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Report on Geotechnical Investigation

Proposed Conference Centre and Office Tower

12 - 16 Campbell Street, Bowen Hills QLD

Prepared for New Urban Villages Pty Ltd

Project 227045.00

4 June 2024

Douglas PARTNERS | GROUNDED EXPERTISE

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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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Reviewer	D Qualischefski	4 June 2024

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Report on Geotechnical Investigation
Proposed Conference Centre and Office Tower
12 - 16 Campbell Street, Bowen Hills QLD

1. Introduction

This report presents the results of a geotechnical investigation undertaken by Douglas Partners Pty Ltd (Douglas) for proposed conference centre and office tower at 12 – 16 Campbell Street, Bowen Hills. The investigation was commissioned in an email dated 6 March 2024 by Tim Johnson of New Urban Villages Pty Ltd, and was undertaken in accordance with Douglas Partners Pty Ltd (DP) proposal 227045 dated 19 December 2023.

It is understood that the proposed development would comprise the construction of an eight-level office building with lower bound conference centre and associated partly inground single level basement carpark.

The aim of the investigation was to assess the subsurface soil and groundwater conditions across the site in order to update the following comments (where required):

- encountered subsurface conditions;
- groundwater levels and control measures;
- site classification in accordance with AS2870 (2011);
- earthworks requirements, excavatability for bulk excavations and detailed footings and services, site preparation, trafficability, compaction of controlled filling, re-use of excavated materials;
- temporary batter slopes;
- suitable retention options and geotechnical retaining wall design parameters (comprising unit weight, active, passive and at rest earth pressure coefficients, ultimate passive pressures in rock);
- allowable bearing pressures for high level or raft footings and associated settlements;
- allowable shaft and end bearing pressures, and suggested pile types;
- assessment of site sub-soil class to AS1170.4-2007 Part 4 within the depths drilled; and
- indicative subgrade California bearing ratio (CBR) value for pavement / slab on ground design purposes (by others);

The investigation included the drilling of two boreholes, followed by laboratory testing, engineering analysis and reporting. The details of the field work and laboratory testing are presented in this report, together with comments and recommendations on the items listed above.

This report must be read in conjunction with the notes entitled 'About This Report' in Appendix A and other explanatory notes, and should be kept in its entirety without separation of individual pages or sections.

2. Site Description

The overall site is rectangular in shape, and encompasses the following lots below:

- Lot 4 on plan RP10074; and
- Lot 5 on plan RP10074.

The site is bounded by: Campbell Street to the north; commercial buildings to the east and west; Local access road to the south (refer Drawing 1 in Appendix B).

At the time of the investigations, the allotments mentioned above were vacant due to being recently demolished which was historically occupied by a former one to two storey building and at grade carpark circa 2023.

Rock outcrops were observed locally at surface level at southern portion of the site where a temporary sump was excavated.

Review of the Brisbane City Council (BCC) interactive mapping contour overlay indicates the proposed administration development site slopes down to the south-west with levels ranging from approximately RL18 m AHD to RL19 m AHD which then slopes to the south-west towards the access road between RL 15 mAHD to RL 18 mAHD.

Photographs of the proposed development site taken during the investigations are indicated in Figures 1 and 2.

Figure 1: Looking south at the rig set up on Bore 1, located within the proposed development site.

Figure 2: Looking south-east towards the rig set up on Bore 2, located in the proposed development site.

3. Regional Geology

The Geological Survey of Queensland digital geological map indicates the site is underlain by late Triassic aged Brisbane Tuff from the Brisbane Tuff formation, that typically comprises "*Rhyolitic tuff, ignimbrite, agglomerate, conglomerate, sandstone, shale*".

The natural subsurface conditions encountered during the investigations comprised fill underlain by residual clay then weathered phyllite and / or conglomerate at depth. The residual soils and weathered phyllite and / or conglomerate are generally consistent with the geological mapping, albeit with a minor change to the rock type.

4. Field Work

4.1 Field Work Methods

The field work was originally undertaken on 19 and 20 March 2023, and comprised the drilling of two boreholes (designated Bores 1 and 2, to between 10.10 m and 12.90 m depth, at or near client nominated locations (refer Drawing 1 in Appendix B).

The boreholes were drilled using a track mounted drilling rig using solid flight auger techniques up to 2.5 m depth, after which rotary washbore techniques and / or NMLC rock coring were used to advance the boreholes to termination.

Standard penetration tests (SPTs), 'undisturbed' (U_{50}) tube samples, and disturbed samples were undertaken at regular depth intervals within the boreholes for visual identification and laboratory testing. Dynamic cone penetrometer (DCP) testing was carried out adjacent to the boreholes to maximum depths of 0.6 m and 0.9 m (or prior refusal) with reference to test method AS 1289.6.3.2 (1997). On completion of drilling, and after checking for groundwater ingress, a groundwater well was installed in Bore 1 to 5 m depth, whilst the remaining borehole were backfilled with drilling spoil.

The test locations were set out by a geotechnical engineer and the UTM coordinates and ground surface levels at the test locations were recorded using a differential GPS accurate to approximately 0.1 m and are presented on the borehole logs in Appendix C. The field work was then completed by an experienced geotechnical engineer who prepared field logs of the subsurface conditions, collected samples for visual and tactile assessment, and laboratory testing.

4.2 Field Work Results

The subsurface conditions encountered in the boreholes are described in detail on the borehole logs in Appendix C. Notes defining the classification methods and descriptive terms used in their preparation are given in Appendix A.

In summary, the subsurface conditions encountered generally comprised **fill** overlying **residual soils** then **weathered phyllite / conglomerate** to the limit of the investigation. The subsurface conditions at each development area are further described below:

- **Fill:** generally medium dense to dense, fine to coarse, gravelly sand fill was encountered from surface in all boreholes and continued to depths of between 0.2 m and 0.3 m.

  The fill appeared to be recently placed as a working platform on site. However, in the absence of documentation to confirm the fill was placed and compacted in a controlled manner under engineering supervision and testing, it should be considered as 'uncontrolled'.
- **Residual Soil:** encountered beneath the fill in all boreholes and continued to 0.4 m and 2.7 m depth. The residual soils were generally stiff to hard, medium plasticity, residual sandy gravelly clay with some localised relict rock structure.

  Residual sandy gravelly clay was also encountered interbedded within the weathered phyllite as crushed seams and continued to bore termination depth. This material may be completely weathered rock.
- **Phyllite / Conglomerate:** encountered beneath the residual soil and continued to borehole termination depths of between 10.10 m and 12.90 m. The phyllite / conglomerate was initially very low to low strength and highly weathered, and graded to medium strength and moderately weathered phyllite / conglomerate generally below depths of 4 m to 8 m. The phyllite / conglomerate has relatively closely spaced fractures throughout the rock.

Free groundwater seepage was not encountered during the auguring to 2.5 m depth, after which water was added to facilitate washboring and NMLC coring techniques. However, was measured at 2.4 m depth in the groundwater well installed in Borehole 1. It should be noted, however, that groundwater depths are affected by climatic conditions, soil permeability, surface and subsurface drainage, human influences, and will therefore vary with time.

5. Laboratory Testing

Collection of "undisturbed" samples intended for shrink-swell test was not possible due to the type and strength of the materials encountered. Consequently, an Atterberg limits and linear shrinkage test was adopted for use in correlating with soil reactivity.

The laboratory test results are summarised in Table 1, and detailed test report sheets are given in Appendix D.

Table 1: Results of Moisture Content, Linear Shrinkage & Plasticity Tests

Bore	Depth (m)	Material	M (%)	W_L (%)	W_P (%)	PI (%)	LS (%)
1	1.00 – 1.45	Sandy Gravelly Clay	16.2	40	26	14	6.5

Legend: M – moisture content; W_L – liquid limit; W_P – plastic limit; PI – plasticity index; LS – linear shrinkage;

Results of the above reported Atterberg limit test indicates the sample tested to be of low plasticity.

6. Proposed Development

It is understood that the proposed development would comprise the construction of an eight-level office building with lower bound conference centre and associated partial inground single level basement carpark.

Details of the building construction were not known at the time of reporting, however, it is envisaged that the structures will be of a reinforced concrete panels with slab on ground and precast or blockworks walls.

Structural working loads were not provided prior to the preparation of this report, however, it is envisaged they will be similar to multi-storey building type loads (i.e. in the order of 6000kN working). It is also anticipated that up to 3.5 m depth of bulk excavation would be required across the sloping site to achieve the single level in-ground basement carpark.

7. Appreciation of Ground Conditions

The subsurface conditions encountered during the field work generally comprised 'uncontrolled' fill up to 0.3 m depth, and then stiff to hard residual clays to between 0.4 m and 2.7 m depth. The residual soils were underlain by very low to low strength, highly weathered phyllite to 4 m and 8 m, grading to medium to high strength, moderately weathered to the borehole termination depths of between 10.10 m and 12.90 m. In Bore 1 below 8m depth the interbedded conglomerate layers were present within the phyllite rock. Groundwater seepage was not encountered during the field works, however, was measured in the groundwater well in Bore 1 at 2.4 m depth on 20 of March 2024.

The main building structure which is anticipated to have 'heavy' column loads will need to be supported on piled foundations penetrating through the fill and residual soils to found in the medium strength (or stronger) phyllite / conglomerate rock.

The proposed single level partial inground basement carpark could be constructed using a combination of battered temporary excavations up to 3.5 m depth (i.e. along the northern boundary) and positive support such as soldier pile walls with shotcrete panels (i.e. along the eastern and western boundaries). Due to the need to excavate relatively close to the boundaries of the site, there will be implications for the design and construction of the basement carpark, as follows:

- stability of adjoining buildings, footpaths, roadways and in-ground services during construction;
- stability of excavated faces during construction;
- potential groundwater seepage (if any) at carpark level; and
- Potentially excavatability of rock (if higher strength rock is encountered in site areas away from the bores).

It would be prudent to commission a dilapidation survey of nearby structures and in-ground services prior to construction.

Further comments on design and construction practice are given in the following sections of this report.

8. Comments

8.1 Groundwater Control

Groundwater was not encountered during auger drilling of the boreholes. However, water was measured at 2.4 m depth in the groundwater well installed in Borehole 1. Given the ground conditions encountered which comprised mainly residual clays and weathered phyllite / conglomerate, and the proposed basement carpark excavation of up to 3.5 m depth, groundwater seepage into the basement excavation is anticipated to be slow and able to be handled via pumping from shallow sumps during construction.

Given the anticipated slow seepage inflows a 'drained' basement will appear suitable for this site which would necessitate full height drainage to be installed behind all basement walls and a subfloor drainage layer installed beneath the on-ground basement floor slabs. All drainage would then need to be connected to sumps with pumps to remove water to the stormwater system following any required treatment. The design of extraction pumps would require a detailed groundwater investigation to determine inflow rates. DP can assist with such an investigation. The alternate to a drained basement is a 'tanked' which requires design for full hydrostatic uplift.

8.2 Basement Construction

8.2.1 General

Excavations of up to 3.5 m depth will generally be required to achieve the in-ground basement excavation along the northern, western and eastern boundaries, while the southern boundary is at a level close to the local laneway. Temporary battering of the excavation face is possible along the northern boundary only as there appears to be suitable space available. However, where the line of the excavation extends close to the site boundaries (ie. western and eastern boundaries) which are surcharged by existing structures, a stiff retention system, such as piled wall will be required, to minimise lateral and vertical ground movements behind the walls.

8.2.2 Excavatability

Based on the conditions encountered in the boreholes, it is anticipated that bulk excavation of the existing fill and residual soils and very low strength phyllite could be undertaken with conventional sized hydraulic excavators (ie 30-35 tonne). A single ripping tyne will probably be required to facilitate excavation in the very low and low strength rock. If high strength phyllite is encountered (as per Bore 2 at 4m depth) within the excavation then rock hammers will be required.

It should be recognised that the above excavatability estimates are based on materials encountered at the test locations only, and that conditions may prove more difficult (or easier) for excavation between and beyond the test locations.

8.2.3 Temporary Battered Excavations

Temporary battering of excavation faces may be suitable for the northern boundary and any internal batters within the bulk excavation up to 3.5m vertical height such as lift pit overruns etc.

Unsurcharged batter slopes cut up to 3.5 m vertical height may be preliminarily designed for temporary batters given in Table 2.

Table 2: Cut Batter Slopes (up to 3.5 m high)

Material	Safe Batter Slope (H:V)
	Short Term
Existing ‘uncontrolled’ fill Controlled fill⁽¹⁾ and / or stiff natural clays Very stiff (or stronger) natural clays	1:1
Very low strength (or stronger) phyllite / conglomerate	0.75:1⁽²⁾

Notes:

(1) Assuming controlled fill is undertaken in accordance with the recommendations of this report.

(2) Subject to geotechnical inspection during construction to confirm the absence of adverse joints.

The stability of shallow excavations should generally remain temporarily stable for near vertical shallow excavations (i.e. up to 1 m depth) provided dry conditions prevail and there are no loads, services, structures or traffic loads within a distance from the crest equal to the excavation height.

The above temporary batter slopes are suggested with respect to slope stability only, and do not allow for lateral stress relaxation which may result in movement of nearby in-ground services or shallow footings. If such services are settlement-sensitive, and are located such that a linear spread at 1H:1V outwards, down and away from the base of the service, intersects the cut face, then the excavation may have to be positively supported.

8.2.4 Positive Support

8.2.4.1 General

Where there is limited space to batter, then positive ground support will be required which will be the case along the western and eastern boundaries. The ground retention system selected will need to minimise ground movements behind the excavation faces to ensure adjacent structures, pavements, and in-ground services are not affected as a result of basement construction.

Cantilevered soldier piles with shotcrete infill panels are commonly used to support the faces in basement excavations. The advantage of a piled wall is that it could be incorporated into the final carpark structure.

Where significant loads surcharge the excavation faces, the piled wall can be made stiffer by decreasing the pile spacing to form a contiguous pile wall or incorporating anchors or props for support.

Driven sheet piles would not be practical on this site due to the presence of weathered phyllite rock at shallow depth and within the bulk excavation level.

8.2.4.2 **Pile Walls**

Based on previous experience with similar subsurface conditions, it is envisaged that a soldier pile wall with shotcrete infill panels would be suitable. Soldier piles are typically spaced at approximately three pile diameters along the basement excavation sidewalls with mesh and shotcrete infill panels between the piles.

If anchors are required to limit pile movements, they could be designed using the following working bond stress of 100 kPa in very low strength, 150kPa in low strength and 300kPa medium strength (or stronger) phyllite / conglomerate.

Anchor bond stresses are largely reliant upon drilling and cleaning techniques, and hence the amount of smear around the sides of the hole. It would be appropriate for checks of bond stress to be made by the contractor installing anchors at the time of construction, by way of pull out testing and proof load testing.

After installation, all temporary anchors should be check stressed to 130% of the nominal working load then locked off at 90% of the working load. Checks should also be made at regular intervals to ensure that load is maintained in anchors and not lost due to creep effects.

The conditions indicated by the investigation suggest that the preparation of anchors at the site should also include:

- a free length equal to their height above the base of the excavation;
- a minimum bond length of 3 m; and
- a maximum bond length of 10 m.

Internal bracing systems are an alternative to anchored support, however, braces can restrict access which must be maintained during building construction. Approval from neighbours and *Council* will be required prior to installation of temporary anchors where they extend beyond property boundaries.

Determination of pile depths, anchor spacing and lengths is a matter for detailed design. DP could assist in the analysis for such a design if required.

8.2.4.3 **Geotechnical Retaining Wall Design Parameters**

The design of flexible or rigid retaining walls with a single row of props or cantilevered could be undertaken using a triangular pressure distribution and the parameters given in Table 3. Flexible walls are those which are free to rotate or tilt (such as cantilevered walls) and should be designed using an active earth pressure coefficient (Ka). Where walls are relatively rigid so they cannot rotate or translate to achieve ‘active’ lateral pressure conditions in the retained soil, the ‘at rest’ earth pressure coefficient (Ko) should be used.

Table 3: Earth Pressure Coefficients (non-sloping crest backfill)

Material	Unit Weight (kN/m^3)	At rest Ko	Active Ka	Passive Kp (pressure)
Existing 'Uncontrolled' fill	18	0.72	0.57	1.75
Controlled fill(1) and / or stiff natural clays	20	0.66	0.49	2.04
Very stiff (or stronger) natural clays	20	0.58	0.41	2.44
Very low strength (or stronger) phyllite / conglomerate	22	0.38	0.24	(400 kPa)

Notes: (1) Assuming controlled fill is undertaken in accordance with the recommendations of this report.

It is recommended that all permanent basement walls be drained for full height in order to minimise hydrostatic pressure build-up behind the walls. Tanked basements would need to be designed for full height hydrostatic pressure.

For design of retaining walls:

- Due allowance should be made for surcharge loadings (over and above the lateral earth pressure coefficients presented above) where the finished ground level above the retaining walls is above horizontal and where additional loading is likely to be applied from existing or future upslope structures, or from traffic. The effects of surcharge can be estimated by multiplying the vertical pressure by the appropriate lateral earth pressure coefficient presented above.
- Drainage material should be installed for the full height of the wall, for a width of at least 0.3 m. The material must be free draining and granular, and have a perforated or slotted drainage pipe at the heel of the wall to rapidly remove the water into the stormwater system.
- Where not fully drained, the walls will need to be designed for full hydrostatic pressure.

It is recommended that factors of safety of 2 against overturning and sliding stability and 1.5 for global stability, be adopted in the design of all retaining walls.

For limit state design methods, the ultimate parameters provided above in Table 3 will need to be factored in accordance with AS 4678 (2002). Guidance of the selection material strength partial factors is provided in Section 5.2 of AS 4678 (2002) and is dependent upon the nature and state of the natural in situ soil.

8.3 Carpark Basement Preparation and Localised Fill Placement

Following bulk excavation to design level for the basement, the exposed subgrade is anticipated to comprise very low to low strength (or stronger) phyllite. Where the exposed subgrade is subjected to increases in moisture content from rainfall and/or overland flow, there is potential for the weathered rock to soften.

A working platform is recommended to prevent softening of the subgrade and would be required or the support of a piling rig if pile foundations are adopted. Temporary piling platform design can only be definitively carried out once the size and loading of the piling rig(s) are known. At this stage, a nominal construction trafficking platform in the order of (say) 0.3 m should be allowed for in costings for light construction vehicles.

It is important that suitable grades be maintained to allow drainage and to minimise the potential for ponding of surface water, which can be collected in screened sumps, tested and treated if required, and pumped from the excavation.

Trafficability across the weathered phyllite subgrade at bulk excavation level, if water softened, will be relatively poor. Placement of the abovementioned construction trafficking platform would also assist trafficability of rubber tyred vehicles.

Any new fill required to achieve design levels beneath on-ground basement slabs should be undertaken under 'Level 2' sampling and testing as detailed in AS 3798 (2007). Any new fill required beneath floor slabs should also be compacted to a minimum dry density ratio of 98% relative to standard dry density at ±2% OMC.

The above procedures will require geotechnical inspection and testing services to be employed during construction. DP is suitably qualified to conduct earthworks testing and supervision services, as well as engineering inspections of batters, footings and piled foundations, as may be required during the development.

8.4 Foundations

8.4.1 General

For the anticipated main structure, the use of high-level footings would only be possible if the loads are minimal and provided low strength (or stronger) phyllite is present at close to bulk excavation level which may occur in the northern site areas (as encountered in Bore 2). Otherwise as mentioned in the above previous sections, the use of pile foundations will be required to support the main building structural loads.

Where limit state methods are used to design the foundations, the ultimate geotechnical strength ($R_{d,ug}$) can be calculated by multiplying the allowable parameters (given in Sections 8.5.2 and 8.5.3) by the adopted safety factor of 2.5, and then multiplied by a suitable geotechnical strength reduction factor (Φ_g) to obtain the design geotechnical strength ($R_{d,g}$). A nominal Φ_g value of 0.5 is recommended for high level footings.

The Piling Code AS 2159 (2009) requires a Φ_g value of 0.45 to 0.65 where there is no testing of pile capacity, rising to 0.65 to 0.85 where a significant number of piles are tested after installation.

It is essential that foundation excavations be inspected by experienced geotechnical personnel to ensure the design parameters adopted are suitable for the ground conditions being encountered and to ensure that there is no soft or loose material remaining at the base of the excavations or smear on the pile side walls. Ground conditions can vary, and it is essential that adequate provision be made throughout the project to vary foundations to suit differing ground conditions.

8.4.2 High Level Foundations

Provided site preparation is carried out in accordance with the recommendations in this report, it is considered that high level pad /strip and spread footings up to 2 m in width could be designed using the maximum allowable bearing pressures given in Table 4.

Table 4: Maximum Allowable Bearing Pressure

Material	Maximum Allowable Bearing Pressure(1) (kPa)
Very low strength phyllite	Not Suitable
Low strength (or stronger) phyllite	1000(2)

Notes: (1) Subject to confirmation through in-situ testing during construction.
(2) Provided no weaker foundation material exists within two footing widths below the base of the footing.

It is recommended that footings be founded in materials of similar strength and compressibility to reduce the potential for differential settlement. Load related settlements for footings loaded as per above are not anticipated to exceed 1 % to 2% of footing width. Where footings wider than 2 m are adopted, specific assessment based on actual applied pressure and ground conditions at that location is recommended to assess specific settlement characteristics.

The above allowable values are based on a factor of safety of 2.5 against bearing capacity failure. Should the above maximum allowable bearing pressures prove too low for the development loads, then the building will need to be supported on bored piles.

8.4.3 Piled Foundations

8.4.3.1 General

Given the ground conditions encountered in the boreholes suitable pile types would comprise conventional bored piles founding in the low strength (or stronger) phyllite / conglomerate rock.

It should be noted that the ability to drill bored piles in rock is not only dependent on the characteristics of the rock (strength, fracture spacing etc) but also the type (power and size) of the drilling rig and the size (diameter) of piles. Bored pile installation in medium strength or stronger rock will require the use of heavy drilling plant such as Casagrande, Soilmec or Bauer rigs.

8.4.3.2 Bored Piles

For the design of bored piles founded a minimum of one pile diameter into weathered rock could be sized using the allowable values given in Table 5. Pile capacities and suitable pile types should be confirmed by prospective piling contractors.

Table 5: Allowable Bored Pile Design Pressures

Material	Allowable Shaft Adhesion[1] (kPa)	Allowable End Bearing[1] (kPa)
Very low strength phyllite / conglomerate	100	Not Suitable
Low strength phyllite / conglomerate	150	1500
Medium strength (or stronger) phyllite / conglomerate	300	3000[2]

Notes: (1) Subject to confirmation through visual and tactile assessment of the material during inspection.
(2) Provided no weaker foundation material exists within four pile diameters and below the base of the pile footing.

For bored pile foundations loaded as per the allowable bearing pressures in Table 5, it is considered that settlements under such applied loading will be less than 1% of the pile diameter.

It is recommended that the upper 0.9 m depth of natural soil or depth of ‘uncontrolled’ fill (whichever is greater) be ignored in pile shaft adhesion calculations due to the effects of seasonal moisture variation and shaft load development effects.

Bored piles should be socketed into similar strength strata to reduce the potential for differential settlement between adjacent piles.

8.5 Site Earthquake Sub-Soil Class

In accordance with AS1170.4-2007, it is recommended that a site sub-soil classification of “Class B_e – Rock” be adopted, in accordance with the definitions presented in *Section 4.2 – Class Definitions*. This is based on a sub-soil profile of no more than 3 m of soil underlain by rock with a compressive strength of between 1 MPa and 50 MPa over the top 30 m.

8.6 Indicative Slab-on-Ground Parameters

Where site preparation is carried out as detailed in Section 8.3, the subgrade conditions are expected to comprise weathered rock.

Based on subgrade type and strength, for design of on-ground basement floor slabs subjected to vehicular trafficking, it is recommended that the following parameters indicated in Table 6 below can be adopted in the design.

Table 6: Presumptive Pavement and Slab on Ground Parameters

Material	Presumptive Soaked CBR Value (%)	Modulus of subgrade reaction (k) (kPa/mm)	Soil Modulus for Short Term Load (MPa)(1)	Soil Modulus for Long Term Load (MPa)(1)
Very low strength (or stronger) phyllite / conglomerate	5	34	44	35

Notes: (1) for well drained subgrade conditions

These values are based on the assumption that the earthworks will be undertaken in accordance with the recommendations in this report. Confirmatory CBR tests will be carried out to verify or not the above presumptive CBR value.

For loaded areas of different proportion or different load intensity to standard highway type wheel loads, DP should be contacted for further advice.

8.7 Site Classification

Site classification of foundation soil reactivity strictly only applies to residential buildings up to two-storeys and to other buildings of similar size, loading and flexibility as defined in accordance with AS 2870 (2011), and would not apply to this development. Such classification, as well as the results of the laboratory testing, provide an indication of the propensity of the ground surface to move with seasonal variation in moisture content. The following is provided for information purposes.

Due to the presence of fill of unknown compaction history (which must be considered as ‘uncontrolled’ fill) up to 0.3 m depth, the site would strictly be given a “Class P” classification, in accordance with AS 2870 (2011), requiring design by engineering principles. However, if the supporting documentation can be provided for it’s placement under controlled conditions then the site would be classified as per natural conditions.

To provide an indication of the reactive surface movements of the sample tested, the results of the Atterberg limits and linear shrinkage test were compared with an in-house database of plasticity and shrink-swell index (I_{ss}) values, to estimate a presumptive I_{ss} value of 1.5 % per ∆pF for the sandy gravelly clay sample tested. Therefore, we have adopted the presumptive shrink-swell index of 1.5% for our calculations. Which is similar to previous investigations at nearby sites undertaken by DP.

The analysis indicates that the y_s values of a full depth soil profile tested in response to seasonal moisture variation, are in the order of up to 20 mm consistent with a “Class S” (slightly reactive) classification, if it were not for the existing “Class P”. This is similar to previous investigations undertaken by DP.

Where existing site soils of similar reactivity won from excavation are reused as controlled fill, y_s values of up to 30 mm consistent with a “Class M” (moderately reactive) classification would also result. This is due to the need to consider uncracked conditions for a five-year period following fill placement and two years following excavation.

It should be noted that the proposed basement carpark level is up to 3.5 m depth, which will be below the depth of seasonal moisture change of 1.8 m depth, and site classification will be of particular importance to inground services founded close to existing ground surface levels.

9. Limitations

Douglas Partners (Douglas) has prepared this report for this proposed conference centre and office tower at 12 - 16 Campbell Street, Bowen Hills QLD in accordance with Douglas' proposal 227045.00.R.001.Rev0 dated 6 April 2024 and acceptance received from Tim Johnson dated 6 April 2024. The work was carried out under Douglas' Engagement Terms. This report is provided for the exclusive use of New Urban Villages 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 Douglas, does so entirely at its own risk and without recourse to Douglas for any loss or damage. In preparing this report Douglas has necessarily relied upon information provided by the client and/or their agents.

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

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

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. Douglas 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 Douglas. This is because this report has been written as advice and opinion rather than instructions for construction.

Asbestos has not been detected by observation or by laboratory analysis, either on the surface of the site, or in fill materials at the test locations sampled and analysed. Building demolition materials, such as concrete, brick, tile [list as appropriate to the field work findings], were, however, located in previous below-ground fill and/or above-ground stockpiles [as appropriate], and these are considered as indicative of the possible presence of hazardous building materials (HBM), including asbestos.

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

The scope of work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of fill 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 fill may contain contaminants and hazardous building materials.

10. **References**

AS 1170.4:2007, *Structural Design Actions, Part 4: Earthquake actions in Australia*, Standards Australia.

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AS 2870:2011, “Residential slabs and footings”; Standards Australia.

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AS 4678:2002, 2002, “Earth-retaining structures”, Sydney, NSW: Standards Australia.

Fox E, 2000, “A Climate-Based Design Depth of Moisture Change Map of Queensland and the Use of Such Maps to Classify Sites Under AS 2870:1996”, Australian Geomechanics, Vol 35, No 4.

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Appendix A

About this Report
Terminology, Symbols and Abbreviations
Soil Descriptions
Rock Descriptions
Sampling, Testing and Excavation Methodology

About this Report

November 2023

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.

continued next page

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.

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Terminology, Symbols and Abbreviations

November 2023

Introduction to Terminology, Symbols and Abbreviations

Douglas Partners' reports, investigation logs, and other correspondence may use terminology which has quantitative or qualitative connotations. To remove ambiguity or uncertainty surrounding the use of such terms, the following sets of notes pages may be attached Douglas Partners' reports, depending on the work performed and conditions encountered:

- Soil Descriptions;
- Rock Descriptions; and
- Sampling, insitu testing, and drilling methodologies

In addition to these pages, the following notes generally apply to most documents.

Abbreviation Codes

Site conditions may also be presented in a number of different formats, such as investigation logs, field mapping, or as a written summary. In some of these formats textual or symbolic terminology may be presented using textual abbreviation codes or graphic symbols, and, where commonly used, these are listed alongside the terminology definition. For ease of identification in these note pages, textual codes are presented in these notes in the following style `XW`. Code usage conforms with the following guidelines:

- Textual codes are case insensitive, although herein they are generally presented in upper case; and
- Textual codes are contextual (i.e. the same or similar combinations of characters may be used in different contexts with different meanings (for example `PL` is used for plastic limit in the context of soil moisture condition, as well as in `PL(A)` for point load test result in the testing results column)).

Data Integrity Codes

Subsurface investigation data recorded by Douglas Partners is generally managed in a highly structured database environment, where records "span" between a top and bottom depth interval. Depth interval "gaps" between records are considered to introduce ambiguity, and, where appropriate, our practice guidelines may require contiguous data sets. Recording meaningful data is not always appropriate (for example assigning a "strength" to a concrete pavement) and the following codes may be used to maintain contiguity in such circumstances.

Term	Description	Abbreviation Code
Core loss	No core recovery	`KL`
Unknown	Information was not available to allow classification of the property. For example, when auguring in loose, saturated sand auger cuttings may not be returned.	`UK`
No data	Information required to allow classification of the property was not available. For example, if drilling is commenced from the base of a hole predrilled by others	`ND`
Not Applicable	Derivation of the properties not appropriate or beyond the scope of the investigation. For example, providing a description of the strength of a concrete pavement	`NA`

Graphic Symbols

Douglas Partners' logs contain a "graphic" column which provides a pictorial representation of the basic composition of the material. The symbols used are directly representing the material name stated in the adjacent "Description of Strata" column, and as such no specific graphic symbology legend has been provided in these notes.

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Soil Descriptions

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Introduction

All materials which are not considered to be "in-situ rock" are described in general accordance with the soil description model of AS 1726-2017 Part 6.1.3, and can be broken down into the following description structure:

The "classification" comprises a two character "group symbol" providing a general summary of dominant soil characteristics. The "name" summarises the particle sizes within the soil which most influence its behaviour. The detailed description presents more information about composition, condition, structure, and origin of the soil.

Classification, naming and description of soils require the relative proportion of particles of different sizes within the whole soil mixture to be considered.

Particle size designation and Behaviour Model

Solid particles within a soil are differentiated on the basis of size.

The engineering behaviour properties of a soil can subsequently be modelled to be either "fine grained" (also known as "cohesive" behaviour) or "coarse grained" ("non cohesive" behaviour), depending on the relative proportion of fine or coarse fractions in the soil mixture.

Particle Size Designation	Particle Size (mm)	Behaviour Model	
		Behaviour	Approximate Dry Mass
Boulder	>200	Excluded from particle behaviour model as "oversize"	
Cobble	63 - 200		
Gravel[1]	2.36 - 63	Coarse	>65%
Sand[1]	0.075 - 2.36		
Silt	0.002 - 0.075	Fine	>35%
Clay	<0.002		

[1] – refer grain size subdivision descriptions below

The behaviour model boundaries defined above are not precise, and the material behaviour should be assumed from the name given to the material (which considers the particle fraction which dominates the behaviour, refer "component proportions" below), rather than strict observance of the proportions of particle sizes. For example, if a material is named a "Sandy CLAY", this is indicative that the material exhibits fine grained behaviour, even if the dry mass of coarse grained material may exceed 65%.

Component proportions

The relative proportion of the dry mass of each particle size fraction is assessed to be a "primary", "secondary", or "minor" component of the soil mixture, depending on its influence over the soil behaviour.

Component Proportion Designation	Definition[1]	Relative Proportion	
		In Fine Grained Soil	In Coarse Grained Soil
Primary	The component (particle size designation, refer above) which dominates the engineering behaviour of the soil	The clay/silt component with the greater proportion	The sand/gravel component with the greater proportion
Secondary	Any component which is not the primary, but is significant to the engineering properties of the soil	Any component with greater than 30% proportion	Any granular component with greater than 30%; or Any fine component with greater than 12%
Minor[2]	Present in the soil, but not significant to its engineering properties	All other components	All other components

[1] As defined in AS1726-2017 6.1.4.4

[2] In the detailed material description, minor components are split into two further sub-categories. Refer "identification of minor components" below.

Composite Materials

In certain situations, a lithology description may describe more than one material, for example, collectively describing a layer of interbedded sand and clay. In such a scenario, the two materials would be described independently, with the names preceded or followed by a statement describing the arrangement by which the materials co-exist. For example, "INTERBEDDED Silty CLAY AND SAND".

Soil Descriptions

Classification

The soil classification comprises a two character group symbol. The first character identifies the primary component. The second character identifies either the grading or presence of fines in a coarse grained soil, or the plasticity in a fine grained soil. Refer AS1726-2017 6.1.6 for further clarification.

Soil Name

For most soils, the name is derived with the primary component included as the noun (in upper case), preceded by any secondary components stated in an adjective form. In this way, the soil name also describes the general composition and indicates the dominant behaviour of the material.

Component[1]	Prominence in Soil Name
Primary	Noun (eg "CLAY")
Secondary	Adjective modifier (eg "Sandy")
Minor	No influence

[1] – for determination of component proportions, refer component proportions on previous page

For materials which cannot be disaggregated, or which are not comprised of rock or mineral fragments, the names "ORGANIC MATTER" or "ARTIFICIAL MATERIAL" may be used, in accordance with AS1726-2017 Table 14.

Commercial or colloquial names are not used for the soil name where a component derived name is possible (for example "Gravelly SAND" rather than "CRACKER DUST").

Materials of "fill" or "topsoil" origin are generally assigned a name derived from the primary/secondary component (where appropriate). In log descriptions this is preceded by uppercase "FILL" or "TOPSOIL". Origin uncertainty is indicated in the description by the characters (?), with the degree of uncertainty described (using the terms "probably" or "possibly" in the origin column, or at the end of the description).

Identification of minor components

Minor components are identified in the soil description immediately following the soil name. The minor component fraction is usually preceded with a term indicating the relative proportion of the component.

Minor Component Proportion Term	Relative Proportion	
	In Fine Grained Soil	In Coarse Grained Soil
With	All fractions: 15-30%	Clay/silt: 5-12% sand/gravel: 15-30%
Trace	All fractions: 0-15%	Clay/silt: 0-5% sand/gravel: 0-15%

The terms "with" and "trace" generally apply only to gravel or fine particle fractions. Where cobbles/boulders are encountered in minor proportions (generally less than about 12%) the term "occasional" may be used. This term describes the sporadic distribution of the material within the confines of the investigation excavation only, and there may be considerable variation in proportion over a wider area which is difficult to factually characterise due to the relative size of the particles and the investigation methods.

Soil Composition

Plasticity

Descriptive Term	Laboratory liquid limit range	
	Silt	Clay
Non-plastic materials	Not applicable	Not applicable
Low plasticity	≤50	≤35
Medium plasticity	Not applicable	>35 and ≤50
High plasticity	>50	>50

Note, Plasticity descriptions generally describe the plasticity behaviour of the whole of the fine grained soil, not individual fine grained fractions.

Grain Size

Type		Particle size (mm)
Gravel	Coarse	19 - 63
	Medium	6.7 - 19
	Fine	2.36 – 6.7
Sand	Coarse	0.6 - 2.36
	Medium	0.21 - 0.6
	Fine	0.075 - 0.21

Grading

Grading Term	Particle size (mm)
Well	A good representation of all particle sizes
Poorly	An excess or deficiency of particular sizes within the specified range
Uniformly	Essentially of one size
Gap	A deficiency of a particular size or size range within the total range

Note, AS1726-2017 provides terminology for additional attributes not listed here.

Soil Condition

Moisture

The moisture condition of soils is assessed relative to the plastic limit for fine grained soils, while for coarse grained soils it is assessed based on the appearance and feel of the material. The moisture condition of a material is considered to be independent of stratigraphy (although commonly these are related), and this data is presented in its own column on logs.

Applicability	Term	Tactile Assessment	Abbreviation code
Fine	Dry of plastic limit	Hard and friable or powdery	w<PL
	Near plastic limit	Can be moulded	w=PL
	Wet of plastic limit	Water residue remains on hands when handling	w>PL
	Near liquid limit	"oozes" when agitated	w=LL
	Wet of liquid limit	"oozes"	w>LL
Coarse	Dry	Non-cohesive and free running	D
	Moist	Feels cool, darkened in colour, particles may stick together	M
	Wet	Feels cool, darkened in colour, particles may stick together, free water forms when handling	W

The abbreviation code NDF, meaning "not-assessable due to drilling fluid use" may also be used.

Note, observations relating to free ground water or drilling fluids are provided independent of soil moisture condition.

Consistency/Density/Compaction/Cementation/Extremely Weathered Material

These concepts give an indication of how the material may respond to applied forces (when considered in conjunction with other attributes of the soil). This behaviour can vary independent of the composition of the material, and on logs these are described in an independent column and are generally mutually exclusive (i.e it is inappropriate to describe both consistency and compaction at the same time). The method by which the behaviour is described depends on the behaviour model and other characteristics of the soil as follows:

- In fine grained soils, the "consistency" describes the ease with which the soil can be remoulded, and is generally correlated against the materials undrained shear strength;
- In granular materials, the relative density describes how tightly packed the particles are, and is generally correlated against the density index;
- In anthropogenically modified materials, the compaction of the material is described qualitatively;
- In cemented soils (both natural and anthropogenic), the cemented "strength" is described qualitatively, relative to the difficulty with which the material is disaggregated; and
- In soils of extremely weathered material origin, the engineering behaviour may be governed by relic rock features, and expected behaviour needs to be assessed based the overall material description.

Quantitative engineering performance of these materials may be determined by laboratory testing or estimated by correlated field tests (for example penetration or shear vane testing). In some cases, performance may be assessed by tactile or other subjective methods, in which case investigation logs will show the estimated value enclosed in round brackets, for example (VS).

Consistency (fine grained soils)

Consistency Term	Tactile Assessment	Undrained Shear Strength (kPa)	Abbreviation Code
Very soft	Extrudes between fingers when squeezed	<12	VS
Soft	Mouldable with light finger pressure	>12 - ≤25	S
Firm	Mouldable with strong finger pressure	>25 - ≤50	F
Stiff	Cannot be moulded by fingers	>50 - ≤100	St
Very stiff	Indented by thumbnail	>100 - ≤200	VSt
Hard	Indented by thumbnail with difficulty	>200	H
Friable	Easily crumbled or broken into small pieces by hand	-	Fr

Relative Density (coarse grained soils)

Relative Density Term	Density Index	Abbreviation Code
Very loose	<15	VL
Loose	>15 - ≤35	L
Medium dense	>35 - ≤65	MD
Dense	>65 - ≤85	D
Very dense	>85	VD

Note, tactile assessment of relative density is difficult, and generally requires penetration testing, hence a tactile assessment guide is not provided.

Soil Descriptions

Compaction (anthropogenically modified soil)

Compaction Term	Abbreviation Code
Well compacted	WC
Poorly compacted	PC
Moderately compacted	MC
Variably compacted	VC

Cementation (natural and anthropogenic)

Cementation Term	Abbreviation Code
Moderately cemented	MOD
Weakly cemented	WEK

Extremely Weathered Material

AS1726-2017 considers weathered material to be soil if the unconfined compressive strength is less than 0.6 MPa (i.e. less than very low strength rock). These materials may be identified as "extremely weathered material" in reports and by the abbreviation code XWM on log sheets. This identification is not correlated to any specific qualitative or quantitative behaviour, and the engineering properties of this material must therefore be assessed according to engineering principles with reference to any relic rock structure, fabric, or texture described in the description.

Soil Origin

Term	Description	Abbreviation Code
Residual	Derived from in-situ weathering of the underlying rock	RS
Extremely weathered material	Formed from in-situ weathering of geological formations. Has strength of less than 'very low' as per as1726 but retains the structure or fabric of the parent rock.	XWM
Alluvial	Deposited by streams and rivers	ALV
Estuarine	Deposited in coastal estuaries	EST
Marine	Deposited in a marine environment	MAR
Lacustrine	Deposited in freshwater lakes	LAC
Aeolian	Carried and deposited by wind	AEO
Colluvial	Soil and rock debris transported down slopes by gravity	COL
Slopewash	Thin layers of soil and rock debris gradually and slowly deposited by gravity and possibly water	SW
Topsoil	Mantle of surface soil, often with high levels of organic material	TOP
Fill	Any material which has been moved by man	FILL
Littoral	Deposited on the lake or seashore	LIT
Unidentifiable	Not able to be identified	UID

Cobbles and Boulders

The presence of particles considered to be "oversize" may be described using one of the following strategies:

- Oversize encountered in a minor proportion (when considered relative to the wider area) are noted in the soil description; or
- Where a significant proportion of oversize is encountered, the cobbles/boulders are described independent of the soil description, in a similar manner to composite soils (described above) but qualified with "MIXTURE OF".

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Rock Descriptions

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Rock Strength

Rock strength is defined by the unconfined compressive strength, and it 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 Point Load Strength Index $I_{s(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

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

[1] Rock strength classification is based on UCS. The UCS to $I_{s(50)}$ ratio varies significantly for different rock types and specific ratios may be required for each site. The point load Index ranges shown above are as suggested in AS1726 and should not be relied upon without supporting evidence.

The following abbreviation codes are used for soil layers or seams of material "within rock" but for which the equivalent UCS strength is less than 0.6 MPa.

Scenario	Abbreviation Code
The material encountered has an equivalent UCS strength of less than 0.6 MPa, and therefore is considered to be soil (as per Note 1 of Table 20 of AS 1726-2017). The properties of the material encountered over this interval are described in the "Description of Strata" and soil properties columns.	SOIL
The material encountered has an equivalent UCS strength of less than 0.6 MPa, and therefore is considered to be soil (as per Note 1 of Table 20 of AS 1726-2017). The prominence of the material is such that it can be considered to be a seam (as defined in Table 22 of AS1726-2017) and the properties of the material are described in the defect column.	SEAM

Degree of Weathering

The degree of weathering of rock is classified as follows:

Weathering Term	Description	Abbreviation Code
Residual Soil[1]	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.	RS
Extremely weathered[1]	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible	XW
Highly weathered	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching or may be decreased due to deposition of weathering products in pores.	HW
Moderately weathered	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable but shows little or no change of strength from fresh rock.	MW
Slightly weathered	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.	SW
Fresh	No signs of decomposition or staining.	FR
Note: If HW and MW cannot be differentiated use DW (see below)		
Distinctly weathered	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching or may be decreased due to deposition of weathered products in pores.	DW

[1] The parent rock type, of which the residual/extremely weathered material is a derivative, will be stated in the description (where discernible).

Rock Descriptions

Degree of Alteration

The degree of alteration of the rock material (physical or chemical changes caused by hot gasses or liquids at depth) is classified as follows:

Term	Description	Abbreviation Code
Extremely altered	Material is altered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.	XA
Highly altered	The whole of the rock material is discoloured, usually by staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is changed by alteration. Some primary minerals are altered to clay minerals. Porosity may be increased by leaching or may be decreased due to precipitation of secondary materials in pores.	HA
Moderately altered	The whole of the rock material is discoloured, usually by staining or bleaching to the extent that the colour of the original rock is not recognisable but shows little or no change of strength from fresh rock.	MA
Slightly altered	Rock is slightly discoloured but shows little or no change of strength from fresh rock	SA
Note: If HA and MA cannot be differentiated use DA (see below)		
Distinctly altered	Rock strength usually changed by alteration. The rock may be highly discoloured, usually by staining or bleaching. Porosity may be increased by leaching or may be decreased due to precipitation of secondary minerals in pores.	DA

Degree of Fracturing

The following descriptive classification apply to the spacing of natural occurring fractures in the rock mass. It includes bedding plane partings, joints and other defects, but excludes drilling breaks. These terms are generally not required on investigation logs where fracture spacing is presented as a histogram, and where used are presented in an unabbreviated format.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

Rock Quality Designation

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

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

where 'sound' rock is assessed to be rock of low strength or stronger. 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

These terms may be used to describe the spacing of bedding partings in sedimentary rocks. Where used, these terms are generally presented in an unabbreviated format

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

Rock Descriptions

Defect Descriptions

Defect Type

Term	Abbreviation Code
Bedding plane	B
Infilled seam	IS
Cleavage	CV
Crushed zone	CZ
Decomposed seam	DS
Fault	F
Joint	JT
Lamination	LAM
Parting	P
Shear zone	SZ
Vein	VN
Drilling/handling break	DB , HB
Fracture	FC

Rock Defect Orientation

Term	Abbreviation Code
Horizontal	H
Vertical	V
Sub-horizontal	SH
Sub-vertical	SV

Rock Defect Coating

Term	Abbreviation Code
Clean	CN
Coating	CT
Healed	HE
Infilled	INF
Stained	SN
Tight	TI
Veneer	VNR

Rock Defect Infill

Term	Abbreviation Code
Calcite	CA
Carbonaceous	CBS
Clay	CLAY
Iron oxide	FE
Manganese	MN

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Rock Defect Shape/Planarity

Term	Abbreviation Code
Curved	CU
Irregular	IR
Planar	PR
Stepped	ST
Undulating	UN

Rock Defect Roughness

Term	Abbreviation Code
Polished	PO
Rough	RF
Slickensided	SL
Smooth	SM
Very rough	VR

Defect Orientation

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

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Sampling, Testing and Excavation Methodology

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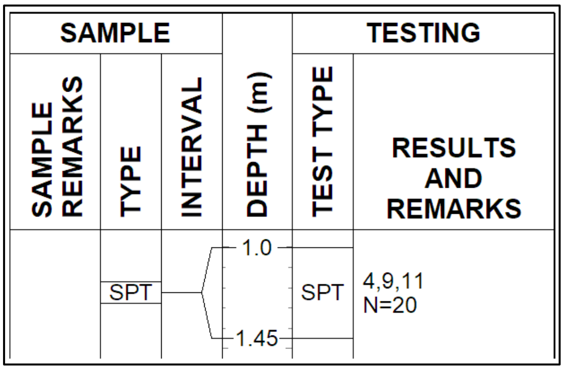
Sampling and Testing

A record of samples retained, and field testing performed is usually shown on a Douglas Partners' log with samples appearing to the left of a depth scale, and selected field and laboratory testing (including results, where relevant) appearing to the right of the scale, as illustrated below:



Sampling

The type or intended purpose for which a sample was taken is indicated by the following abbreviation codes.

Sample Type	Code
Auger sample	A
Bulk sample	B
Core sample	C
Disturbed sample	D
Sample from SPT test	SPT
Environmental sample	ES
Gas sample	G
Undisturbed tube sample	U[1]
Water sample	W
Piston sample	P
Core sample for unconfined compressive strength testing	UCS
Material Sample	MT

[1] – numeric suffixes indicate tube diameter/width in mm

The above codes only indicate that a sample was retained, and not that testing was scheduled or performed.

Field and Laboratory Testing

A record that field and laboratory testing was performed is indicated by the following abbreviation codes.

Test Type	Code
Pocket penetrometer (kPa)	PP
Photo ionisation detector (ppm)	PID
Standard Penetration Test x/y =*x* blows for *y* mm penetration HB = hammer bouncing HW = fell under weight of hammer	SPT
Shear vane (kPa)	V
Unconfined compressive strength, (MPa)	UCS
Point load test, (MPa), axial (A), diametric (D), irregular (I)	PLT(_)
Dynamic cone penetrometer, followed by blow count penetration increment in mm (cone tip, generally in accordance with AS1289.6.3.2)	DCP/150
Perth sand penetrometer, followed by blow count penetration increment in mm (flat tip, generally in accordance with AS1289.6.3.3)	PSP/150

Groundwater Observations

▷	seepage/inflow
▽	standing or observed water level
NFGWO	no free groundwater observed
OBS	observations obscured by drilling fluids

Drilling or Excavation Methods/Tools

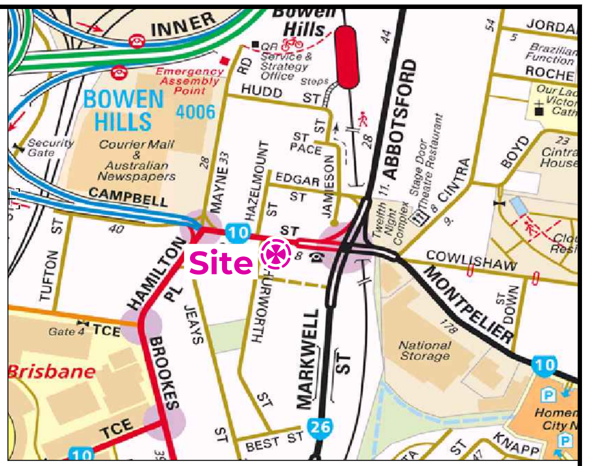
The drilling/excavation methods used to perform the investigation may be shown either in a dedicated column down the left-hand edge of the log, or stated in the log footer. In some circumstances abbreviation codes may be used.

Method	Abbreviation Code
Toothed bucket	TB[1]
Mud/blade bucket	MB[1]
Ripping tyne/ripper	R
Rock breaker/hydraulic hammer	RB
Hand auger	HA[1]
NMLC series coring	NMLC
HMLC series coring	HMLC
NQ coring	NQ3
HQ coring	HQ3
PQ coring	PQ3
Push tube	PT[1]
Rock roller	RR[1]
Solid flight auger. Suffixes: /T = tungsten carbide tip, /V = v-shaped tip	AD[1]
Sonic drilling	SON[1]
Vibrocore	VC[1]
Wash bore (unspecified bit type)	WB[1]
Existing exposure	X
Hand tools (unspecified)	HAND
Predrilled	PD
Diatube	DT[1]
Hollow flight auger	HSA[1]
Vacuum excavation	VE

[1] – numeric suffixes indicate tool diameter/width in mm

Appendix B

Site and Test location Plans - Drawing 1



Location Plan

LEGEND:-

⌖	Borehole Location and Number
▪▪▪▪	Approximate Site Boundary

REV	DESCRIPTION/COMMENT	DATE	DRAWN BY
0		05/06/2024	JST

: ZONE 56
COORDINATE REFERENCE SYSTEM: MGA2020

NOTE:

- Base image from MetroMap (Dated 18.06.2023)
- Site locality obtained from street-directory.com.au. Not to scale.

Douglas PARTNERS

OFFICE: BRISBANE
439 Montague Road, West End, QLD, 4101

CLIENT: **New Urban Villages Pty Ltd**

SCALE: 0 2 4 6 8 10
SCALE 1:300 (A4)

PROJECT NAME: **Proposed Conference Centre and Office Tower**

PROJECT ADDRESS: **12 - 16 Campbell Street, Bowen Hills**

DRAWING TITLE: **SITE AND TEST LOCATION PLAN**

PROJECT No: **227045.00**

DRAWING No: **1**

REVISION: **0**

Appendix C

Field Work Results

BOREHOLE LOG

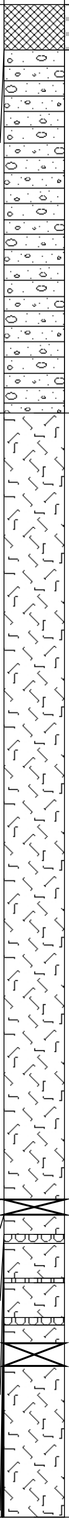
CLIENT: New Urban Villages Pty Ltd
PROJECT: Proposed Conference Centre and Office Tower
LOCATION: 12 - 16 Campbell Street, Bowen Hills, QLD 4006

SURFACE LEVEL: 16.8 AHD
COORDINATE: E:503652.0, N:6963950.9
DATUM/GRID: MGA2020 Zone 56
DIP/AZIMUTH: 90°/---°

LOCATION ID: 1
PROJECT No: 227045.00
DATE: 19/03/24
SHEET: 1 of 2

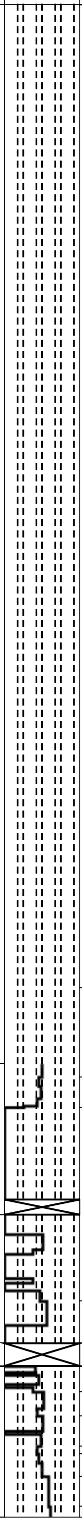
CONDITIONS ENCOUNTERED

DEPTH (m)	DESCRIPTION OF STRATA	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE
	FILL / Gravelly SAND (SW), with clay: brown; fine to coarse; fine to medium, sub-angular to angular, with coarse sub-angular to angular gravel.	FILL	D VD	M
0.30	Sandy Gravelly CLAY (CI), with silt: orange; medium plasticity; fine to coarse sand; fine to medium, sub-angular gravel; low plasticity silt. 1.50m-1.60m: interbedded extremely weathered conglomerate	RS	St	w<PL
2.70	PHYLLITE: grey-blue, fine grained; (Neranleigh Fernvale Beds) 7.29m: Interbedded Crushed Seam - Sandy Gravelly Clay at 7.29m, 8.26m, 8.45m, 9.04m, 9.12m, 9.43m, 10.59m, 11.2m, 11.57m, 11.73m			
7.90	CORE LOSS			
8.00 8.13 8.19 8.42	PHYLLITE: grey-blue, fine grained; (Neranleigh Fernvale Beds)			
8.45 8.68 8.73 8.85	CONGLOMERATE: grey-orange, fine to coarse grained. (Neranleigh Fernvale Beds)			
9.00	PHYLLITE: grey/red/orange, fine grained; fractured, foliated (Neranleigh Fernvale Beds)			



ROCK

DEPTH (m)	WEATH.	STRENGTH	RECOVERY (%)	RQD
2.70	HW	VL		
5.70		L		
7.00	MW		90	0
7.09	XW			
7.29				
8.00	XW	M	85	12
8.13	MW			
8.23	HW	L		
8.50	MW	M		
8.65	HW	L		
9.00	MW	H	100	57



DEFECTS & REMARKS

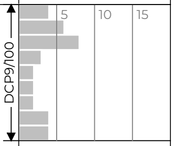
6.50m : , Unless otherwise stated, defects are: JT/10,20,55°, IR or UN or ST, SN FE, Clay, RF or if it is closed (CS)
7.09m: P/30°, IR, VNR Clay, RF
7.29m : , 50-70°, CS/SH
8.57m: P/80°, ST, CT Clay, RF
9.00m: DB
9.17m : P/20°, ST, SN Fe, RF
9.33m: P/35°, ST, VNR Clay, RF, Fe SN
9.53m: JT/10°, UN, VNR Clay, SM, Fe SN

SAMPLE

TYPE	DEPTH (m)
D	
D	0.30 0.50
D	1.00
SPT	1.00 1.45
D	1.50
U50	1.50 1.77
D	2.40 2.50
SPT	2.50 2.79
D	3.80
D	4.00
SPT	4.00 4.23
SPT	5.50
SPT	7.00
C	7.00 8.00
C	8.00 9.00
C	9.00

TESTING

TEST TYPE	RESULTS AND REMARKS
DCP9/100	
SPT	2,4,7 N=11
SPT	12,30/140
SPT	18,30/80
SPT	10/30
SPT	15/40
PLT	PL(A)=0.39MPa
PLT	PL(D)=7.14MPa
PLT	PL(A)=1.18MPa
PLT	PL(D)=0.6MPa



NOTES: (#)Soil origin is "probable" unless otherwise stated. (*)Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

PLANT: Hanjin DB8
OPERATOR: Terratest
LOGGED: JB
METHOD: Solid flight auger to 2.5m, washbore to 7.0m, then NMLC coring
CASING: HWT to 2.5m, then HQ to 7m
REMARKS: Well measured to be 2.4m

Refer to explanatory notes for symbol and abbreviation definitions

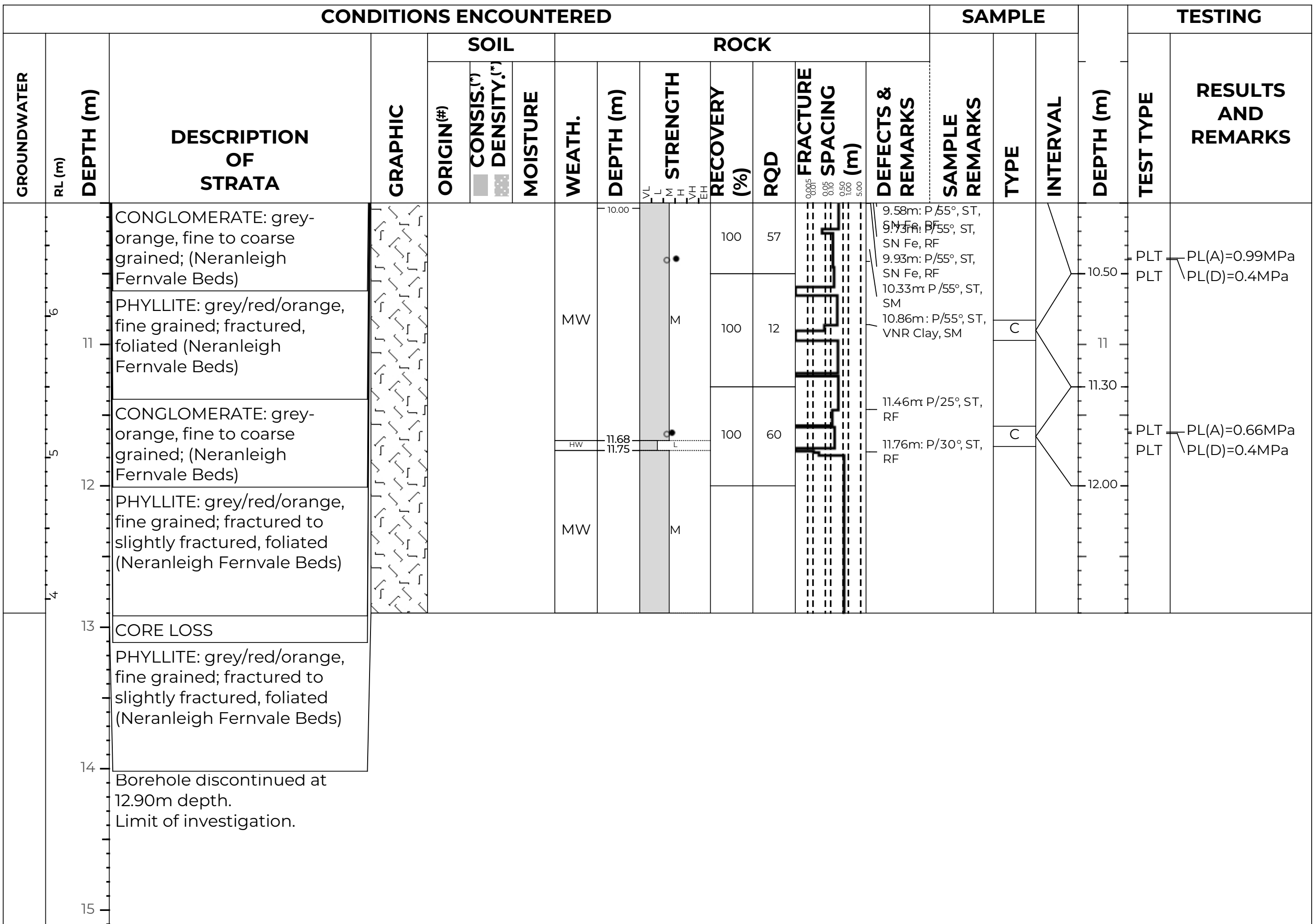
Douglas PARTNERS

BOREHOLE LOG

CLIENT: New Urban Villages Pty Ltd
PROJECT: Proposed Conference Centre and Office Tower
LOCATION: 12 - 16 Campbell Street, Bowen Hills, QLD 4006

SURFACE LEVEL: 16.8 AHD
COORDINATE: E:503652.0, N:6963950.9
DATUM/GRID: MGA2020 Zone 56
DIP/AZIMUTH: 90°/---°

LOCATION ID: 1
PROJECT No: 227045.00
DATE: 19/03/24
SHEET: 2 of 2



DEPTH (m)	DESCRIPTION OF STRATA
10.00	CONGLOMERATE: grey-orange, fine to coarse grained; (Neranleigh Fernvale Beds)
	PHYLLITE: grey/red/orange, fine grained; fractured, foliated (Neranleigh Fernvale Beds)
	CONGLOMERATE: grey-orange, fine to coarse grained; (Neranleigh Fernvale Beds)
	PHYLLITE: grey/red/orange, fine grained; fractured to slightly fractured, foliated (Neranleigh Fernvale Beds)
	CORE LOSS
	PHYLLITE: grey/red/orange, fine grained; fractured to slightly fractured, foliated (Neranleigh Fernvale Beds)
	Borehole discontinued at 12.90m depth. Limit of investigation.

WEATH.	DEPTH (m)	STRENGTH
MW	10.00	M
HW	11.68	L
MW	11.75	M

RECOVERY (%)	RQD
100	57
100	12
100	60

DEFECTS & REMARKS

9.58m: P/55°, ST, SN Fe, RF
9.73m: P/55°, ST, SN Fe, RF
9.93m: P/55°, ST, SN Fe, RF
10.33m: P/55°, ST, SM
10.86m: P/55°, ST, VNR Clay, SM
11.46m: P/25°, ST, RF
11.76m: P/30°, ST, RF

TYPE	INTERVAL DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
C	10.50 – 11.30	PLT	PL(A)=0.99MPa
		PLT	PL(D)=0.4MPa
C	11.30 – 12.00	PLT	PL(A)=0.66MPa
		PLT	PL(D)=0.4MPa

Generated with CORE-GS by Geroc - Combined Log

NOTES: (#)Soil origin is "probable" unless otherwise stated. (*)Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

PLANT: Hanjin DB8
OPERATOR: Terratest
LOGGED: JB
METHOD: Solid flight auger to 2.5m, washbore to 7.0m, then NMLC coring
CASING: HWT to 2.5m, then HQ to 7m
REMARKS: Well measured to be 2.4m

Refer to explanatory notes for symbol and abbreviation definitions

Douglas PARTNERS

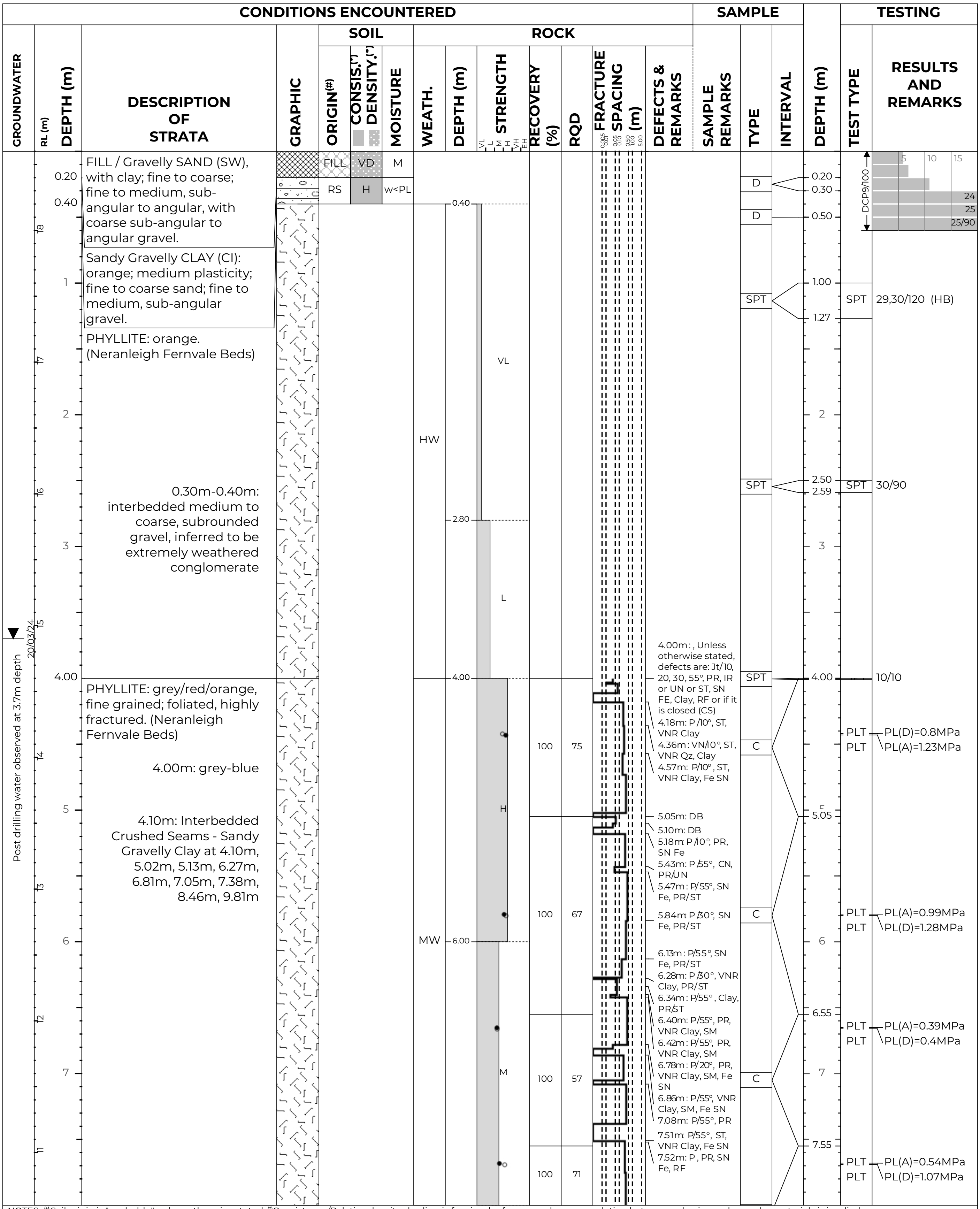
Douglas Partners Geotechnics / Environment / Groundwater	CLIENT:	New Urban Villages Pty Ltd	**Core Photograph – Bore 1, Box 1 of 1**	PROJECT No:	227045.00
	OFFICE:	Brisbane	**Propose Conference Centre and Office Tower**	PLATE No:	1
	DATE:	4 June 2024	**12 – 16 Campbell Stret, Bowen Hills**	REVISION:	0

BOREHOLE LOG

CLIENT: New Urban Villages Pty Ltd
PROJECT: Proposed Conference Centre and Office Tower
LOCATION: 12 - 16 Campbell Street, Bowen Hills, QLD 4006

SURFACE LEVEL: 18.6 AHD
COORDINATE: E:503664.2, N:6963767.0
DATUM/GRID: MGA2020 Zone 56
DIP/AZIMUTH: 90°/---°

LOCATION ID: 2
PROJECT No: 227045.00
DATE: 20/03/24
SHEET: 1 of 2



DEPTH (m)	DESCRIPTION OF STRATA
0.00	FILL / Gravelly SAND (SW), with clay; fine to coarse; fine to medium, sub-angular to angular, with coarse sub-angular to angular gravel.
0.20	Sandy Gravelly CLAY (CI): orange; medium plasticity; fine to coarse sand; fine to medium, sub-angular gravel.
0.40	PHYLLITE: orange. (Neranleigh Fernvale Beds) 0.30m-0.40m: interbedded medium to coarse, subrounded gravel, inferred to be extremely weathered conglomerate
4.00	PHYLLITE: grey/red/orange, fine grained; foliated, highly fractured. (Neranleigh Fernvale Beds) 4.00m: grey-blue 4.10m: Interbedded Crushed Seams - Sandy Gravelly Clay at 4.10m, 5.02m, 5.13m, 6.27m, 6.81m, 7.05m, 7.38m, 8.46m, 9.81m

RUN (m)	RECOVERY (%)	RQD
4.00–5.05	100	75
5.05–6.55	100	67
6.55–7.55	100	57
7.55–	100	71

DEFECTS & REMARKS

4.00m : , Unless otherwise stated, defects are: Jt/10, 20, 30, 55°, PR, IR or UN or ST, SN FE, Clay, RF or if it is closed (CS)
4.18m: P /10°, ST, VNR Clay
4.36m : VN/10°, ST, VNR Qz, Clay
4.57m: P/10°, ST, VNR Clay, Fe SN
5.05m: DB
5.10m: DB
5.18m: P /10°, PR, SN Fe
5.43m: P /55°, CN, PR/UN
5.47m : P/55°, SN Fe, PR/ST
5.84m: P /30°, SN Fe, PR/ST
6.13m : P/55°, SN Fe, PR/ST
6.28m: P /30°, VNR Clay, PR/ST
6.34m : P/55°, Clay, PR/ST
6.40m: P/55°, PR, VNR Clay, SM
6.42m : P/55°, PR, VNR Clay, SM
6.78m : P/20°, PR, VNR Clay, SM, Fe SN
6.86m : P/55°, VNR Clay, SM, Fe SN
7.08m: P/55°, PR
7.51m: P/55°, ST, VNR Clay, Fe SN
7.52m: P , PR, SN Fe, RF

TYPE	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
	0.00–0.50	DCP9/100	5, 10, 15, 24, 25, 25/90
D	0.20–0.30		
D	0.50		
SPT	1.00–1.27	SPT	29,30/120 (HB)
SPT	2.50–2.59	SPT	30/90
SPT	4.00		10/10
C	4.00–5.05	PLT PLT	PL(D)=0.8MPa PL(A)=1.23MPa
C	5.05–6.55	PLT PLT	PL(A)=0.99MPa PL(D)=1.28MPa
C	6.55–7.55	PLT PLT	PL(A)=0.39MPa PL(D)=0.4MPa
	7.55–	PLT PLT	PL(A)=0.54MPa PL(D)=1.07MPa

NOTES: (#)Soil origin is "probable" unless otherwise stated. (*)Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

PLANT: Hanjin DB8
OPERATOR: Terratest
LOGGED: JB
METHOD: 125mm diameter solid flight auger to 0.7m, washbore to 4.m, then NMLC coring
CASING: HWT to 0.7m, then HQ to 4m
REMARKS: Post drilling water observed at 3.7m depth

Refer to explanatory notes for symbol and abbreviation definitions

Douglas PARTNERS

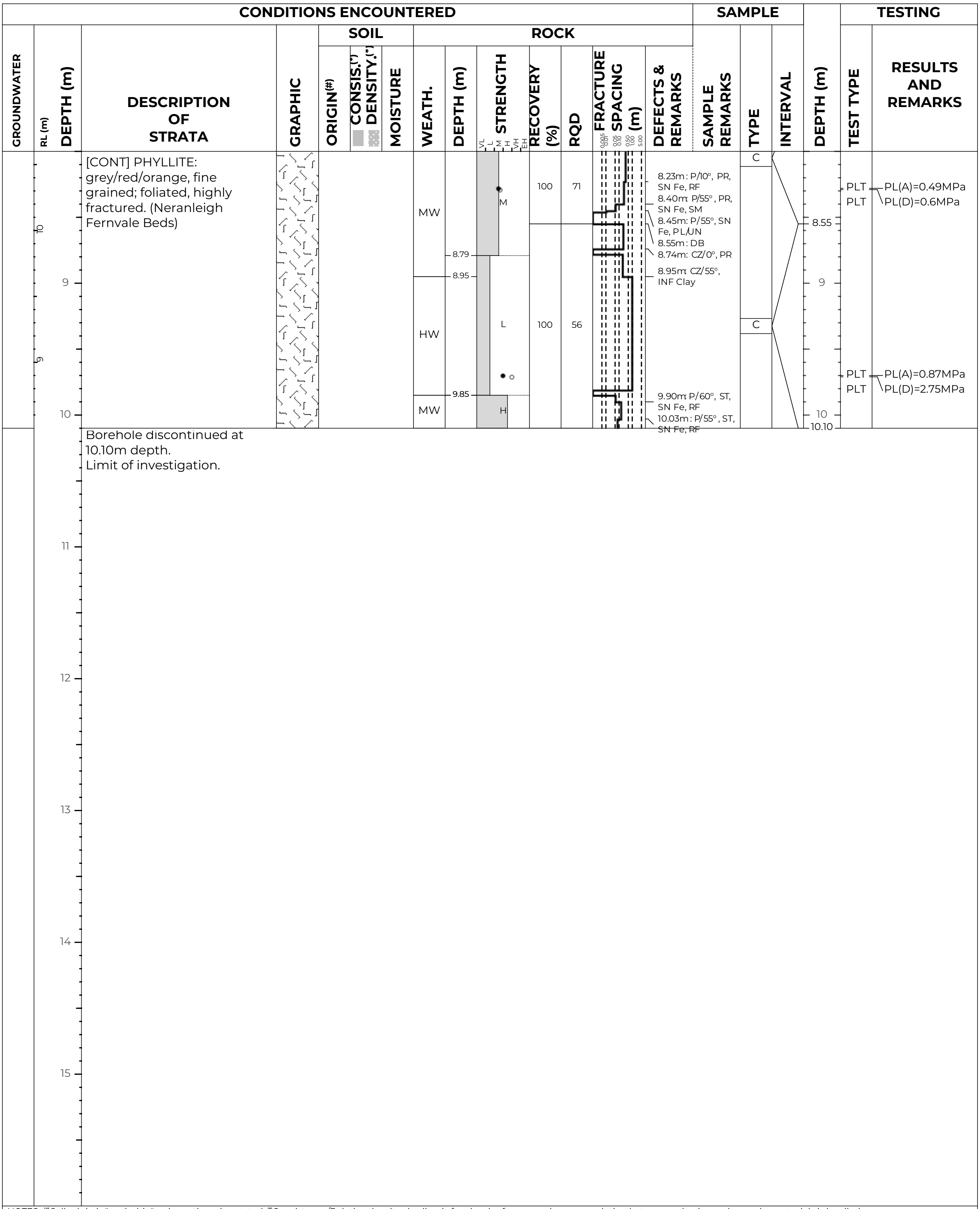
Generated with CORE-GS by Geroc - Combined Log

BOREHOLE LOG

CLIENT: New Urban Villages Pty Ltd
PROJECT: Proposed Conference Centre and Office Tower
LOCATION: 12 - 16 Campbell Street, Bowen Hills, QLD 4006

SURFACE LEVEL: 18.6 AHD
COORDINATE: E:503664.2, N:6963767.0
DATUM/GRID: MGA2020 Zone 56
DIP/AZIMUTH: 90°/---°

LOCATION ID: 2
PROJECT No: 227045.00
DATE: 20/03/24
SHEET: 2 of 2

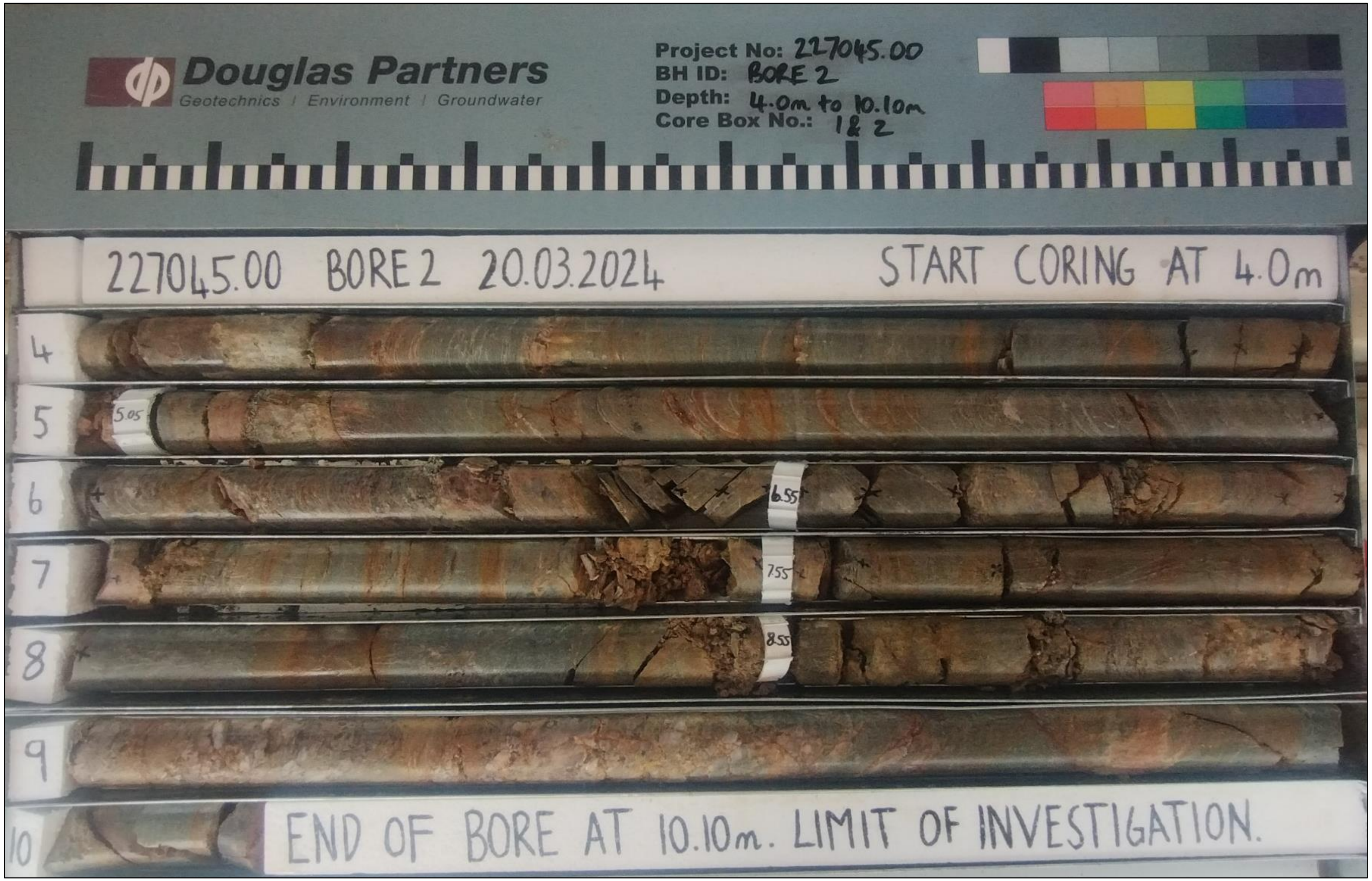


DEPTH (m)	DESCRIPTION OF STRATA	WEATH.	DEPTH (m)	STRENGTH	RECOVERY (%)	RQD	DEFECTS & REMARKS	TYPE	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
	[CONT] PHYLLITE: grey/red/orange, fine grained; foliated, highly fractured. (Neranleigh Fernvale Beds)	MW		M	100	71	8.23m: P/10°, PR, SN Fe, RF 8.40m: P/55°, PR, SN Fe, SM 8.45m: P/55°, SN Fe, PL/UN 8.55m: DB 8.74m: CZ/0°, PR	C	8.55	PLT PLT	PL(A)=0.49MPa PL(D)=0.6MPa
			8.79	L	100	56	8.95m: CZ/55°, INF Clay	C		PLT PLT	PL(A)=0.87MPa PL(D)=2.75MPa
		HW	8.95								
		MW	9.85	H			9.90m: P/60°, ST, SN Fe, RF 10.03m: P/55°, ST, SN Fe, RF		10.10		
10.10	Borehole discontinued at 10.10m depth. Limit of investigation.										

NOTES: [#]Soil origin is "probable" unless otherwise stated. [*]Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

PLANT: Hanjin DB8
OPERATOR: Terratest
LOGGED: JB
METHOD: 125mm diameter solid flight auger to 0.7m, washbore to 4.m, then NMLC coring
CASING: HWT to 0.7m, then HQ to 4m
REMARKS: Post drilling water observed at 3.7m depth

Refer to explanatory notes for symbol and abbreviation definitions



Douglas Partners Geotechnics / Environment / Groundwater	CLIENT:	New Urban Villages Pty Ltd	**Core Photograph – Bore 2, Box 1 of 1**	PROJECT No:	227045.00
	OFFICE:	Brisbane	**Propose Conference Centre and Office Tower**	PLATE No:	2
	DATE:	4 June 2024	**12 – 16 Campbell Stret, Bowen Hills**	REVISION:	0

Appendix D

Laboratory Test Results

Material Test Report

Report Number: **227045.00-1**
Issue Number: 1
Date Issued: 28/03/2024
Client: New Urban Villages Pty Ltd
c/- Milanovic Neale Consulting Engineers, Woolloongabba QLD 4102
Contact: Tim Johnson
Project Number: 227045.00
Project Name: Proposed Conference Centre and Office Tower
Project Location: 12 - 16 Campbell Street, Bowen Hills QLD
Work Request: 16338
Sample Number: BN-16338A
Date Sampled: 19/03/2024
Dates Tested: 22/03/2024 - 26/03/2024
Sampling Method: Sampled by DP Brisbane Engineering Department
The results apply to the sample as received
Sample Location: **BH01 , Depth: 1.00 - 1.45 m**
Material: Sandy gravelly CLAY

Douglas Partners Pty Ltd
Brisbane Laboratory
439 Montague Road West End QLD 4101
Phone: (07) 3237 8900
Email: aimee.cartwright@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Aimee Cartwright
Laboratory Technician
Laboratory Accreditation Number: 828

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1 & Q252)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Passing 0.425 (%)	80		
Liquid Limit (%)	40		
Plastic Limit (%)	26		
Plasticity Index (%)	**14**		
Weighted Plasticity Index (%)	1121		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	**6.5**		
Cracking Crumbling Curling	None		

Moisture Content (AS 1289 2.1.1)		Min	Max
Moisture Content (%)	16.2		