Douglas Partners
Geotechnics | Environment | Groundwater

Report on
Geotechnical Desktop Assessment

Proposed Commercial Development
29-57 Christie Street, St Leonards NSW 2065

Prepared for
Arrow Capital Partners

Project 99528.00
January 2020

Integrated Practical Solutions

Douglas Partners
Geotechnics / Environment / Groundwater

Document History

Document details

Project No.	99528.00	Document No.	R.001.Rev1
Document title	Report on Geotechnical Desktop Assessment Proposed Commercial Development		
Site address	29-57 Christie Street, St Leonards NSW 2065		
Report prepared for	Arrow Capital Partners		
File name	99528.00.R.001.Rev1 Geotechnical Desktop Study		

Document status and review

Status	Prepared by	Reviewed by	Date issued
Revision 0	Slaiman Shirzai/Peter Hunt	John Braybrooke	20 December 2019
Revision 1	Peter Hunt	John Braybrooke	6 January 2020

Distribution of copies

Status	Electronic	Paper	Issued to
Revision 0	1	0	Stephen Day, Arrow Capital Partners
Revision 1	1	0	Stephen Day, Arrow Capital Partners

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.

	Signature	Date
Author		6 January 2020
Reviewer	pp	6 January 2020

ISO 9001 Quality Management
FS 604853

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Report on Geotechnical Desktop Assessment
Proposed Commercial Development
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1. Introduction

This report presents the results of a geotechnical desktop assessment carried out for a proposed commercial development at 29-57 Christie Street, St Leonards NSW 2065 (Lot 1 - DP 773862). The investigation was commissioned by Stephen Day of Arrow Capital Partners and was carried out in accordance with Douglas Partners' proposal SYD191135.P.001.Rev0 dated 18 November 2019.

We understand that a preliminary geotechnical desktop assessment is required to support Phase 1 of the project – 'Development Application (DA) period' for submission to Lane Cove Council. It is further understood that the concept design for DA comprises 3 buildings separated by two through-site links. Building A and B will be commercial buildings of 8 storeys and 16 storeys respectively. Building C will comprise the retention of the existing 7 storey, commercial building as a podium, which will accommodate a future tower above. It is also understood that the proposed development will extend the existing basement to provide 298 car parking spaces over four levels.

In the preparation of this report, findings and information from previous, nearby geotechnical investigations, as well as other available information (geological maps etc.) have been used. The report includes preliminary comments on expected ground conditions, groundwater and vibration. It also provides initial design parameters for foundations and shoring walls.

2. Site Description

The proposed development is located at 29 - 57 Christie Street, St Leonards (see Figure 1). The site occupies an area of approximately 7,636 m^2 and is bounded by Christie Street to the west, Nicholson Street to the east and Oxley Street to the south. The northern boundary of the site is bounded by the 3 storey Nature Care College with an unknown number of basement levels. The site is currently occupied by a 7-storey building with at least one basement level at the northern end and a two storey building at the southern end. The original topography of the site comprised a natural ground surface dipping gently to the west, with a level difference of approximately 8.0 m across the site, however the footprints of the two existing buildings were levelled out prior to their construction.

Figure 1. Location of site and lot boundary (Ref. sixmaps.nsw.gov.au)

3. Site Geology

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale of Triassic Age. The Ashfield Shale generally comprises black and dark-grey shale and laminite. The Ashfield Shale, part of the Wianamatta Group of rocks comprises four siltstone and laminite subgroups. The predominantly shaly rock is typically closely bedded and contains an orthogonal pair of steeply dipping (70° to 90°) joint sets typically striking NNE and ESE and spaced at 0.5 m to 5 m. Randomly oriented, 30° to 45° dipping slickensided joints are also ubiquitous.

The Ashfield Shale overlies the Mittagong Formation, which is a transitional unit between the Ashfield Shale and underlying Hawkesbury Sandstone. The Mittagong Formation typically comprises fine-grained quartz sandstone with interbeds of siltstone and laminite. Hawkesbury Sandstone is typically pale to mid-grey in colour, has massive and cross-bedded facies and strength properties typically in the medium to high strength range. The Hawkesbury Sandstone formation normally has near-horizontal bedding partings spaced from less than 1 m to well over 3 m in places, and is typically cut by the following two orthogonal sets of steeply dipping joints spaced between 1 m and 10 m:

- Set 1: Strike 020° to 035° / Dip 70° - 90° E and W
- Set 2: Strike 110° to 130° / Dip 70° - 90° N and S

A mapping extract from the Geological Series Sheet is presented in Figure 2.

Figure 2: Mapped geological units underlying and adjacent to the site

The acid sulphate soil risk map published by the Department of Conservation and Land Management indicates that the site is in area with no known occurrence of acid sulphate soils.

4. Previous Investigations

DP have carried out a number of investigations in the vicinity of the proposed project, included some within and adjacent to the subject site:

- **'St Leonards Square' (DP Ref. 10180.00)** – Geotechnical Investigation including rock cored boreholes to depths of approximately 5 m to 18 m with some boreholes on the subject site.
- **29 – 67 Christie Street, St Leonards (DP Ref. 85238.00)** – Geotechnical Desktop Study carried out across the existing site and a number of adjacent properties.
- **472 – 494 Pacific Highway, St Leonards (DP Ref. 44388.03)** – The site is located almost immediately opposite 29-57 Christie Street. The development comprised two high rise buildings sharing a seven level basement. DP carried out:
    - o The Preliminary Geotechnical Investigation;
    - o The Design Geotechnical Investigation comprising 6 boreholes to depths of 24 m below surface;
    - o Numerical Analysis of the impacts of the development;
    - o Geotechnical services during construction including face mapping, footing inspections and crane outrigger assessments; and

- o Prepared that Geotechnical Monitoring Plan.

- **496 – 520 Pacific Highway, St Leonards (DP Ref. 85818.00)** – DP provided geotechnical services during construction, including inclinometer readings, working platform assessments, anchor testing, and footing inspections.
- **69 Christie Street, St Leonards (DP Ref. 4340.00)** – Geotechnical Investigation for foundation design.

5. Geotechnical Model

The interpreted geotechnical profile for the site is summarised in Table 1 below. It is noted, however, that there is some variability across the site based on the information from the desktop study and further investigations are recommended to refine the profile.

Table 1: Interpreted Geotechnical Profile

Geological Unit	Layer	Description
1	Fill	Fill – Generally poorly compacted fill to depths of about 0.5 m to 1.0 m.
2	Residual Soil	Firm to hard, silty clay and shaley clay to depths of between 1.0 m and 4.0 m.
3	Shale	Siltstone/shale and laminite (Ashfield Shale); generally highly to moderately weathered, fractured and extremely low to very low strength to depths of between 3 to 5 m, then low to medium strength with some higher and lower strength bands to depths of about 5 m (in the west) to 10 m (in the east).
4	Sandstone	Sandstone; slightly weathered and fresh, slightly fractured, medium to high strength sandstone (Hawkesbury Sandstone). The upper sandstone at some locations may include more variable sandstone/laminite (Mittagong Formation).

The profile outlined in Table 1 is from the natural surface. A large proportion of the site has been excavated and levelled for the construction of the existing buildings. Where excavation of greater than 4 m depth has been carried out, rock is expected to be present at the surface. The shale is expected to be thicker on the eastern boundary of the site, with sandstone observed in the valley to the west of the site.

Groundwater seepage should be expected from along the contact of clayey soils and bedrock and is also likely to occur from fractured zones and joints within the rock. Higher seepage flows may also be encountered near the interface of shale and sandstone. Seepage flows are likely to increase following periods of extended wet weather.

6. Proposed Development

The proposed development is located at 29 - 57 Christie Street, St Leonards (see Figure 1). It is understood that the development will comprise 3 buildings separated by two through-site links (see Figure 3). Building A and B will be commercial buildings of 8 storeys and 16 storeys respectively. Building C will comprise the retention of the existing 7 storey, commercial building as a podium, which will accommodate a future tower above. It is understood that the proposed development will extend the existing basement to provide 298 car parking spaces over four levels.

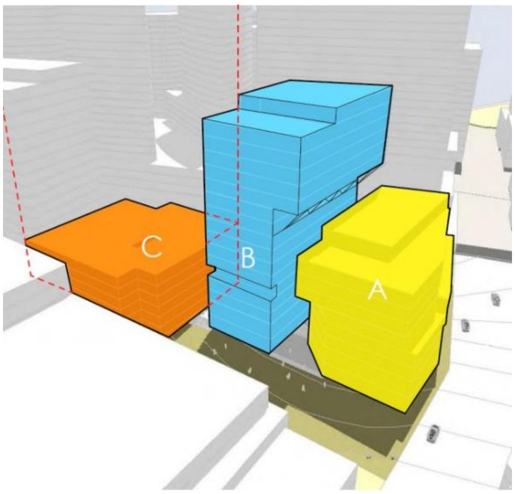

Figure 3: Concept Design (image supplied by Arrow Capital Partners)

7. Comments

7.1 Site Preparation and Earthworks

7.1.1 Excavation Conditions

Excavation is expected to encounter some sub-slab fill and natural clays (Units 1 and 2) followed by very low strength, then medium strength Shale (Unit 3), with the remainder of the excavation, if required, in medium to high strength sandstone and laminite (Unit 4). Where there is an existing basement or previous excavation works of greater than 4 m depth, rock is expected to be present below the sub-slab fill.

Excavation within the fill, clay and the extremely low strength rock (Units 1, 2 and partly 3) should be readily achieved using conventional earthmoving equipment such as hydraulic excavators with bucket attachments. Excavation of the stronger rock will largely depend on rock strength and discontinuity spacing and will require medium to heavy ripping or rock breaking equipment (hydraulic hammers) aided by rock sawing. Rock breaking equipment will generally cause noise and vibration that may be disturbing to people in adjacent or nearby buildings. Monitoring of ground-borne vibration and control of the rock hammering is likely to be required (see Section 7.6). Rock sawing will probably be required to assist excavation and potentially reduce noise/vibration.

Excavation of the medium to high strength shales and sandstone will cause some stress relief within the rock. This is discussed further in Section 7.2.4.

7.1.2 Disposal of Excavated Material

All surplus excavated materials will need to be disposed of in accordance with the Protection of the Environment Operations Act 1997 (POEO Act). All materials removed from the site are defined as waste under the POEO Act and must be disposed of in accordance with one of the following:

- Virgin Excavated Natural Materials (VENM) as defined under the POEO Act, permitting reuse on site; or,
- a waste category meeting the criteria set out in the NSW EPA Waste Classification Guidelines 2014, with the materials disposed to a landfill licenced to receive the waste under the assigned classification; or,
- material complying with a Resource Recovery Order (RRO) as defined under the Protection of the Environment Operations (Waste) Regulation 2014, with complying materials able to be reused under certain conditions.

7.1.3 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 not consistent with acid sulphate soils. On this basis, it is considered that the site is unlikely to be underlain by acid sulphate soils and therefore an acid sulphate soil management plan is not relevant for this site.

7.2 Excavation Support

7.2.1 Retaining/Shoring Walls

Excavation in soil and extremely low to medium strength shale and laminite will require both temporary and permanent lateral support. Support for this type of ground typically comprises anchored soldier piles with shotcrete infill panels. Excavation faces in medium strength or stronger sandstone/laminite are generally self-supporting and can be cut vertically without any support, provided there are no adversely oriented joints or other defects present.

Soldier pile shoring walls with shotcrete infill panels are often used for temporary support of soils and weaker rock. The soldier piles are usually spaced at approximately 1.8 m to 2.4 m centres, however, closer spaced piles may be required to reduce wall movements, or prevent collapse of granular infill materials, particularly where pavements, structures or services are located close to the excavation.

Preferably, shoring piles should be founded in medium strength or stronger laminite/sandstone below depths of about 5 m (west) to 10 m (east). Toes of piles terminating above bulk excavation level will need to be restrained with toe bolts or anchors. It is critical that the quality of the rock below the toe of each pile is confirmed prior to proceeding with excavation below the base of the piles. Additional support will be required if adversely dipping discontinuities are encountered. Alternatively, piles could be extended to below the bulk excavation level to eliminate the risk of adverse joints in the rock undermining the pile toe, however, this may require drilling through a significant thickness of medium to high strength rock.

All vertical faces in medium strength or stronger sandstone should be inspected by a geotechnical engineer at regular depth intervals to check for adversely inclined joints and to assess whether additional stabilisation measures (such as rock bolts or shotcrete) are required.

7.2.2 Preliminary Lateral Earth Pressures for Design

Preliminary design for lateral earth pressures for a multi-anchored wall system may be based on a trapezoidal earth pressure distribution. A uniform lateral earth pressure for the retaining wall of 4H kPa (H = Height (m) to be retained above the medium strength sandstone) or 6H (where lateral movements are to be limited) should be adopted.

The earth pressure loading described above does not include either earthquake loads or hydrostatic pressure due to the build-up of groundwater behind impermeable walls, both of which must be considered in the design. Unless positive drainage measures are 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.

Shoring will also need to be designed to support any surcharge loads and possible rock wedges due to jointing within the rock. It is recommended that as-built drawings of the neighbouring buildings be requested and that additional investigation is carried out to determine the exact founding level and founding conditions of their footings. It is further recommended that allowance is made for the possibility that 45 degree joints in the shale/laminite will daylight near the base of the shoring, which may result in lateral loads in excess of that normally allowed for in the shoring design.

Passive resistance for piles founded in rock below the base of bulk excavation may be based on a working passive restraint of at least 1000 kPa in medium or greater strength shale or sandstone not adversely affected by discontinuities. The first 0.5 m of rock socket below the bulk excavation level should not be taken into account for the purpose of passive restraint. The minimum socket depth should be equal to the greater of one pile diameter or 1.0 m below the lowest level of any nearby excavation (including any detailed excavations) unless determined otherwise by analysis.

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

7.2.3 Preliminary Anchor Design

It is anticipated that the new structure will support the shoring walls over the long term and therefore any anchors are expected to be temporary only. The use of permanent anchors would require careful attention to corrosion protection including full column grouting and the use of an internal corrugated sheath over the full length of the anchor. A detailed specification would need to be prepared for the installation and stressing of permanent anchors.

Anchors can be used to laterally support existing walls, new shoring or unstable rock masses. These support elements should be bonded in the stronger rock, inclined as required, but preferably not steeper than 30° below the horizontal. Table 2 provides allowable bond stresses for estimating purposes.

Table 2: Allowable Bond Stresses

Material	Allowable Bond Stress (kPa)
Very low to low strength shale	100
Medium strength shale/sandstone/laminite	350
Medium to high strength sandstone/laminite	600

Ultimately, it is the contractor's responsibility to ensure that the correct design values (specific to the support system and method of installation) are used and that the support element holes are carefully cleaned prior to grouting.

After support elements have been installed, it is recommended that they are tested to 125% of their nominal working load. Where stress relief or further unavoidable movement of the shoring is expected, it is recommended that the support elements are locked-off at between 60% and 80% of their working loads, as required to accommodate the additional movement and subsequent increase in stress in the support elements. Checks should be carried out to confirm that the load in the support elements has been maintained and that losses due to creep effects or to other causes have not occurred.

Shorter elements (rockbolts dowels and pins) may be required to support any unstable rock wedges, slivers or blocks. Short dowels and pins may be required to support feather edges where sub-parallel joints intersect the face. Shotcrete or mesh may be required where beds/seams of extremely or very low strength rock are encountered within higher strength sandstone, secured with anchors, rockbolts, dowels and pins, as required. Also all shale faces will need weathering protection with dowelled reinforced shotcrete to prevent long term slaking and ravelling.

Care should be exercised to ensure that anchors are installed progressively during excavation and stressed prior to excavation of the next drop to ensure that stability is maintained at all times.

It should be noted that permission will be required from adjacent property owners prior to installing rockbolts/ground anchors below their land. Also the possibility of below-ground tunnels, basements and services will need to be considered.

7.2.4 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 (Braybrooke, 1990).

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 at the planning and construction phases.

7.3 Groundwater

Groundwater seepage should be expected from the interface of clayey soils and bedrock, as well as from fractured zones and joints within the rock. Higher seepage flows may also be encountered near the interface of shale and sandstone.

Inflow into the basement is expected to be relatively minor but could increase after periods of heavy rain. 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.

At this stage it is not possible to accurately estimate the likely extent and rate of seepage, although it is anticipated that seepage rates will be relatively low (less than 3 ML/year) given the expected low permeability of the rock mass.

The disposal requirements of water collected on-site will be dependent on the chemical composition of the water and local council requirements. Normally, water is disposed of to a stormwater or sewer system subject to approval from the relevant government authority. It is noted that a drained basement will act as a low point to which groundwater will flow from the surrounding area.

Seepage water is likely to have red-brown iron oxide sludge precipitate from the water as it comes into contact with air. Perimeter and sub-floor slab drains should be designed for easy access to allow for inspection, maintenance (e.g. rodding) and periodic cleaning. Additional sumps should be provided to promote settling and removal of the iron oxide precipitate, prior to reaching the pumping system, for disposal.

7.4 Foundations

It is expected that bulk excavation for the basement will expose medium strength or stronger shale and/or medium to high strength sandstone/laminite.

Preliminary design of pad or strip footings on medium strength or stronger shale or sandstone/laminite may be carried out using the values given in Table 3. Higher allowable bearing pressures are possible for design purposes, however, additional investigation drilling within the site will be required to confirm the rock quality below the proposed founding levels.

Table 3: Recommended Design Parameters for Foundation Design

Foundation Stratum	Allowable End Bearing Pressure (kPa)	Ultimate End Bearing Pressure (kPa)	Young's Modulus, E (MPa)
Very stiff clay	200	400	30
Very low to low strength shale	1,000	8,000	100
Medium strength shale/ sandstone/laminite	3,500	30,000	700
Medium to high strength sandstone/laminite	6,000	60,000	1200

Note: Shaft adhesion applicable to the design of bored piles, uncased over the rock socket length, where adequate sidewall cleanliness and roughness are achieved

Foundations proportioned on the basis of the allowable bearing pressure in Table 3 would be expected to experience total settlements of less than 1% of the foundation width under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value where founded on the same material.

All footing excavations should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. Spoon testing should be carried out in at least one third of the footings which are designed for an allowable end bearing pressure of greater than 3500 kPa. Spoon testing generally involves drilling a 50 mm diameter hole below the base of the footing, to a depth of 1.5 times the footing width, followed by testing to check for the presence of weak/clay bands. If weak seams are detected, footings may need to be taken deeper to reach suitable foundation material.

7.5 Seismicity

A Hazard Factor (*Z*) of 0.08 would be appropriate for the development site in accordance with Australian Standard AS 1170.4 – 2007 *Structural design actions – Part 4: Earthquake actions in Australia*. The site sub-soil class is considered to be Class B_e based on the building being founded directly on rock.

Where excavation is near the boundary of the site, it is recommended that the building is not designed to rely on any passive resistance from the excavated rock faces surrounding the site due to the potential for the removal of this rock by future neighbouring developments.

7.6 Vibration

Ground-borne vibration will be generated during excavation and construction works, particularly when using hydraulic hammers. It will be necessary to use appropriate methods and equipment to keep ground-borne vibration within acceptable limits. The standards listed below are considered appropriate documents on which to base the management of ground vibration:

- German Standard DIN4150-3-1999 "Structural vibration – effects of vibrations on structures"; and,

- Australian Standard AS2670.2-1990 "Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)".

7.6.1 Provisional Allowed Vibration Limit

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 (up to 25 mm/s PPVi for concrete framed structures). 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 AS2670.2-1990, 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 human comfort considerations, although this vibration limit may need to be reduced if there are sensitive buildings or equipment in the area.

7.6.2 Excavation Plant

DP maintains a database of vibration trial results which can provide guidance for the selection of excavation plant. Trial data is dependent on-site conditions and equipment, hence actual vibration levels may differ from predictions and a specific trial is recommended at the commencement of rock excavation. The database suggests buffer distance ranges, such as those shown for selected plant in Table 4, which should be maintained between excavation plant and adjacent buildings. These estimates should be examined in relation to the distances between adjacent buildings and the proposed excavation footprint, in order to select suitable plant.

Table 4: Approximate Buffer Distances for Selected Plant (PAVL 8 mm/s VSPPV)

Excavation Plant	Distance from plant by which vibration would attenuate to the Provisional Allowed Limit	
	From DP trial maxima [1]	From DP trial averages
Rock Saw on Excavator [2,3]	1.1 m	0.6 m
Ripper on 20t Excavator	3.4 m	1.2 m
Rock Hammer < 500 kg operating weight	7.4 m	3.0 m
Rock Hammer 501 - 1000 kg operating	7.5 m	3.3 m
Rock Hammer 1001 - 2000 kg operating	12.4 m	5.4 m
Rock Hammer > 2000 kg operating weight	7.4 m	4.9 m

Note:
1. Smaller distances can generally be determined from individual trials, as indicated by those from trial averages;
2. Buffer distances for rock hammers may be reduced by prior saw cutting along, or parallel to, excavation boundaries; and
3. Loading effects from adjacent buildings may reduce vibration levels, to enable boundary saw cuts with few exceedances;

7.6.3 Building Condition Surveys

It is recommended that dilapidation surveys (structural condition surveys) of adjacent buildings, pavements and any major services be carried out before commencement of any excavation work and that the building foundation types and conditions be determined, where possible, to allow further assessment of the maximum acceptable vibration levels and to provide a record in the event of any damage claims. Follow up dilapidation surveys should also be carried out during and after excavation/construction.

7.7 Further Investigation

Although drilling has been conducted on the site, the levels and exact locations of the boreholes are unknown, with the site undergoing significant changes due to the construction of the existing structures. The proposed basement may also extend to below the previous boreholes.

Further investigation will be required to assess the rock profile on the site prior to detailed design and construction. As a guide, this may include five to six rock cored boreholes drilled to at least 3 m below the proposed bulk excavation level, along with the installation of two to three groundwater monitoring wells. 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. References

Braybrooke, J.C. (1990 a) – Some geotechnical phenomena related to high stresses in the Hawkesbury Sandstone. *24th Newcastle Symposium, 1990*.

Braybrooke, J.C. (1990 b) – Excavations in a high horizontal stress field. *Proc. 6th Cong. Int. Assoc. Eng. Geol. Amsterdam, Aug 1990*.

Pells et al (1998) "Foundations on Sandstone and Shale in the Sydney Region" Aust. Geomech Jrnl., Dec, 1998.

9. Limitations

Douglas Partners (DP) has prepared this report for this project at 29-57 Christie Street, St Leonards in accordance with DP's proposal dated 18 November 2019 and acceptance received from Stephen Day of Arrow Capital Partners. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Arrow Capital Partners for this project only and for the purposes as described in the report. It should not be used by or be relied upon for another project or other purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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

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

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires 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 components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

Introduction

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

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

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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.