# REPORT

TO
ALBION HOTEL

ON
PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR
PROPOSED MULTI LEVEL DEVELOPMENT

AT
CORNER OF HARRIS STREET AND GEORGE STREET
PARRAMATTA, NSW

14 January 2015
Ref: 27941Prpt

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5001
www.jkgeotechnics.com.au
Jeffery & Katauskas Pty Ltd, trading as
JK Geotechnics ABN 17 003 550 801

Date: 14 January 2015
Report No: 27941Prpt
Revision No: 0

Report prepared by:

P. Wright

Peter Wright
Senior Associate

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670



This Report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JK) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JK and its Client and is therefore subject to:

a) JK's proposal in respect of the work covered by the Report;

b) the limitations defined in the Client's brief to JK;

c) the terms of contract between JK and the Client, including terms limiting the liability of JK.

If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JK which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JK does so entirely at their own risk and to the fullest extent permitted by law, JK accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

## TABLE OF CONTENTS

## 1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed multi-level development at the corner of Harris Street and George Street, Parramatta, NSW. The investigation was commissioned by Mr Bruce Solomon on behalf of the Albion Hotel by return of an 'Acceptance of Proposal' form dated 13 November 2014. The commission was based on our revised fee proposal, (Ref: P39604Prev1) dated 12 November 2014, with additional work completed on the basis of our email dated 12 December 2014.

Based on our correspondence we understand that the development is currently at concept stage, though will most likely comprise commercial lower floor(s) with residential levels above, and possibly between two and five car parking basement levels. We understand that a high rise residential building is proposed with a few levels of commercial space above multiple basement levels.

The purpose of this limited investigation was to obtain preliminary geotechnical information on the subsurface conditions to allow assessment of the difficulty of designing and constructing a basement for the proposed development. Preliminary recommendations on other geotechnical aspects are also provided in this report.

## 2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 14 November and 18 December 2014 and comprised the drilling of three boreholes (BH1 to BH3 inclusive) to refusal depths ranging between 9.0m (BH3) and 9.7m (BH1 and BH2) using our truck mounted JK350 drill rig. All boreholes were completed using spiral auger drilling techniques with a tungsten carbide (TC) bit attached to the augers.

The borehole locations are shown on the attached Figure 1 which is based upon the supplied survey plan prepared by Denny Linker & Co. (Ref: 140926 and dated 13.10.2014) and were set out by taped measurements from features shown on that plan. The reduced levels of the ground surface at each borehole location were interpolated from spot levels shown on the survey plan, and as such should be considered to be approximate.

One of our geotechnical engineers was present on a full-time basis during the fieldwork, and they set out the borehole locations, nominated the testing and sampling, and prepared the borehole logs. The borehole logs are attached to this report, together with a glossary of the terms and symbols used in the logs.

The composition of the soil profiles was assessed by visual and tactile examination of the materials recovered during drilling. The strengths of the soils were assessed from Standard Penetration Test (SPT) 'N' values and from hand penetrometer readings on clayey samples recovered in the SPT split tube sampler. The strength of the bedrock was assessed from the observation of the penetration resistance of the augers, and examination of the recovered rock cuttings. We note that rock strengths estimated in this way are subjective and variations of say one strength order should not be unexpected.

Groundwater observations were made during and on completion of each borehole. A hand slotted PVC standpipe was installed in BH1 for monitoring of the water levels. During the second day of fieldwork on 18 December 2014, the water level in BH1 was remeasured.

For further details of the investigation techniques adopted, and their limitations, reference should be made to the attached Report Explanation Notes.

Testing for possible contamination of the soil and groundwater was beyond the agreed scope of this investigation. Laboratory testing of the soil and rock was also beyond the scope of this preliminary investigation.

## 3 RESULTS OF INVESTIGATION

### 3.1 Site Description

The site is located within relatively flat low-lying topography and is about 60m south of the Parramatta River.

At the time of the fieldwork the site was occupied over the northern half by the 'Albion Hotel' and over the southern half by a segmental block paved car park. The hotel comprised a 1 and 2 storey brick building which appeared in good external condition based on a cursory inspection. The paved car park surface also appeared to be in good condition. Small trees and shrubs were located in garden beds within the car park, and there were medium to tall trees in the neighbouring property to the south.

To the south of the site was an asphaltic concrete surfaced car park which appeared in fair to poor condition with significant root jacking of the surface observed. The surface level across the site boundary was essentially similar.

To the west of the site was a 1 & 2 storey brick warehouse building which was mostly set back by about 9m from the site boundary, with the exception of one portion toward the mid-length of the western boundary for a length of about 18m where the single storey warehouse extended up to the boundary. An AC surfaced driveway and car parking area extended between the warehouse and the common site boundary.

The site had frontages to Harris Street to the east and George Street to the north, with AC surfaced footpaths along both frontages.

### 3.2 Subsurface Conditions

Reference to the 1:100,000 Geological Series Sheet for Sydney suggest the site to be underlain by Ashfield Shale, but with alluvial soils a short distance to the east and south. The boreholes encountered pavement and then alluvial soils extending to depths ranging between 6.2m and 8.5m, which was underlain by weathered shale bedrock. Some of the more pertinent aspects of the strata encountered are discussed below. For further details of the conditions encountered at each location, reference should be made to the attached borehole logs.

***Pavements***

Borehole 1 (BH1) and BH2 were drilled through the southern car park of the hotel and encountered pavement comprising 80mm thick segmental block pavers over silty gravelly sand which extended to depths of 0.7m and 0.8m respectively. BH3 was drilled through the parking lane of George Street, and this encountered approximately 100mm of asphaltic concrete over silty gravel. The material below the car park and George Street appeared to be a crushed igneous rock roadbase material.

***Alluvial Soils***

Alluvial soils were encountered to depths ranging from 6.2m to 8.5m with the shallower soil profile being closer to the river. The alluvial soils primarily comprised fine to medium grained silty sand with minor proportions of clay fines, and with bands of silty clayey sand predominantly in the deeper portions of the boreholes. The sands were of loose relative density, becoming medium dense with depth.

The exception was in BH1 where low plasticity silty clay of hard strength and low plasticity silty sandy clay of very stiff strength were encountered from depths of 4.8m and 6.0m respectively. Organic clay of low plasticity and firm strength with fine to medium grained sand were then encountered from 7.0m in BH1.

***Shale Bedrock***

Shale bedrock was encountered from depths ranging between 6.2m and 8.5m with the deeper shale being to the south. The upper 0.7m to 1.4m of shale in each borehole was of lower strength, ranging between extremely low and low to medium strength. Shale of at least medium strength was encountered after not more than 1.4m penetration into the bedrock, which corresponds with a depth of about 9m.

***Groundwater***

Groundwater seepage was noted during the drilling at depths ranging between 5m and 6m, with water levels being measured at 5.4m to 5.7m depth on completion of drilling each borehole. A PVC standpipe was installed in BH1 on 14 November 2014 and a water level was measured in this standpipe at a depth of 4.12m below ground level on our return visit on 18 December 2014.

## 4 COMMENTS AND RECOMMENDATIONS

### 4.1 Demolition

Typically asphaltic concrete pavements surround the site, while to the south-west of the existing Albion Hotel Building, there is a warehouse structure extending locally to the site boundaries. The adjoining asphaltic pavements are not likely to be particularly sensitive to vibration damage, however care must be taken when demolition is being carried out in close proximity to the warehouse. This work will need to be carried out by suitably experienced (and insured) contractors.

Prior to the commencement of demolition, dilapidation reports should be prepared for the adjoining properties and infrastructure, providing photographs and descriptions of any defects. Copies of these reports should be signed by the owners of the adjoining properties to confirm they present a fair record of the existing conditions so they can be used as a benchmark in assessing potential claims of damage caused during the demolition or excavation.

Demolition of concrete footings, floor slabs and paved surfaces may require the use of hydraulic rock breaker attachments to tracked excavators. Although this site is not particularly problematic good practice dictates that saw cut slots be provided near adjoining buildings to reduce the risk of demolition vibrations being transferred to those adjoining buildings. Wherever possible, concrete slabs should be removed using saw cutting and the buckets of hydraulic equipment rather than hydraulic impact hammers.

### 4.2 Excavation Conditions

We assume the proposed basement will extend close to the property boundary, and in that regard, it will be necessary to install a shoring system prior to the commencement of excavation. As some of the soils will comprise sands below the water table, it will be necessary to have a shoring system that is relatively waterproof to prevent the uncontrolled inflow of water into the excavation, which could result in the loss of sand from beyond the shoring.

The excavation recommendations provided below should be complemented by reference to the Code of Practice ‘Excavation Work’, prepared by Safe Work Australia July 2014.

The boreholes indicated that soils were encountered to depths ranging from 6.2m to 8.5m, and the excavation of these soils should be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. The lower portion of the excavation will require dewatering to allow effective excavation of the deeper sandy soils.

Shale bedrock of at least low to medium strength was encountered at depths ranging from 6.5m (north) to 9.2m (south). Shale bedrock of at least low strength will require the use of techniques such as hydraulic rock hammers, ripping tynes, rotary grinders and/or rock saws.

Percussive excavation techniques, such as hydraulic rock hammers, are not ideal as there is a risk of densification and settlement of the sands from the vibrations, and a risk of damage being caused to the nearby structures. If hydraulic hammers are used, the vibrations transmitted to the adjoining buildings should be quantitatively monitored on a full-time basis during excavation. Where the transmitted vibrations are considered excessive it will be necessary to change to alternate excavation equipment, such as smaller rock hammers, ripping tynes, rotary grinders or rock saws.

## 4.3 Retention

As mentioned above, a shoring system will need to be constructed around the perimeter of the proposed basement prior to the commencement of excavation, and it will be important for this to be relatively waterproof to prevent the loss of soil by sand runs through the shoring wall.

It may be possible to install a secant pile wall to support the soil above the shale bedrock, and in that case the shoring should contain a nominal socket into the shale of at least medium strength. This may require some grouting of the soil beyond the pile wall if a lack of verticality in the piles results in gaps between the piles. The drilling of these piles to significant socket into medium and high strength shale would cause a high risk of causing settlements in the ground around the pile wall from removing excess sandy soil during the grinding of the rock sockets. We note however that it is very likely that the shale will require some degree of retention, as even good quality shale can contain continuous inclined joints that can cause stability issues, and their presence is usually not known until after they have been exposed in the basement excavation. Therefore it would be necessary following the excavation of the soil to install an additional pile wall set in from the perimeter secant pile wall through the soil to support the shale bedrock; this would then require the re-establishment of the piling plant, and also reduce the footprint of the proposed basement.

We consider a better approach would be to construct an anchored diaphragm wall constructed with a shield with rotating rock cutters such as the Hydrofraise system. During the excavation of each panel, the soil and weathered rock is supported by a bentonite slurry which is also used to transfer the spoil from the cutting head to the surface. This shoring system would then provide a relatively impermeable wall extending below the proposed basement level. The advantages of a diaphragm wall would be that its stiffness could be used to allow rows of temporary anchors to be relatively widely spaced, and the cut-off wall would then reduce the inflow of groundwater into the excavation. To allow the optimal design of the cut-off system would require the completion of packer tests within cored boreholes, and numerical analyses of the potential groundwater seepage.

Propped or anchored shoring systems may be provisionally designed using a rectangular/uniform lateral earth pressure distribution with a magnitude of 6H kPa within the soil and 4H kPa in the shale respectively, where H is the depth of excavation in metres, provided there are no settlement sensitive structures or services within a distance equal to the proposed excavation depth from the perimeter of the shoring. Where there are movement sensitive structures or services, the lateral earth pressures should be increased to 8H kPa and 6H kPa for the soil and shale respectively. It is likely that the shoring designed in this way will be relatively conservative, and it is likely that a soil/structure interaction analysis using programs Wallap and Plaxis would result in a less conservative design. We could assist with such analyses if requested.

It will be necessary to provide temporary lateral support to the shoring system using temporary anchors or props. Temporary anchors with bond zones in shale of at least medium strength may be provisionally designed for a bond value of 150kPa. Higher pressures may be feasible following additional investigation incorporating cored boreholes. Where the shoring system is socketed into shale of at least medium strength below the proposed basement excavation, the length of socket more than 0.5m below the proposed lowest excavation, including allowance for services and footing excavations, may be designed for an allowable lateral bearing pressure of 300kPa.

We assume the permanent support of the basement excavation will be provided by bracing from the proposed building.

### 4.4 Footings

A subsequent cored borehole investigation will be required to optimise footing design parameters, though we expect footings founded below the proposed basement level within consistent shale of at least medium strength could be designed for allowable bearing pressures of the order of 3,500kPa to 6,000kPa. Proving these pressures would require the drilling of at least five cored boreholes, probably with some degree of spoon testing of individual footings during construction.

### 4.5 Basement Floor Slab

As the basement floor slab will be subjected to traffic loads, a sand bedding layer is not suitable for use as a separation layer between the bedrock and the underside of the slab unless the basement will be tanked. This separation layer should comprise either a free draining gravel material, or a compacted DBG20 layer which would be resistant to the pumping of fines through joints in the slab.

If the building is tanked, the floor slab must be designed to be waterproof and to resist the uplift from the hydrostatic pressures, and in that case a sand bedding would be appropriate as there would be no potential for the pumping of fines.

### 4.6 Groundwater

If the project is referred to the New South Wales Office of Water (NOW), it is likely they will impose a discharge limit of 3 megalitres per year from the basement dewatering. In that case it would be necessary to undertake detailed seepage analyses to estimate the flows, though we consider it likely that this limit would be exceeded without waterproof shoring walls extending below the proposed basement excavation level.

The flowrate of the seepage will be dependent upon the depth to which impermeable shoring can be installed around the perimeter of the basement, and the permeability of the shale. It may be preferable to install a deeper and more expensive cut-off to prevent the costs associated with tanking the basement.

If the seepage will be at a rate that requires tanking of the basement, hydrostatic uplift pressure will increase with basement depth below the groundwater table, and these pressures will need to be resisted by the lowest basement floor slab which we expect would be restrained by the self-weight of the building in the long-term, though dewatering and/or temporary anchoring will be required during construction.

During the construction dewatering, it will be necessary to monitor the quality of the water being discharged, and on site treatment of the groundwater may also be required prior to disposal.

## 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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between and below 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 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 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.

# JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

# BOREHOLE LOG

Borehole No. **1**
1/2

**Client:** ALBION HOTEL
**Project:** PROPOSED MULTI LEVEL DEVELOPMENT
**Location:** CORNER HARRIS AND GEORGE STREETS, PARRAMATTA, NSW

**Job No.** 27941P | **Method:** SPIRAL AUGER JK350 | **R.L. Surface:** ≈ 6.5m
**Date:** 14/11/14 | **Datum:** AHD
**Logged/Checked by:** D.S./P.W.

| Groundwater Record | SAMPLES (ES, U50, DB, DS) | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel. Density | Hand Penetrometer Readings (kPa.) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 0 | | - | SEGMENTAL BLOCK PAVERS: 80mm.t | D | | | |
| | | N = SPT 8/5mm REFUSAL | | | | FILL: Silty gravelly sand, fine to coarse grained, brown, fine to coarse grained igneous gravel. | | | | |
| | | | 1 | | SM | SILTY SAND: fine to medium grained, brown. | D | L | | ALLUVIAL |
| | | N = 7 3,4,3 | | | | as above, but trace of clay fines. | | | | |
| | | | 2 | | | SILTY SAND: fine to medium grained, orange brown, with clay fines. | | | | |
| | | N = 30 9,13,17 | 3 | | | SILTY SAND: fine to medium grained, red brown. | | MD | | |
| ON 18-12-14 | | | 4 | | SC | SILTY CLAYEY SAND: fine to medium grained, orange brown. | M | | | |
| ON COMPLETION AND AFTER 2 HRS | | N = 14 6,6,8 | 5 | | CL | SILTY CLAY: low plasticity, light grey and red brown, with fine to medium grained ironstone gravel. | MC>PL | H | 400 400 400 | |
| | | N = 11 4,5,6 | 6 | | | SILTY SANDY CLAY: low plasticity, light grey, fine to medium grained sand. | | VSt | 200 250 280 | |
| | | | 7 | | | | | | | |