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Level 7 / 50 Bridge Street
SYDNEY NSW 2000

85061.00.R.002.Rev1
11 December 2015
HS:pc

Attention: Mrs Catherine Hyland

Email: chyland@savills.com.au

Dear Sirs

Geotechnical Desktop Study
85 Harrington Street, The Rocks

1. Introduction

This report presents the findings of a desktop assessment for a proposed mixed commercial/ residential development at 85 Harrington Street, The Rocks, on the western side of Sydney Cove.

The site is located in an area of heritage and modern commercial and residential properties. The land adjoining the site to the south, between the site and the Cahill Expressway, is vacant and is understood to be under the ownership of TfNSW.

DP understands that the proposed development involves demolishing the existing building and constructing two luxury residential apartment buildings (5 and 9.5 above ground storeys), with ground floor retail and a three level basement car park. Based on the land survey drawing (LandPartners Drawing SY073523-SV1, dated 13/10/2014), existing surface levels at the Gloucester Street and Harrington Street frontages are approximately RL 27 m and RL 17 m respectively. Based on the point of entry to the basement being from Harrington Street (FJMT drawing GA85H, DA-2000 (Rev F), dated 25/11/2015), and the updated basement excavation Section 2 (FJMT drawing GA85H, DA-4002 (Rev 01), dated 25/11/2015), the bulk excavation level is RL7.4 m Australian Height Datum (AHD). The extent / depth of the basement excavation are shown in Drawings 1 and 2.

It is understood that a desktop geotechnical investigation is required for a Development Application (DA) approval, with preliminary comments made (where possible) on:

- the likely subsurface conditions including groundwater;
- excavatability of likely subsurface materials;
- shoring requirements;
- suitable foundation options; and
- anticipated geotechnical issues, including addressing the issue of the effects the project may have on groundwater and possible mitigation approaches (if required).

The existing City Circle rail tunnel, a 9 m wide double track tunnel operated by Sydney Trains, is understood to pass just beyond the south-eastern corner of the site. It is noted that information on the easement was not available for review by DP as part of this study. If the excavation is within 25 m of the rail corridor, detailed assessment study of the impact of the excavation on the tunnels will be required. This desktop study does not address any specific impacts that the proposed development may have on TfNSW assets.

The desktop assessment comprised a walk-over survey of the surrounding area by a principal geotechnical engineer from Douglas Partners Pty Ltd (DP) and a review of available and published geological information.

A Contamination Desktop Study is currently being undertaken by DP and will be reported separately.

2. Site Description

The site, located at 85 Harrington Street, The Rocks, incorporates a property to the north of the pedestrian stairs joining Harrington and Gloucester Streets, see Drawing 1. The architectural drawings indicate that the site is bounded to the west by Gloucester Street, to the east by Harrington Street and to the north by Cumberland Place and the Cumberland Place Steps. It is understood that a three storey residential terrace with a frontage to Gloucester Street ("Bakers Terrace") is being retained as part of the development, with excavation for the basement occurring at the rear of the Terrace.

Available information and observations made during a brief site visit indicate that the site is on the eastern side of the ridge, which runs between the Observatory and Dawes Point and carries the Bradfield Highway. The ground surface slopes in an easterly direction from the ridge towards Sydney Cove, with the surface elevation of the site falling from about RL 27 m to RL 17 m over a horizontal distance of approximately 25 m, between the Gloucester Street and Harrington Street frontages, respectively.

3. Regional Geology

Reference to the Sydney 1:100 000 Geological Sheet indicates that the site is underlain by the Triassic aged Hawkesbury Sandstone. This rock is a quartz sandstone which is predominantly medium to coarse grained with minor shale lenses. It has both massive and cross-bedded units, 0.5 m - 5 m thick.

The formation is cut by two main joint sets:

- Set 1: a north-north-east striking set, dipping 80°- 90°, generally to the west but sometimes to the east, spaced 0.5 m – >10 m, and generally persistent over many metres; and
- Set 2: an east-south-east striking set, dipping 80°- 90° to the north and south, often strata bound and with spacing often similar to the bed thickness.

4. Geological Site Profile and Observations

DP has carried out numerous investigations in the area, including drilling at the site and inspecting excavations directly opposite the site on Harrington Street (circa 1987). The positions of the boreholes drilled at the site by DP are shown on Drawing 1. It is noted that since the drilling was undertaken at the site in 1987, filling of parts of the site was undertaken as part of the construction of the existing building. It is understood that the base of the excavation for the existing development at the site was "stepped".

Observations made by DP (in 1987) of the excavation opposite the site on Harrington Street indicated sandstone at depths of between 1 - 2 metres, with bedding on the western face of the excavation appearing to be generally horizontal, and the surface of the sandstone forming benches stepping down in an easterly direction.

Observations made during the site walkover indicated sandstone exposures along the western frontage to Gloucester Street at the base of the wall, west of and above the site, and sandstone exposures within the northern portion of the site in the existing basement car park entry ramps, sides and plenums. Groundwater was not observed within the car park basement or plenums during the site walkover.

Borehole data from the site is summarised in Drawing 2.

Existing borehole data and experience in the area indicates that the subsurface profile is likely to comprise:

- surficial fill to depths of between about 0.5 m and 1.5 m over much of the site, expected to comprise gravelly sand or clay fill with some rubble;
- residual sandy soil beneath the filling, with anticipated thickness of between 0.1 m and 0.5 m over the site;
- highly weathered, medium to high strength sandstone, from depths of between 1 m and 3 m below existing surface; and
- groundwater is anticipated to be encountered as discrete seepage points from defects within the rock to depths of up to 6 m, with generally low inflow rates.

5. Comments

5.1 Excavation Conditions

Based on the likely subsurface conditions, the excavation will require the removal of up to about 1.5 m of surficial filling, then medium to high strength sandstone to the base of the excavation, at 10 m - 20 m depth.

Excavation within the filling and soils should be readily achieved by conventional earthmoving equipment such as hydraulic excavators with bucket attachments and bulldozer with blade attachments.

Excavation of the bedrock will largely be dependent on the rock's strength and discontinuity spacing encountered. The medium strength rock is expected to have an Unconfined Compressive Strength (UCS) of 6 – 20 MPa and high strength rock is expected to have a UCS of 20 – 60 MPa. Depending on the discontinuity spacing actually encountered during excavation, the medium and high strength sandstone will require the use of ripping / rock breaking / rock sawing equipment. Experience indicates that sufficient bedding planes and vertical joints are present in the upper layers of the sandstone to enable ripping by large dozers (i.e. D9 or D10). Previous DP reports document that the Contractor on the site directly opposite the site on Harrington Street (circa 1987) was able to rip the rock to depths of 3 - 4 m, but thereafter it was necessary to excavate closely spaced parallel trenches in the sandstone with a hydraulic rock breaker to allow the dozer to continue ripping. Therefore, it is likely that effective removal of the medium or higher strength sandstone within the basement can initially be achieved by moderate to heavy ripping, followed by use of large rock hammers in conjunction with heavy ripping. Detailed excavations adjacent to boundary lines can be achieved by use of hydraulic rock saws. Once excavated, the sandstone is anticipated to break up into 0.2 m to 0.5 m diameter fragments with appreciable amounts of finer fragments. Excavators on rock benches are likely to be required to pass material from the excavation base up to a loading out platform at ground level.

Where rock hammers are required in the vicinity of adjacent structures (closer than 20 m) it would be prudent to monitor and limit vibrations on these structures. Based on DP's experience and with reference to AS2670, a maximum peak particle velocity of 8 mm/sec (in any component direction) at the foundation level of adjacent structures is suggested for human comfort considerations. Vibration trials are suggested during initial excavation of rock to verify vibration levels and the effectiveness of rock saw cutting in reducing vibration.

It is understood that excavation will take place in close proximity to the rear of Bakers Terrace, which is a three storey brick building, probably in excess of 100 years old. This building is currently proposed to be underpinned as part of the project, prior to the basement excavation proceeding. Our previous investigation report indicates that excavations undertaken near similar buildings in Harrington Street, in the late 1980's, encountered footings bearing on clay or filling, despite the presence of near surface, medium strength rock. There is therefore a possibility that Bakers Terrace is founded on similar material, and could be damaged during excavation due to settlement caused by removal of the lateral support from surficial soils supporting the footings. Even if all footings for the Terrace are on sandstone, lateral expansion and slight settlement will probably occur progressively as excavation proceeds. Experience indicates that these movements can be as high as 0.5 mm vertically and up to 1 mm horizontally per metre depth of excavation proposed. These displacements are sufficient to cause distress to old masonry buildings and it is therefore recommended that a dilapidation survey is carried out prior to excavation and that Bakers Terrace be monitored to help minimise the possibility of further damage.

Vibration from machinery operating near Bakers Terrace may also cause damage, particularly during heavy ripping or rock hammering. It may therefore be desirable to saw cut along this boundary prior to excavating in this vicinity.

All excavated materials will need to be disposed of in accordance with current EPA policies. Under the Waste Avoidance and Resource Recovery Act (NSW EPA, 2001) a waste/fill receiving site must be satisfied that materials received meet the environmental criteria for the proposed land use. This includes filling and virgin excavated natural materials (VENM), such as may be removed from site. Accordingly, environmental testing will need to be carried out to classify spoil prior to disposal. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the receiving site.

It is likely that there will be some seepage of groundwater into the excavation along the top of rock and from joints and bedding planes within the rock. Such seepage will need to be collected during construction in sumps and intermittently pumped off site. At this stage, it is not possible to accurately estimate the likely extent and rate of seepage. It is anticipated that seepage rates will be relatively low (less than 3 ML/year) given the expected low permeability of the rock mass. Inflow rates such as these are readily handled by sump and pump measures. It is suggested that monitoring of flow during the early phases of excavation be undertaken to assess long term pumping requirements.

5.2 Excavation Support

The estimated 10 m to 20 m deep excavation (i.e. three basement levels to RL7.4 m on a sloping site) is likely to encounter up to 1.5 m of filling, and then medium to high strength sandstone. Careful consideration must be given to the planning and design of excavation and excavation retention system(s) to reduce the risks of destabilising and causing damage to the adjacent buildings and surrounding public footpaths/roads.

As excavation will be required virtually to the boundaries of the site, battering of the sides of the excavation will not be feasible. Where present, re-use of the existing retaining walls associated with the existing building could be considered. This method would need to be detailed by the structural engineer but is anticipated to include supporting the walls with structural props or anchors possibly in conjunction with a shotcrete covering.

Where no existing walls are present and where vertical excavation in the filling, soils and rock of less than medium strength is expected, both temporary and permanent lateral support will be required during excavation and as part of the final construction. Shoring is therefore likely to be required down to approximately 2 to 3 m below surface, dependent on the levels that medium strength sandstone is encountered. Typical shoring systems could include soldier piles with shotcrete infill panels, shotcrete supported by dowels and sitting on a medium strength sandstone ledge (subject to no adversely oriented joints being present in the rock beneath) or a reinforced shotcrete wall, supported by bolts or anchors constructed in 4 m wide, anchored, panels using a hit-and-miss type construction sequence.

Excavation in the medium to high strength sandstone within the site can be cut vertically and left unsupported as the excavation progresses, subject to a detailed assessment of jointing and rock conditions every 1.5 m drop by a suitably qualified geotechnical engineer/engineering geologist. If there are adversely oriented joints or faults present, there is a risk of instability developing in these faces that that could adversely affect wall stability and the footing performance of the adjacent buildings and infrastructure. Steeply dipping joints, which have been observed in this unit, could form unstable wedges that require immediate stabilisation. Allowance should be made for spot rockbolting or ground anchors to stabilise any adversely dipping joints and/or faults identified during excavation inspections. In addition, shotcreting (with reinforcing mesh and dowels) of all very low strength weathered layers or clay seams greater than 100 mm will be required to prevent the otherwise inevitable weathering and degradation of exposed faces.

This requirement for regular geotechnical inspections every 1.5 m drop should be explicitly stated on the drawings and a hold point instruction developed until inspection for each section of excavation is carried out.

As previously outlined, the proposed excavation on the southern side of the site (adjacent to the Cahill Expressway) will approach close to the RailCorp easement. Depending on the location of electricity cables in this area relative to the excavation face, it may be necessary to provide temporary support for these services.

5.3 Internal Temporary Batters

During bulk excavation, the maximum unprotected batter slopes in Table 1 are recommended for the temporary battering of internal excavations.

Table 1: Temporary Batter Slopes

Material Description	Batter Slope (H:V)
Filling and Residual soils	1.5:1
Weathered Rock	0.75:1[1]

Note: 1 - Subject to Geotechnical Inspection every 2 m lift of excavation to determine if flatter batters or stabilisation measures are required.

5.4 Shoring Wall Design

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

Table 2: Lateral Earth Pressure Coefficients

	Lateral Earth pressure coefficient (K_a)*[1]		K_0*[2]
	Temporary Support	Permanent Support	
Filling, clay and extremely low strength rock	0.35	0.4	0.6
Low to medium strength sandstone	0	0	0

*[1] Assuming a level surface behind the wall
*[2] Assuming no adverse dipping joints are present

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

Surcharge loading from adjacent building footings, sloping ground, traffic or other loadings should be taken into account. Unless positive drainage measures can be incorporated to prevent water pressure build-up behind the walls, full hydrostatic head should be allowed for in design while, at the same time, allowing for the soil unit weight to reduce to the buoyant condition.

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

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

Where:
H_z = horizontal pressure at depth z
γ = unit weight of soil or rock
K = Earth pressure coefficient
z = depth (m)
p = vertical surcharge pressure

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

Care should be exercised in construction to ensure that anchors are installed progressively during excavation and stressed prior to excavation of the next drop. It should be noted that stress relief (see Section 5.6) related movement may lead to an increase in the stress in anchors.

5.5 Anchor Design

The shoring wall can be supported with prestressed type rock anchors. It is suggested that these be inclined, but not steeper than 30° below the horizontal, to allow anchoring in the stronger rock. For estimating purposes it is suggested that an allowable bond stress of 250 kPa be adopted in low strength sandstone and 500 kPa in medium strength sandstone (refer Table 3 below).

Table 3: Allowable Bond Stress

Material	Allowable Bond Stress
Low strength sandstone	250 kPa
Medium strength sandstone and better	500 kPa

These values should be confirmed by a pull out test prior to installation of anchors.

Ultimately, it is the contractor's responsibility to ensure that the correct design values (specific to the anchor system and method of installation) are used and that the anchor holes are carefully cleaned out prior to grouting. After anchors have been installed, it is recommended that they be tested to 125% of nominal working load and then locked off at between 60% and 80% of their working loads. Checks should be carried out, however, to ensure that the load is maintained in the anchors throughout the construction period and is not lost due to creep effects or to other causes.

Permission should be sought from neighbours or relevant authorities, if the proposed anchors extend beyond the boundary of the site.

5.6 Excavation Induced Ground Movements

For a major excavation, such as is proposed on this site, there will be inward horizontal movement of the excavation face due to stress relief effects. Release of high horizontal in-situ stresses will cause horizontal movements along the rock bedding surfaces and defects.

Based on previous experience in the Sydney area it is estimated that at the midpoint of the crest of a deep excavated face, stress relief will generally cause a horizontal movement of approximately 0.5 mm/m to 1 mm/m depth of excavation. The amount of horizontal movement will diminish along the crest away from the midpoint, down the excavated face away from the crest and back from the face. The movement would be expected to occur progressively during the excavation and should be completed shortly after excavation is completed. If bulk excavation leaves a relatively thin bed of sandstone just below the base then there can also be compression or shear failure of this layer.

The stress relief movements may cause cracks within buildings, infrastructure and underground services adjacent to the excavation. The effects of this movement on the various buildings and infrastructure should be considered by a structural engineer. Appropriate allowance should be made for the potential repair of these structures. Dilapidation surveys of adjacent buildings should be carried out, both prior to and at the completion of bulk excavation.

Consideration should be given to the locations of internal columns, connections with perimeter walls and other design issues particularly at the northern end of the site, so that potential stress relief movements, as a result of excavation of future basements for nearby buildings, do not affect the building.

5.7 Foundations

High strength sandstone is expected to be encountered at the base of the excavation (shown on the architectural cross-section drawing to be at RL7.4 m) and it is expected that the proposed building will be supported by pad or strip footings. Due to the previous boreholes being drilled for a shallower basement level, the existing borehole information does not extend to the base of the proposed excavation plus 3 m as would be usually required. Additional drilling to confirm the expected ground condition will be necessary.

Based on our previous investigation report, however, high strength sandstone is anticipated at final basement level, probably meeting the requirements for at least Class II sandstone as given in Pells et. al. (1998). At this stage, a maximum allowable bearing pressure of 6000 kPa is suggested for design purposes for this material though it is possible that a lower strength layer of rock is present at foundation level. Spoon testing will be required in at least 50% of the footings and the base of all footings inspected and approved by geotechnical engineer prior to placing reinforcement. Spoon testing should extend below the footing bases for a depth of at least 1.5 times the footing width or 2.5 m, whichever is the shallower. Higher allowable bearing pressures of 8000 kPa or more may also be achievable on high strength sandstone, subject to a more extensive program of coring and spoon testing.

For high level footings constructed on high strength sandstone and footings on the edge of excavations, a maximum allowable bearing pressure of 4000 kPa is suggested. Inspection of the excavated face below the footing will be necessary to ensure that there are no significant defects or adversely dipping joints that may affect footing performance.

These parameters apply to the design of spread footings, such as pads or strip footings. Wherever possible all footings should be founded below a line drawn upwards at 60 degrees from adjacent excavations.

Foundations proportioned on the basis of the allowable parameters would be expected to experience total settlements of less than 1% of the minimum footing width under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value (based on a standard 8 m x 8 m grid).

5.8 Seismic Design

In accordance with Section 4 of the Earthquake Loading Standard, AS1170.4 - 2007 the site is assessed to have a Site Sub-Soil Class of “B_e”.

5.9 Ground Slabs

The ground floor slab at the lowest level of the basement is expected to be used for car parking and hence will probably only be lightly loaded. The base of the excavation will be high strength sandstone which will provide adequate support for a slab-on-grade.

It is recommended that a gravel layer be provided beneath the floor slab and should slope towards the sump pit to allow sub-floor drainage.

5.10 Further Investigation

Transport for New South Wales (TfNSW) will need to be approached for information on the location of the City Circle Tunnels with respect to the property.

Additional geotechnical drilling (two boreholes to depths of 23 m) is recommended along the Gloucester Street frontage of the site, to confirm the depth to sandstone, and to obtain sub-surface information at the deepest part of the excavation (approximately 20 m deep).

Additional geotechnical drilling (two boreholes to an average depth of 12 m dependent on location and surface RL) is recommended along the Harrington Street frontage of the site, to obtain information on the sandstone beneath the basement excavation.

The above advice is based on a desktop assessment of predicted subsurface conditions at 85 Harrington Street, The Rocks. It is suitable for preliminary design only and confirmation of ground conditions will be required. This document will be superseded once the results of any future field investigation become available.

5.11 Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at 85 Harrington Street, The Rocks, in accordance with proposal dated 4 December 2014, and acceptance received on 20 August 2015. The work was carried out under DP's Standard Conditions of Engagement. This report is provided for the exclusive use of Savills Project Management Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or be relied upon for other projects or purposes on the same or other sites or by a third party. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The comments provided in the report are indicative of the sub-surface conditions on the site only and are based on the information to hand. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences.

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.

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.

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.

References

- Douglas Partners Site Investigation Report, October 1987. Proposed Office Development, The Rocks, SSI/10523.
- Pells PGN, Mostyn G and Walker BF, 1998, Foundation on sandstone and shale in the Sydney region, *Australian Geomechanics*, December 1998, p 17-29.

Please contact either of the undersigned for clarification of the above as necessary.

Yours faithfully
Douglas Partners Pty Ltd

Reviewed by

Huw Smith / Hugh Burbidge
Senior Engineering Geologist

pp **John Braybrooke**
Principal

Attachments: About this report
Drawing 1 - Location of Tests
Drawing 2 - Cross Section A-A'

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.

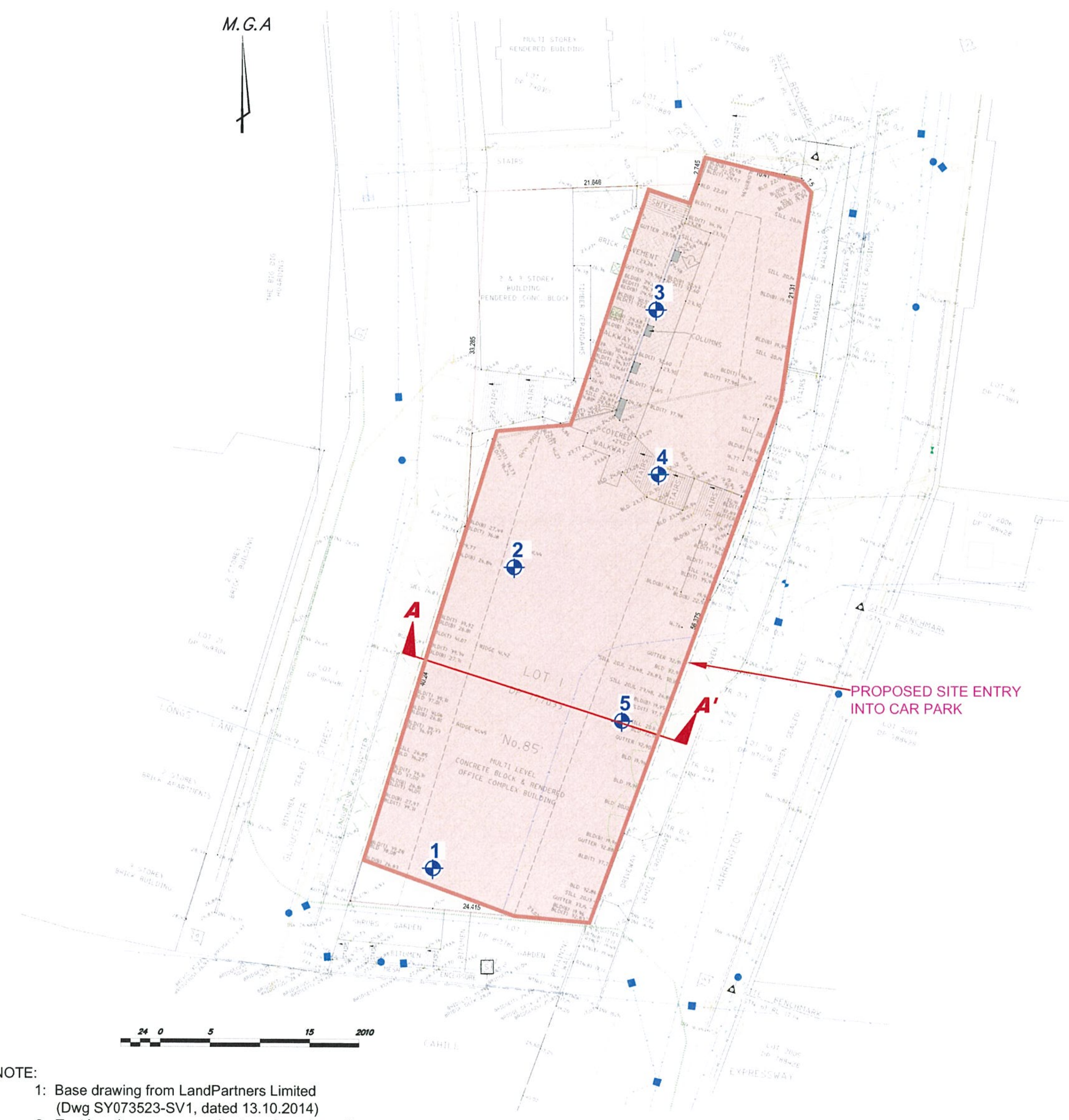
Information for Contractual Purposes

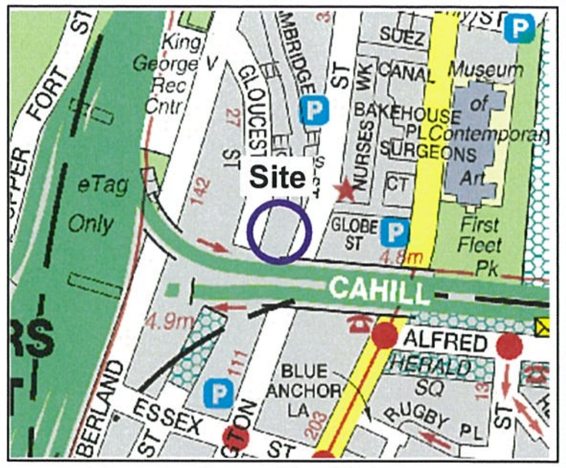
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

M.G.A





Locality Plan

NOTE:

1: Base drawing from LandPartners Limited (Dwg SY073523-SV1, dated 13.10.2014)

2: Test locations are approximate only and are shown with reference to existing featues.

LEGEND

DIAL BEFORE YOU DIG DETAILS
(ALL SERVICE DETAILS SHOWN APPROXIMATE ONLY)

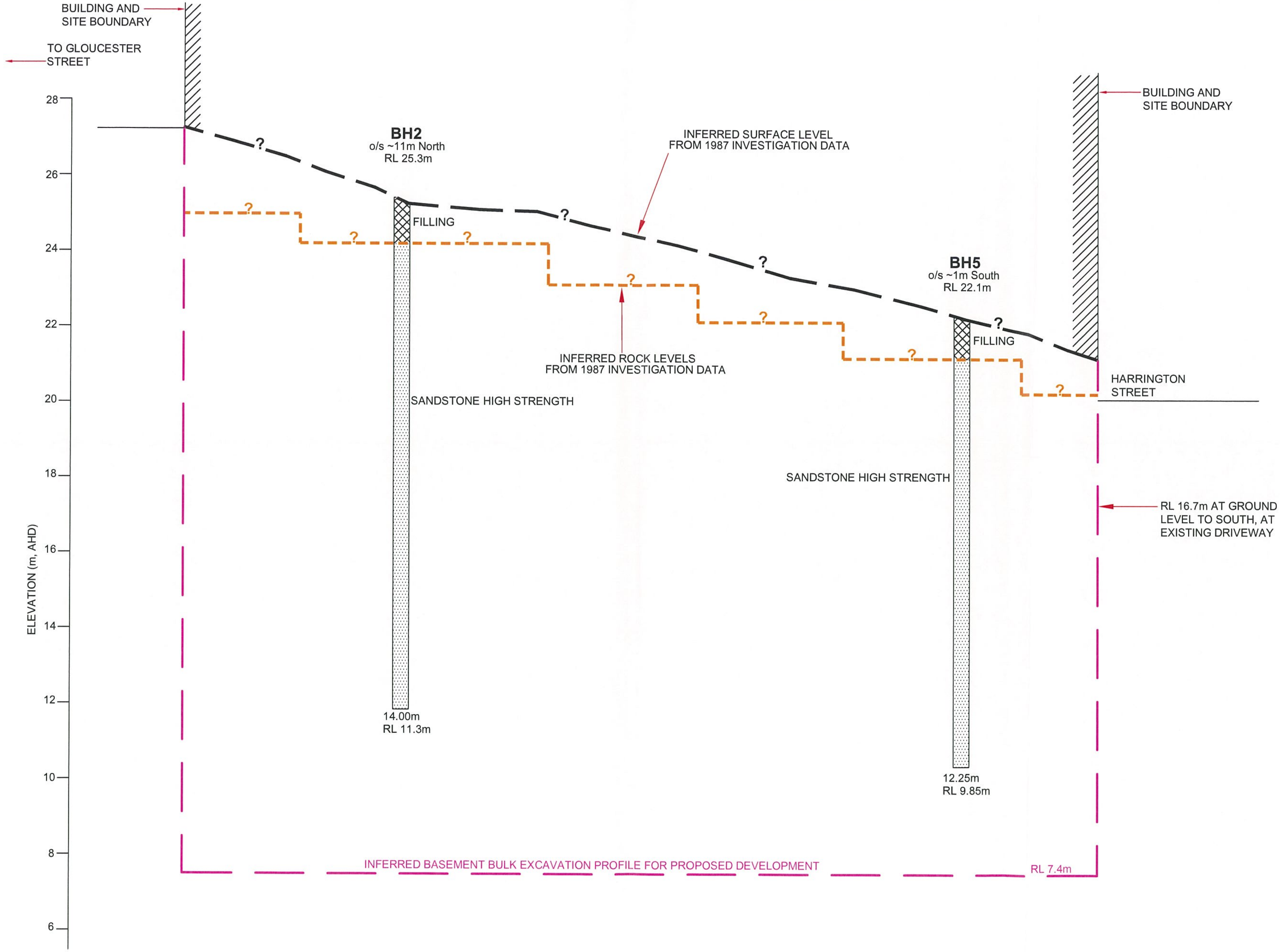
Indicative location of previous DP borehole (1987)

A — A' Geotechnical Cross Section A-A'

Outline of proposed basement, refer Drawing GA85H-DA-2011 (25/11/15)

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Golden Age & Hannas The Rocks Pty Ltd		TITLE: **Location of Tests**
	OFFICE: Sydney	DRAWN BY: PSCH	**Geotechnical Desktop Study**
	SCALE: 1:500 @ A3	DATE: 8.12.2015	**85 Harrington Street, THE ROCKS**

PROJECT No:	85061.00
DRAWING No:	1
REVISION:	0



Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Golden Age & Hannas The Rocks Pty Ltd	
OFFICE: Sydney	DRAWN BY: PSCH
SCALE: 1:100 @ A3	DATE: 8.12.2015

TITLE: **Cross Section A-A'**
Geotechnical Desktop Study
85 Harrington Street, THE ROCKS

PROJECT No:	85061.00
DRAWING No:	2
REVISION:	0

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