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REPORT

TO
BARBARY COAST INVESTMENTS

ON
PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR
PROPOSED MIXED USE DEVELOPMENT

AT
94 CARRINGTON ROAD AND 223 TO 227 BRONTE ROAD, WAVERLEY, NSW

14 May 2018
Ref: 29613ZRrpt Rev1

dlcs ISO 9001 certified
AS/NZS 4801 certified
ISO 14001 certified

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1 INTRODUCTION

This report presents the results of a limited scope geotechnical investigation for the proposed mixed use development at 94 Carrington Road and 223 to 227 Bronte Road, Waverley, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by David Whitten (Barbary Coast Investments Pty Ltd), by signed 'Acceptance of Proposal' form dated 20 April 2018. The commission was on the basis of our fee proposal (Ref. P42877R rev1 let) dated 20 April 2018.

We have been provided with the following information:

- Site survey plan (Ref. 160101_A, dated 11 February 2016) prepared by Geosurv Pty Ltd.
- Site survey plan (Ref. 20150283, dated 21 October 2015) prepared by Ballenden Surveyors.
- A Planning Study (VER 2.1, dated May 2018) prepared by H&E Architects.

Based on a review of the provided information, we understand that following demolition of existing buildings over the southern and western portions of the site, the proposed mixed use development will comprise:

- Over the northern portion of the site, maintaining the existing Robin Hood Hotel building. The existing building over the eastern portion of the site (No. 211 Bronte Road) will remain and will not be included in the development.
- Over the southern portion of the site, two multi-level buildings (the seven storey southern building and the six storey western building) constructed over one basement level. Proposed finished floor levels have not been provided and we have assumed that excavations to a maximum depth of 3m will be required to achieve the basement bulk excavation level. A driveway ramp access into the basement will extend down from the western (Carrington Road) frontage.
- Courtyards and laneways will be provided at ground level between the multi-level buildings.

Structural loads have not been supplied and we have assumed typical loadings for this type of development.

The purpose of this investigation was to obtain geotechnical information on subsurface conditions as a basis for preliminary comments and recommendations on geotechnical issues relevant to the proposed development including demolition, excavation, retention, groundwater, footings, on-grade floor slabs, drainage and the scope of further geotechnical input.

We note that we prepared a previous report (Ref. 29613ZRrpt) dated 31 August 2016 for a previous development proposal. This current report supersedes our previous report.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 21 July 2016 and was restricted by access constraints to the south-western corner of the site. The fieldwork comprised the auger drilling of two boreholes (BH1 and BH2) to respective depths of 1.23m and 1.46m, using our track mounted JK308 drilling rig. The boreholes were then extended by diamond core drilling using NMLC coring techniques (with water flush) to final depths of 9.15m and 9.59m below existing ground surface levels.

Prior to commencement of the fieldwork, the borehole locations were scanned for the presence of buried services by a specialist sub-contractor.

The borehole locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The surface reduced levels (RLs) shown on the attached borehole logs were estimated by interpolation between spot levels shown on the provided survey plan prepared by Geosurv. The survey plan forms the basis for Figure 2. The survey datum is Australian Height Datum (AHD).

The strength of the bedrock within the augered portion of the boreholes was assessed from observation of drilling resistance when using a tungsten carbide (‘TC’) bit and examination of the recovered rock cuttings. The strength of the bedrock within the cored portions of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with laboratory Point Load Strength Index testing.

Groundwater observations were made in the boreholes during auger drilling, and on completion of auger drilling and coring. We note that water is used as part of the coring process, and therefore water levels at the completion of coring may not have stabilised in the short time period after drilling. No longer term groundwater monitoring has been carried out.

For more details of the investigation procedures, reference should be made to the attached Report Explanation Notes.

The fieldwork for the investigation was carried out under the direction of our geotechnical engineer (Joel Dalberger), who was present full-time on site, and set out the test locations, directed the buried services scan, logged the encountered subsurface profile, and nominated in-situ testing and sampling. The borehole logs are attached, together with a glossary of logging terms and symbols used.

The recovered rock core was returned to the Soil Test Services Pty Ltd (STS) NATA registered laboratory where it was photographed and Point Load Strength Index tests completed. A summary of the Point Load Strength Index tests and estimated Unconfined Compressive Strengths are attached in STS Table A and plotted on the cored borehole logs. The core photographs are included opposite the relevant borehole log.

A contamination screen of site soils and groundwater was outside the agreed scope of the investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within gently undulating topography near the crest of a hillside that slopes down to the south at approximately 5˚.

The site has an approximately triangular shape (in plan) with western and eastern frontages onto Carrington Road and Bronte Road, respectively.

At the time of the fieldwork, the northern portion of the site was occupied by the one and two storey brick Robin Hood Hotel. The majority of the remaining southern portion of the site was occupied by one and two storey brick and rendered residential and commercial buildings. A dilapidated brick garage with gravel surfaced surrounds was situated over the south-western corner of the site. The eastern end of the gravel surfaced surrounds was lined by a dilapidated brick retaining wall (maximum 0.6m high), which supported a garden bed and concrete paved surface.

The neighbouring three story rendered Eastern Suburbs Legion Club (No. 211 Bronte Road) lined the central stepped (in plan) section of the eastern site boundary.

Neighbouring two storey brick residential and commercial buildings lined the southern site boundary.

Site surface levels were similar across the site boundaries.

Based on a cursory inspection from within the site and along the street frontages, unless otherwise noted above, the buildings and structures within and neighbouring the site appeared to be in good condition.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone. The boreholes disclosed a subsurface profile comprising a thin surficial layer of fill overlying weathered sandstone bedrock. No discernible groundwater was encountered over the depth of the investigation. For detailed subsurface conditions at the borehole locations, reference should be made to the attached borehole logs. A summary of the pertinent subsurface issues is presented below:

Fill

A 0.4m thick layer of silty sand fill with variable gravel content was respectively encountered from below the gravel surface (BH1) or from surface level (BH2). The fill was of insufficient thickness to carry out SPT testing and so no assessment of its state of compaction was possible.

Sandstone Bedrock

Sandstone bedrock was encountered below the fill in both boreholes. On first contact, the sandstone was assessed to be extremely to distinctly weathered and of extremely low to very low strength. In BH1, the sandstone improved to distinctly weathered and low to medium strength below 0.8m depth. In the cored portion of the borehole the sandstone was distinctly weathered and low to medium strength down to 6m depth. In BH2, the extremely to distinctly weathered sandstone extended down to 4.3m depth, where it then improved to distinctly weathered and medium strength.

Below depths of 6m (BH1) and about 5.6m (BH2), the bedrock typically comprised extremely weathered shale (occasionally sandstone) of extremely low strength, with occasional distinctly weathered low or medium strength bands of shale or sandstone. Below depths of 7.65m (BH1) and 7.4m (BH2), the sandstone was slightly weathered and medium (becoming high) strength to the borehole termination depths.

In the cored sections of the boreholes, the following defects were recorded:

- Occasional sub-horizontal bedding partings.

- A number of sub-horizontal extremely weathered seams and clay seams ranging between about 5mm and 70mm thickness.
- Occasional planar and undulating joints dipping at between approximately 20° and 90°.

The following core loss zones were also encountered:

- In BH1 at 2.99m depth: 0.33m thick, and
- In BH2 at 2.84m depth: 1.4m thick.

The core loss zones may be interpreted as representing clay seams, extremely weathered seams, fractured bands and/or crushed zones.

An indicative engineering classification of the bedrock (in accordance with Pells et al. 1998) has been carried out for the cored boreholes and is tabulated below:

Borehole	**Approx. Surface RL (m) AHD**	**Indicative Engineering Classification of Sandstone Bedrock Depths (m)**			
		Class V	**Class IV**	**Class II**	**Class I**
1	87.7	6.0 – 7.87[1]	0.4 – 6.0*		7.87 – 9.15
2	87.3	0.4* - 7.40[1]	-	7.40 – 9.59	-

***: Includes upper augered portion of borehole.**

[1]: Includes shale bands

Groundwater

Groundwater seepages or standing water levels were not encountered whilst auger drilling the boreholes, or the short time whilst setting up for core drilling.

In the cored boreholes, we note that water is introduced during core drilling and obscures groundwater measurements and groundwater levels may not have stabilised during the relative short period between borehole completion and measurement of water levels.

Water flush returns ranging from 100% to 75% were recorded whilst core drilling which indicates a relatively impermeable rock mass. On completion of coring, the standing water flush levels were recorded at depths of 1.3m (BH1) and 1.4m (BH2). No longer term groundwater monitoring has been carried out.

3.3 Laboratory Test Results

The point load test results indicated that the rock within the cored portion of the boreholes was of very low to high strength with estimated Unconfined Compressive Strengths (UCS) ranging between <1MPa and 26MPa. However, the majority of the test results indicated very low, low or medium strength.

4 COMMENTS AND RECOMMENDATIONS

The comments and recommendations provided below are preliminary and will need to be reviewed once architectural details for the proposed development have been finalised. This is of particular importance in relation to basement retention, as the upper portion of the bedrock in the two boreholes ranging from Class V to Class IV. Further geotechnical investigation will therefore be required following demolition in order to confirm the bedrock quality, the amount of ‘hard rock’ excavation, the extent of engineer designed shoring systems and allowable bearing pressures for footing design.

4.1 Demolition and Excavation

4.1.1 General

Excavation recommendations provided below should be completed by reference to the Safe Work Australia Code of Practice ‘Excavation Work’, dated July 2015.

The footprint of the proposed basement is indicated on the attached Figure 2 and excavations to a maximum depth of about 3m have been assumed.

We note that existing neighbouring buildings and paved surfaces line the southern, western and eastern sides of the proposed basement excavation. In addition, the existing hotel building that will remain, lines the northern side of the proposed basement excavation. To maintain the stability of the adjacent buildings, structures and paved surfaces, demolition and excavation will need to be completed with care. We recommend that demolition and excavation be completed using suitably experienced (and insured) contractors.

During demolition, we recommend that test pits be excavated to expose the footings of the neigbouring buildings to the south and east and the existing hotel building to the north. The test pits should be inspected by the geotechnical and structural engineers. Where footings are not founded on bedrock, then they may need to be underpinned, unless the basement retention system is designed to provide long term support. In addition, the width of existing footings can be assessed during the inspection; the existing footings may have an impact on the installation of shoring piles if they extend into the site.

The structural engineer should also assess the condition and stability of the adjacent hotel building to the north and the neighbouring buildings to the south and east, and assess the need for any temporary propping or bracing.

On the basis of the investigation results, following demolition, the proposed excavations will encounter the limited thickness soil profile and penetrate sandstone bedrock.

We expect the demolition and excavation to be completed using tracked excavators. A bucket attachment to the excavator will be required to excavate the soil profile and extremely weathered bedrock. In addition, rock breakers (attached to the excavator) may be required for demolition of existing concrete paved surfaces, footings and floor slabs.

Excavation of low or higher strength sandstone bedrock may be achieved using rock breakers, rock grinders and ripping attachments to the tracked excavator. We expect that small to medium sized rock breaker attachments will be used. If space permits, a medium to large sized dozer may also be used.

Care will be required to control ground vibrations associated with the use of rock breakers, such as the provision of rock saw cuts (see Section 4.1.2, below). Rock saws may also be used to create ‘smooth’ finishes on cut faces and aid in detailed excavation of footings, services trenches etc. Where rock breakers, rock saws and/or rock grinders are used, the resulting dust should be suppressed with water.

4.1.2 Potential Vibration and Ground Surface Movement Risks

The surficial sandy fill (which may be poorly compacted) encountered in the investigation may be of greater depth over portions of the site not currently investigated. Such fill we expect will extend across the site boundaries, and we therefore advise that sudden stop/start movements of tracked equipment should be avoided in order to reduce transmission of ground vibrations to the adjacent sections of buildings and structures within and neighbouring the site.

Care should be taken where rock breakers are used during demolition and for excavation of sandstone bedrock so that ground vibrations do not adversely affect nearby buildings, structures and paved surfaces within and neighbouring the site. If there is any cause for concern then demolition and/or excavation should cease and further geotechnical advice sought.

While the rock breakers are being used to excavate bedrock, continuous vibration monitoring of the neighbouring buildings and structures to the south and east will be required, to confirm that peak particle velocities (PPV) fall within acceptable limits. Subject to the results of the dilapidation reports (see Section 4.1.3, below), we would recommend that the PPV along the site boundaries do not exceed 5mm/sec during bedrock excavation using rock breakers. Consideration should also be given to completing similar vibration monitoring of the existing hotel building that will remain. Should higher vibrations be measured they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be acceptable depending on the vibration frequency. We note that this vibration limit will reduce the risk of vibration damage to the buildings and structures within and neighbouring the site. However, these vibrations may still result in discomfort to occupants of the neighbouring buildings and the adjacent hotel building. If excessive vibrations are confirmed, it will be necessary to use lower energy equipment such as smaller rock breakers and/or use rock saw cuts with the base of the slot maintained below the level at which the rock breaker is being used.

Where rock breakers are used, to reduce vibrations we recommend that the rock breaker be continually orientated towards the face, and be operated one at a time and in short bursts only to reduce amplification of vibrations.

4.1.3 Dilapidation Surveys

Prior to demolition and excavation commencing, detailed dilapidation reports should be compiled on the neighbouring buildings and structures to the south and east. In addition, Council may also require that dilapidation survey reports be completed on their assets lining the street frontages to the east and west, i.e. the paved footpaths, the roadways and kerbs and gutters. The property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the client in defending themselves from unfair damage claims.

4.1.4 Drainage

Seepage inflow may be expected within the excavations, particularly after periods of heavy rain within the sandy soil profile close to, or at, the contact with the underlying bedrock. In addition, concentrated flows along defects within the rock mass may also be encountered. In general, we expect the inflows to be ephemeral, of small volume and managed by conventional sump and pump techniques.

We recommend that all cut faces and retaining walls incorporate spoon drains or subsoil drains to intercept any potential seepage which could occur along the soil bedrock interface or open defects within the bedrock profile (if present). As noted above, seepage is expected to be of limited volume, and readily controlled by drainage to the stormwater system.

Inspection and monitoring of groundwater seepage during excavations is recommended, so that any unexpected conditions, which may be revealed can be incorporated into the drainage design.

4.2 Retention

4.2.1 Temporary Batters and Retention Methods

The proposed basement excavation will extend to the eastern, western and southern site boundaries and the southern side of the hotel building that will remain.

Due to the thickness of the soil profile and assuming buildings lining the excavation are founded on, or will be underpinned down to bedrock, sub-vertical temporary soil batters will be feasible. However, underpinning of the paved surfaces to the west and east would also likely be required. Underpins supporting a soil profile would need to be designed in accordance with the advice presented in Section 4.2.3, below.

However, over the south-western corner of the site, the bedrock encountered in BH2 was of poor quality, and full depth retention will be required. The retention system should comprise an engineer designed anchored contiguous bored pile wall installed prior to excavation commencing. Soldier pile walls with reinforced shotcrete infill panels may be considered. However, there would be an increased likelihood of collapse of the soil profile and weak rock between the soldier piles which could lead to damage to adjacent buildings and structures. We therefore do not recommend the use of soldier pile walls.

The contiguous pile wall construction will need to be of high quality with strict control of the vertical and lateral alignments of the piles. In this regard, soil loss through gaps between adjacent contiguous piles and/or seepage from gaps between the piles may occur will result in sub-soil erosion and removal of soil from outside the excavation. This could possibly cause settlement outside the excavation. Any such gaps between contiguous piles that are revealed as excavation proceeds must be immediately plugged with concrete.

We note that the piles must be socketed into bedrock and care will be required whilst drilling the piles into the bedrock so as not to cause excessive sand draw-down and possibly induce ground surface movements within surrounding properties. However, underpinning adjacent footings and paved surfaces would satisfactorily manage this issue.

We recommend that competent piling contactors be used. The piling contractor should be provided with a copy of this geotechnical report and any additional reports once the additional geotechnical investigations outlined in Section 4.6 below, have been completed, in order that appropriate piling rigs and equipment are brought to site.

To reduce excavation induced ground movements, the retention system will need to be progressively anchored as excavation proceeds. Excavations must not extend beyond the nominated anchor point until the anchors have been installed, stressed and tested.

The contiguous piles may be incorporated into the footing system, and based on BH2, they would need to be founded sufficient depth below bulk excavation level for stability and founding considerations. However, where more competent bedrock is encountered, shoring piles may be terminated in the excavation face and the toes would need to be temporarily restrained by rock bolts. The basement floor slabs may provide permanent support for the pile toes and up-turns or down-turns may need to be provided, otherwise the toe rock bolts would need to be permanent.

4.2.2 Sandstone Cut Face Stability

The proposed excavations will encounter sandstone bedrock. Class IV (or better) sandstone bedrock may be cut vertically, subject to geotechnical inspection. Geotechnical inspections should be completed by an experienced geotechnical engineer or engineering geologist at regular intervals of no more than 1.5m vertical excavation ‘lifts’.

The presence of potentially unstable wedges, clay seams and extremely weathered seams of sandstone and/or shale within the sandstone bedrock may adversely affect the stability of the cut faces and/or footings located close to the crests of cut faces. Such features may require shotcreting and rock bolting. Provision should be made in the contract documents (budget and programme) for such inspections and stabilisation measures.

For walls founded at the crest of excavation faces, lateral restraint may be provided by starter bars drilled and grouted to a depth of at least 0.5m into the sandstone bedrock. The starter bars should be installed at a downward angle into the rock face and be provided with a vertical cogged length. Where cross bedded units within the sandstone bedrock are identified during geotechnical inspections and slope down into the excavation, then the starter bars may have to be extended to stabilise the potentially unstable cross bedded units.

4.2.3 Retention Design Parameters

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

- For design of conventional walls that will be supported by the structure and any underpins supporting a soil profile, we recommend the use of a triangular lateral earth pressure distribution with an ‘at rest’ earth pressure coefficient (k_o) of 0.55 for the soil profile and Class V bedrock, assuming a horizontal backfill surface.
- For progressively anchored or propped walls, where minor movements may be tolerated (i.e. along the eastern and western street frontages, assuming no movement sensitive buried services), we recommend the use of a uniform rectangular earth pressure distribution of 6H kPa for the retained profile, where ‘H’ is the retained height in metres.
- For progressively anchored or propped walls, which support areas that are highly sensitive to lateral movement (i.e. the northern and southern sides of the basement), a uniform rectangular earth pressure distribution of 8H kPa for the retained profile should be adopted.
- Any surcharge affecting the walls (e.g. due to traffic loads, adjacent footings, construction loads, etc) should be allowed in the design using the above ‘at rest’ earth pressure coefficient.
- The piled walls and any underpins supporting a soil profile must be designed as permanently drained and PVC pipes should be installed at nominal 1.2m horizontal spacing just above the adjacent floor level and just above the bedrock surface. Holes will need to be drilled to allow installation of the pipes and/or use of gaps between contiguous piles. The end of the pipe penetrating the retained profile behind the wall must be wrapped in a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The pipes should discharge into the perimeter drainage system.
- Conventional retaining walls should be designed as drained and provision made for permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a

non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system.

- For piles embedded into the underlying bedrock below bulk excavation level, an allowable lateral toe resistance of 200kPa may be adopted for sockets embedded a minimum depth of 1m into Class V sandstone or shale bedrock. Increased allowable lateral toe resistances of 500kPa or 1000kPa may be adopted for sockets embedded a minimum depth of 1m into Class IV or Class III sandstone, respectively. This value assumes excavation is not carried out within the zone of influence of the wall toe and the rock does not contain unfavourable defects, etc. The upper 0.3m depth of rock below the adjacent excavation level should not be taken into account in the socket design to allow for disturbance and tolerance effects during excavation.
- Ground anchors, any rock bolts that may be required and starter bars providing lateral restraint for retaining walls at the crests of rock cut faces should be designed for an allowable bond strength of 200kPa assuming they are installed into Class IV (or better) sandstone bedrock.
- Rock bolts and dowels should be ‘nipped’ tight. All ground anchors should be proof tested to 1.3 times the working load under the direction of an experienced engineer independent of the anchor contractor. We recommend that only experienced contractors be considered for the rock bolt and anchor installation.
- Where rock bolts, ground anchors etc extend beyond the site boundaries, permission from the neighbouring property owners will be required. Permanent rock bolts or ground anchors will need to be designed with due regard for long term corrosion protection.

4.3 Footings

Sandstone (and possibly shale) bedrock of variable strength is expected to be exposed at bulk excavation level. We envisage that the footing system will comprise a combination of pad and strip footings founded below bulk excavation and perimeter shoring piles founded in the cut face and/or below bulk excavation levels

Based on the indicative engineering classification of the bedrock (in accordance with Pells et al. 1998) provided in Section 3.2 above, we recommend the following allowable bearing pressures be adopted for design of footings:

• Class V shale:	700kPa
• Class V sandstone:	1,000kPa
• Class IV sandstone:	1,500kPa
• Class III sandstone:	3,5000kPa

- Class II (or better) sandstone: 6,000kPa

The load carrying portions of any pile rock sockets may be designed using values of 10% or 5% of the above allowable bearing pressures in compression and tension, respectively. The design of rock sockets must take into account the presence of localised excavations for service trenches, lift pits etc.

For piles founded on Class IV (or better) sandstone bedrock within the excavation depth, an allowable end bearing pressure of 600kPa may be adopted. However, geotechnical inspection of the cut face below the pile toe would need to be carried out to check for the presence of any adversely orientated joints, extremely weathered seams etc which may compromise the stability of the pile bases.

We recommend that footing bases be inspected by a geotechnical engineer to confirm our assumptions regarding sandstone quality.

However, we note that the adoption of the allowable bearing pressures for Class III (or better) sandstone will require the completion of additional cored boreholes. Further, a comprehensive spoon testing regime for at least 50% of pad or strip footings designed for an allowable bearing pressure of 6,000kPa is likely to be required, depending on the number of additional cored boreholes completed. We recommend that a further cored borehole investigation be undertaken to confirm assumptions regarding sandstone quality across the site, as outlined in Section 4.6, below.

All footings should be excavated/bored, inspected and poured with minimal delay. All footings should be free from all loose or softened materials prior to pouring. If water ponds in the base of the footing excavations or bored pile drill holes then it should be pumped dry and then re-excavated/over-drilled to remove all loose and softened materials prior to pouring. If a delay in pouring high level footings is anticipated consideration should be given to provision of a blinding layer of concrete to protect the base of the footing excavations in any Class V sandstone or shale bedrock.

4.4 Subgrade Preparation and Engineered Fill

4.4.1 Subgrade Preparation

Prior to the placement of on-grade floor slabs over any areas of poor quality (Class V) bedrock, the following preparation should be undertaken following completion of bulk excavations:

- Proof-roll the exposed subgrade with a minimum of eight passes of a five tonne minimum deadweight smooth drum vibratory roller. The purpose of proof rolling is to identify any soft or unstable areas. All proof-rolling should be conducted under the direction of an experienced earthworks superintendent or geotechnical engineer. Care should be taken when proof rolling under vibration if movement sensitive structures are located nearby. If transmitted vibrations are considered excessive, proof rolling should be completed using the static (no vibration) mode.
- All soft or heaving areas identified during proof-rolling should be excavated to a sound base and reinstated with engineered fill as described below.
- Any areas of extremely weathered (Class V) bedrock subgrade may be found to be unstable if proper site drainage is not implemented during construction. It is therefore important to provide good drainage in order to promote run-off and reduce ponding. Earthworks platforms should be graded to maintain cross-falls during construction. If the Class V is exposed to periods of rainfall, softening may result and site trafficability will be poor. If softening occurs, the subgrade should be over-excavated to below the depth of moisture softening. The material removed should be replaced with engineered fill. Trafficability may be improved by the use of a sacrificial surface layer of demolition rubble.

4.4.2 Engineered Fill

Engineered fill required to treat any poor subgrade areas should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm. We expect the soils and weathered bedrock sourced from the excavations may be used as engineered fill. Engineered fill comprising clayey materials (including weathered sandstone bedrock) should be placed in layers of maximum 100mm loose thickness and compacted with the above mentioned roller to at least 98% of Standard Maximum Dry Density (SMDD).

Backfill to any conventional retaining walls should also comprise engineered fill. The well graded granular materials such as crushed sandstone and demolition rubble would be suitable for this purpose. This granular fill should be free of deleterious substances and should have a maximum particle size of 40mm. Such fill should be compacted in horizontal layers using a hand held plate compactor as outlined above. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

Density tests should be carried out the frequency outlined in AS3798. At least Level 2 testing of backfill should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

Single sized granular material (or ‘no fines’ gravel) may be used as backfill to retaining walls and this would also act as the drainage behind the wall and would only require nominal compaction (with no compaction testing). The drainage material should be wrapped in a non woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.

4.5 Basement Floor Slab and Drainage

The basement floor slab should be designed as drained and an underfloor drainage layer will be required. The drainage layer should connect to the perimeter drainage system and the water discharged to the stormwater system. Discharging of the basement drainage will require a sump and pump system although in our experience permission from Council will be required to discharge into the stormwater drainage; further advice from Council will be required in this regard.

The basement slab should be designed to be separated from all walls, columns, footings, etc to permit relative movements (i.e. ‘floating’). The concrete floor slab should be provided with effective shear connection of joints by using dowels or keys. Additional dowels may be required at the interface between Class IV (or better) bedrock and Class V bedrock subgrade areas.

The basement underfloor drainage should comprise a continuous layer of durable, single sized, washed gravel (e.g. ‘blue metal’). A non-woven geofabric such as Bidim A34 will need to be provided at the interface between the drainage layer and any sections of poor quality subgrade to act as a filter against sub-soil erosion. The thickness of the drainage layer will need to be a minimum of 100mm.

4.6 Additional Geotechnical Investigation

We recommend that following demolition an additional geotechnical investigation be undertaken to include at least four cored boreholes to be drilled across the remainder of the proposed basement where access was currently not feasible. The boreholes are required so as to confirm bedrock quality and the extent of the poor quality bedrock encountered in BH2 quality. The boreholes will assist in confirming excavation conditions, retention methods and optimisation of allowable bearing pressures.

It should be appreciated that the completion of the geotechnical investigations following demolition will cause a delay to the project as detailed design will not be able to be completed until after the investigation and an additional report is prepared.

4.7 Further Geotechnical Input

Provided below is a summary of further geotechnical input outlined in the preceding sections of this report:

- Further geotechnical investigation including additional cored boreholes on completion of demolition.
- Inspection of test pits to expose adjacent footings.
- Dilapidation reports of adjoining buildings and structures.
- Monitoring of seepage into the excavations.
- Quantitative vibration monitoring.
- Inspection of the shallow and piled footing bases.
- Direction of proof rolling poor subgrade areas
- Density testing of engineered fill.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between and below the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1
1 / 3

BOREHOLE LOG

Client: BARBARY COAST INVESTMENTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 94 CARRINGTON ROAD, WAVERLEY, NSW

Job No.: 29613ZR **Method:** SPIRAL AUGER **R.L. Surface:** ~87.7 m
Date: 21/7/16 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** J.D./P.R.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING				■				▨		FILL: Gravel, coarse grained, angular to sub angular igneous, grey.	M			
				■	N > 1 5,1/ 10mm REFUSAL			░	-	FILL: Silty sand, fine to medium grained, dark grey, trace of fine to medium grained igneous gravel and slag.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
AT 2PM ▼				■		87	1	░		SANDSTONE: fine to medium grained, light grey and orange brown.	DW	L - M		LOW TO MODERATE RESISTANCE
						86 85 84 83 82 81	2 3 4 5 6			REFER TO CORED BOREHOLE LOG				

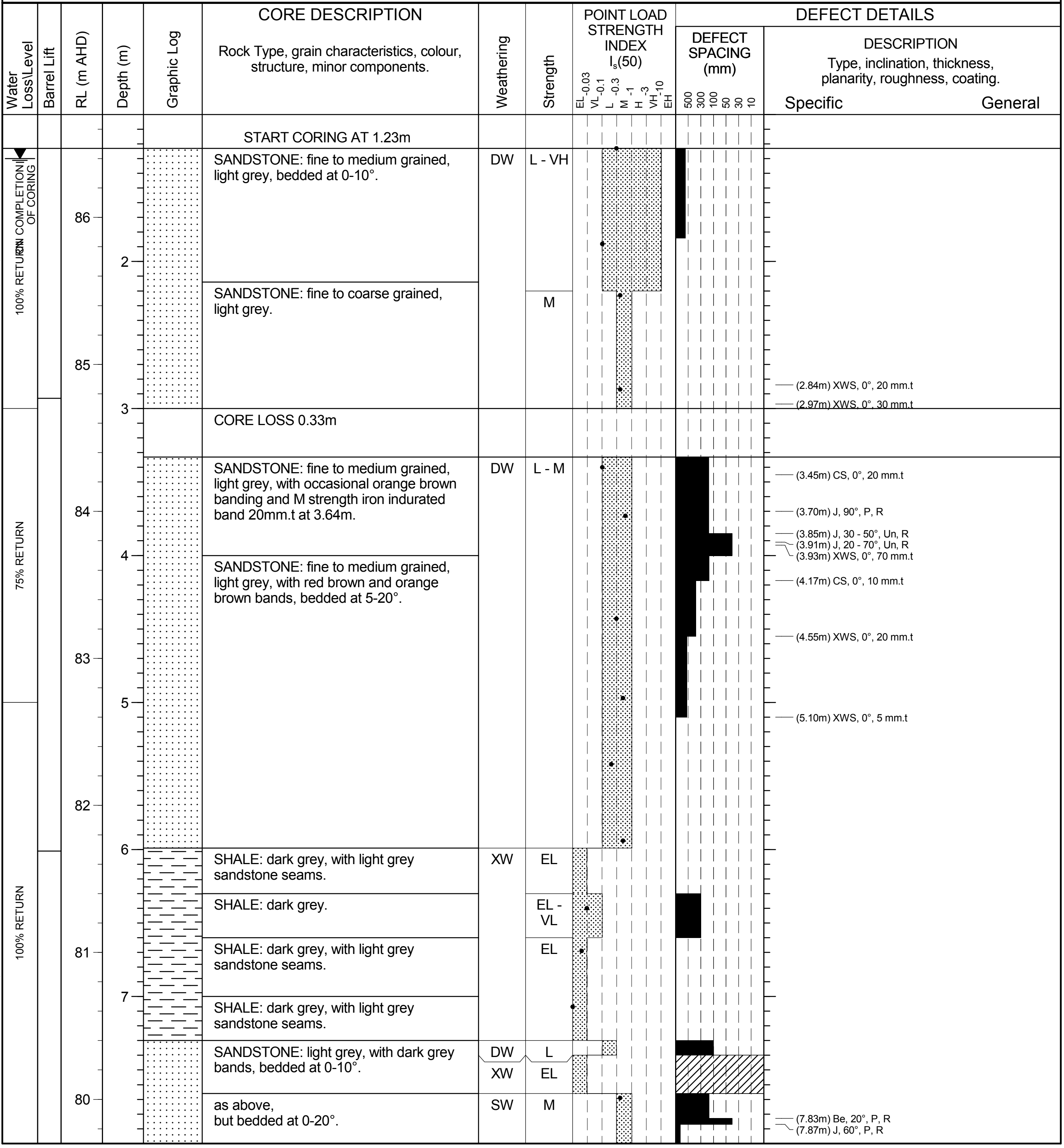
JK_LIB_CURRENT - V8.00.GLB Log J & K AUGERHOLE - MASTER 29613ZR WAVERLEY.GPJ <<DrawingFile>> 30/08/2016 16:13 Produced by gINT Professional, Developed by Datgel

CORED BOREHOLE LOG

Borehole No.
1
2 / 3

Client: BARBARY COAST INVESTMENTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 94 CARRINGTON ROAD, WAVERLEY, NSW

Job No.: 29613ZR	**Core Size:** NMLC	**R.L. Surface:** ~87.7 m
Date: 21/7/16	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** J.D./P.R.



Water Loss\Level	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
	START CORING AT 1.23m				
ON COMPLETION OF CORING 100% RETURN	SANDSTONE: fine to medium grained, light grey, bedded at 0-10°.	DW	L - VH		
	SANDSTONE: fine to coarse grained, light grey.		M	(2.84m) XWS, 0°, 20 mm.t (2.97m) XWS, 0°, 30 mm.t	
75% RETURN	CORE LOSS 0.33m				
	SANDSTONE: fine to medium grained, light grey, with occasional orange brown banding and M strength iron indurated band 20mm.t at 3.64m.	DW	L - M	(3.45m) CS, 0°, 20 mm.t (3.70m) J, 90°, P, R (3.85m) J, 30 - 50°, Un, R (3.91m) J, 20 - 70°, Un, R (3.93m) XWS, 0°, 70 mm.t	
100% RETURN	SANDSTONE: fine to medium grained, light grey, with red brown and orange brown bands, bedded at 5-20°.			(4.17m) CS, 0°, 10 mm.t (4.55m) XWS, 0°, 20 mm.t (5.10m) XWS, 0°, 5 mm.t	
	SHALE: dark grey, with light grey sandstone seams.	XW	EL		
	SHALE: dark grey.		EL - VL		
	SHALE: dark grey, with light grey sandstone seams.		EL		
	SHALE: dark grey, with light grey sandstone seams.				
	SANDSTONE: light grey, with dark grey bands, bedded at 0-10°.	DW XW	L EL		
	as above, but bedded at 0-20°.	SW	M	(7.83m) Be, 20°, P, R (7.87m) J, 60°, P, R	

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1
3 / 3

CORED BOREHOLE LOG

Client: BARBARY COAST INVESTMENTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 94 CARRINGTON ROAD, WAVERLEY, NSW

Job No.: 29613ZR
Core Size: NMLC
R.L. Surface: ~87.7 m
Date: 21/7/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: J.D./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH	DEFECT DETAILS DEFECT SPACING (mm) 500, 300, 100, 50, 30, 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
100% RETURN		79	9		SANDSTONE: light grey, with dark grey bands, bedded at 0-20°.	SW	H			(8.69m) CS, 0 - 10°, 5 mm.t	
		78 77 76 75 74 73	10 11 12 13 14		END OF BOREHOLE AT 9.15 m						

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JK Geotechnics
Job No. 29613ZR BH01 Start coring at 1.23m
1
1.23m
2
3
CORE LOSS 0.33m thick
4
5
6
7
8
9
EOH AT 9.15m

BOREHOLE LOG

Borehole No.
2
1 / 3

Client: BARBARY COAST INVESTMENTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 94 CARRINGTON ROAD, WAVERLEY, NSW

Job No.: 29613ZR
Method: SPIRAL AUGER
R.L. Surface: ~87.3 m
Date: 21/7/16
Datum: AHD
Plant Type: JK308
Logged/Checked By: J.D./P.R.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING				■		87		▨		FILL: Silty sand, fine to medium grained, dark grey, trace of fine to medium grained gravel.	M			GRAVEL COVER
ON COMPLETION OF CORING ▼				■		86	1	░	-	SANDSTONE: fine to coarse grained, light grey.	XW - DW	EL - VL		VERY LOW 'TC' BIT RESISTANCE
						85 84 83 82 81	2 3 4 5 6			REFER TO CORED BOREHOLE LOG				

JK Geotechnics

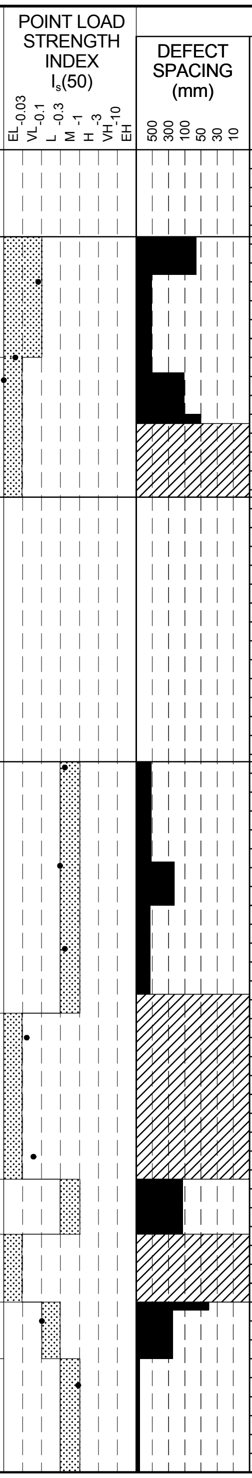
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2
2 / 3

CORED BOREHOLE LOG

Client: BARBARY COAST INVESTMENTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 94 CARRINGTON ROAD, WAVERLEY, NSW

Job No.: 29613ZR
Core Size: NMLC
R.L. Surface: ~87.3 m
Date: 21/7/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: J.D./P.R.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		86		START CORING AT 1.46m				
80% RETURN			2	SANDSTONE: fine to coarse grained, light grey.	XW - DW	EL - VL	(1.50m) J, 70 - 90°, Un, R (1.60m) CS, 0°, 30 mm.t (1.66m) J, 80°, P, R	
		85		as above, but fine to coarse grained.	XW	EL	(2.18m) XWS, 0°, 10 mm.t (2.27m) J, 45°, P, R (2.40m) J, 80 - 90°, Un, R	
		84	3 4	CORE LOSS 1.40m				
		83 82	5	SANDSTONE: fine to coarse grained, light grey, with orange brown bands, bedded at 0-25°.	DW - SW	M	(4.77m) Be, 0°, P, R, IS (5.00m) J, 70 - 80°, Un, R, IS	
		81	6	SHALE: dark grey, with light grey bands and thin VL ironstone bands, bedded at 0-5°.	XW	EL		
				SANDSTONE: fine to medium grained, light grey.	DW	M	(6.59m) Be, 0 - 5°, 5 mm.t, Un, R, Clay (6.70m) CS, 0°, 5 mm.t	
			7	SHALE: light grey.	XW	EL		
		80		SHALE: dark grey, with light grey sandstone bands up to 20mm thick.	DW	L	(7.14m) XWS, 5°, 10 mm.t (7.39m) XWS, 0°, 15 mm.t	
				SANDSTONE: fine to medium grained, light grey, with dark grey bands. bedded at 10-25°.	SW	M		

Borehole No.
2
3 / 3

CORED BOREHOLE LOG

Client: BARBARY COAST INVESTMENTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 94 CARRINGTON ROAD, WAVERLEY, NSW

Job No.: 29613ZR **Core Size:** NMLC **R.L. Surface:** ~87.3 m
Date: 21/7/16 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** J.D./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL -0.03 VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
80% RETURN		79 78	9		SANDSTONE: fine to medium grained, light grey, with dark grey bands. bedded at 0-25°.	SW	H				
		77 76 75 74 73	10 11 12 13 14		END OF BOREHOLE AT 9.59 m						

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JK Geotechnics
Job No. 29613ZR
BH02
Start coring at 1.46m
1
1.46m
2
CORE LOSS
3
CORE LOSS 1.40m
4
5
6
7
8
9
EOH AT 9.59m

SOURCE: http://www.whereis.com/

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

This plan should be read in conjunction with the JK Geotechnics report.

Title:

SITE LOCATION PLAN

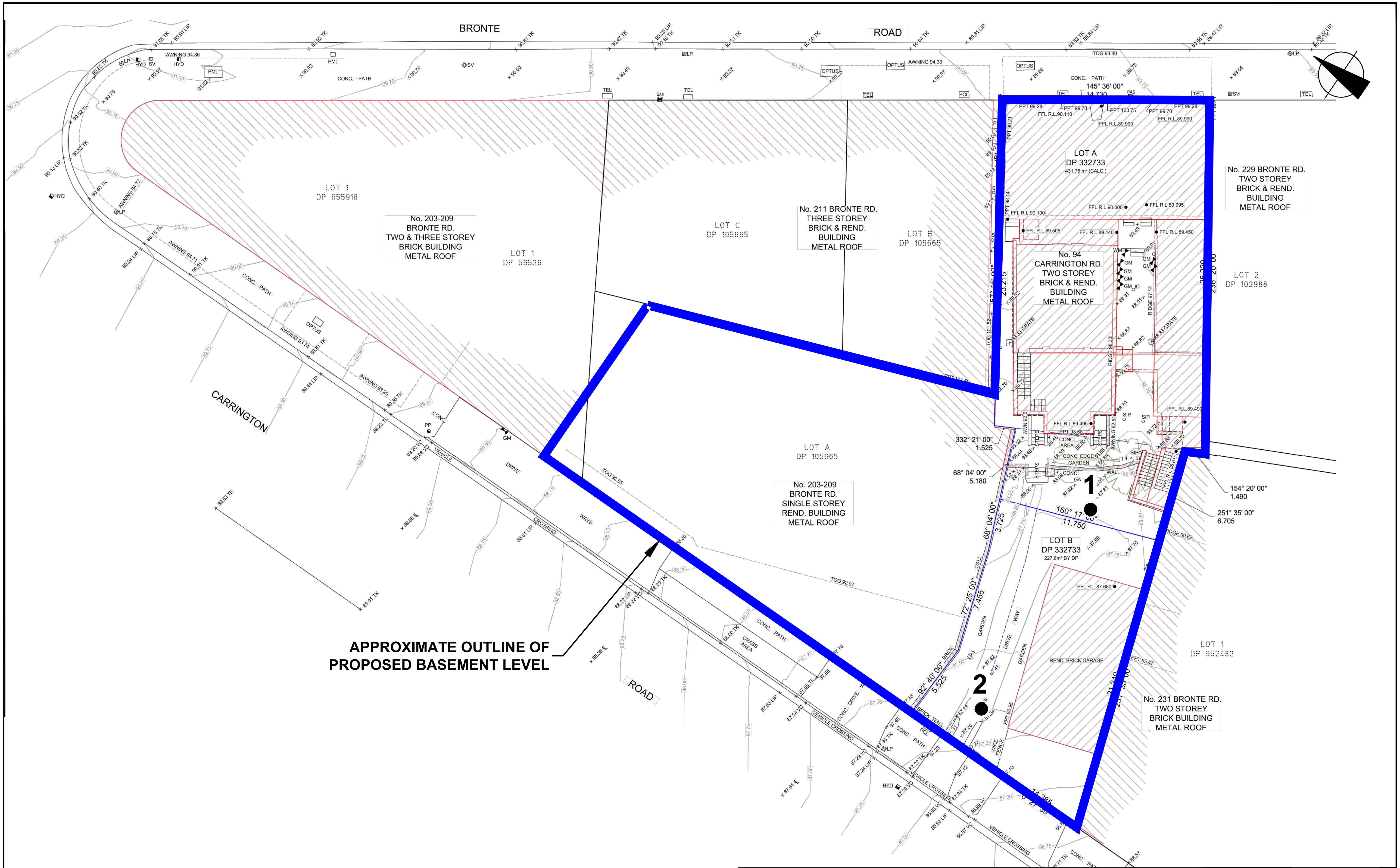
Project: PROPOSED MIXED USE DEVELOPMENT
94 CARRINGTON RD & 223-227 BRONTE RD, WAVERLEY, NSW

Report No: 29613ZR

Figure No: 1

JK Geotechnics

PLOT DATE: 14/05/2018 4:38:46 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\29000'S\29613ZR WAVERLEY\CAD\29613ZR.DWG



This plan should be read in conjunction with the JK Geotechnics report.

Title:
BOREHOLE LOCATION PLAN

Project: PROPOSED MIXED USE DEVELOPMENT
94 CARRINGTON RD & 223-227 BRONTE RD, WAVERLEY, NSW

Report No:	Figure No:
29613ZR	2

JK Geotechnics

PLOT DATE: 14/05/2018 4:38:30 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\29000'S\29613ZR WAVERLEY\CAD\29613ZR.DWG

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

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 (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 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 subsurface 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 or where only a limited investigation has been completed or where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer.

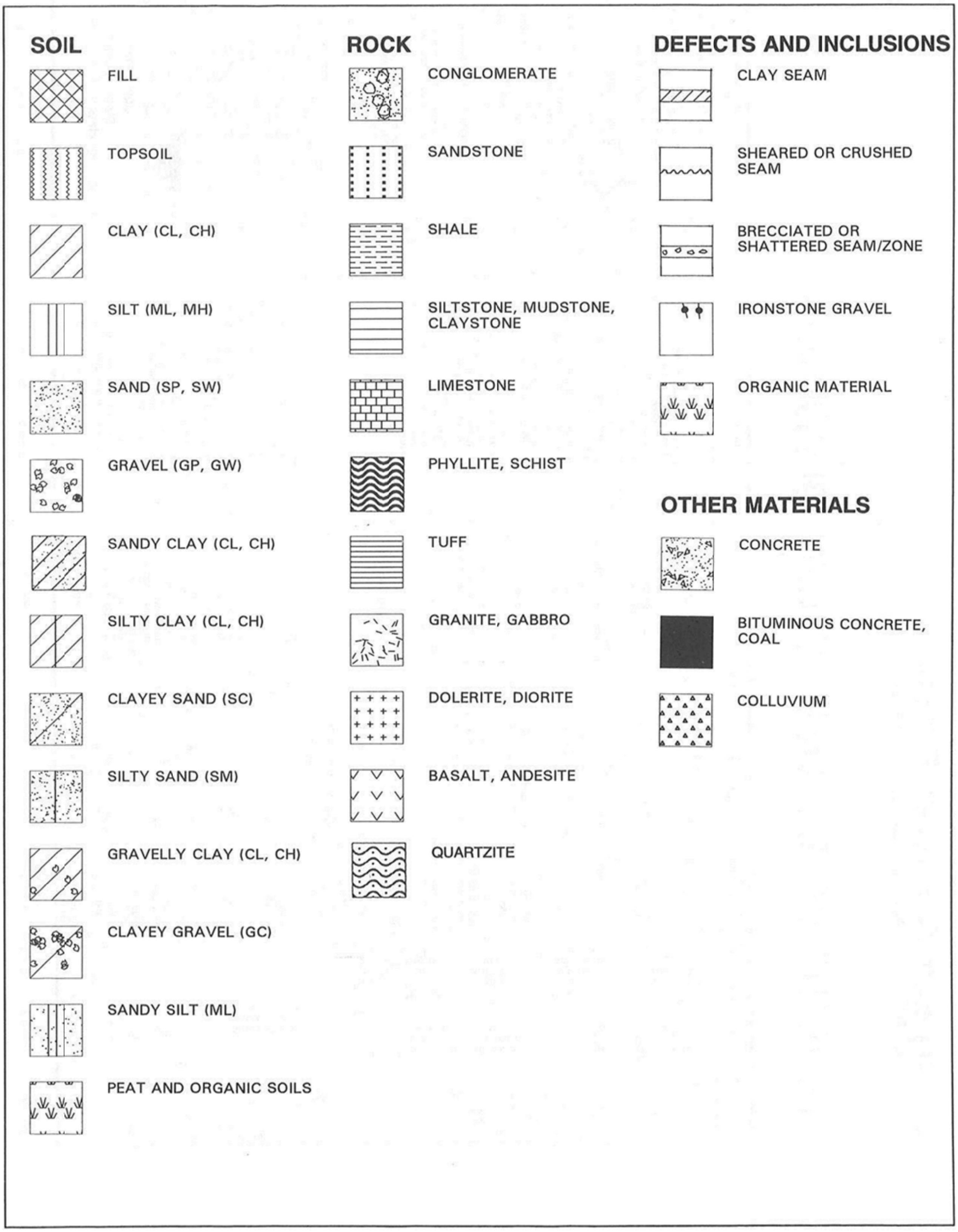
SITE INSPECTION

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

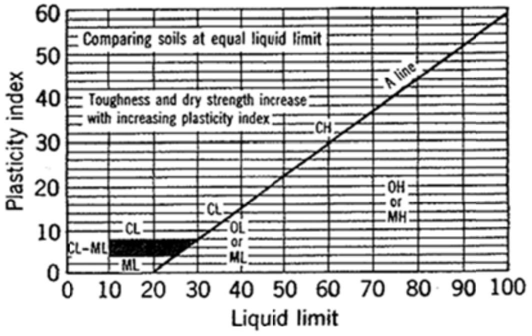
DEFECTS AND INCLUSIONS

- 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)						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[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	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 For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics 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*)	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	$C_U = \frac{D_{60}}{D_{10}}$ 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			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			$C_U = \frac{D_{60}}{D_{10}}$ 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
	Identification Procedures on Fraction Smaller than 380 μm Sieve Size									
			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)					
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 For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)		 Plasticity chart for laboratory classification of fine grained soils
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity			
	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			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils			

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. 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.

LOG SYMBOLS

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

LOG SYMBOLS continued

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	