Attachment A10

Geotechnical Assessment

REPORT TO

DEXUS CPA PTY LTD

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED TOWER DEVELOPMENT

AT

56 TO 60 PITT STREET & 3 SPRING STREET, SYDNEY, NSW

Date: 5 April 2024

Ref: 33070BMrptRev1

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1 INTRODUCTION

This report presents the results of a geotechnical desk top study assessment for a proposed tower development at 56 to 62 Pitt Street and 3 Spring Street, Sydney, NSW. The location of the site is shown in Figure 1.

From the supplied ‘For Information’ architectural drawings (Ref. PP- 9.2.01 to 9.2.09, 9.3.1, 9.3.4 and 9.4.1, all dated 31 January 2024) prepared by fjcstudio, we understand it is proposed to demolish the existing buildings and construct a multi- storey tower to a height of about 300m to 304m over a ground floor, lower ground floor plus 2 to 4 basement levels. The lowest basement level, B4, will be at RL -10.16m, about 18m to 20m below existing ground surface levels requiring the full depth of excavation where there are no prior existing basement levels. The northern end of the proposed building extends over the existing City East Cable Tunnel and the proposed Sydney Metro (City) tunnel. These tunnels are shown on our Figure 2. The Sydney Metro Tunnel First Reserve (RL-5.150 to RL-24.270) and Second Reserve, and the "Easement for Substratum Tunnel” (RL-24.770 to RL-35.320) are shown on drawings 9.2.09 and 9.4.1 which also indicate the northern extents of the lower basement levels, B3 and B4, are restricted by the inferred Sydney Metro Tunnel First Reserve.

The purpose of this desk top study assessment was to obtain geotechnical information on likely subsurface conditions from a previous geotechnical investigation completed nearby as a basis for preliminary comments and recommendations on development in relation to tunnels, excavation, retention, groundwater, footings, basement slabs, geotechnical investigation and analysis, and monitoring.

2 ASSESSMENT PROCEDURE

The assessment involved the following:

- A search of our project database for previous geotechnical investigations we have completed in the vicinity of the subject site and a desk top study of the results of those geotechnical investigations;
- Review of the published information including geological maps; and
- A walkover of the site and its surrounds by our Associate Geotechnical Engineer on 6 April 2020.

The assessment did not include application to the tunnel authorities for details of their assets.

3 RESULTS OF INVESTIGATION

3.1 Site Description

Located relatively central to the ‘width’ of the Sydney CBD, the site is about 300m south of Circular Quay. Its surrounds fall gently down north-westwards at about 2° to 3° to Pitt Street which appears to be aligned along a shallow ‘valley’ depression stretching down towards Circular Quay.

The site is bound by Pitt Street to the west, Bridge Street to the north, Gresham Street to the east, Spring Street to the south-east and No 62 Pitt Street to the south. It has an irregular shape extending about 90m north to south and up to 45m east-west. Surface levels drop about 3.4m from the eastern side (corner of Spring and Gresham Streets) to the north-western corner (junction of Pitt and Bridge Streets). Surface levels around the site range from about RL5.65m to RL9.4m, based on the provided survey plan by Project Surveyors, Ref. B04580-DETAIL-1, dated August 2019.

The site is currently occupied by four buildings ranging from about 12 to 28 above ground storeys. The various building addresses within the site are shown on Figure 2. Ramps off Gresham Street and Pitt Street into No. 56 Pitt Street (also known as 21 Bridge Street) indicate at least one basement level beneath this property, under the northern half of the site. There is also a steep lane (Little Spring Street) providing vehicular access to a basement under No 3 Spring Street, and to a fire escape at the rear of No 58 Pitt Street.

The site does not include the property in the southern corner of the block which is fully occupied by an 8-storey building (Bank of Sydney).

Footpaths around the site are stone paved with some small trees within each footpath, except on Bridge Street. The roads are asphaltic concrete surfaced.

As shown on Figure 2, two tunnels pass under the northern end of the site. The City East Cable Tunnel, operated by Ausgrid, crosses from east to west and is thought to be about 40m deep. We understand it is about 4m diameter and concrete lined. The Sydney Metro City tunnel is aligned north-west to south-east, across the north-eastern corner of the site. According to architectural drawing PP9.2.09, the 'First Reserve' (a zone in which no development, including footings, is permitted) is above RL-5.15m. The 'Second Reserve' (a zone within which development will be limited to prevent adverse effects on the tunnel) is indicated on Figure 2, extending 25m laterally from the first reserve.

The buildings within the adjacent blocks are generally multi-storey commercial buildings, some with basement carparks, but to the east is a three to four-storey heritage sandstone building (Lands Department, 23 Bridge Street.) Within 40m to the south-west of the site is the Australia Square high-rise building with multiple basement parking levels.

Before You Dig Australia (BYDA) plans provided by Sydney Metro indicate the top and bottom levels of the acquisitioned rail corridor, which match the levels shown on the architectural section. The tunnel itself is not detailed on the BYDA plans. No details are given for the second reserve.

The provided BYDA metro survey plans also indicate the deep Proposed Electrical Easement at RL-25m to RL-35m. The BYDA Ausgrid plan indicates the tunnel crossing the northern portion of the site but provide no details.

In close proximity to the north of the site, the architectural plan PP-9.2.08 shows a 'Telstra Tunnel Centre Depth RL-2.5m'. BYDA Telstra plans show near surface conduits but no tunnel.

3.2 Geology and Likely Subsurface Conditions

The Sydney 1:100,000 geological map of Sydney indicates the site to be underlain by Hawkesbury Sandstone and man-made fill, the latter being associated with infilling of the Tank Stream behind Circular Quay. The fill may comprise dredged estuarine sand and mud, demolition rubble, industrial and household waste. Hawkesbury Sandstone comprises quartz sandstone and very minor shale and laminite lenses.

From review of the 'Map and Selected Details of Near Vertical {Geological} Features in the Sydney CBD', prepared by Pells, Braybrook and Och (2004), we understand the GPO Fault Zone is plotted trending about 030° (north-eastwards to south-westwards) past the site on the far side of the junction between Spring and Gresham Streets. A dyke (named Pittman LIV) trending approximately east-westwards, is less than 50m to the north of the site.

We previously completed a geotechnical investigation in 2008 (Ref 22233SYrpt) for the City East Cable Tunnel which included a series of deep cored boreholes. The nearest borehole to the site (CECT3) was located in Gresham Street about 15m to the east of the site, and about 25m south of the junction with Bridge Street.

In summary, the borehole CECT3 encountered distinctly weathered sandstone bedrock from a depth of 0.5m, initially of low to medium strength improving to medium strength below a depth of about 3m. Below a depth of about 6.5m, slightly weathered and then fresh, high strength sandstone was encountered and extended to the termination depth of 53.5m. The sandstone contained relatively few defects, mostly sub-horizontal bedding partings with occasional joints and a single sub-horizontal extremely weathered seam of 50mm thickness. The only features of note were two portions of 'no core' totalling 0.5m between depths of 8.6m and 9.4m. Portions of no core recovery usually indicates weak material flushed away during the coring process, such as clay seams, and extremely weathered seams.

Based on this borehole information and our walkover inspection, we expect the eastern side of the site to be underlain by shallow sandstone bedrock conditions and that only negligible depths of fill or natural soil would be present on site given the site has already been fully developed. However, the "Tank Stream" is located on the western side of the site and deeper soils may be present on that side of the site.

While the proposed excavation will be lower than 0mAHD and the groundwater table might therefore be expected, the presence of nearby tunnels and deep basements on nearby properties, at lower levels, may have already caused the site to be drained to the depth of the excavation. Further investigation of the tunnel designs is required to confirm which tunnels were designed as drained and which are undrained. Generally, minor groundwater seepage should be expected through sandstone bedrock, which may increase following long periods of rain, but will be limited by the permeability of the defects within the sandstone. If the rock levels drop off on the western side of the site there may be flows within the fill above the bedrock.

4 COMMENTS AND RECOMMENDATIONS

4.1 Principal Geotechnical Issues

The principal geotechnical issues for this 'landmark' development will be the design of foundations to resist high column loads (and up-lift forces) whilst limiting deflections within the underlying tunnels to tolerable limits, maintaining stability to adjacent pavements and infrastructure, and limiting vibrations during excavation to prevent potential structural damage to existing structures. The existing and proposed tunnels will place limitations on the footing system and approval for the development from the tunnel authorises will be required. We expect that extensive finite element analysis will be required, to assess the effect of the development on the tunnels, in order to gain approval.

Our desktop study is primarily based on information from a nearby cored borehole from a previous investigation which indicates good quality sandstone bedrock from shallow depth. Assuming these conditions to be similar throughout the site, the subsurface materials are favourable in relation to most of the principal geotechnical issues, although sandstone of medium to high strength will require high energy excavation techniques and there is a possibility that rock depths will increase to the western side of the site towards the nearby Tank Stream.

The comments and recommendations contained herein may be used for planning and preliminary design, but numerous boreholes will be required to confirm the subsurface conditions below the site and variations in rock depth and quality. Not all of these boreholes will not need to be drilled as deep as the existing borehole drilled for the cable tunnel, but some will need to extend to such depths to determine the subsurface profile below the existing and proposed tunnels. Some boreholes may be able to be drilled prior to demolition from within Little Spring Street and possibly inside trafficable basements. However, the results will need to be confirmed by additional boreholes drilled following demolition.

As part of the planning phase of the project., we recommend that a survey of all of the existing basements and lowest floor levels within and around the site be carried out. In addition, details of all of the existing and proposed tunnels below and close to the site should be sought from the respective authorities (i.e. Sydney Metro, Telstra and Ausgrid). Limitations or approval may also be required due to the Tank Stream culvert, which we understand is below or to the west of Pitt Street, and the relevant authority in control of this should also be contacted.

The above geotechnical issues are discussed in more detail in the following sections of this report.

4.2 Planning, Analysis and Monitoring for Tunnel Authorities

As shown on Figure 2, the existing City East Cable Tunnel (CECT) passes below the northern end of the site and the Sydney Metro tunnels are below the north-eastern corner. Sydney Metro imposes limitations on development within the 'First Reserve', which we understand extends approximately 5m radially from the tunnels, and the 'Second Reserve', which we understand extends approximately 30m radially from the tunnels.

Based on the Sydney Metro Underground Corridor Protection - Technical Guidelines Rev1 dated 16 Nov 2017, no excavation or footings will be permitted within the first reserve. Geotechnical investigation and instrumentation may be permissible within the First Reserve, subject to Sydney Metro assessment.

Some excavation and footings may be acceptable within the Second Reserve, subject to Sydney Metro assessment. No adverse effects will be permissible on their assets, such as deflection from pressure increases from the building loads or anchors, or pressure relief from excavation and release of in-situ stresses. As a result, piles may be needed to transfer the building loads below the zone of influence of the tunnel, which may provisionally be taken as a line of at least 1V:1H drawn up from the base of the tunnel. The piles may need to be lined to prevent shaft friction placing loads on the sandstone within the second reserve. The actual toe levels and shaft lining required can be refined following finite element analysis.

We expect that similar constraints and requirements may be imposed for the CECT, but we are not aware of published guidelines detailing these requirements. We recommend that the extent of the first and second reserves for both the Sydney Metro tunnels and for the existing CECT should be confirmed by the authorities and their requirements for geotechnical investigation and analysis.

An engineering impact assessment will be required to demonstrate that no adverse effects will occur on the existing and proposed tunnels as a result of the demolition, excavation, temporary works and construction of the building. The assessment will comprise finite element analysis comprising probably a 3-dimensional geotechnical model, based on site specific boreholes, predicted in-situ stresses and surveyed geometry (basements and lower ground floor levels). The analysis must also account for the staged effects of the demolition, excavation and construction phases for which proposed loads, footing layout and structural details of the tunnel structures are required. Given the limitations of drilling boreholes within the site due to the existing buildings, the initial analysis may be able to be carried out using the results of borehole CECT3 from our previous investigation. However, it would be preferable to drill boreholes where access is possible to assess variations in the subsurface profile. The analysis may need to be repeated or checked following demolition when a more complete geotechnical investigation can be carried out. The outcome of the analysis, which is inherently complex, will be the major factor in the Sydney Metro assessment, hence they may require a peer review of the analysis.

A geotechnical and hydrogeological monitoring plan will be required, which will detail monitoring during demolition, excavation and construction. Monitoring is likely to include installation of inclinometers, piezometers, extensometers and settlement markers in the ground and on buildings. Instrumentation will also be required in the tunnels and will probably include tunnel convergence, tilt meters, crack meters, vibration sensors, track distortion monitoring and possibly strain gauges, pressures cells and optical prism laser scanning. Again, the relevant authorities will need to confirm their monitoring requirements.

4.3 Dilapidation Surveys

Prior to demolition and excavation, we recommend that detailed dilapidation surveys be carried out on the adjoining properties and buried structures located within a horizontal distance from the edge of the proposed excavation of at least twice the excavation depth. The dilapidation surveys should comprise detailed inspections of the structures, both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective property owners should be provided with a copy of the dilapidation reports and be asked to confirm that they present a fair representation of the existing conditions.

Such reports can be used as a baseline against which to assess possible future claims for damage arising from the works and in this way can guard against opportunistic claims for damage that was present prior to the start of the works.

4.4 Excavation

Excavation for the proposed basement is expected to extend to a maximum depth of about 20m below existing ground surface levels. Existing basement levels at No 56 Pitt Street, No. 3 Spring Street and lower ground floor of No 58 Pitt Street should be surveyed to better inform the depths of required excavation. Based on expected subsurface profile, excavation to such depths will predominantly encounter sandstone bedrock.

It is possible that council may make it a development condition that the rock be quarried for heritage purposes, if deemed of sufficient quality. The quality of the sandstone will need to be assessed by the quarry operator by inspection of the rock core recovered as part of the geotechnical investigation. The quarry operators may also require testing of the cores to assess the rock durability. Such an assessment will not be able to be finally made until all boreholes have been drilled, but an initial assessment could be made as part of any preliminary investigations carried out prior to demolition involving the drilling of boreholes in accessible areas.

Excavation of fill, residual soil and extremely weathered sandstone bedrock will be readily achievable using the buckets of medium to large sized hydraulic excavators.

Excavation of low or higher strength sandstone will require medium and large excavators with a combination of rock hammers, ripping tynes and rock sawing, particularly around the perimeter. Where the rock is quarried for re-use it will need to be saw cut to form block for removal from the excavation. Sawn rock faces prevent unnecessary overbreak and reduce excavation related fracturing of the rock face resulting in less remediation to any cut faces that could be left unsupported. In addition, saws cuts can assist in reducing the transmission of vibrations outside of the excavation. The site is large enough for dozers fitted with ripping tynes to be considered to assist with the bulk excavation, depending on access considerations and productivity.

Excavation using hydraulic rock hammers must be carried out with care due to the risk of damage to nearby structures from the vibrations generated by the hammer. In this respect, we recommend that excavation commence away from likely critical areas (i.e. commence within the central portion of the site) to allow monitoring of transmitted vibrations prior to excavation close to the boundaries. We recommend that the vibrations transmitted to nearby structures be quantitatively monitored during rock hammer excavation works. Vibration monitors should be solidly fixed to the nearest structures, with the monitors attached to flashing warning lights, or other suitable warning systems, so that the operator is aware when acceptable limits have been reached at which point such excavation techniques should cease. If permission is not given to attach monitors to the nearby structures then they should be set up on the site boundaries. The completed dilapidation reports should be reviewed to assess the most appropriate location for monitors.

Vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec for most nearby structures but 3mm/sec for the heritage Lands Department building. The lower limit should also be adopted any other particularly sensitive structures or equipment. Sydney Metro will have their own specifications for tolerable vibrations.

If higher vibrations are recorded than the target limits, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be feasible depending on the associated vibration frequency. However, any on site warning devices can only be set against the PPV and not the associated vibration frequency so will need to be set for the lower PPV values. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use smaller plant or alternative lower percussion techniques, e.g. grid sawing in conjunction with ripping and rock grinders. The use of these alternative techniques will have lower productivity, but will limit vibrations. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

We recommend use of excavation contractors with experience in such work and with a competent supervisor who is aware of vibration damage risks. The contractor should be provided with a full copy of the geotechnical investigation report (once completed) and have all appropriate statutory and public liability insurances.

4.5 Sandstone Cut Faces and Retention

The basement will extend to the site boundaries so there is no space to batter except perhaps for internal steps etc. We expect that the excavations will generally encounter a shallow soil cover over good quality sandstone bedrock, which will be able to be left unsupported subject to geotechnical inspection. Some shoring walls may be required, particularly along the western side of the site, where deeper soils are more likely to be encountered and may comprise soldier or contiguous pile walls installed prior to the start of excavation. Where such walls are terminated above the base of the excavation within good quality rock additional lateral support of the pile toes would be required by the use of external anchors or internal props.

Where the existing basement walls extend to the site boundaries the method of support required will depend on the location and nature of the existing basement retaining walls. Survey measurements of the existing walls should be made so these can be compared with the proposed basement extents. If the proposed basement walls are located outside of the existing basements then piled walls may need to be installed behind the existing walls prior to demolition of the existing walls. Alternatively, if the new walls are in front of the existing walls then the existing walls may be able to be used for temporary support to allow construction of the new walls and then the resulting gap backfilled. More likely the old and new walls will be coincident and a methodology for partial and progressive removal will have to be devised. Similarly, the existing basements should be inspected by a geotechnical engineer to assess if sandstone is exposed and if such faces can be left unsupported during excavation.

Vertically cut unsupported excavations would be appropriate where sandstone of at least low strength with infrequent defects is present. The extent of such rock must be assessed as part of the geotechnical investigation. Sandstone cut faces must be inspected by a geotechnical engineer a depth interval of no more than 1.5m to assess if any weak zones or inclined joints are present that require additional support. Such additional support may comprise rock bolts, shotcrete and mesh, and/or dental treatment of weak seams. All additional support recommended by the geotechnical engineer must be installed prior to further excavation.

Although good quality sandstone will be adequate to stand unsupported in the long term, the sandstone face will deteriorate and fret over time with the debris collecting at the base of the cut faces, potentially blocking drainage. Allowance for ongoing maintenance to clear such debris is unlikely to be feasible and so the sandstone cut faces should be protected by the placement of shotcrete or the construction of permanent walls in front of the cut faces with the resulting gaps filled with gravel.

4.6 Groundwater

Given the ground profile is predominantly sandstone bedrock, and nearby basements and tunnels may have already drained localised groundwater, groundwater may not be a significant issue for the proposed development. Some water may be perched within the soil above the rock or within defects the bedrock, but this is expected to be of limited volume and should drain quickly. Such seepage from rock usually reduces following excavation, but should be expected to increase during and following rain. Dykes can also be a source of higher rates of seepage and if a dyke is encountered in the excavation significant seepage may occur. Provided no dykes are encountered seepage is expected to be readily managed by sump and pump techniques.

For such a significant development, authorities may require detailed investigation of groundwater levels and estimates of potential seepage discharge rates. This will require site specific groundwater monitoring wells to be installed as part of the geotechnical investigation and permeability testing carried out. We recommend at least 3 groundwater monitoring wells be installed as part of the first round of the geotechnical investigation.

In the long term, drainage should be provided behind all basement retaining walls, and possibly below the basement slab, to control and direct any seepage that does occur. The completed excavation should be inspected by the geotechnical and hydraulic engineers to confirm if the designed drainage system is adequate for the actual seepage flows. It is possible that a tanked basement will be required in which case drainage would only be required during the construction period.

4.7 Footings

Sandstone bedrock will be exposed at bulk excavation level so all footings should be founded within sandstone bedrock. Shallow pad and strip footings will be feasible at the southern end of the site, but piled footings are likely to be required at the central and northern ends to transfer the building loads below the zones of influence of the tunnels.

Piles may have to be specially detailed (lined and "lubricated") to prevent shaft adhesion and thus prevent surcharging the tunnels. Piling through high strength sandstone will require large high-powered piling rigs. Establishing such rigs should be carefully planned prior to completion of bulk excavation. Simply mounding excavated spoil to form a temporary berm may not provide adequate support for such large piling rigs. It may be beneficial to complete piling after demolition but before further excavation.

While we expect that good quality sandstone will be present across the majority of the site, core loss indicating weak seams occurred at a depth of about 9m in borehole CECT3. Although it is likely footings would be founded below such seams, the extent and dip direction of this feature is unknown. Additional geotechnical investigation is critical to determine the present of such weak zones and the effect on footing design, particularly since we expect that high bearing pressures will be required. Other features such as joint swarms and dykes may be present and these would affect footing design. However, such features can be narrow and may not be encountered within the boreholes drilled for the geotechnical investigational and redesign of the affected footing design may be required once these features are exposed during excavation. The use of additional footings and transfer beams to span across such features may be required.

The geotechnical investigation should include a good spread of boreholes where possible in advance of demolition for general footing design, followed after demolition by further detailed cored boreholes, probably one per footing location to maximise bearing pressures. While there is a relatively low risk of encountering geological features (e.g. dykes and joint swarms), it would be prudent to include angled boreholes as part of the investigation to check for previously unknown features. However, even with angled boreholes such features may not be encountered until excavation.

Allowable bearing pressures within sandstone would start at 1000kPa for sandstone of low strength, increasing to say 3500kPa for sandstone of medium strength and 6000kPa or more for sandstone of high strength without significant effects. The final bearing pressure will need to be determined as part of the geotechnical investigation and design will be based on limit state methods where settlements are calculated for each footing in order to maximise bearing pressures. During construction, we expect that significant proving will be required, involving cored boreholes at all or most of the footings locations and possibly spoon testing of cored holes in pad footings.

4.8 Preliminary Earthquake Site Classification

Based on expected good quality rock from shallow depths, we consider the site sub-soil class would be 'Class B_e – Rock' in accordance with AS1170.4-2007 with Amendments 1 and 2. This must be reviewed following the geotechnical investigation.

4.9 Basement Floor Slab

If drainage is required/permitted below the basement slab it may comprise either a closely spaced grid of subsoil drains or a (single sized) gravel blanket. The drainage will need to be connected to a permanent 'fail-safe' pump out system, which is fitted with automatic level control pumps to prevent flooding.

If a drainage blanket is not adopted, the basement slab should be designed with a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 unbound base material (or other approved good quality and durable fine crushed rock), which is compacted to at least 100% of Standard Maximum Dry Density (SMDD). This subbase layer will provide a separation between the sandstone subgrade and the slab and provide a uniform base for the slab. The grid of subsoil drains, if required, would then be formed below this layer.

4.10 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Clarification of easements and tunnels and their development reserves.
- Detailed surveys of the site's basements and adjacent basements.
- Site specific geotechnical investigation comprising cored boreholes, unconfined compressive strength testing, point load strength testing. We expect that this will be staged with some boreholes drilled prior to demolition and some following demolition, included angled boreholes to check for joint swarms or dykes.
- Inspection of any cut rock faces present in the existing basements.
- Installation and reading of groundwater monitoring wells and preparation of a hydrogeological report including assessment of likely inflows.
- Geotechnical 3D finite element analysis to demonstrate effects on adjacent tunnels and other structures.
- Preparation of a geotechnical and hydrogeological monitoring plan.
- Installation of geotechnical instrumentation in accordance with monitoring plan.
- Dilapidation Surveys.
- Vibration monitoring.
- Inspection of cut rock faces.
- Inspection of footing excavations and pile drilling.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the detailed design and construction phases of the project. In the event that any of the detailed design or construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the JK Geotechnics report.

Title: **SITE LOCATION PLAN**

Location: 56-62 PITT STREET AND 3 SPRING STREET, SYDNEY, NSW

Report No: 33070BM

Figure: 1

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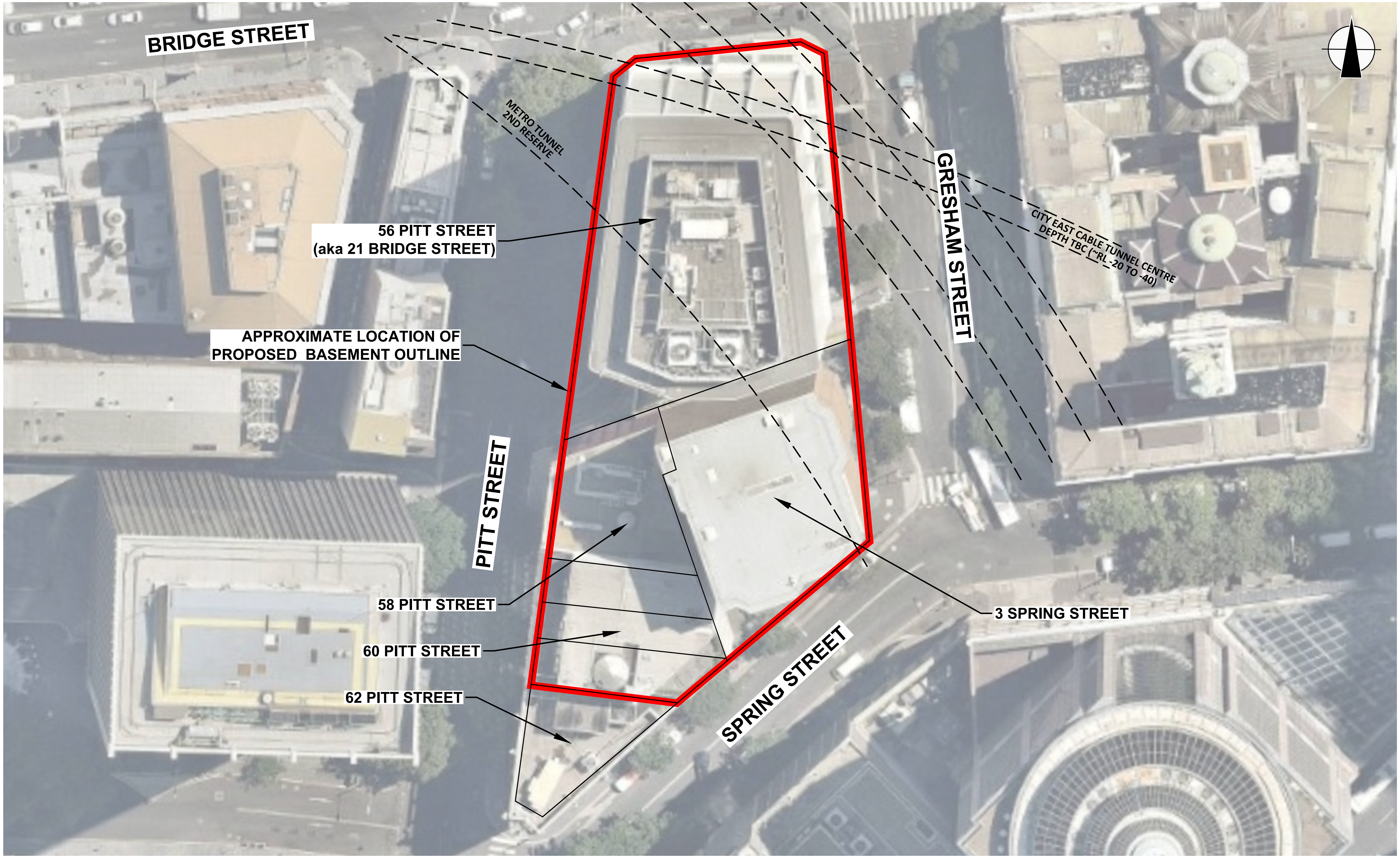
PLOT DATE: 27/03/2024 2:46:40 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33070LM SYDNEY\CAD\33070BM.DWG



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LEGEND

– – – – RAIL TUNNEL CORRIDORS

SCALE 0 5 10 15 20 25 METRES 1:500 @A3	Title: **PROPOSED DEVELOPMENT LOCATION PLAN**
	Location: 56-62 PITT STREET AND 3 SPRING STREET, SYDNEY, NSW
	Report No: 33070LM — Figure: 2
This plan should be read in conjunction with the JK Geotechnics report.	**JKGeotechnics**

PLOT DATE: 14/04/2020 12:24:03 PM DWG FILE: Y:\33000'S\33070LM SYDNEY\CAD\33070LM.DWG

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.