



REPORT

TO

PTI GROUP

ON

**GEOTECHNICAL AND HYDROGEOLOGICAL
ASSESSMENT**

FOR

PROPOSED RETAIL AND RESIDENTIAL DEVELOPMENT

AT

**CORNER LINDFIELD AVENUE, KOCHIA LANE AND
HAVILAH LANE, NSW**

31 March 2011

Ref: 24013SPrptrev1

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS



TABLE OF CONTENTS

1	INTRODUCTION	1
2	PROPOSED DEVELOPMENT	1
3	ASSESSMENT PROCEDURES	2
4	SITE OBSERVATIONS	2
5	SUBSURFACE CONDITIONS	3
6	COMMENTS AND RECOMMENDATIONS	4
	6.1 Geotechnical Issues	4
	6.2 Excavation Conditions	5
	6.3 Excavation Batters and Retention	6
	6.4 Footing Design	8
	6.5 Hydrogeological Considerations	9
	6.6 Car Park Floor Slabs	9
	6.7 Geotechnical Subsurface Investigations	10
7	GENERAL COMMENTS	10

REPORT EXPLANATION NOTES



1 INTRODUCTION

This report presents the results of a geotechnical desktop assessment on the likely geotechnical issues for a proposed retail and residential development at the corner of Lindfield Avenue, Kochia Lane and Havilah Lane, Lindfield, NSW. The assessment was commissioned by Cissy Kang of PTI Group in an email dated 30 March 2011, and was carried out in accordance with our proposal of that same date.

The purpose of our assessment was to review available subsurface information from nearby projects and to provide our opinion on likely geotechnical issues associated with the proposed development and how such issues can be addressed during the design and construction of the development. We have also provided preliminary comments and recommendations on geotechnical and hydrogeological aspects of the proposed development.

A preliminary contamination assessment of the site has been undertaken by Environmental Investigation Services (EIS), and reference should be made to the EIS report reference E24013Krpt for details of the contamination assessment.

2 PROPOSED DEVELOPMENT

As part of our assessment we have been provided with the following architectural drawings prepared by PTI Group, Reference P194.2;

- Drawing DA00 Cover Sheet, Revision B
- Drawing DA02.5 Basement 3 Plan
- Drawing DA03 Basement 2 Plan, Revision B
- Drawing DA04 Basement 1 Plan, Revision B
- Drawing DA05 Ground Floor Plan, Revision D
- Drawing DA06 Level 1 Plan, Revision D
- Drawing DA07 Level 2 Plan, Revision C
- Drawing DA08 Level 3 Plan, Revision B



- Drawing DA09 Level 4 Plan, Revision B
- Drawing DA10 Level 5 Plan, Revision B
- Drawing DA11 Level 6 Plan, Revision B
- Drawing DA12 Level 7 Plan, Revision B
- Drawing DA14 Elevations Sheet 1, Revision B
- Drawing DA15 Elevations Sheet 2, Revision B
- Drawing DA16 Elevations Sheet 3, Revision B
- Drawing DA17 Sections Sheet 1, Revision A
- Drawing DA18 Sections Sheet 2, Revision B

From these drawings we understand that it is proposed to construct two levels of retail development with towers of four and six levels of residential apartments above, over three basement levels. Excavation for the basement will extend to about 86.2m AHD to achieve a finished floor level of 86.5m AHD, which relates to depths of about 8m to 12m below the existing surface levels. The basement will extend to the site boundaries.

3 ASSESSMENT PROCEDURES

Our assessment of the subject site has included the following;

- A brief site visit by a geotechnical engineer to assess existing site conditions.
- Review of subsurface information from nearby sites as a basis for preliminary comments and recommendations on the likely geotechnical issues affecting the site and proposed development.

4 SITE OBSERVATIONS

The site is located on the eastern side of the ridgeline passing in a north-south direction through Lindfield, with the surface in the vicinity of the site generally sloping down to the north-east at about 3° to 4°.



The site is irregularly shaped and is bounded to the west, south and east by Lindfield Avenue, Kochia Lane and Havilah Lane, with the exception of the south-eastern corner of the site where there is another property at the Kochia Lane/Havilah Lane intersection. The Avenue and Lanes are asphaltic concrete surfaced with concrete kerbs and gutters and concrete footpaths.

The property to the south-east of the site contains a two storey brick building over a lower parking area which is near the level of Havilah Lane at its eastern end, and cut about 2.5m below the adjacent level at the western end of that site.

To the north of the eastern end of the site is an on-grade asphaltic concrete carpark. To the north of the western end of the site are two brick terrace buildings which have been converted into retail. These structures appear to be in fair condition, with minor cracking in the brickwork at roof level on the front walls of these structures. These structures extend to the common site boundary.

The existing structures on the subject site comprise two and three level brick masonry retail developments and two level concrete frame structures with brick infill panels. These structures appear to be in fair condition.

5 SUBSURFACE CONDITIONS

The Sydney 1:100 000 Geological Series Sheet indicates that the site is located within an area mapped as being underlain by Ashfield Shale, but is close to the Hawkesbury Sandstone region which is just to the east/downhill.

Based on nearby investigations, we expect the subsurface conditions would comprise the following;



- Surficial fill will be present over some areas of the site, with slightly deeper fill in some areas possibly up to a depth of about 1m;
- Residual silty clays of high plasticity and typically very stiff strength;
- Weathered shale bedrock at relatively shallow depths of about 1m to 2m or so. We note that the weathered shale is likely to be of very poor quality, generally no stronger than very low to low strength and is likely to contain significant thicknesses of extremely weathered shale and shaly clay, but with stronger sandstone, siltstone and ironstone seams and bands;
- Shale bedrock of medium and high strength from depths of possibly 3m to 4m below the existing surface levels;
- Only minor groundwater seepage will probably be encountered within the excavation.

6 COMMENTS AND RECOMMENDATIONS

6.1 Geotechnical Issues

In our opinion from a geotechnical perspective, the proposed development is suitable for the subject site and will involve relatively common construction techniques and methodologies carried out on many sites throughout Sydney and within this area. We consider that the primary geotechnical issues relating to this development will be as follows;

- There will be variable excavation conditions and the requirement for retention of at least the soil and shale parts of the profile for both temporary and permanent cases. The temporary support of the shoring may require rock anchors extending beyond the site boundaries.
- We expect excavation conditions to be quite variable, comprising residual clay in the shallower areas and poor quality weathered shale with thick zones of clay, while the lower metres of excavation are likely to be in medium and high strength



sandstone probably requiring the use of rock breaker attachments to hydraulic excavators. Vibration effects (associated with general excavation but more critically sandstone excavation) on adjoining structures must be considered.

- The footings are likely to found within competent sandstone bedrock.
- Minor groundwater seepage could occur above bulk excavation level and therefore drainage behind the shoring and below the basement slabs is likely to be necessary.

These issues are discussed in more detail below.

6.2 Excavation Conditions

All excavation works will need to be carried out with reference to the Workcover NSW Code of Practice – Excavation Work (Cat 312).

Prior to any works commencing on site we consider that it would be wise to carry out dilapidation reports on adjoining buildings and roads. Dilapidation reports provide a benchmark for assessing any damage claims and it is recommended that the owners of the adjoining structures be asked to sign the reports to confirm that they present a fair assessment of existing conditions.

Excavation for the basement level will be to maximum depths of about 12m below the existing surface levels. We expect that the upper portion of the excavation will be through fill and residual soils and then into the weathered shale of up to very low to low strength, with medium and high strength sandstone expected in the deeper portions of the excavation. Excavation of soils and weathered rock up to very low strength will be able to be achieved with the buckets of larger hydraulic excavators. If very low to low or low strength shale is encountered then this will probably require some light ripping with say a ripping tyne fitted to a large excavator. Shale or sandstone of medium or higher strength is likely to prove effectively unrippable and



will require the use of hydraulic rock breakers fitted to excavators or a combination of sawing the sandstone into blocks with large diameter excavator mounted saws and ripping of the blocks with ripping tynes. We expect large hydraulic rock breakers will be used and therefore full-time quantitative vibration monitoring should be undertaken to confirm vibrations on adjoining structures are within tolerable limits.

6.3 Excavation Batters and Retention

As the excavation will extend to the site boundaries, temporary batters will not be feasible in this instance and permanent shoring will be required.

Where sandstone bedrock of medium or high strength is encountered within the proposed excavation depth, it will be difficult to pile through this material to found below the base of the proposed excavation and consideration could be given to founding above excavation level and providing additional anchors for lateral support. The shoring system will probably comprise soldier or contiguous pile walls probably with two levels of anchors for the temporary case, and possibly soil nail walls where there are no structures adjacent to the site. Further details of these options are provided below.

Anchored Soldier or Contiguous Pile Wall

If a pile wall is to be adopted, this could be soldier pile wall with reinforced shotcrete infill panels where it is adjacent to roadways and carparks. Where the pile wall will be adjacent to other buildings, a stiffer contiguous pile wall is preferable. Such walls should be socketed into the medium to high strength sandstone and, as this is likely to occur above the proposed bulk excavation level, at least two rows of temporary anchors will be needed until permanent support can be provided by the floor slabs. We note that it would be necessary to obtain permission from neighbours where anchors are to extend beyond the site boundaries. Further specific advice on lateral



earth pressures for design of an anchored soldier pile shoring system can be provided after specific subsurface investigations on the site are carried out. However as a guide a trapezoidal earth pressure of $6H$ kPa (where H is the depth of the excavation in metres) could be used for preliminary design purposes; this maximum pressure should be assumed to apply over the central 60% of the height of the shoring, tapering to zero at the crest and toe of the shoring. Where there are settlement sensitive structures or services within a distance equal to the depth of shoring from the excavation, the maximum earth pressure magnitude should be increased to $8H$ kPa. Appropriate surcharge loads and hydrostatic pressures are additional to the above. Depending on seepage levels and flows it may be necessary to pile using grout injected Continuous Flight Auger (CFA) techniques. Geotechnical inspections should be completed during the drilling of representative shoring piles to confirm the piles are extending to adequate depths and into appropriate strata.

It is likely that the sandstone bedrock of medium and high strength would be effectively self supporting, though this should be confirmed by investigation when access to the site is possible. Even if the sandstone is self supporting, geotechnical inspections will be necessary on each lift of excavation to observe for potentially unstable wedges of rock in the face which may require stabilisation.

Soil Nail Wall

An alternative to a soldier pile wall would be to use soil nailing with a reinforced shotcrete face. This system involves excavating to a depth of about 1.5m, installing a grid of rock bolts, tying mesh to the rock bolts and spraying the face with shotcrete. Following the application of the shotcrete, the next 1.5m of excavation can be conducted, and the process repeated to the base of the excavation or until better quality bedrock is encountered. On this site we expect that the soil nailing and shotcrete will probably extend to the top of the medium and high strength sandstone. Such a system has been found to be quite cost effective on other similar sites such as this. The soil nail option would be subject to further specific



geotechnical design, but the following could be used for preliminary costing purposes. Assume:

- The soil nails will be on a grid of 1.5m both vertically and horizontally;
- The upper soil nail must not be more than 0.5m from the surface;
- The lowest soil nail must not be more than 0.8m from the base of the shotcrete;
- The soil nail lengths would be approximately equal to the height of material to be retained (for a vertical face), slightly less if the face is laid back at about 4V in 1H , however soil nails would have an absolute minimum length of 4m;
- As the soil nail support will extend beyond the boundary, it will not be possible to use it as a permanent retention system and it will be necessary to brace separately constructed retaining walls from the slabs of the proposed structure.
- Shotcrete thickness would probably be 120mm to 150mm with centrally located SL82 mesh.

For this soil nail option permission would also need to be obtained from neighbours prior to installing soil nails into their property.

We would be pleased to carry out a soil nail wall design for you once the site specific subsurface investigations have been carried out.

6.4 Footing Design

Following bulk excavation, the exposed subsurface conditions in the base of the excavation are likely to expose sandstone bedrock of medium and high strength. Footings embedded into such rock should be suitable for an allowable bearing pressure of 3500kPa, though this is subject to confirmation following site specific investigation. Higher bearing pressures may also be proved feasible following investigation.



6.5 Hydrogeological Considerations

Minor seepage could be encountered above the bulk excavation level and may occur during or immediately following periods of extended wet weather. Therefore long term drainage of the basement should be allowed. This would comprise construction of subsoil drainage behind the shoring system and any retaining walls as well as subsoil drainage connected to a permanent failsafe sump and pump system placed below the car park floor slabs.

Seepage into such excavations is usually associated with seepage at the soil/bedrock interface, from perched water trapped in localised depressions in the bedrock and from general seepage flows from within fractures in the rock mass. Considering the expected low permeability of the site soils and weathered rock, we consider that removal of seepage is very unlikely to adversely affect nearby structures.

6.6 Car Park Floor Slabs

As discussed above we expect that bulk excavation will expose medium and high strength sandstone bedrock. Therefore the design of the car park floor slabs should incorporate a subbase layer of DGB20 or similar crushed rock, compacted to at least 98% of Standard Maximum Dry Density (SMDD). This will act as a separation/debonding layer from the weathered rock subgrade. Sand layers should not be used below trafficable slabs.

Joints in concrete pavements should be dowelled or keyed to resist shear forces but not bending moments.



6.7 Geotechnical Subsurface Investigations

The above comments and recommendations have been based on nearby investigations and experience. Prior to detailed design we recommend that a comprehensive geotechnical investigation be carried out on the subject site to assess the specific subsurface conditions and to provide an updated geotechnical report suitable for design purposes. We envisage at least 5 cored boreholes should be completed. In addition we recommend PVC piezometer standpipes be installed in two of the boreholes to check groundwater levels. At present access for a drill rig is quite limited and therefore at least some demolition would be required to enable access for drilling equipment.

7 GENERAL COMMENTS

The recommendations presented in this report are based on our nearby investigations. Therefore it is possible that the subsurface conditions on the subject site may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

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. 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 handwritten signature in cursive script that reads "P. Wright".

P Wright
Senior Associate

For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.