Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Desktop Study

Proposed Mixed Use Development
2 Wilson Street, Chatswood

Prepared for
853 Pacific Highway Pty Ltd ATF 2017 PHC Unit Trust

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Integrated Practical Solutions

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

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Report on Geotechnical Desktop Study
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1. Introduction

This report presents the results of a geotechnical desktop study undertaken for a proposed mixed use development at 2 Wilson Street, Chatswood. The investigation was commissioned on 22 September 2020 by Michael Andrijic on behalf of 853 Pacific Highway Pty Ltd ATF 2017 PHC Unit Trust and was undertaken in accordance with Douglas Partners' proposal SYD200997.P.001.Rev1 dated 22 September 2020.

This report has been prepared on behalf of 853 Pacific Highway Pty Ltd ATF 2017 PHC Unit Trust in support of a Planning Proposal (PP) for a proposed development which includes:

- Demolition of the existing buildings on site and excavation and construction of a 28 level building with basement parking, retail/commercial use on ground and first floor and residential use above; and
- Dedication of the western fringe of the site for a future road widening along Pacific Highway.

The desktop study comprised a review of existing geotechnical information in the vicinity of the site and preparation of the report together with preliminary comments relating to design and construction practice.

2. Site Description and Topography

The site covers approximately 3166 m^2 and is bound by Pacific Highway to the west, Wilson Street to the north, O'Brien Street to the south and an adjacent property and rail corridor to the east (including the North Shore, Western and North West Metro Lines - refer Figure 1).

The site is understood to be currently occupied by four, three to four storey brick residential buildings, including (from north to south):

- 2 Wilson Street;
- 859 Pacific Highway;
- 853 Pacific Highway; and
- 849 Pacific Highway.

Figure 1: Approximate Location of the Site

The site is situated on a ridge at approximately 106 m Australian Height Datum (AHD) as shown in Figure 2. Gentle slopes at the site are down to the south and south-east. The rail corridor to the east of the site, has been ‘cut’ into the ridge and lies approximately 5 m below the adjacent land.

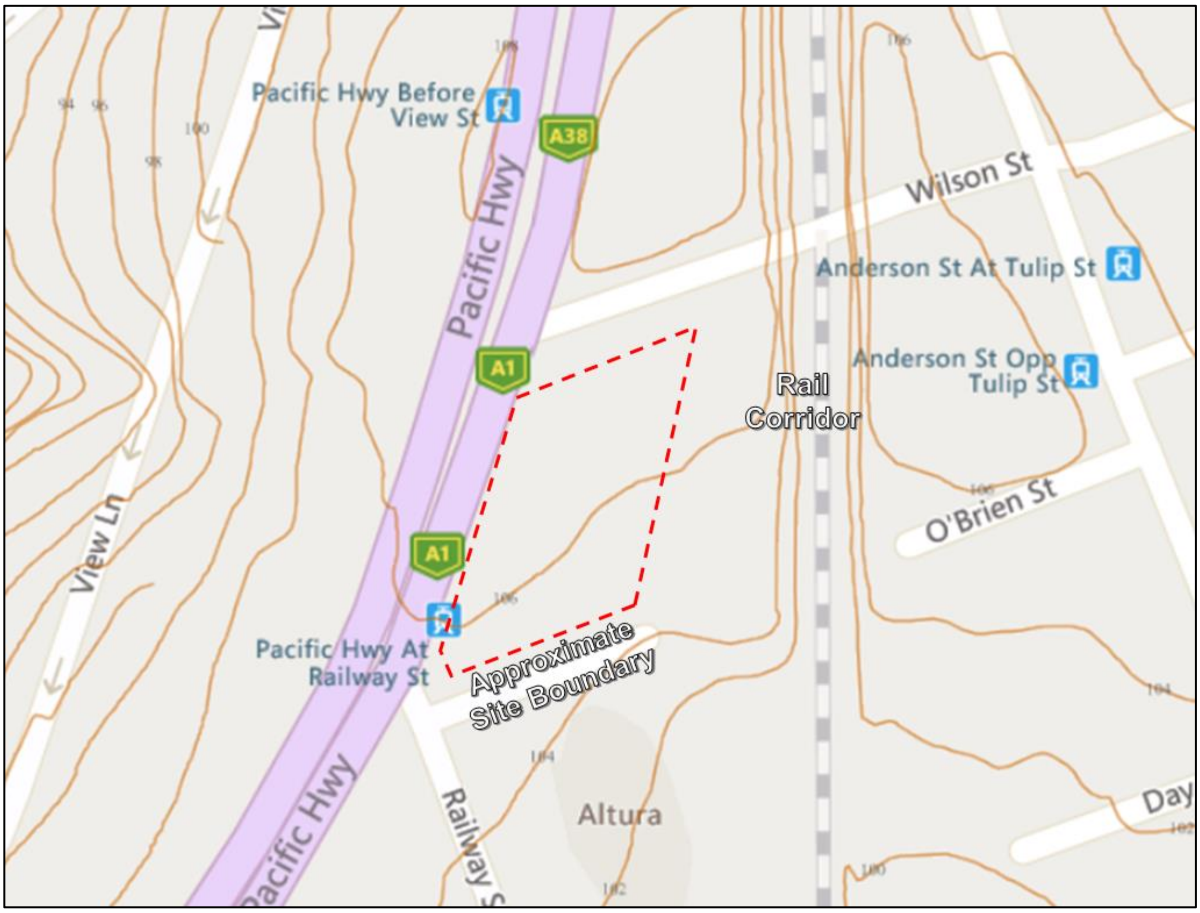

Figure 2: Site Topography

3. Published Data

3.1 Geology

Reference to the Sydney 1:100 000 Series Geological Sheet indicates that the site is underlain by Ashfield Shale of Triassic age. The Ashfield Shale, which typically comprises black to dark grey shale and laminite, is underlain by a transitional unit, the Mittagong Formation which typically includes interbedded fine-grained sandstones and siltstones.

3.2 Soil Landscape

According to the NSW Soil Landscapes Central Eastern NSW (v2) mapping, the site is within the Glenorie soil landscape where soils are predominately formed by erosion typically comprising red and yellow, moderately reactive clay soils. The landscape comprises undulating to rolling hills; a local relief of 50-80 m; slopes of 5-20%; narrow ridges, hillcrests and valleys; and extensively cleared tall open-forest. Soils are shallow to moderately deep (<1 m) on crests; moderately deep (0.7-1.5 m) on upper slopes; and deep (>2 m) along drainage lines. Dominant soil materials include: dark brown loam; brown clay loam; reddish-brown clay; mottled grey clay; and brownish-grey silty clay. Red ironstone nodules are often found in the deep subsoil.

3.3 Groundwater

A search of the publicly available Water NSW registered groundwater bore database revealed that there are two registered groundwater bores (GW114837 and GW114838) approximately 500 m to the north of the site. The bores were installed for monitoring purposes. GW114837 was drilled to a depth of 5 m and a standing water depth of 2.6 m was recorded. GW114838 was drilled to a depth of 9.7 m and a standing water depth of 5 m was recorded.

3.4 Acid Sulphate Soils

According to NSW Acid Sulphate Soil Risk data, the site is not in an area, or near an area, associated with a probability of occurrence of acid sulphate soils.

According to the Willoughby Local Environment Plan 2012 Acid Sulphate Soil Map, the site is in Class 5 land. For Class 5 land, development consent is required for the carrying out of works within 500 m of adjacent Class 1, 2, 3 or 4 land that is below 5 m AHD and by which the water table is likely to be lowered below 1 m AHD on adjacent Class 1, 2, 3 or 4 land. According to the Acid Sulphate Soil Map, there is no Class 1, 2, 3 or 4 land within 500 m of the site.

4. Previous Investigations

DP has access to and have carried out several geotechnical investigations near the site. The nearby Geotechnical Factual Reports (GFR) include:

- GFR for 871 – 877 Pacific Highway (Report no. 84722 dated April 2015), comprising 2 boreholes to a maximum depth of 8.25 m, confirming fill, clay, weathered shale and fresh shale at depth;
- GFR for 7 Railway Street, Chatswood (Pacific Place; Report no. 45679.01 dated April 2011), comprising ten boreholes to a maximum depth of 31.5 m and three cone penetration tests, confirming fill, clay, weathered shale and fresh shale at depth then Mittagong Formation and Hawkesbury Sandstone; and
- GFR for Anderson, Help and McIntosh Streets, Chatswood (Report no. 73524.00 dated July 2017), comprising 4 boreholes to a maximum depth of 15.2 m below surface, confirming fill, clay, weathered shale and laminite and fresh shale at depth.

5. Geotechnical Model

Generally, the interpreted geotechnical model for the site may be described as follows:

- **FILL**: Variable sand, clay and gravel fill with various inclusions to depths of between 0.5 m and 1.0 m; overlying;
- **RESIDUAL SOIL**: Stiff clay with high plasticity to depths of between 2.5 m and 3.0 m; overlying;
- **ASHFIELD SHALE**: Extremely low to very low strength, extremely to highly weathered, fractured shale to depths of 6.0 m; overlying very low and low strength, highly to moderately weathered, fractured and slightly fractured shale, to depths of 7 m; overlying, medium strength, fresh, slightly fractured shale becoming high strength from depths of 13 m. At depth, this unit may be underlain by;
- **MITTAGONG FORMATION:** comprising a sequence of fresh, fine grained sandstone, siltstone and laminite, generally of medium to high strength from depths of 20 m; underlain by; and
- **HAWKESBURY SANDSTONE:** fresh, medium and coarse-grained sandstone that is typically medium to high strength.

The soil profile and depth of fill may be variable in the north east portion of the site due to construction of the Wilson Street bridge and rail corridor.

Ephemeral, perched groundwater is likely to be present at the transition from residual soil into rock at a depth of about 3 m, and seepage may occur from along bedding planes within the rock mass. Inflow rates from both the residual clays and shale bedding planes are typically very low.

6. Proposed Development

The proposed development is understood to include:

- Demolition of the existing buildings on site and excavation and construction of a 28 level building with basement parking, retail/commercial use on ground and first floor and residential use above; and
- Dedication of the western fringe of the site for a future road widening along Pacific Highway.

Specific details of the proposed basement depth have not been finalised.

7. Comments

Preliminary comments on earthworks, excavation support, groundwater and foundations are provided in the following sections and will need to be reviewed once details of the proposed development have been confirmed and geotechnical investigations conducted.

7.1 Site Preparation and Earthworks

7.1.1 Excavation Conditions

Basement excavation to depths of 3 m to 6 m (typical of a 1 to 2 level basement) may require excavation of soils and extremely low to very low strength shale. Basement excavation to greater depths may require excavation of medium to high strength shale.

Excavation of soil and extremely low to low strength rock should be achievable using conventional earthmoving equipment, however, the assistance of rock hammering or ripping may be required to excavate any medium to high strength ironstone bands within the weathered rock profile. It is anticipated that excavation of medium to high strength rock will require hydraulic rock breakers in conjunction with heavy ripping for effective removal of this material. Rock sawing can be used to reduce over-break and vibration around the site.

7.1.2 Dilapidation Surveys

Dilapidation surveys should be carried out on surrounding buildings, bridges, rail infrastructure and pavements that may be affected by the basement excavation. The dilapidation surveys should be undertaken before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed.

7.1.3 Vibration

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard AS2670.2-1990 “Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)” indicates an acceptable day time limit of 8 mm/s PPVi for human comfort.

Based on the experience of DP and with reference to AS2670, it is suggested that a maximum PPVi of 8 mm/s (applicable at the floor level of existing buildings) be employed at this site for both architectural finishes and human comfort considerations, although this vibration limit may need to be reduced if there are heritage/sensitive buildings or equipment in the area.

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation. The trial may indicate that smaller or different types of excavation equipment should be used for bulk (or detailed) excavation purposes.

7.1.4 Disposal of Excavated Material

All excavated materials 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 fill and natural materials that may be removed from the site.

7.1.5 Unsupported Temporary Excavation Batters

Depending on the earthworks sequence and basement depth, batters may be required in some areas of the site, such as internal to the shoring wall in the north east portion of the site if temporary propping is required adjacent to the rail corridor. The following batter slopes (Table 1) are suggested for the design of temporary unsupported cuts of up to 4 m height.

Table 1: Recommended Maximum Safe Batter Slopes for Preliminary Design

Material	Recommended Temporary Batter
Fill / Residual Clay	1.5:1
Extremely low to very low strength shale – Class IV / V	1:1
Very low strength shale – Class IV	0.75:1
Low strength shale – Class III	0.5:1*
Medium strength shale – Class II (or better)	Vertical*

* Subject to jointing assessment by experienced Engineering Geologist

Regular inspections of the excavated faces should be undertaken by an experienced Engineering Geologist or Geotechnical Engineer at the time of excavation to assess whether localised stabilisation (e.g. rock bolting) is required to control any adversely oriented jointing within the face. Based on the significant amount of jointing and faulting with moderate and subvertical dips, noted in the rock core from nearby sites, it is expected that some dowelling/rock bolt support may be required to prevent the loss of adversely oriented wedges. If such wedges are not supported prior to excavation significant overbreak may result.

7.2 Excavation Support

7.2.1 Retaining/Shoring Walls

The fill, clay, and extremely low strength shale (shaly clay) encountered has a limited ability to stand unsupported for other than a short period of time. Therefore, some form of shoring will be required. In addition, it is likely that some positive support measures will also be needed because of predominant 45° joints common in the Ashfield Shale which could lead to large wedge failures if some form of anchoring is not adopted (Note, such 45° wedges were present in the adjacent Pacific Place excavation).

Temporary shoring could be designed to be incorporated within the permanent excavation support. Alternatively, the final structure may be used to prop or brace the retaining wall system in the longer term, so that temporary anchors may be detensioned.

Shoring support methods will generally require tie-back anchors for stability, particularly where limiting ground movements behind the wall is essential. The legal implications of the use of rock anchors extending onto neighbouring properties and public land will need to be considered. Approval should be sought from Council and adjacent property owners.

The following shoring options may be considered for the support of such excavations:

- **Contiguous Pile Wall** – Consisting of closely spaced, or almost touching, bored (or Continuous Flight Auger) and socketed reinforced concrete piles. The wall may form part of the final structure, sealed by a shotcrete panel facing that is constructed as the bulk excavation progresses. Multiple rows of ground anchors tied into waling beams are generally required; and

- **Soldier Pile/Infill Panel Wall System** – Consisting of bored or CFA, rock socketed piles installed on or behind the excavation line, at typically 2 – 3 m centres. As excavation proceeds, structurally reinforced shotcrete infill panels, or similar, are constructed in between the piles. The piles can often be designed to also provide foundation support for the perimeter of the structure. Piles are normally drilled with a minimum "toe in" design to provide lateral restraint at the base of the excavation based on the passive resistance of the rock in which the pile is socketed. Again, multiple rows of ground anchors are generally required.

Soldier piles in conjunction with reinforced shotcrete panels have been used on the other nearby development sites within Chatswood and are commonly used in Sydney for excavation support in cohesive soils overlying weak rocks. The exposed soil and rock profile in between the soldier piles is expected to be temporarily self-supporting for panel depths of up to about 2 m, until the ground anchors are installed and the reinforced infill panels are constructed. At no stage should progressive vertical excavation exceed about 2.5 m without infill panel support being constructed. A maximum depth increment of 1.0 – 1.5 m is recommended for excavation over the upper 2 – 5 m of the soil and extremely low strength rock profile. It is possible that adverse jointing may cause localised instability in the exposed material (e.g. unstable wedges), which may require remedial measures prior to shotcreting. Regular inspection of the excavated faces in between soldier piles should be carried out by an experienced Engineering Geologist/Geotechnical Engineer during excavation works to advise further on such stabilisation measures, as well as to record the orientation of the joints and faults.

Consideration of two complementary wall loading conditions will generally be required for the design of retaining structures in shale. The first condition, considering horizontal pressures due to the soils and weathered rock material, and the second, taking account of potential sliding along adversely oriented, continuous joints (refer Section 7.2.2).

7.2.2 Preliminary Lateral Earth Pressures for Design

The preliminary design of the shoring system may be based on an average bulk unit weight of 20 kN/m^3 and up to 24 kN/m^3 for soil and rock respectively, with a triangular earth pressure distribution based on lateral earth pressure coefficients as given in Table 2 below.

Table 2: Preliminary Design Parameters for Shoring Systems

Material Type	Unit Weight (kN/m^3)	Earth Pressure Coefficient	
		Active (K_a)	At Rest (K_o)
Fill and residual clay	20	0.30 (0.4)	(0.50)
Extremely low to very low strength shale – Class IV	22	0.25 (0.30)	(0.40)
Low strength shale – Class III	22	0.20 (0.25)	(0.35)
Medium strength shale – Class II	24	0 (0.1)*	(0.15)*
Medium and High strength rock	24	0.0*	0.0*

Note: * Subject to discontinuity assessment by experienced Geotechnical Engineer/Engineering Geologist.
Bolting and possibly shotcrete, will be required if the rock is adversely affected by defects.
() Permanent earth pressure coefficients shown in brackets

Areas where deflections of the wall are to be minimised and for the first metre below the adjacent footpaths/roads, should be designed for K_0 conditions (lateral earth pressure coefficient at rest).

Provision should be made where surcharging occurs either from adjacent building footings, traffic or other loadings. Unless positive drainage measures can be incorporated to prevent water pressure build-up behind the walls, full hydrostatic head should be allowed for in design while, at the same time, allowing for the soil unit weight to reduce to the buoyant condition.

The horizontal or lateral pressures acting on the wall can be calculated based on the following triangular earth pressure distribution:

$$H_z = K(\gamma z + p)$$

Where:

H_z = horizontal pressure at depth z

γ = unit weight of soil or rock

K = Earth pressure coefficient

z = depth (m)

p = vertical surcharge pressure

An anchored shoring system supporting the overburden and extremely low and low strength sandstone may be designed based on a rectangular earth pressure distribution of 8H (H = retained height in metres) where adjoining building foundations and sensitive services are present. Where no building foundations or services lie within the area of influence a uniform pressure of 4H could be adopted. These pressures assume horizontal ground surfaces behind the wall, no hydrostatic pressure and no surcharge pressure; these pressures should be allowed for in the final design.

Wall support piles may be designed to extend below the base of the excavation and develop passive resistance at the base of the piles. In estimating the minimum “toe-in” length, where this approach is adopted, the ultimate passive pressures shown in Table 3 below are suggested for preliminary design.

Table 3: Preliminary Design parameters for shoring piles

Material Description	Estimated Ultimate Passive Pressure (kPa)
Very low strength shale – Class IV	400
Low strength shale – Class III	1,500
Medium strength shale – Class II (or better)	5,000

It should be noted that the ultimate values shown above may be considered below one pile diameter beneath the bulk excavation level and will need to incorporate a factor of safety. Piles should also found at least two pile diameters below the bulk excavation level (or perimeter drainage trenches), except when required to carry structural loadings from the proposed building, where longer rock sockets will generally be required. Soldier piles supporting structural compression loads should be apportioned based on the allowable foundation pressures given in Section 7.5.

Shoring will also need to be designed to support earth pressures and surcharge loads and should also consider the possibility that 45 degree joints in the shale will daylight near the base of the shoring, which may result in rock wedges and lateral loads in excess of that normally allowed for in the shoring design.

For preliminary design of anchors, the maximum allowable bond stresses shown in Table 4 should be adopted. The parameters given in Table 4 assume that the drill holes are clean and adequately flushed. The anchors should be bonded behind a line drawn up at 45 degrees from the base of the shoring. Testing should be carried out to confirm the anchor capacities. It should be noted that permission may be required from adjacent property owners prior to installing bolts/anchors below their land.

Table 4: Ultimate Bond Stresses for Preliminary Anchor Design

Material Description	Ultimate Bond Stress (kPa)
Very stiff residual clays	75*
Extremely low to very low strength rock	150
Low strength rock	350
Low to medium strength rock	1200
Medium to High strength rock	2000

Note: * Applicable for soil nails where no clay smearing is present in the hole.

It is anticipated that the new structure will prop/support the shoring walls over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors, if required, would require careful attention to corrosion protection and further geotechnical advice should be sought.

Care should be exercised in construction to ensure that anchors are installed progressively during excavation and stressed prior to excavation of the next drop. It should be noted that stress relief (see Section 7.2.3) related movement may lead to an increase in the stress in anchors.

The final or detailed design of retaining walls is normally undertaken using interactive computer programs such as WALLAP or FLAC, which can take soil-structure interaction into account during the progressive stages of wall construction, anchoring and bulk excavation.

7.2.3 Excavation Induced Ground Movements

It is likely that horizontal movements due to stress relief will occur during the excavation works. Based on published literature and experience in the Sydney region, the lateral deflections could be in the order of 0.5 mm to 2 mm per metre depth of excavation in low to medium strength and stronger rock.

It is not practicable to provide restraint for the relatively high in-situ horizontal stresses associated with stress relief movements. Therefore, it is recommended that appropriate allowance be made for movements during planning and construction.

7.3 Acid Sulphate Soils

The site is in an area mapped as having no known occurrence of acid sulphate soils. The relatively shallow residual clay soils encountered in the previous boreholes on the site are also considered not to be consistent with acid sulphate soils.

7.4 Groundwater

During construction and in the long term, it is anticipated that seepage into the excavation should be readily controlled by perimeter drains connected to a "sump-and-pump" system. A drained basement will require permanent drainage below the basement floor slab to direct seepage to the stormwater drainage system. It is likely that iron oxides will precipitate from any such seepage, possibly leading to a build-up of an iron-oxide sludge. Allowance for periodic cleaning of such sludge should be made in the long-term maintenance requirements.

Tests previously conducted on groundwater samples on a nearby site in Chatswood indicate moderately acidic conditions. If similar groundwater conditions are present on the subject site, this could have an adverse effect on buried steel due to corrosion. Corrosion rates are unlikely to affect the strength of steel components during the construction phase, however, the design of permanent steel structural elements should be reviewed considering the moderately aggressive groundwater. Laboratory testing of groundwater for aggressivity should be conducted, preferably taken from groundwater monitoring wells, before finalising the design.

7.5 Foundations

It is presumed that basement excavations will expose low to medium strength or stronger rock and that pad footings will probably be suitable. However, this will depend on the depth of excavation and magnitude of the foundation loads.

Preliminary foundation design values for a range of foundation materials are presented in Table 5 which also includes parameters for clay and extremely low to low strength shale.

Table 5: Recommended Design Parameters & Moduli for Preliminary Foundation Design

<table>
<tr><th rowspan="2">Foundation Stratum</th><th rowspan="2">Classification[1]</th><th colspan="2">Maximum Allowable Pressure</th><th rowspan="2">Field Elastic Modulus (MPa)</th></tr>
<tr><th>End Bearing (kPa)</th><th>Shaft Adhesion[2] (kPa)</th></tr>
<tr><td>Firm Clay</td><td>Not Applicable</td><td>50</td><td>20</td><td>5</td></tr>
<tr><td rowspan="3">Ashfield Shale</td><td>Extremely Low Strength (Class V)</td><td>700</td><td>60</td><td>50</td></tr>
<tr><td>Very Low Strength (Class IV)</td><td>1,000</td><td>100</td><td>200</td></tr>
<tr><td>Low Strength (Class III)</td><td>2,000</td><td>200</td><td>500</td></tr>
</table>

Foundation Stratum	Classification[1]	Maximum Allowable Pressure		Field Elastic Modulus (MPa)
		End Bearing (kPa)	Shaft Adhesion[2] (kPa)	
	Medium Strength (Class II)	3,500	300	1,000
	Medium to High Strength (Class I)	6,000	500	2,000

Notes:

1 Classification based on Reference 1 (Pells et al. 1998).

2 Shaft adhesion applicable for the design of bored piers, uncased over rock socket length, where adequate sidewall cleanliness and roughness is achieved.

The foundation design parameters given in the above table assume that the foundation excavations (e.g. pads or piles) are clean and free of loose debris, with pile sockets free of smear and adequately rough immediately prior to concrete placement.

Foundations proportioned based on the above parameters would be expected to experience total settlements of less than 1% of the footing width (or pile diameter) under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

Heavily loaded footings should be founded below a line drawn up at 45 degrees from the toe of adjacent excavations, such as shallow pad footings in the proximity of lift shafts and service pits or alternatively a reduction of the allowable bearing pressure or shaft adhesion parameters may be appropriate. These parameters apply to the design of pad or strip (shallow) footings, and for socketed bored piles.

All footing excavations should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. Spoon testing/cored boreholes will be required for all foundations with allowable bearing pressures of 3,500 kPa or greater.

7.6 Design for Earthquake Loads

In accordance with AS1170 - 2007 “Structural Design Actions, Part 4 : Earthquake Actions in Australia” a hazard factor (Z) of 0.08 and a site subsoil Class Be is considered to be appropriate for the site assuming that all structural loads are uniformly founded on very low strength or better rock.

7.7 TfNSW Geotechnical Considerations

It is noted that the proposed development is located near the Railway Corridor and various roads and associated infrastructure and is likely to be subject to Transport for NSW (TfNSW) conditions. TfNSW Technical Direction GTD 2020/001, Version No. 01, dated 2 July 2020 sets out the requirements for TfNSW concurrence for developments with expected influence of temporary works affecting the road reserve and TfNSW easements. This technical direction supersedes RMS document GTD 2012/001.

As a general requirement, TfNSW will likely require accurate survey drawings and an assessment of likely ground movements that may occur within TfNSW property or the road reserve. The permissible movements on TfNSW assets should be clarified with TfNSW before the design, however, as a general rule for non-sensitive TfNSW assets, the total serviceability horizontal movement of the site's retaining wall in any one direction is to be limited to 0.5% of the excavated height, or 30 mm, or the structural serviceability requirements of the retaining wall, whichever is the lesser.

Instrumentation and monitoring may be required and will need to be carried out by a suitably qualified geotechnical professional which could include at least 2 inclinometers and survey stations. A monitoring plan will be required with a hold point release system based on agreed movement trigger levels.

It is recommended that the design team familiarise themselves with the TfNSW technical direction for future consultation with TfNSW.

7.8 Further Investigation

A site investigation will be required to assess the site's rock profile prior to detailed design and construction. As a guide, this may include at least 4 rock cored boreholes drilled to at least 3 m below the proposed bulk excavation level, along with the installation of groundwater monitoring wells (the appropriate scope will depend on the scale of the proposed development and TfNSW requirements). Further geotechnical review and advice will be required once these investigations have been completed and after specific details of the proposed development have been confirmed.

8. Limitations

Douglas Partners (DP) has prepared this report for this project at 2 Wilson Street, Chatswood in accordance with DP's proposal SYD200997.P.001.Rev1 dated 22 September 2020 and acceptance received from Michael Andrijic on behalf of 853 Pacific Highway Pty Ltd ATF 2017 PHC Unit Trust dated 22 September 2020. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of 853 Pacific Highway Pty Ltd ATF 2017 PHC Unit Trust for this project only and for the purposes as described in the report. It should not be used by or be relied upon for other projects or purposes on the same or another 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 this desk-top report are indicative of the expected sub-surface conditions on the site based on investigation of sites at some distance from the subject site. Sub-surface conditions can change abruptly both vertically and laterally due to variable geological processes and also as a result of human influences. .

DP's advice is based upon the conditions encountered during nearby investigations. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions between and beyond the sampling and/or testing locations

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

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

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be found, 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 a risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the (geotechnical / environmental / groundwater) components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Introduction

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

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

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

Borehole and Test Pit Logs

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

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

Groundwater

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

- 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.

About this Report

Site Anomalies

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

Information for Contractual Purposes

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

Site Inspection

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