REPORT

TO
FRASERS BROADWAY

ON
GEOTECHNICAL INVESTIGATION

FOR
BLOCK 3A – KENSINGTON STREET PRECINCT

AT
CORNER OF BROADWAY AND KENSINGTON STREET,
SYDNEY, NSW

2 November 2012
Ref: 22905Srpt3A Rev2

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J&K BOREHOLE LOGS B4, B6, B16, P4, P5, P14, P15
URS BOREHOLE LOGS MW101, MW311, SB201 TO SB205 (INCLUSIVE)
JBS ENVIRONMENTAL BOREHOLE LOGS B01 TO B15 (INCLUSIVE)
FIGURE 1: BOREHOLE LOCATION PLAN
REPORT EXPLANATION NOTES

1 INTRODUCTION

This report presents the results of a compilation of geotechnical information obtained from geotechnical investigations undertaken within the Central Park site, for the proposed boutique hotel development for Block 3A. This geotechnical assessment was commissioned by Anthony Green of Frasers Property Australia Pty Ltd, in an email dated 17 September 2012. The scope of works was set out in the proposal by Jeffery and Katauskas Pty Ltd (now trading as JK Geotechnics) dated 8 September 2011 (Ref: 22905SBlocks3,6,7,10prop).

We have been provided with plans prepared by HBO+EMTB Heritage Pty Ltd which detail the existing 'Clare Hotel' and 'Admin Building' structures in which the facades are to remain. We have also been supplied with architectural drawings issued for planning approval prepared by Tonkin Zulaikha Greer Architects (Dwg Nos.A000 to A003, A100 to A107, A300 to A302, A400 to A403, A500 to A502, A700 and A701, Revision A, dated November 2012). Based on these drawings we understand the proposed development will comprise the demolition of parts of the internal structure of the 'Clare Hotel' and 'Admin Building' and construction of a four and five-storey boutique hotel within the remaining shell of the existing buildings. The hotel will be over the existing single basement level and also have a roof-top 'pool deck'. We understand that the new building will be supported in part by the remaining shell of the existing building and also by new columns which will be added to the "Admin Building'. As part of this development, the existing basement levels of the two buildings will be connected by a new underground ramp. We understand that at basement level the typical finished floor levels (FFLs) of the 'Clare Hotel' and 'Admin Building' are RL14.45m and RL14.85m, respectively. At ground level, the two buildings will be connected by a 'hotel foyer'. Above ground level, there will be bridges connecting the two buildings. As part of the development, a new lift pit, new stairwells and internal columns will be added.

As no structural loads for the proposed new works have been provided, we have assumed typical loads for this type of development.

The purpose of this assessment was to provide a compilation of information on the sub-surface conditions around Block 3A and to present our comments and recommendations on geotechnical aspects of the proposed development.

2 INVESTIGATION PROCEDURE

A number of previous investigations have been carried out on the site by ourselves and other consultants. The most relevant reports for this geotechnical assessment were carried out by

Jeffery and Katauskas and covered the whole of Basement 2, 5 and 9 (Ref:22905rpt259, dated 28 November 2009) and a pavement investigation along Kensington Street and Kent Road (Ref:22905Sroadsrpt, dated 2 March 2010). Other relevant reports were carried out by URS and covered the whole Kent Brewery Site and associated properties (Ref: 43187193 dated 5 October 2006) and boreholes were carried out by JBS Environmental Pty Ltd within the basement of Block 3a (Ref: JBS42253-51722, investigation carried out in August 2012). We have shown the relevant approximate borehole locations on Figure 1 and have used the information as part of this assessment. Copies of relevant borehole logs and explanatory notes which describe terms and symbols used and details of the exploratory techniques are attached with this report.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located at the Central Park development, Chippendale, NSW. The area of the current investigation lies to the north-east of the main One Central complex and has frontages to Broadway to the north and Kensington Street to the east and Carlton Street to the west.

During the time of writing, Block 3A contained two separate buildings referred to herein as the 'Clare Hotel' (northern building) and 'Admin Building' (southern building). These two buildings are separated by a disused loading dock/beer garden at ground level.

The key elements of the topography of the site are that it lies on the northern side of an old creek which drained westwards to the river channel which originally occupied the area of Abercrombie Street and in turn drained northwards to Blackwattle Bay. The creeks were culverted and infilled a long time ago and an old brick ovoid culvert lies a little to the south of Outram Street. Ground slopes have been modified by the filling but the area of interest is fairly level, with Broadway having a slope slightly down to the west.

3.2 Subsurface Conditions

The geology of the site and subsurface conditions including groundwater have been discussed extensively in our previous report Ref:22905Srpt259, dated 28 November 2009. In brief there is filling in the old creek channel which ran to the south of the site, overlying a shallow sandy deposit which infills the creek and covers the surrounding area to shallow depth and which form part of the Botany Basin deposits. These sands are known to have formed as sand dune deposits (aeolian sands) which have blown over the underlying shale and sandstone. These sands are found at higher elevations than the sandstone and are also overlying shales and residual clays derived from the shales. The site itself is located at the edge of the southern extent of Ashfield

Shales of the Wianamatta Group which have been found along the Broadway frontage of the site. Weathering of the Ashfield Shales produces residual clays which usually grade into weathered shale. These shales overlie the sandstone belonging to the Hawkesbury formation.

The more pertinent details of the encountered subsurface profile in the area of interest are discussed below. Reference should be made to the attached borehole logs for detailed descriptions of the subsurface conditions.

Pavements and Fill

In general, the JBS Environmental boreholes (B01 to B15 inclusive) which had been drilled within the basements of Block 3A, penetrated a concrete slab ranging in thickness from 0.1m to 0.35m. In eight of these boreholes, the concrete slab was overlying fill comprising gravelly clayey sand, bituminous gravels or sandy clay. Where fill was encountered, the fill extended to depths ranging from 0.4m to 1.1m depth. In seven boreholes fill was not encountered, and the concrete slabs were underlain by natural soils.

With reference to the J&K boreholes, which had been drilled outside Block 3A area, fill comprising sands or gravelly clays was encountered at depths ranging in thickness from 0.2m (B6) to 1.5m (P15). The deeper fill could be associated with buried services. In one pavement borehole (P4) fill was not encountered as natural soils were encountered from surface levels.

With reference to the URS boreholes, concrete slabs or pavers were penetrated in five nearby boreholes. Three boreholes encountered fill beneath the slabs comprising gravelly sands and extending up to 0.4m to 0.8m. Where fill was not encountered, natural soils were either encountered from surface levels or directly beneath the slabs.

Natural Sands

Natural silty sands and clayey sands were encountered in three JBS Environmental boreholes (B01, B03 and B13) and in five J&K boreholes (P4, P5, B4, B10, and B6) and all URS boreholes. The sands extended to depths ranging from 0.7m (B4) to 4.9m (MW101). In MW101, the sands were interbedded with a 700mm thick clay band at 2.8m depth. In general, the sands varied in relative density from very loose to medium dense.

Natural Clays

Natural silty clays were encountered beneath the natural sands in URS boreholes SB201, SB202, and MW311 and J&K boreholes B4, B6, B10. Natural sandy clays were encountered beneath the fill in B16. The silty clays were generally of medium to high plasticity, with some localised areas of low plasticity. The natural silty clays are of a residual nature derived from the weathering of the Ashfield Shales and either graded into shale or sandstone bedrock. The sandy clays were of low to medium plasticity and graded into weathered sandstone bedrock.

Weathered Shale Bedrock

Weathered shale bedrock was encountered below the silty clays in J&K boreholes B4 and B6 at the northern end of the site. The shale was assessed to be extremely to distinctly weathered and was encountered at 4.3m (B6) and 4.1m (B4) depth. The shale ranged from 0.4m (B6) to 1.0m (B4) thickness overlying weathered sandstone bedrock

Weathered Sandstone Bedrock

Weathered sandstone bedrock was encountered in six boreholes (B4, B6, B10, B16, SB201 and MW101) at depths between 3.8m (B16) and 5.1m (B4) below existing surface levels. The sandstone was assessed to be extremely to distinctly weathered and very low or low to medium strength on initial contact, becoming medium to high strength with depth.

Groundwater

Groundwater was encountered at depths between 2.7m (B16) to 3.8m (B4) depth within the natural sands. Groundwater seepage was noted within the fill in B16 at a depth of 1.3m.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

A summary of some of the main geotechnical issues is presented below. Further details are provided in the following sections of this report.

1. A retention system is required for the new underground ramp construction between 'Admin building' and 'Clare Hotel.'
2. New footings are to be founded within the sandstone bedrock as near-surface soils will not be capable of supporting substantial structures.
3. Footing details of the existing buildings are not known at the time of writing. Test pits should be carried out at the existing building walls to determine footing dimensions and founding materials.

4.2 Site Preparation

For any new pavement areas or slab on grade construction, the comments and recommendation provided below should be followed.

Following excavation to design levels, we expect the exposed subgrade at the new connecting basement level will consist of natural clays. If clayey subsoil is exposed to prolonged periods of rainfall, softening will result and site trafficability will be poor. If soil softening occurs, the subgrade should be over-excavated to below the depth of moisture softening and the excavated material replaced with engineered fill. If shrinkage cracking of the clay surface occurs during dry weather, then prior to pouring concrete slabs, the exposed surface should be sprayed with water and re-rolled to close up the surface cracks.

4.3 Excavation Conditions

All excavation recommendations should be complemented by reference to the Code of Practice *'Excavation Work'*, Cat 312 dated 31 March 2000 by WorkCover.

Excavation for this project is expected to be limited to excavation for the new underground ramp at basement level between the two buildings and excavation for new lift structures, stairwell bases, and new footings. Exact levels of the proposed 'service connection' have not been supplied, however we expect that levels will be similar to the existing basement levels. Therefore, we expect excavation in this area will be no greater than 3m to 4m deep.

Such excavation is expected to encounter existing pavements, fill, natural sands and natural clays, which are anticipated to be readily excavated using conventional earthwork techniques, such as the buckets of hydraulic excavators.

Excavations and retention systems will need to be carefully planned and scheduled so as not to have any adverse effects on the buildings and structures adjoining or above the excavation. During the excavation, every care should be taken to not undermine or render unstable the footings of any adjoining structures and to maintain stability in the long term. Test pits should be carried out under geotechnical supervision to assess the footing details and founding materials of the existing buildings at the earliest practicable opportunity.

4.4 Retention

Excavation will be required for the basement level underground ramp between the 'Clare Hotel' and 'Admin Building' structures. The following recommendations are based on the assumption that excavation will not extend below the water table.

Where space permits, temporary batters within the fill and natural soil profile should be no steeper than 1 Vertical (V) in 2 Horizontal (H). Such batters should remain stable in the short term, provided all surcharge loads, including construction loads, adjoining structural loads, etc., are kept well clear of the crest (at least 2H from the crest where H is the depth of excavation in metres). We anticipate that such temporary batters will be able to be accommodated for the lift pit excavations and the eastern side of the underground ramp excavation.

Where there is insufficient space, as is the case for the ‘new service connection’ excavation, the use of temporary batters will not be feasible. Therefore, the excavation will need to be supported by a properly designed retention system installed prior to the start of excavation.

A contiguous pile retaining wall is likely to be appropriate. We recommend that the piles be socketed at least 0.3m into the underlying sandstone bedrock, but deeper as necessary for bearing capacity or lateral stability.

Given the limited depth of excavation, piled retaining walls with no structures or surcharge loads within a zone of influence of 2H : 1V could be braced off the existing basement walls at each end of the shoring walls. Where walls are propped (or anchored) an ‘at rest’ earth pressure coefficient (K_o) of 0.55 should be used. The bulk unit weight of the soil may be taken as 20kN/m^3.

The recommended lateral earth pressure coefficients and pressures assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads, including adjacent structures, should be allowed for in the design. Full hydrostatic pressures should be considered unless measures are undertaken to provide complete and permanent drainage of the ground behind the wall.

Passive toe resistance of pile walls may be estimated on a passive earth pressure coefficient (K_p) of 3 provided an adequate factor of safety is applied due to the large strains necessary to generate full passive resistance. Alternatively a maximum allowable lateral resistance of 200kPa may be assumed for rock of very low strength; this is not additional to passive toe pressures due to the large discrepancy in strains at which the resistance is mobilised.

4.5 Footings

Reuse of Existing Footings

As the overall dimensions and depth of the existing footings along the existing walls of the buildings are unknown, assessing the load capacity of the existing walls is not feasible at this stage. We recommend test pits be carried out under geotechnical supervision to determine the footing details and founding materials of the existing buildings.

However, we consider that one possible approach would be to assess the loads that the existing walls are currently carrying, and keeping any additional loads imposed on the walls to equal to or less than the current loads. However, if the pattern or distribution of loading from the new structure differs from the current structure, different movements between different portions of the building may occur.

We note that given the age of the existing building, we consider that the existing walls and footings most likely do not meet current structural design standards, and adopting the above approach will require some risk. Alternatively, the new structure within the existing building shells could be supported on new footings as discussed below.

New Footings

Due to the moderate structural loads expected and to limit differential settlement between the new buildings and the existing structures/facades, we recommend that the new footings be supported on piles or deep pad/strip footings founded within the sandstone bedrock. Given the nature of the silty clays expected below basement level, we expect conventional bored piling techniques will be suitable. However, if subsurface conditions differ from expected, ie. substantial fill or natural sands are encountered at basement level, the use of Continuous Flight Auger (CFA) piling techniques may be required.

From the boreholes contained within our previous geotechnical report (ref:22905Srpt259, dated 28 November 2009) sandstone was encountered at the following depths/RLs:

Borehole Number	Surface RL (m)	Class V	Class IV	Class III (or better)
B4	16.78	4.1m / RL12.68	5.1m / RL11.68m	6.06m / RL10.72m
B6	16.95	4.3m / RL12.65m	4.7m / RL12.25m	5.63m / RL11.32m
B10	17.16	4.4m / RL12.76m	6.2m / RL10.96m	6.91m / RL10.25
B16	14.87	*3.8m / RL11.07m	4.29m / RL10.58	4.60m / RL10.27m
SB201	17.58	4.9m / RL12.68	-	-
MW101	17.55	4.9m / RL12.65m	-	-

Note: *Less than Class 5 rock strength.

Rock classification is based upon the system described by Pells et al, Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics, Dec 1998.

It should be noted that boreholes SB201 and MW101 only penetrated the top 0.2m of the underlying sandstone and that poor quality sandstone was observed within B16 down to RL10.58m.

Due to the limited information on the strength and consistency of the underlying sandstone at the site and the known poor quality sandstone observed within B16, it should be assumed that the Allowable End Bearing Pressure (AEBP) on the upper sandstone should be limited to 700kPa. Higher bearing pressures of 1000kPa may be used for piles founded within Class IV sandstone. Piles socketed at least 0.3m in Class III sandstone may be provisionally designed for a maximum AEBP of 3000kPa. However, further cored boreholes will need to be completed to assess the extent and competency of the underlying sandstone bedrock.

We recommend that at least the initial stages of footing excavation be inspected by a geotechnical engineer to confirm that a suitable founding stratum is being achieved. The base of all footings must be clean of any loose or softened material and unless tremie methods are used, must be free of any standing water prior to pouring concrete

4.6 Slab On Grade Construction

Slab-on-grade construction is considered feasible on condition that the subgrade is prepared in accordance with the recommendations given in Section 4.2 above.

Where concrete slabs are formed on-grade they should be separated from all other building elements. The purpose of this separation is to permit differential movements. Joints in the on-grade floor slabs should be provided with keyed or dowelled joints to accommodate shear forces but not bending moments.

A subbase layer of minimum 100mm of DGB20, compacted to at least 98% of Standard Maximum Dry Density (MMDD) should be considered beneath any concrete on-ground floor slabs as the clay is a very poor subgrade.

5 GENERAL COMMENTS

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

Occasionally, the subsurface conditions between the completed boreholes 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.

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.

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

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