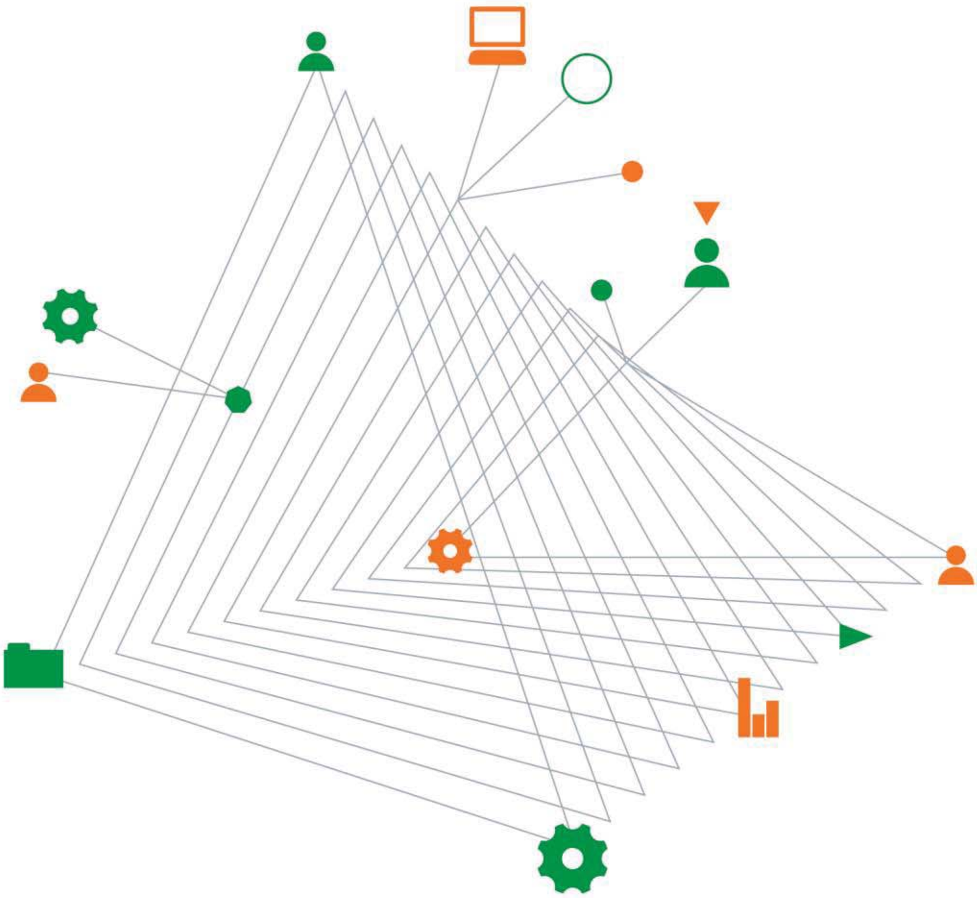
One Investment Management Pty Ltd as Trustee for Recap Management No. 4 Trust

4-6 Bligh Street, Sydney NSW

Planning Proposal - Geotechnical Desktop Study and Rail Impact Statement Rev02

26 July 2017



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4-6 Bligh Street, Sydney NSW

Prepared for
One Investment Management Pty Ltd as Trustee for Recap Management No. 4 Trust

Prepared by
Coffey Geotechnics Pty Ltd

26 July 2017

Document authorisation

Our ref: 754-SYDGE204536-AC Rev02

For and on behalf of Coffey

Ross Best
Senior Principal

Quality information

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1. Introduction

This report presents our findings from our preliminary geotechnical desktop study for the proposed development of 4-6 Bligh Street, Sydney.

The work was commissioned by One Investment Management Pty Ltd as trustee for Recap Management No. 4 Trust (Recap) and was undertaken in accordance with Coffey proposal ref. 754-SYDGE205019-AA issued on 21 April 2017, which includes:

- Desktop geotechnical review to assess the site's subsurface conditions and assess the suitability of the site for the proposed development
- Identify potential engineering measures required for the construction of the proposed development
- Rail Corridor Impact Statement.

The purpose of this report is to support the Planning Proposal for the proposed development.

2. Proposed development

We understand that the indicative architectural scheme for the 4-6 Bligh Street project provides for a new mixed use hotel and commercial building with height of 55-storeys or 205 metres / RL 225.880. The indicative architectural scheme comprises:

- 10 storey podium, including hotel entrance lobby, commercial lift lobby, food and beverage facilities, plant, commercial offices, meeting/conference rooms, gym space, and landscaped podium with formal hotel lobby
- 37 storeys of hotel (each level including 11 rooms, with a total of 407 rooms)
- 4 levels at rooftop including hotel club lounge, function space, restaurant and bar, and publicly accessible landscaped terrace
- 4 basement levels including 17 car parking spaces, 2 loading spaces, plants, end of trip facilities and waste management facilities
- Site footprint area of approximately 1216 m^2.

The concept reference design plan dated 20 July 2017 provided by Architectus show the floor level of the lowest basement at 6.010 mAHD.

The site is located approximately 500 m south of Circular Quay, and approximately 480 m north of Hyde Park, as shown on Figure 1.

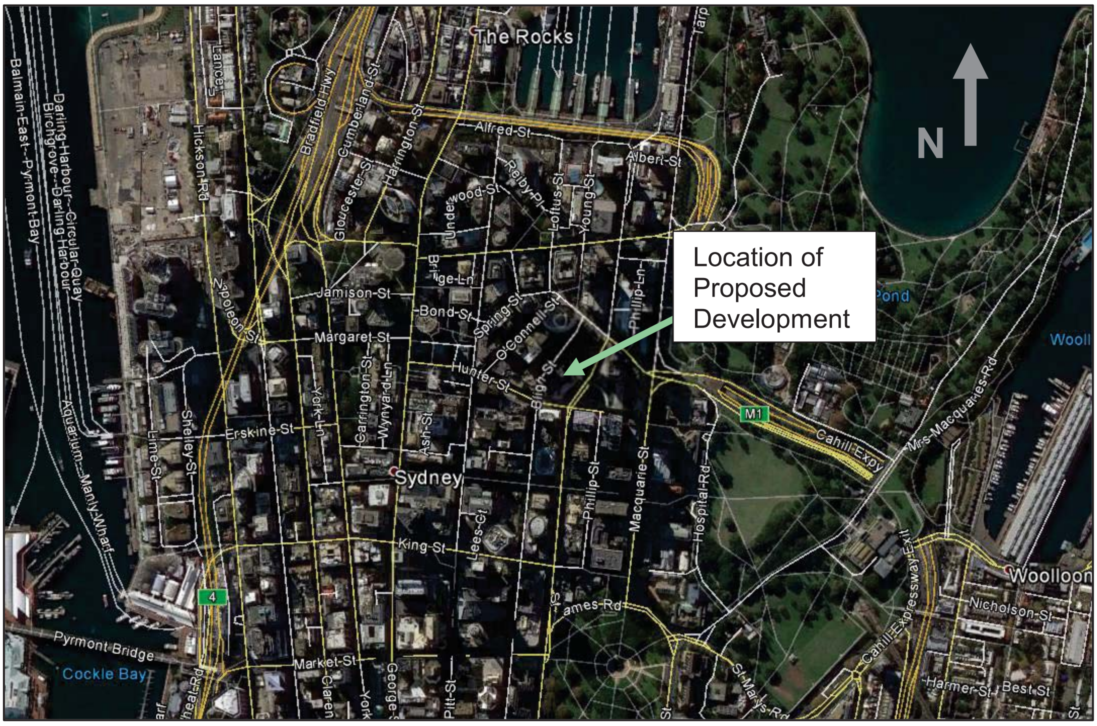


Figure 1: Location of Proposed Development

The proposed development is within close proximity to multiple heritage buildings as shown on Figure 2. The development is bounded by the Bligh Street to the northwest, the Sofitel Wentworth Heritage Building to the northeast, the Qantas House heritage building to the southeast, and the 61-101 Phillip Street heritage building to the southwest.

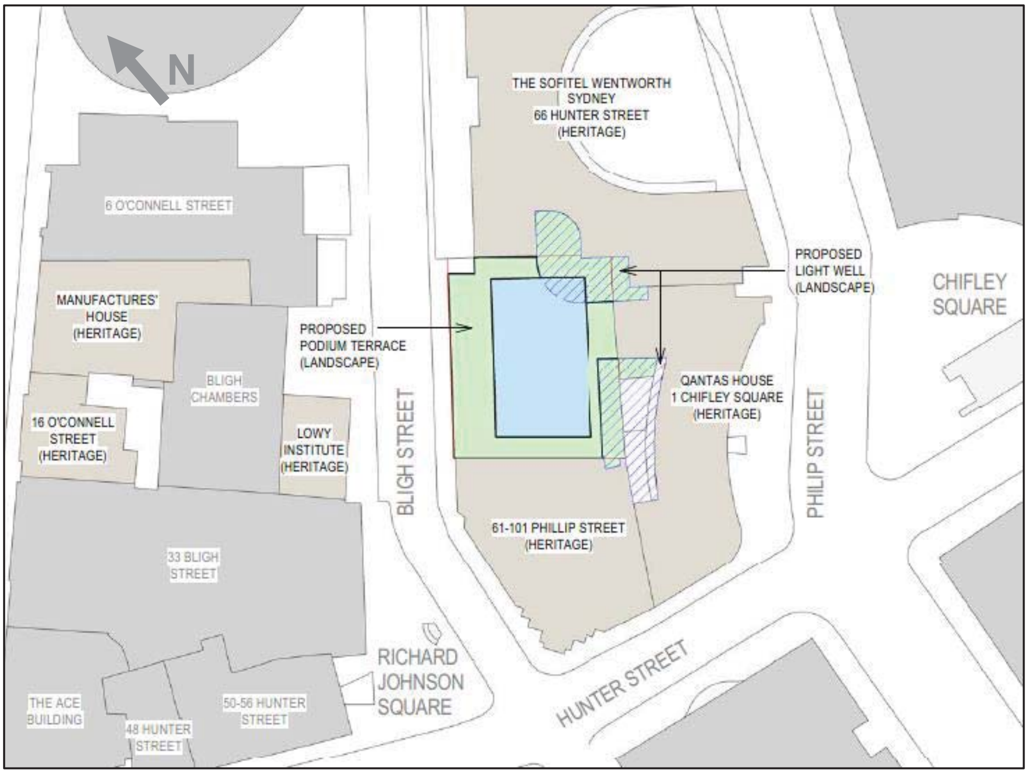


Figure 2: Heritage Buildings Surrounding 4-6 Bligh Street

We understand that Recap is proposing to demolish the existing 22 storey tower building on 4-6 Bligh Street for this development. The ground floor level of the existing tower building is approximately 21 mAHD. The existing tower building contains a two level basement, and the floor levels of the level one and level two basement are 17.9 mAHD and 12.8 mAHD respectively.

The existing ground level surrounding the site generally falls towards west south west, with the lowest point (at the south western corner) at a level of approximately 19.5 mAHD.

3. Desktop Geotechnical Review

3.1. Local Geology

The Sydney Harbour 1:25,000 Acid Sulfate Soils Risk Map indicates that there is no known occurrence of Acid Sulfate Soils in the locality. This is consistent with the presence of residual rather than alluvial soils.

The Sydney 1:100,000 Geological Sheet indicates that the site locality is underlain Hawkesbury Sandstone which is composed of predominantly medium to coarse grained quartzose sandstone typically comprising 1 m to 3 m thick beds, with major joint sets trending north-south and east-west as an orthogonal pattern, with a subordinate northwest-southeast trending set. The north-south (trending about 10° to 15° east of north) joint set is more dominant set, with a subvertical dip and typical spacing of 1m to 5m. The east-west trending joints tend to be spaced at 5m to 15m intervals.

A geological map showing near vertical structural features in Sydney CBD (Pells, Braybrooke & Och – 2004) shows geological features in the vicinity of the proposed development, however none of these geological feature are expected to intersect the site as shown on Figure 3. Three geological features in the proximity of the site are:

- The Pittman LIV dyke running east west across the CBD, located to the north of the site
- The GPO Fault Zone orientated in a NNE – SSW direction, located to the west of the site. From Energy Australia Cable Tunnel investigation along Bridge Street it is expected that the GPO Fault Zone may be up to 60m wide. There are two distinct sections of faulted rock forming the edges of the fault zone, each approximately 20 m wide, with relatively intact rock in between
- Martin Place Swarm Joint running in a NNE – SSW direction sub-parallel to the GPO Fault Zone. The Martin Place Swarm Joint is located to the east of the site. This zone comprises closely spaced jointing with minor normal and reverse faulting.

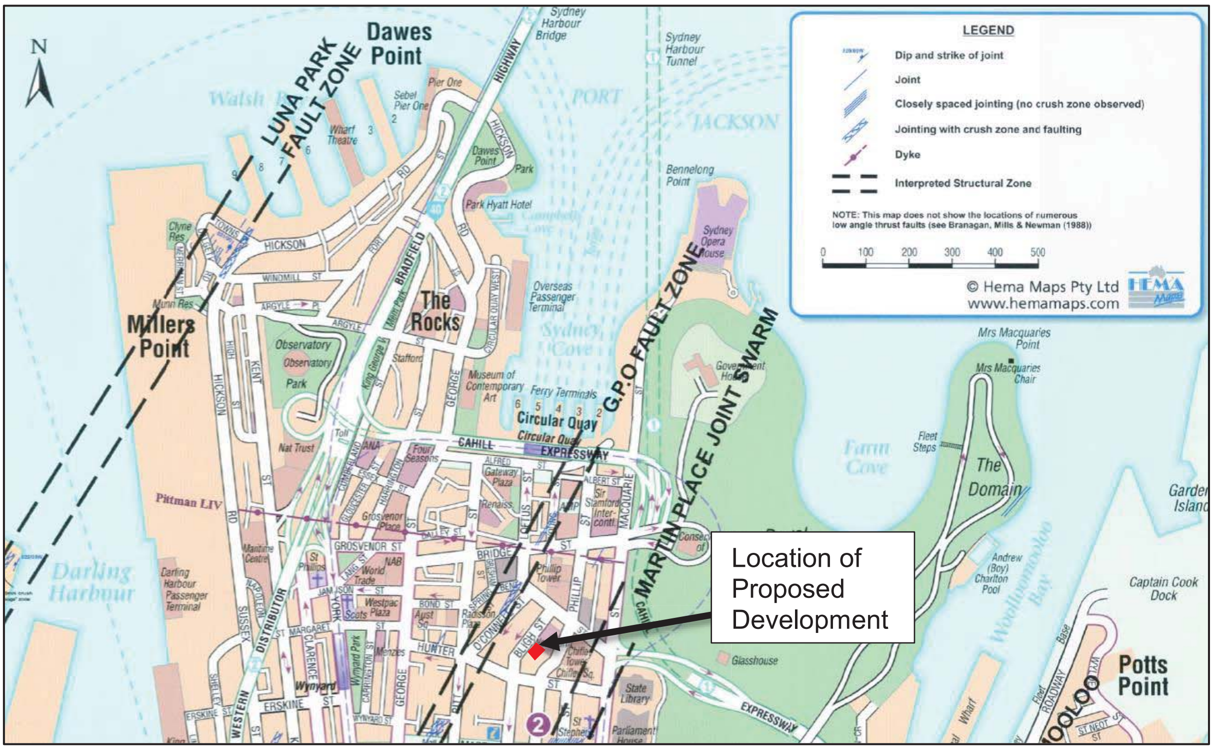


Figure 3: Geological Map Showing near Vertical Structural Features in Sydney CBD (Pells, Braybrooke & Och – 2004)

Groundwater movement would typically occur at the soil/rock interface and in bedrock joints and bedding partings. Bedrock seepage in sandstone bedrock could be assumed as typically flowing toward local drainage lines or regional water table, along horizontal bedding planes and sub-vertical joints. In the absence of development, the local groundwater table would be expected to fall to the north towards Circular Quay. Monitoring in the area by Coffey at nearby locations has revealed the groundwater levels are influenced by drainage to basements and by services. The site is some 500 m from the waters of Circular Quay, and is not expected to fluctuate significantly with tidal water level changes.

3.2. Subsurface Conditions

3.2.1. General

Coffey has conducted a number of geotechnical studies in the general area of the site. Figure 4 show investigation locations in the vicinity of the site.

The expected subsurface conditions, based on our experience, is summarised in the following sub-sections.

3.2.2. Encountered Material

Surficial fill of varying thickness is present over Hawkesbury Sandstone.

The fill generally comprised sandy gravel and gravelly sand, and contains some concrete and brick rubbles, and sandstone fragments. A concrete or asphalt layer is typically found overlying the gravelly material. Based on our experience, we anticipate fill thickness from 0 m to 4 m in the vicinity of the site.

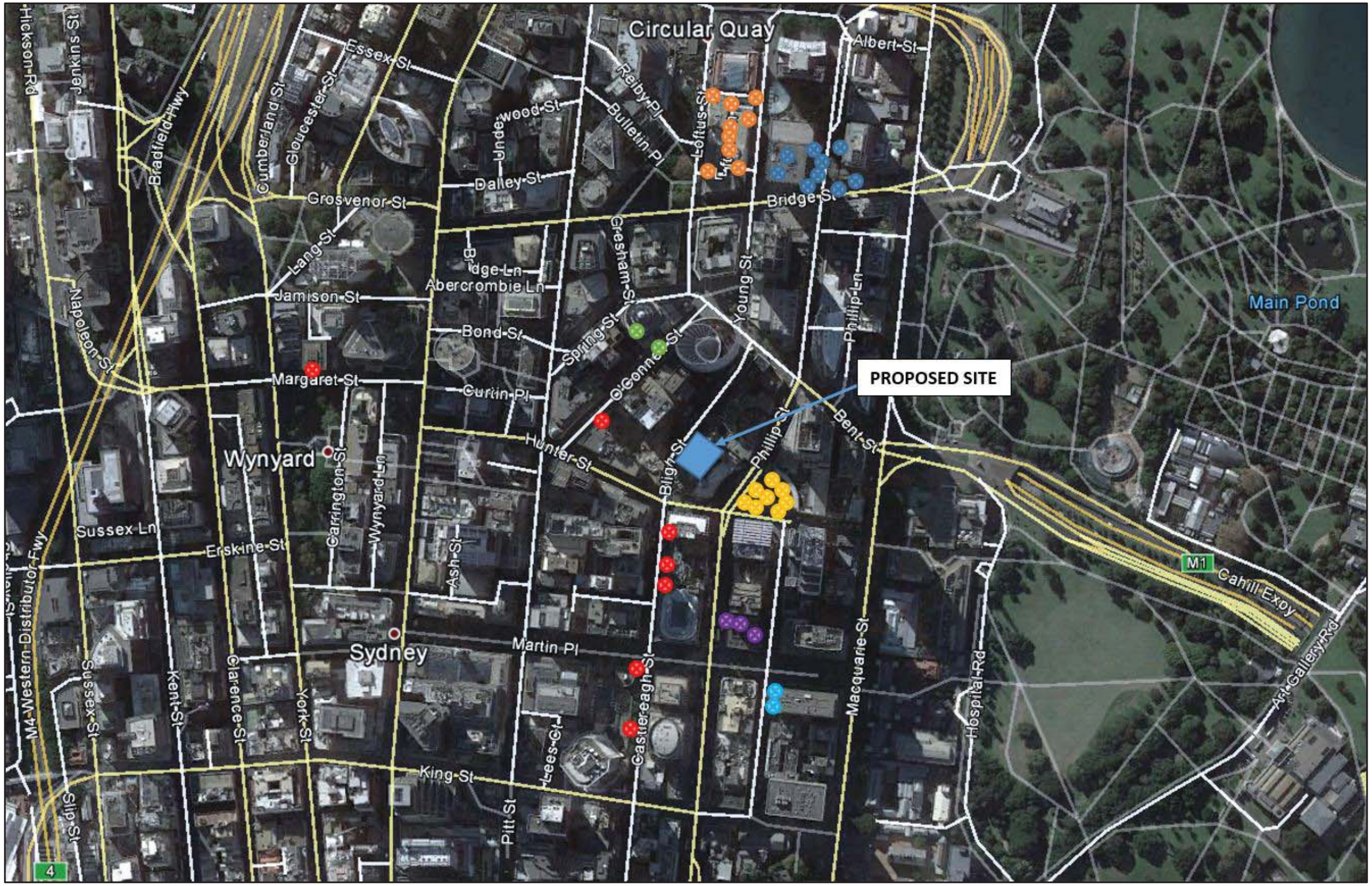


Figure 4: Locations of Investigation Drilling by Coffey on Various Projects

The Hawkesbury Sandstone underlying the fill exhibits varying degree weathering/strength profile from borehole to borehole. The sandstone typically varied from moderately weathered to fresh, and from low to high strength. Hawkesbury Sandstone at the development site is expected to comprise:

- Residual to extremely weathered, very low to low strength sandstone –about 2 m thick; underlain by,
- Highly weathered to moderately weathered, low strength sandstone – up to about 3 m thick; underlain by,
- Moderately weathered to fresh, medium to high strength sandstone.

3.2.3. Stratigraphy

Based on the information obtained from the boreholes and CPT's a geotechnical model has been developed and is presented in Table 1. The layer thicknesses shown in Table 1 are in a downward progression starting from ground surface level.

Table 1: Subsurface Profile at Test Locations

Geological Unit	Material	Thickness of Layer [m]
Fill (Including Surficial Concrete)	Generally comprises sandy gravel and gravelly sand, and contains some concrete and brick rubbles. Overlain by concrete/asphalt.	0 to 4
Hawkesbury Sandstone	Residual to extremely weathered, very low to low strength sandstone	1 to 2
	Highly weathered to moderately weathered, low strength sandstone	1 to 3
	Moderately weathered to fresh, medium to high strength sandstone	-
(1) Expectations based on experience in the area and subject to review following site specific investigation.		

3.2.4. Groundwater

Groundwater level measurement below 0 mAHD have been recorded in the vicinity of the site. This is a clear indication that drainage to subsurface basements or services has occurred.

Based on our experience we anticipate that the groundwater level at the site is above 2 mAHD.

The permeability of the sandstone is expected to be governed by seepage along joints, shear fractures, bedding partings, and other defects. The lowest basement level (anticipated to be at 6.010 mAHD) is not expected to be significantly below the existing groundwater table. As a result seepage into the basement excavation during construction is expected to be minor. Coffey anticipates drainage to sumps will be sufficient to manage seepage during construction.

3.2.5. Key Findings from Geological Desktop Study

Key findings from our preliminary desktop geotechnical study are as follows:

- Rock generally occurs at shallow depths across the site (typically within 4 m of the surface). It is overlain by fill comprising sandy gravel and gravelly sand which contains some rubbles and fragments, and covered by pavement of floor slab
- It may be possible to reuse the site-won fill material for temporary crane platforms
- The sandstone rock has typically weathered to residual soil and is highly weathered at the rock surface, and quickly grades through to a moderately weathered to slightly weathered, medium to high strength sandstone. Fresh Sandstone is expected close to the bottom of the excavation
- Due to the relatively high rock strength and the relatively wide spacing of defects of the fresh sandstone, it is likely the rock towards the bottom of the proposed excavation would be difficult to excavate
- The lower basement level (floor level at 6.010 mAHD) is not expected to be significantly below the predevelopment groundwater level, and as a result seepage to the excavation during construction is expected to be small. Seepage from perched groundwater could occur.

4. Geotechnical Consideration for Proposed Development

4.1. Excavatability

A summary of the excavatability of the encountered soil and rock is contained in Table 2, and is suggested as a guide only. Excavation contractors should inspect the rock core, engineering logs and core photographs to make their own judgement as to likely productivity and specific plant.

Table 2: Guidelines for Excavation

Material	Likely Minimum Plant Requirements
Fill	Bulldozer blade, excavator bucket
Residual to extremely weathered, very low to low strength sandstone	Bulldozer blade, excavator bucket
Highly weathered to moderately weathered, low strength sandstone	Bulldozer with ripper, excavator bucket. Higher strength zones may require a rock breaker
Moderately weathered to fresh, medium to high strength sandstone	Cat D10 or equivalent. Higher Strength bands may require a rock breaker.

The use of hydraulic impact hammers for bulk excavation, trimming the sides of excavations, and detailed excavation, will cause vibrations that could affect vibration sensitive structures and services.

Assessment of the potential impacts of excavation induced vibrations should be considered as part of detailed design and excavation planning. Additional discussion of the vibration monitoring requirements is contained in Section 4.5.

The above indications are not based on direct observation of the site and contractors should form their own assessment for selection of excavation equipment and estimation of production rates.

4.2. Groundwater Conditions

Based on the identified groundwater conditions it is expected that a drained basement will be feasible. It is considered likely that the finished level of the lower basement will be near of below the pre-development groundwater table.

For a drained excavation structure, permanent floor and wall drainage will need to be maintained throughout the life of the structure. It is expected that such a drainage system would include a sub-floor drainage blanket with slotted drainage pipes and sump and pump system with the ability to effectively back flush the system for long-term maintenance.

Alternatively a tanked basement could be employed with design for the maximum anticipated groundwater level.

Groundwater levels vary in response to rainfall and as a result of development in the area.

4.3. Excavation Support

4.3.1. General Excavation Support Requirements

Permanent excavation support is typically controlled by site constraints, tolerable ground movements and requirements to restrain poor quality rock. Table 3 contains a preliminary assessment of support options for the geotechnical units.

Table 3: Preliminary support options

Material	Support Options
Fill and residual to extremely weathered, very low to low strength sandstone	• Unsupported excavation at suitable batter slopes • Retaining walls • Soil nails • Mesh and shotcrete (minimum 75 mm thick), with adequate drainage for the Class IV sandstone.
Highly weathered to moderately weathered, low strength sandstone	• Retaining walls. • Pattern rock bolting in low strength sandstone and in fractured zones within the better quality rock • Mesh support by doweling and shotcrete (minimum 75 thick) or fibre reinforced shotcrete, with adequate drainage.

Material	Support Options
Moderately weathered to fresh, medium to high strength sandstone	• Fractured areas : Shotcrete and pattern bolting (allow 2m grid) • Unfractured areas: Spot bolting as needed The above will need to be assessed on site by geotechnical professional based on exposed condition of rock.

Support requirements should be assessed during excavation and Coffey recommends regular face mapping by a geotechnical professional at vertical intervals of not greater than 2 m.

4.3.2. Retaining Walls for Soil

Where excavations cannot be battered, soil and more weathered sandstone could be supported using shoring walls such as conventional soldier pile wall. Use of the existing basement walls could be considered subject to suitable support during demolition of the existing building.

Temporary anchor installation would require the permission of adjacent property owners where anchors cross boundaries or easements.

It is recommended that a detailed analysis be undertaken, including assessment of surcharge loads, to develop a suitable retention support system. As a guide, Table 4 below presents typical design parameters for retaining wall design. These parameters should be reviewed following geotechnical investigation of the site.

Table 4: Parameters for retaining wall design

	Bulk unit Weight γ (kN/m3)	**'Active' Earth Pressure Coefficient, K_a**	**'At Rest' Earth Pressure Coefficient, K_0**	**'Passive' Earth Pressure Coefficient, K_p**	**C' (kPa)**	**ϕ' (degrees)**
Fill	20	0.4	0.5	2.5	0	25
Residual Soil and Extremely Weathered Sandstone	20	0.27	0.5	3.7	30	35

Active earth pressure coefficients should be adopted where wall movements of about 1% of the wall height can be tolerated. At rest pressure coefficients should be adopted where less movement can be tolerated. However, it should be understood that a well-constructed wall will still undergo movements of the order of 0.1% to 0.3% of the wall height where at rest pressures are adopted.

Applicable surcharge loads should be added to earth pressures.

4.3.3. Support for Better Quality Rock

Vertical cuts are generally feasible in Class III or better Hawkesbury Sandstone without support from a retaining wall. Rock bolt support, possibly supplemented with shotcrete and mesh, may be necessary in sections of the excavated rock faces below any existing shoring walls in order to retain fractured zones of rock.

To assess final shoring requirements an experienced geotechnical engineer or engineering geologist should carry out regular inspections as the excavation progresses. To assess the need for bolting and rock face support (for short and long term safety) it is recommended that the rock faces be assessed every 2 m depth on all excavated faces.

Where long-term support is required below the site retention system rock bolts must be provided with a high level of corrosion protection if they cannot be maintained (i.e. inspected and replaced, if necessary). Stainless steel bolts or multiple layers of corrosion protection such as encapsulating plain or galvanised bolts in both grout and PVC sheaths may be required.

4.3.4. Rock Anchors

Temporary anchors should be inclined downwards to anchor in the better quality sandstone.

The actual design load capacity of anchors should be based on a performance specification, verified by proof-testing.

Where subsequent rock excavation exposes the toes of the retention system piles, rock bolts may need to be installed above the toe of the pile to provide restraint and local shotcrete and mesh may be required to support and protect the foundation of the piles.

4.4. Foundation Design

It is expected that bulk excavations for the proposed development are expected to expose predominantly sandstone of good quality. It is likely that column loads for the proposed development may be supported using pad, strip or piled footings founded on sandstone bedrock.

Table 5 below presents serviceability and Limit State geotechnical design parameters that may be used for design of pad footings and bored piles into the different classes of sandstone.

Table 5: Geotechnical Foundation Design Parameters for Sandstone

Unit	Serviceability End Bearing Pressure (MPa)	Ultimate End Bearing Capacity (MPa)	Ultimate Shaft Adhesion (kPa)	Young's Modulus (MPa)
Class IV Sandstone [d]	1 [b]	8 [b]	250 [a]	400
Class III Sandstone [d]	5 [b]	20 [b]	800 [a]	1000
Class II Sandstone [d]	8 [c]	30 [c]	1,500 [a]	2,000

a) *For piles, shaft adhesion should only be assumed where piles have a minimum socket of at least 1 pile diameter and a clean socket of roughness category R2 or better is required. Values may have to be reduced if wall smear or polish is present.*

b) *Assumes that at least 40% of footings are proved by core drilling or spoon testing.*

c) *Assumes that the ground condition for each footing is proved by core drilling or spoon testing.*

d) *Rock classification according to Pells et.al (1998)*

For pad footings either a working stress or limit state design method could be adopted. For piles a limit state design method should be used if the design is to comply with AS2159-2009 “Piling – Design and installation”.

The above parameters should be confirmed by a geotechnical professional onsite based on the ground conditions encountered.

Footings designed using the serviceability end bearing pressures given above should result in settlements of less than 1% of the least footing dimension.

In accordance with AS2159-2009, the geotechnical strength reduction factor, Φg, is dependent on assignment of an Average Risk Rating (ARR) which takes into account various geotechnical uncertainties, redundancy of the foundation system, construction supervision, and the quantity and type of pile testing. The assessment of Φg therefore depends on the structural design of the foundation system as well as the design and construction method, and testing (if any) to be employed by the designer and piling contractor.

To assist you with preliminary design we recommend Φg of 0.6 be adopted for footings on sandstone. The final selection of Φg should be reviewed by Coffey at the detailed design stage.

If foundations are to resist uplift, the ultimate shaft adhesion should be reduced by a factor of 0.7. Uplift piles should also be checked for an inverted cone pull-out mechanism.

4.5. Protection of Adjacent Buildings and Infrastructure

The proposed excavation will cause adjacent ground movements. Due to the abundance of heritage buildings surrounding the proposed excavation, it is important to undertake a study on the potential impact on the heritage buildings and other surrounding structures due to the proposed basement excavation.

Many factors can influence the size of these movements, such as ground conditions, design and construction quality.

For excavation in soils (fill, residual soil and extremely weathered rock), documented data has shown that for well-designed and constructed shoring, vertical and lateral movements can be about 0.1% to 0.3% of the retained thickness of soil. Likely ground movements should be assessed during design of the shoring system.

In rock excavation, lateral movement occurs due to relief of *in situ* locked-in horizontal stresses and must be considered as part of design. There are relatively high natural horizontal stresses within Sydney rock, the magnitude of which varies with rock quality. From our past experience of deep basements in Sydney, typical lateral movements range from 0.5 mm to 2 mm per metre depth of excavation, depending on rock quality and presence of bedding seams.

Lateral ground movements due to stress relief have been measured at distances of up to 1.5 to 2 times the basement depth from the edge of excavations. These typically show that movements can be up to 30% of the displacement around the excavation perimeter at a distance approximately equal to the excavation depth. Stress relief ground movements are unlikely to be significant at distances greater than twice the excavation depth. However, these approximations will be affected by local geological structures and should only be used as a rough guide.

The use of excavation plant such as impact hammers will generate vibrations that may affect any surrounding sensitive structures and buried services. Alternative excavation methods (such as saw cutting or rock grinding of sandstone) may be preferred at property boundaries to avoid over-breaking and to reduce vibrations caused by mechanical excavation. The vibration limits in Table 6 below are commonly recommended to reduce the risk of vibration damage to sensitive receptors.

Table 6: Ground Vibration Limits for Various Types of Structures

Type of Structure	Peak Particle Velocity (mm/s)
Historic buildings or monuments	2
Residential or low rise buildings in good condition	10
Reinforced concrete commercial and industrial buildings in good condition	25

It is recommended that a limit is selected considering the structure of concern. It should be noted that limits set by the relevant authorities may override these recommendations.

As-built information of the surrounding buildings will be required for the impact assessment, and to ensure any proposed wall reinforcements (such as ground anchors, if required) does not clash with the basement and foundation of the existing buildings. It is recommended that condition survey, and background settlement surveys of the surrounding buildings to be undertaken prior to, during, and after excavation.

5. Metro Rail Corridor Impact Statement

5.1. General

Careful consideration is required for the potential impact on the Sydney Metro tunnel, which is programmed to be constructed prior to the proposed development.

Based on available information, the following assumptions were made on the Sydney Metro tunnel in relation to the proposed development for our impact assessment:

- The top of the rail tunnel acquisition level is at RL 5.6 mAHD. The crown of the tunnel is anticipated to be 7 m below the top of the acquisition level (RL -1.4 m)
- The crown of the tunnel closest to the proposed excavation is anticipated to be approximately 7 m below the base of the excavation (based on a final basement floor level of RL 6.010 mAHD), and approximately 3 to 4 m horizontal distance beyond the footprint of the development
- The tunnel structure is outside the footprint of the proposed development, as shown on Figure 5. The proposed excavation for the development is anticipated to be outside the rail acquisition zone.

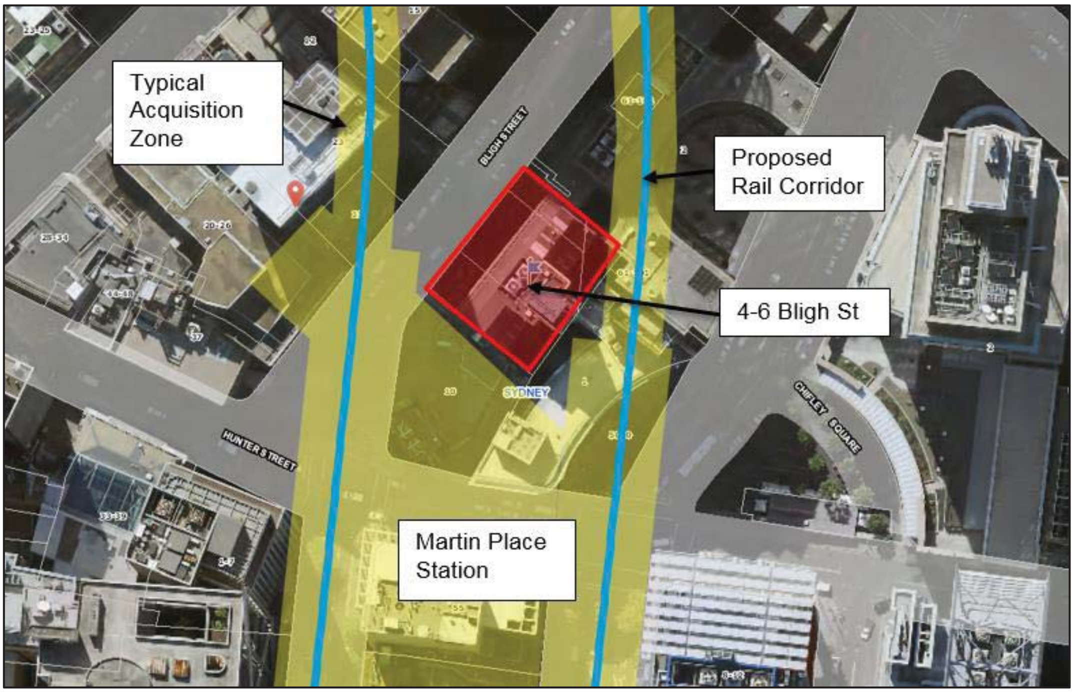


Figure 5: Indicative Location of Future Sydney Metro Rail Corridor

We are currently awaiting advice from Sydney Metro with regards to the exact locations and levels of the proposed track, and also the dimensions of the acquisition zone.

It is unlikely that the development will impose significant engineering challenges to the rail tunnel. However it is anticipated that review and approval by the relevant authorities will be necessary for any proposed works in the immediate vicinity.

Section 5.2 and Section 5.3 summarises our findings from our assessment of potential influences between the proposed development and the Metro Rail Tunnel.

5.2. Influence of the Proposed Development on the Sydney Metro Rail Tunnel

Interaction with the proposed Metro Rail infrastructure will be a source of project risk and we recommend liaison with the relevant authorities early in the design process.

The findings in this report indicate that the development is not located in an area of known major structural features such as major fault zones or igneous intrusions. It is inferred that the existing basement floor is underlain by sandstone bedrock. Similarly, the Sydney Metro tunnels in the vicinity of the development are likely to be constructed within sandstone of good quality.

Historical results from numerical analysis and inclinometer monitoring of the deep excavation undertaken for the World Square Project in the Sydney CBD (presented in Carter, et al. (1995) and shown on Figure 6) shows that the horizontal displacement below the excavation depth is very minor (less than 5 mm).

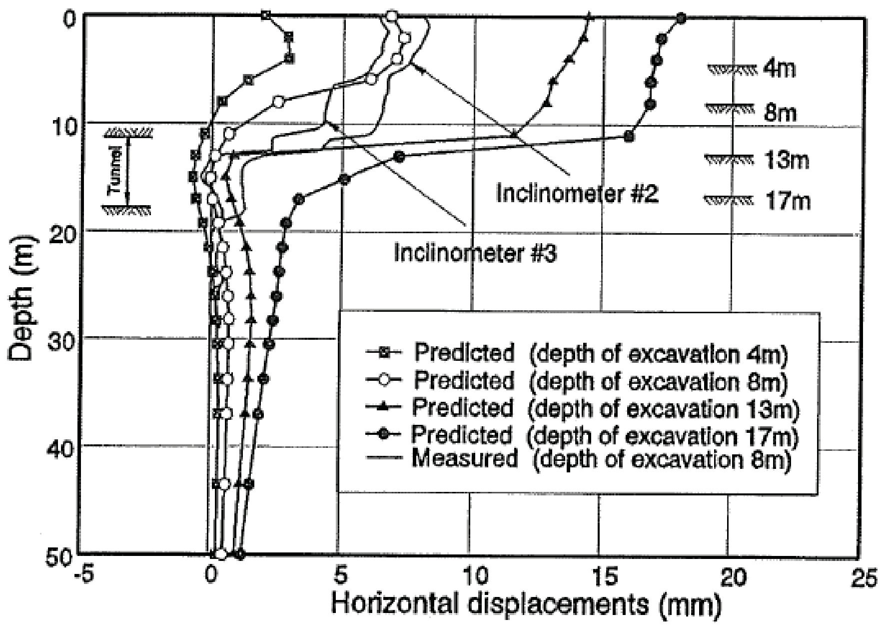


Figure 6: Numerical Analysis and inclinometer monitoring of the deep excavation undertaken for the World Square Project in the Sydney CBD (Carter, et al. (1995))

High increase in stress changes generally occurs near the corner of the basement of a deep excavation, as shown in Figure 7 for a 38 m deep excavation. From our experience with finite element modelling of deep excavations, the substantial stress increases generally within approximately 5 m below the bottom corner of the basement.

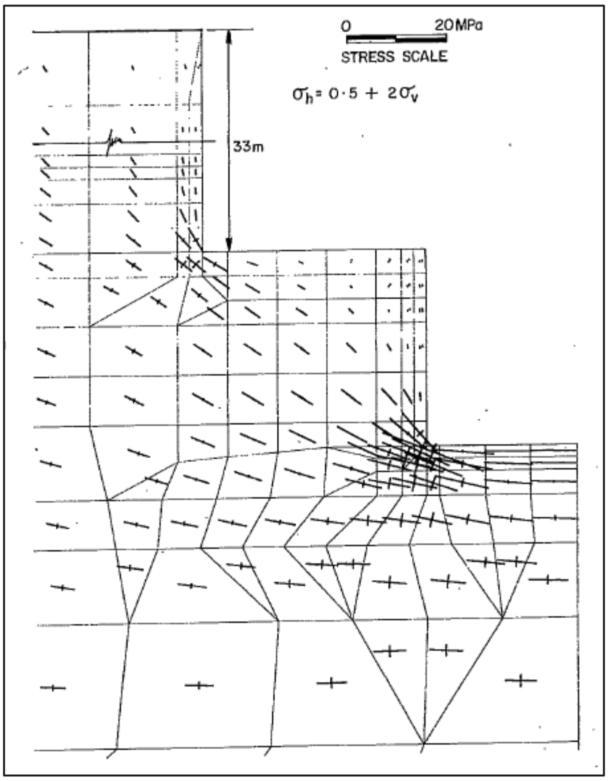


Figure 7: Stresses around a 38m Deep Excavation (from Pells, 1990)

Based on the findings above, we consider the influence of the proposed excavation on the Metro Sydney Rail would be minimal. A more detailed assessment of potential impacts on the Sydney Metro tunnels (and associated structures) will be undertaken at detailed design stage of the basement once more definitive information is received.

We have noted a potential underground structure running adjacent to the south western boundaries of the site, which connects the Martin Place Station to Bligh Street. Information regarding this underground structure will be required during detailed design stage for to assess the interactions with the proposed development.

5.3. Influence of the Metro Rail Tunnel on the Proposed Development

The highest stress changes due to the tunnel construction are expected to occur near the tunnel boundary, diminishing with distance from the tunnel.

Terzaghi and Richart (1952) have produced a plot showing the normal stresses adjacent to a circular tunnel along the major and minor principal stress axes and is shown on Figure 8. Key points to note are as follows:

- Stress changes in the major axis becomes insignificant at approximately one and a half tunnel diameter away from the tunnel boundary
- Stress changes in the major axis becomes insignificant at approximately one and a half tunnel diameter horizontal distance away from the tunnel boundary
- Stress changes in the minor axis becomes insignificant at approximately one diameter vertical distance above the tunnel boundary.

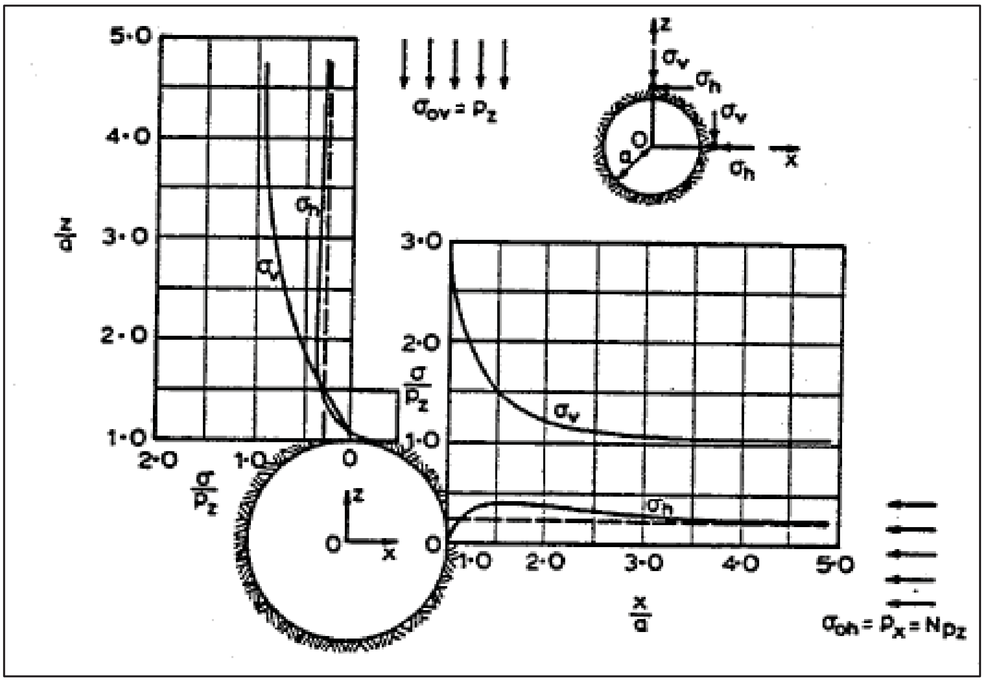


Figure 8: Stress Distribution around a Circular Tunnel for $K_o = P_x/P_z = 0.25$ (Terzaghi and Richart, 1952)

Note that the plots above relates to an unlined tunnel opening. The impacts on stress in the surrounding rock would be reduced by the presence of a lining.

Based on the findings of Terzaghi and Richart (1952), we anticipate the tunnel is sufficiently away from the proposed development, and will have minimal effects on the construction of the proposed basement. A more detailed assessment of potential impacts on the Sydney Metro tunnels (and associated structures) is recommended as part of design studies.

6. Additional Investigations

It is recommended that additional investigations are to be undertaken to verify our findings from this desktop study, and to assess the presence and nature of the rock jointing. The impact of identified adversely oriented joints will be considered in the basement excavation design.

The additional investigation will also be used to assess whether any features from the GPO Fault Zone and Martin Place Joint Swarm intersects with the proposed development. The presence of such features may considerably reduce allowable bearing capacities for foundations, increase groundwater inflow, and present instabilities to the excavations.

A detailed geotechnical model will be developed for the basement excavation which will include the findings of the ground investigation together with borehole information from nearby sites.

We consider four boreholes down to a depth of 30 m below existing ground level would provide a sound bases for assessment of subsurface conditions for the design of the foundation and basement.

7. Monitoring

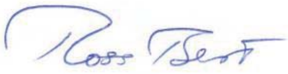
A geotechnical monitoring programme should be implemented during the construction phase as a check of design assumptions and to enable excavation support to be installed progressively as required by the revealed conditions. The programme should include, as a minimum, the following components:

- Monitoring of surface survey points located on existing structures, on any retaining wall, and on the ground surface at lateral distance from the excavation. Survey monitoring should be undertaken on a weekly basis during construction. Monitoring points should provide for accurate recording of both vertical and horizontal movements
- Undertake regular geotechnical assessments of exposed rock faces. Installation of rock face support as required
- Inclinometers to be installed at site boundaries to measure impact at the level of the Metro tunnels
- Vibration monitoring on vibration sensitive structures located close to the excavation, such as the adjacent buildings.

8. Closure

The descriptions of subsurface conditions described in this report are based on experience in the vicinity of the site. Ground conditions can change over relatively short distances. The recommendations of this report should be reviewed following a geotechnical investigation comprising drilling and groundwater measurements. In addition, assessment during construction with appropriate input from an experienced geotechnical engineer is also recommended.

The document entitled “Important Information about Your Coffey Report” presents additional information on the uses and limitations of this report.



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Attachments:

- Information about Your Coffey Report

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Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

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Important information about your **Coffey** Report

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way. Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction,
speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.

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