REPORT

TO
ABM BUILT

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED MIXED USE DEVELOPMENT

AT
62 – 70 KING GEORGES ROAD &
280 – 292 LAKEMBA STREET, WILEY PARK NSW

22 June 2017
Ref: 30501Srpt

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TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed mixed use development at 62 to 70 King Georges Road and 280 to 292 Lakemba Street, Wiley Park NSW. The investigation was commissioned by Abdallah Saboune of ABM Built, transmitted by email, dated 18 May 2017. The commission was on the basis of our proposal, Ref: P44714S, dated 5 April 2017.

Architectural drawings by Marchese Partners are preliminary at this stage and referenced 15063 No DA 0.00, DA 1.01 to 1.07 and DA 2.01 and 2.02, all Rev A and dated 20 April 2017. Based on the supplied drawings, we understand that following demolition of existing structures on site, the development comprises an 8 storey residential and commercial building with 3 levels of basement car parking.

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

We were also commissioned to carry out a Stage 1 Environmental Site Assessment. This work was carried out by Environmental Investigation Services (EIS) in conjunction with the geotechnical investigations. Reference should be made to the EIS report, (Ref: E30501KG) for further details.

2 INVESTIGATION PROCEDURE

Prior to commencement of the fieldwork a 'Safe Work Method Statement' was prepared, a 'Dial Before You Dig' (DBYD) search was undertaken and the borehole locations were electromagnetically scanned by a specialist sub-contractor for buried services.

The fieldwork was carried out between 23 and 24 May 2017 and comprised the drilling of four boreholes. These boreholes were drilled using spiral auger techniques and a Tungsten Carbide 'TC' bit to depths ranging from 2.61m to 6.68m and were then extended to total depths ranging from 11.67m to 12.5m by diamond coring techniques using an NMLC core barrel with water flush.

The borehole locations, are shown on the attached Figure 2, and these were set out by taped measurements from existing surface features shown on the supplied survey plans by Higgins Surveyors (Ref: 10310, dated 17 May 2017) . The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels and contours shown on the survey plans. The datum of the levels is Australian Height datum (AHD).

The strengths of the subsurface soils were assessed from Standard Penetration Test (SPT) ‘N’ values, augmented by hand penetrometer test results on cohesive samples returned by the SPT split tube sampler. Within the augered portions of the boreholes, the strength of the underlying shale was assessed by observation of the drilling resistance of the Tungsten Carbide (TC) bit attached to the augers, together with examination of the recovered rock chips and subsequent correlation with laboratory moisture content test results.

Where the bedrock was core drilled, the recovered core was returned to our NATA registered laboratory (Soil Test Services) for photographing and Point Load Strength Index (Is_{50}) testing. Using established correlations the Unconfined Compressive Strength (UCS) of the bedrock was then inferred from the Is_{50} results. The Point Load Strength index test results are summarised in the attached STS Table B and are plotted on the cored borehole logs. Colour photographs of the rock core are provided with the borehole logs.

Groundwater observations were made both during drilling and after completion of the boreholes. Slotted PVC standpipes were installed in BH1, BH3 and BH4 to allow further groundwater level monitoring during the fieldwork period. Water levels were also obtained on 13 June 2017 which is approximately 3 weeks after drilling was completed. No long term monitoring of the groundwater levels has been carried out.

The fieldwork was completed in the full time presence of our geotechnical engineer who set out the boreholes, nominated sampling and testing locations and prepared logs of the strata encountered. The borehole logs, which include field test results and groundwater observations, are attached to the report together with colour photographs of the rock core and our Report Explanation Notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories to determine moisture contents, point load strength index, pH, sulphate content, chloride content and resistivity. The results of the testing are summarised in the STS Tables A and B and Envirolab report No. 167921. Samples were also collected from the boreholes for testing as part of the environmental site assessment by EIS.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within relatively flat topography on a slight north-west facing slope of 2° to 3°. The site encompasses several properties and is approximately 87m long and 67m wide. The site is bound to the north-west by Lakemba Street and to the south-west by King Georges Road.

At the time of fieldwork, the site was occupied by several single storey brick and fibro structures comprising houses and commercial buildings. All building across the site were in a relatively fair condition based on an external cursory inspection. The existing buildings were surrounded by mainly grass and concrete covered surfaces. 68 King Georges Road was surfaced with asphaltic concrete, and was used as a temporary carpark. 282 Lakemba Road was also partially surfaced with asphaltic concrete which was in a poor condition.

To the north-east of the site was a two storey brick apartment building, set back approximately 4m from the common boundary, which appeared in good condition based on a cursory inspection. To the south-east of the site was a 4 to 5 storey brick and concrete apartment building appearing to be in a good condition. The apartment building was set back approximately 3m from the common boundary and appeared to have basement level car parking. The Bankstown Railway line is located approximately 50m to the south of the site.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicated the site is underlain by Ashfield and Bringelly Shale of the Wianamatta Group.

Generally, the boreholes disclosed shallow fill overlying residual silty clay with weathered shale and sandstone bedrock at moderate depths. Reference should be made to the attached borehole logs for details at each specific location. Further comments on the subsurface conditions encountered are provided below.

Fill

Fill comprising gravelly sand was encountered in each borehole and extended to depths ranging from 0.3m to 0.4m below existing surface levels. The fill was covered by concrete and asphaltic concrete pavements.

Residual Silty Clay

Residual silty clay of medium to high plasticity was encountered below the fill in each borehole and was assessed to be of firm to hard strength. Inclusions of fine to coarse grained ironstone were present within the residual soils

Weathered Bedrock

Weathered bedrock was encountered in each borehole at depths ranging from 0.7m to 1.8m and extended to the borehole termination depths.

Sandstone bedrock was initially encountered in BH1 and BH4 underlying the residual clays and extended to depths between 5.4m and 5.8m below existing surface levels. Both BH1 and BH4 encountered an interbedded shale and sandstone layer underlying the sandstone, with BH1 transitioning to shale bedrock at a depth of 8.5m which extended to the termination depths. Shale bedrock was encountered underlying the residual clays in BH2 and BH3 and extended to the termination depths.

The bedrock profile was generally distinctly weathered and of medium to high strength in BH1 and BH4 at the higher end of site improving to slightly weathered and fresh with depth. At the lower end of site the bedrock in BH2 and BH3 was initially of extremely weathered and of extremely low strength, increasing with depth to high strength by its termination depths. Defects within the cored portions of the shale included inclined joints, bedding partings, clay seams and extremely weathered seams.

The difference in elevation between the two ends of the site is not sufficient to directly account for the difference in geology between the boreholes there must be some depositional or structural geological anomaly to account for the difference.

Groundwater

Generally the boreholes were ‘dry’ on completion of auger drilling. In cored holes, it is difficult to measure a true groundwater once coring has commenced as water is introduced during the coring process and obscures the true groundwater levels. Standpipes were installed in boreholes BH1, BH3 and BH4 to between 11.6m and 12.2m depth. Groundwater measurements within the standpipes were recorded over the course of the fieldwork and again on 13 June 2017. A summary of the results are presented in the table below and are also shown on the borehole logs.

Groundwater Depth	After Augering (Reduced level)	On Completion (Reduced level)	Groundwater Depth After 1 day (Reduced level)	Groundwater Depth After 3 Weeks (Reduced level)
BH1	Dry to 2.6m (RL39.9m)	-	1.0m (RL41.5m)	2.5m (RL40.0m)
BH2	Dry to 6.7m (RL 33.0m)	-	-	-
BH3	Dry to 6.6m (RL 32.6m)	0.9m (RL 38.4m)	-	1.2m (RL38.1m)
BH4	Dry to 2.7m (RL39.6m)	2.2m (RL 40.1m)	-	4.0m (RL38.3m)

The above levels seem to be anomalous, particularly the difference in depth/level between BH1 and BH4 which have similar ground levels and are only 35m apart. The shallower recorded levels are therefore potentially due to seepage in the unsaturated zone above the water table and indeed it is possible that the deeper level in BH4 is similarly affected. This simple method of establishing groundwater levels which has been used does suffer from this type of problem from time to time and the issue can only be resolved by installation of sophisticated instruments that measure water pressure, rather than collecting flows into an open hole.

3.3 Laboratory Test Results

The results of the moisture content tests carried out on recovered rock cuttings correlated well with our field assessment of bedrock strength. The point load strength index test results also correlated well with the field logging strength assessments, showing the bedrock to have an Unconfined Compressive Strength of between 8MPa and 124MPa which are at the high end of the strength range associated with these formations. Reference should be made to the attached Table B for specific test results.

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
BH1	0.5-0.95	Residual Clay	5.3	370	22	4,500
BH2	0.5-0.7	Residual Clay	5.4	160	<10	8,800
BH4	2-2.5	Sandstone Bedrock	7.2	69	100	4,100

The above results indicate that the soils and weathered shale would have an exposure classification of 'Mild' when assessed in accordance with the criteria for concrete piling exposure classification given in Table 6.4.2 (C) of AS2159-2009 "Piling Design and Installation". Any concrete exposed to these conditions (e.g. piles) should have a characteristic concrete strength and cover as recommended in Table 6.4.3 of the standard.

The above results indicate that the soils and weathered shale would have an exposure classification of 'Non Aggressive' when assessed in accordance with the criteria for steel piling exposure classification given in Table 6.5.2 (C) of AS2159-2009. Any steel exposed to these conditions should have a uniform corrosion allowance as recommended in Table 6.5.3.

3.4 Bedrock Classification

Based on the Pells et al 1998 system, bedrock classifications have been applied to the shale and sandstone bedrock encountered in the cored boreholes. We note that the rock classification can vary depending on the size of footing and its influence zone. Therefore these classifications should be treated as a guide only. When footing types, sizes and founding depths are determined these classifications should be reviewed to confirm that they are suitable for the specific footing details. Some engineering judgment has been applied to the augered portions of the boreholes and those portions are approximate only.

Borehole No. Surface RL (mAHD)	Depth (m) and RL (mAHD) to Top of Assessed Rock Class				
	Class V	Class IV	Class III	Class II	Class I
BH1 (RL42.5)	-	1.8m (RL40.7)	6.3m (RL36.2)	9.3m (RL33.2)	-
BH2 (RL39.7)	0.7m (RL39.0)	7.4m (RL32.3)	-	7.9m (RL31.8)	-
BH3 (RL39.3)	1.3m (RL38.0)	5.5m (RL33.8)	-	7.8m (RL31.5)	-
BH4 (RL42.3)	-	1.8m (RL40.5)	-	2.8m (RL39.5)	-

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

As discussed in Section 3.2, the boreholes disclosed fill overlying residual soils that grade into shale and sandstone bedrock at depths ranging from 0.7m (~RL39.0m) in BH2 to 1.8m (~RL40.5m) in BH4 below existing surface levels. Groundwater was encountered at 1.2m (~RL38.1m) depth in BH3 and 4.0m (~RL38.3m) in BH4. Based on these results, we consider the following are the principal geotechnical issues for the proposed development:

1. The excavation to form the underground basement car park will remove the fill and residual soils and will encounter a varied bedrock profile, with relatively shallow high to very high strength sandstone in the south of the site (~RL39.5m), and a deeply weathered shale profile in the north, where moderate to high strength shale is encountered at about RL31.6m.
2. Dilapidation surveys of adjoining buildings that fall in the area of influence of the excavation are recommended prior to excavation commencing. During the excavation, every care should be taken not to undermine or render unstable the footings of any adjoining structure.
3. Excavation of the shale and sandstone bedrock will require hydraulic rock breaking equipment. Vibration effects (associated with general excavation but more critically rock excavation) on adjoining structures must be considered.
4. The excavation sides should be formed as temporary batters or supported by appropriate shoring/retention systems.
5. Vertical cuts in sandstone and shale of at least medium strength are a feasible option but suitable pile wall toe restraint, in the form of anchors, would be necessary if shoring is founded above bulk excavation level (BEL) as well as frequent geotechnical inspections of the excavated cut faces.
6. We recommend that the buildings are supported on footings uniformly founded in the shale and interbedded shale and sandstone bedrock below BEL.
7. Groundwater is not expected to be an issue for the excavation as the profile comprises relatively low permeability silty clay and bedrock and should be manageable through conventional gravity drains and/or sump and pump techniques, though seepage should be expected.
8. Based on the Envirolab test results the soils have a mild exposure classification to concrete piles and non-aggressive exposure classification to steel piles.
9. As King Georges Road is an RMS road a detailed analysis will have to be submitted to RMS to demonstrate that the excavation will not have adverse effects on the roadway.

Further comments on the above issues are provided in the subsequent sections of this report.

4.2 Dilapidation Surveys

Prior to commencement of demolition, excavation or construction, we recommend that dilapidation survey reports be carried out on the neighbouring buildings and structures/infrastructure that fall within the zone of influence of the excavation, which is defined by a distance back from the excavation perimeter of twice the total depth of the excavation. The reports would provide a record of existing conditions prior to commencement of the work. A copy of each report should be provided to the adjoining property owner who should be asked to confirm that it represents a fair assessment of existing conditions.

The reports should be carefully reviewed prior to excavation commencing. Excavations and retention systems will need to be carefully planned and constructed so as not to have any adverse effects on the adjoining buildings and structures.

4.3 Demolition, Excavation and Retention System

4.3.1 Demolition and Excavation

Prior to any excavation commencing we recommend that reference be made to the WorkCover Authority of NSW's "Code of Practice – Excavation Work" Safe Work Australia dated July 2014.

We expect that excavation of up to 13.5m will be required in order to achieve the bulk excavation level (BEL) of the lowest basement car park. An assessment of the excavation characteristics of the various strata is presented below.

The excavation of the soils and weathered shale and sandstone, and the selection of appropriate excavation equipment have been assessed on the basis of results from the boreholes and correlations with point load test results (Table B).

Assessment of excavation characteristics and productivity is not an exact science and contractors must make their own evaluation based on experience with specific equipment, and their own study of the borehole information. The ease with which excavation of rock is achieved depends upon the equipment used, the skill, and experience of the operator and the characteristics of the rock. The contractor must make their own judgement on all of these factors.

In the southern part of the site the basement excavation will be through soil and for the most part high to very high strength sandstone (about RL39.5m) and interbedded sandstone and shale bedrock. In the northern part of the site a deeper weathered bedrock profile is expected with the excavation through mainly extremely low to very low shale bedrock for the most part then moderate to high strength shale expected at about RL31.5m. The soil materials including weathered bedrock of less than low strength should be readily excavated using the buckets of conventional earthmoving equipment (such as hydraulic excavators) with some ripping.

Sandstone and shale bedrock of low or higher strength will require assistance using rock excavation techniques (such as ripping tynes fitted to excavators or dozers, hydraulic impact hammers, rock saws or rock grinders). Care will need to be taken when excavating using hydraulic impact hammers that vibrations transmitted to adjoining structures do not exceed tolerable levels. We recommend that where hydraulic impact hammers are to be used, full time quantitative vibration monitoring be carried out on the nearest residential buildings to the east and south to assess that transmitted vibrations are within tolerable limits. Where vibrations are found to be excessive then alternative lower vibration emitting equipment would need to be used.

4.3.2 Vibration Monitoring

If rock hammers are to be used, we recommend that the initial excavation in rock be commenced away from likely critical areas, with instrumental vibration monitoring undertaken. Trial excavations can then be undertaken together with vibration monitoring to assess how close the hammer can operate to any critical boundary, while maintaining transmitted vibrations within acceptable levels. Guideline levels of vibration velocity for evaluating the effects of vibration in structures are given in the attached Vibration Emission Design Goals sheet. We recommend that the acceptable limit for transmitted vibrations be set at a peak particle velocity of 5mm/s for frequencies of less than 10Hz at foundation level.

To fall within these limits, we recommend that the size of rock hammers initially used during the trial not exceed medium sized rock hammer say 900kg such as Krupp 580. If it is found that transmitted vibrations are unacceptable, then it would be necessary to change to a smaller excavator with a smaller rock hammer, or to a rotary grinder, rock saws, or jackhammers. The use of heavier hammers may well prove to be feasible. Continuous vibration monitoring should be carried out for the remainder of all percussive rock excavation with warning systems to alert site staff if tolerable threshold levels are exceeded.

The use of a rotary rock grinders or grid rock sawing in conjunction with ripping and/or hammering provide relatively low vibration excavation options. When using a rock saw or rotary rock grinder, the resulting dust must be suppressed by spraying with water.

Only excavation contractors with experience in similar work using a competent supervisor who is aware of vibration damage risks and rock face instability issues, etc. should be used. The contractor should have all appropriate statutory and public liability insurances.

4.3.3 Retention System and Batter Slopes

Excavations within the soils will not be self-supporting and batter slopes will need to be formed or shoring systems will need to be installed prior to the start of excavation. It should be possible to install an anchored soldier pile wall with the piles terminated once good quality shale or sandstone is encountered provided structural loads are not supported on the shoring piles in the long term. Alternatively the piles could be socketed below the bulk excavation into the shale and interbedded sandstone and shale but this would seem expensive and unnecessary, especially within the southern portion of the site, where high to very strength bedrock is encountered at relatively shallow depths unless the shoring piles have to carry permanent structural loads.

As there are substantial set-backs of the basement from the southern boundary, It should be feasible to form temporary batters at slopes of 1 Horizontal (H) to 1 Vertical (V). This batter must extend to the rock of at least moderate strength which occurs at a depth of 1.8m in the borehole locations (BH1 and BH4). The batter slope will necessitate considerable over-excavation and the spoil will have to be removed from site and later replaced with good quality material to backfill behind the retaining walls. It may be ultimately more cost effective and will produce a better outcome to install a soldier pile shoring system on all boundaries as outlined below. A shoring system will be required along the northern, eastern and western boundaries.

An anchored soldier pile wall is required and therefore permission for anchors will need to be obtained from the owners of the adjoining properties prior to the installation of the anchors below their properties. Temporary rock bolts are also likely to be required for stabilisation of rock face excavations. Such permission can take some time to obtain and should be sought early to allow time for negotiation. We expect that permanent lateral support would be provided by the floor slabs inside the excavation.

A soldier pile shoring system should comprise reinforced concrete piles which extend at least to the medium to high strength Class II shale and sandstone which occurs at depths ranging between 1.8m (~RL40.5m) in the south and at 7.70m (RL31.6m) in the north, as shown in the boreholes and Section 3.4. The depth of the piles also depends upon the level of the intermediate floor slab which will be needed to prop the base of the piles in the long term. Depending on the strength and quality of the bedrock at depth it may be feasible to found the shoring piles above the bulk excavation level, however this would need to be confirmed following additional cored borehole testing along the eastern and western boundaries, which should be completed following the demolition of the existing structures. Due to the variable conditions numerous boreholes will be necessary.

The following are guidelines for lateral pressures for design of shoring/retention systems. Again the presence of adjoining buildings must be taken into consideration in the assessment of lateral pressures, including those imposed from adjoining footing loads. The following lateral pressures may be assumed for design of shoring/retention systems if ‘Wallap’ or similar computer analysis is not completed.

1. A trapezoidal pressure distribution is applicable for anchored and propped walls. For the temporary case where there are no movement sensitive structures in the zone of influence of the wall a pressure of 4H kPa may assumed for all soil and weathered shale and sandstone of up to medium strength; the pressure should be uniform over the central 60% of the loaded area, tapering to zero at the top and base. For the long term case the pressure should be increased to 5H kPa. Where movement sensitive structures are present the pressure should be increased to 6H kPa in the short term and 8H kPa in the long term.
2. Assumed bulk unit weight of 20kN/m^3 for the subsoils and 25kN/m^3 for the bedrock.
3. The retaining walls should be designed to withstand hydrostatic pressure unless measures are taken to introduce complete and permanent drainage of the ground behind the wall. All surcharge loads should also be considered in the design of retaining walls.

Passive toe resistance of the retention system below the base of the bulk excavation, where piles extend below the base of the excavation, may be estimated based on a maximum allowable lateral resistance of 350kPa for shale and sandstone of medium or higher strength. The passive resistance should be ignored to at least 0.3m below the base of the excavation, including footing and service excavations. Where the retention piles are to accommodate building loads we recommend that these piles be founded below the base of the bulk excavation. Bored piers may be used on this site although allowance should be made for the removal of water from the base of the piers (where depths are moderate) during construction as some groundwater seepage into the piles should be expected. Where piers extend to substantial depths of 6m or more it will be necessary for concrete to be placed by tremie methods.

Temporary rock anchors will be required and unless piles are socketed below bulk excavation level there will need to be at least two rows of anchors.

Vertical excavations within the sandstone and shale would be feasible, but regular geotechnical inspections will be required during excavation to confirm this. Excavation should be terminated above the pile toe level to allow the installation of anchors for temporary lateral support and at this time a geotechnical inspection of the conditions exposed should be carried out. Following this, the geotechnical engineer should inspect the cut faces at regular intervals, say 1.5m, to check for any weak seams or inclined joints that require additional support, such as rock bolts and/or shotcrete and mesh. Any additional support required should be installed prior to further excavation. For the long term condition rock bolts which extend beyond site boundaries must be replaced by propped or braced support. The depth to which the bedrock is considered to be largely self-supporting varies considerably from north to south across the site and with the current borehole information we cannot determine how far (south to north) the shallow high to very high strength sandstone extends. As a minimum we recommend coring at least several additional boreholes along both the eastern and western boundaries.

Anchors should have their bond formed within sandstone of at least medium strength and may be provisionally designed based on an allowable bond stress of 400kPa. The anchor bond should be formed outside a line drawn up at 45° from the bulk excavation level, with a minimum free length of 4m and a minimum bond length of 3m. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 80% of the working load. Lift-off tests should be carried out on at least 10% of the anchors, 24 to 48 hours following locking off, to confirm that the anchors are holding their load. If any anchor fails the lift-off test then all anchors should be tested. No anchor stands should be cut such that a jack cannot be reinstalled without the permission of the geotechnical engineer. Generally anchors are installed on a design and construct contract so that optimisation of bond stresses does not become a contractual issue in the event of an anchor failing the test load.

4.3.4 Long Term Retention

Shale bedrock is not a durable material and if temporary shoring systems do not extend to BEL then additional long term support to the shale faces will be required. One such system would be to install prefabricated walling such as Dincel or CSR - AFS which is propped off the floor slabs and backfilled with no-fines concrete. Such walls could be designed for a lateral loading of 5kPa provided there are no adverse defects in the rock mass.

Where adverse defects are identified during the excavation and stabilised with temporary rock anchors, the anchor support must be replaced by propping from the internal structures whether the shoring is full depth or not.

4.4 Groundwater Considerations

Groundwater seepage was encountered in the boreholes during auger drilling and groundwater was measured at 1.2m (~RL38.1m) in BH3 and 4.0m (~RL38.3m) in BH4, three weeks after completion of drilling.

Groundwater is unlikely to be a significant issue for the proposed basement excavation. Considering the low permeability residual soils and sandstone bedrock beneath the site, the seepage rate into the excavation is expected to be comparatively low and therefore, we do not expect the construction of the basement to be any more difficult than deep basements in similar strata in the surrounding area. We expect some groundwater seepage flows will occur at the soil-rock interface and through joints and bedding planes within the completed cut faces, particularly after periods of heavy rain. In this regard we recommend that strip drains be incorporated behind all retaining walls and under slab drainage placed below all slabs. Hydraulic design should allow to pick up the strip and below slab drainage and re-direct the intercepted groundwater to the stormwater system. Seepage is expected to be satisfactorily controlled by gravity drainage systems or sumps and pumps where required.

We do not consider that there is a likelihood of the construction of the basement causing significant interference to the regional groundwater flow due to relatively impermeable nature of the subsurface profile, nor will the basement be untowardly affected by the groundwater provided proper drainage systems are designed and installed by a qualified hydraulic/drainage engineer.

4.5 Footing System

Shale and interbedded sandstone and shale bedrock will be exposed over the full excavation footprint at the bulk excavation level. Consequently, we recommend that the proposed structure be uniformly founded on bedrock using a pad and strip footings.

The moderate to high strength rock is deeper in the northern portion of the site (BH2 and BH3), and variations in the hidden topography could result in irregular depths to rock elsewhere on site not covered by the present boreholes. As mentioned in Section 4.3.3 we recommend that additional cored boreholes should be completed following demolition, not only to refine the shoring system design but within the site generally to check for any variations in the quality of the bedrock at the proposed founding level.

The following criteria are recommended for design and construction of these footings:

1. Strip/pad footings may be proportioned for an allowable bearing pressure (ABP) of up to 6000kPa for shale and sandstone of medium strength or better and should generally be applicable based on the existing cored borehole information. All footing excavations should be inspected by a geotechnical engineer and about one third of footings are spoon tested to confirm there are no significant defects in the zone of influence of the footings.
2. All loose or softened debris should be cleaned from the base of all pad footings and strip footings prior to concreting. All footings should be poured immediately after excavation, removal of water, cleaning and inspection.

4.6 Basement Slab and Pavement Design

The basement floor slabs will be placed on shale and interbedded sandstone and shale bedrock and as such no particular subgrade preparation is required apart from placing a 100mm layer of roadbase over the rock to form a separation or debonding layer. Alternatively a layer of free draining granular (20mm blue metal or similar) may be used to act as a drainage blanket as well as a separation layer.

The design of any proposed external pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of fill excavated or imported to the site, as well as vehicle loadings and use. Various alternative types of construction could be used for the pavements. Concrete construction would undoubtedly be the best in areas where heavy vehicles manoeuvre. Flexible pavements may have a lower initial cost but maintenance will be higher. These factors should be considered when making the final choice.

4.7 Further Work

The above comments and recommendations have been based on the four boreholes completed to date. As mentioned in Section 3.2 the difference in elevation between the two ends of the site is not sufficient to directly account for the difference in geology between the boreholes and there must be some depositional or structural geological anomaly to account for the difference. Following demolition of the existing structures or when access is made available, we recommend that additional cored boreholes be carried out to further assess the ground conditions and refine the design of the shoring and footing systems.

It is also possible that seepage analysis may be required at a later date if the Office of Water become involved in the application process, and that finite element modelling of the excavation and shoring system will be required if Road and Maritime Services are required to approve the development proposal.

5 GENERAL COMMENTS

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

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

Occasionally, the subsurface conditions between 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.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT TEST REPORT

Client: JK Geotechnics
Project: Proposed Mixed Use Development
Location: 62-70 King Georges Road &
280-292 Lakemba Street, Wile Park, NSW

Ref No: 30501S
Report: A
Report Date: 31/05/2017
Page 1 of 1

AS 1289	**TEST METHOD**	**2.1.1**
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
2	2.50-3.00	12.9
2	5.50-6.00	9.3
3	2.70-3.00	11.7
3	5.50-6.00	9.3

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	30501S
Project:	Proposed Mixed Use Development	**Report:**	B
Location:	62-70 King Georges Road & 280-292 Lakemba Street, Wiley Park, NSW	**Report Date:**	5/06/2017
		Page 1 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	2.73-2.76	2.1	42
	3.13-3.16	2.5	50
	3.90-3.93	1.6	32
	4.18-4.20	2.3	46
	4.60-4.63	0.9	18
	5.31-5.33	3.8	76
	5.75-5.78	2.2	44
	6.05-6.07	2.1	42
	6.92-6.95	2.3	46
	7.26-7.28	1.7	34
	7.74-7.78	1.6	32
	8.28-8.31	1.1	22
	8.80-8.83	1.1	22
	9.14-9.17	1.2	24
	9.69-9.72	1.3	26
	10.30-10.33	0.8	16
	10.74-10.76	0.8	16
	11.26-11.29	1.1	22
2	8.14-8.17	1.1	22
	8.78-8.82	1.4	28
	9.17-9.20	2.1	42
	9.81-9.85	1.2	24
	10.18-10.21	1.4	28
	10.73-10.77	1.2	24
	11.15-11.19	1.2	24
	11.73-11.76	4.1	82

NOTES: See Page 3 of 3

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TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	30501S
Project:	Proposed Mixed Use Development	**Report:**	B
Location:	62-70 King Georges Road & 280-292 Lakemba Street, Wiley Park, NSW	**Report Date:**	5/06/2017
		Page 2 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
3	6.80-6.83	0.4	8
	7.17-7.19	0.4	8
	7.79-7.82	1.1	22
	8.39-8.43	1.0	20
	8.80-8.83	0.8	16
	9.13-9.17	1.3	26
	9.74-9.78	1.1	22
	10.17-10.20	0.8	16
	10.79-10.82	2.5	50
	11.26-11.29	1.3	26
	11.85-11.87	1.2	24
	12.07-12.09	1.8	36
	12.40-12.43	1.3	26
4	2.81-2.84	4.2	84
	3.16-3.19	4.6	92
	3.76-3.79	2.5	50
	4.09-4.11	6.2	124
	4.82-4.85	3.6	72
	5.25-5.28	4.9	98
	5.94-5.96	2.6	52
	6.24-6.28	3.3	66
	6.78-6.80	4.6	92
	7.19-7.22	5.7	114
	7.82-7.84	1.9	38
	8.12-8.16	2.7	54

NOTES: See Page 3 of 3

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	30501S
Project:	Proposed Mixed Use Development	**Report:**	B
Location:	62-70 King Georges Road & 280-292 Lakemba Street, Wiley Park, NSW	**Report Date:**	5/06/2017
		Page 3 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
4	8.70-8.73	1.3	26
	9.26-9.29	1.1	22
	9.86-9.90	2.0	40
	10.25-10.27	1.5	30
	10.79-10.82	2.0	40
	11.21-11.24	1.1	22
	11.82-11.84	3.0	60
	12.16-12.18	3.0	60

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: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. 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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Envirolab Services Pty Ltd - Sydney | ABN 37 112 535 645

CERTIFICATE OF ANALYSIS 167921

Client:
JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: K Singh

Sample log in details:

Your Reference:	**30501S, Wiley Park**
No. of samples:	3 Soils
Date samples received / completed instructions received	25/05/17 / 25/05/17

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:	1/06/17 / 30/05/17
Date of Preliminary Report:	Not Issued

NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025 - Testing **Tests not covered by NATA are denoted with *.**

Results Approved By:

David Springer
General Manager

Misc Inorg - Soil				
Our Reference:	UNITS	167921-1	167921-2	167921-3
Your Reference	-------------	BH1	BH2	BH4
Depth	------------	0.5-0.95	0.5-0.7	2-2.5
Date Sampled		23/05/2017	23/05/2017	23/05/2017
Type of sample		Soil	Soil	Soil
Date prepared	-	28/05/2017	28/05/2017	28/05/2017
Date analysed	-	28/05/2017	28/05/2017	28/05/2017
pH 1:5 soil:water	pH Units	5.3	5.4	7.2
Chloride, Cl 1:5 soil:water	mg/kg	22	<10	100
Sulphate, SO4 1:5 soil:water	mg/kg	370	160	69
Resistivity in soil*	ohm cm	4,500	8,800	4,100

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.

QUALITY CONTROL Misc Inorg - Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			28/05/2017	[NT]	[NT]	LCS-1	28/05/2017
Date analysed	-			28/05/2017	[NT]	[NT]	LCS-1	28/05/2017
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	92%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	109%
Resistivity in soil*	ohm cm	1	Inorg-002	<1.0	[NT]	[NT]	[NR]	[NR]

Report Comments:

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
NR: 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 batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

BOREHOLE LOG

Borehole No.
1
1 / 3

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S **Method:** SPIRAL AUGER **R.L. Surface:** ~42.5 m
Date: 23/5/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.S./P.S.

Groundwater Record	SAMPLES ES	U50	DB	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 ON AUGERING							0	■		ASPHALTIC CONCRETE: 100mm.t				
	■							▩	-	FILL: Gravelly sand, fine to coarse grained, dark grey.	M			ROADBASE
AFTER 1 DAY	■			■	N = 5 1,2,3	42		▨	CL-CH	SILTY CLAY: medium to high plasticity, light grey mottled orange brown, with fine to coarse grained ironstone gravel.	MC~PL	St	150 160 130	RESIDUAL
				■	N > 13 3,6,7/ 50mm REFUSAL	41	1	▨				VSt	280 220 270	
13/6/17				■		40	2	░		SANDSTONE: fine grained, grey.	DW	M - H		MODERATE TO HIGH 'TC' BIT RESISTANCE
						39 38 37 36	3 4 5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 11.6m, MACHINE SLOTTED PVC STANDPIPE 8.6m TO 11.6m, CASING 0m TO 8.6m, 2mm SAND FILTER PACK 7.5m TO 11.6m, BENTONITE SEAL 5.0m TO 7.5m, BACKFILLED WITH SAND TO SURFACE AND COMPLETED WITH A CONCRETED GATIC COVER

JK Geotechnics

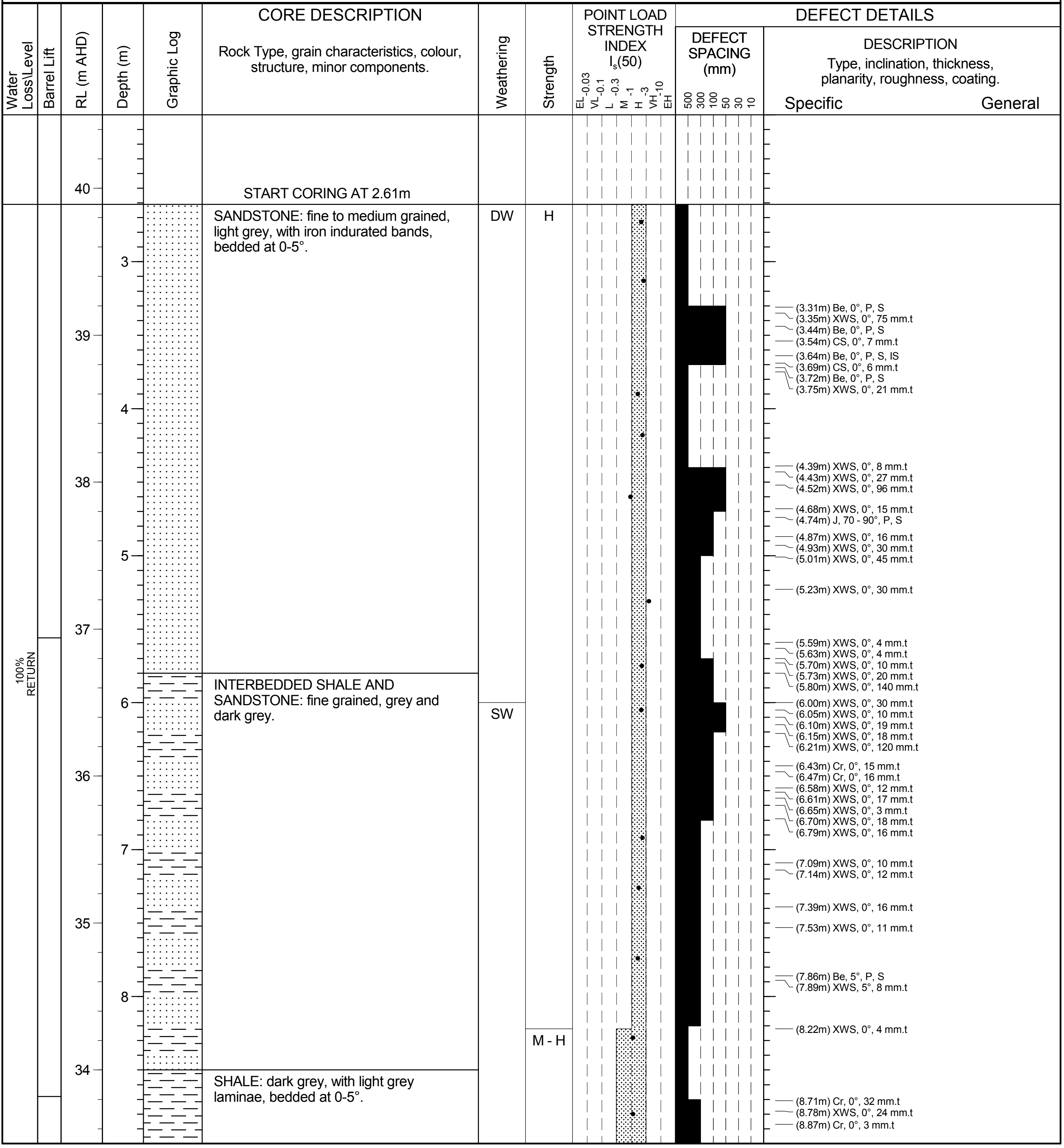
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1

CORED BOREHOLE LOG

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S
Core Size: NMLC
R.L. Surface: ~42.5 m
Date: 23/5/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: K.S./P.S.



START CORING AT 2.61m

SANDSTONE: fine to medium grained, light grey, with iron indurated bands, bedded at 0-5°. DW H

INTERBEDDED SHALE AND SANDSTONE: fine grained, grey and dark grey. SW

M - H

SHALE: dark grey, with light grey laminae, bedded at 0-5°.

(3.31m) Be, 0°, P, S
(3.35m) XWS, 0°, 75 mm.t
(3.44m) Be, 0°, P, S
(3.54m) CS, 0°, 7 mm.t
(3.64m) Be, 0°, P, S, IS
(3.69m) CS, 0°, 6 mm.t
(3.72m) Be, 0°, P, S
(3.75m) XWS, 0°, 21 mm.t
(4.39m) XWS, 0°, 8 mm.t
(4.43m) XWS, 0°, 27 mm.t
(4.52m) XWS, 0°, 96 mm.t
(4.68m) XWS, 0°, 15 mm.t
(4.74m) J, 70 - 90°, P, S
(4.87m) XWS, 0°, 16 mm.t
(4.93m) XWS, 0°, 30 mm.t
(5.01m) XWS, 0°, 45 mm.t
(5.23m) XWS, 0°, 30 mm.t
(5.59m) XWS, 0°, 4 mm.t
(5.63m) XWS, 0°, 4 mm.t
(5.70m) XWS, 0°, 10 mm.t
(5.73m) XWS, 0°, 20 mm.t
(5.80m) XWS, 0°, 140 mm.t
(6.00m) XWS, 0°, 30 mm.t
(6.05m) XWS, 0°, 10 mm.t
(6.10m) XWS, 0°, 19 mm.t
(6.15m) XWS, 0°, 18 mm.t
(6.21m) XWS, 0°, 120 mm.t
(6.43m) Cr, 0°, 15 mm.t
(6.47m) Cr, 0°, 16 mm.t
(6.58m) XWS, 0°, 12 mm.t
(6.61m) XWS, 0°, 17 mm.t
(6.65m) XWS, 0°, 3 mm.t
(6.70m) XWS, 0°, 18 mm.t
(6.79m) XWS, 0°, 16 mm.t
(7.09m) XWS, 0°, 10 mm.t
(7.14m) XWS, 0°, 12 mm.t
(7.39m) XWS, 0°, 16 mm.t
(7.53m) XWS, 0°, 11 mm.t
(7.86m) Be, 5°, P, S
(7.89m) XWS, 5°, 8 mm.t
(8.22m) XWS, 0°, 4 mm.t
(8.71m) Cr, 0°, 32 mm.t
(8.78m) XWS, 0°, 24 mm.t
(8.87m) Cr, 0°, 3 mm.t

JK_LIB_CURRENT - V8.00.GLB Log J & K CORED BOREHOLE - MASTER 30501S WILEY PARK.GPJ <<DrawingFile>> 13/06/2017 12:18 Produced by gINT Professional, Developed by Datgel

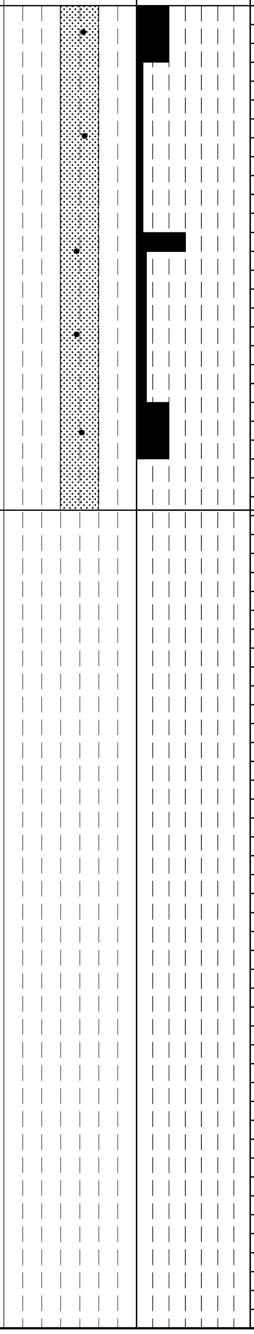
CORED BOREHOLE LOG

Borehole No.
1
3 / 3

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S
Core Size: NMLC
R.L. Surface: ~42.5 m
Date: 23/5/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: K.S./P.S.

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
100% RETURN		33 32 31	10 11		SHALE: dark grey, with light grey laminae.	FR	M - H			(9.20m) FRAGMENTED ZONE, 26 mm.t (9.35m) Be, 0°, P, S (10.17m) Be, 0°, P, S (10.32m) Be, 0°, P, S (11.04m) Be, 0°, P, S (11.44m) Be, 0°, P, S	
		30 29 28 27	12 13 14 15		END OF BOREHOLE AT 11.67 m						




JK Geotechnics
30501S BH1 START CORING AT 2.61m
2
3
4
5
6
7
8
9
10
11
END OF BH1 AT 11.67m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2
1 / 2

BOREHOLE LOG

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S **Method:** SPIRAL AUGER **R.L. Surface:** ~39.7 m
Date: 23/5/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.S./P.S.

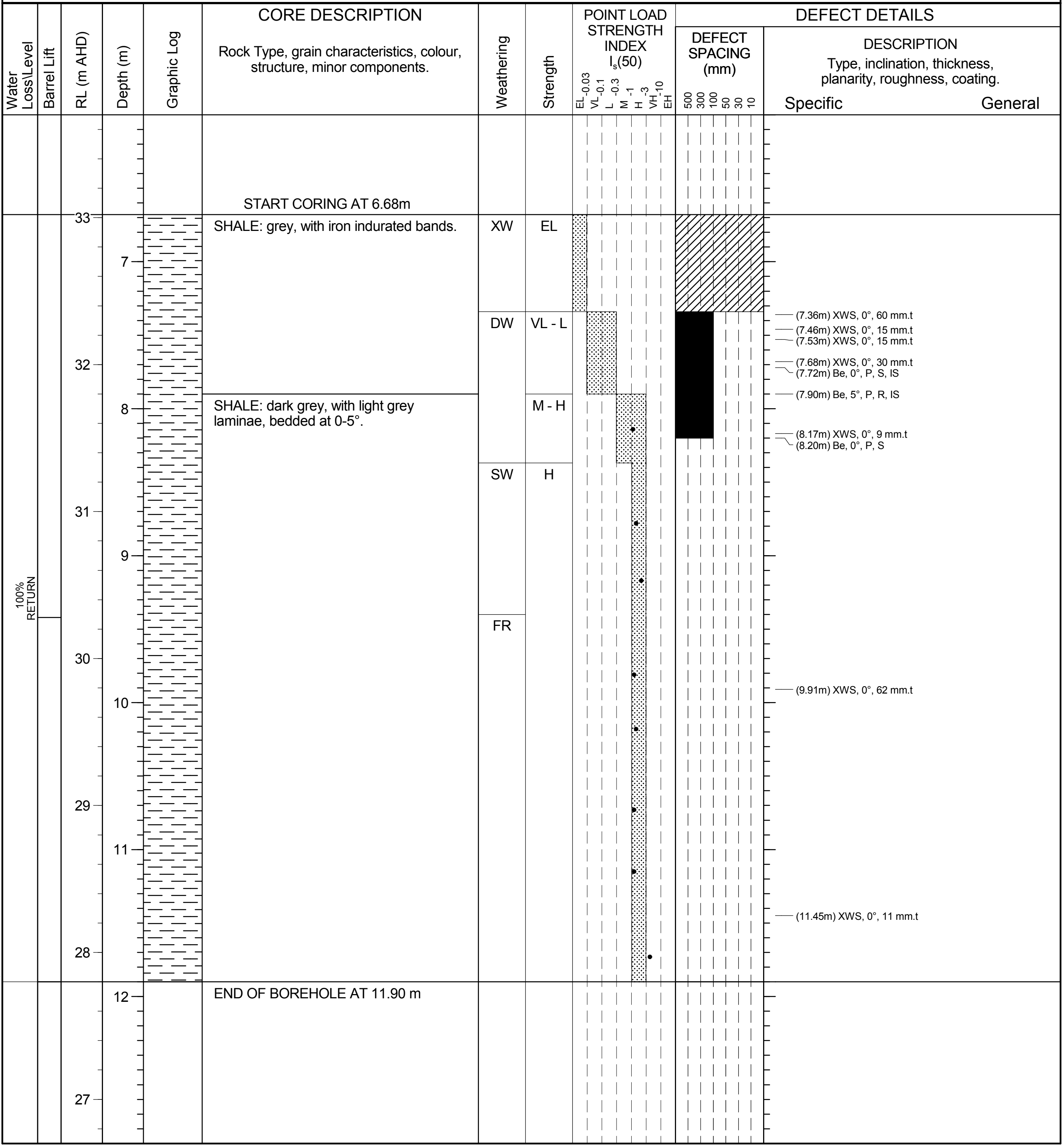
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION ON AUGERING	■									CONCRETE: 130mm.t				
									-	FILL: Gravelly sand, fine to coarse grained, grey and brown.	D			
	■			■	N > 5 2,5/ 50mm REFUSAL				CL-CH	SILTY CLAY: medium to high plasticity, grey mottled orange brown, with fine to coarse grained ironstone gravel.	MC~PL	VSt	200 270 300	RESIDUAL
						39				SHALE: grey, with iron indurated bands.	XW	EL		
				■			1							
						38								
							2							
				■		37								
							3							
						36								
				■			4				XW - DW	EL - VL		VERY LOW TO LOW 'TC' BIT RESISTANCE
						35								
							5							
				■		34								
							6			SHALE: grey, with XW seams.	DW	VL		LOW RESISTANCE
						33				REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
2
2 / 2

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S
Core Size: NMLC
R.L. Surface: ~39.7 m
Date: 23/5/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: K.S./P.S.



Water Loss\Level	Barrel Lift	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 6.68m				
100% RETURN		SHALE: grey, with iron indurated bands.	XW	EL		
			DW	VL - L	(7.36m) XWS, 0°, 60 mm.t (7.46m) XWS, 0°, 15 mm.t (7.53m) XWS, 0°, 15 mm.t (7.68m) XWS, 0°, 30 mm.t (7.72m) Be, 0°, P, S, IS (7.90m) Be, 5°, P, R, IS	
		SHALE: dark grey, with light grey laminae, bedded at 0-5°.		M - H	(8.17m) XWS, 0°, 9 mm.t (8.20m) Be, 0°, P, S	
			SW	H		
			FR		(9.91m) XWS, 0°, 62 mm.t (11.45m) XWS, 0°, 11 mm.t	
		END OF BOREHOLE AT 11.90 m				


JK Geotechnics
305015 BH2 START CORING AT 6.68m
6
7
8
9
10
11
END OF BH2 AT 11.90m
18.80m

BOREHOLE LOG

Borehole No.
3
1 / 2

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S **Method:** SPIRAL AUGER **R.L. Surface:** ~39.3 m
Date: 24/5/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.S./P.S.

Groundwater Record	SAMPLES ES	U50	DB	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 ON AUGERING						39			-	ASPHALTIC CONCRETE: 40mm.t	M			
										FILL: Gravelly sand, fine to coarse grained, brown, fine to coarse grained igneous gravel.				
13/6/17 EOH					N = 5 1,2,3		1		CL-CH	SILTY CLAY: medium to high plasticity, light grey mottled orange brown, with fine to coarse grained ironstone gravel.	MC~PL	F - St	90 110 120	RESIDUAL
						38				SHALE: grey, with iron indurated bands, some L strength bands.	XW	EL		
						37, 36, 35, 34	2, 3, 4, 5							GROUNDWATER MONITORING WELL INSTALLED TO 12.1m, MACHINE SLOTTED PVC STANDPIPE 9.1m TO 12.1m, CASING 0m TO 9.1m, 2mm SAND FILTER PACK 7.3m TO 12.1m, BENTONITE SEAL 5.0m TO 7.3m, BACKFILLED WITH SAND TO SURFACE AND COMPLETED WITH A CONCRETED GATIC COVER
											DW	VL		VERY LOW TO LOW 'TC' BIT RESISTANCE
						33	6					VL - L		LOW RESISTANCE
										REFER TO CORED BOREHOLE LOG				

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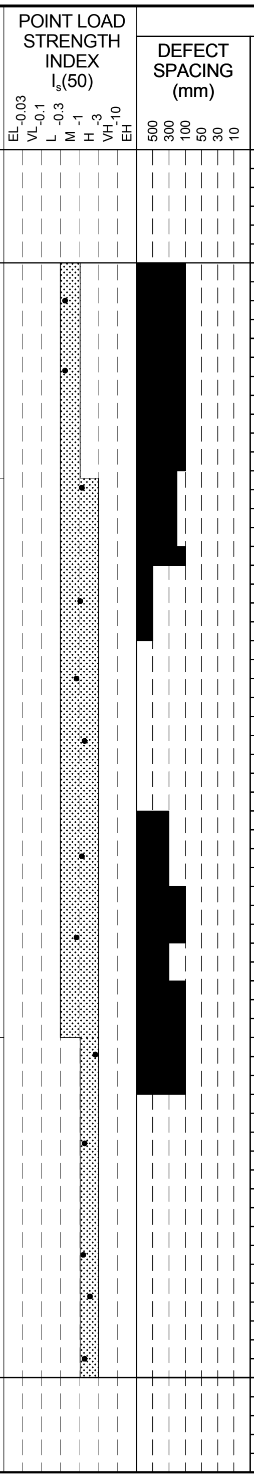
CORED BOREHOLE LOG

Borehole No.
3
2 / 2

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S
Core Size: NMLC
R.L. Surface: ~39.3 m
Date: 24/5/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: K.S./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		33			START CORING AT 6.60m				
100% RETURN		32	7		SHALE: grey and orange brown, with iron indurated bands.	DW	M	(6.63m) XWS, 0°, 30 mm.t (6.75m) XWS, 0°, 17 mm.t (6.89m) XWS, 0°, 4 mm.t (7.06m) XWS, 0°, 40 mm.t (7.16m) XWS, 0°, 10 mm.t (7.20m) XWS, 0°, 15 mm.t (7.39m) XWS, 0°, 16 mm.t (7.46m) XWS, 0°, 12 mm.t (7.55m) XWS, 0°, 70 mm.t (7.71m) XWS, 0°, 20 mm.t	
		31 30 29	8 9 10		SHALE: dark grey, with light grey laminae.	SW	M - H	(8.03m) J, 80°, P, S (8.12m) J, 70°, P, S (8.60m) Cr, 0°, 7 mm.t (9.54m) Be, 0°, P, S (9.92m) Cr, 0°, 4 mm.t (10.04m) XWS, 0°, 10 mm.t (10.13m) Be, 0°, P, S (10.35m) J, 80°, P, S (10.44m) FRAGMENTED ZONE, 0°, 45 mm.t (10.58m) XWS, 0°, 9 mm.t (10.63m) XWS, 0°, 14 mm.t	
		28 27	11 12			FR	H	(10.79m) Be, 0°, P, S (10.93m) XWS, 0°, 10 mm.t (11.78m) Be, 0°, P, S (12.48m) XWS, 0°, 40 mm.t	
					END OF BOREHOLE AT 12.50 m				




JK Geotechnics
30501S BH3 START CORING AT 6·60m
6
7
8
9
10
11
12
END OF BH3 AT 12·50m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
4
1 / 3

BOREHOLE LOG

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S
Method: SPIRAL AUGER
R.L. Surface: ~42.3 m
Date: 24/5/17
Datum: AHD
Plant Type: JK350
Logged/Checked By: K.S./P.S.

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 ON AUGERING										ASPHALTIC CONCRETE: 50mm.t				
	■					42			-	FILL: Gravelly silty sand, fine to coarse grained, brown, trace of clay.	M			
	■			■	N = 13 4,6,7				CL-CH	SILTY CLAY: medium to high plasticity, light grey and red brown, with fine to coarse grained ironstone gravel.	MC<PL	H	490 500 520	RESIDUAL
						41	1							
				■	N > 21 2,18,3/ 20mm REFUSAL								>600 >600	
ON COMPLETION OF CORING ▼				■		40	2			SANDSTONE: fine to medium grained, grey.	DW	M		MODERATE 'TC' BIT RESISTANCE
						39 38 37 36	3 4 5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 12.2m, MACHINE SLOTTED PVC STANDPIPE 9.2m TO 12.2m, CASING 0m TO 9.2m, 2mm SAND FILTER PACK 7.0m TO 12.2m, BENTONITE SEAL 2.0m TO 7.0m, BACKFILLED WITH SAND TO SURFACE AND COMPLETED WITH A CONCRETED GATIC COVER

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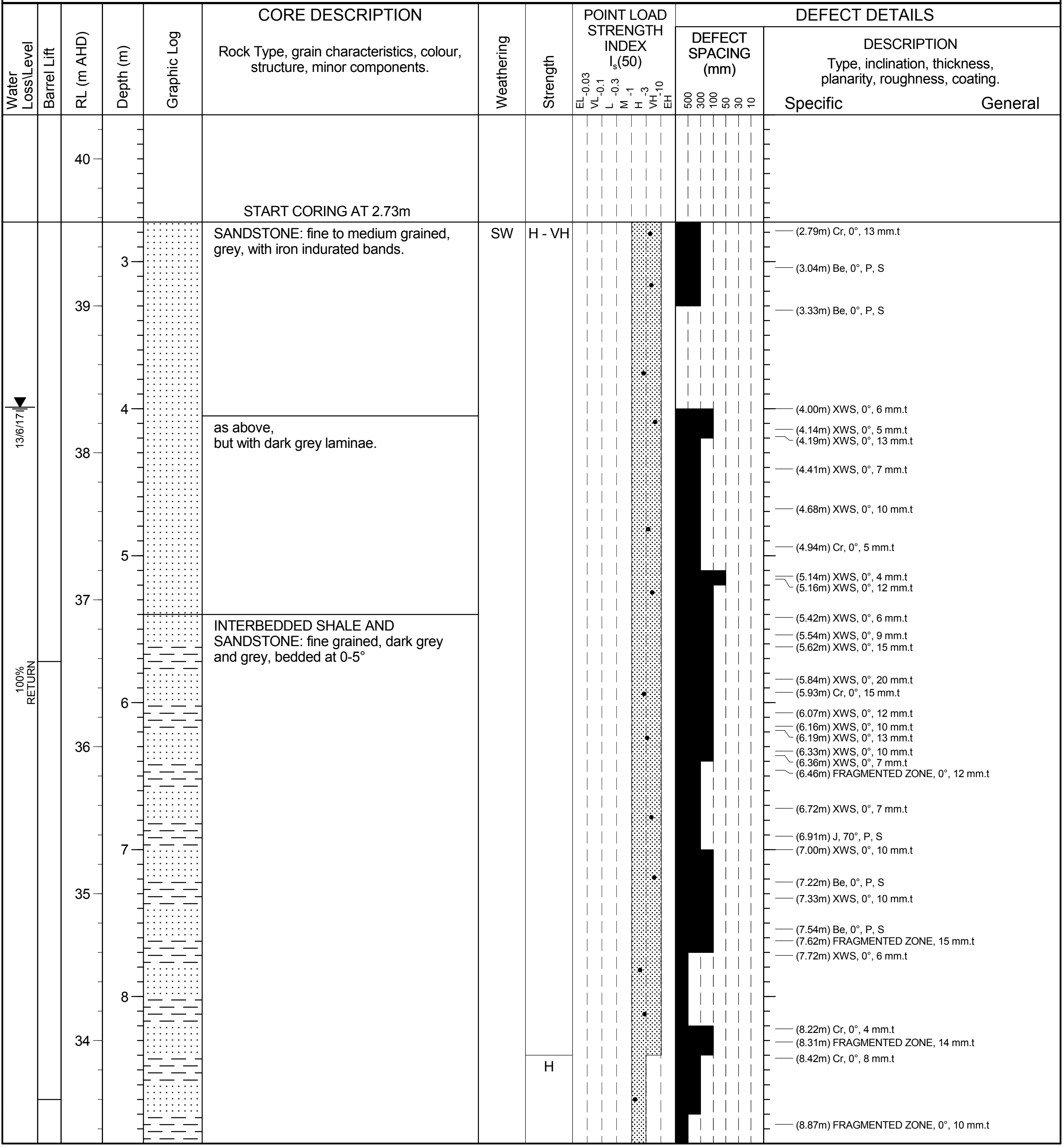
CORED BOREHOLE LOG

Borehole No.
4

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S
Core Size: NMLC
R.L. Surface: ~42.3 m
Date: 24/5/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK350
Bearing: N/A
Logged/Checked By: K.S./P.S.



START CORING AT 2.73m

CORE DESCRIPTION	Weathering	Strength
SANDSTONE: fine to medium grained, grey, with iron indurated bands.	SW	H - VH
as above, but with dark grey laminae.		
INTERBEDDED SHALE AND SANDSTONE: fine grained, dark grey and grey, bedded at 0-5°		H

(2.79m) Cr, 0°, 13 mm.t
(3.04m) Be, 0°, P, S
(3.33m) Be, 0°, P, S
(4.00m) XWS, 0°, 6 mm.t
(4.14m) XWS, 0°, 5 mm.t
(4.19m) XWS, 0°, 13 mm.t
(4.41m) XWS, 0°, 7 mm.t
(4.68m) XWS, 0°, 10 mm.t
(4.94m) Cr, 0°, 5 mm.t
(5.14m) XWS, 0°, 4 mm.t
(5.16m) XWS, 0°, 12 mm.t
(5.42m) XWS, 0°, 6 mm.t
(5.54m) XWS, 0°, 9 mm.t
(5.62m) XWS, 0°, 15 mm.t
(5.84m) XWS, 0°, 20 mm.t
(5.93m) Cr, 0°, 15 mm.t
(6.07m) XWS, 0°, 12 mm.t
(6.16m) XWS, 0°, 10 mm.t
(6.19m) XWS, 0°, 13 mm.t
(6.33m) XWS, 0°, 10 mm.t
(6.36m) XWS, 0°, 7 mm.t
(6.46m) FRAGMENTED ZONE, 0°, 12 mm.t
(6.72m) XWS, 0°, 7 mm.t
(6.91m) J, 70°, P, S
(7.00m) XWS, 0°, 10 mm.t
(7.22m) Be, 0°, P, S
(7.33m) XWS, 0°, 10 mm.t
(7.54m) Be, 0°, P, S
(7.62m) FRAGMENTED ZONE, 15 mm.t
(7.72m) XWS, 0°, 6 mm.t
(8.22m) Cr, 0°, 4 mm.t
(8.31m) FRAGMENTED ZONE, 14 mm.t
(8.42m) Cr, 0°, 8 mm.t
(8.87m) FRAGMENTED ZONE, 0°, 10 mm.t

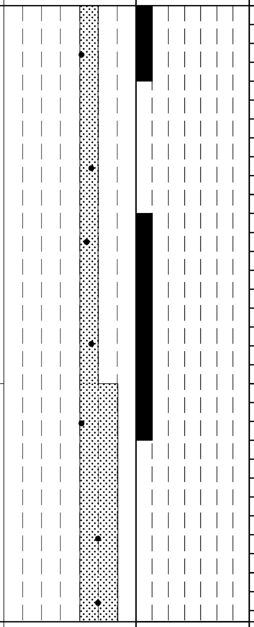
CORED BOREHOLE LOG

Borehole No.
4
3 / 3

Client: ABM BUILT
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 62-70 KING GEORGES ROAD & 280-292 LAKEMBA STREET, WILEY PARK, NSW

Job No.: 30501S
Core Size: NMLC
R.L. Surface: ~42.3 m
Date: 24/5/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK350
Bearing: N/A
Logged/Checked By: K.S./P.S.

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
100% RETURN		33		INTERBEDDED SHALE AND SANDSTONE: fine grained, dark grey and grey.	FR	H	(9.46m) Be, 0°, P, S
		32	10				(10.13m) J, 90°, P, S
							(10.57m) XWS, 0°, 5 mm.t
		31	11			H - VH	(10.97m) CS, 0°, 2 mm.t
							(11.26m) CS, 0°, 2 mm.t
		30	12				(12.09m) Be, 0°, P, S
				END OF BOREHOLE AT 12.26 m			
		29	13				
		28	14				
		27	15				



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30501S BH4 START CORING AT 2·73m
2
3
4
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12
END OF BH4 AT 12·26m

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

PLOT DATE: 26/05/2017 2:33:13 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30501S WILEY PARK\CAD\30501S.DWG

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:

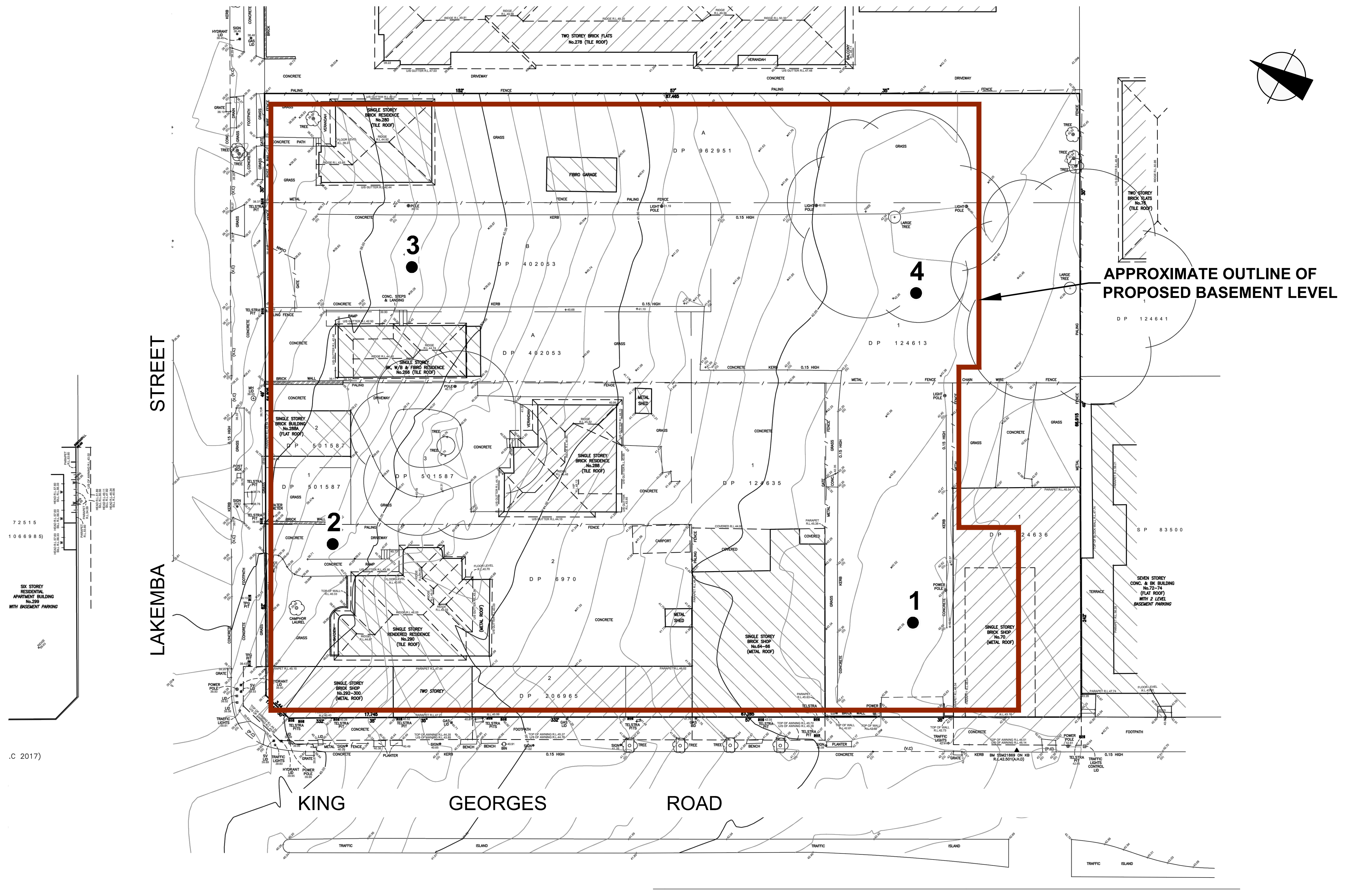
SITE LOCATION PLAN

Location:
62-70 KING GEORGES ROAD &
280-292 LAKEMBA STREET, WILEY PARK, NSW

Report No: 30501S

Figure No: 1

JK Geotechnics



0 4 8 12 16 20

SCALE 1:400 @A3 METRES

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

Title:

BOREHOLE LOCATION PLAN

Location: 62-70 KING GEORGES ROAD &
280-292 LAKEMBA STREET, WILEY PARK, NSW

Report No: 30501S

Figure No: 2

JK Geotechnics

PLOT DATE: 21/06/2017 11:54:23 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30501S WILEY PARK\CAD\30501S.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 (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 a Cone Penetrometer Test (CPT). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm or 44mm 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 a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm 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 CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT 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.

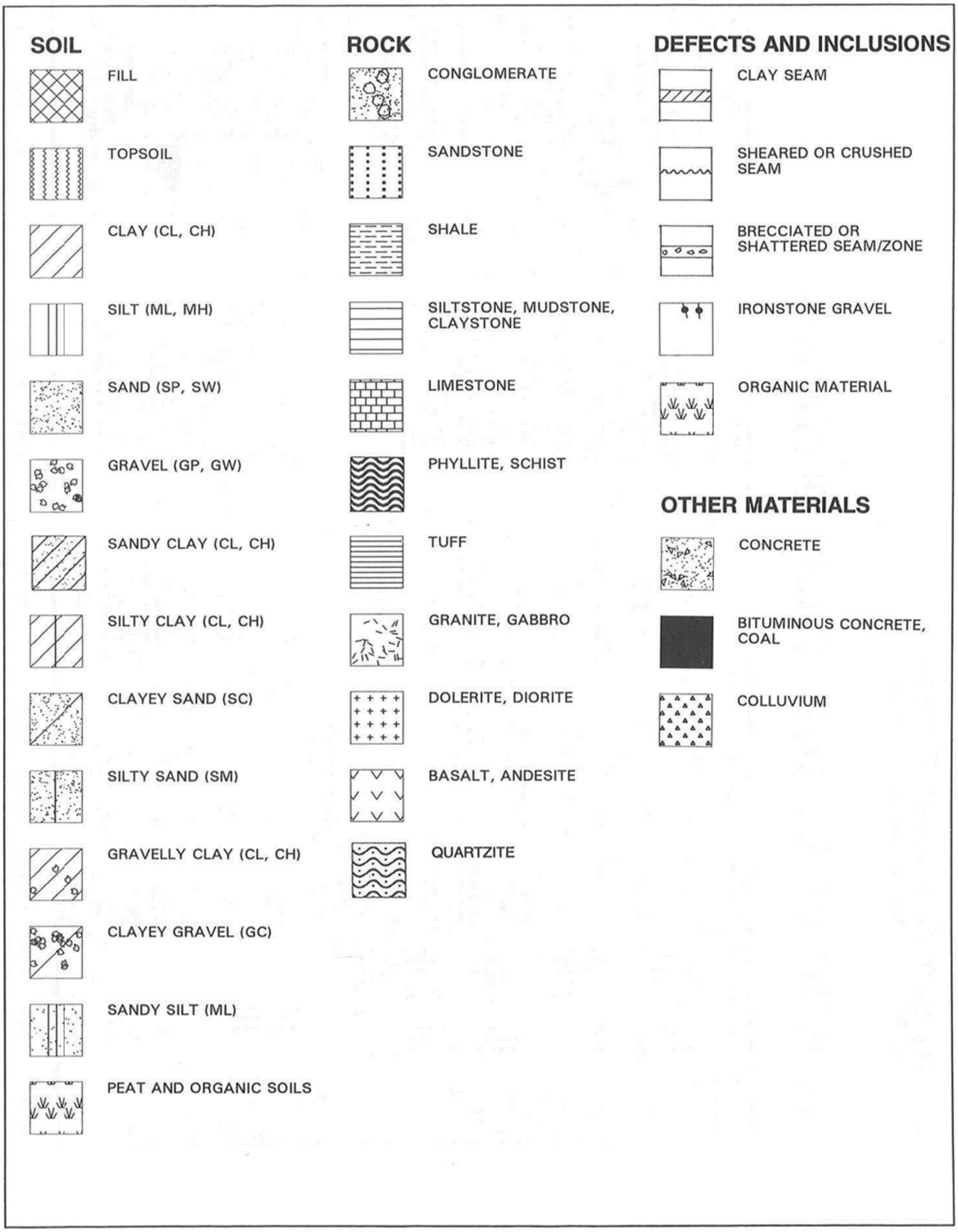
Requirements could range from:

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

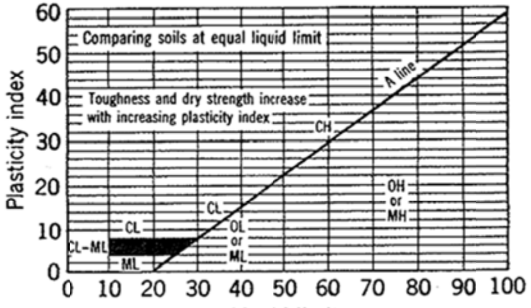
- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

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

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	
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size											
			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)							
	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.

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LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION		
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.		
	—C—	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.		

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	