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REPORT

TO

HEALTH INFRASTRUCTURE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED SHOALHAVEN CANCER CARE CENTRE

AT

CORNER NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

8 April 2011

Ref: 24682WHrpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Shoalhaven Cancer Care Centre (SCCC) at the corner of North Street and Scenic Drive, Nowra, NSW. The investigation was commissioned by Mr Robert Mackellar of Taylor Thomson Whitting (NSW) Pty Ltd (TTW), on behalf of Health Infrastructure (HI), in an email dated 11 February 2011. The commission was on the basis of our fee proposal, Ref: P33479WH, dated 25 January 2011.

Based on the supplied architectural drawings prepared by Hassell (Project No. AX003042, Dwg No. SK-02_F, dated 25 March 2011 and Dwg Nos. SK-03_H and SK-04_H, dated 23 March 2011), we understand that a two to three storey SCCC building will be constructed on the western side of the site. Several smaller single storey residential units will be constructed on the eastern side of the site. A driveway with car parking spaces is proposed between the main SCCC building and the residential units. We have assumed that the driveway and car parking areas will be surfaced with a flexible asphaltic concrete (AC) pavement. The approximate outline of the proposed buildings, car parking and driveway areas, are shown on Figure 1.

The supplied architectural drawings do not indicate any design finished floor levels. We contacted Mr Peter Monckton of Hassell on 31 March 2011 and 1 April 2011 who indicated to us that the finished floor level (FFL) of the proposed Lower Ground Floor Level of the main SCCC building and the proposed residential units will be at reduced level (RL) 28.0m. Furthermore, Mr Monckton indicated that the FFL of the proposed Ground Floor Level of the main SCCC building will be at RL32.0m, with the south-eastern corner suspended over the proposed RT Treatment bunkers. To achieve the FFL for the proposed Lower Ground Floor Level, excavation to a maximum depth of about 3.8m below existing grade, will be required. Due to the sloping nature of the site, cut and fill earthworks to maximum depths/heights up to about 1m will also be required for the remaining portions of the proposed buildings.

In the supplied *'Geotechnical and Environmental Investigation – Brief and Offer of Service'* prepared by TTW, dated 20 January 2011, footings loads up to 1,000kN, have been indicated.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at six borehole locations, and based on the results obtained, to present our comments and recommendations on excavation conditions and support, site earthworks, retaining wall design parameters, footings, earthquake design parameters, the Lower Ground Floor Level slab-on-grade and external pavements.

We were also commissioned to carry out a Stage 1 Preliminary Environmental Site Assessment. This work was carried out by Environmental Investigation Services (EIS) [the environmental consulting division of the Jeffery and Katauskas Group] who prepared a report, Ref: E24682Krpt, dated April 2011. This geotechnical report must be read in conjunction with the above EIS report.

2 INVESTIGATION PROCEDURE

Prior to the commencement of the fieldwork, a 'Dial Before You Dig' search was undertaken and the borehole locations were electromagnetically scanned by a specialist sub-contractor for buried services. The borehole locations were nominated by Mr Troy Harvey of HI and were shown on a sketch plan which appeared to be overlayed on an extract of the site survey plan. Mr Harvey emailed the sketch plan showing the nominated borehole locations to us on 7 March 2011.

The fieldwork for the investigation was carried out on 10 & 11 March 2011 and comprised the drilling of six boreholes (BH1 to BH6), at the locations shown on

Figure 1. Figure 1 is based on the supplied survey plan prepared by Allen, Price and Associates (Reference No. 25451-01, dated 15 September 2010). The borehole locations were set out using tape measurements from existing surface features, as close as practical to the nominated borehole locations.

The boreholes were auger drilled to depths between 1.5m (BH2) and 4.3m (BH4), below existing grade, using our truck mounted JK350 drill rig. BH1 and BH4 were extended into the underlying bedrock by rotary diamond coring techniques, using an NMLC triple tube core barrel with water flush, to final depths of 5.00m (BH1) and 7.46m (BH4).

The approximate surface RLs indicated on the attached borehole logs were interpolated between spot level heights and ground contour lines shown on the supplied survey plan. The datum for the levels is the Australian Height Datum (AHD).

The nature and composition of the subsoils were assessed by logging the materials recovered during drilling. The strength of the subsoil profile was assessed from the Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer readings on clayey samples recovered in the SPT split spoon 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 examination of recovered rock cuttings and correlation with subsequent moisture content tests. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Groundwater observations were made in each borehole during the fieldwork. A slotted 50mm diameter PVC standpipe was also installed into BH3 for groundwater monitoring during the fieldwork and for possible future groundwater monitoring.

Our geotechnical engineer (Mark Tsang) was present on a full-time basis during the fieldwork to set out the borehole locations, direct the electromagnetic scanning, nominate the testing and sampling, direct the standpipe installation and to prepare the attached borehole logs, all under the direction of our Associate (Adrian Hulskamp). The Report Explanation Notes define the logging terms and symbols used.

Selected soil and rock chip samples were returned to NATA registered laboratories (Soil Test Services Pty Ltd and Envirolab Services Pty Ltd) for moisture content, Atterberg Limits, soil pH, chloride and sulphate, Standard compaction and four day soaked CBR testing. The test results are summarised in Table A, C and D. The Envirolab Services Pty Ltd *"Certificate of Analysis"* is presented in the attached Appendix A.

The recovered rock cores were photographed and returned to STS for Point Load Strength Index testing. The photographs are enclosed facing the relevant cored borehole logs. The Point Load Strength Index test results are plotted on the borehole logs and are also summarised in the attached Table B. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table B.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located within Nowra Park partway down an east facing hillside, which grades at about 3°. Sandstone cliffs at least 30m high are located about 25m to the west of the site. The Shoalhaven River meanders along the toe of the sandstone cliffs.

Nowra Park is bound by Scenic Drive to the west, North Street to the south and Shoalhaven Street to the west. The proposed SCCC development occupies the south-western corner of Nowra Park. The surrounding roads were surfaced with AC which all appeared to be in good condition.

At the time of the fieldwork, the site was undeveloped and vacant and covered with grass, scattered shrubs and medium to tall trees. In some areas the grass cover was patchy and residual silty sands and silty sandy clays were exposed. Trafficability for our 8.5 tonne truck mounted drill rig across the site was good.

During the fieldwork, our geotechnical engineer carried out a cursory inspection of the sandstone cliffs to the west of the site, from the top of the cliffs only. The sandstone bedrock was assessed to be sub-horizontally bedded, distinctly weathered and of at least medium rock strength, based on sounding with a geological hammer. We did not observe any groundwater seepage over the cliff faces. We did observe a number of large detached blocks or boulders of sandstone across and along the toe of the sandstone cliff face. We note that these blocks have been derived from previous collapses of detached blocks from the sandstone cliffs and have occurred over geological time. There was also vegetation growing within the cliff face, probably through extremely weathered bands and/or clay bands present within the rock mass.

We note that the supplied survey plan indicates a north-east/south-west oriented water main which passes through the proposed development footprint. Furthermore, an east/west oriented sewer main, with an approximate north/south oriented offshoot on the southern side of the main line, also passed through the southern side of the proposed development footprint. The diameter of the water and sewer mains, their invert levels, installation details etc are unknown.

The Shoalhaven District Memorial Hospital was located to the north of Nowra Park. The closest building within the neighbouring hospital to the north was set back at least 100m from the proposed development footprint. An AC surfaced on-grade car park belonging to the neighbouring hospital abutted Nowra Park, adjacent to the aforementioned closest building.

3.2 Subsurface Conditions

The 1:250,000 geological map of Wollongong indicates that the site is underlain by Nowra Sandstone of the Shoalhaven Group and this was confirmed by the investigation results and our site observations.

In summary, the boreholes encountered fill and residual soils overlying weathered sandstone bedrock at shallow depth. Reference should be made to the attached borehole logs, for details at each specific location. A graphical borehole summary is presented as Figure 2, which also shows the level of the proposed Ground and Lower Ground Floor levels. A summary of the encountered subsurface conditions is presented below:

Fill

Fill comprising silty sand was encountered from the ground surface in BH1, BH4 and BH5 and extended down to depths between 0.2m (BH4) and 0.5m (BH1) below existing grade. The fill contained inclusions of igneous and sandstone gravel. The fill was covered with grass.

Residual Soils

Residual soils comprising silty sand and silty sandy clay were encountered from the ground surface in BH2, BH3 and BH6 and below the fill in BH4 and BH5 and extended down to depths between 0.5m (BH2) and 1.3m (BH5). The residual silty sands were medium dense. The residual silty sandy clay was assessed to be of low and medium plasticity and very stiff and hard strength.

Where not underlying the fill, the residual soils were covered with a thin layer of silty sand topsoil of either 100mm (BH2 and BH3) or 200mm (BH6) thickness.

Weathered Sandstone Bedrock

Weathered sandstone bedrock was encountered below the base of the fill and residual soils in each borehole at depths between 0.5m (BH1 and BH2) and 1.3m (BH5) and extended down to the borehole termination depths.

The weathering and strength of the sandstone bedrock profile was extremely variable and ranged erratically from extremely weathered sandstone of extremely low strength to slightly weathered and fresh sandstone of high strength. The sandstone bedrock profile often comprised medium and high strength iron indurated bands, particularly within the extremely low and very low strength profiles. The weathered sandstone ranged from fine to coarse grained and also contained quartz gravel inclusions.

The cored portions of the bedrock contained defects including extremely weathered seams/bands, crushed seams and bedding partings. These defects were sub-horizontal. There were no inclined joints encountered in the cored bedrock portions. Several core loss zones were encountered and ranged from 0.13m to 1.14m thick and are inferred to be extremely weathered bands and/or clay band which have "washed away" during the coring process.

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 are therefore only indicative. These classifications should be reviewed once footing sizes/pile types and diameters and founding levels have been selected to confirm applicability within the zone of influence of such footings/piles.

BH	Approximate Surface RL (mAHD)	Depth(m)/ Top of RL Class V	Depth(m)/ Top of RL Class IV	Depth(m)/ Top of RL Class III	Depth(m)/ Top of RL Class II	Depth(m)/ Top of RL Class I
1	32.3	0.5/31.8	-	3.56/28.74	-	-
2	31.3	0.5/30.8*	-	-	-	-
3	30.5	0.71/29.79*	-	-	-	-
4	29.9	1.1/28.8	-	-	-	5.54/24.36
5	29.0	1.3/27.7*	-	-	-	-
6	27.6	0.9/26.7*	1.5/26.1*			

* Based on the weathering and rock strength from the augered borehole.

Groundwater

All boreholes were 'dry' during auger drilling and on completion of auger drilling. The standpipe in BH3 was also 'dry' after 24 hours. No further longer term groundwater monitoring has been carried out.

3.3 Laboratory Test Results

The results of the moisture content and Point Load Strength Index tests carried out on recovered rock chip samples and recovered rock cores, generally correlated well with our field assessment of bedrock strength. However, several of the moisture content tests were much lower than expected, probably due to high proportions of quartz gravel and/or ironstone gravel. The estimated UCSs ranged between 2MPa and 50MPa.

The Atterberg Limits and linear shrinkage test result confirmed our field classification of the site soils and indicated that the residual silty sandy clay sample tested from BH4 was of medium plasticity and had a moderate potential for shrink-swell reactivity with changes in moisture content.

The soil pH tests results ranged between values of 5.5 and 6.1, which show the samples tested to be slightly acidic. The soil sulphate and chloride test results were all less than 10mg/kg, which indicates very low sulphate and chloride contents.

The four day soaked CBR tests carried out on residual silty sand and silty sandy clay samples from BH2 and BH5, resulted in values of 25% and 8%, respectively when compacted to 98% of Standard Maximum Dry Density (SMDD) and surcharged with 9kg. The samples were compacted prior to CBR testing at close to their respective Standard Optimum Moisture Contents (SOMC). The insitu moisture contents of the samples tested from BH2 and BH5 were 8.6% and 2.7% 'dry' of their respective SOMCs.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation Conditions

Prior to the commencement of excavation, we recommend that reference be made to the WorkCover Authority of NSW's "Code of Practice – Excavation Work".

We note the presence of the existing water and sewer mains which run through the southern half of the proposed development footprint, as shown on the supplied site survey plan. Prior to the commencement of excavation and other site earthworks as discussed further below, we recommend that further details be obtained on these buried services from the utility provider, so that the services are not damaged and/or destabilised as a result of excavation. Depending on the invert levels of the pipes, temporary or permanent diversion of these services may be required. If the pipes will not be diverted, we recommend that the condition of the pipes be assessed by CCTV survey. The CCTV survey may then be used as a benchmark against which to assess possible future claims for damage arising from the works.

Dilapidation surveys of potential neighbouring buildings constructed within 30m of the proposed development footprint may also be required if construction of these neighbouring structures precedes construction on the subject site.

The initial stage of excavation will require all trees and shrubs (including their root balls), all grass, topsoil and any deleterious or contaminated fill within the development footprint stripped and then disposed appropriately off site. Reference should be made to the EIS report for guidance on the off-site disposal of soil.

To achieve the design FFL of the proposed Lower Ground Floor Level and Ground Floor Level of the main SCCC building, excavation to maximum depths of about 3.8m and 1m below existing grade, will be required, respectively. To achieve the design FFL of the proposed residential units, excavation to a maximum depth of about 0.5m below existing grade will be required. We also expect some minor excavation to achieve design subgrade levels for construction of the proposed car park areas and internal driveways.

Based on the borehole logs, most of the excavations will extend through the soil profile and into the underlying weathered sandstone bedrock.

Excavation of the soil profile and any extremely weathered sandstone bedrock can be carried out using a bucket fitted to a large hydraulic excavator. More effective excavation may be possible using buckets fitted with "tiger teeth". The sandstone bedrock of at least very low to low strength would be most effectively excavated using hydraulic impact rock hammers and/or by using ripping tynes fitted to a large excavator. The hydraulic impact rock hammers would also be required for breaking up of boulders or blocks or for trimming rock excavation side slopes and for detailed rock excavations such as for footings or buried services. Grid sawing techniques in conjunction with ripping or hammering will also help to facilitate excavation.

Higher bit wear of excavation attachments should be envisaged for this site due to the presence of medium and high strength iron indurated sandstone bedrock and bedrock which contains quartz gravel inclusions.

4.1.1 Potential Vibration Risks

We recommend that caution be taken during rock excavation on this site as there will likely be direct transmission of ground vibrations to nearby buried services.

Excavation procedures and the CCTV survey should be carefully reviewed prior to the commencement of excavation so that appropriate equipment is used.

Excavation with hydraulic impact rock hammers, if used, should commence furthest away from the sewer and water mains (i.e. commence over the northern side of the proposed Lower ground Floor Level) employing a hydraulic excavator fitted with a relatively low energy hydraulic rock hammer no larger than say, a Krupp 900 size or equivalent. To reduce the transmission of vibrations, we recommend that a perimeter vertical saw cut slot be provided and the base of the slot maintained at a lower level than the adjoining rock excavation at all times.

If hydraulic impact rock hammers are to be used in close proximity to the existing buried sewer and water mains, then the transmitted vibrations should be qualitatively monitored by a geotechnical engineer, at least in the early stages. Subject to review of the CCTV footage the ground vibrations in the vicinity of the sewer and water mains should be limited to 8mm/sec. Transmitted vibrations, if excessive, may cause damage to these services. If the transmitted vibrations are considered to be excessive, then it would be necessary to use a smaller rock hammer or alternative rock excavation techniques, such as rock sawing or using a rotary grinder.

When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations in the vicinity of the buried sewer and water mains, if hydraulic impact rock hammers are used:

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

4.1.2 Seepage

Based on the investigation results, we do not expect groundwater seepage flows into the excavation cuts. However, if excavation is carried out during or following periods of wet weather, groundwater seepage may occur at the soil/bedrock interface and through joints, bedding planes and other defects within the cut faces.

Seepage, if any, during excavation is expected to be satisfactorily controlled by conventional sump and pump techniques and/or gravity drainage down to the lower eastern areas of Nowra Park.

We recommend that groundwater seepage, if any, into the excavation be monitored by site personnel and the results (quantity, location, source, etc) reported to the geotechnical and hydraulic engineers so that any unexpected conditions can be promptly addressed.

4.2 Excavation Support

Based on the required excavation depths, temporary batter slopes through the soil profile would be feasible and should be cut no steeper than 1 Vertical (V) on 1 Horizontal (H), provided surcharge loads are kept well clear of the crest of the temporary batters, i.e. at least a distance away from the crest equivalent to the depth of the excavation through soil. Retaining walls can then be constructed along the toe of the temporary batters and subsequently backfilled.

If seepage occurs at the soil/bedrock interface, then localised instability at the toe of the soil batters may occur and therefore an allowance should be made for sandbag support at the toe of the batters.

We warn that where temporary batter slopes are adopted, particular care and design considerations will need to be made in relation to backfilling between the proposed retaining walls and the temporary batter slopes, particularly in those areas where the proposed Ground Floor Level extends beyond the footprint of the proposed Lower Ground Floor Level. Poorly placed backfill or backfill which has been inadequately compacted may settle resulting in damage to paved surfaces and landscaped retaining walls which are founded within the backfill. There is still a likelihood of settlement occurring over time even if the backfill is adequately placed and compacted which may be undesirable or not acceptable.

To reduce the likelihood of post construction settlements, the backfill materials will need to comprise a good quality granular material, such as the excavated sandstone bedrock (of at least low strength) or “blue metal”. The backfill materials would need to be properly placed, compacted and density tested. Where the excavated bedrock is used as backfill, the material would need to be compacted to a density ratio of not less than 98% of Standard Maximum Dry Density (SMDD). Particle sizes would need to be limited to 40mm and therefore would most likely require crushing of the excavated bedrock.

As an alternative, the backfill materials could be nominally compacted, with the proposed Ground Floor Level above entirely suspended and supported by footings founded within the underlying sandstone bedrock, as discussed further below in Section 4.4.

We expect that the weathered sandstone bedrock could be cut sub-vertically, although our preference is for a slight batter of about 1V on 0.25H. We recommend that the cut faces through the weathered sandstone bedrock be inspected at not more than 1.5m depth intervals to assess whether stabilisation of the rock cuts (e.g. shotcrete, mesh and dowels, rockbolts etc.) are required. Based on the investigation results, we expect that cut faces will require some stabilisation, such as by using shotcrete and mesh, particularly if thick extremely weathered bands and/or clay bands are encountered. ‘Dental’ treatment is also expected for any clay seams, extremely weathered seams etc that may be present. A provision should be made in the contract documents (budget and program) for the above inspections and expected stabilisation measures.

A toe drain should be provided at the base of all rock cuttings to collect groundwater seepage and lead it to a sump for pumping or for gravity fed drainage to the stormwater system.

If permanent batter slopes are envisaged, then we recommend slope angles no steeper than 1V on 2H through soil, though flatter batters may be required to facilitate maintenance, such as mowing (say 1V on 4H or even flatter). All permanent batter slopes should be protected from erosion by a quickly establishing grass cover, covering with shotcrete or similar approved material. This advice also applies to engineered fill, where required.

4.2.1 Retaining Wall Design Parameters

The major consideration in the selection of earth pressures for the design of the retaining walls is the need to limit deformations occurring outside the excavations. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of the retaining walls.

- All retaining wall footings should be uniformly founded in the underlying sandstone bedrock. For allowable bearing pressure recommendations, refer to Section 4.4 below.
- For free-standing cantilever walls which are retaining areas where movement is of little concern (i.e. where only garden or grassed areas are to be retained), a triangular lateral earth pressure distribution may be adopted with an 'active' earth pressure coefficient, K_a, of 0.35, for the soil profile and Class V sandstone bedrock, assuming a horizontal backfilled surface.
- For cantilever walls where the tops are restrained by the permanent structure or which retain areas where movement is of concern or for propped walls, a triangular lateral earth pressure distribution should be adopted with an 'at rest'

earth pressure coefficient, K_o, of 0.55, for the soil profile and Class V sandstone bedrock, assuming a horizontal backfilled surface.

- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile and Class V sandstone bedrock.
- Any surcharge affecting the walls (e.g. traffic loading, construction loads, compaction stresses during backfilling, inclined backfill etc) should be allowed for in the design using the appropriate earth pressure coefficient from above.
- The retaining walls should be designed as drained and measures taken to induce complete and permanent drainage of the ground behind the wall. Subsurface drains should incorporate a non-woven geotextile filter such as Bidim A34 to control subsoil erosion. All drainage water should be piped to the stormwater system.
- Lateral toe restraint may be achieved by fixing the walls to sandstone bedrock above bulk excavation level using rock dowels or by keying the walls into sandstone bedrock below bulk excavation level and below any adjacent footing or buried service trench excavations. An allowable bond/lateral stress of 150kPa may be adopted for dowel/key design. The dowels must be of sufficient length to engage a volume of rock to give global stability against sliding and overturning and should be installed at 45° to the cut face. For long term corrosion considerations we recommend that all permanent dowels be either hot dipped galvanised or stainless steel.

4.3 <u>Site Earthworks</u>

4.3.1 Site Drainage

The subgrade at the site, which is expected to be predominantly clayey, is expected to undergo a reduction in strength when wet. Furthermore, the clayey subgrade is expected to have a moderate shrink-swell reactive potential. Therefore, it is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade may become untraffickable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

4.3.2 Removal of Existing Trees

We note that the existing trees have likely caused localised "drying out" of the surrounding clayey soils. Removal of the trees for the proposed SCCC development will therefore lead to the recovery of the clay soil moisture content, resulting in differential swell movements across the site. The swell movements generated by the removal of the trees are in addition to the shrink-swell movements, which can occur in the clayey soils due to weather related natural moisture changes and by the reduction in surface evaporation subsequent to covering the site with buildings and pavements. The latter shrink/swell movements are outlined in AS2870-2011 ("Residential Slabs and Footings").

It is likely that moisture equilibrium in the clayey soils, following removal of the tree stumps and roots, could take at least one to two years to develop. In order to reduce the effects that removal of the trees will have on the proposed building and external pavements, we recommend they be removed as early as possible ahead of construction. Alternatively, the effects of tree removal can be reduced if the soils encapsulated within the primary root zone are boxed out following tree removal and replaced with nominally compacted clayey fill. The clayey fill should be slightly wet of SOMC and compacted in 100mm thick layers.

4.3.3 Subgrade Preparation

Following stripping of all vegetation and root affected soils and completion of bulk excavation, we recommend that in areas where a soil subgrade is exposed and pavements/on-grade floor slabs are proposed, the soils be proof rolled with at least eight passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes deadweight. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas.

If subgrade heaving is detected during proof rolling, then the heaving areas should be locally removed down to a stable base and replaced with engineered fill, as outlined below in Section 4.3.4, or further geotechnical advice should be sought. Further guidance on the treatment of heaving areas will be provided during the proof rolling inspection. Based on the investigation results, we do not expect significant heaving areas to be encountered, provide the earthworks are carried out during dry weather and not immediately following a period of wet weather.

If soil softening occurs after prolonged rainfall, then the subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the clayey subgrade exhibits shrinkage cracking, then the surface should be watered and rolled until the shrinkage cracks are no longer evident.

Engineered fill must be used to raise site levels up to design subgrade level.

4.3.4 Engineered Fill

Preferred materials suitable for use as engineered fill are well graded granular materials, such as crushed sandstone, on condition the materials are "clean", free of organic matter and free of particle sizes greater than 75mm. The residual soils and ripped sandstone would also be suitable, however, and as noted below, the compaction specification is more stringent than for granular materials.

Engineered fill comprising well graded granular materials, such as ripped or crushed sandstone, should be compacted in maximum 250mm thick loose layers to achieve a minimum density ratio of 98% of SMDD. Engineered fill comprising the excavated residual soils (which are expected to be predominantly clayey) and ripped extremely weathered sandstone bedrock should be compacted in maximum 200mm thick loose layers to a density ratio between 98% and 102% of SMDD and at a moisture content within 2% of SOMC.

Where space permits, we recommend that the engineered fill, where required, extend a horizontal distance of at least 1.5m beyond the design fill embankment slope so that adequate edge compaction can be achieved. On completion of filling, any excess fill can be trimmed off.

Density tests should be regularly carried out on the engineered fill and must confirm that the above specifications are achieved.

- The frequency of density testing for engineered fill should be at least one test per layer per $2500m^2$ or one test per $500m^3$ distributed reasonably evenly throughout the full depth and area, whichever requires the most tests.

- The frequency of density testing for backfill behind retaining walls should be at least one test per two layers per $50m^2$.

- The frequency of density testing for granular pavement materials should be at least one test per layer per 25 lineal metres, or three tests per layer, whichever requires the most tests.

We recommend Level 1 control of fill compaction, as defined in AS3798-2007, be adhered to on this site, as the pavements and on-grade floor slabs will be subjected to vehicular traffic. The geotechnical testing and inspection authority (GITA) should be directly engaged by HI and not by the earthworks contractor.

4.4 Footings

Based on the investigation results, weathered sandstone bedrock will be exposed at bulk excavation level within the Lower Ground Floor Level of the proposed main SCCC building and at Ground Floor Level within some of the proposed residential units. Sandstone bedrock is also expected at shallow depth even after completion of the earthworks and any site filling to achieve design levels. Therefore, for uniformity of support, we recommend that all footings for the proposed new buildings be founded within the underlying sandstone bedrock.

Pad and/or strip footings or bored piers founded in the sandstone bedrock would be suitable. Pad or strip footings and any bored piers founded in weathered sandstone bedrock, should be designed for a maximum allowable end bearing pressure of 800kPa. We have downgraded the allowable end bearing pressure that normally applies for Class V sandstone bedrock (i.e. 1,000kPa) due to the presence of thick extremely weathered sandstone bands within the weathered rock profile.

For bored piers, an allowable shaft adhesion of 80kPa (compression) and 40kPa (tension) would apply for rock sockets deeper than a nominal 0.3m socket into the weathered sandstone bedrock.

For footings located directly behind the crest of a sandstone cut face, the sandstone exposed below the toe of the footing must be inspected by a geotechnical engineer to identify any adverse defects or presence of poor quality bedrock, so that the bearing pressure for each particular footing can be assessed. A maximum allowable end bearing pressure of 400kPa would be applicable.

All footings should be excavated/drilled, cleaned out, inspected and poured with minimal delay. If a delay in pouring is expected, then we recommend a blinding layer of concrete be placed in the base of pad and strip footings excavations to protect the bases from deterioration due to weathering.

The above maximum allowable bearing pressures are based on serviceability, which results in settlement of less than 1% of the minimum footing dimension.

Where the proposed Ground Floor Level of the proposed buildings will be constructed at or close-to existing grade or engineered fill surface, we recommend that the perimeter and internal ground beams between pier heads be poured over void formers, which can accommodate heave movements of at least 20mm. This is due to the moderate shrink-swell nature of the residual clayey soils which are expected to be exacerbated by the removal of the existing trees.

Footings on the sandstone bedrock may also be designed using "Limit State Design" principles as detailed by Pells et al. (1998). An ultimate bearing capacity of 3MPa and an ultimate shaft adhesion value of 150kPa (compression) could be adopted for the Class V sandstone bedrock where piles are used. Settlement limitations to the structure will need to be satisfied and can be estimated using an elastic modulus value of 75MPa for the Class V sandstone bedrock.

It should be noted that such ultimate bearing pressures must be used in conjunction with an appropriate "*Geotechnical Strength Reduction Factor*" (ϕ_g). Provided there is good workmanship and quality control during footing construction, we recommend that a ϕ_g value of not greater than 0.55 be adopted for end bearing and shaft adhesion.

We note the above design recommendations for bearing and shaft adhesion are contingent on achieving good construction practice and an appropriate inspection and test plan.

4.5 On-Grade Floor Slabs

Based on the investigation results, the proposed Lower Ground Floor Level floor slab will directly overlie sandstone bedrock. We therefore recommend that underfloor drainage be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The underfloor drainage should connect with the wall drains and lead groundwater seepage to a sump for either pumped or gravity fed disposal to the stormwater system.

On-grade floor slabs should be separated from all walls, columns, footings, etc., to permit relative movements (i.e. designed as 'floating' slabs). However, an integral slab could be adopted for the proposed Lower Ground Floor Level due to uniform founding on the underlying sandstone. Joints in the concrete on-grade floor slabs should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

For any proposed on-grade floor slabs which will overlie soil and unless the soil comprises engineered fill, we recommend the exposed subgrade be proof rolled with at least six passes of a large sized (preferably at least ten tonnes by dead-weight) smooth drum roller. The last two passes should be under the direction of an experienced earthworks superintendent or geotechnical engineer. The objective of the proof rolling is to assist in the detection of unstable areas. If any unstable areas are exposed during proof rolling, these areas should be removed down to a sound base and replaced with engineered fill (as discussed above) or further geotechnical advice should be sought.

We note that unless a construction joint is provided in the Ground Floor Level Slab directly above the side walls of the Lower Ground Floor Level, that portion of the ground floor slab which extends beyond the side walls of the lower level, should be designed as suspended, to reduce differential settlements across the slab join.

We note that if a suspended floor slab design is adopted for the entire proposed ground floor level, then there is no necessity for proof rolling.

4.6 Soil Aggression

Based on the soil chemistry test results, a "mild" exposure classification results in accordance with AS2159-2009.

4.7 Earthquake Design Parameters

Based on the investigation results and in accordance with AS1170.4–2007, a Hazard Factor (Z) of 0.09 is applicable for the site, together with a subsoil Class B_e.

4.8 External Pavements

Based on the laboratory test results, we recommend that the proposed new pavements be designed for a CBR value of 8% or a short-term Young's Modulus of 30MPa for the subgrade.

We recommend that all base course materials comprise DGB20 in accordance with RTA QA Specification 3051 unbound base. The DGB20 material should be compacted using a large static roller to at least 98% of Modified Maximum Dry Density (MMDD). If recycled crushed concrete is proposed as a pavement material, then further advice should be sought in relation to design, QA testing and expected performance. Consideration must also be given to possible re-cementing of the recycled material and potential detrimental effects on the proposed AC surfacing.

We further recommend that all sub-base course materials comprise DGS40 in accordance with RTA QA Specification 3051 unbound base. Recycled materials may be used provided they conform to the specification requirements of DGS40 but consideration must be given to possible re-cementing of the materials. If the recycled materials contain brick or ceramic fragments, it is highly unlikely that they will conform to the specification requirements. The DGS40 material should be compacted using a large static roller to at least 95% of MMDD.

Adequate moisture should be added during placement of the base and/or sub-base so as to reduce the potential for material breakdown during compaction.

Density tests should be regularly carried out on the granular materials to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 1000m^2, or three tests per layer, or three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. The GITA should be directly engaged by HI and not by the earthworks contractor or sub-contractors.

We recommend that subsoil drains be provided with invert levels of at least 200mm below design subgrade level. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

4.9 Additional Geotechnical Input

We summarise below the previously recommended additional work that needs to be carried out:

1. Obtain further details of the existing water and sewer mains.
2. CCTV survey of water and sewer mains.
3. Vibration monitoring.
4. Rock face inspections.
5. Groundwater seepage monitoring.

6 Proof-rolling inspections.

7 Density testing of all engineered fill and granular pavement materials.

8 Footing inspections.

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 Jeffery and Katauskas Pty Ltd 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.

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 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.

Adrian Hulskamp
Associate

Reviewed by:

Agi Zenon
Senior Associate
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
SUMMARY OF LABORATORY TEST RESULTS

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
3	1.00-1.50	14.5				
3	2.50-3.00	2.8				
4	0.50-0.95	15.0	41	16	25	12.0
4	2.50-3.00	8.4				
4	3.50-4.00	6.9				

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions

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TABLE B
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	1.20-1.24	1.0	20
	1.81-1.85	0.1	2
	2.10-2.13	0.3	6
	3.56-3.60	0.9	18
	4.11-4.14	1.5	30
	4.79-4.82	1.2	24
4	4.35-4.38	0.2	4
	4.84-4.88	1.9	38
	5.25-5.28	1.1	22
	5.86-5.89	1.7	34
	6.31-6.35	2.5	50
	6.84-6.88	1.9	38
	7.27-7.31	2.2	44

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S(50)}$

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE C
SUMMARY OF FOUR DAY SOAKED C.B.R.TEST RESULTS

BOREHOLE NUMBER	2	5
DEPTH (m)	0.10 - 0.50	0.40 - 1.00
Surcharge (kg)	9.0	9.0
Maximum Dry Density (t/m^3)	1.78 STD	1.77 STD
Optimum Moisture Content (%)	12.0	15.5
Moulded Dry Density (t/m^3)	1.74	1.74
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	101	101
Moisture Contents		
Insitu (%)	3.4	12.8
Moulded (%)	12.1	15.6
After soaking and After Test, Top 30mm(%)	14.3	16.6
Remaining Depth (%)	13.2	16.0
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	0.0	0.0
C.B.R. value: @5.0mm penetration	25	8

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1

NATA

NATA Accredited Laboratory Number:1327

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Approved Signatory
(A.Tatikonda)

Date: 1/4/11

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TABLE D
SUMMARY OF LABORATORY RESULTS
SOIL CHEMISTRY - pH, SULPHATE & CHLORIDES

Borehole Number	Sample Depth (m)	Sample Description	pH Units	Sulphate (mg/kg)	Chloride (mg/kg)
BH1	0.50 - 0.63	XW Sandstone	6.1	<10	<10
BH2	0.50 - 0.95	XW Sandstone	5.9	<10	<10
BH3	0.50 - 0.71	Residual Silty Sandy Clay	5.5	<10	<10

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BOREHOLE LOG

Borehole No. **1**

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED SHOALHAVEN CANCER CARE CENTRE
Location: CNR NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

Job No. 24682WH
Date: 11-3-11
Method: SPIRAL AUGER JK350
Logged/Checked by: M.L.T./AJH
R.L. Surface: ≅ 32.3m
Datum: AHD

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
DRY ON COMPLE -TION OF AUGER -ING	ES		0			FILL: Silty sand, fine to medium grained, dark brown, with roots and fine to medium grained sandstone and igneous gravel.	M			GRASS COVER
	DS	SPT 17/130mm REFUSAL			-	As above, but light brown and brown with fine to coarse grained sandstone gravel.				
			1			SANDSTONE: fine to coarse grained, light grey and orange brown, with a trace of fine grained quartz gravel.	XW DW	EL M		VERY LOW 'TC' BIT RESISTANCE MODERATE RESISTANCE HIGH RESISTANCE
						REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL
			2							
			3							
			4							
			5							
			6							
			7							

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JEFFERY & KATAUSKAS PTY LTD
JOB NO.24682WH BH1 START CORING AT 1.20m
1
1.2m
CORE LOSS 0.28m
2
CORE LOSS
2.13 0.15m 2.28
2.37
CORE LOSS 1.14m
3
3.51
4
END BH1 AT 5.0m

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CORED BOREHOLE LOG

Borehole No. **1** 2/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED SHOALHAVEN CANCER CARE CENTRE
Location: CNR NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

Job No. 24682WH **Core Size:** NMLC **R.L. Surface:** ≅ 32.3m
Date: 11-3-11 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK350 **Bearing:** - **Logged/Checked by:** M.L.T./AJH

Water Loss/Level	Depth (m)	CORE DESCRIPTION (Rock Type, grain characteristics, colour, structure, minor components)	Weathering	Strength	Defect Description – Specific
	0–1.2	START CORING AT 1.2m			
	1.2	SANDSTONE: fine to coarse grained, light grey and brown, bedded at 0-5°.	DW	H	Cs,0°,30mm.t
		CORE LOSS: 0.28m			
		SANDSTONE: fine to coarse grained, orange brown and light brown, bedded at 0-5°.	XW-DW / DW	EL-VL / L	
		CORE LOSS: 0.15m			
FULL RET-URN		SANDSTONE: fine to coarse grained, orange brown and red brown, bedded at 0-5°.	DW	L	XWS,0°,50mm.t
		CORE LOSS: 1.14m			
	3.55	SANDSTONE: fine to coarse grained, light brown and dark brown, bedded at 0-10°.	DW / SW	M / H	Cr,0°,30mm.t
		as above, but light grey and light brown.			Be,0°,P,R,IS; Be,0°,P,R,IS
	5.00	END OF BOREHOLE AT 5.00m			

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BOREHOLE LOG

Borehole No.
2
1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED SHOALHAVEN CANCER CARE CENTRE
Location: CNR NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

Job No. 24682WH
Method: SPIRAL AUGER JK350
R.L. Surface: ≅ 31.3m
Date: 11-3-11
Datum: AHD
Logged/Checked by: M.L.T./ASH

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
DRY ON COMPLE -TION AND AFTER 2HRS						0			TOPSOIL: Silty sand, fine grained, dark brown, with root fibres.	D			
								SM	SILTY SAND: fine grained, brown with fine to medium grained sandstone gravel.	D	(MD)	-	RESIDUAL
					N > 29 10,14, 15/ 120mm END	1		-	SANDSTONE: fine to medium grained, orange brown and red brown, with M-H strength iron indurated bands and fine to coarse grained quartz gravel.	XW	EL	-	VERY LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
										DW	VL-L		
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

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Borehole No.
3
1/1

BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED SHOALHAVEN CANCER CARE CENTRE
Location: CNR NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

Job No. 24682WH **Method:** SPIRAL AUGER JK350 **R.L. Surface:** ≅ 30.5m
Date: 10-3-11 **Datum:** AHD
Logged/Checked by: M.L.T./ AJA

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLE-TION AND AFTER 24 HRS		0		TOPSOIL: Silty sand, fine grained, dark brown, with roots.	D			GRASS COVER
	N > 8 3,8/60mm REFUSAL		CL	SILTY SANDY CLAY: low plasticity, orange brown, fine grained sand, with fine to coarse grained ironstone gravel.	MC<PL	H	- 440 410	RESIDUAL
		1	-	SANDSTONE: fine to coarse grained, light grey and red brown.	DW	L	-	MODERATE 'TC' BIT RESISTANCE
		2		As above, but light grey, with fine to coarse grained quartz gravel.	XW	EL		VERY LOW RESISTANCE PROPOSED LOWER GROUND FLOOR LEVEL, RL 28.0m
		3		As above, but coarse grained, light grey and orange brown.	DW	M / H		MODERATE RESISTANCE / HIGH RESISTANCE
		4		END OF BOREHOLE AT 4.0m				'TC' BIT REFUSAL 50mm DIA.PVC STANDPIPE INSTALLED TO 4.0m DEPTH. SLOTTED BETWEEN 2.5m AND 4.0m DEPTH. GATIC COVER CONCRETED FLUSH WITH SURFACE
		5						
		6						
		7						

Samples: ES U50 DB DS

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BOREHOLE LOG

Borehole No.
4
1/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED SHOALHAVEN CANCER CARE CENTRE
Location: CNR NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

Job No. 24682WH **Method:** SPIRAL AUGER JK350 **R.L. Surface:** ≅ 29.9m
Date: 11-3-11 **Datum:** AHD
Logged/Checked by: M.L.T./AJH

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
DRY ON COMPLE-TION OF AUGER-ING			0			FILL: Silty sand, fine grained, dark brown, with roots and fine to coarse grained igneous gravel.	D			GRASS COVER
					SM	SILTY SAND: fine grained, brown.	D	(MD)	-	RESIDUAL
		N = 16 6,8,8			CL	SILTY SANDY CLAY: medium plasticity, orange brown, fine grained sand, with fine to medium grained ironstone gravel.	MC<PL	H	>600 >600 >600	
			1		-	SANDSTONE: fine to coarse grained, light grey, orange brown and red brown, with M-H strength iron indurated bands.	DW	M-H		MODERATE 'TC' BIT RESISTANCE WITH HIGH BANDS
			2					H		PROPOSED LOWER GROUND FLOOR LEVEL RL 28.0m
										HIGH RESISTANCE
						As above, but light grey, without iron indurated bands.	XW	EL		VERY LOW RESISTANCE
			3							
			4				DW	L		LOW RESISTANCE
						REFER TO CORED BOREHOLE LOG				
			5							
			6							
			7							

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JOB NO. 24682WH BH4 START CORING AT 4.30m
4.30m
5
CORE LOSS
0.13m
6
7
END BH4 7.46m

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CORED BOREHOLE LOG

Borehole No. 4
2/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED SHOALHAVEN CANCER CARE CENTRE
Location: CNR NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

Job No. 24682WH | **Core Size:** NMLC | **R.L. Surface:** ≅ 29.9m
Date: 11-3-11 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK350 | **Bearing:** - | **Logged/Checked by:** M.L.T./AJH

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		4		START CORING AT 4.30m						
FULL RETURN				SANDSTONE: fine to coarse grained, light grey, with dark brown bands, bedded at 0-5°.	SW	L				
						H				
		5		as above, but light grey.	SW-FR				- XWS,10°,5mm.t - XWS,10°,3mm.t - XWS,0°,20mm.t	
				CORE LOSS: 0.13m						
				SANDSTONE: fine to coarse grained, light grey, bedded at 0-5°.	SW-FR	H				
		6		As above, but orange brown, with light brown bands, bedded at 0-20°, with quartz gravel.	DW					
		7								
				END OF BOREHOLE AT 7.46m						
		8								
		9								
		10								

Point load strength index scale: EL VL L M H VH EH
Defect spacing scale: 500 300 100 50 30 10

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BOREHOLE LOG

Borehole No. **5**
1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED SHOALHAVEN CANCER CARE CENTRE
Location: CNR NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

Job No. 24682WH
Date: 10-3-11
Method: SPIRAL AUGER JK350
R.L. Surface: ≅ 29.0m
Datum: AHD
Logged/Checked by: M.L.T./AJH

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
DRY ON COMPLE -TION			0			FILL: Silty sand, fine grained, dark brown, with roots, a trace of fine to coarse grained sandstone gravel.	M			GRASS COVER
		N = 10 4,5,5			CL	As above, but brown, with concrete fragments. SILTY SANDY CLAY: low plasticity, orange brown and red brown, with a trace of fine grained ironstone gravel.	MC<PL	VSt- H	- 300 450 470	RESIDUAL
			1							
		SPT 17/100mm REFUSAL			-	SANDSTONE: fine to coarse grained, orange brown, light brown and red brown.	XW-DW	EL-VL	-	VERY LOW 'TC' BIT RESISTANCE
						END OF BOREHOLE AT 1.6m				
			2							
			3							
			4							
			5							
			6							
			7							

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Borehole No. 6
1/1

BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED SHOALHAVEN CANCER CARE CENTRE
Location: CNR NORTH STREET AND SCENIC DRIVE, NOWRA, NSW

Job No. 24682WH
Date: 11-3-11
Method: SPIRAL AUGER JK350
R.L. Surface: ≅ 27.6m
Datum: AHD
Logged/Checked by: M.L.T./AH

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLE -TION		0		TOPSOIL: Silty sand, fine grained, dark brown, with roots.	D			
	N = 22 10,11,11		SM	SILTY SAND: fine grained, brown, with fine to coarse grained cemented sand nodules.	D	MD	-	RESIDUAL
		1	-	SANDSTONE: fine to coarse grained, orange brown and red brown.	XW-DW	EL-VL		VERY LOW 'TC' BIT RESISTANCE
				as above, but with M-H strength iron indurated bands.	DW	L		LOW RESISTANCE WITH MODERATE BANDS
		2		As above, but grey and orange brown.				LOW RESISTANCE WITH VERY LOW AND MODERATE BANDS
		3						HIGH RESISTANCE
				END OF BOREHOLE AT 3.6m				'TC' BIT REFUSAL
		4						
		5						
		6						
		7						

Samples columns: ES, U50, DB, DS

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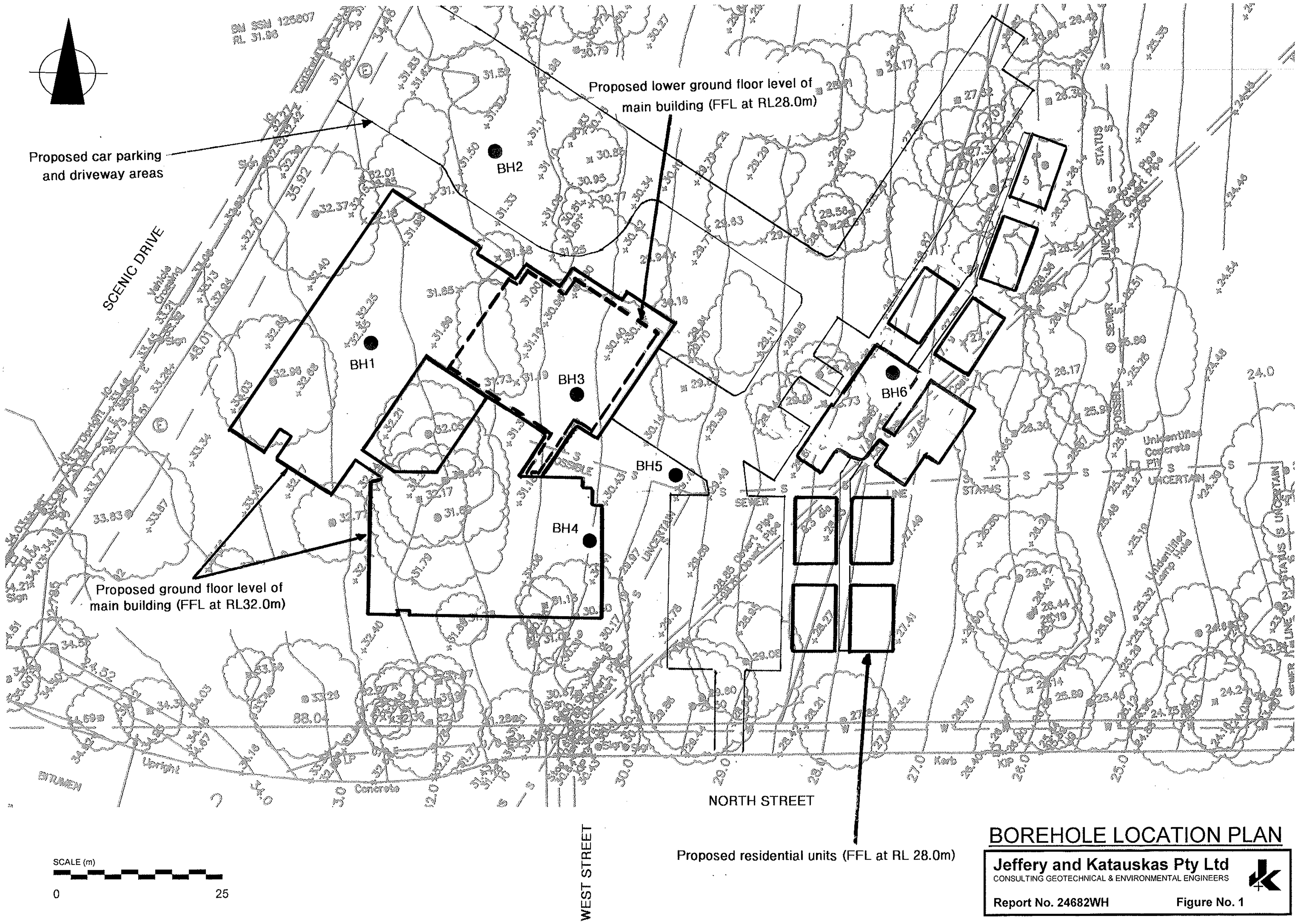

BOREHOLE LOCATION PLAN

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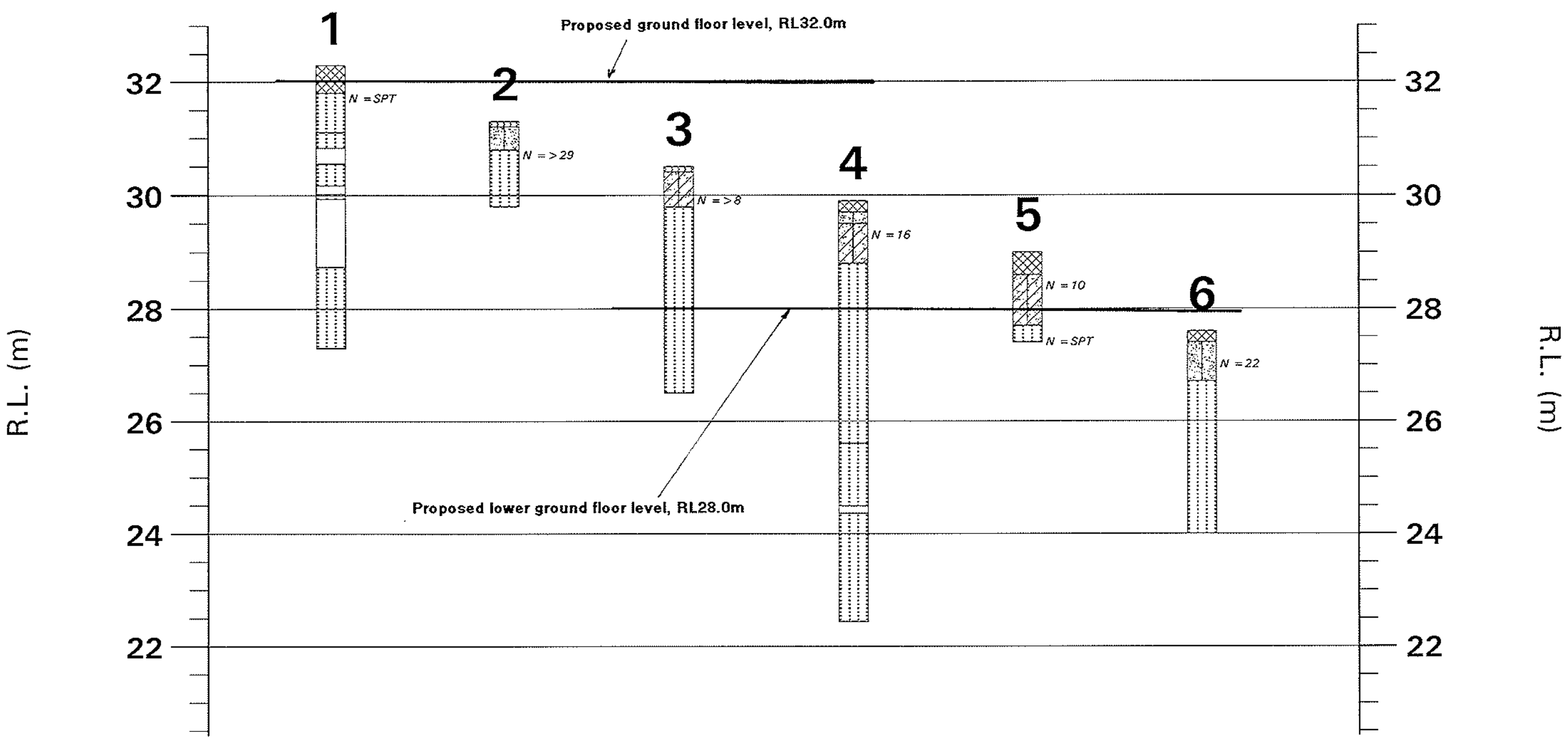
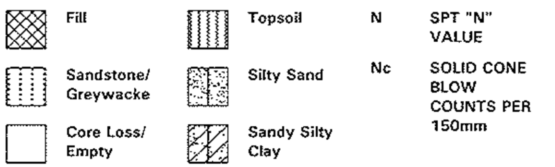
Report No. 24682WH **Figure No. 1**

GRAPHICAL BOREHOLE SUMMARY





NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 24682WH

Figure No.: 2

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REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, the SAA Site Investigation Code. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the grading of other particles present (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'Shale' is used to describe thinly bedded to laminated siltstone.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock fragments. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N>30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as "N_c" on the borehole logs, together with the number of blows per 150mm penetration.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The attached explanatory notes define the terms and symbols used in preparation of the logs.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than "straight line" variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg bricks, steel etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soil for Engineering Purposes'*. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg to a twenty storey building). If this happens, the company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer.

SITE INSPECTION

The company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CH)
- SILTY CLAY (CL, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML)
- PEAT AND ORGANIC SOILS

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)

(The 75 μm sieve size is about the smallest particle visible to naked eye)

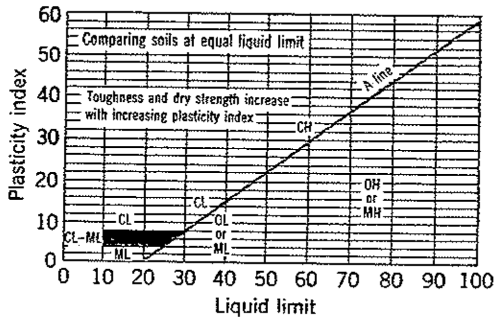
Major division	Subdivision	Fines	Field Identification Procedures	Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below "A" line, or *PI* less than 4 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7

Laboratory Classification Criteria (coarse-grained): Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% — *GW, GP, SW, SP*; More than 12% — *GM, GC, SM, SC*; 5% to 12% — *Borderline* cases requiring use of dual symbols

Identification Procedures on Fraction Smaller than 380 μm Sieve Size

Major division	Subdivision	Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	

Highly Organic Soils	Field Identification	Group Symbols	Typical Names
Highly Organic Soils	Readily identified by colour, odour, spongy feel and frequently by fibrous texture	*Pt*	Peat and other highly organic soils



Plasticity chart
for laboratory classification of fine grained soils

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. GW-GC, well graded gravel-sand mixture with clay fines).

2) Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

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LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
		Extent of borehole collapse shortly after drilling.
		Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY - runs freely through fingers.
	M	MOIST - does not run freely but no free water visible on soil surface.
	W	WET - free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT - Unconfined compressive strength less than 25kPa
	S	SOFT - Unconfined compressive strength 25-50kPa
	F	FIRM - Unconfined compressive strength 50-100kPa
	St	STIFF - Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF - Unconfined compressive strength 200-400kPa
	H	HARD - Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 — 0-4
	L	Loose 15-35 — 4-10
	MD	Medium Dense 35-65 — 10-30
	D	Dense 65-85 — 30-50
	VD	Very Dense >85 — >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T 60	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
		0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
		0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
		0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
		1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
		3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
		10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	

APPENDIX A

Envirolab Services Pty Ltd
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CERTIFICATE OF ANALYSIS **52978**

Client:
Environmental Investigation Services
PO Box 976
North Ryde BC
NSW 1670

Attention: Mark Tsang

Sample log in details:

Your Reference:	**24682WH, Nowra**		
No. of samples:	3 Soils		
Date samples received / completed instructions received	14/03/11	/	16/03/11

This report supersedes the previous report due to the changes in sample ID

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	22/03/11	/	1/04/11
Date of Preliminary Report:	Not issued		

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This document is issued in accordance with NATA's accreditation requirements.
Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

Nick Sarlamis
Inorganics Supervisor

Miscellaneous Inorg - soil Our Reference: Your Reference Depth Type of sample	UNITS ------------- ------------	52978-1 BH1 0.5-0.63 Soil	52978-2 BH2 0.5-0.95 Soil	52978-3 BH3 0.5-0.71 Soil
Date prepared	-	22/3/2011	22/3/2011	22/3/2011
Date analysed	-	22/3/2011	22/3/2011	22/3/2011
pH 1:5 soil:water	pH Units	6.1	5.9	5.5
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	<10	<10

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA 21st ED, 4500-H+.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 21st ED, 4110-B.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			22/3/2011	[NT]	[NT]	LCS-1	22/3/2011
Date analysed	-			22/3/2011	[NT]	[NT]	LCS-1	22/3/2011
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	102%
Chloride, Cl 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	92%
Sulphate, SO4 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	106%

Report Comments:

Chloride\Sulphate:PQL raised due to sample matrix.

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NA: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batched of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes and LCS: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable.