Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Desktop Study

Proposed Commercial Development
2-8a Lee Street, Haymarket

Prepared for
Toga Pty Ltd

Project 86884.00
February 2020

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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1. Introduction

This report presents the results of a geotechnical desktop study undertaken for a proposed multistorey building re-development located at 2-8a Lee Street, Haymarket (the site). The assessment was commissioned by Toga Pty Ltd (the client) and was undertaken in accordance with Douglas Partners Pty Ltd (DP) proposal SYD190732.P.001.Rev0 dated 29 July 2019.

The aim of the geotechnical desktop assessment is to indicate geotechnical constraints to the re-development of the site into a multi-storey commercial development to provide preliminary advice on design and construction issues. The assessment was undertaken using available published information and knowledge from previous DP projects surrounding the site. Intrusive investigation will be required to confirm subsurface conditions and to provide information for detailed design.

DP has also prepared a Preliminary Site investigation (PSI) report for the site, reference 86864.01.R.001.Rev0. This geotechnical desktop report should be read in conjunction with the PSI report.

2. Site Description

2.1 Site Identification

The site is identified as Lot 13, Deposited Plan 1062447 (8A Lee Street, Haymarket) and Lot 30 D.P877478 (2 Lee Street Haymarket) within the local government area of City of Sydney. The site is irregular in shape and has an approximate area of 0.5 ha, the general layout of which is provided on Drawing 1, Appendix B.

Based on the Section 10.7 Planning certificates, the site is zoned as B8 Metropolitan Centre.

2.2 Site Description

A DP environmental scientist inspected the site on 7 August 2019. At the time of DP's presence on site Adina hotel along with the Henry Dean Plaza occupied the site. It is understood that Adina hotel has a single level basement.

Based on a preliminary inspection:

- The external brickwork of the commercial buildings on the site appeared to be in a relatively good condition; and

- The pavements within the site appeared to be in a fair condition with some signs of minor cracking evident.

The site is situated within an area developed for a variety of uses. A summary of the current land uses adjacent to the location of the proposed building at the time of DP's presence on site is provided in Table 1.

Table 1: Summary of Adjacent Land Use

Direction Relative to the Site	Land Use Description
North	Ambulance Avenue followed by Railway Colonnade Drive which provides driveway access to Central Station.
East	Central Station which adjoins the eastern site boundary. It is noted that an underground pedestrian tunnel which crosses central station extends through the approximate central part of the site in an east-west direction, refer to Drawing 1. The extent and depth of this tunnel should be confirmed prior to final design. It is noted that the eastern site boundary is adjoining the Sydney Trains Rail corridor with the nearest train track having a setback of approximately 15 m from the eastern site boundary.
South	Six to eight-storey commercial building with possible below-ground parking adjoining the southern site boundary. During DP's presence on site, access to these buildings were not possible. The extent and depth of these nearby basement car parks should be confirmed prior to final design.
West	Lee Street followed by a Railway Square and George Street. Lee Street and George Street are both a Roads and Maritime Services (RMS) asset.

3. Regional Geology and Hydrogeology

3.1 Geology

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale which typically comprises black to dark grey shale and laminite (interlaminated siltstones and sandstones). However, the areas to the north and east of the site are underlain by Quaternary dune sand deposits and the area to the west is underlain by Hawkesbury sandstone. An extract of the mapping is shown on Figure 1 below. Previous investigation on and near the site however have severally encountered shallow to deep fill over residual soils underlain by sandstone with some shale beds and also an alluvial channel comprising sandy soils.

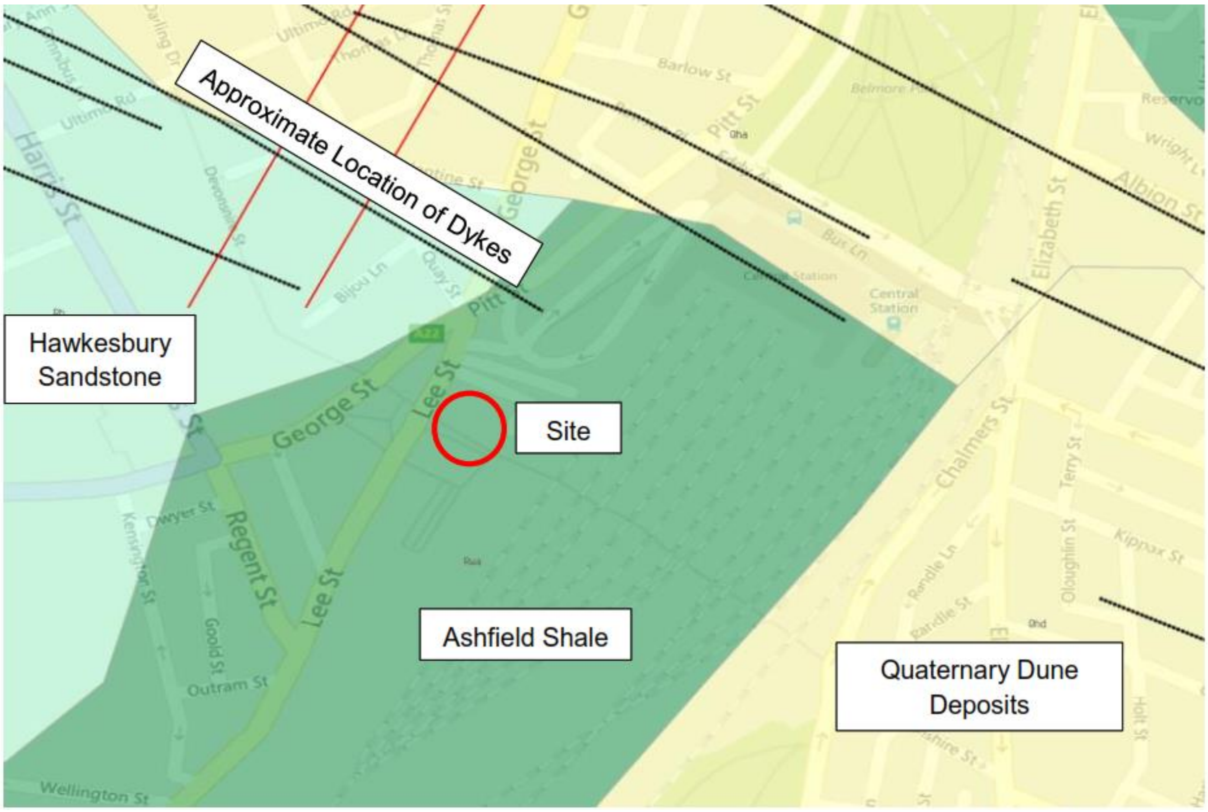


Figure 1 – Site Geology

4. Possible Geotechnical Conditions

DP has carried out several investigations within a 100 m radius of the site. Based on the results of nearby investigations, observations made during the site inspection and DP's general understanding of the geology in Haymarket, the anticipated sequence of subsurface materials likely to be encountered at the site, in increasing depth order, is summarised in Table 2.

Table 2: Summary of the Anticipated Subsurface Ground Profile

Material	Anticipated Depth Range to Top of Material (m)	Anticipated Thickness (m)	General Description
Filling (Sand and Clay)	0	0.2 to 8	Based on previous investigations near the site, filling is estimated to be typically present to depths of about 0.2 m near the northern site boundary and deepening to about 8.0 m near the southern site boundary, i.e. away from the Adina hotel and towards the Henry Dean Plaza. The previous borehole logs indicate the filling to be of a variable nature and include both sand and clay layers.
Sand	0.6 to 6.2	1.5 to 3.5	Typically very loose to loose fine to medium grained sand and clayey sand. It is noted that this 'pure' natural sand / clayey sand layer was not observed uniformly across the site and was only observed in a few of the previously drilled boreholes.
Silty Clay / Sandy Clay	0.2 to 9.6	0.5 to 3.5	Typically very stiff to hard residual clay observed above the sandstone bedrock.
Sandstone	1.0 to 13	(1)	Typically very low and low strength with medium strength bands, becoming medium and high strength with increasing depth.

Notes:
(1) Likely to be present below depths of about 1 m to 13 m below existing surface levels and below the proposed bulk excavation level across parts of the site

Groundwater measurements in previous nearby investigations completed by DP indicated that groundwater is likely to be encountered below depths of about 2 m to 8 m below the existing surface level (typical at RL 15 m to RL 12 m). Some of the previously drilled boreholes encountered groundwater within the fill / sand and some boreholes encountered groundwater within or close to the residual clay and rock interface.

It should be noted that groundwater levels are transient and that fluctuations may occur in response to climatic and seasonal conditions. Ongoing monitoring of water levels should be carried out to assess likely fluctuations for basement design.

5. Proposed Development

Based on the preliminary information supplied by the client it is understood that the proposed development will include the demolition of some of the existing structures and construction of a multi-storey commercial development over a three-level basement. At the time of writing this report the basement extent and building location had not yet been finalised. It is understood that the existing Adina Hotel will be incorporated into the development. Excavation for the basement is anticipated to extend to depths of about 9 m below existing surface levels. Locally deeper excavation may be required for service trenches and lift cores.

6. Geotechnical Considerations

The site is considered suitable for the proposed development from a geotechnical perspective. Intrusive investigation will be required to confirm the subsurface conditions and to provide information for detailed design.

6.1 Geotechnical Issues

Some of the primary geotechnical issues that need to be considered for development are:

- Groundwater is likely to be present and dewatering will be required for construction of basements;
- Excavation induced movement adjacent to Lee Street which is a RMS asset;
- Excavation induced movement adjacent to the eastern site boundary which is a Sydney Trains Rail corridor;
- Maintaining the stability of adjoining structures, in particular the Adina hotel during construction;
- Maintaining the stability of the existing pedestrian tunnel that extends through the site during construction;
- Shoring walls will need to be designed to reduce groundwater inflow and to control drawdown of water levels on adjacent sites as this has the potential to cause settlement;
- The shoring will need to be socketed into competent rock which can be problematic for some shoring systems and can result in decompression and loosening of the surrounding soils;
- If cut-off walls into rock are successfully constructed to reduce inflow and drawdown of water levels then it is technically feasible to construct a drained basement. This however will be subject to review and approval by both the Council and by Water NSW;
- Alternatively, a tanked basement could be constructed to reduce the need for long term collection, possible treatment and removal of groundwater inflows. A tanked basement will need to be designed for hydrostatic uplift.

6.2 Dilapidation Surveys

Dilapidation surveys should be carried out on adjacent / existing buildings, pavements and infrastructure that may be affected by the excavation works.

6.3 Excavation Conditions

Excavation in fill, sand and extremely low to very low strength rock should be readily achieved using of conventional earthmoving equipment, particularly if fitted with 'rock teeth'. Excavation in low strength (or stronger) rock will probably require the use of rock hammers and / or rock saws for effective removal.

6.4 Vibrations

Excavation in fill, sand and extremely low to very low strength rock should be readily achieved using of conventional earthmoving equipment, particularly if fitted with 'rock teeth'. Excavation in low strength (or stronger) rock will probably require the use of rock hammers and / or rock saws for effective removal.

6.5 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.

6.6 Dewatering and Tanking

6.6.1 General and Seepage Rates.

The proposed bulk excavation will likely extend below the groundwater table. If dewatering on the site results in excessive drawdown (lowering of the water level) beneath surrounding sites, then this has the potential to induce settlement. Existing groundwater contamination on the site, if applicable, should also be considered in the planning.

Provided that the depth of bedrock is shown (through detailed investigation works) to be encountered either above the proposed basement level or not at significant depths beneath it, it is anticipated that the most cost effective basement construction would comprise the construction of a relatively water tight perimeter 'cut-off' wall extended to base of the excavation and socketed at least 2 m into competent slightly weathered to fresh, slightly fractured and unbroken, medium to high strength bedrock in order to construct a fully tanked basement structure. This option would be expected to significantly reduce seepage flows as seepage will only occur though the relatively low permeability medium to high strength rock below the basement floor. This option may effectively reduce inflow rates into the basement to the extent that a drained basement may be justified without significant impact on groundwater levels on surrounding sites.

Further detailed investigations and groundwater modelling would be required to predict seepage rates and drawdown in the short and long term. This would also be required to assess whether a cut-off wall into rock below the bulk excavation may be used to allow a drained basement. However, a drained basement will be subject to review and approval by Council and by Water NSW.

If a drained basement slab is not possible then a water-tight 'tanked' basement will be required for the permanent basement structure. A tanked basement would need to be designed to resist uplift forces associated with (hydrostatic) groundwater pressures.

6.6.2 Drawdown and Settlement

It is suggested that the design and construction of the basement should be carried out to target a drawdown on adjacent properties of less than 1.5 m. As a minimum this will require perimeter cut-off walls into rock, and possibly installed into rock below the bulk excavation level to cut off horizontal flows through rock into the excavation. Further modelling may indicate that a tanked basement is required to reduce long term drawdown to acceptable levels.

6.6.3 Groundwater Disposal

The groundwater removed from the site will require disposal. A dewatering management plan (which includes a groundwater quality assessment) may be required.

6.7 Excavation Support

Shoring will be required around the perimeter of the site. It may be necessary or beneficial to install cut off walls into rock below bulk excavation level, however this will be subject to a detailed investigation of the groundwater and rock quality.

6.7.1 Retaining Wall Systems

The final basement structure should incorporate a watertight retaining wall system around the basement perimeter if sandy soils (both natural or fill) are present within proposed basement excavated and are below the groundwater table.

The following options may be considered:

- Diaphragm walls may be used as the permanent basement wall. These walls are associated with lower risk but are relatively slow to construct and consequently more expensive. Diaphragm walls are constructed using a large grab, which excavates the soil and rock in panels which are supported by bentonite fluid. Each panel is then cast using concrete tremmied into the bentonite supported excavation, with reinforcement cages installed prior to the concrete being tremmied. The joints between the panels are sealed with a waterstop so that a completely water-tight wall is achieved. This option is probably not warranted for this site.
- Interlocking secant pile wall (temporary and permanent) – secant pile walls are typically formed by drilling alternate 'soft' grout or concrete piles and then installing 'hard' reinforced concrete piles by cutting into the previously drilled soft piles. This overlap typically ensures that piles are sealed, but even at relatively shallow depths, some misalignment can occur and hence minor gaps appear in the wall. The potential for misalignment and therefore seepage and sand loss through gaps in

deep secant pile walls is very high. Drilling of piles into rock will also be problematic for secant piles and may result in decompression of the surrounding sands which can result in damage to adjacent buildings. The use of segmental casing would be required to avoid issues associated with decompression.

- Deep soil mix (DSM) or cutter soil mix (CSM) wall (temporary) – DSM/CSM walls involve blending or mixing of grout with the site soils in situ to form cement stabilised soil panels with universal column sections "plunged" into the "wet" panel at regular intervals along the wall to provide bending stiffness. However, experience with the DSM/CSM walls has indicated that the mixing consistency, and consequently the permeability and durability of the wall need to be carefully considered, particularly within clayey soils and rock. In addition, the construction of these walls become significantly more difficult within deep variable fill. This option is unlikely to be suitable at the site and may not achieve an effective seal at the rock interface

Should the intrusive geotechnical and groundwater investigation indicate that sandy soils within the basement excavation are above the groundwater table then a contiguous pile wall may also be feasible. However, this will have to be confirmed at a later stage following the intrusive works.

6.7.2 Retaining Wall Design

The shoring will need to be supported by internal bracing and / or ground anchors to control deflections. It is noted that Sydney Trains do not allow any anchors (temporary or permanent) within their corridor and as such internal bracing / props will likely (depending on the final basement configuration) be required along the eastern site boundary.

Preferably, shoring walls should be founded in rock at least 1.0 m below the bulk excavation level (possibly deeper to reduce water inflow) in order to provide lateral restraint at the base of the excavation and to avoid the risk of adversely inclined joints or wedges undermining the base of the shoring.

The preliminary design of shoring systems with one row of anchors may be based on the earth pressure coefficients provided in Table 3. 'Active' earth pressure coefficient (K_a) values may be used where some wall movement is acceptable, and 'at rest' earth pressure (K_o) values should be used where the wall movement needs to be reduced.

Table 3: Preliminary Design Parameters for Shoring Systems

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient		Effective Cohesion c' (kPa)	Effective Friction Angle (Degrees)
		Active (Ka)	At Rest (Ko)		
Filling	19	0.3	0.5	0	28
Very loose to loose sand	18	0.35	0.5	0	28
Very Stiff to Hard Clay	20	0.25	0.4	5	25
Extremely low to low strength sandstone	22	0.1	0.15	100	25
Medium strength or stronger sandstone	24	0*	0*	300	40

Note * subject to geotechnical inspection

The design for lateral earth pressures where multiple rows of anchors or propping (i.e. two rows or more) may be based on a trapezoidal earth pressure distribution. The following earth pressure magnitudes are considered appropriate, where H is the height of soil and rock to be retained, in metres:

- 4H kPa, where some lateral movement is allowed; and
- 6H kPa, where lateral movements need to be minimised (e.g. next to buildings and services).

In each case the maximum pressure generally acts over the central 60% of the wall height, reducing to zero at the top and base of the wall.

Passive resistance for shoring founded in rock below the base of the bulk excavation (including allowance for services or footings) may be based on the ultimate passive restraint values provided in Table 4. These ultimate values represent the pressure mobilised at high displacements and therefore it will be necessary to incorporate a factor of safety of say 2 or more to limit wall movement. The top 0.5 m of the socket should be ignored due to possible disturbance and over-excavation.

Table 4: Preliminary Passive Resistance Values

Foundation Stratum	Ultimate Passive Pressure (kPa)
Very low to low strength sandstone	2,000
Medium strength or stronger sandstone	4,000

Detailed design of shoring should preferably be carried out using WALLAP, PLAXIS or other accepted computer analysis programs capable of modelling progressive excavation and anchoring, and predicting potential lateral movements, stresses and bending moments. PLAXIS (or similar) would be required if it is necessary to assess ground movements on surrounding properties (e.g. Lee Street and Sydney Trains Rail Corridor / Tracks) as WALLAP will only assess wall movements.

6.7.3 Ground Anchors

For estimation purposes the design of temporary ground anchors for the support of shoring systems may be carried out on the basis of the maximum bond stresses given in Table 5. The anchors should preferably have their bond length within the low and medium strength and stronger rock.

Table 5: Preliminary Bond Stresses for Rock Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)	Maximum Ultimate Bond Stress (kPa)
Very low strength sandstone	100	200
Low to medium strength sandstone	200	400
Medium strength or stronger sandstone	500	1000

It will be necessary to obtain permission from neighbouring landowners prior to installing anchors that will extend beyond the perimeter of the site. In addition, care should be taken to avoid damaging buried services and pipes, and possibly neighbouring piled footings, during anchor installation. Anchoring should only be carried out by an experienced contractor with demonstrated experience in similar ground conditions.

6.8 Excavation Induced Ground Movement

6.8.1 RMS Infrastructure and Sydney Trains Rail Corridor

Lee Street and George Street is a Roads and Maritime Services (RMS) owned asset. Reference should be made to the *RMS Geotechnical Technical Direction 2012/001 dated April 2012*, which outlines requirements for excavations adjacent to RMS infrastructure and includes the level of geotechnical investigation required, dilapidation surveying, instrumentation and monitoring during construction, trigger levels and contingency plans.

A Geotechnical Impact Assessment (GIA), i.e. numerical modelling, will typically be required as part of the DA application (imposed by RMS and Sydney Trains). The purpose of the GIA is to assess the likely amount of excavation induced ground movement as a result of proposed excavation.

During construction, instrumentation (e.g. inclinometers) and survey monitoring is typically required where the excavation exceeds 3 m in height (for cantilevered shoring walls) or 6 m in height (for anchored or propped shoring walls). A geotechnical monitoring plan will also be typically required by RMS prior to construction for this site.

Depending on the setback of the basement excavation from the Sydney Trains Rail corridor, a site-specific track monitoring plan may also be required. It should be noted that this will likely involve the placement of survey markers within the rail corridor and on the nearest track which has its own complications regarding the delays / costs associated in obtaining the necessary approvals from Sydney Trains.

6.9 Foundations

Depending on the final design bulk excavation level, it is anticipated that variable founding conditions, ranging from filling, clay, very low to low strength sandstone, and medium to high and high strength sandstone is likely be encountered at the bulk excavation level. The new buildings should be uniformly founded on bedrock.

Pad footings and piles may be designed using the preliminary maximum pressures for the various rock strata presented in Table 6. Shaft adhesion values for uplift (tension) may be taken as being equal to 70% of the values for compression.

Table 6: Preliminary Design Parameters for Foundation Design

Foundation Stratum	Maximum Allowable Pressure		Maximum Ultimate Pressure	
	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)
Extremely low to very low strength sandstone	700	50	3,000	150
Low to medium strength sandstone	2,000	150	5,000	400
Medium strength sandstone	3,500	350	20,000	600
Medium to high strength or stronger sandstone	6,000	600	40,000	1,200

Serviceability limit-state is likely to govern the design and the ultimate bearing pressures provided in Table 6 will probably need to be lowered in order to limit settlements to an acceptable level. An appropriate geotechnical strength reduction factor should be applied when using the limit-state approach.

Foundations proportioned on the basis of the allowable bearing pressures in Table 6 would be expected to experience total settlements of less than 1% of the footing width / pile diameter under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

All footings 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 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 then footings may need to be taken deeper to reach suitable foundation material.

6.10 Seismic Design

In accordance with the Earthquake Loading Standard, AS1170.4, 2007, a hazard factor (z) of 0.08 and a site sub-soil class of either Class D_e or C_e, which is dependent on the strength of the materials underlying the site. This will have to be confirmed at a later date following the intrusive investigation.

7. Further Geotechnical Input

Below is a summary of the recommended additional works that should be carried out:

- Intrusive geotechnical investigation comprising a minimum of four to six cored boreholes drilled to at least 3 m below the proposed bulk excavation level and 3 m into medium strength sandstone or stronger (whichever is deeper) in order to confirm subsurface conditions and to provide information for detailed design. It is noted that the total number of boreholes required will be dependent on the required foundation design parameters (e.g. if high performance footings of 6,000 kPa or more are required) and final basement layout;
- Installation of monitoring wells across the site to confirm the depth to groundwater within the basement excavation;
- Completion of groundwater analysis to assess the feasibility of a drained basement at the site;
- Numerical modelling of the shoring wall adjacent to Lee Street (RMS asset) and eastern site boundary (Sydney Trains Rail corridor) to assess the likely amount of excavation induced ground movement as a result of proposed excavation. It is noted that both RMS and Sydney Trains will typically require this as part of the DA application;
- Preparation of geotechnical monitoring plan (Lee Street for RMS) and track monitoring plan (eastern site boundary for Sydney Trains). It is noted that both RMS and Sydney Trains will typically require this as part of the DA application;
- Instrumentation (inclinometers and survey markers) during construction to monitor excavation induced movements and confirm that they are within approved /tolerable limits as specified in the geotechnical monitoring plan and track monitoring plan;
- Dilapidation surveys;
- Waste Classification of all material to be excavated and transported off site; and
- Footing inspections during construction.

It is recommended that a meeting be held after the initial design has been completed to confirm that these recommendations have been interpreted correctly.

8. Conclusion

This report has discussed various geotechnical aspects of the proposed development and has outlined appropriate construction methods, monitoring requirements, and design parameters. Similar basements have been constructed in Sydney without significant impacts to surrounding properties. It is considered that once a detailed intrusive geotechnical investigation is completed at the site, the basement could be designed and constructed without significant adverse impacts to surrounding properties.

9. Limitations

Douglas Partners (DP) has prepared this report for this project at 2-8a Lee Street, Haymarket (the site) in accordance with DP's proposal SYD190732.P.001.Rev0 dated 29 July 2019 and acceptance received from Toga Pty Ltd (the client) on 30 July 2019. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Toga Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

DP's advice is based upon the conditions encountered during previous investigations completed by DP near the site. The geological model provided in the report is only indicative of the anticipated sub-surface conditions at the site. Sub-surface conditions can change abruptly due to variable geological processes and as a result of human influences, particularly as some of DP's field testing nearby was undertaken many years ago.

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

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

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

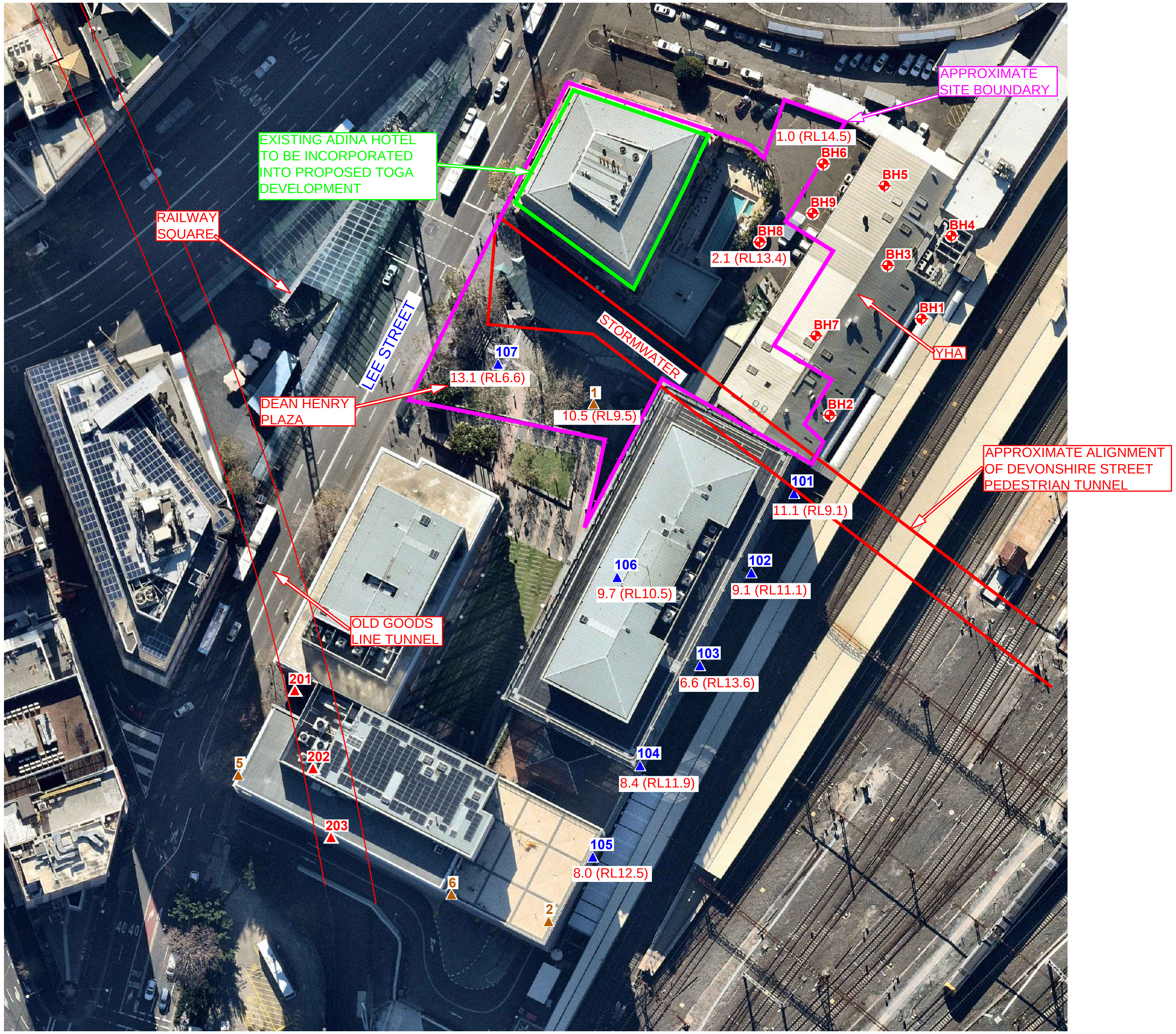
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

Appendix B

Site Plan



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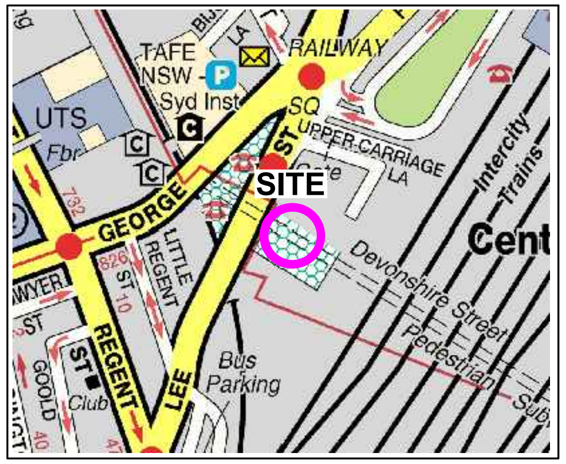

Locality Plan

LEGEND

PREVIOUS NEARBY INVESTIGATIONS

- Current borehole location (July 2019, Ref. 86767.00)
- Borehole location (1999, Ref. 27282A)
- Borehole location (1999, Ref. 27282B)
- Borehole location (1998, Ref. 27282)
- CPT location (1998)

8.0 (RL12.5) — Depth to top of rock (m); Reduced level of top of rock (m, AHD)

NOTE:
1: Base image from Nearmap.com (Dated 1.7.2019)
2: Test locations are approximate only and are shown with reference to existing features.

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Toga Development and Construction Pty Ltd		TITLE: **Site Plan**	PROJECT No: 86884.00
OFFICE: Sydney	DRAWN BY: PSCH	**Proposed Commercial Development**	DRAWING No: 1
SCALE: 1:1000 @ A3	DATE: 9.8.2019	**2 & 8A Lee Street, HAYMARKET**	REVISION: 0