Attachment A18

Report on Desktop Geotechnical Assessment

Douglas PARTNERS | GROUNDED EXPERTISE

Report on Desktop Geotechnical Assessment

Park Royal Hotel Redevelopment

150 Day Street, Sydney, NSW

Prepared for UOL Group Limited

Project 231572.00

19 March 2025

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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.

Signature		Date
Author		19 March 2025
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Douglas Partners acknowledges Australia's First Peoples as the Traditional Owners of the Land and Sea on which we operate. We pay our respects to Elders past and present and to all Aboriginal and Torres Strait Islander peoples across the many communities in which we live, visit and work. We recognise and respect their ongoing cultural and spiritual connection to Country.

Douglas PARTNERS | GROUNDED EXPERTISE

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Report on Desktop Geotechnical Assessment
Park Royal Hotel Redevelopment
150 Day Street, Sydney, NSW

1. Introduction

This report prepared by Douglas Partners Pty Ltd (Douglas) presents the results of a desktop geotechnical assessment undertaken for the proposed Park Royal Hotel redevelopment at 150 Day Street, Sydney, NSW (the site). The assessment was undertaken in accordance with Douglas' proposal 231572.00.P.001.Rev0 dated 2 September 2024.

It is understood that the proposed redevelopment includes extending the existing 10-storey building with an additional 11 storeys plus plant, whilst maintaining similar setbacks to the existing structure. The proposed redevelopment aims to retain as much of the existing structure as possible. As part of the redevelopment, loads on existing footings will increase and may be double at some locations.

The purpose of this desktop assessment is to provide preliminary information on subsurface soil, rock and groundwater conditions to support the submission of a Planning Proposal, and to provide preliminary information for the design of the proposed redevelopment.

The assessment involved a review of previous investigations undertaken by Douglas at the site during the development of Day Street Hotel (now Park Royal Hotel), in 1988. A summary of findings from neighbouring sites is also provided in this report, together with preliminary comments relating to design and construction practice.

This report must be read in conjunction with all appendices including the notes provided in Appendix B.

2. Site Description

The site is located at 150 Day Street within the Council of the City of Sydney local government area. The site is comprised of one allotment, legally described as Lot 20 in DP 1046870. The existing development comprises a 11-storey hotel building with a 2-level basement below the building footprint.

The site lot has a combined area of approximately 2,250 m^2 and is irregular in shape, bounded by Day Street to the west, Bathurst Street to the south, and Sands Street to the east. Additionally, located to the north of the site is the Druitt Street Western Distributor on-ramp, and the exit of the westbound Cross City Tunnel. The south of the site is further enclosed by the eastbound Cross City Tunnel under Bathurst Street. The existing building has frontages of approximately 70 m to Day Street and 18 m to Bathurst Street.

The site topography is relatively flat, located at approximately 5 m AHD. Surface levels across the site fall away from the south-eastern corner with a maximum fall of around 3 m towards the north-west, in the direction of Darling Harbour.

A site aerial plan is provided in Figure 1, detailing the site location.

Figure 1: Location of Subject site

3. Published data

3.1 Geological landscape

Reference to the Sydney 1:100 000 Geological Map Sheet indicates that the site is underlain by Hawkesbury Sandstone, a Triassic age medium to coarse grained quartz sandstone unit with minor shale and laminite bands or lenses. Rock is anticipated to be at a shallow depth.

In its fresh condition, Hawkesbury Sandstone is typically light to mid grey in colour, has massive and cross-bedded facies and strength properties typically in the medium and high strength range. The 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 sets of steeply dipping joints:

- Set 1 (NNE): Strike 020° - 035° / Dip 70° - 90° E and W; and
- Set 2 (ESE): Strike 110° - 130° / Dip 70° - 90° N and S

Reference to Sydney CBD fault zones map (Pells, Braybrooke & Och, 2004), indicates the site is located within the G.P.O Fault zone. The fault zone extends from the north-east of the subject site to the south-west. Weaker rock and a higher degree of fracturing within the rock mass is expected in the fault zone. An extract from the geological map overlain by 2 m (AHD) contours is shown in Figure 2.

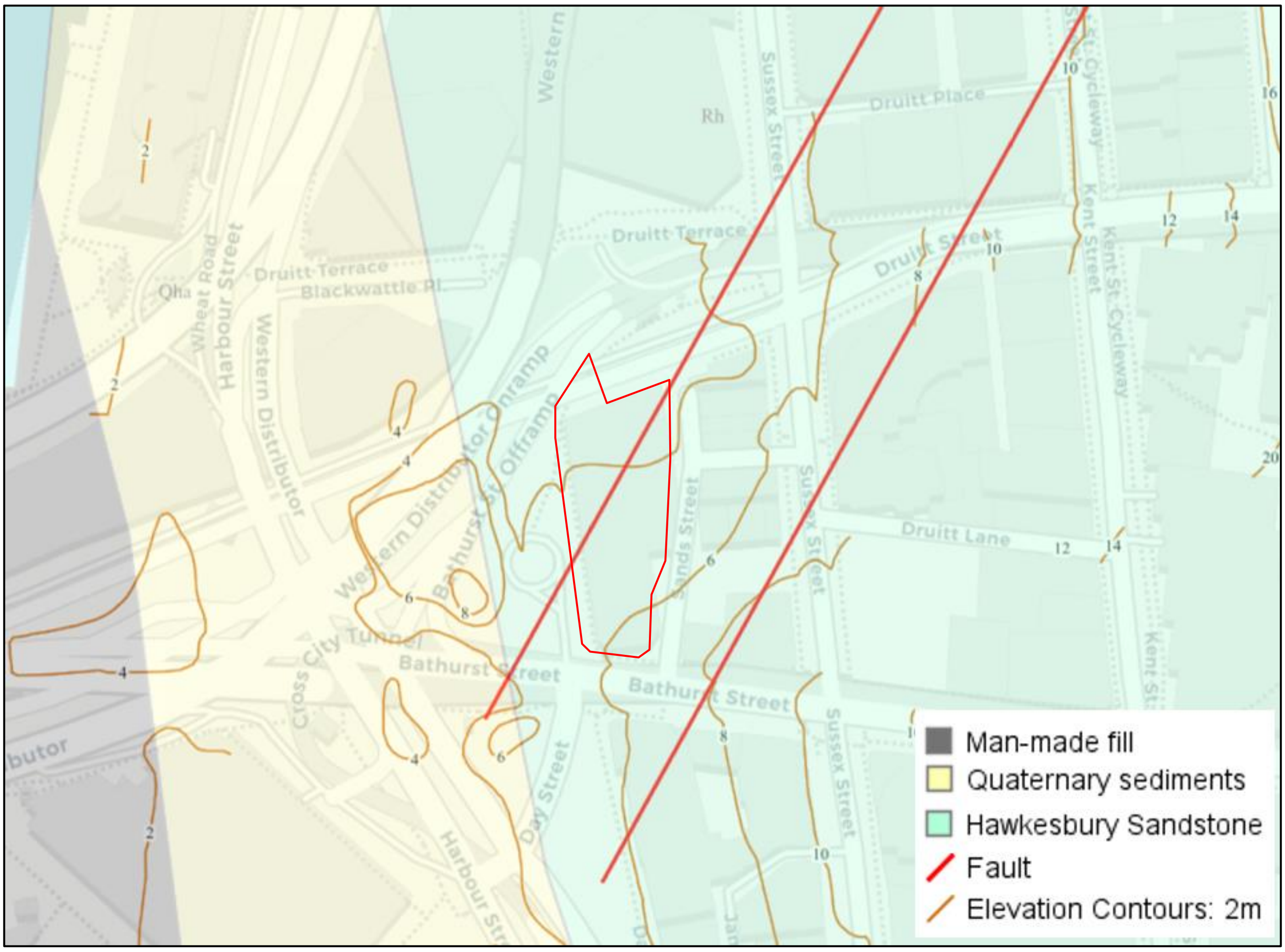

Figure 2: Subject site geology detailing GPO fault zone

3.2 Acid Sulphate soils

Reference to the NSW 1:25,000 Acid Sulphate Soil Risk Mapping, prepared by NSW Department of Environment and Climate Change, indicates the area is in low lying, disturbed terrain where there is the potential for Acid Sulphate Soils to occur. An area of high probability of Acid Sulphate soil occurrence is mapped within Darling Harbour approximately 160 m northwest of the site.

4. Previous Investigations

A number of investigations have been undertaken at the site and surrounds. The reports associated with these investigations have been reviewed as part of this desktop geotechnical assessment and are summarised below.

4.1 Previous Investigation on Site

- *DP Project No. 10878, (May 1988):* Douglas undertook geotechnical investigations for the development of Day Street Hotel (currently Park Royal Hotel) to inform structural design and to provide construction advice for the current hotel development. Six geotechnical boreholes (BH1 – 6) were drilled to depths of 5.3 – 16.3 m below ground level.

  The boreholes generally encountered fill and natural stiff sandy clays below thin asphalt and concrete pavement layers to depths of between 2.6 m and 5.6 m. Extremely weathered and very low to low strength sandstone was encountered in most borehole locations below the soil layers. Medium strength (of better) sandstone was typically encountered at depths of between 3.0 m and 9.3 m. Two boreholes (BH1 and BH5) were noted to encounter highly fractured and fragmented rock of highly variable strength. These bores were inferred to be within the fault effected zone running through and adjacent to the site.

  Groundwater was noted within five of the boreholes at depths of between 1.9 m and 3.0 m below ground level.

- *DP Project No. 10878/1, (Nov 1988):* A subsequent investigation was undertaken to further define the location of the G.P.O Fault Zone within the site. Eleven additional boreholes (BH7 – 17) were drilled to depth of between 9.8 – 20 m.

  The boreholes generally encountered fill below thin asphalt and concrete pavement layers to depths of between 0.9 m and 4.0 m. Thin layers of natural soil, less than 0.5 m thick, where noted below the fill in two boreholes only. Extremely weathered and very low to low strength sandstone was encountered in borehole locations BH7, BH12, BH14 and BH17 below the soil layers. Medium strength (of better) sandstone was typically encountered below depths of between 2.1 m and 7.0 m. Two boreholes (BH8 and BH12) were noted to encounter highly fractured and fragmented rock of highly variable strength. These bores were inferred to be within the fault effected zone running through the site. Based on the additional boreholes the approximate position of the GPO fault zone was inferred to be about 7 m wide and striking 030° across from the northeast corner to about the mid-point of the western boundary.

  Groundwater was noted within two of the additional boreholes at depths of between 1.7 m and 3.1 m below ground level.

- *DP Project No. 10878/2, (Nov 1989):* Footing inspections were conducted by Douglas during construction. Detailed inspection records are no longer held, however a summary letter of the inspections remains on file. The summary letter states that 14 inspections were completed. Spoon testing during the inspections confirmed that the founding material was generally considered to be Class III sandstone, suitable for the design allowable bearing pressure of 3,500 kPa, in the areas outside of the fault zone. Within the highly fractured and jointed material within the fault zone observed from the northeast to southwest corner of the excavation, the rock was classified as Class IV sandstone suitable for the design allowable bearing pressure of 1,500 kPa.

4.2 Nearby investigations

Historic investigations have occurred in the vicinity of and adjacent to the proposed development. These investigations are summarised in Table 1.

Table 1: Summary of nearby investigations

DP Job. No.	Year	Location	Work Description
10420	1987	Corner of Bathurst and Day Street	3 test pits excavated to depths 1.0 – 1.5 m, for pedestrian link bridge.
23776	1996	Bathurst Street, Sussex Street and Day Street	3 bore holes, ground investigation for residential multistorey unit developments.
85060.00	2015	273 – 279 Sussex Street	1 borehole, and desktop review. For 14 storey building + 4 basement levels.
85060.02	2017	273 – 279 Sussex Street	Site inspection, rock face mapping.
85146.01	2017	286 Sussex Street	Rock face mapping, inclinometer monitoring.

5. Geotechnical Model

Based upon the available data, the subsurface profile at the subject site is expected to include the following:

Table 2: Summary of Geotechnical Model

Unit	Description
Unit 1: Fill	Fill including sand, gravel, clay, bricks, concrete, sandstone boulders and organic soils, encountered to depth of between 1 m to 5 m below ground level.
Unit 2: Natural Soils	Encountered below the fill at depths of between 2.5 m to 5.5 m below ground level in some locations and typically comprising stiff sandy clays.
Unit 3: Extremeley Weathered and Very Low to Low Strength Sandstone	Highly weathered, very low to low strength sandstone with some extremely weathered zones encountered to depths of between 2.1 to 9.3 m below ground level.
Unit 4: Medium Strength (or better) Sandstone	Moderately weathered, medium strength (or better) sandstone

The extent of weathering and fracturing in the sandstone is expected to vary significantly in the vicinity of the G.P.O fault zone which crosses the site. In areas outside the estimated extent of the fault zone, the sandstone is generally noted to exhibit a higher degree of fracturing and an increased number of steeply oriented or subvertical joints, than typically seen in Hawkesbury Sandstone.

Groundwater levels were measured during the 1988 investigation in boreholes 1-4, 6, 8, and 16, at depths of 1.2 to 3.2 m below ground level. However, groundwater levels can vary seasonally and in the long term, due to climatic effects and following prolonged periods of rainfall. It will also vary due to local factors, such as changes to drainage conditions and subsequent nearby developments.

Table 3 provides a summary of the ground conditions and groundwater levels encountered in the previous investigation at the subject site in 1988.

The approximate borehole locations and position of the fault zone are indicated on the attached foundation plan included in Appendix B, which was provided by the Structural Engineer for the project (TTW).

Table 3: Summary of Depth/Level to Top of Strata (Depth: m, RL: m AHD)

Bore	Surface RL	Unit 1: Fill		Unit 2: Natural Soil		Unit 3: Very Low to Low Strength Sandstone		Unit 4: Medium Strength (or better) Sandstone		Base of Borehole		Ground Water Level	
		Depth	RL	Depth	RL	Depth	RL	Depth	RL	Depth	RL	Depth	RL
BH1**	4.3*	0.1	4.2	2.5	1.8	5.6**	-1.3**	9.3**	-5.0**	16.3	-15.8	2.9	1.4
BH2	3.7*	0.0	3.7	NE	NE	4.8	-1.1	NE	NE	5.3	-5.4	1.9	1.8
BH3	4.2*	0.2	4.0	3.7	0.5	4.3	-0.1	5.4	-1.2	13.0	-12.6	3.0	1.2
BH4	5.1*	0.0	5.1	2.7	2.4	NE	NE	3.0	2.1	8.5	-7.2	2.3	2.8
BH5**	6.5*	0.5	6.0	NE	NE	2.2**	4.3**	7.8**	-1.3**	12.9	-10.2	NE	NE
BH6	4.1*	0.3	3.8	1.2	2.9	2.6	1.5	5.9	-1.8	8.5	-8.2	2.6	1.5
BH7	3.7	0.3	3.4	NE	NE	3.7	0.0	5.2	-1.5	10.0	-6.3	NE	NE
BH8**	3.7	0.4	3.3	3.0	0.7	NE	NE	3.4**	0.3**	10.0	-6.3	5.4	1.7
BH9	3.7	0.2	3.5	NE	NE	NE	NE	2.3	1.4	10.1	-6.4	NE	NE
BH10	3.7	0.2	3.5	NE	NE	NE	NE	4.0	-0.3	10.5	-6.8	NE	NE
BH11	3.7	0.4	3.3	NE	NE	NE	NE	3.2	0.5	10.2	-6.5	NE	NE
BH12**	3.7	0.2	3.5	3.5	0.2	4.0**	-0.3**	5.2**	-1.5**	9.8	-6.1	NE	NE
BH13	5.1	0.3	4.7	NE	NE	NE	NE	3.3	1.8	20.0	-14.9	NE	NE
BH14	5.1	0.2	4.9	NE	NE	0.9	4.2	2.1	3.0	10.0	-4.9	NE	NE
BH15	5.1	0.2	5.0	NE	NE	NE	NE	3.9	1.2	10.0	-4.9	NE	NE
BH16	5.1	0.2	4.9	NE	NE	NE	NE	2.4	2.7	10.0	-4.9	1.9	3.1
BH17	5.0	0.2	4.9	NE	NE	3.7	1.3	3.3	1.8	10.0	-5.0	NE	NE

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Notes: * Arbitrary site datum adopted during original investigation. Level calculated based on strata and groundwater RLs provided in subsequent report which referenced a provided survey plan.

** Rock significantly affected by fault zone and includes highly fractured and jointed rock of variable strength.

6. Proposed development

It is understood that the proposed development of the site includes the expansion of the existing 11-storey hotel building with an additional 11-storeys. This includes the retention of as much of the existing structure as possible in order to reduce the embodied carbon emissions of the construction and to align with the sustainability objectives of the City of Sydney council. The existing building setbacks will also be maintained, as the new structure will be utilizing the existing perimeter columns.

As part of this redevelopment, it is expected that some of the existing building footings will need to be strengthened where loads are increased beyond their existing allowable bearing capacity. Localised excavation may also be required for new footings, lift pits and services.

7. Geotechnical Considerations

Preliminary comments on earthworks, excavations, groundwater, and suitable footing types are provided in the following sections. These comments may need to be reviewed after more detailed plans of the proposed development have been confirmed.

7.1 Geotechnical issues

The primary geotechnical issues that should be considered for the proposed development are:

- Due to the presence of the G.P.O Fault zone, which runs from the northeast to the southwest of the site, areas of lower strength rock and a high degree of fracturing occurs within the rock mass. As a result, a reduced bearing pressure of 1,500 kPa was adopted for foundations within the fault zone. Outside of the fault zone, fracturing and discontinuities in the rock mass limit the allowable bearing pressure to 3,500 kPa.
- To accommodate the expected two-fold increase in bearing pressure, the footings of the existing building will need to be assessed and strengthened.
- Groundwater levels were measured during the 1988 investigation at depths between 1.9 m to 5.4 m below ground level. These levels can vary seasonally and long-term due to climatic conditions, prolonged rainfall, and local factors such as drainage changes and nearby developments. Localised excavations within the existing basement levels (such as lift pits) may need to consider the possible presence of groundwater within the design.

7.2 Excavation Conditions

Excavation through fill, natural soil, and very low-strength sandstone (Units 1, 2 and some of Unit 3) can typically be readily achieved with conventional earthmoving equipment. However, excavating low to medium-strength (or stronger) sandstone will likely require rock hammers for effective removal.

The excavation of trenches or any other localised excavations should not be undertaken within the zone of influence of any existing building footings (or other structures). The zone of influence for rock can be determined by drawing an (imaginary) 45° line (i.e., 1H:1V) from the base of the excavation back to the ground surface. Any footing or structures bearing above this line could be considered to be within the zone of influence of the excavation. The zone of influence for soils is shallower and possibly 2H:1V depending on soil type and surcharge loads and should be assessed on a case by case basis. The depth and founding conditions of any footings (or other sensitive structures) within the zone of influence of any proposed excavation should be confirmed prior to excavation. Footings and structures within the zone of influence of proposed excavation may require underpinning or temporarily support during excavation.

7.3 Vibration

During excavations in rock, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. The level of acceptable vibration is dependent on various factors including the type of structure (e.g., reinforced concrete or brick structures etc.), its structural condition, the frequency range of vibrations produced by the construction equipment, the natural frequency of the structure and the vibration transmitting medium.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s vector sum peak particle velocity (VSPPV). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard (AS 2670.2, 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 VSPPV (vector sum peak particle velocity) for human comfort. This value may need to be reduced if there any sensitive structures or equipment nearby.

Based on the experience of Douglas and with reference to AS 2670, it is suggested that a maximum VSPPV of 8 mm/s (applicable at the foundation level of existing buildings) be provisionally adopted for this site for both architectural and human comfort considerations.

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. Enquiries should also be made in regard to the (vibration) sensitivity of the neighbouring buildings, housed equipment or any existing utilities on or around the site.

7.4 Excavation support

7.4.1 Batter slope

Vertical excavations in fill, soil and rock up to low strength (Units 1 to 3) are not expected to be stable and are required to have both temporary and permanent support. Rock significantly affected by faulting is also not expected to be stable due to the variable strength characteristics and high degree of fracturing and will also require support.

Where space permits, temporary batters of 1.5(H):1(V) could be used for excavations up to a maximum depth of 2 m within fill and natural soils above the water table.

Temporary batters for excavations in very low to low strength sandstone (or fault effected sandstone) should be maintained at no steeper than 0.75(H):1(V), or at 1(H):1(V) (or flatter) if permanent. Permanent batters will require shotcrete protection or similar.

Excavations in sandstone of medium (or greater) strength away from existing footings can be cut vertically provided there are no adversely orientated discontinuities. All unsupported excavations in rock should be progressively inspected by a geotechnical engineer or engineering geologist at each 1.5 m drop in excavation for the presence of adversely orientated discontinuities.

Where batters cannot be accommodated, some form of shoring or retaining support will be required.

7.4.2 **Shoring**

For the support of trenches or other smaller excavations, trench boxes or similar propped support systems could be used and could be designed on the basis of the preliminary parameters outlined in Table 4.

It is suggested that design of shoring with one row of props be based on a triangular earth pressure distribution using the earth pressure coefficients provided in Table 4. For walls where two or more rows of props are used, the shoring can be designed using a rectangular or trapezoidal earth pressure distribution. 'Active' earth pressure coefficient (Ka) values may be used where some movement is acceptable. 'At rest' earth pressure coefficient (Ko) values should be used where the wall movement needs to be reduced.

Table 4: Recommended Preliminary Design Parameters for Shoring

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient		
		Active (K_a)	At Rest (K_0)	Passive (K_p)
Unit 1 & 2: Fill / Natural Soil	20	0.4	0.6	-
Unit 3: Very Low to low strength sandstone	22	0.2	0.25	400 kPa
Unit 4: Medium strength (or better) sandstone	22	0[1]	0[1]	6000 kPa[1]

Notes: [1] Subject to inspection for the presence of adversely orientated discontinuities by geotechnical engineer or engineering geologist

Any significantly fault effected rock should be treated as Unit 3 material regardless of apparent rock strength due to the high degree of jointing and fractures likely to be present.

7.5 **Rock bolts and anchors**

Pre-stressed ground anchors and rockbolts (support elements) can be used to laterally support unstable rock masses (or shoring walls, if required). These support elements should be bonded into the stronger material, inclined as required, but preferably not steeper than 30° below the horizontal. 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/rockbolts used to support shoring walls should be bonded behind a line drawn up at 45° from the base of the proposed excavation.

The preliminary design of temporary and permanent ground anchors/rockbolts may be carried out using the allowable bond stresses given in Table 5 below.

Table 5: Recommended Preliminary Bond Stresses for Ground Anchor and Rockbolt Design

Material Description	Allowable Bond Stress (kPa)
Unit 3: Very Low to Low Strength Rock	75
Unit 4: Medium strength (or better) sandstone	350

The values adopted in the design should be confirmed by pull-out tests, carried out prior to installation of support elements, which may also justify slightly higher values. Ultimately, it is the anchoring 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 the ground anchors 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 anchors are locked-off at a lower value (say, 70% of the Working load), as required to accommodate the additional movement and subsequent increase in stress in the anchors, due to stress relief, swelling of clayey soils and other factors. Checks should be carried out to confirm that the load in the anchors is maintained and that significant losses due to creep or other causes do not occur.

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 from adjacent property owners will be required, prior to installing rockbolts/anchors below their land. Due consideration should also be given to buried services and any excavations nearby. TfNSW, Cross City Tunnel, Sydney Water and other service providers may require assessment on the effects that rockbolts and anchors may have on their assets.

7.5.1 Existing Shoring System

It is understood that the existing shoring system and basement depth is to be maintained as part of the redevelopment. It should be noted that modification of the existing basement retention system, if required, will need to consider the impact on neighbouring structures and services, such as TfNSW infrastructure, Sydney Water Assets and the Cross City Tunnel.

If such modifications are required, impact assessments should be undertaken to estimate the effect on neighbouring structures. Similarly, modification to the basement depth or retention system should also consider the effect such modification may have on groundwater inflow into the basement.

7.6 Foundations

Given the large magnitude of the increased load expected for the addition of 11 storeys to the existing building, the strengthening of existing footings or construction of additional footings is likely to be necessary. Footing strengthening could comprise locally increasing the size of existing footings to increase the foundation area.

New footings could also be constructed to carry the additional load. However, this will require an increase in load-bearing columns within the hotel, potentially leading to significant changes in the hotel layout.

Based on the desktop assessment, it is understood that the existing building footings were designed to bear on material suitable allowable bearing capacity of 3,500 kPa outside the fault zone and 1,500 kPa within the fault zone. Reference to Douglas letter (10878/2, dated November 1989) suggests that the footing excavations encountered material suitable for the design allowable bearing capacity during construction.

For preliminary design purposes, footings or footing strengthening could be designed based on the parameters provided in Table 6 below.

Table 6: Preliminary foundation design parameters

Material	Allowable Bearing Capacity (kPa)	Allowable Shaft Adhesion (kPa)	Ultimate Bearing Capacity (kPa)	Ultimate Shaft Adhesion (kPa)	Youngs Modulus E (MPa)
Unit 3: Very Low to Low Strength Sandstone	1,000	75	3,000	150	100
Unit 4: Medium Strength Sandstone	3,500	350	20,000	800	350
Low Strength Sandstone / Unit 4 Rock in the Fault Zone	1,500	100	4,000	250	150

Notes: 1. Classification system based on "Foundation on Sandstone and Shale in the Sydney Region, by Pells, Mostyn & Walker (1998)"

2. Ultimate values occur at large settlements, typically >5% minimum footing dimension

For footings proportioned on the basis of the allowable bearing capacities within Table 6, settlements would be expected to be less than 1% of the minimum footing dimension, with differential settlements between footings bearing upon the same material expected to be half this value.

It is expected that the majority of settlement of the existing footings will have already occurred under the existing building loads. For both new footings and areas of footing strengthening, some differential settlements may occur relative to the existing footings which have already experienced loading. The additional settlement of footings under increased loading and the potential for differential settlement between new and existing footings should be considered within the design.

All footing excavations should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the adopted design parameters.

Precise survey monitoring of columns subject to load increases of more than 20% should be carried out progressively to confirm settlements and footing performance is in line with the design assumptions. This should include baseline readings prior to load increases.

8. Recommendations for Further Investigation

Existing footings that need to support additional loads should be assessed to confirm their capacity and the suitability of the underlying foundation material for the required load. Such investigation can also be used to inform the design of footing strengthening that may be required. A preliminary assessment of a number of footings is currently underway and will be reported separately.

Assessment of the possible increased settlement below the existing or strengthened footings should also be completed once detailed information is available about the increased loading and foundation design.

All new foundation excavations or foundation strengthening should be inspected by a geotechnical engineer to confirm the suitability of the material for the design allowable bearing capacity. In addition, any unsupported excavations within rock should be inspected and geologically mapped by a geotechnical engineer/engineering geologist for the presence of adversely oriented discontinuities which may form unstable wedges.

9. References

AS 2670.2. (1990). *Evaluation of Human Exposure to whole-body vibration, Part 2: Continuous and shock-induced vibrations in buildings (1 to 80 Hz)*. Standards Australia.

Pells, P. J., Mostyn, G., & Walker, B. F. (1998). Foundations on Sandstone and Shale in the Sydney Region. *Australian Geomechanics, No 33 Part 3*, 17-29.

10. Limitations

Douglas Partners Pty Ltd (Douglas) has prepared this report for this project at 150 Day Street, Sydney, NSW in line with Douglas' proposal dated 2 September 2024 and acceptance received from Jack Rixon of UOL Group Limited dated 18/09/2024. The work was carried out under

Douglas' Engagement Terms. This report is provided for the exclusive use of UOL Group Limited for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of Douglas, does so entirely at its own risk and without recourse to Douglas for any loss or damage. In preparing this report Douglas has necessarily relied upon information provided by the client and/or their agents.

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

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

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

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

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

Appendix A

About This Report

Appendix B

Provided Drawings