Attachment A16

Geotechnical Desktop Assessment - 15-25 Hunter and 105-107 Pitt Street, Sydney

REPORT TO

MILLIGAN GROUP PTY LTD AND ITS SUBSIDIARY FT SYDNEY PTY LTD AS TRUSTEE FOR FT SYDNEY UNIT TRUST

ON

PRELIMINARY GEOTECHNICAL DESKTOP ASSESSMENT

FOR

PROPOSED HUNTER & PITT DEVELOPMENT

AT

15-23 HUNTER STREET & 105-107 PITT STREET, SYDNEY, NSW

Date: 30 July 2020
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Table of Contents

1 FOREWORD

In the interests of clarity and consistency with other consultants reports for this project, the following has been requested for inclusion in this report.

This geotechnical report has been prepared by JK Geotechnics Pty Ltd in support of a Planning Proposal to amend the Sydney Local Environmental Plan 2012 (Sydney LEP). This report has been prepared on behalf of Milligan Group Pty Ltd (the Proponent) and its related entities and consultants, representatives and agents and FT Sydney Pty Ltd as trustee for FT Sydney Unit Trust. It relates to an amalgamated site at 15-21 Hunter Street and 105-107 Pitt Street (the site).

The purpose of this Planning Proposal is to amend the site's Floor Space Ratio (FSR) development standard, and the Maximum Building Height to align with the Martin Place Sun Access Plane contained within the concurrent Central Sydney Planning Proposal.

This Planning Proposal supports the City of Sydney Council's draft Central Sydney Planning Strategy (Draft CSPS) by unlocking additional employment generating floor space within a designated tower cluster. The proposed Sydney LEP amendment is part of the broader redevelopment plan for the site to facilitate a new commercial office tower. It will also facilitate significant public benefits through additional site activation and embellishment of the public domain.

The Planning Proposal is accompanied by amendments to the Sydney Development Control Plan 2012 (Sydney DCP). The site specific DCP amendments reflect the proposed outcome to provide a podium tower scheme. This is reflected in the accompanying reference design prepared by Bates Smart which serves as a baseline proof of concept for this Planning Proposal. This 2,108m2 strategic site presents a unique opportunity to deliver a landmark premium commercial office tower that will exhibit design excellence and offer significant employment opportunities for global Sydney.

The uplift being sought is consistent with the strategic intent of the draft CSPS, which contains the City's requirements and expectations for projects pursuing this pathway. Following the Planning Proposal, the planning approval pathway involves a competitive design process and a detailed Development Application. As such, this report reflects the concept stage of the proposal, and may be embellished as the detailed design and required works evolve.

2 INTRODUCTION

This report presents the results of a preliminary geotechnical desktop assessment for the proposed 'Hunter & Pitt' multi-storey development at 15-23 Hunter Street and 105-107 Pitt Street, Sydney, NSW.

We have been supplied with the reference design drawings prepared by Bates Smart dated 24 July 2020 for a 51-storey tower over four basement levels with a lower floor level of RL-6.7mAHD. In the northern half of the basement, there is proposed to be a double level car stacker extending down to a level of RL-12.2m. The basement and car stacker will require excavation to depths ranging from 14.5m to 22m below existing surface levels up to the northern, eastern and southern site boundaries. The existing building at 15-17 Hunter Street appears to be retained in the proposed design at the crest of the proposed western basement wall. Proposed loads were not available at the time of writing, though we assume high column loads will apply.

The purpose of the assessment was to infer the likely geotechnical subsurface conditions from a desktop review of available information, and to use this as a basis for providing preliminary comments and recommendations on excavation conditions, retention systems, footing design and hydrogeological considerations.

A summary of the principal geotechnical issues, based on this assessment, are summarised in Section 5.1.

3 ASSESSMENT PROCEDURE

The desktop assessment comprised a review of:

- Published geological data including the 1:100,000 Sydney Geological Series Sheet and mapping of fault zones in Sydney CBD.
- Review of nearby projects completed by JK Geotechnics.
- Review of online imagery.

4 SITE DESCRIPTION

The site is located within gently sloping topography in a shallow gully between low-height ridgelines on the sandstone headland occupied by the CBD. The site has a northern frontage with Hunter Street and an eastern frontage with Pitt Street, with the streets appearing to be relatively level in front of the site.

The site comprises five lots each occupied by a multi-storey building with the following observations made for each lot (in clockwise order from the north-western corner):

- No. 15-17 Hunter Street is a three-storey rendered building which appears to be of an early twentieth century construction. No basements appear to be present at this property;
- No. 19-21 Hunter Street is a four-storey rendered building which does not appear to contain any basement levels;
- No. 23 Hunter Street is a 15-storey building appearing to be of concrete frame construction;
- Nos. 105 and 107 Pitt Street are both 8-storey concrete framed buildings with a single lower-ground floor level.

Each of the buildings typically extends up to the boundaries on their respective lot although along the northern boundary of No. 105 is a narrow concrete paved laneway, Empire Lane which separates the buildings at 105 Pitt Street and 23 Hunter Street.

West of the site is a 19 storey (No. 9 Hunter Street) concrete framed building which does not appear to contain any basement levels. South of the site is a five-storey building (No. 109 Pitt Street) which appears to contain at least one level of basement parking extending close to the site boundary.

5 SUBSURFACE CONDITIONS

The Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Hawkesbury Sandstone comprising medium to coarse-grained quartz sandstone with minor shale and laminite lenses. Nearby boreholes have disclosed a subsurface profile generally comprising fill and residual sandy clay/clayey sand overlying sandstone bedrock at depths of 2m to 5m though possibly deeper near the Tank Stream. Around the Tank Stream it is possible that loose alluvial sands will be encountered.

The Tank Stream is a heritage-listed brick and sandstone oviform and rectangular tunnel that follows the alignment of an open-air fresh water creek that was the first colonial water source. The present Tank Stream forms a drain for stormwater runoff from the CBD and discharges at an outlet in the seawall at Circular Quay to the north of site. The outlet for the Tank Stream is at sea level in the seawall at the southern edge of Circular Quay which would probably be situated between RL0m and RL1m. At the site the invert is likelyto be close to RL1m and the crown at about RL3m which is about 5m below ground surface levels; the level of the tank stream tunnels should be determined as part of the future site investigation and design phases.

The GPO Fault Zone is north-north-east trending fault zone that from a review of published maps appears to be close to the south-eastern corner of the site. Fault zones are linear zones within the bedrock which have been sheared by regional movements, resulting in zones of more fractured and lower strength rock than the surrounding undisturbed material. These zones often have a higher permeability than the surrounding undisturbed rock mass.

6 COMMENTS AND RECOMMENDATIONS

6.1 Summary of Principal Geotechnical Issues

Based on the results of the preliminary desktop assessment, and our understanding of the proposed development, we have summarised the principal geotechnical issues associated with the planning, design and construction of the proposed development as below:

1. Prior to demolition or excavation we recommend detailed dilapidation surveys be completed on the neighbouring structures to the west and south of the site. Dilapidation surveys may also be required on Hunter and Pitt Streets including associated infrastructure.
2. The upper fill and soil profile will need to be supported by shoring walls. Shoring walls will need to extend into the upper sandstone and be supported laterally by at least two rows of anchors and/or props.

3. Excavation for the proposed basement will likely primarily extend through sandstone bedrock which will require 'hard rock' excavation equipment, which may transmit vibrations through the rock mass that could affect adjoining structures, therefore requiring the completion of vibration monitoring.
4. Excavation is anticipated to extend below the groundwater table, and testing and analysis will be required to estimate the groundwater seepage inflows during construction and the long term, and to allow coordination with authorities with regard to the disposal of groundwater from a drained basement.
5. Shoring along the western site boundary will require careful consideration with respect to the heritage-listed Tank Stream and the existing building at 15-17 Hunter to be retained.
6. The column loads are expected to be high, requiring high bearing pressures on the sandstone bedrock which will require detailed investigation and proving during construction.
7. The impact of the proposed basement on the adjacent properties including the heritage-listed Tank Stream must be assessed, such as by using finite element analysis. If possible, we recommend obtaining drawings of the neighbouring buildings as well as an inspection of any basement levels.

Further comments on these issues and geotechnical design parameters are provided in the subsequent sections of this report.

To allow detailed design, a site-specific geotechnical investigation must be carried out. The geotechnical investigation should comprise the drilling of cored boreholes to depths below the proposed basement levels for a classification of rock quality and strength testing.

Prior to demolition, some boreholes may be drilled in Empire Lane and possibly within the existing buildings to confirm the site geological model, however, this will tend to reduce the drilling efficiency. Further drilling would likely be required to complete rock mass permeability testing and to complete drilling where access is not possible prior to demolition.

The comments and recommendations contained herein may be used for planning and preliminary design though they must be reviewed and amplified as part of the detailed geotechnical investigation.

6.2 Dilapidation Surveys

Prior to the commencement of demolition, we recommend that dilapidation surveys be completed on the neighbouring buildings around the site. Depending on Council and utility authorities' requirements, dilapidation surveys may also be required for Hunter and Pitt Streets and adjacent infrastructure.

The dilapidation surveys should include internal and external inspections of the buildings, with all defects including defect locations, type, length and width being described and photographed. With larger buildings it may be appropriate to limit the detailed study to areas closest to the subject site and perhaps to representative floors rather than all floor levels. Any reduction in the detail needs to be agreed between all parties prior to commencement. The respective owners of the neighbouring buildings, assets and infrastructure should be asked to confirm that the dilapidation survey reports present a fair record of the existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future claims for damage arising from the works.

6.3 Excavation

6.3.1 Excavation Conditions

All excavation recommendations should be complemented by reference to the NSW Government "Code of Practice Excavation Work" dated January 2020.

Excavation for the proposed basements will extend through fill and natural soils, though will be mostly within sandstone bedrock likely to be predominantly of medium and high strength. Excavation of the soils and extremely weathered sandstone will be achievable using conventional excavation equipment such as the buckets of hydraulic excavators. Excavation through sandstone of low or greater strength will require the use of 'hard rock' excavation techniques which will likely include a combination of hydraulic rock hammers, rock saws and rock grinders. We expect the majority of the excavation would be completed using rock hammers on large, say 30 tonne, tracked excavators.

Grid sawing of the bedrock would increase the ground borne vibration path to the adjacent buildings (i.e. would reduce vibrations) provided the base of the saw slot is maintained below excavation level. Dust suppression by spraying with water should be carried out whenever rock saws are being used.

Excavation along the southern boundary will need to consider the extent and depth of the neighbouring basement and footings, and details on this structure will be required. Ideally, the as-built construction and structural plans of the adjacent building should be obtained for review. If plans are unavailable, then as a minimum a walkover inspection should be completed within the basement, and test pits must be excavated in advance of the bulk excavation to assess the footing founding level and foundation material.

6.3.2 Vibration Monitoring

We recommend that full-time quantitative vibration monitoring be carried out on the adjoining building at 109 Pitt Street and the nearby 9 Hunter Street building during the demolition and the excavation for the proposed basements.

The vibration monitoring should include geophones affixed to the surrounding buildings and a warning system (e.g. flashing lights, audible alarm, etc.) which is set to trigger when the permissible vibration limits have been recorded. The locations of the geophones should be assessed following review of the dilapidation survey reports, and should be jointly nominated by the geotechnical engineer and the acoustic consultant.

It is possible that vibration monitoring on or near the Tank Stream may be required to protect the heritage-listed structure. The requirements for vibration monitoring should be checked with the relevant authority.

The vibrations limits should be set by the structure engineer following their review of the structural integrity and sensitivity of the adjoining structures.

If higher vibrations are recorded then they should be reviewed in light of their associated frequency, as higher vibrations may be acceptable depending on the frequency. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to change to alternative rock excavation methods such as a tighter grid of rock saw slots and/or a smaller rock hammer.

6.3.3 Hydrogeology

Based on information from nearby sites, the groundwater level across the site is expected to be between about RL2m to RL3m, which would probably be within the sandstone bedrock. Groundwater at these levels would result in the lower 9m to 15m of excavation for basements occurring below the water table. Groundwater inflows into the excavation are expected to occur as local seepage flows through joints and bedding partings within the bedrock profile. The depth to groundwater and anticipated seepage volumes must be confirmed during detailed investigation, and this will require rock mass permeability testing and subsequent seepage analysis.

It is expected that groundwater seepage will be controllable using conventional sump and pump discharge systems. Piped discharge from the drainage system into the stormwater system can only be completed once the approvals have been obtained, and the groundwater has been treated in accordance with any environmental recommendations. If excavation occurs through the GPO Fault Zone then higher seepage flows would be anticipated, and grouting or tanking may be required to reduce or eliminate the volume of seepage.

6.4 Excavation Retention

6.4.1 Retention Systems

Excavation is proposed to extend up to each boundary except to the west where the extent of excavation is limited by the Tank Stream. Due to the extent of excavation, temporary battering will not be possible and cuts through any soil and the upper more weathered sandstone bedrock must be supported by an engineered shoring system.

Subject to the results of the investigation, it is likely this would comprise a soldier pile wall with reinforced shotcrete infill panels, If the upper soils are particularly sandy, or the neighbouring buildings and footings are quite sensitive, then it may be necessary for some of the shoring walls to be constructed as contiguous pile walls.

It is likely that conventional bored piles would be suitable for the shoring wall construction.

The shoring will require the use of at least two rows of anchors or props in the short term, with the permanent support to the shoring being provided by bracing from the floor slabs.

6.4.2 Retention Design Parameters

For preliminary design of in-situ retention systems the following earth pressure coefficients and subsoil parameters may be adopted however these must be confirmed, and refined where necessary, following detailed subsurface investigations.

- For progressively anchored or propped basement walls which are highly sensitive to lateral movement (which we expect to be on all sides of the proposed basement), we recommend the use of a trapezoidal lateral earth pressure distribution, with a maximum pressure of 8H (kPa), where H is the height of retained material in metres. This pressure should be applied over the central 60% of the height of the shoring, tapering to zero at the crest and toe.
- Any surcharge affecting the walls (e.g. traffic and construction loading, adjacent building footings, adjacent basement floor slabs, etc.) should be allowed in the design using an 'at rest' earth pressure coefficient (K_0) of 0.55 for the soil profile, assuming a horizontal retained surface.
- A bulk unit weight of $20kN/m^3$ should be adopted for the soil and more weathered rock profile.
- If temporary rock anchors are adopted to support the shoring walls, permission must be sought from the neighbouring property owners and Council prior to their installation. The anchors must have free lengths and bond lengths of at least 3m each, and the bond zone must be entirely behind a line drawn upward at 45° from the toe of the shoring wall. Anchors bonded into the sandstone of at least low strength may be provisionally designed for an allowable bond of 150kPa, while bond zones in rock of at least medium strength may be provisionally designed for a bond of 300kPa. All anchors should be proof tested to 1.3 times the working load in the presence of an experienced engineer independent of the anchor contractor. We recommend that only experienced contractors be considered for the anchor installations.

At an early stage of the planning, it must be determined whether anchors can be used above the tank stream structure, as if not, temporary bracing will need to be designed by the structural engineers to transfer loads and limit deflections.

Analysis of the proposed retaining wall design must be completed to assess wall deflections and additional advice should be sought from the geotechnical engineer with respect to appropriate design parameters for the sections being modelled following the detailed investigation. In addition to the shoring deflections, there will be additional movements associated with the release of the high in-situ horizontal stress field as discussed in Section 5.4.3 below.

6.4.3 Unsupported Cuts in Sandstone Bedrock

Sandstone bedrock of medium or greater strength will generally be self-supporting and where encountered below the toe of the shoring walls, may be cut sub-vertically subject to progressive inspection by an experienced geotechnical engineer as the excavation proceeds at no more than 1.5m depth intervals. The purpose of the inspections is to identify defects and assess the need for temporary support (e.g. rock bolts, dowels, shotcrete etc.) of potentially unstable wedges or weathered seams.

Provision must be made in the construction program and budget for the inspections and stabilisation works. A detailed Geotechnical Inspection and Monitoring Program will be required.

Within the unsupported rock excavation through the more competent sandstone, additional lateral movements will likely occur due to relief of in-situ horizontal stresses. Such in-situ horizontal stresses can be in the order of 2MPa or greater, and therefore cannot be economically restrained by typical shoring measures. Based on experience with deep basements in Hawkesbury Sandstone in the Sydney CBD, typical lateral movements due to stress relief are in the order of 0.5mm to 2mm per metre depth of excavation into those materials. The maximum lateral movements are likely to occur towards mid-height of the unsupported rock faces (i.e. between bulk excavation level and the toe of the shoring walls), but can occur along pre-existing bedding planes and therefore at any level and along multiple bedding planes.

6.5 Footings

Following bulk excavation we anticipate that sandstone bedrock will be encountered across the building footprint and it is likely that internal pad and/or strip footings will be adopted. The sandstone is likely to be of at least Class II quality, and so an allowable serviceability bearing pressure of between 6MPa and 10MPa may be used for the preliminary design, subject to detailed investigation and the degree of proving adopted during construction. These serviceability pressures are based upon having footing settlements of less than 1% of the footing width. Where Ultimate Limit State (ULS) design is adopted, ultimate bearing pressures in the order of 20MPa to 40MPa may be achievable, though serviceability analysis must also be undertaken to estimate the settlements of the footings under these pressures. For ULS design, an appropriate Geotechnical Strength Reduction Factor (ϕ_g) should be adopted, which would likely be in the order of 0.5 to 0.6, again depending on the amount of investigation and site proving to be undertaken.

All footings would need to be cleaned out and subsequently inspected by a geotechnical engineer (prior to the installation of reinforcement cages). As part of the inspection process for pad and strip footings, it is likely that spoon testing and/or cored boreholes will be required at a nominated proportion of footings, to assess the foundation material below the footings. The depth of the cored boreholes and/or spoon testing would typically be at least 1.5 times the footing width, below the base of the footing. The proportion of footings tested will be dependent upon the bearing pressure adopted and the uniformity of the bedrock encountered within cored boreholes. Deeper footing excavations may be required if stress relief effects cause base heave of the sandstone foundation material. All footings must be poured shortly after inspection and approval by the geotechnical engineer.

If there will be any footings on the high side of the cut between the general basement level and the car stacker, these must be founded below a line drawn upward at 1 vertical in 1 horizontal from the toe of that cut. If there are light loads within this zone, it may be possible to found these at the higher level, subject to geotechnical inspection of the rock quality and defects in the cut face.

6.6 Basement Floor Slabs

Sandstone bedrock will almost certainly be present at the proposed bulk excavation level. Therefore a slab on grade would be feasible for the basement floor slab. A drainage layer comprising free-draining, hard and durable aggregate should be placed on the base of the excavation to lead any seepage to sumps for pumped disposal. This will also act as a separation layer between the concrete slab and the bedrock to prevent curling of the floor slabs due to cracking. The slabs should be jointed to resist shear forces but not bending moments.

6.7 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:

- Completion of a detailed subsurface investigation.
- Completion of rock mass permeability testing and seepage analysis to estimate potential seepage inflows.
- Review of the shoring design and footing design.
- Progressive rock face inspections (below the toes of secant pile walls and within the central lift core) as the excavation proceeds.
- Inspections of footing excavations prior to pouring.

7 GENERAL COMMENTS

The preliminary recommendations presented in this report include specific issues to be addressed during the design and construction phase of the project. In the event that any of the design and 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 preliminary 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.