Statement of Environmental Effects, Proposed Wollongong Hospital Carpark

APPENDIX F

GEOTECHNICAL ASSESSMENT (JK GEOTECHNICS)

Cardno
Shaping the Future

REPORT

TO
NSW HEALTH INFRASTRUCTURE

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED CAR PARK

AT
WOLLONGONG HOSPITAL, 3 to 15 DUDLEY STREET
WOLLONGONG, NSW

3 December 2012
Ref: 26129Srpt

AS/NZS ISO 9001
Certified
Davis Langdon Certification Services

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed multistorey car park at Wollongong Hospital, NSW. The investigation was commissioned by Mei Soh of TSA Management Pty Ltd on behalf of the NSW Health Infrastructure, and was carried out in accordance with our proposal dated 21 September 2012 (Ref: 36018S).

Based on the supplied preliminary floor plan and cross section drawings by Cardno (Project No. NA80813070-, Sheet No. SK-10 & SK-11), we understand that the proposed development comprises the following:

- Demolition of properties Nos 104 to 108 New Dapto Street.
- Demolition of property at No. 13 Dudley Street.
- Demolition of existing engineering building along Dudley Street.
- Excavation down to an RL of approximately 30.27m along Dudley Street side, resulting in an excavation of approximately 10.0m below existing ground levels.
- Excavation down to RL 32.3m along New Dapto Road, resulting in an excavation of approximately 13.0m below existing ground level.
- Construction of a 5 level car park.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation, retention, footings, groundwater and pavement slabs.

2 INVESTIGATION PROCEDURE

Boreholes BH1 to BH7 were drilled using our truck mounted JK350 rig to total depths ranging from 5.97m to 14.45m below the existing ground surface. The boreholes were initially auger drilled to depths ranging from 2.96m to 11.7m and were then continued by diamond coring techniques using a NMLC core barrel with water flush. BH8 was hand augered to a depth 0.4m to obtain samples for contamination analysis and forms part of a separate report issued by EIS.

The borehole locations, as shown on Figure 1, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs, were estimated interpolation between spot levels shown on the supplied survey plan by Allen, Price & Associates (Ref: 25461-31, dated 15/8/11). The datum of the levels is Australian Height Datum (AHD).

The apparent compaction of the fill and the strength and relative density of the subsurface soils was assessed with reference to Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer test results on cohesive samples returned by the SPT split tube sampler. The strength of the upper weathered bedrock profile was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with the examination of the recovered rock cuttings and hence are very approximate. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with the correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests. The results of the point load strength index test are summarised on the cored borehole logs.

Groundwater observations were made during and on completion of drilling. However, the use of water for core drilling would have affected the groundwater readings taken on completion of drilling. No longer term monitoring of groundwater levels was carried out.

Our engineering geologist, Mr Ian Squibbs, set out the borehole locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs, and photographs of the rock core, are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA registered laboratories, for testing to determine soil pH, chloride, sulphate, standard compaction and four day soaked CBR testing. The test results are summarised in Tables A and B and Envirolab Test Report 81890.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on the upper reaches of a north facing hillside and is situated to the west of the main campus of Wollongong Hospital. The site is bounded to the south by Urunga Parade, to the east by New Dapto Road and to the west by Dudley Street.

The southern half of the site slopes down from Urunga Parade at varying degrees of between 5° and 10° to the north and northeast, with Dudley Street sloping down to the north-east at approximately 5°. The entry slope accessing the site from Dudley Street slopes down to the west at approximately 7°.

At the time of investigation the site was being used as an on grade car park, with numerous single and two storey weatherboard and brick buildings associated with the hospital, situated along the eastern boundary and southern half of the western boundary. The ground level of the buildings along the eastern boundary were between one metre and three metres higher than the car park levels, and were retained by concrete walls. The property at No. 106 New Dapto Road had a two metre high batter at approximately 45° down to car park level, and was covered in grass. The concrete retaining wall at the centre of the eastern boundary had a vertical crack approximately 10mm wide that extended up to one metre in length.

Along the western boundary a two storey brick engineering building cut back into the site from Dudley Street, with the maximum cut being approximately 6m towards the southern corner of the building. The cut was supported by a reinforced concrete retaining wall in seemingly good condition.

To the north of the site was a multi-storey car park with a helicopter landing pad on its top level. Entry to the lower level of the car park could be made from the subject site.

3.2 Subsurface Conditions

Reference to the 1:100,000 Wollongong-Port Hacking Geological Sheet indicates the site to be underlain by Budgong Sandstone which is a volcanic sandstone unit of the Shoalhaven Group.

In summary, below the asphalt pavements, the boreholes encountered a varying amount of fill over sandstone bedrock. Towards the north of the site a clay profile was encountered beneath the fill and above the sandstone. Further comments on the subsurface conditions encountered are provided below. Reference should be made to the attached borehole logs for detailed descriptions of the subsurface conditions.

Asphalt Pavement and Fill

Asphalt pavement was encountered in every borehole ranging from 80mm to 400mm thick. This was generally underlain by a gravelly sand sub-base layer. In BH2 to BH4 the sub-base was underlain by silty sand down to a maximum of 1.4m depth and contained trace charcoal and ash. In BH7 the pavement layer was underlain by a silty clay fill down to a depth of 1.0m.

Residual Soils

The thickness of residual soil varied substantially, extending to 4.7m depth in BH5 but only 1.2m and 2.4m in BH6 and BH7. In BH5 and BH7 a layer of silty clay was encountered beneath the fill which was assessed to be of low to medium plasticity and very stiff in strength in BH5 and soft to firm in BH7. This was underlain by sandy clay which was assessed to be of low plasticity and very stiff to hard in strength. In BH6 a sandy gravelly clay layer was encountered beneath the fill and was assessed to have medium plasticity and hard strength.

Sandstone

Sandstone was encountered in all of the boreholes from depths of 0.4m in BH1 down to 4.7m in BH5. Generally the sandstone was initially assessed to be of medium to high strength and distinctly weathered. In BH4 the sandstone decreased in strength to very low down to a maximum depth of 11.5m, and BH2 and BH3 both reduced to low and very low in strength after initial medium to high strength bands.

The cored sandstone in BH1, BH2, BH3 and BH5 was initially assessed to be distinctly weathered and of high strength, but all contained very low and low strength bands with core loss zones of up to 1.0m (BH1). The sandstone improved to slightly weathered and consistently high strength from depths ranging from 6.8m (BH1) to 11.5m (BH4). The cored sandstone in BH6 and BH7 was generally assessed to be slightly weathered to fresh and of high strength.

The boreholes in the southern half of the site contained numerous defects that comprised extremely weathered seams up to 140mm thick, bedding partings and joints inclined up to 90°. The boreholes in the northern part of the site were less fractured and only contained bedding partings.

In BH2 at a depth of 2.2m, white silty clay approximately 1m thick was encountered between the sandstone layers. This could be indicative of an igneous intrusion (dyke or sill) that has weathered down to a residual clay, but this could not be confirmed without petrographic analysis. In short, there is considerable variation in the degree and depth of weathering and fracturing in the rock at the site, the geological explanation for which is not clear but may well become apparent during the bulk excavation.

A preliminary engineering classification of the sandstone bedrock (in accordance with Pells et al. 1998) has been carried out based on the boreholes and is tabulated below. We note that the engineering classification has not taken into account specific footing sizes, pile types, pile diameters and founding levels, and is therefore only indicative.

BH	Approximate Surface RL (mAHD)	Depth(m)/Top of RL(mAHD)				
		Class V	Class IV	Class III	Class II	Class 1
1	42.5	-	0.5/42.0	6.0/36.5	6.8/35.7	-
2	44.0	-	3.2/40.8	6.5/37.5	7.2/36.8	-
3	42.0	-	1.0/41.0	-	-	-
4	39.8	1.4/38.4	-	-	11.7/28.1	13.0/26.8-
5	33.0	4.8/28.2-	6.8/26.2	9.0/24.0	10.0/23.0	-
6	37.0	-	1.2/35.8	-	3.0/34.0	-
7	35.4	-	2.4/33.0	-	-	3.8/31.6

Groundwater

All of the holes were dry on completion of augering with the exception of BH4 which encountered groundwater at a depth of 11m. Groundwater was measured within the boreholes on completion of coring at depths ranging from 2.5m down to 5.5m. As water is used in the coring process, these groundwater levels have most likely been affected by the introduced drill flush water.

A pre-existing monitoring well was located in the northern half of the car park and had a standing groundwater level of 3.2m when dipped.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on recovered rock cores correlated well with our field assessment of bedrock strength. The Unconfined Compressive Strength (UCS) of the sandstone, estimated from the point load strength index test results, generally ranged from 8MPa to 60MPa, with some lower results of 2MPa to 4MPa and some higher results up to 76MPa.

The soil pH test results were between values of 6.3 and 8.4, which showed the samples tested to be slightly acidic to slightly alkaline. The sulphate and chloride results were all below 100mg/kg with the exception of the high strength sandstone in BH6 which recorded a sulphate value of 480mg/kg.

The four day soaked CBR tests carried out on the residual silty clay samples resulted in values of between of between 2% and 8%, when compacted to 98% of Standard Maximum Dry Density (SMDD) and surcharged with 9kg.

4 COMMENTS AND RECOMMENDATIONS

4.1 Principal Geotechnical Issues

The primary geotechnical issues associated with the proposed development will be to maintain stability to adjoining buildings, roads, buried services and footpaths, both during excavation and in the long term, as well as reducing the risk of vibration induced damage to adjoining buildings during excavation.

The geotechnical investigation has provided a basis for the comments and recommendations which follow. However, it will be essential during excavation and construction that frequent geotechnical inspections are carried out to assess exposed subsurface conditions, so as to provide appropriate geotechnical advice, especially in view of the anomalous geological conditions which have been identified.

As no boreholes were drilled along the New Dapto Road boundary, all assumptions have been made based on the holes drilled within the car park, which has ground levels up to 4m lower than New Dapto Road. Furthermore Boreholes 1, 2, 3 and 6 do not penetrate sufficiently below the proposed founding level to indicate the quality of the bedrock foundation; this arose as we were not informed of the proposed bulk excavation level until after the fieldwork was completed. Additional boreholes should be completed as soon as the opportunity arises.

4.2 Excavation

We recommend that prior to the commencement of demolition and excavation, detailed dilapidation survey reports be completed on the properties at 15 & 17 Dudley Street, 4 Urunga Parade and 110 New Dapto Road which are to remain and border the excavation.

The dilapidation surveys should include detailed internal and external inspections where all defects should be rigorously described including defect type, length, width and location. A photographic record is not sufficient.

The respective owners should be 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 arising from the works. We could prepare a fee proposal to carry out the dilapidation surveys, if requested.

Council may also request dilapidation surveys be completed on the adjoining footpaths and road surfaces adjacent to the subject site.

Excavations for the lowest level of car park will be required to depths ranging up to about 13.0m in the south-eastern corner along the New Dapto Road side. Such excavations will encounter fill, residual silty clays and sandstone bedrock. The sandstone bedrock will range from low to high strength, with higher strength bedrock being encountered towards the base of the lowest level at the northern end of the site.

Excavation of the soils and any upper rock of extremely low strength should be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Excavation of sandstone bedrock of low strength or stronger will require assistance with rock excavation equipment, such as ripping tynes, hydraulic rock hammers, rotary grinders or rock saws. It is likely that the medium to high and high strength sandstone bedrock with widely spaced fractures will require the full time use of rock saws and hydraulic rock breakers. As the site is of substantial size it will be feasible to use a heavy tractor (such as Caterpillar D10 or equivalent) to rip substantial quantities of rock, though the high strength sandstone with widely spaced fractures will not be rippable. Hydraulic rock hammers must be used with care due to the risk of damage to the adjoining structures from vibrations generated by such equipment. Where hydraulic rock hammers are used quantitative monitoring of vibrations transmitted to the adjoining properties should be carried out during the excavation works to reduce the risk of vibrations exceeding tolerable limits. It is likely that the use of hydraulic rock hammers would be limited where excavations are required close to adjoining structures (such as New Dapto Road) and equipment that results in less vibrations would need to be used, such as ripping hooks, rotary grinders or rock saws.

Any soil to be removed from site will need to be classified for disposal purposes prior to being removed from site. Environmental testing may also be required for the site as part of the approval process.

Some groundwater seepage will occur into the excavation, particularly during or following periods of wet weather. This seepage is expected to be controlled during excavation and construction, by using sump and pump techniques. In the long term drainage will need to be provided below the lowest car park level to collect and dispose of any seepage that occurs.

4.3 Retention

It is understood that the design will include shoring systems along the eastern and western boundaries and temporary batter slopes along the southern boundary. Such retention systems may comprise soldier pile retaining walls with shotcrete infill panels, or contiguous pile retaining walls where adjoining structures or movement sensitive services, are close to the subject site boundaries. Soldier piles should generally be at centre to centre spacings of about 1.8m, but where the excavations are within a distance of twice the excavation depth from adjoining structures, a closer spacing or contiguous pile walls should be adopted to provide a stiffer retention system and reduce adjacent ground movements.

Our preference is for shoring piles to extend at least 1m below the base of the excavations, including any local excavations for footings, services, lift pits, etc. This will require drilling shoring piles through sandstone bedrock of up to high strength and this would require the use of large pile drilling rigs. The alternative is to found the shoring piles on at least medium strength sandstone bedrock above the bulk excavation level. However this option requires temporary anchoring close to the toe of the piles, detailed inspections of the rock faces below the toe of the piles and staged excavation processes to reduce the risk that any adverse defects within the rock below the toe of the piles will affect the stability of the shoring wall. In the long term structural propping of the pile toes will have to replace the temporary anchors. A detailed construction and inspection methodology would need to be prepared and approved by the geotechnical engineers if this option is adopted.

Along the southern boundary where there is sufficient space, temporary batters within the soils and poor quality sandstone of extremely low to very low strength should be no steeper than 1 Vertical in 1 Horizontal (1V:1H) but with berms at least 1m wide at height intervals not greater than 3m reducing the overall slope somewhat. Vertical excavations should be feasible within good quality sandstone. Such batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well clear, say at least twice the batter height, of the crest of the batters. Permanent batters should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation. Permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, following construction to reduce erosion. All stormwater run-off should be directed away from all temporary and permanent slopes to also reduce erosion.

Vertical unsupported excavations would be possible within good quality sandstone of at least low to medium strength. However, the suitability of the sandstone to stand unsupported must be confirmed by regular geotechnical inspections of the cut faces to check for inclined joints or weak seams that require additional support. The geotechnical inspections should be carried out at depth intervals of no more than 1.5m to allow for any such additional support to be installed progressively during excavation. Allowance for some additional support in the form or rock bolts and/or shotcrete and mesh should be made within the project budget.

Design of cantilevered retaining walls no more than 3m in height may be based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.33 and a bulk unit weight of $20kN/m^3$ for the soils and poor quality sandstone. Where walls are constructed in front of good quality sandstone the wall loads would be dependent upon the defects within the rock which can only be determined by inspection of the cut faces and confirmation by a geotechnical engineer. For budgetary purposes we suggest an assumption of design loadings of 10kPa.

Propped or anchored walls should be designed based on a rectangular lateral pressure distribution of magnitude 6H kPa (where H is the retained height of soils and poor quality sandstone in metres) where some movements are tolerable and existing structures or movement sensitive services are located beyond 2H of the wall. Where structures or movement sensitive services are located within 2H of the walls, a higher rectangular lateral pressure distribution of 8H kPa should be used.

The above coefficients and pressure distributions assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients or pressures would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads 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.

Anchors should have their bond within sandstone of at least low strength and may be designed based on a bond stress of 250kPa for sandstone of low strength or 400kPa for sandstone of medium or higher strength. The anchors bond should have a minimum free length of 3m and a bond length of at least 3m formed outside of the active zone of the retention system. Anchors should be proof loaded to at least 1.3 times their working loads before locking off at about 85% of the working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to check that the anchors are holding their load.

Permission will need to be obtained from the RMS and council with regards to anchoring below the New Dapto Road and Dudley Street. Such permission can take some time to obtain and we recommend that permission be sought early in the process to avoid project delays.

4.4 Groundwater

There is no reliable groundwater information for the site, however, given the depth of excavation, seepage must be expected primarily through defects in the rock and retention systems will require drainage behind walls and below basement slabs to be provided in order to prevent the build-up of hydrostatic pressures. Drainage below the basement slab should be installed through either a drainage blanket or a network of subsoil drains. The volume of seepage flow is not expected to be very great, but this can only be confirmed following completion of bulk excavation at which stage the hydraulic engineer should confirm any nominated design is fit for purpose.

4.5 Footings

Following bulk excavations, sandstone will be exposed within the vast majority of the proposed building area. Therefore, the proposed building should be supported on footings founded entirely within the sandstone to provide uniform support and reduce the risk of differential settlements. Pad or strip footings would be appropriate for the majority of the site, but piles will be required in the area of BH5 to extend through the residual clay into the underlying sandstone bedrock. As Boreholes 1, 2, 3 and 6 do not penetrate to sufficient depth to provide direct information on the quality of the bedrock below footing level, the foundation quality can only be inferred at present and if bearing pressures greater than about 1,000 kPa are proposed then additional cored boreholes will be required to verify ground conditions. We also recommend additional boreholes adjacent to New Dapto Road.

At bulk excavation level footings founded on the Class III sandstone of at least medium strength may be designed for an allowable bearing pressure (ABP) of 3,500kPa, in which case all the footing excavations should be inspected by a geotechnical engineer to confirm that adequate sandstone has been encountered (provided that additional boreholes are completed prior to construction to confirm rock quality at depth). If allowable bearing pressures are limited to 1,500kPa on Class IV sandstone then only the inspection of selected footings spread throughout the basement would be required unless the bulk excavation reveals unusual geological conditions.

In the area of BH4, sandstone of very low strength was encountered at bulk excavation level so a reduced ABP of 1,000kPa should be adopted for this area. Alternatively a piled solution may be adopted founding on the better quality sandstone encountered from 11.5m (RL28.3m) depth, where an APB of 3,500kPa can be used.

In the area of BH5 bored piles extending to at least 4.8m below existing ground level can be designed for an ABP of 1,000kPa, which can be increased to an ABP of 3,500kPa if taken down to 9.0m depth. Piles in the area of BH4 and BH5 are likely to extend below the water table, so the use of pumps and/or tremie techniques may be required.

For piles in compression, allowable shaft adhesions of 10% of the above allowable bearing pressures may be used, below the nominal 0.3m socket and provided socket roughness and cleanliness is maintained.

The above allowable bearing pressures are based on a serviceability criteria of settlement of the footings or piles of no more than 1% of the footing width or pile diameter.

Based on the pH, sulphate and chloride results, the soils would be classified as 'mild' exposure classification for concrete piles and 'non-aggressive' exposure classification for steel piles in accordance with AS2159-2009 'Piling-Design and Installation'.

4.6 Car Park Floor Slab

As detailed above, we expect that the bulk excavation will encounter sandstone bedrock below much of the building footprint.

We recommend that the design of the car park floor slabs should incorporate a sub-base layer of DGB20 or similar crushed rock, compacted to at least 100% of Standard Maximum Dry Density (SMDD). This will act as a separation/debonding layer from the weathered rock subgrade and will also reduce the risk of pumping of fines from the subgrade through joints in the basement slab. Sand layers should not be used below trafficable slabs.

In the area of BH5 the pavement layer will be on the sandy clay which had a CBR value of 2%. Due to this low CBR value, thick pavements would be required unless some form of subgrade improvement is carried out. Such subgrade improvement works may comprise placement of a select layer of at least 0.3m thickness of good quality granular fill, such as crushed sandstone, with a CBR of at least 10%. The pavement thickness may then be designed taking into account the select layer, which could be achieved by adopting an overall equivalent CBR of say 7%.

4.7 Summary of Additional Geotechnical Work Recommended

- Additional boreholes near the New Dapto Road boundary and in the southern half of the site.

- Geotechnical inspection of excavation to check for adversely inclined defects and geological anomalies.
- Geotechnical inspection of founding material for shoring pile wall.
- Monitoring ground anchor installation.
- Geotechnical inspections/testing of founding material for footings.
- Assessment of drainage requirements after bulk excavation.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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