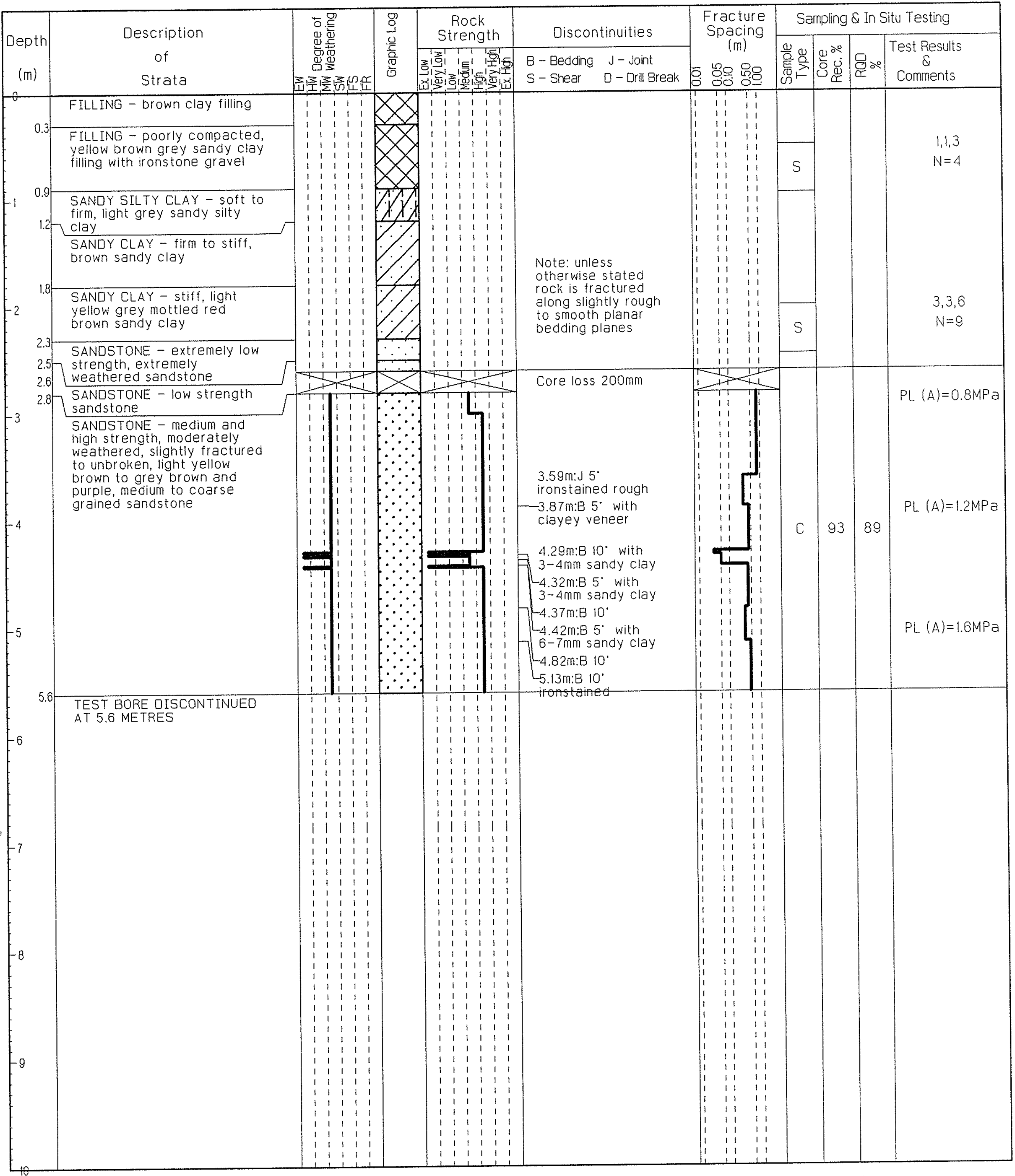
TEST BORE REPORT

BORE No: 3
DATE: 2/8/00
SHEET 1 **OF** 1
AZIMUTH:

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTISTOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

PROJECT No: 29190
SURFACE LEVEL: 46.76
DIP OF HOLE: 90°



Depth (m)	Description of Strata	Discontinuities	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0	FILLING - brown clay filling					
0.3	FILLING - poorly compacted, yellow brown grey sandy clay filling with ironstone gravel		S			1,1,3 N=4
0.9	SANDY SILTY CLAY - soft to firm, light grey sandy silty clay					
1.2	SANDY CLAY - firm to stiff, brown sandy clay	Note: unless otherwise stated rock is fractured along slightly rough to smooth planar bedding planes				
1.8	SANDY CLAY - stiff, light yellow grey mottled red brown sandy clay		S			3,3,6 N=9
2.3	SANDSTONE - extremely low strength, extremely weathered sandstone					
2.5						
2.6		Core loss 200mm				
2.8	SANDSTONE - low strength sandstone		C	93	89	PL (A)=0.8MPa
	SANDSTONE - medium and high strength, moderately weathered, slightly fractured to unbroken, light yellow brown to grey brown and purple, medium to coarse grained sandstone	3.59m:J 5° ironstained rough 3.87m:B 5° with clayey veneer 4.29m:B 10° with 3-4mm sandy clay 4.32m:B 5° with 3-4mm sandy clay 4.37m:B 10° 4.42m:B 5° with 6-7mm sandy clay 4.82m:B 10° 5.13m:B 10° ironstained				PL (A)=1.2MPa PL (A)=1.6MPa
5.6	TEST BORE DISCONTINUED AT 5.6 METRES					

RIG: B40
DRILLER: DRIVER
LOGGED: PARMAR
CASING: GL TO 2.6m
TYPE OF BORING: SPIRAL FLIGHT AUGER TO 2.6m,NMLC CORING TO 5.6m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp pocket penetrometer (kPa)	V Shear Vane (kPa)

CHECKED:
Initials: WD
Date: 10/8

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TEST BORE REPORT

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTI STOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

DATE: 1 AUGUST 00
PROJECT No.: 29190
SURFACE LEVEL: 47.3

BORE No. 4
SHEET 1 **OF** 1

Depth m	Description of Strata	Sampling & In Situ Testing: Type	Depth (m)	Results	Headspace PID (ppm)
0	FILLING - poorly compacted, brown, slightly sandy clay filling	A*	0.5		2
		S	0.5 – 0.95	1,2,4 N=6	
	- 0.95m - traces of wood				
1.3	CLAY - red brown clay with a trace of silt and sand				
1.7	SILTY SANDY CLAY - grey silty sandy clay	A	1.8		2
2.0	CLAY - firm, red brown clay	S	2.0 – 2.45	2,3,5 N=8	
2.8	SANDSTONE - extremely low strength, light grey sandstone with some clay				
3.5	TEST BORE DISCONTINUED AT 3.5 METRES - auger refusal				

RIG: B40
DRILLER: DRIVER
LOGGED: CARLE
CASING:
TYPE OF BORING: 100mm DIAMETER SPIRAL FLIGHT AUGER
GROUND WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED
REMARKS: *DENOTES DUPLICATE SAMPLE Z1 TAKEN

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp Pocket Penetration (kPa)	V shear vane (kPa)

CHECKED:
Initials:
Date: 10/8

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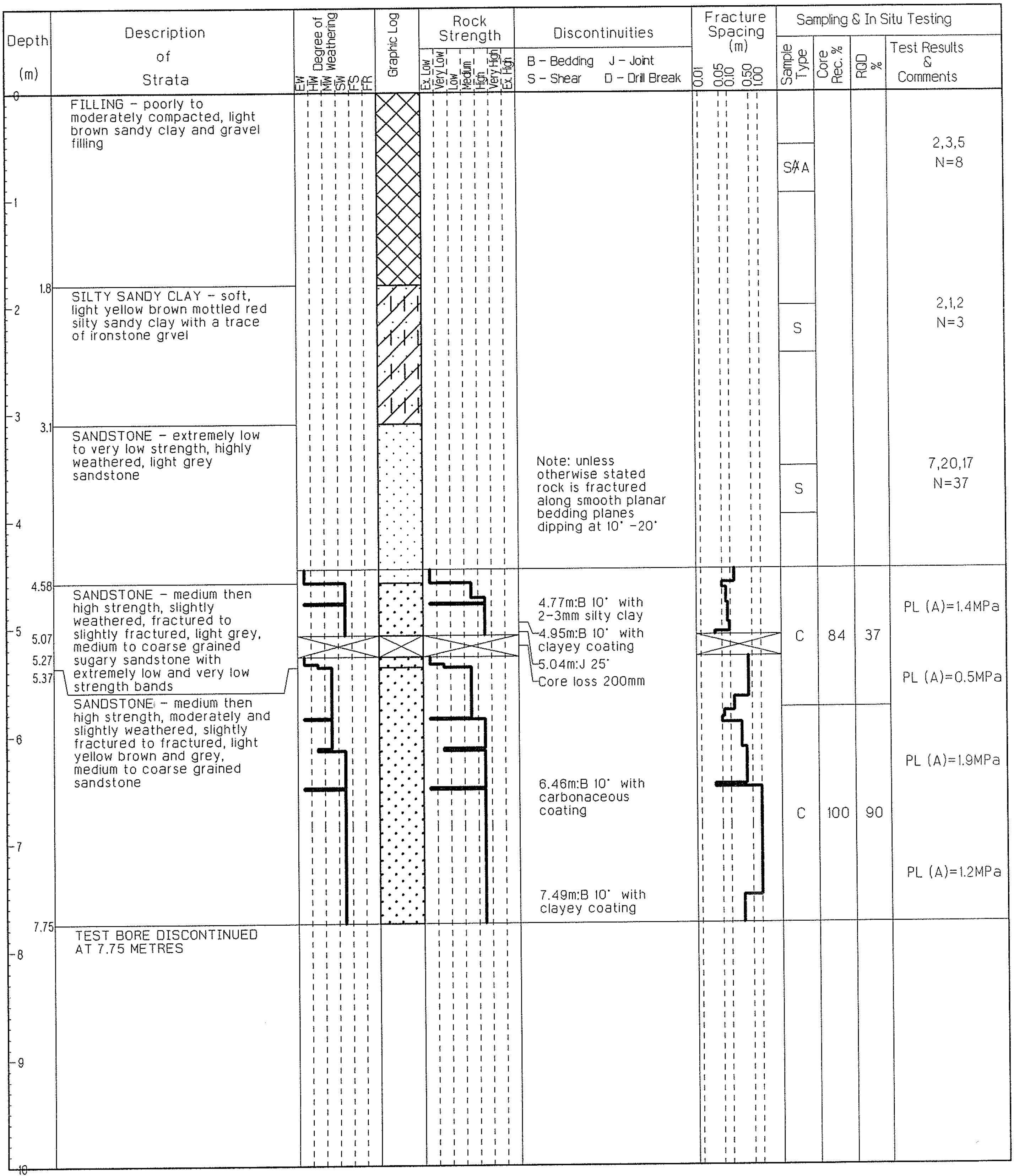
TEST BORE REPORT

BORE No: 5
DATE: 3/8/00
SHEET 1 **OF** 1
AZIMUTH:

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTISTOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

PROJECT No: 29190
SURFACE LEVEL: 48.05
DIP OF HOLE: 90°



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0	FILLING - poorly to moderately compacted, light brown sandy clay and gravel filling		S/A			2,3,5 N=8
1.8	SILTY SANDY CLAY - soft, light yellow brown mottled red silty sandy clay with a trace of ironstone grvel		S			2,1,2 N=3
3.1	SANDSTONE - extremely low to very low strength, highly weathered, light grey sandstone	Note: unless otherwise stated rock is fractured along smooth planar bedding planes dipping at 10° -20°	S			7,20,17 N=37
4.58 5.07 5.27 5.37	SANDSTONE - medium then high strength, slightly weathered, fractured to slightly fractured, light grey, medium to coarse grained sugary sandstone with extremely low and very low strength bands	4.77m:B 10° with 2-3mm silty clay 4.95m:B 10° with clayey coating 5.04m:J 25° Core loss 200mm	C	84	37	PL (A)=1.4MPa PL (A)=0.5MPa
	SANDSTONE - medium then high strength, moderately and slightly weathered, slightly fractured to fractured, light yellow brown and grey, medium to coarse grained sandstone	6.46m:B 10° with carbonaceous coating 7.49m:B 10° with clayey coating	C	100	90	PL (A)=1.9MPa PL (A)=1.2MPa
7.75	TEST BORE DISCONTINUED AT 7.75 METRES					

RIG: B40 **DRILLER:** DRIVER **LOGGED:** PARMAR **CASING:** GL TO 4.45m
TYPE OF BORING: SPIRAL FLIGHT AUGER TO 4.45m,NMLC CORING TO 7.75m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_S (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp pocket penetrometer (kPa)	V Shear Vane (kPa)

CHECKED:
Initials:
Date: 10/8

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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

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.

Soil Descriptions

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726-1993, 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:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

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

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

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:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

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:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

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 ($Is_{(50)}$) 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 - 2007. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approximate Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	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.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

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.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and longer sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

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

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

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:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Symbols & Abbreviations

Introduction

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

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	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

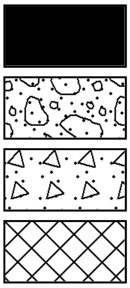
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

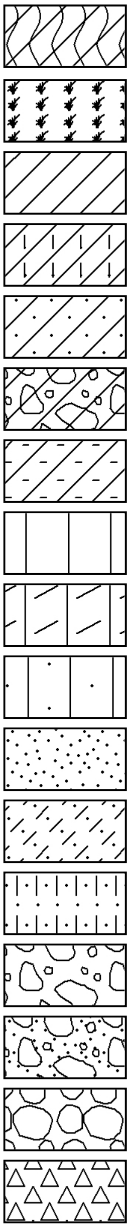
Graphic Symbols for Soil and Rock

General



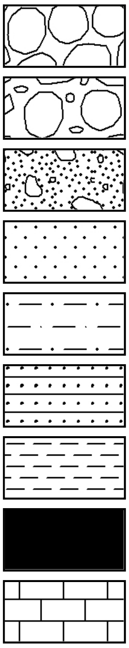
- Asphalt
- Road base
- Concrete
- Filling

Soils



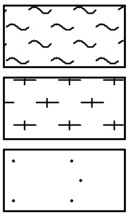
- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks



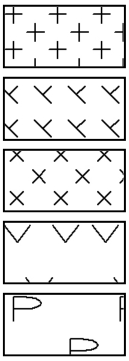
- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks



- Slate, phyllite, schist
- Gneiss
- Quartzite

Igneous Rocks



- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry