REPORT

TO

MOWEND PTY LTD

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED REDEVELOPMENT

AT

588 TO 592 PRINCES HIGHWAY (CNR OF LISTER AVENUE)

ROCKDALE, NSW

4 December 2015

Ref: 28959SBrpt

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Date: 4 December 2015
Report No: 28959SBrpt
Revision No: 0

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1 INTRODUCTION

This report presents the results of a geotechnical assessment for the proposed redevelopment of 588 to 592 Princes Highway, Rockdale, NSW. The assessment was commissioned by Mr Nick Vranas of Mowend Pty Ltd by returned Acceptance of our proposal dated 23 November 2015 (Ref: P41598SB).

As shown on the supplied preliminary development drawings by Anthony Vavayis + Associates (Ref: 15041, drawing Nos DA1001 to DA1006, DA1012 and DA1013, dated Nov 15) the existing buildings on the site will be demolished, and a new building constructed with ground floor retail and 13 levels of apartments above. At this stage the building is proposed to have two basement levels, with excavation to depths ranging from about 4m to 7m, but additional basement levels may be required, possibly four levels. The basement will extend to the eastern and southern boundaries, but will be offset about 2m to 3 from the northern and western boundaries.

The purpose of the assessment was to assess the likely subsurface conditions from a previous geotechnical investigation carried out at the site as a basis for preliminary comments and recommendations on geotechnical issues for the proposed development to assist with planning and preliminary concept design.

2 ASSESSMENT PROCEDURE

The assessment comprised a walkover inspection of the site by our Senior Associate, Mr Daniel Bliss, on 27 November 2015. Observations made during the walkover inspection are summarised in Section 3.1 below.

We were also provided with a previous geotechnical investigation report for the northern portion of the site (No 588 to 590) by Peter J Burgess & Associates Pty Ltd dated 16 June 1993 (Ref: S3442.R1/AA) and the borehole logs included within that report has been reviewed as part of this assessment, as discussed in Section 3.2.

3 RESULTS OF ASSESSMENT

3.1 Site Observations

The site is located within undulating topography and on the side of a hill that slopes down towards the south at about 3°. The site is located on the corner of the Princes Highway and Lister

Avenue. The site comprises two properties, being No. 588 to 590 at the northern end and No. 592 at the southern end.

No. 588 to 590 is occupied by a brick and rendered building with two above ground levels and one basement level. The basement is about 2m lower than the Lister Avenue frontage on the northern side of the site, but due to the hillside slope is at about the ground surface on the southern side of the site. The basement walls are constructed of concrete blocks so any exposed sandstone within the excavation could not be seen. The building appeared to be in good external condition. Mr Vranas advised that sandstone was encountered during excavation and that the excavation was extended for a depth of about 3m below the existing basement floor and subsequently backfilled.

No. 592 is occupied by a two storey rendered unit building, with a concrete paved parking area at the rear. The parking area is accessed via a concrete ramp off Lister Avenue on the eastern side of the building at No. 588 to 590. This ramp is suspended above the ground surface with a void below the ramp. The building at No. 592 appeared to be in fair external condition.

To the east of the site are two properties fronting Hayburn Avenue. The northern property, extending about 20m from Lister Avenue, contains a three storey brick unit building which was under construction at the time of the site visit. This building contains a single basement level, which is set back about 2m from the boundary. The southern property, which extends for the remainder of the common boundary, contains a three storey unit building with a single basement level, which is set back about 5m from the boundary. The ground surface adjacent to the boundary is at a similar level to the parking area at the rear of No. 592, but is lower than the suspended ramp on the eastern side of the building at No. 588 to 590.

To the south of the site is a concrete paved car yard, with a single storey office/workshop building at the rear. A concrete retaining wall is located on the boundary, which retains the subject site for a maximum height of about 1.5m. This retaining wall appeared to be in good condition. The wall of the office/workshop building at the rear also acts as a retaining wall.

3.2 <u>Previous Nearby Geotechnical Investigations</u>

Reference to the Sydney 1:100 000 Geological series Sheet indicates that the site is located in an area mapped to be underlain by Hawkesbury Sandstone.

The previous geotechnical investigation of No. 588 to 590 by Peter J Burgess & Associates Pty Ltd involved the drilling of five boreholes to depths ranging from 1.6m to 4.25m. Three of the boreholes were auger drilled, terminating at depths of 1.6m to 2.1m, with the sandstone cored in the remaining two boreholes to depths of 4.25m and 3.45m.

The boreholes encountered surface concrete and bitumen and fill to shallow depths covering residual soils grading into sandstone bedrock at depths ranging from 0.2m to 1.25m. Within the augered boreholes the sandstone was described to be “very weak to strong”, but the cored sandstone was generally assessed to be slightly weathered to fresh and of “medium strong to strong” strength. However, some “weak” strength sandstone was encountered in one borehole to a depth of 2.8m. Based on the currently used strength classification system, “weak” strength would be “low” strength and “medium strong to strong” strength would be “medium to high” strength. Defects within the cored sandstone comprised sub horizontal clay seams and joints at spacings of about 300mm. No groundwater was encountered within the boreholes.

4 COMMENTS AND RECOMMENDATIONS

4.1 Subsurface Conditions and Additional Geotechnical Investigation

Based on the results of the previous geotechnical investigation carried out at the site, sandstone is present at shallow depths, with medium to high strength sandstone at depths of about 1m to 3m. However, due to the basement excavation within No. 588 to 590 we understand that the sandstone has been excavated to depths of about 3m to 5m.

Since the previous boreholes were only drilled to shallow depths, well above the proposed excavation depth for the two or possibly four basement levels proposed, we recommend that to allow detailed design additional cored boreholes be drilled. These boreholes should be drilled to depths below the base of the proposed excavation and so should be carried out once the final excavation depth has been determined. The drilling of cored boreholes will also allow assessment of the sandstone at the proposed footing level in order to optimise the allowable bearing pressure for the design of the building footings. Prior to demolition, cored boreholes could be drilled at the rear of No. 592, and possibly at the front using a small rig. Boreholes within No. 588 to 590 would need to be carried out following demolition, with a ramp constructed into the existing basement excavation to provide access for the drilling rig.

Comments are provided below on the geotechnical issues for the site and proposed development based on the above subsurface profile. The comments and recommendations must be confirmed and amplified following completion of the additional cored boreholes.

4.2 Geotechnical Issues

Based on the above subsurface profile the main geotechnical issues for the proposed development as described in Section 1 are discussed below. Overall, we consider that the site is suitable for the proposed development and will be similar to other developments constructed within nearby properties.

The comments and recommendations provided herein are preliminary only and may be used for planning and preliminary concept design only. The comments and recommendations will need to be confirmed and amplified as part of the detailed geotechnical investigation of the site.

Excavation

Variable conditions will be encountered within different parts of the site due to the existing basement excavation. We have been advised that at the time of development of the building at No. 588 to 590 excavation occurred below the level of the current basement with this backfilled prior to construction of the basement. Therefore, in that area of the site the excavation will encounter fill for a few metres and then sandstone below the base of the previous excavation. Within No. 592, where no basements are present, the excavation will encounter soils to a shallow depth, but for the majority of the excavation sandstone bedrock is expected.

Prior to the start of excavation, dilapidation surveys should be carried out on the existing structures within the adjoining properties to the east and south. The dilapidation surveys should include detailed inspections of the adjoining properties, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The respective owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The preparation of such reports will also help to guard against opportunistic claims for damage that was present prior to the start of construction.

Excavation of the existing fill and natural soils will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Some of the upper sandstone of extremely low strength may also be able to be excavated using such equipment. Excavation of the sandstone will require assistance with rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws. The sandstone of medium to high strength will represent 'hard rock' excavation conditions and the drilling of cored boreholes will be important for the excavation contractor to be able to assess the excavation characteristics of the rock so that appropriate equipment is mobilised to site. Due to the medium to high strength of the sandstone slow productivity and significant equipment wear should be expected during excavation.

Hydraulic rock hammers must be used with care due to the risk of damage to adjoining structures from vibrations generated by such equipment. If hydraulic rock hammers are used the transmitted vibrations to the adjoining buildings to the east and south should be quantitatively monitored at all times during rock hammer operation. The rock hammer should commence work away from the adjoining buildings and slowly progress towards the adjoining buildings so that work can cease when acceptable limits have been reached. The vibration monitors should be attached to flashing warning light so that the operator is aware when acceptable limits have been reached and work can stop. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are excessive it would be necessary to change to excavation equipment that results in lower vibrations, such as ripping hooks, rotary grinder or rock saws. A rock saw could be used to cut a slot around the perimeter of the excavation before excavating the rock using a rock hammer to reduce the transmitted vibrations, but the effectiveness of this would need to be confirmed by the results of the vibration monitoring. Our preference would be to cut all perimeter excavation faces using a rock saw to reduce the risk of overbreak and to provide a 'clean' face if it is exposed within the basement car park.

The excavated material will need to be classified in accordance with environmental guidelines for waste classification prior to disposal from site.

Groundwater

Groundwater seepage may occur into the excavation and will tend to occur along the soil/rock interface and through bedding partings and joints within the sandstone, particularly during and following rainfall. Any such seepage that does occur should be able to be adequately controlled using conventional sump and pump techniques. In the long term, drainage should be provided behind all retaining walls, at the base of rock cuts and below the basement slab. The drainage should lead to a sump containing and automatic pump to reduce the risk of basement flooding.

The depth of the groundwater table is not known, but may be intercepted if a deep basement is proposed.

Retention

During demolition consideration will need to be given to the upper soils that may be supported by the existing basement walls. Careful demolition of the walls will be required so that support of the soils is maintained. Information should be sought on retention that has been provided for the previous development to assist in determining the demolition precautions and methodologies required. If may be possible to incorporate the existing walls within the new shoring system, say by anchoring the existing walls, if the alignment of the existing walls matches that of the proposed basement.

Any soils and poor quality sandstone of extremely low to very low strength, which based on the previous boreholes may occur to depths of about 1m to 3m, will need to be formed at suitable batters or retention systems installed prior to the start of excavation. However, since the proposed basement extends to the eastern and southern boundaries insufficient space will be available for such batters. Batters may be possible on the northern and western sides of the basement where some space is available inside the excavation.

Temporary batters within the soils and extremely low to very low strength sandstone would be of the order of 1 Vertical in 1 Horizontal (1V:1H), but this should be confirmed by the results of the detailed geotechnical investigation. Where these batters cannot be accommodated suitable retention systems may comprise soldier pile walls with shotcrete infill panels. Although where the rock is shallow, say up to about 1m, the retention system could comprise the excavation of a trench, the drilling of dowels into the rock and pouring a concrete wall within the trench, prior to bulk excavation, provided the rock is of good quality on first contact.

If soldier pile walls are adopted the piles may be terminated once good quality sandstone is encountered, but external anchors or internal props will be required to provide lateral support. The presence of basements towards the east may limit the use of external anchors on that side of the site.

Propped or anchored retaining walls may be provisionally designed using an 'at rest' earth pressure coefficient, K_0, of 0.6 and a bulk unit weigh of $20kN/m^3$, assuming horizontal backfill.

Excavation within the good quality sandstone of low to medium or higher strength would be able to be cut vertically without support. However, regular inspection of the cut faces by a geotechnical engineer, say at depth intervals of no more than 1.5m, would be required to assess the quality of the rock and check for the presence of weak seams or inclined joints that require additional support. Such additional support may comprise dental treatment of thin weak seams, placement of shotcrete and mesh over thick weak seams, and rock bolts to support wedges isolated by inclined joints. Allowance for some additional support should be made within the project budget, but this should be further assessed from the cored boreholes drilled below the base of the proposed excavation.

Footings

Since sandstone will be encountered within the excavation the proposed structure should be supported on pad footings founded within the sandstone at the base of the excavation.

Allowable bearing pressures within the sandstone would start at 1000kPa for the upper sandstone of very low strength, increasing to 1200kPa for sandstone of low strength and 3500kPa or more for sandstone of medium to high strength without significant defects. The final design parameters should be assessment as part of the detailed geotechnical investigation.

5 GENERAL COMMENTS

The recommendations presented in this report are based on an inferred subsurface profile assessed from previous limited scope geotechnical investigation carried out within the site. A detailed geotechnical investigation of the site should be carried out to depths below the proposed excavation to confirm these comments and recommendations to allow detailed design of the proposed structure.

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 will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 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) should be expected. We strongly recommend that this issue 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.

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 structures.

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.

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