

# **APPENDIX K**

## **GEOTECHNICAL DESK STUDY - COFFEY**

Our ref: GEOTLCOV25104AA-AB

Mirvac  
Level 26,60 Margaret Street  
Sydney NSW 2000

Attention: Dominic Hunt

Dear Dominic

**Geotechnical Desk Study, Proposed Residential Development, 505 George Street, Sydney**

## 1. Introduction

This report presents the results of our initial geotechnical assessment for the planning proposal to amend the height from 150 metres to 235 metres for the residential tower to be constructed at 505-523 George Street in the Sydney CBD, NSW. This geotechnical assessment was commissioned by Mr Dominic Hunt of Mirvac. We understand that the proposed residential development comprises the construction of a 74 level, 260 m tall, residential tower with up to six basement levels.

The purpose of this study was to review existing Coffey archival borehole log information from previous investigations around the site and other publically available information to develop a preliminary site geotechnical model as a basis for general discussion on the geotechnical aspects and feasibility of the proposed development.

Our study was based on review of the following archived Coffey and published information:

- Coffey & Hollingsworth Pty Ltd Report S1325 (Feb.1965) for Greater Union site located to immediate north of Albion Lane.
- Coffey Report S6923/1 (1982) for MWS&D Building on George St.
- Coffey CBD Metro investigations for tender design including borehole drilling along George Street.
- Previous geotechnical investigation for the existing "Sydney Water" building on 115 Bathurst Street as supplied to Coffey for project work on the adjoining building.
- Genting Centre, Sydney – Deep Excavation Adjacent to Railway Tunnels, 1999, Hewitt et al, Proceeding's 8th Australia New Zealand Conference on Geomechanics, Hobart.

## 2. Site Description

The site location and footprint is shown in the attached Figure 1, Site Location Plan.

The site is located towards the southern end of a localised low ridge line that runs through the Sydney CBD from approximately Liverpool Street in the south, north east to Sydney Harbour, and contains Hyde Park and Oxford Street. The topography to the north and east of the site is relatively level. The topography to the south and west of the site slopes down to the south and west at approximately 2° to 4°.

Currently the site is occupied by the Events Cinema Centre. The cinema building is of a similar height as a four to five level office building and has one basement level for plant and equipment.

To the north the site is bounded by the Lumiere Building of 40 levels, (formerly known as the Genting Centre) and Fraser Suites building of 30 levels. We understand that these buildings have up to 8 levels of basement car parking.

To the south, the site is bounded by building also occupied by Event Cinema, and then a four to five level brick pub/hotel. On the opposite side of these buildings lies the Meriton Tower residential and office building. The second Event Cinema building matches the building type and height that is on the subject site. The Meriton Tower is of approximately 40 levels with a 10 level basement carpark.

To the east of the site, and passing beneath George Street, are six single track rail tunnels associated with the City Rail and Eastern Suburbs Rail tunnels. These brick lined tunnels are between 5.4 m and 17.5 m depth below George Street. It is understood (Hewitt 1999) that some of the tunnels may have been constructed as driven tunnels with the arched roof of the tunnels being unreinforced concrete.

## 3. Proposed development

We understand that the proposed site redevelopment will comprises the construction of a 74 level, 260 m tall, residential tower with up to six basement levels. Excavations for a six level basement are expected to extend up to 20.5m below existing ground surface levels.

From our experience in similar Sydney CBD projects it is expected the key geotechnical issues for the proposed redevelopment would be as follows:

- Protection of adjoining structures and the 'City Rail' tunnel Easement.
- Design of new building foundations.

## 4. Regional geology

The Sydney 1:100,000 Geological Sheet indicates that the site locality is underlain by the Hawkesbury Sandstone Formation, the geological contact with the Ashfield Shale of the Wianamatta Group is located to the east of the site and trending north-northwest near the intersection of George and Bathurst Streets.

Hawkesbury Sandstone is composed of predominantly medium to coarse grained quartzose sandstone typically comprising 1m to 3m thick beds. The major joint sets in the Hawkesbury Sandstone trend approximately north-south and east-west as an orthogonal pattern with a subordinate northwest-southeast trending set. The north-south trending joint set is the more dominant set (trending about 10° to 15° east of north), 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.

The Ashfield Shale which overlies the Hawkesbury Sandstone is described as dark grey to black claystone and siltstone, often with fine sandstone laminae. Between the Ashfield Shale and Hawkesbury Sandstone there is often a relatively thin discontinuous transitional unit called the Mittagong Formation comprising interbedded fine grained sandstone and siltstone.

Located to the east of the site and trending sub-parallel to George Street is the Martin Place Joint Swarm. The Martin Place Joint Swarm comprises a concentration of structural features such as faulting, sub vertical joint swarms and low angled thrust faults and the bedrock in such zones may be more weathered, of lower strength, and sometimes fragmented. In addition to the Martin Place Joint Swarm, previous investigations at the northeast corner of the intersection of George Street and Bathurst Street, and elsewhere in the CBD by Coffey identified low angle thrust fault features, of typically 50 mm to 150 mm thickness comprising broken rock material.

## 5. Initial site Geotechnical Model

Using the subsurface information from previous Coffey geotechnical investigations, published data and archived information, our proposed geotechnical units for the site have been developed to characterise the soil and rock strata and are presented in Table 1 below.

**Table 1: Proposed Geotechnical Units**

Unit	Geological Formation	Material Description	Rock Mass Classification <sup>1</sup>	Estimated Unit Thickness (m)
1	Fill/Residual Soil	Clays, high plasticity, stiff to hard consistency.	N/A	0.5
2	Ashfield Shale or Mittagong Formation	Extremely weathered rock	Class V	0.5 to 2.5
3	Hawkesbury Sandstone	Sandstone: Highly to moderately weathered, typically of low to high strength, fractured	Class IV some III	2.5 to 3.5
4	Hawkesbury Sandstone	Interbedded Sandstone & Siltstone: Highly to slightly weathered, low to high strength, slightly fractured to fractured	Class III Sandstone & Shale	6.5 to 7.5
5	Hawkesbury Sandstone	Sandstone: Slightly weathered to fresh, high strength, slightly fractured	Class II Sandstone and better	> 10m

<sup>1</sup> Rock classified as shale in accordance with Pells et al (1998) "Foundations on Sandstone and Shale in the Sydney Region" Aust. Geomech. Jnl, Dec 1998

The available information suggests that geotechnical conditions may vary across the site, with the Ashfield Shale (and possibly the Mittagong Formation) and underlying Hawkesbury Sandstone encountered within the northeast portion of the site. Further to the west and south (towards the Kent

Street boundary and Albion Place) the Ashfield Shale and Mittagong Formation may be thin or absent, with the Hawkesbury Sandstone anticipated over the full depth of the excavation.

Within the Hawkesbury Sandstone, investigation data from the Lumiere Building to the north of the site indicate the possible presence of an Interbedded Sandstone and Siltstone unit up to approximately 6.5m to 7.5m thick between approximately RL.12 m and RL.4 m. This unit has been interpreted by others as being part of the transitional Mittagong Formation, however, for the purposes of this preliminary study we have included this unit within the Hawkesbury Sandstone.

Although the Martin Place Joint Swarm is inferred to be located to the east of the site, the potential exists for more closely spaced sub-vertical joints to be located within the sandstone on or near to the eastern site boundary.

Regional groundwater is expected to be present at levels between RL0m to RL-10m, however the presence of deep basements to the north and south of the site, as well as localised drainage to the Cross City Tunnel and nearby railway tunnels are likely to have impacted and lowered the regional groundwater levels.

Groundwater seepage would usually be encountered at the soil/rock interface and in joints and bedding partings within the bedrock. Seepage in sandstone bedrock may be assumed as typically flowing downwards toward local drainage lines or regional water table, along horizontal bedding planes and sub-vertical joints. As the site is located toward the crest of a low ridgeline, resulting in a limited catchment area, near surface groundwater flows at the site are expected to be transient and rainfall dependent, rather than exist as standing water levels. The rock mass permeability will be governed by the joints, faults and bedding planes. Due to the anticipated relatively intact bedrock with tight defects across the site it is anticipated that the permeability of the sandstone will be relatively low.

## **6. Geotechnical considerations for the proposed development**

### **6.1. Basement excavation**

#### **6.1.1. Excavation works**

Excavations for the basements are expected to penetrate all soil and rock units and to terminate in Unit 5 Sandstone.

Unit 1 soils and Unit 2 Shale should be able to be excavated using a large excavator with a toothed bucket. Unit 3 Sandstone may be excavated with a large excavator fitted with rock teeth, however the lower Unit 4 and 5 Sandstone of predominantly high strength will be relatively difficult to excavate in the confined space of a basement excavation. Ripping is likely to be difficult and will require large excavation plant such as Class 300/400C dozers (Cat D10 or equivalent). Ripping productivity rates in the high strength sandstone will be low and may produce blocky material. If ripping proves to be impracticable, rock saws, impact hammers and milling machines could be used for all bulk and detailed excavation and trimming works.

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.

### **6.1.2. Excavation induced ground movements**

Ground movements induced by excavation of the proposed basements, have the potential to affect surrounding in-ground structures and services.

Within the retained fill/residual soils, the magnitude of adjacent ground movements will depend on the ground conditions, design lateral pressure, shoring system adopted, construction sequence and workmanship. Documented data has shown that for well-constructed shoring, vertical and lateral movements may be in the order of 0.1% to 0.3% of the retained thickness. Detail analysis should be carried out to assess likely ground movements when designing the appropriate shoring system.

Where it is important to limit adjacent ground movements due to the presence of nearby structures supported on high level footings, the use of a relatively stiff shoring system with bracing and/or tie-back anchors designed to resist higher than active earth pressures may be required. We suggest that such cases be specifically addressed during detailed design when details of adjacent footings and loadings are known.

Horizontal stress relief in the bedrock will also result in ground movement. Based on past excavation experience in sandstone in the Sydney CBD, typical lateral ground movements at the excavation face of the order of 0.5 mm to 2 mm per metre depth of excavation may be expected, depending on rock quality and bedding.

Lateral displacements of retaining walls and rock faces may also result in vertical settlements of the surrounding ground. For preliminary assessment of impacts, we recommend that potential settlement be assumed to be equal to predicted lateral displacements. Typically, ground movements (lateral displacement and settlement) are greatest at the excavation face and decrease to negligible values at a distance of up to three times the excavation depth.

We recommend that the effects of stress redistribution and potential ground settlement in the vicinity of excavations should be assessed as part of the detailed design.

### **6.2. Groundwater**

We are aware of relatively deep basement excavations to the north and south of the site, but have no knowledge whether or not these basements have under-slab drainage in place. If such under-slab drainage systems are in place, groundwater inflows would be directed to and drawn down by these basements sumps.

Where excavations extend below the toe of any retaining walls, appropriate treatment of joints or other defects will be required to reduce the hydraulic connection to groundwater within the soils.

Groundwater inflows through the bedrock are not expected to be significant if the rock is relatively free of defects and there is not a strong hydraulic connection to the overlying soils. Minor groundwater inflows during excavation from the bedrock should be able to be managed by a sump and pump drainage system. Should unacceptably high groundwater inflows occur during excavation, targeted grouting could then be used to reduce inflows.

Groundwater seepage into the proposed basement could be collected from the perimeter walls and floor and directed to an internally located holding tank or pit. Licencing and approvals may be required from authorities such as Council and NSW Office of Water to collect and release groundwater inflows into the sewer system.

### 6.3. New building foundations

For the design of new building footings with the anticipated high foundation loads it is expected that pile footings into moderately weathered, or better sandstone will be required. On the basis of our review of ground conditions it is expected that the moderately or less weathered sandstone underlying the site would typically be of Class III or better quality Sandstone.

As a preliminary for pad footings and bored piles into sandstone Table 2 below presents indicative serviceability, and Limit State geotechnical design parameters.

**Table 2: Preliminary 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)
Unit 4 Class III Sandstone	6 <sup>B</sup>	20 <sup>B</sup>	1,000 <sup>A</sup>	1000
Unit 5 Class II and I Sandstone	10 <sup>C</sup>	80 <sup>C</sup>	3,000 <sup>A</sup>	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.
- C. Assumes that the ground condition for each pile is proved by core drilling.
- D. Higher values may be feasible if specific foundation settlement is carried out.

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".

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

In accordance with AS2159-2009, the geotechnical strength reduction factor,  $\Phi_g$ , 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  $\Phi_g$  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 a  $\Phi_g$  of 0.6 be adopted for footings on Sandstone. The final selection of  $\Phi_g$  should be reviewed by an experienced geotechnical engineer 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 pullout mechanism.

#### **6.4. Protection of adjoining buildings**

For the protection of adjoining buildings and basements the type of underground structures, location, layout, and depth should be determined at the commencement of project design works. This information could then be used in conjunction with available information on site ground conditions and the results of any subsequent investigations for geotechnical assessments to determine whether the new development may affect existing structures. For this proposed development scheme Coffey expects that a design issue will be whether new building footings at a higher level than adjoining basement floors would surcharge adjoining deeper basement structures.

Where new footings/foundation loads may affect existing structures then detailed geotechnical assessments into the nature and magnitude of the potential effect will be required. Depending on the complexity of the geotechnical problem analytical methods would range from a simple empirical assessment, through to 3-dimensional finite element analyses and consultation with the project structural engineers to assess possible load influences, resulting ground movements/stresses, and additional support requirements.

Possible measures to mitigate the effect of the proposed foundations on existing basements that could be adopted include:

- Deeper founding levels for bored piles, and/or
- Sleeving of bored piles to below basement lowest floor level.

#### **6.5. Protection of nearby rail tunnels**

Along the George Street boundary, the location, i.e. depth and set back, of the rail tunnels will have a major influence on the choice of retention systems used here. Depending on final development plans, an anchored contiguous bored pile wall could be expected to be feasible for the support of Unit 1 and 2 materials along the George Street boundary. Permanent support of the retaining wall could then be provided by floor slabs isolated acoustically to reduce noise effects from the railway. Depending on actual ground conditions, below these depths, the bedrock could be cut vertically with permanent support of the excavation face comprising a system of ground anchors, rock bolts and where required mesh reinforced shotcrete.

Depending on final excavation depths, consideration will need to be given to estimation of potential ground movements and distortion around the railway tunnels from the proposed works. Such modelling utilising two-dimensional plain strain finite difference programs such as FLAC or three-dimensional finite difference programs such as PLAXIS would be required. This numerical modelling would need to be carried out following completion of geotechnical site investigations, so that major rock mass discontinuities could be modelled.

The following figure overleaf from Hewitt (1999) relating to the Genting Centre Development (now Lumiere Building) located to the north of the site, provides a summary of measured displacements resulting from the excavations carried out at that site. In particular, the maximum displacement recorded at the site was 15mm into the excavation at the base of the existing Cinema Centre. With regard to the impact on the nearby tunnels, displacements of up to 8 mm towards the excavation were recorded, with maximum displacements of the tunnel roof of 3 mm downwards.



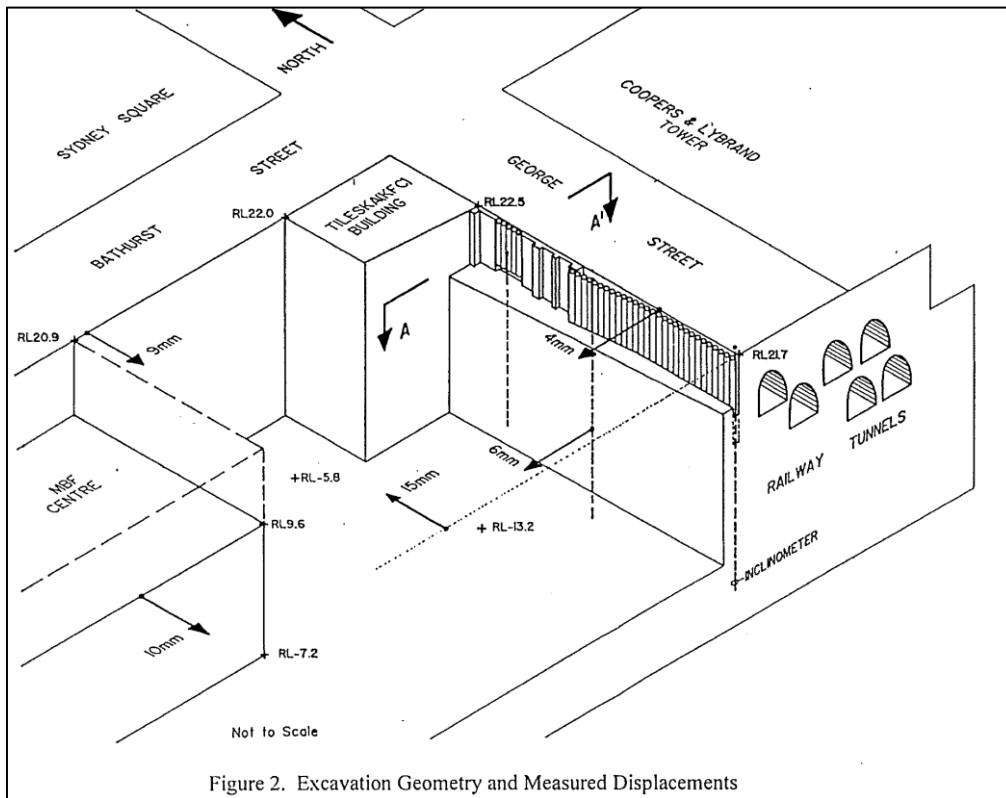


Figure 2. Excavation Geometry and Measured Displacements

As part of the construction phase work, instrumentation would need to be installed along the edge of the basement footprint and possibly within the nearby tunnels to monitor progress of the works. Such monitoring may include inclinometers, tunnel survey, tunnel convergence monitoring, vibration monitoring and surface survey monitoring.

### 6.6. Intrusive geotechnical site investigations

Given anticipated generally uniform subsurface conditions at the site, and our knowledge of adjoining developments, intrusive geotechnical investigations involving the drilling of cored boreholes will be required to support building design works.

In particular, this work will be required to obtain further information of ground conditions adjacent to the George Street boundary and the nearby railway tunnels. Following finalisation of the pile footing layout and design, and in consultation with the project structural designers, further foundation proving boreholes at selected footing location requiring bearing on Class II or Class I sandstone would also be necessary.

We recommend that a staged investigation be carried out, with the Stage 1 Investigations to comprise the following:

- Drilling of three cored boreholes adjacent to the George Street boundary, to at least 3m below proposed basement levels or the invert of the lowest nearby rail tunnels.
- Drilling of three cored boreholes on Kent Street to at least 3m below proposed basement excavations in this area.
- Carry out geotechnical laboratory Point Load Strength Index and UCS/modulus strength testing of rock core samples.

- Prepare a detailed site investigation report presenting the investigation results together with recommendations and geotechnical design parameters to support detailed design of project elements.

Subject to input from the project structural designers, we envisage that the Stage 2 geotechnical investigations may comprise the drilling of additional cored boreholes within the footprint of the site following demolition of the existing buildings. The purpose of Stage 2 borehole drilling investigation would be to confirm design assumptions and ground conditions at proposed footing locations. The location, depth and number of boreholes required would be dependent on the final footing and basement design/layout.

## 7. Conclusion

Based our site observations, preliminary geotechnical model, and experience on similar projects, the proposed development is considered feasible from a geotechnical perspective. The proposed development is assessed to have a low impact geotechnically on surrounding structures and the groundwater environment, provided appropriate additional site investigation, design assessments, and construction monitoring normally associated with this type of development are carried out.

## 8. Closure

The description of subsurface conditions is based on a desk top study, site surface observations, published geology maps, and our experience in similar projects. The preliminary geotechnical model and geotechnical engineering comments/advice presented in this report are based on professional judgment, and should be revised following intrusive site investigations and laboratory testing.

The attached document entitled "Important Information About Your Coffey Report" presents additional information on the uses and limitation of this report.

Should you have further questions or require further information please contact the undersigned on 9406 1000.

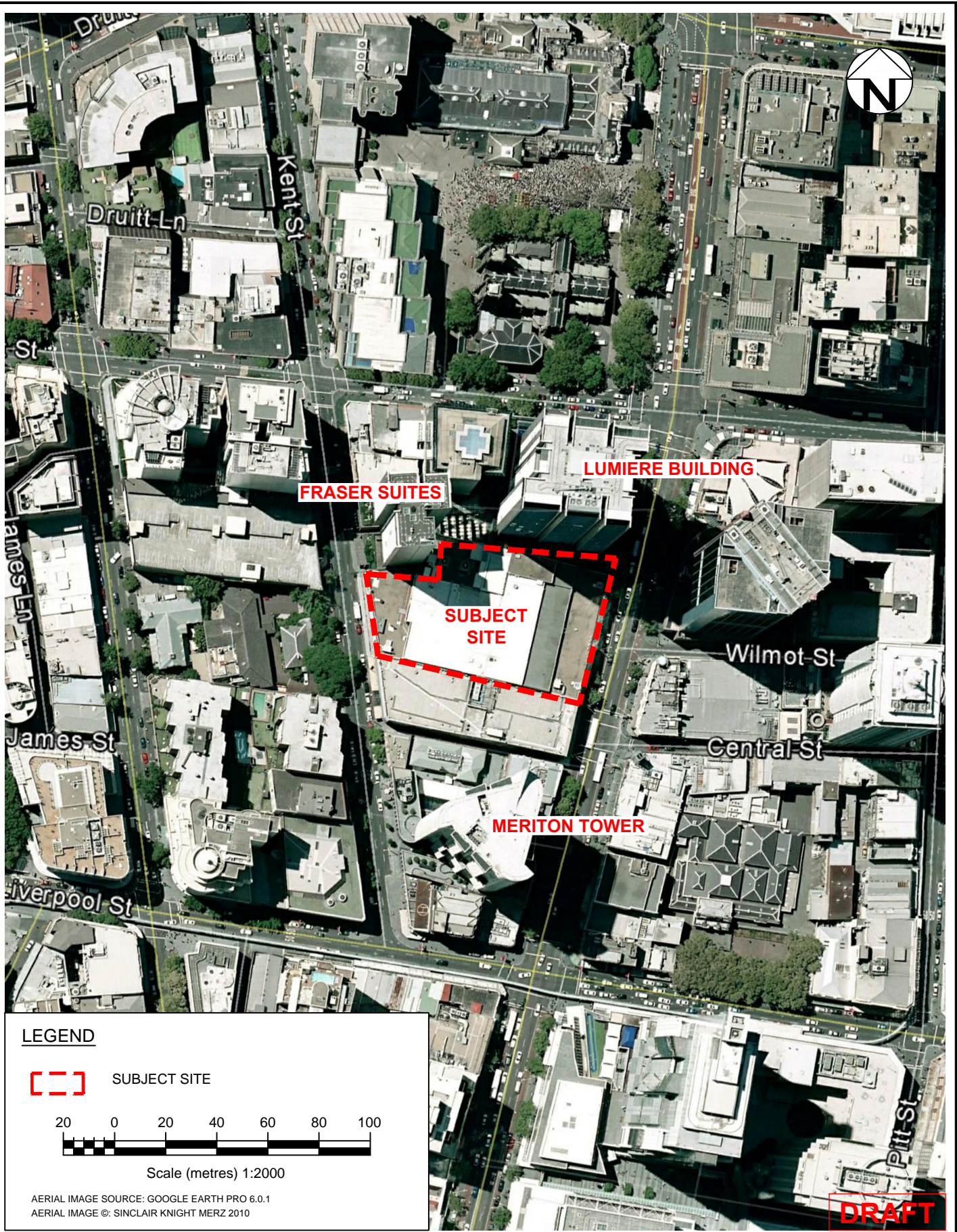
For and on behalf of Coffey



**Sven Padina**  
Associate Geotechnical Engineer

Attachments: Important Information about Your Coffey Report

Figure 1 Site Locations Plan



**DRAFT**

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drawn	SP / LH
approved	-
date	14 / 04 / 14
scale	1:2000
original size	A4



client:	MIRVAC		
project:	PROPOSED RESIDENTIAL TOWER 505 GEORGE ST, SYDNEY, NSW		
title:	SITE LOCATION PLAN		
project no:	GEOTLCOV25104AA	figure no:	FIGURE 1
		rev:	A

## 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.

## Important information about your **Coffey** Report

### **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.

### **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.