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REPORT

TO

HUDSON SQUARE PTY LTD

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED COMMERCIAL OFFICES & STUDENT ACCOMMODATION DEVELOPMENT

AT

157-163 CLEVELAND STREET REDFERN, NSW

10 March 2010

Ref: 22386Zrpt3

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1 INTRODUCTION

This report presents our geotechnical assessment for the proposed commercial offices and student accommodation development at 157-163 Cleveland Street, Redfern, NSW. The assessment was commissioned by Mr Robert Sargis of Hudson Square Pty Ltd, by signed 'Acceptance of Proposal' form dated 17 February 2010. The commission was on the basis of our proposal (Ref: P22386Zemail) dated 1 February 2010.

We understand from the provided architectural drawings (Job No 0722, Drawing Nos DA0-1 to DA-10, all Amendment A) prepared by Fortey & Grant Architecture, that the proposed development will comprise a four storey building over a basement level. The proposed basement level will extend to the eastern, and to the eastern end of the northern site boundaries and will require a maximum excavation depth of about 3.3m to achieve the finished basement reduced level (RL) at 14.88m. We have assumed that typical structural loads for this type of development apply.

The purpose of this assessment was to infer the geotechnical information on subsurface conditions at the site as a basis for preliminary comments and recommendations on excavation conditions, excavation support, footings and on-grade floor slabs.

2 ASSESSMENT METHODOLOGY

A desk study was carried out of previous geotechnical investigations we have carried out within approximately 500m of the site. In addition, published geological maps and orthophotos of the site area were reviewed. The search of our database revealed a total of ten relevant investigations within the study area, two of which were within 100m to the north-west and south-west of the site respectively.

3 SITE DESCRIPTION

We recommend that the site description which follows be read in conjunction with attached Figure 1.

The site is located just back from the south-eastern corner of the Abercrombie and Cleveland Streets intersection, and extends to Hudson Street along the south and Hart Street along the east. The site itself has a truncated triangular plan shape, being between approximately 30m and 59m deep (north to south) by an average of about 29m wide (east to west) and covers a total area of 2,688m^2.

At the time of this assessment, two buildings occupied the entire site. The western portion was occupied by a one and two storey rendered building. The eastern portion was occupied by a one storey rendered building.

Relatively new three storey rendered buildings and older two storey brick industrial style buildings were located across the street frontages.

4 SUBSURFACE CONDITIONS

The 1:100,000 geological map of Sydney indicates that the site is located close to the contact between alluvial deposits to the north-west and the underlying Ashfield Shales, which in turn are underlain by the Hawkesbury Sandstones. Previous investigations carried out in the immediate vicinity indicate that the likely subsurface profile will comprise sands over clays with shale and/or sandstone bedrock at moderate depth. A detailed description of the likely subsurface horizons is presented below:

- Sands (both fill and natural) are likely to be encountered from surface and extend to depths up to 3m. The sands are likely to be loose improving to medium dense with depth.
- The sands are likely to be underlain by soft to stiff alluvial clays of medium plasticity and/or very stiff to hard residual clays of high plasticity.
- Sandstone bedrock is anticipated at depths between 5m and 10m and may possibly be overlain by shale bedrock. The sandstone is probably of low strength improving to medium and high with depth, although medium and high strength sandstone from first contact cannot be discounted.
- Groundwater is expected at around RL11m to RL13m (ie, about 3m to 4m depth).

5 PRELIMINARY COMMENTS AND RECOMMENDATIONS

The comments and recommendations which follow are of a preliminary nature as they are based on inferred subsoil conditions obtained from other nearby sites. We recommend that once the existing buildings on site have been demolished, a comprehensive geotechnical investigation be carried out (refer Section 5.5 below).

We note that if the sands in particular, and also possibly clays, which are associated with the alluvial deposits to the north-west extend to the subject site, then groundwater could be an issue with significant impact on the design and construction of the basement.

5.1 Excavation Conditions

5.1.1 Excavation Methods

The proposed bulk excavation to 3.3m depth will encounter the soil profile and may extend into the underlying shale bedrock.

Excavation of the soil profile (dewatered, if appropriate) may be readily carried out using conventional earthworks equipment, such as hydraulic excavators. Some of the underlying weathered shale bedrock of extremely or very low strength, if encountered, may also be excavated by a large bucket excavator, possible with some ripping. However, in the unlikely event that low to medium and higher strength bedrock is encountered, we expect it would be most effectively excavated using hydraulic impact rock hammers. This equipment would also be required for breaking up boulders or blocks, for trimming rock excavation side slopes, and for detailed rock excavations, such as for footings or buried services.

5.1.2 Vibration Risks

We recommend that considerable caution be taken if rock hammers are used for excavation on this site, as there will likely be direct transmission of ground vibrations to surrounding buildings and structures. The proposed excavation will be within approximately 5m of the neighbouring buildings to the east (across Hart Street) and 15m of the neighbouring buildings to the south (across Hudson Street). Prior to excavation with rock hammers commencing, detailed dilapidation reports should be compiled on the buildings and structures, to the east, west and south, and the owners asked to confirm that the reports present a fair record of existing conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage resulting from the proposed works. The dilapidation reports should be carefully reviewed prior to excavation commencing, so that appropriate equipment is used.

The excavation with hydraulic hammers, if used, should commence over the central portion of the site, using a hydraulic excavator fitted with a moderately sized hammer no larger than a Krupp 900 size or equivalent. Quantitative vibration monitoring should be carried out at the commencement of rock excavations. Subject to review of the dilapidation reports, we recommend that vibrations, measured as Peak Particle Velocity (PPV), be limited to no higher than 8mm/sec on the surrounding buildings and structures. If it is found that transmitted vibrations are excessive, then it would be necessary to change to a considerably smaller rock hammer or to use alternative excavation techniques. Alternative excavation techniques which will significantly reduce vibrations include the use of a rotary grinder or grid sawing in conjunction with ripping and hammering. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations if rock hammers are used:

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Operate hammers in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience in confined work, with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

5.1.3 Seepage

As stated above, if the site is underlain by alluvial deposits (sands in particular), then a dewatering system using wells or spear points, may be required so that excavation and construction can be completed under 'dry' conditions. The effects of dewatering on the surrounding buildings will need to be assessed during the detailed geotechnical investigation (refer Section 5.5 below).

If not, we would expect some groundwater seepage flows into the bulk excavation, particularly after periods of heavy rain. Seepage, if any, during excavation would be expected to be satisfactorily controlled by conventional sump pumping.

5.2 Excavation Support

5.2.1 Support Methods

On the basis of the provided architectural drawings, temporary batters (1Vertical (V) in 1.5 Horizontal (H) can only be accommodated along the western and southern excavation faces. Where such temporary batters cannot be accommodated, the proposed bulk excavation will require to be supported by an engineered retention system which is installed prior to excavation commencing. A suitable retention system includes a contiguous or secant pile wall. The use of grout injected auger (also known as CFA) piles may be necessary if sands and/or groundwater are encountered. The piles should be progressively anchored as excavation proceeds.

Should the groundwater level be within the alluvial sand profile above bulk excavation level, then a secant pile wall would be necessary and should extend into the clay or shale profile to act as a cut-off. Alternatively, a contiguous pile wall would suffice. If the perimeter pile wall is to support structural loads, then the piles would need to be installed into bedrock below bulk excavation level.

Construction of the contiguous pile walls should be of high quality, taking care to prevent soil loss through gaps that will most likely occur between the piles, as this could result in sand 'runs' leading to settlement occurring outside the excavation. Such gaps should be rectified progressively during excavation, such as by mass concrete infill or shotcrete.

5.2.2 Lateral Earth Pressures

The major consideration in the selection of earth pressures for the design of retaining walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of temporary or permanent systems to retain the excavation:

- For progressively anchored or propped walls, where minor movements can be tolerated (such as along the street frontages provided there are no movement sensitive buried services), we recommend the use of a lateral earth pressure distribution as indicated in attached Figure 2, of 6H kPa, where 'H' is the retained height in metres.
- For progressively anchored or propped walls which are supporting areas highly sensitive to lateral movement, the rectangular pressure should be increased to 8H kPa, where 'H' the retained height in metres.
- Any surcharge affecting the walls (eg. traffic, construction loads, nearby high level footings, etc), should be allowed for in the design using an 'at rest' earth pressure coefficient, K_o, of 0.55.
- Where a tanked basement is proposed, the lateral pressures due to the groundwater must be taken into account. Alternatively, for a drained basement, provision must be made to provide permanent and effective drainage of the ground behind the walls. Subsurface drains should comprise 20mm diameter PVC pipes which are grouted into holes or gaps between piles, with the embedded end wrapped in a non-woven geofabric (such as Bidim A34), to act as a filter against subsoil erosion.
- For piles founded in bedrock below bulk excavation level, an allowable lateral stress of 200kPa may be assumed for embedment design to achieve toe restraint.

- The proposed anchors will extend beyond the site boundaries, and the permission of the owners of the surrounding properties will be required prior to installation. It is likely that Cleveland Street is an RTA road and their permission will therefore also be required. Anchors should have a free length of at least 3m and should be bonded into bedrock of at least low strength, where an allowable bond stress of 200kPa may be tentatively adopted. All anchors should be proof-tested to 1.3 times the working load, under the direction of an experienced engineer or construction superintendent, independent of the anchor contractor. We recommend that only experienced contractors be considered for the anchor installation. We have assumed that permanent lateral support of the perimeter pile walls will be provided by the new structure. If not, permanent anchors will be required which should be designed for corrosion resistance and for long term durability.

It is inevitable that the excavation will induce movements of the ground that falls within the zone of influence of the excavation. Therefore, any existing buried services or infrastructure which falls within the zone of influence, would be susceptible to some damage due to excavation induced movements. The zone of significant influence can be defined as extending a horizontal distance out from the excavation perimeter equal to twice the soil depth. The actual wall movements are highly dependent on the construction sequence, detailing and quality of installation, and should be closely monitored in critical areas.

5.3 Footings

Given the number of suspended levels (4 No) and the approximate column grid of about 8mx10m within the basement, the proposed structural loads will need to be founded on relatively competent bedrock.

For piles founded in Class 4 shale or Class 5 sandstone (Pells *et al*, 1998) an allowable end bearing pressure of 1,000kPa is applicable. In addition, an allowable shaft adhesion value of 100kPa may be applied for rock sockets into the Class 4 shale or Class 5 sandstone. Alternatively, bored piles founded in Class 3 shale or Class 4 sandstone may be designed for an allowable end bearing pressure of 2,000kPa to 3,500kPa and an allowable shaft adhesion value of 200kPa to 350kPa. The above are based on serviceability criteria of settlements less than 1% of the minimum footing dimension.

All footing excavations should be inspected by a geotechnical engineer prior to pouring to confirm that adequate founding conditions have been achieved.

5.4 Lower Basement Floor Slab

Where a tanked basement is envisaged, the slab design must take the uplift forces due to external groundwater into account.

Alternatively, for a drained basement, underfloor drainage must be provided. The underfloor drainage should comprise a strong, durable, single sized, washed aggregate (such as 'blue metal' gravel). The underfloor drainage should connect with the wall drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system. Joints in the on-grade floor slab for a drained basement, should be designed to accommodate shear forces but not bending moments by using dowels or keys.

5.5 Geotechnical Investigation

Once the existing buildings have been demolished and access is available for a truck/track mounted rig, we recommend that a comprehensive geotechnical investigation of the site be carried out. We consider that a total of four boreholes would provide a suitable site coverage for the site area. The boreholes should be drilled to a depth of at least 2m below bulk excavation level of at least 2m into bedrock. The bedrock within at least two of the boreholes should be core drilled.

We further recommend that PVC standpipes be installed into the two augered boreholes to allow longer term groundwater monitoring. We note that the excavation conditions and excavation support will be highly dependent on groundwater conditions, and this aspect must be specifically investigated.

The comprehensive geotechnical investigation will provide specific geotechnical information on subsurface conditions on the site, as a basis for comments and recommendations on excavation conditions, excavation support, footings and on-grade floor slabs and groundwater.

5.6 Further Geotechnical Input

The following summarises the further geotechnical input which is likely to be required and which has been detailed in the preceding sections of this report:

- Comprehensive geotechnical investigation once existing buildings have been demolished.
- Dilapidation surveys of surrounding buildings and structures.
- Quantitative vibration monitoring during rock excavation.
- Geotechnical inspections of cut rock faces.

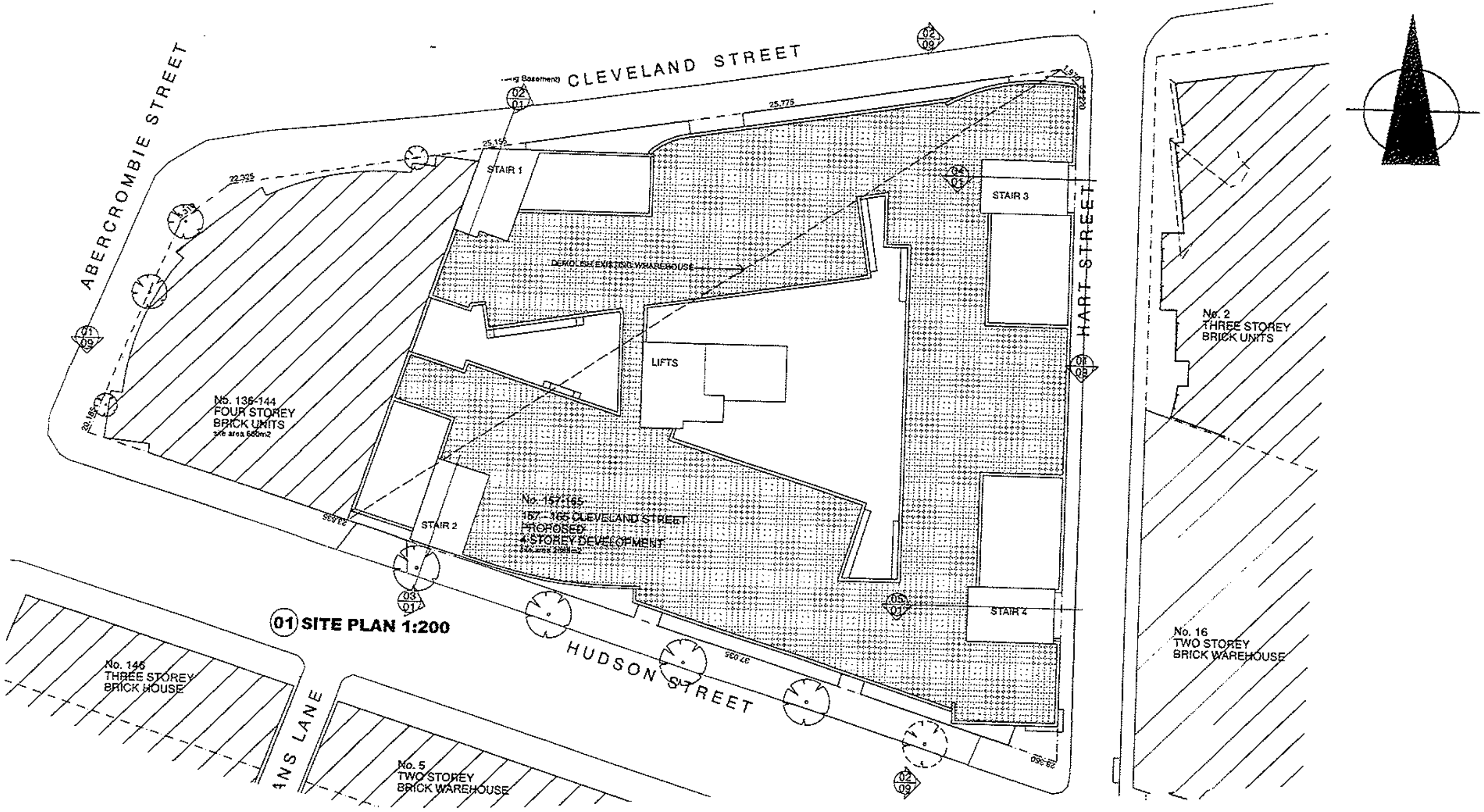
- Geotechnical footing inspections.
- Groundwater monitoring into bulk excavation.

6 GENERAL COMMENTS

This report has been prepared for the particular project and purpose 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 preliminary recommendations should be reviewed. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. 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.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

A ZENON
Senior Associate
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.



(01) SITE PLAN 1:200

SCALE (m)

0 25

SITE LAYOUT PLAN

Jeffery and Katauskas Pty Ltd
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Report No. 22386 Z **Figure No. 1**