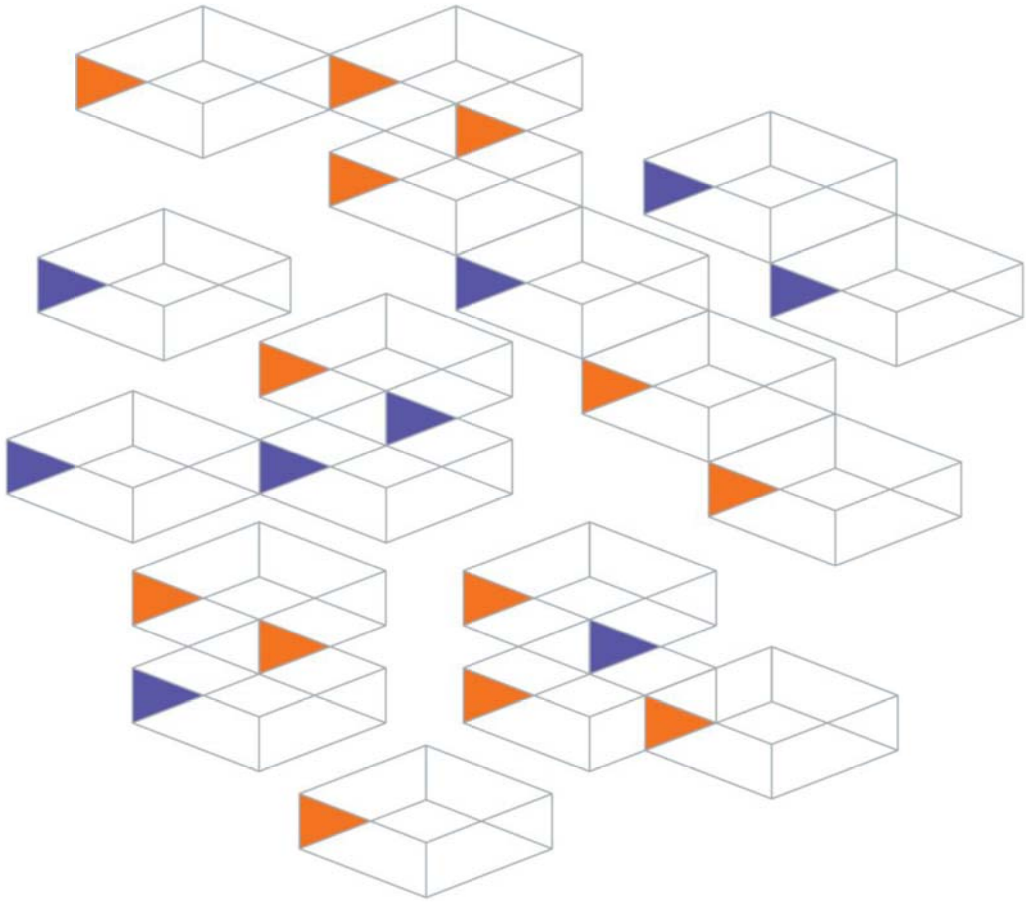
Karimbla Construction Services Pty Ltd

230 - 238 Sussex Street, Sydney, NSW

Geotechnical Desktop Study

16 June 2015



Boundaries
are set by those
who are afraid
to push them

230 - 238 Sussex Street, Sydney, NSW

Prepared for
Karimbla Construction Services Pty Ltd

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16 June 2015

Document authorisation

Our ref: GEOTLCOV25423AA-AB

For and on behalf of Coffey

Bernice Cahill
Geotechnical Engineer

Quality information

Revision history

Revision	Description	Date	Author	Reviewer	Signatory
0	GEOTLCOV25423AA-AB	16/06/2015	Bernice Cahill	Robert Turner	Bernice Cahill

Distribution

Report Status	No. of copies	Format	Distributed to	Date
0	1	PDF	Karimbla Construction Services Pty Ltd	16/06/2015

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1. Introduction

This report presents the results of our initial geotechnical assessment for the proposed development at 230-238 Sussex Street, Sydney CBD, NSW. The proposed development will comprise a 30 level mixed use residential/commercial building with up to 4 levels of basement. This geotechnical assessment was commissioned by Karimbla Construction Services Pty Ltd (Karimbla).

The purpose of this study was to review previous Coffey investigations in the locality 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. From our experience in Sydney CBD projects, the key geotechnical issues for the project are expected to be:

- Design of new building foundations.
- Excavation design and planning.
- Protection and support of the existing heritage building and neighbouring structures.
- Design to ensure that the development will not impact or limit future development of the CBD Rail Link tunnel reserve.
- Scoping and completion of further geotechnical investigation and design assessments to support project delivery.

2. The site and proposed development

The study site is situated at the corner of Sussex Street and Druitt Lane, approximately 200 m east of Cockle Bay Wharf. The site is rectangular, measuring approximately 50m north/south and 40m east/west. A plan of the site is attached as Figure 1. The site topography slopes down towards the wharf and ground elevation ranges from approximately 11 m AHD to the east to 9 m AHD in the west.

The site is currently occupied by two structures, a multi storey commercial building with no basement to the south and a three storey heritage structure with a single level basement to the north. The proposed CBD Rail Link (CBDRL) traverses the site in a north-northwest/south-southeast direction. The site falls within the interim rail corridor nominated by the NSW Department of Planning for the future CBD Rail Link (refer Interim Rail Corridor Map 7 of 9 provided in Appendix A).

The redevelopment will retain the heritage structure to the north of the site. To the south, a 30 storey mixed use tower with up to four basement levels is proposed. Basement excavations are expected to extend to approximately 12 m below existing ground level. The supplied drawings indicate that it is not proposed to extend the basement under the existing heritage structure.

3. Geotechnical Model

3.1. Information sources

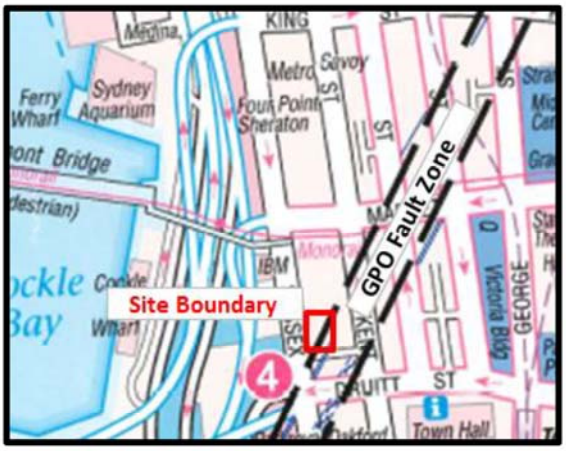
Our geotechnical model was developed from published geological data and previous Coffey investigations at the following sites:

- 252 Sussex Street Geotechnical Investigation to the immediate south of the site.
- 286 Sussed Street Geotechnical Desktop Study (120 m south of the site).
- Cockle Bay Marine Wharves Geotechnical Investigation (200 m west of the site).
- Cnr of Market, Clarence and York Street Geotechnical Investigation. (180 m northeast of the site)

3.2. Local geology

The Sydney 1:100,000 Geological Sheet indicates that the site locality is underlain by the Hawkesbury Sandstone Formation. Hawkesbury Sandstone is composed of predominantly medium to coarse grained quartzose sandstone typically comprising 1 m to 3 m 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 1 m to 5 m. The east-west trending joints tend to be spaced at 5 m to 15 m intervals.

The GPO Fault Zone traverses the southeast site corner and trending in a southwest to northeast direction as shown in below.



The GPO Fault Zone comprises a concentration of near vertical faulting or closely spaced joints with sympathetic, sub-parallel, isolated faults located on either side of the main zone of faulting.

Groundwater seepage would typically be present at the soil/rock interface and in bedrock joints and bedding partings. The local groundwater table is expected to be at a similar level to water levels in Cockle Bay approximately 0 m AHD; however it is not expected to fluctuate with tidal water level changes.

3.3. Preliminary geotechnical model

The geotechnical units in Table 1 below have been developed from the listed information as a preliminary characterisation of the soil and rock strata below the site. For the inferred distribution of the various geotechnical units across the site, reference should be made to the geotechnical sections in Figure 2

Table 1: Proposed Geotechnical Units

Unit	Geological Formation	Material Description	Rock Mass Classification[1]	Estimated Unit Thickness (m)
1	Fill/Residual Soil	Clays, high plasticity, stiff to hard consistency.	N/A	5
2	Hawkesbury Sandstone	Sandstone: Moderately weathered, typically medium strength	Class III or better	3
3		Sandstone: Slightly weathered to fresh, high strength, slightly fractured	Class II or better Sandstone	> 10

[1] Rock classified as shale using the classification system by Pells et al (1998) “Foundations on Sandstone and Shale in the Sydney Region” Aust. Geomech. Jnl, Dec 1998.

The geotechnical conditions are likely to be relatively variable across the site due to the presence of the GPO Fault Zone.

Groundwater is expected to be present at levels between RL 0 m to RL -10 m, however localised drainage to surrounding basements may have impacted and lowered local groundwater levels.

Localised seepage would typically be encountered at the soil/rock interface and in joints and bedding partings within the bedrock. Seepage in the 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. The rock mass permeability will be governed by the joints, faults and bedding planes. Due to the expected variability in rock conditions across the site, it is anticipated that the permeability will also vary, with higher permeability in zones of “broken” rock or open defects associated with the GPO Fault Zone and lower permeability in areas of relatively intact bedrock with tight defects.

4. Geotechnical considerations

4.1. Basement excavation

4.1.1. Excavation works

It is expected that Unit 1 may be excavated using conventional earthmoving plant such as large excavators with toothed buckets.

Excavation within Units 2 and 3 would require the use of hard rock excavation techniques such as large dozers fitted with rippers, or large excavators fitted with rock hammers, rock saws, or rock grinders.

Ripping is likely to be difficult and would 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 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.

Excavation contractors should make their own judgement as to likely productivity, bulking factors, or specific plant requirements.

4.1.2. Excavation support

Where space permits excavation in Unit 1 soils may be temporarily cut at 1 Horizontal (H): 1 Vertical (V), up to a maximum depth of approximately 4m, provided there are no surcharge loads behind batter crests. If this is not possible then excavation retention will be required.

It is expected that vertical cuts in Units 2 and 3 would be feasible subject to the nature and orientation of rock defects. Rock bolt support in sandstone where there is jointing or faulting, and possibly shotcrete of shale/fine grained sandstone lenses may be necessary in some sections of vertically cut rock, with support requirements to be assessed and confirmed during excavation works.

The type of retention system for temporary and/or permanent excavation support will depend upon the requirements for a stiff and/or watertight wall. Where ground movements assessed for cantilevered walls are excessive, additional lateral support could be provided by temporary anchors installed progressively as the excavation proceeds. If anchors are to be installed beneath the adjacent properties, prior permission of adjacent property owners would be required. If the use of anchors is not possible, then top-down construction or internal bracing would then be required.

The design of retaining walls for multi-anchored or braced walls is geotechnically complex. The relative stiffness of the wall and support system chosen will strongly influence the resulting earth pressure magnitude and distribution. Earth pressure coefficients adopted for design will depend on the analytical tools utilised in the design, and whether the numerical analysis methods used allow for stress re-adjustment to occur with wall movements.

Retaining wall analyses will also need to consider surcharges, footing loads from adjacent structures, and hydrostatic pressures. If drained walls are to be used then adequate drainage will need to be provided behind the walls, and a permanent water collection system will be required together with flushing points for drainage system periodic maintenance. Nevertheless an allowance of potential water pressure build-up equivalent to one-third the wall height is considered to be prudent with such drainage measures installed.

4.1.3. Excavation induced ground movements

The proposed excavation will cause some ground movements. Many factors can influence the size of these movements, from ground conditions to design and construction quality. 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 stiff clay and medium dense sand soils. If this aspect is critical, we can assess (possibly by numerical analysis) likely ground movements during design of the shoring system.

Due to the sensitive heritage structure on site, a relatively stiff shoring with bracing and/or tie-back anchors designed to resist pressures higher than active earth pressures may be required. Such cases should be specifically addressed by Coffey during detailed design when adjacent footing layouts and loadings are known.

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 Hawkesbury Sandstone, the magnitude of which varies with rock quality.

From our 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.

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.

4.1.4. Groundwater

Depending on the depth of basement, excavations may extend below the anticipated groundwater level.

Based on Coffey experience with similar projects, it is expected that seepage through rock defects is likely to be controllable during construction by conventional sump pumping methods, with discharge to the stormwater network (subject to receipt of regulatory approval). However a detailed basement inflow assessment will be required to establish this.

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.

Approvals for the use of a drained basement would also be required from the NSW Office of Water. It is expected that for the approvals process detailed assessment and calculation of potential groundwater inflows/drawdown demonstrating whether the development satisfies the requirements for minimal impact to aquifers.

4.2. New building foundations

For the design of new building footings, it is expected that pad or 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.

Table 2 below presents indicative serviceability and Limit State geotechnical design parameters that may be used for preliminary design of pad footings and bored piles into sandstone.

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 2: Class III Sandstone	6 [B]	20 [B]	1,000 [A]	1000
Unit 3: Class II and I Sandstone	10 [C]	80 [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.*

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".

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 pullout mechanism.

4.3. Protection of adjoining buildings

For the protection of adjoining structures the type of structure, location, layout, and depth should be determined at the commencement of excavation 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 excavations may affect existing structures. 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 will be required to assess possible load influences, resulting ground movements/stresses, and additional support requirements.

The use of excavation plant such as impact hammers will generate vibrations that may affect any surrounding sensitive structures and buried services. Measures to mitigate the risks associated with vibration such as the use of rock saws or rock grinders should be considered. The vibration limits in Table 3 below are commonly recommended to reduce the risk of vibration damage to sensitive receptors.

Table 3: 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.

Dilapidation surveys should be carried out on neighbouring structures or sensitive services prior to commencing excavation as a baseline record of their condition. Excavation trials with vibration monitoring should also be carried out to assess appropriate distances for various excavation plant to be used to limit generated vibrations, and need for ongoing vibration monitoring during site works to confirm that the limits are not exceeded.

4.4. Protection of the rail corridor

At this stage we have not been supplied with any plans showing the layout and extent of the rail corridor first and second reserves. Assuming that excavations and footings are located outside the first and second rail reserves, and that appropriate additional site investigation, design assessments, and construction monitoring normally associated with this type of development are carried out, it is expected that the proposed development will not limit future development of the rail tunnel corridor.

4.4.1. Foundation Loads

We expect that the new buildings and structures would be supported on either pad footing, or bored piles, into the sandstone exposed at the bottom of basement excavations.

The ground conditions are anticipated to comprise Units 2 and 3 outside the fault zone with more weathered or lower strength jointed/faulted rock where the GPO Fault Zone is encountered.

In general the resulting footing loads on the rock mass are not expected to impose surcharge on the rail corridor provided they remain outside the rail reserves. However it is recommend that at detailed design stage a geotechnical assessment of the potential to surcharge the rail reserves be carried out.

4.4.2. Ground Deformation

The construction of tunnels induces ground movements, the magnitude of which is dependent on several factors, including tunnelling method, the quality of workmanship, ground and groundwater conditions and geotechnical properties. Ground movements can occur during and post-construction.

During the tunnelling process the permanent support to the tunnel is installed after excavation, therefore some movement occurs ahead of the excavation face. The permanent support can be passive in nature in that the ground deflects until the support takes up the load. This form of settlement is often described as being induced by stress redistribution within the rock mass through which the tunnel is being driven.

The future rail tunnels are likely to be constructed within medium to high strength sandstone with limited thicknesses of residual soil and fill cover therefore the majority of the settlement induced due to tunnel construction will be a result of stress redistribution within the rock mass. Excavation of a tunnel provides an opening into which the surrounding ground can deform. The movement of the ground into the opening can be related to the concept of “loss of ground”, which is defined as volume of material that has been excavated in excess of the theoretical design volume of excavation. Based on our tunnel construction experience in Sydney, volume loss of 0.2% to 0.3% may be anticipated in Class III or better sandstone.

At detailed design stage, an assessment of the changes in ground stress and potential ground movements associated with the future tunnel construction should be completed using appropriate empirical and/or numerical methods (e.g. finite element analysis).

4.5. Intrusive geotechnical site investigations

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 associated with the GPO Fault Zone. We expect that such an investigation would comprise the drilling of cored vertical and angled boreholes into the bedrock below proposed foundation locations/levels. The aim of the investigation would be to assess the bedrock quality for foundation design and delimitate the extent of and faulted or deeply weathered bedrock. Following finalisation of the footing layout and design, and in consultation with the project structural designers, further foundation proving boreholes at selected footing locations may also be necessary.

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

- Drilling of cored boreholes to at least 3 m below the proposed basement level.
- 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 building. 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.

5. Closure

The preliminary geotechnical assessment and recommendations of this report are based on a desk study limited to regional information and existing subsurface investigation data that is not located on the site.

Subsurface conditions can be complex and vary over relatively short distances – and over time. Site specific investigations will be required to support detailed design. Detailed design and construction should not proceed on the basis of this desk study report without further advice from us.

The attached document entitled “Important information about your Coffey report” forms an integral part of this report and presents additional information about it uses and limitations.

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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.

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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

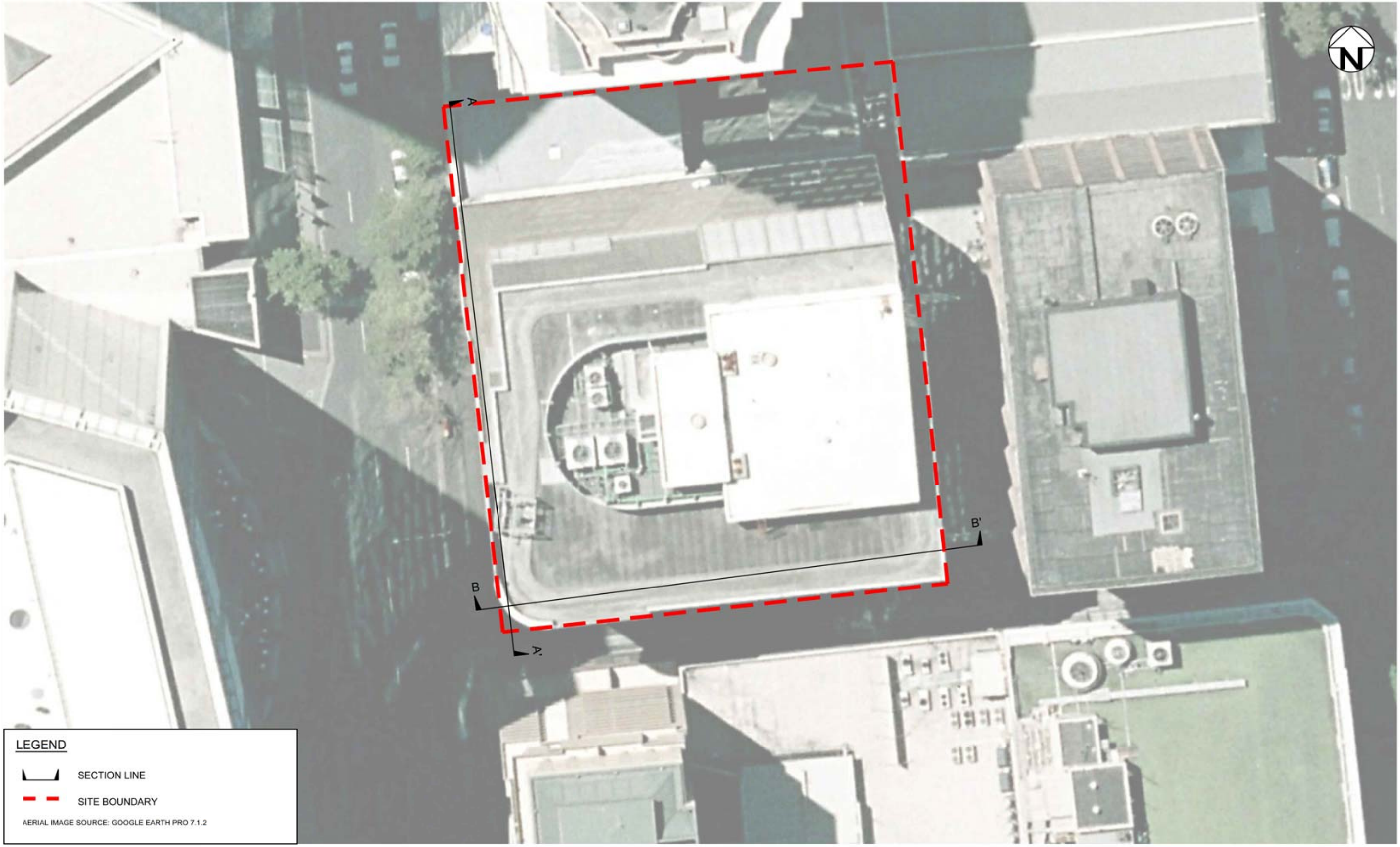
Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As 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.

Figures



LEGEND

SECTION LINE

SITE BOUNDARY

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.2

no.	description	drawn	approved	date
A	ORIGINAL ISSUE			

revision

5 0 5 10 15

Horizontal Scale (metres) 1:300

drawn	BC / JP
approved	-
date	15 / 06 / 15
scale	AS SHOWN
original size	A3

coffey

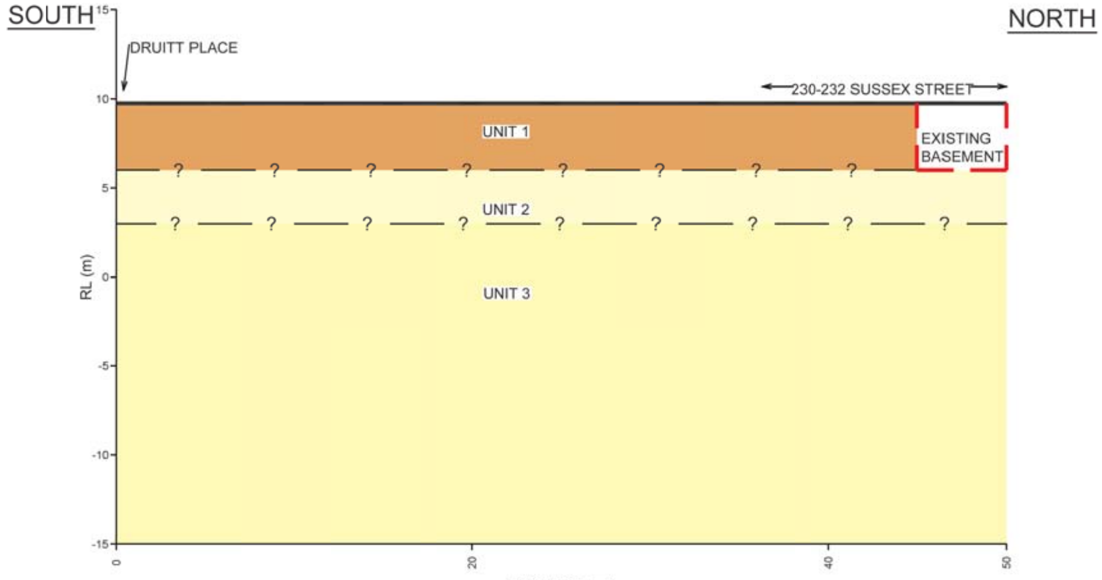
client: KARIMBLA CONSTRUCTION SERVICES PTY LTD

project: 230 - 238 SUSSEX STREET SYDNEY CBD, NSW

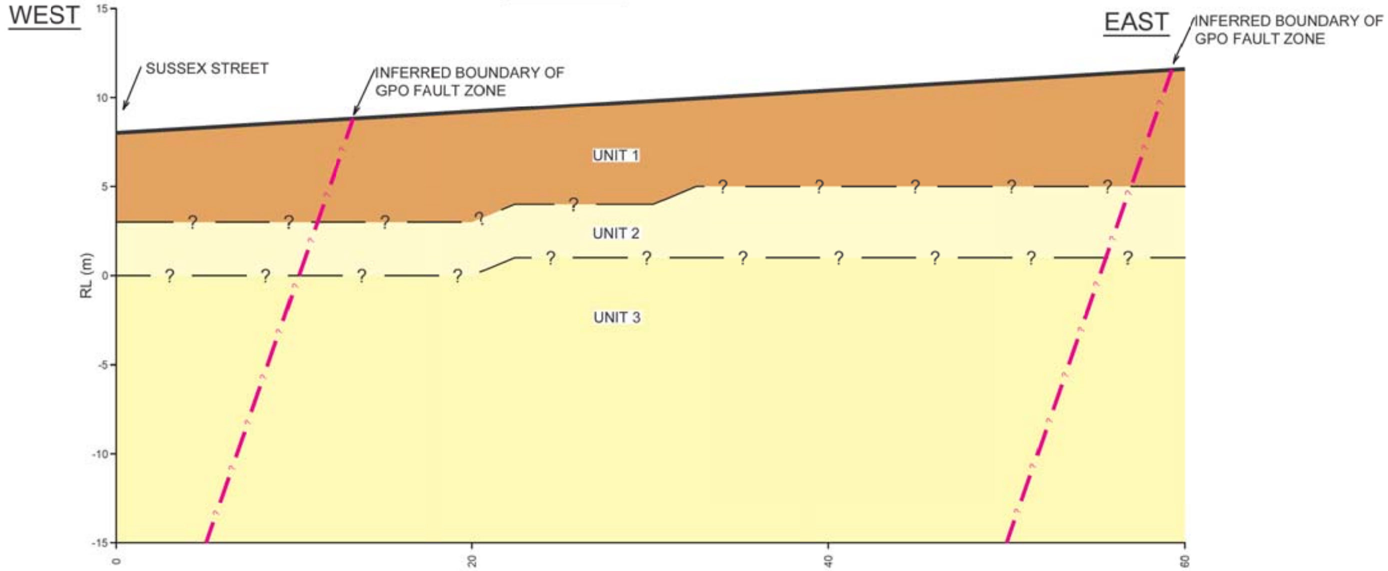
title: SITE PLAN

project no: GEOTLCOV25423AA-AB | figure no: FIGURE 1 | rev: A

PLOT DATE: 16/06/2015 11:03:00 AM DWG FILE: F:\GEOTECHNICS\1 PROJECTS\GEOTLCOV254\GEOTLCOV25423AA 234-238 SUSSEX STREET\CAD\GEOTLCOV25423AA.DWG



SECTION A-A'



SECTION B-B'

LEGEND

- UNIT 1 - FILL AND RESIDUAL SOIL
- UNIT 2 - MODERATELY WEATHERED SANDSTONE
- UNIT 3 - SLIGHTLY WEATHERED TO FRESH SANDSTONE

no.	description	drawn	approved	date
A	ORIGINAL ISSUE			

revision

Horizontal Scale (metres) 1:200

drawn	BC / JP
approved	-
date	15 / 06 / 15
scale	AS SHOWN
original size	A3

coffey

client:	KARIMBLA CONSTRUCTION SERVICES PTY LTD
project:	230 - 238 SUSSEX STREET SYDNEY CBD, NSW
title:	SUBSURFACE SECTIONS A-A' & B-B'
project no:	GEOTLCOV25423AA-AB
figure no:	FIGURE 2
rev:	A

PLOT DATE: 16/06/2015 12:32:01 PM DWG FILE: F:\GEOTECHNICS\1 PROJECTS\GEOTLCOV25423AA 234-238 SUSSEX STREET\CAD\GEOTLCOV25423AA.DWG

Appendix A : Referenced Drawings

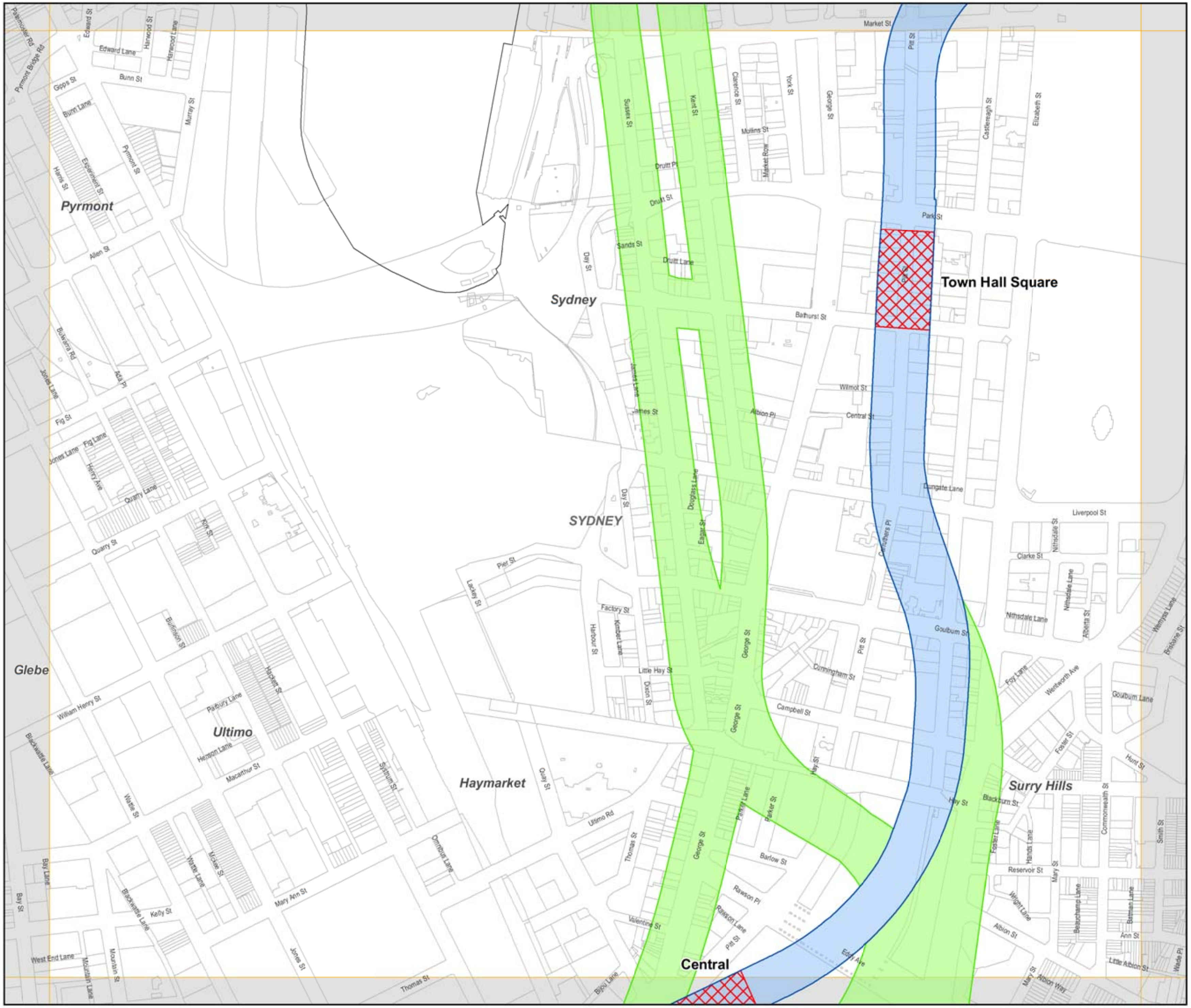
NSW GOVERNMENT
Department of Planning

State Environmental Planning Policy (Infrastructure) 2007

Interim Rail Corridor
CBD Rail Link & CBD Metro
Map 7 of 9

- CBD Rail Link (Zone B - Tunnel)
- CBD Metro (Zone A - Above Ground Including Cut & Cover Tunnel)
- CBD Metro (Zone B - Tunnel)
- CBD Station Extent
- LGA Boundary
- Land Title Boundary 20/07/08 © Dept of Lands

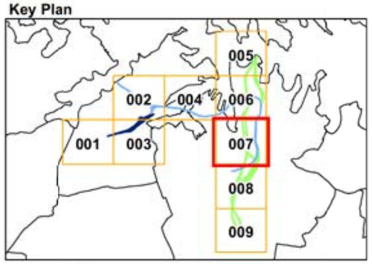
The CBD Metro is stage 1 of the Sydney Metro Network



Minister for Planning

Date: ....

Key Plan



N

0 50 100 Metres

Scale: 1:5,000 @ A3

Projection: GDA 1994
MGA Zone 56

CERTIFIED BY:

SydneyMetro

Date: ....

Map identification number:
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