REPORT TO

WEST END MAZDA

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED COMMERCIAL DEVELOPMENT

AT

574-584 CHURCH STREET, PARRAMATTA, NSW

Date: 20 October 2020
Ref: 33532SCrpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed commercial development located at 574-584 Church Street, Parramatta, NSW. The investigation was commissioned by Mr Stephen Craig of Greenwich Projects on behalf of West End Mazda by email dated 10 September 2020. The commission was on the basis of our fee proposal (Ref: P52550S, dated 28 August 2020). The location of the site is shown in Figure 1.

Based on the following documents supplied to us:

- Geotechnical Brief prepared by Northrop Consulting Engineers (Northrop) (Ref: SY190111, dated 26 August 2020;
- Detailed survey drawings prepared by Usher & Company (UC) (Ref: 6003-DET, dated 27 November 2018).
- Preliminary architectural drawings for Development Application approval, prepared by Gray Puksand (GP) (Ref 218024; Drawing No. DA010, DA100, DA101, DA102, DA210, DA211 and DA200, Revision P3; dated 24 & 28 August 2020).

We understand that the proposed commercial development will comprise the following:

- Demolition of the existing brick and concrete block buildings (single storey and two storey);
- Demolition of the existing external asphaltic concrete and concrete pavements;
- Construction of a proposed car dealership and workshop. The building will comprise a lower ground level with a one and two-storey concrete slab and column layout above. The proposed lower ground level (FFL: RL 19.19m) will require excavation to a maximum depth of about 3.6m over the southern portion of the proposed building footprint;
- Customer carparks to the south and north of the proposed building;
- External car display area at the south-western corner of the site
- Two-lane driveway around the proposed building.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions from seven borehole locations and, based on the information obtained, to present our comments and recommendations on geotechnical aspects of the proposed development as outlined in the brief including site classification, excavation, demolition, retention, footing design, potential settlement, possible dewatering issues, soil aggression, earthquake design parameters and pavement design etc.

This geotechnical investigation was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E33532BA for the results of the environmental site assessment.

2 BACKGROUND INFORMATION

In 2016, JK Geotechnics completed a geotechnical investigation for the West End Mazda showroom and office building, located at 588 Church Street. Our 2016 investigation included one augered borehole and one cored borehole. Sandstone bedrock was encountered at 4.6m depth below the ground surface level at the time (approx. RL18.9m) in the cored borehole, while no competent bedrock was encountered over the 12m investigation depth in the augered borehole (It appeared that the site was located over an infilled gully feature (paleochannel). As the subsurface conditions encountered in the current investigation differ greatly to those encountered previously, we have not referred to the 2016 investigation results in the preparation of the current report.

3 INVESTIGATION PROCEDURE

The fieldwork for the current investigation was carried out on 16, 17 and 18 September 2020 and comprised the drilling of seven boreholes (BH1 to BH7) using our track mounted JK308 drill rig, at the locations shown on Figure 2. The boreholes were initially auger-drilled to depths ranging from 1.35m to 4.8m and were then continued by diamond coring techniques using an NMLC core barrel with water flush to total depths ranging from 4.35m to 7.78m below existing ground surface level.

The borehole locations were set out by tape measurements from apparent site boundaries. The surface RL's indicated on the attached borehole logs were interpolated between spot level heights and ground contour lines shown on the supplied survey plan prepared by Usher & Company (Ref: 6003-DET, dated 27 November 2018) and are therefore only approximate. The survey datum is AHD.

The relative compaction of the fill and strength of the natural alluvial/residual soil was assessed by Standard Penetration Test (SPT) 'N' values, which were then augmented, where possible, by hand penetrometer tests on cohesive soil samples recovered in the SPT split tubes. The strength of the bedrock profile in the augered portion of the boreholes, was assessed by observation of the auger penetration resistance using a Tungsten Carbide (TC) bit, together with examination of the recovered rock cuttings and subsequent correlation with laboratory moisture content tests. The moisture content test results are summarised in the attached STS Table A. It should be noted that strengths assessed in this way are approximate only and variances of one strength order should not be unexpected. The strength of the cored bedrock was assessed by examination of the recovered rock core, together with correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests. The point load strength index tests results are summarised on the cored borehole logs.

Groundwater observations were made during and on completion of auger drilling. Thereafter, the use of water for core drilling limited further meaningful measurements of groundwater levels. Slotted PVC monitoring wells were installed within BH3 and BH4 on completion of the drilling (details are shown on the borehole logs). A return visit to the site was made on 23 September 2020 to measure groundwater levels. No longer term monitoring of groundwater levels was carried out.

Our geotechnical engineer (Bryan Zheng) was present full-time during the fieldwork to make surface observations, set out the borehole locations, direct the electro-magnetic scanning, nominate testing and sampling, and prepare logs of the strata encountered. The borehole logs, including colour photographs of the recovered core, are attached to this report together with a set of explanatory notes which describe the investigation techniques, and their limitations. The Report Explanation Notes also define the logging terms and symbols used.

Selected soil samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories, for testing to determine moisture contents, Atterberg limits, linear shrinkages, standard compaction properties, CBR values, point load strength index values, pH, sulphate contents, chloride contents and resistivity. The results of the laboratory testing are presented in the attached STS Tables A, B and C and Envirolab Certificate of Analysis 251973.

4 RESULTS OF INVESTIGATION

4.1 Site Description

As shown in Figure 1, the site is located approximate 250m to the east of Darling Mills Creek in Parramatta. Within a gently undulating topography, the site is located mid-slope on a hill which falls towards the north-west at about 3°. For descriptive purpose, Church Street has been assumed to be oriented north-south.

At the time of this investigation, the site comprised several industrial type buildings which were being used as a Bridgestone Auto & Tyre Workshop, and the Service Department, New Car Delivery/Storage, Pick-Up Yard of West End Mazda. The external ground surface was predominantly AC or concrete paved apart from the vegetated strip (tree and shrubs) along the eastern site boundary which has a frontage to Church Street.

The Bridgestone Workshop and New Car Delivery Building were located in the western one third of the subject site. The buildings comprised single storey and two-storey concrete block buildings with single storey car workshops attached.

The Service Department of West End Mazda is centrally located within the site and it comprised a two-storey rendered and clad (northern part) admin area and single storey brick workshop (southern part). Due to the topography mentioned above, the southern part of the Service Department (main workshop) appeared to be built into an excavation towards Barney Street, with a ramp located at the southern gate of the workshop to provide access off Barney Street, while the northern gate facing Ferris Street was at about street level. Also, there was an independent lower ground level basement partially underlying the Mazda workshop behind the Bridgestone shop.

Immediately to the east of the Service Department was the Pick-Up Yard. The Yard was retained by the brick wall of the eastern building and the surface level of the yard appeared to be about 3m higher than the basement level when viewed inside the main workshop.

All the buildings above-mentioned were in reasonable external condition based on a cursory inspection. The buildings on site were set back about 1.3m to 8m from the site boundaries. The ground levels were similar across site boundaries.

The site was bounded by Ferris Street, Church Street and Barney Street to the north, west and south respectively. To the east of the site there were two properties, 9 Ferris Street and 14 Barney Street. Both were single storey brick buildings and in reasonable external condition. The ground surface levels were similar across the site boundary.

4.2 Geology and Subsurface Conditions

The 1:100,000 series geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates that the site is underlain by Hawkesbury Sandstone.

In summary all boreholes encountered fill, which extended to depths ranging from 0.2m to 2.2m. Apart from BH6 and BH7, where shallow sandstone bedrock was encountered directly beneath fill, the other boreholes (BH1 to BH5) encountered natural alluvial soil, then followed by residual soil overlying either extremely weathered bedrock or competent quality bedrock. Reference should be made to the attached borehole logs for details at each specific location. A summary of the encountered subsurface characteristics is provided below:

Pavements

Either AC pavement or concrete pavement were encountered in all borehole locations. The AC pavement (in BH1, BH2, BH3, BH4, BH7) comprised approximately 50mm thick AC overlying 50mm thick gravelly road base. The concrete encountered in BH5 and BH6 was about 100mm to 120mm thick respectively, with no observed reinforcement.

Fill

Fill comprised both sandy and clayey soils with various amounts of gravel and extended to depths ranging from 0.2m to 2.2m below ground surface level. Inclusions of brick fragments and slag were observed in the fill. Based on the SPT test results and limited hand penetrometer readings, the fill was assessed to be poorly to moderately compacted.

Alluvial Soil

Alluvial soil was encountered in BH1 to BH5 beneath the fill and extended to depths ranging from 2m to 3.6m below ground surface level. The alluvial soils mainly comprised clayey sand and sandy clay with inclusions of sub-rounded ironstone gravel and ash. The relative density / strength of the alluvial soil was initially very loose to loose for clayey sands and firm for the sandy clays in the upper alluvial profile, improving with depth to stiff for the sandy clay, and loose to medium dense for the clayey sand, in the lower alluvial profile.

Residual Soil

Residual soil comprising silty clay was encountered below the alluvial soils in BH1, BH2, BH3 and BH4. The residual soil profile extended to the top of the weathered sandstone bedrock at depths varying from 2.75m to 4.5m. The strength of the residual soil ranged from firm to very stiff.

Weathered Sandstone Bedrock

Extremely weathered bedrock was initially encountered beneath the residual soils in BH3, BH4 and beneath the alluvial soils in BH5. Extremely weathered bedrock comprised hard/dense ‘soil’ strength to very low strength rock. In very general terms the rock levels through the larger part of the site are similar but the surface falls away in the western/north-western area.

Better quality sandstone bedrock was encountered in all boreholes.

- Boreholes BH5 and BH6 encountered moderately to slightly weathered sandstone up to high strength over the full length of coring.
- Boreholes BH1, BH2, BH4 and BH7 initially encountered moderately weathered sandstone bedrock of low to medium strength with various amount of extremely weathered bands. However, the rock strength shortly improved to consistently high strength with depth.
- An additional coring run was made in BH3 since the first run encountered weak bedrock comprising mainly extremely weathered sandstone and a 100mm thick no-core zone. However, within the second core run, the strength improved to consistently high from 5.2m below ground surface level.

The cored sandstone contained some defects, including inclined joints, clay /extremely weathered seams and a weakly cemented sand / iron deposit coated bedding partings.

The following table summarises the rock levels and rock classification in accordance with Foundations on Shale and Sandstone by Pells et al 1998.

Borehole	**Approx. Surface RL (m) AHD**	**Depth (m)/ RL Class V**	**Depth (m)/ RL Class III or better**
BH1	18.2m	-	4.7m/RL13.5m
BH2	20.5m	4.2m /RL16.3m	4.9m/RL15.6m
BH3	21.4m	2.8m/RL18.6m	4.8m/RL16.6m
BH4	22.7m	4m/RL18.7m	5.3m/RL17.4m
BH5	22.5m	3.6m/RL18.9m	4.3m/RL18.2m
BH6	21.3m	-	2.1m/RL19.2m
BH7	20m	-	1.8m/RL18.2m

Groundwater

Groundwater seepage was encountered during auger drilling of BH1 and BH2 at 2.5m depth. Once coring was commenced water is introduced which obscures the true groundwater level. Groundwater monitoring wells were installed BH3 and BH4 to depths of 5.8m and 6.1m, respectively. The groundwater monitoring wells were allowed to stabilise over several days and return visits were made to site on the 23 September 2020 and groundwater was measured at depths of 2.78m (RL18.62m) in BH3 and 2.95m (RL19.75m) in BH4.

4.3 Laboratory Test Results

Based on the Atterberg limits and linear shrinkage test results, the silty clays tested are of medium plasticity and are assessed to have a moderate potential for shrink/swell movements with changes in moisture contents.

The moisture content and point load strength index results showed reasonably good correlation with our field assessment of rock strength. The Unconfined Compressive Strength (UCS) of the rock core, estimated from the point load strength index test results, generally ranged from 2MPa to 40MPa with some higher results of up to 52MPa.

The four-day soaked CBR test on samples of the clayey fill and sandy fill from BH4 and BH5 compacted to 98% of their Standard Maximum Dry Density (SMDD) gave results of 9% and 30%, respectively.

The pH values on samples of the alluvial and residual soils ranged from 4.3 to 7.2, indicating slightly acidic to neutral soil conditions. The sulphate contents ranged from 10mg/kg to 110mg/kg, the chloride contents ranging from <10mg/kg to 10mg/kg, and the resistivity ranged from 110,000ohm.cm to 38,000ohm.cm. Based on these results, the alluvial and residual soils would classified as 'mild' exposure classification for concrete piles in accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation' and 'non-aggressive' exposure classification for steel piles in accordance with Table 6.5.2(C) of AS2159-2009.

5 COMMENTS AND RECOMMENDATIONS

5.1 Excavation & Groundwater

Prior to the start of excavation, we recommend that dilapidation surveys be completed on structures located within a horizontal distance from the excavation perimeter of at least twice the excavation depth. The dilapidation surveys should comprise detailed inspections of the adjoining buildings, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The respective owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The preparation of the dilapidation reports will also help to guard against opportunistic claims for damage that was present prior to the start of excavation.

Excavation to the required depths of about 3.6m within the southern portion of the site will encounter fill, alluvial clays and sands, residual silty clay and a limited thickness of weathered sandstone bedrock ranging in strength from very low to high.

Excavation of the soils will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Some of the upper weathered siltstone may also be able to be excavated using such equipment.

Excavation of the rock of low strength or higher strength will require assistance using rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws. It may be found that such rock excavation equipment will be required to break through bands of higher strength rock and then the weaker bands being able to be removed using the excavator bucket.

Hydraulic rock hammers must be used with care due to the risk of damage to the adjacent structures from the vibrations generated by such equipment. If hydraulic rock hammers are used the vibrations transmitted to the adjoining properties to the east should be quantitatively monitored at all times during rock hammer use. The monitors should be attached to flashing warning lights, or other suitable devices, to warn the operator when acceptable limits have been reached so that excavation works can cease. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are excessive it would be necessary to change to alternative excavation equipment, such as smaller rock breakers, ripping hooks, rotary grinders or rock saws. A rock saw could be used to cut a slot around the perimeter of the excavation prior to the use of a hydraulic hammer to break the rock from between the saw cuts in order to limit the transmitted vibrations. However, the effectiveness of this must be confirmed by the results of vibration monitoring.

Groundwater was encountered within the wells installed in BH3 and BH4 at levels between RL18.62m and RL19.75m, which is close to the proposed lower ground floor level at RL19.1m. Due to the variability in levels within the wells we expect that the groundwater measured comprises seepage flowing above and through the weathered rock and collecting within the wells. Therefore, during construction we expect that any seepage that does occur within the excavation may occur at various locations within the site and may emerge at variable depths within the rock profile. The seepage would tend to occur along the soil/rock interface and through bedding partings and joints within the rock profile and diminish with time, though becoming greater during wet weather periods.

During construction any such seepage that does occur should be able to be controlled using conventional sump and pumps techniques.

In the long term, drainage should be provided behind all retaining walls and possibly below the basement slab. The completed excavation should also be inspected by the hydraulic consultant to confirm that the designed drainage system is adequate for the actual seepage flows.

5.2 Subgrade Preparation

Following completion of the bulk excavation some relatively minor areas of fill will still be present below the proposed Lower Ground Floor Level. We are unaware of any records of placement or compaction control for the fill and as such it must be considered “uncontrolled”. Therefore, we consider that the fill is unsuitable to support floor slabs and should be stripped back to the underlying natural soils.

Where on-grade floor slabs are proposed we recommend that where the fill is not excavated as part of the bulk excavations that it be removed and replaced with controlled, engineered fill. Alternatively, if the fill is left in place the ground floor slab should be designed as a fully suspended slab supported on the piled footing system. For the proposed pavements the fill may be left in place provided it is treated as required following proof rolling.

Within areas where floor slabs are proposed all existing fill should be fully stripped to expose the natural alluvial or residual soils. Within pavement areas the vegetation and root affected soils should be stripped, but any clean fill below may be left in place subject to proof rolling. This root affected fill is not suitable to reuse as engineered fill, but may be reused within landscaped areas.

Following stripping, the exposed subgrade should be proof rolled with at least 7 passes of a minimum 10 tonne dead weight, smooth drum, vibratory roller. The final pass of the proof rolling should be carried out without vibration and in the presence of a geotechnical engineer to detect any weak subgrades areas. Care must be taken during rolling due to the risk of damage to adjoining structures from the vibrations generated by the roller. If vibrations are of concern the rolling may need to be carried out with a static roller only. Any weak or unstable areas detected during proof rolling should be locally excavated to a sound base and the excavated material replaced with controlled, engineered fill, or as directed by the geotechnical engineer during proof rolling. Some weak subgrade areas may be experienced where the existing fill is left in place or where the clays are allowed to soften due to water ponding. Following treatment of weak areas, engineered fill should be placed in thin layers as recommended in Section 5.2.1 below.

In view of the moderate reactivity potential of some of the clays, particular attention should be given to providing adequate drainage both during construction and for long term site maintenance. The principal aim of the drainage should be to promote run-off and reduce ponding. Placement of a blinding layer of durable granular fill or subbase material to provide a trafficable surface during construction may be necessary or desirable. The earthworks should be carefully planned and scheduled to maintain cross-falls during construction. If the clay is exposed to prolonged periods of rainfall, softening will result and site trafficability will be poor. If soil softening occurs, the subgrade should be over-excavated to below the depth of moisture softening and the excavated material replaced with engineered fill.

5.3 Engineered Fill and Compaction Control

Engineered fill should preferably comprise well graded granular materials, such as ripped rock or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.

The excavated material may be reused as engineered fill, provided it is moisture conditioned, free of deleterious materials and particles greater than 75mm in size. All excavated material should be inspected and approved by a geotechnical engineer prior to reuse. Any clay fill should be compacted in maximum 200mm loose thickness layers to a density strictly between 98% and 102% of SMDD and at moisture contents within 2% of Standard Optimum Moisture Content (SOMC).

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m^2 or three tests per visit, whichever requires the most tests. Where fill is to support footing loads it should be placed under Level 1 control as defined by AS3798-2007. Preferably the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

5.4 Shoring and Slope Batters

Given the space along the sides of the excavation, temporary batters may be used through the clayey soils and poor-quality sandstone bedrock. Such batters may be formed at no steeper than 1 Vertical (V): 1 Horizontal (H). Where adopted all surcharge loads such as stockpiles, traffic loads etc must be kept well clear of the crest of the batters (ie below a 45° line drawn upwards from the toe of the batter) Where permanent batters are adopted they should be formed at no steeper than 1Vertical (V) : 2 Horizontal (H) and should be protected from erosion by vegetation, shotcrete and mesh or similar. For maintenance purposes it may be more practical to from permanent batters at no stepper than 1V:3H or 4H.

Free-standing retaining walls constructed at the base of the batters may be designed as cantilevered walls based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.33 and a bulk unit weight of 20kN/m^3. Where walls are restrained from some lateral movements, such as by other structural elements in front of the wall, or where movements are to be kept low, an 'at rest' earth pressure coefficient, K_0, of 0.6 should be used.

The use of temporary batters would require the removal of material from the site to form batters and then the importation of material for backfilling once permanent walls have been constructed and it may be more cost effective to install a shoring system along the southern, eastern and western sides to excavation. Such a retention system may comprise soldier pile walls with shotcrete infill panels. Bored piers would be appropriate for the piled walls, but some groundwater seepage may be encountered requiring the use of pumps and tremie concreting techniques. The piers should be founded at least 1m below the base of the excavation, including excavations for footings and services, but more as required for stability design. The seepage may also affect the stability of the soil exposed between piles prior to shotcreting, in which case additional temporary works would be required such as timber lagging or steel sheeting between piles or even local sandbagging.

Walls retaining more than about 3m will require additional lateral support in the form of external anchors or internal props, which must be installed progressively as each restraining point is uncovered. Where anchors extend below adjoining properties permission will need to be obtained from the owners of the adjoining properties before installation of the anchors.

Propped or anchored retaining walls may be designed based on a trapezoidal earth pressure distribution of magnitude 6H kPa (where H is the retained height in metres) where some resulting ground movements are tolerable and existing structures are located beyond a horizontal distance of 2H from the wall. Where movements are to be kept low and structures are located within a horizontal distance of 2H from the wall, a trapezoidal earth pressure distribution of 8H kPa should be used. These lateral pressures should be held constant for the central 50% of the pressure distribution.

The above coefficients and lateral pressures assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads must be allowed for in the design, plus full hydrostatic pressures unless measures are undertaken to provide complete and permanent drainage behind the wall.

Anchors should have their bond formed within rock of at least low strength, with the bond formed beyond a line drawn up at 45° from the base of the excavation. Preliminary design of anchors may be based on an allowable bond stress of 200kPa for rock of low strength or 300kPa for rock of medium strength. All anchors should be proof loaded to at least 1.3 times the 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. Anchors are generally carried out on a design and construct basis so that failure of the anchors to hold their test load does not become a contractual issue.

Where batters are used, the space between the batters and the permanent retaining walls will need to be carefully backfilled to reduce future settlement of the backfill. Only light compaction equipment should be used for compaction behind retaining walls so that excessive lateral pressures are not placed on the walls. This will require the backfill to be placed in thin layers, say 100mm loose thickness, appropriate to the compaction equipment being used. The excavated clay will be difficult to properly compact within the limited space available behind the walls and consideration should be given to the use of more readily compactable materials, such as ripped or crushed rock or gravel. The compaction specification for the backfill will depend on whether paving or structures are to be supported on the fill. If the fill is to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular fill materials, but if it is only to support landscaped areas a lower compaction specification, say 95% of SMDD, may be appropriate, provided the risk of future settlement and maintenance can be accepted. If clay fill is to be used a greater control of fill compaction and moisture control will be required and further geotechnical advice on the use of such material should be obtained. An alternative for backfill would also be to use a uniform granular material, such as crushed concrete of 30mm to 70mm in size, surrounded in a geofabric, as this requires only nominal compaction. A capping layer of clay should be used to reduce infiltration behind the wall where pavements will not cover the surface.

5.5 Footings and Site Classification

5.5.1 Site Classification

Classification of the site is in accordance with AS2870-2011 is not directly relevant for the proposed development, but has been included herein as a guide to the reactivity. Due to the existing fill the site in its current state would be classified as a Class P in accordance with AS2870-2011. However, considering the reactivity potential of the alluvial soils and residual clays we would expect movements similar to Class M range, as defined by AS2870-2011 in the absence of modifying factors. As the soil profile is quite variable and parts of the site will be excavated to rock, differential movements are a bigger consideration than shrink-swell per se.

The designer needs to consider the extent of any cut and fill earthworks and the likely extent of shrink-swell movements when completing the footing design. We note that the proposed development is of a size extent that is strictly outside the scope of AS2870-2011. Therefore, standard footing designs may not be relevant and footing design must be carried out using sound engineering principles.

5.5.2 Footings

Given the size of the proposed structure and relatively shallow depth to sandstone bedrock below the bulk excavation level we recommend that it be entirely supported on footings founded within rock to provide uniform support and reduce risk of differential settlements.

Where rock is exposed or is at shallow depth of less than 1m, such as towards the southern end of the site, pad or strip footings may be used. Where the depth of rock is more than about 1m bored piers would be more practical. It should be noted that seepage occurs above rock level in the soils and that some difficulty with water inflow and stability may be experienced with bored piers particularly towards the north-western corner of the site. Casing piers to control seepage and instability is an option but to quantify the extent of the problem we recommend completing some trial piers during the demolition period. If piers are found to be too troublesome then CFA piles would be a more expensive but easier option. Steel screw piles could be considered but would have limited bearing capacity due to their inability to socket into the bedrock. If any above ground portions of the building extend outside of the basement footprint these portions should be supported on piles founded within the rock below a line drawn up at 45° from the base of the excavation so that additional loads are not placed on the basement walls, unless the walls have been designed for such loads.

The rock classification table in Section 4.2 above provides depths and levels for the rock class encountered within each borehole. For design of piles and pad/strip footings, we consider that the Class V sandstone bedrock is adequate for an allowable bearing pressure (ABP) of 700kPa and the Class III sandstone bedrock is adequate for an allowable bearing pressure of 3,500kPa.

Where footings are designed for an allowable bearing pressure of 700kPa, at least the initial stages of pile drilling/footing excavation should be inspected by a geotechnical engineer. Where the higher bearing pressures of 3500kPa are adopted the drilling of all piles should be inspected by a geotechnical engineer to confirm that appropriate quality roc has been encountered.

Where piles are used, allowable shaft adhesions equivalent to 10% of the allowable end bearing pressure may be used for the design of piles in compression, below a nominal 0.3m socket and provided socket roughness and cleanliness is maintained.

5.6 Floor Slabs & Pavements

As discussed in the sections above, we expect the proposed basement level to be underlain by a mixture of fill, sand and clay soils and bedrock. Subgrade preparation in accordance with the guidelines in Section 5.2 is essential. As the clays will be affected by seepage water, irrespective of whether it rains, a crushed rock or crushed concrete working platform will be required; a nominal thickness of 300mm is likely to be enough to allow pier boring rigs access to drill the footing piers though a thicker layer might be needed where any non-cohesive soils may occur.

We recommend that underfloor drainage be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The underfloor drainage should direct the collected seepage to a sump containing an automatic level control pump to avoid flooding of the basement level. The wall drains should be connected into the underfloor drainage system. A complete drainage blanket may be adopted over bedrock or, alternatively and elsewhere, a network of drains at centres not greater than (say) 6m - 8m may be provisionally adopted. Permission for discharge of the drainage system will need to be obtained from the relevant authorities.

On-grade floor slabs should be separated from all walls, columns, footings, etc., to permit relative movements (i.e. designed as 'floating' slabs).

The design of new 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 such as truck turning and manoeuvring. Flexible pavements may have a lower initial cost but maintenance will be higher. These factors should be considered when making the final decision on pavements. The subgrade below pavements should be carefully prepared in accordance with the recommendations given in Section 5.2.

The subgrade is going to be very variable as there is so much fill and variable alluvial soils at the site, as shown by the range of CBR values of 7% and 30%. After stripping and subgrade preparation an assessment will have to be made to decide what is appropriate, but provisionally a moderate CBR value of, say, 5% could be used for the sandy clay/clayey sands. Any areas of more plastic clay may have to be capped with about 300mm of good quality sandstone fill. For concrete or rigid pavement design, an equivalent modulus of subgrade reaction of 35kPa/mm (300mm plate) may be adopted.

For flexible pavements, in-situ lime stabilisation of the clayey subgrade could be undertaken to reduce total pavement thickness but the area involved is too small for this to be economical. A select fill layer comprising good quality well-graded granular material such as sandstone is more likely to be appropriate.

Concrete pavements should be provided with effective shear connection at joints by using dowels or keys. Concrete pavements subject to traffic loadings should be supported on a sub-base layer of RTA Specification 3051 unbound or equivalent good quality crushed rock, compacted to a density of at least 100% SMDD.

Subsoil drains could be provided on the uphill side and along the perimeter of external pavements, with inverts not less than 0.3m below clay subgrade level. Drainage trenches should be excavated with a longitudinal fall to appropriate discharge points so as to minimise the risk of water ponding. The pavement subgrade should be graded to promote water flow or infiltration towards subsoil drains.

5.7 Earthquake Design

The site classification in accordance with AS1170.4 *Earthquake Actions in Australia,* is governed by conditions at the northern and north-western margins of the site where the soil profile is in excess of 3m depth and therefore the site is Class C_e – shallow soil site.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

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

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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North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMIT AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Commercial Development
Location: 574-584 Church Street, Parramatta, NSW

Ref No: 33532S
Report: A
Report Date: 1/10/2020
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	2.80 - 3.15	12.9	36	14	22	8.5
1	4.50 - 4.60	6.9	-	-	-	-
2	1.60 - 1.95	12.7	25	12	13	7.5
3	2.70 - 2.75	12.2	-	-	-	-
4	0.20 - 1.00	14.0	-	-	-	-
4	0.60 - 0.95	13.5	35	13	22	10.0
4	4.20 - 4.50	10.5	-	-	-	-
5	0.10 - 1.60	12.1	-	-	-	-

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 23/09/2020.
- Sampled and supplied by client. Samples tested as received.

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01/10/2020

Authorised Signature / Date
(D. Treweek)

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33532S
Project:	Proposed Commercial Development	**Report:**	B
Location:	574-584 Church Street, Parramatta, NSW	**Report Date:**	30/09/2020
		Page 1 of 1	

BOREHOLE NUMBER	BH 4	BH 5
DEPTH (m)	0.20 - 1.00	0.10 - 1.60
Surcharge (kg)	4.5	4.5
Maximum Dry Density (t/m^3)	1.94 STD	1.96 STD
Optimum Moisture Content (%)	13.1	11.3
Moulded Dry Density (t/m^3)	1.89	1.92
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	104	101
Moisture Contents		
Insitu (%)	14.0	12.1
Moulded (%)	13.6	11.4
After soaking and		
After Test, Top 30mm(%)	16.1	13.1
Remaining Depth (%)	13.3	12.8
Material Retained on 19mm Sieve (%)	0	2*
Swell (%)	0.0	0.0
C.B.R. value: @2.5mm penetration	9	
@5.0mm penetration		30

NOTES: Sampled and supplied by client. Samples tested as received.
- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 23/09/2020.
- * Denotes not used in test sample.
- BH 4 dried back prior to testing.

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30/09/2020
Authorised Signature / Date
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SOIL TEST SERVICES

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33532S
Project:	Proposed Commercial Development	**Report:**	C
Location:	574-584 Church Street, Parramatta, NSW	**Report Date:**	23/09/2020
		Page 1 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	4.83 - 4.87	0.8	16
	5.20 - 5.24	1.7	34
	5.63 - 5.67	1.2	24
	6.14 - 6.18	0.5	10
	6.79 - 6.83	1.9	38
	7.11 - 7.14	1.4	28
	7.47 - 7.51	1.0	20
2	4.85 - 4.89	0.7	14
	5.15 - 5.20	1.1	22
	5.74 - 5.78	0.9	18
	6.34 - 6.38	1.1	22
	6.80 - 6.84	1.3	26
	7.24 - 7.29	2.0	40
3	3.10 - 3.15	0.2	4
	3.73 - 3.77	0.1	2
	4.24 - 4.28	1.3	26
	4.74 - 4.77	0.6	12
	5.20 - 5.24	1.4	28
	5.77 - 5.82	1.8	36
	6.18 - 6.21	1.5	30
	6.63 - 6.68	1.5	30
	7.26 - 7.30	1.4	28
	7.75 - 7.78	1.6	32
4	5.17 - 5.20	0.5	10
	5.70 - 5.73	2.6	52

NOTES: See Page 2 of 2

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Commercial Development
Location: 574-584 Church Street, Parramatta, NSW

Ref No: 33532S
Report: C
Report Date: 23/09/2020
Page 2 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
4	6.21 - 6.24	1.4	28
	6.85 - 6.89	1.4	28
	7.11 - 7.15	1.1	22
	7.56 - 7.61	1.2	24
5	4.45 - 4.48	1.8	36
	4.81 - 4.85	1.1	22
	5.17 - 5.21	1.5	30
	5.74 - 5.78	1.6	32
	6.14 - 6.17	1.9	38
	6.71 - 6.76	1.3	26
6	2.20 - 2.24	2.0	40
	2.70 - 2.75	1.1	22
	3.28 - 3.33	1.2	24
	3.71 - 3.75	1.4	28
	4.27 - 4.32	1.6	32
	4.66 - 4.70	1.2	24
7	1.10 - 1.15	0.2	4
	1.79 - 1.83	0.7	14
	2.17 - 2.21	1.3	26
	2.69 - 2.73	1.4	28
	3.30 - 3.35	1.0	20
	3.67 - 3.70	1.2	24
	4.22 - 4.26	1.7	34

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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CERTIFICATE OF ANALYSIS 251973

Client Details

Client	JK Geotechnics
Attention	Bryan Zheng
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**33532S, Parramatta**
Number of Samples	3 Soil
Date samples received	24/09/2020
Date completed instructions received	24/09/2020

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	01/10/2020
Date of Issue	30/09/2020

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Misc Inorg - Soil				
Our Reference		251973-1	251973-2	251973-3
Your Reference	UNITS	BH5	BH4	BH2
Depth		1.6-1.95	2.7-3.15	2.9-3.15
Date Sampled		17/09/2020	17/09/2020	16/09/2020
Type of sample		Soil	Soil	Soil
Date prepared	-	25/09/2020	25/09/2020	25/09/2020
Date analysed	-	25/09/2020	25/09/2020	25/09/2020
pH 1:5 soil:water	pH Units	7.2	4.3	5.1
Chloride, Cl 1:5 soil:water	mg/kg	<10	10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	110	42	10
Resistivity in soil*	ohm m	110	210	380

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-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 (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

Client Reference: 33532S, Parramatta

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			25/09/2020	[NT]	[NT]	[NT]	[NT]	25/09/2020	[NT]
Date analysed	-			25/09/2020	[NT]	[NT]	[NT]	[NT]	25/09/2020	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	99	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	102	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

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.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

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: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-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.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

pH/EC

Samples were out of the recommended holding time for this analysis.

BOREHOLE LOG

Borehole No.
1
1 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Method: SPIRAL AUGER
R.L. Surface: ~18.2 m
Date: 16/9/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: B.Z./T.C.

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 OF AUGERING						18			-	ASPHALTIC CONCRETE: 55mm.t over ROADBASE: 50mm.t.	M			APPEARS POORLY COMPACTED
										FILL: Silty sand, fine to medium grained, dark brown, and clay, with fine to coarse grained igneous gravel.	w>PL			
					N = 4 3,1,3	17	1			FILL: Silty sandy clay, low plasticity, trace of fine to coarse grained igneous gravel and fragments of slag.			110 140 150	
					N=0 1,0,0	16	2		SC	Silty clayey SAND: fine to medium grained, grey and brown.	M	VL	110 80 40	ALLUVIAL
▶									CI	Silty sandy CLAY: medium plasticity, grey, fine to medium grained sand.	w>PL	(St)		
					N = 21 5,11,10	15	3			Silty CLAY: medium plasticity, red brown, orange brown and grey, with fine to medium grained ironstone gravel.		VSt	210 220 300	RESIDUAL
					N > 7 5,7/ 130mm REFUSAL	14	4			as above, but with fine to medium grained sand.				SPT SAMPLE TOO FRIABLE TO CARRY OUT HAND PENETROMETER
									-	SANDSTONE: fine to medium grained, brown.	MW	M		HAWKESBURY SANDSTONE MODERATE 'TC' BIT RESISTANCE
						13 12	5 6			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
1
2 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Core Size: NMLC
R.L. Surface: ~18.2 m
Date: 16/9/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: B.Z./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
		14			START CORING AT 4.70m						
100% RETURN		13	5		SANDSTONE: fine to coarse grained, red brown and light brown, bedded at 10°.	MW - SW	M - H	0.80 1.7 1.2		(5.27m) CS, 10°, Un, R, Clay, 12 mm.t (5.67m) CS, 8°, Un, Vr, 8 mm.t, gravelly Clay (5.77m) CS, 14°, Un, Vr, 8 mm.t, gravelly Clay	Hawkesbury Sandstone
		12 11	6 7		as above, but red brown and brown, bedded at 20°.			0.50 1.9 1.4 1.0	600 200 60 20	(6.55m) XWS, 20°, Vr, Clay, 90 mm.t, HP reading: 250,300,300	
		10 9 8	8 9 10		END OF BOREHOLE AT 7.72 m				600 200 60 20		

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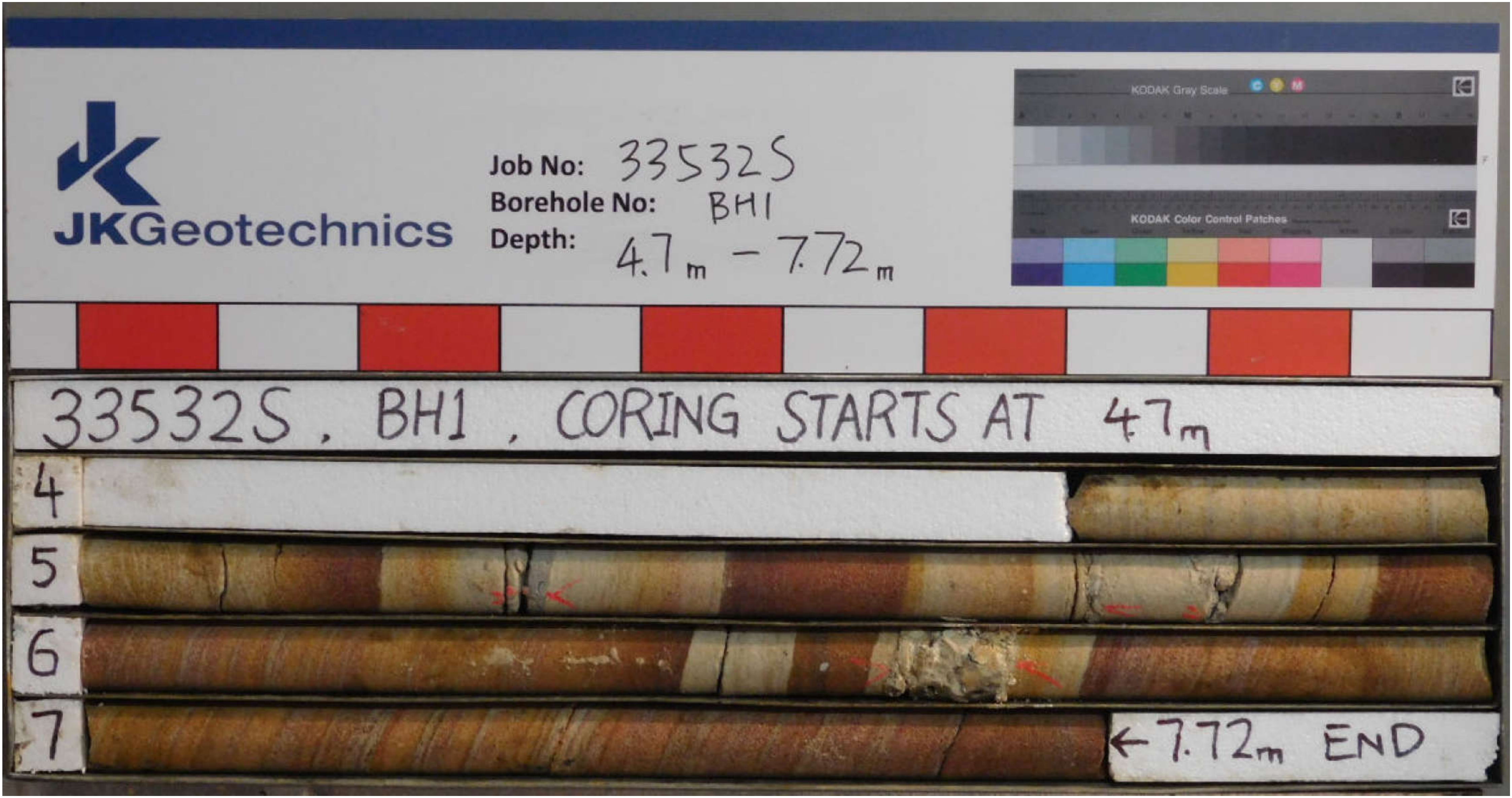
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Job No: 33532S
Borehole No: BH1
Depth: 4.7m – 7.72m

33532S , BH1 , CORING STARTS AT 4.7m

←7.72m END

JKGeotechnics

BOREHOLE LOG

Borehole No.
2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Method: SPIRAL AUGER
R.L. Surface: ~20.5 m
Date: 16/9/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: B.Z./T.C.

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 OF AUGERING										ASPHALTIC CONCRETE: 50mm.t				
									-	over ROADBASE: 50mm.t.	w<PL			APPEARS POORLY TO MODERATELY COMPACTED
					N = 12 5,6,6					FILL: Sand and Silty clay mixture, medium plasticity clay, fine to medium grained sand, dark brown and grey, trace of fine to medium grained, sub angular igneous gravel and slag.			380 300	
										FILL: Sandy silty clay, medium plasticity, dark brown mottled red brown, trace of fine to medium grained sub angular sandstone gravel, slag and ash.				
					N = 8 4,4,4					FILL: Clayey silty sand, fine to medium grained, brown mottled dark grey and red brown, trace of slag.	M			
									SC	Clayey silty SAND: fine to medium grained, brown and grey, trace of ash.	M	L		ALLUVIAL
					N = 7 3,3,4					as above, but strong hydrocarbon odour.				
									CI	Silty sandy CLAY: medium plasticity, brown and red brown mottled grey, strong hydrocarbon odour.	w>PL	(F - St)		RESIDUAL
					N=SPT 2/ 10mm REFUSAL				-	SANDSTONE: fine to medium grained, brown and red brown.	MW	M		HAWKESBURY SANDSTONE MODERATE 'TC' BIT RESISTANCE
										REFER TO CORED BOREHOLE LOG				

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33532S PARRAMATTA.GPJ <<DrawingFile>> 16/10/2020 16:42 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

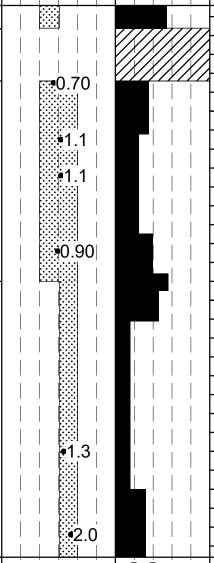
CORED BOREHOLE LOG

Borehole No.
2
2 / 2

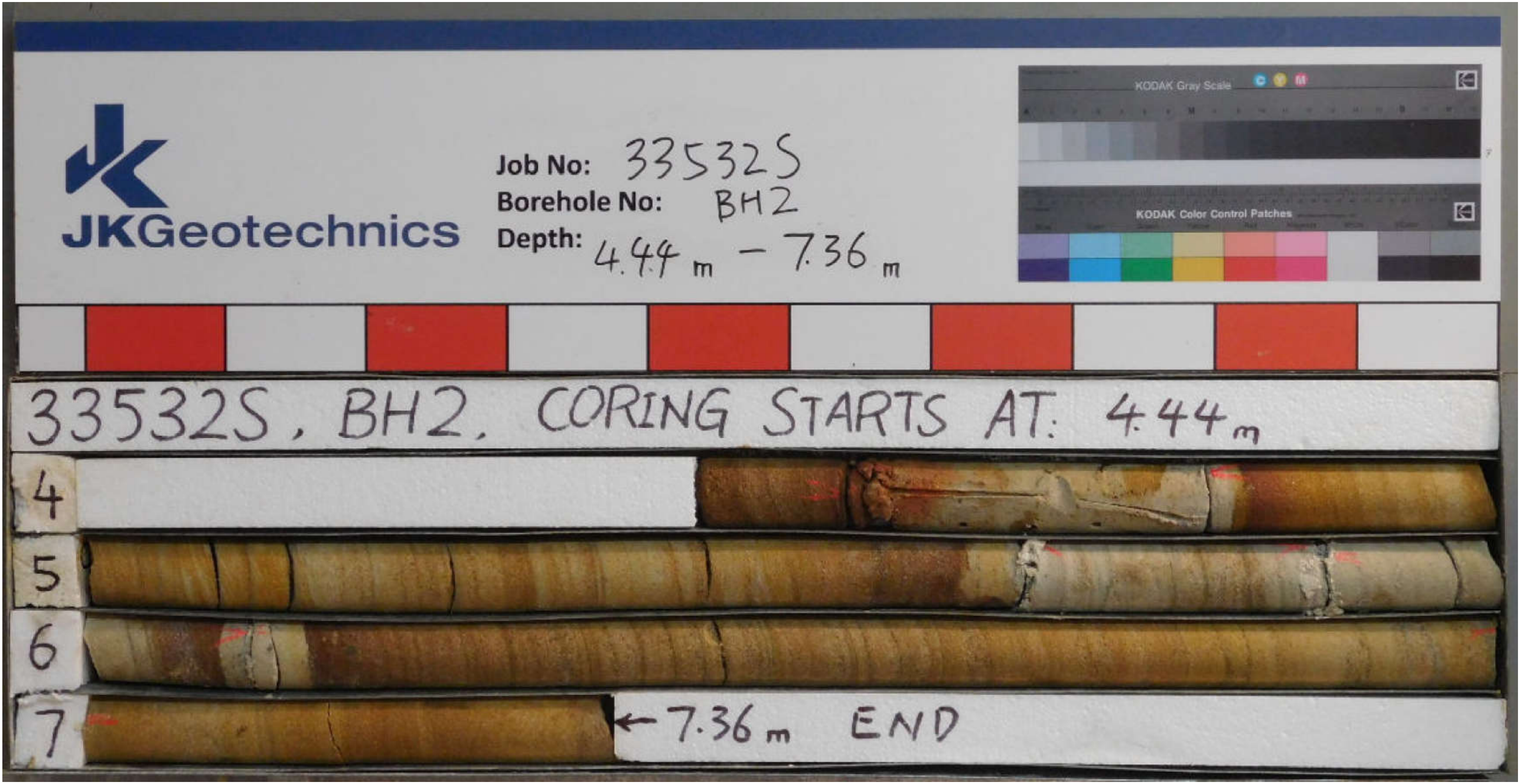
Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Core Size: NMLC
R.L. Surface: ~20.5 m
Date: 16/9/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: B.Z./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					START CORING AT 4.44m							
100% RETURN		16			SANDSTONE: fine to medium grained, red brown.	MW	M					Hawkesbury Sandstone
					Extremely Weathered sandstone: sandy CLAY, medium plasticity, light grey, fine to medium grained sand.	XW	VSt			(4.56-4.84m) HP READING 250,300,300,320		
			5		SANDSTONE: fine to medium grained, yellow brown and light brown, bedded at 20°.	MW	M - H	0.70 1.1 1.1		(5.12m) Be, 22°, P, R, Cn		
		15			as above, but light grey, bedded at 10°.			0.90		(5.65m) CS, 4°, Un, Vr, Sandy Clay, 7mm.t (5.86m) CS, 4°, Un, Vr, Sandy Clay, 10mm.t (5.95m) CS, 23°, P, R, Sandy Clay, Ct		
			6		as above, but brown, bedded at 20°.		H	1.3 2.0		(6.11m) XWS, 2°, Un, Vr, Sand, 3mmt (7.00m) XWS, 3°, Un, Vr, Sand, 2mm.t		
		14	7									
		13			END OF BOREHOLE AT 7.36 m							
		12	8									
		11	9									
		10	10									



FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Job No: 33532S
Borehole No: BH2
Depth: 4.44m – 7.36m

BOREHOLE LOG

Borehole No.
3
1 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S **Method:** SPIRAL AUGER **R.L. Surface:** ~21.4 m
Date: 16/9/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** B.Z./T.C.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■							■		ASPHALTIC CONCRETE: 50mm.t over GRAVELLY ROADBASE: 50mm.t				
	■			■	N = 5 9,3,2	21		▩	-	FILL: Silty sand, fine to medium grained, dark brown, trace of fine to coarse grained sub angular igneous gravel and slag.	M			APPEARS POORLY COMPACTED
	■						1	⁙	SM	Silty SAND: fine to medium grained, brown, with low to medium plasticity clay.	M	(L)		ALLUVIAL
				■	N = 17 3,6,11	20		⁙	SC	Clayey silty SAND: fine to medium grained, brown and red brown, low to medium plasticity clay, with fine to medium grained, sub angular ironstone gravel.		MD		
	■			■		19	2	▨	CI	Silty CLAY: medium plasticity, red brown, with fine to medium grained sand, trace of fine to medium grained sub angular ironstone gravel.	w<PL	(VSt)		RESIDUAL
				■	N=SPT 6/ 100mm REFUSAL			▨		as above, but light grey mottled yellow brown, grading into XW bedrock.		(Hd)		
						18 17 16 15	3 4 5 6			REFER TO CORED BOREHOLE LOG				Groundwater monitoring well installed to 5.8m. Class 18 machine slotted 50mm dia. PVC standpipe 5.8m to 3.0m. Casing 3.0m to 0.1m. 2mm sand filter pack 6.0m to 2.5m. Bentonite seal 2.5m to 1.5m. Backfilled with sand and cuttings to the surface. Completed with a concrete gatic cover

CORED BOREHOLE LOG

Borehole No.
3
2 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Core Size: NMLC
R.L. Surface: ~21.4 m
Date: 16/9/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: B.Z./T.C.

Column headings: Water Loss\Level | Barrel Lift | RL (m AHD) | Depth (m) | Graphic Log | CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components) | Weathering | Strength | POINT LOAD STRENGTH INDEX $I_s(50)$ (VL 0.1, L 0.3, M 1, H 3, VH 10, EH) | DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20) | DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General | Formation

RL (m AHD): 19, 18, 17, 16, 15, 14, 13
Depth (m): 3, 4, 5, 6, 7, 8

Water Loss\Level: ON COMPLETION; 100% RETURN

CORE DESCRIPTION

START CORING AT 2.75m

SANDSTONE: fine to medium grained, red brown.

Extremely Weathered sandstone: silty SAND, fine to medium grained, light grey.

SANDSTONE: fine to medium grained, light grey and red brown, with iron indurated bands.

Extremely Weathered sandstone: silty clayey SAND, fine to medium grained, light grey.

SANDSTONE: fine to coarse grained, light grey, bedded at 25°.

Extremely Weathered sandstone: silty clayey SAND, fine to medium grained, light grey.

SANDSTONE: fine to medium grained, light brown and brown, bedded at 20°.

NO CORE 0.10m

SANDSTONE: fine to medium grained, light brown and brown, bedded at 20°.

SANDSTONE: fine to coarse grained, light brown, bedded at 15°, occasional wavy carbonaceous laminae.

as above, but bedded at 20-25°.

END OF BOREHOLE AT 7.78 m

Weathering	Strength
MW	L
XW	VD
MW	L
XW	VD
MW	L
XW	VD
MW	M - H
MW	M - H
	H
MW - SW	

Point load strength index $I_s(50)$: 0.20, 0.10, 1.3, 0.60, 1.4, 1.8, 1.5, 1.5, 1.4, 1.6

DEFECT DETAILS – Specific

- (2.80m) CS, 25°, Un, Vr, Clay Ct, 17 mm.t
- (3.15m) Ji, 65°, Un, Clay Ct
- (3.50m) J, 70°, Un, FILLED, 10 mm.t, Sand and Roots
- (3.70m) J, 70°, Un, FILLED, 5 mm.t, Sand
- (3.90m) XWS, 20°, Un, Vr, 28 mm.t, Sand
- (4.37m) Jix3, 90°, Un
- (4.98m) XWS, 5°, 20 mm.t, Sand
- (5.08m) CS, 0°, Un, R, Clay, 6 mm.t
- (6.36m) Be, 4°, Un, R, Cb, 5 mm.t, Sand
- (7.22m) Be, 6°, Un, Vr, 3 mm.t, Sand
- (7.62m) Be, 25°, Un, Vr, Ct, Sand

Formation: Hawkesbury Sandstone; Hawkesbury Sandstone

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 33532S

Borehole No: BH3

Depth: 2.75m – 7.78m

33532S, BH3, CORING STARTS AT 2.75m

JKGeotechnics

BOREHOLE LOG

Borehole No.
4
1 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Method: SPIRAL AUGER
R.L. Surface: ~22.7 m
Date: 17/9/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: B.Z./T.C.

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 OF AUGERING	●									ASPHALTIC CONCRETE: 50mm.t				
									-	over GRAVELLY ROADBASE: 50mm.t	M			
	●		●						CI	FILL: Gravelly sand, fine to medium grained, dark brown, fine to coarse grained sub angular igneous gravel.	w>PL	F		ALLUVIAL
			●	●	N = 7 3,3,4	22	1			Sandy silty CLAY: medium plasticity, red brown and brown, with fine to medium grained sub angular and sub rounded ironstone gravel, and fine to medium grained sand.			80 70 80	
	●								SC	Clayey silty SAND: fine to medium grained, brown, medium plasticity clay.	M	L		
				●	N = 7 5,3,4	21	2		SM	Silty SAND: fine to medium grained, brown, with low plasticity clay.				
									CI	Silty CLAY: medium plasticity, red brown and grey, with fine to medium grained sand.	w>PL	F - St		RESIDUAL
ON COMPLETION ▽				●	N = 7 2,3,4	20	3			as above, but grey.			80 90 120	
						19	4			as above, but light grey, grading into XW bedrock.		VSt		
				●	N > 12 8,12/ 150mm REFUSAL	18			-	Extremely Weathered sandstone: Silty SAND, fine to medium grained, light grey, trace of clay, with very low strength sandstone bands and iron indurated bands.	XW	VD		HAWKESBURY SANDSTONE VERY LOW TO LOW 'TC' BIT RESISTANCE
						17 16	5 6			REFER TO CORED BOREHOLE LOG				Groundwater monitoring well installed to 6.1m. Class 18 machine slotted 50mm dia. PVC standpipe 6.1m to 3.1m. Casing 3.1m to 0.1m. 2mm sand filter pack 6.1m to 2.5m. Bentonite seal 2.5m to 1.5m. Backfilled with sand and cuttings to the surface. Completed with a concreted gatic cover

CORED BOREHOLE LOG

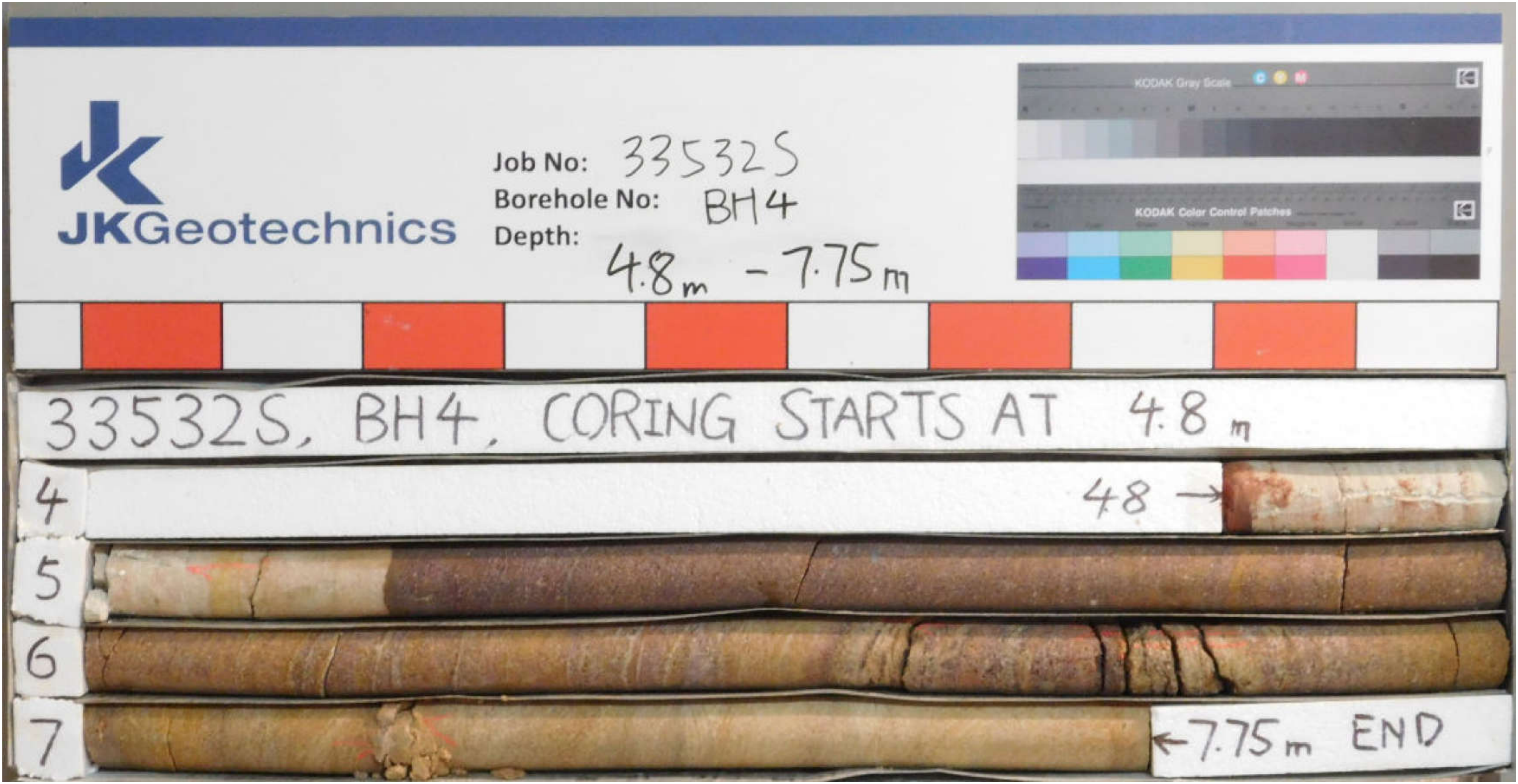
Borehole No.
4
2 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Core Size: NMLC
R.L. Surface: ~22.7 m
Date: 17/9/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: B.Z./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
		18			START CORING AT 4.80m						
100% RETURN		17	5		Extremely Weathered sandstone: clayey SAND, fine to medium grained, light grey, with very low strength sandstone bands.	XW	VD				Hawkesbury Sandstone
			6		SANDSTONE: fine to coarse grained, red brown and brown, bedded at 20°.	SW	M - H H	0.50 2.6 1.4			
		16			as above, but with fine to coarse grained quartz grains inclusion.			1.4		(6.58m) Be, 18°, Un, Vr, Fe & Qz, Cb, Sn (6.70-6.80m) Be x 4, Un, Vr, Fe & Qz, Cb, Sn	
		15	7		as above, but brown and light brown, without quartz grain inclusion.			1.1 1.2	200 60 20	(7.21m) XWS, 0°, Un, Vr, 40 mm.t, Sand	
		14	8		END OF BOREHOLE AT 7.75 m						
		13	9								
		12	10						600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


Job No: 33532S
Borehole No: BH4
Depth: 4.8m - 7.75m

JKGeotechnics

BOREHOLE LOG

Borehole No.
5
1 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Method: SPIRAL AUGER
R.L. Surface: ~22.5 m
Date: 17/9/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: B.Z./T.C.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			CONCRETE: 100mm.t.				NO REINFORCEMENT OBSERVED
	N = 12 8,6,6	-	FILL: Sand, fine to coarse grained, grey and brown, with fine to coarse grained sub angular igneous sandstone gravel and brick fragments.	M			APPEARS POORLY TO MODERATELY COMPACTED
	N = 4 2,2,2	SC	Clayey silty SAND: fine to medium grained, brown and red brown, with fine to medium grained sub angular ironstone gravel.	M	L		ALLUVIAL
	N = 7 3,3,4	CI	Sandy silty CLAY: medium plasticity, brown and red brown, fine to medium grained sand, trace of fine to medium grained ironstone gravel.	w>PL	(St)		SPT NO RECOVERY
		-	Extremely Weathered sandstone: Silty CLAY, medium plasticity, red brown and light grey, with fine to medium grained sand.	XW	(Hd)		HAWKESBURY SANDSTONE
			SANDSTONE: fine to medium grained, light grey and red brown.	HW			LOW 'TC' BIT RESISTANCE
			REFER TO CORED BOREHOLE LOG				

RL (m AHD): 22, 21, 20, 19, 18, 17, 16
Depth (m): 1, 2, 3, 4, 5, 6

SAMPLES: ES, U50, DB, DS

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33532S PARRAMATTA.GPJ <<DrawingFile>> 16/10/2020 16:43 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

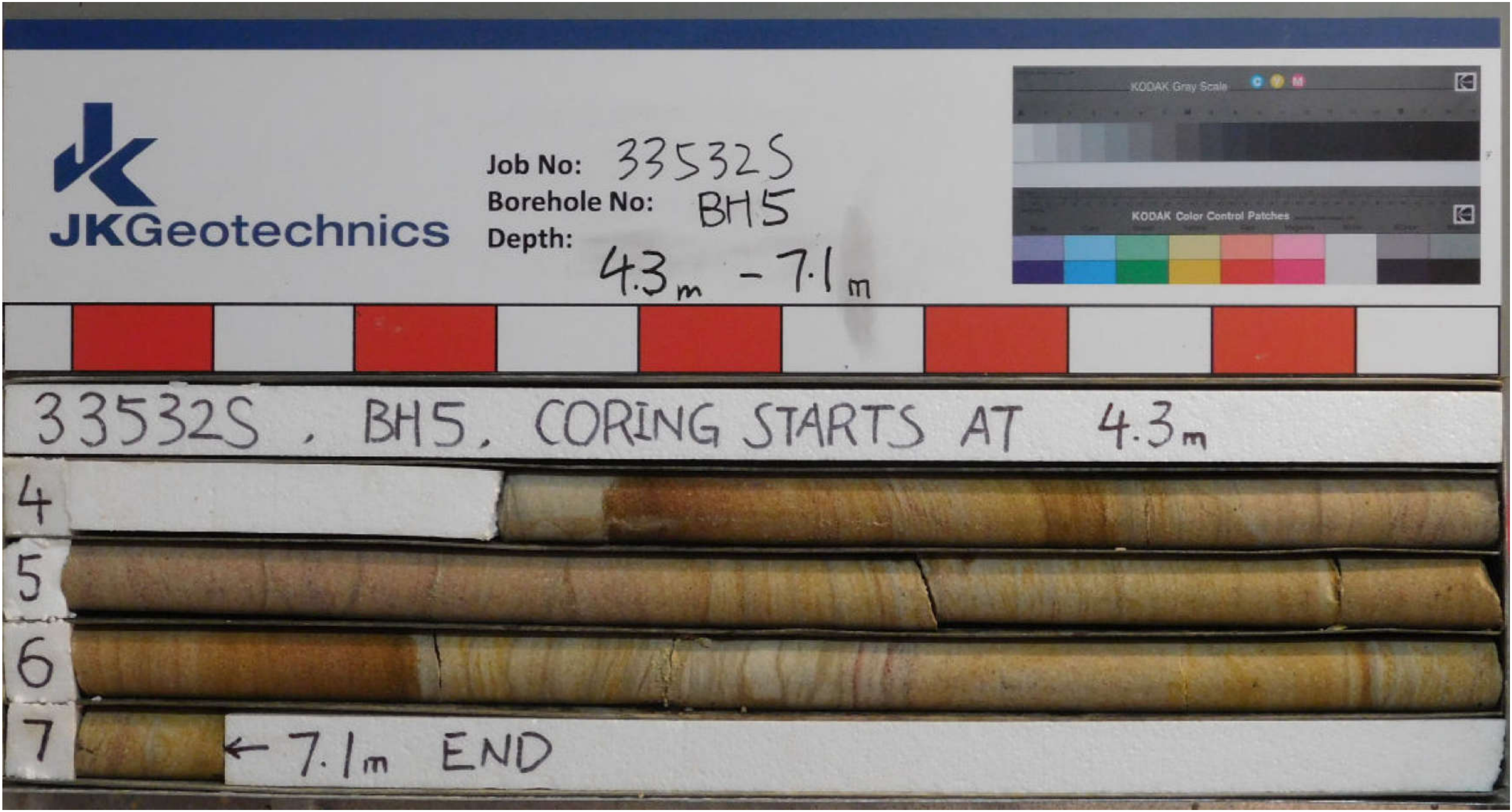
Borehole No.
5
2 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Date: 17/9/20
Plant Type: JK308
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: ~22.5 m
Datum: AHD
Logged/Checked By: B.Z./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					START CORING AT 4.30m							
100% RETURN		18 17 16	5 6 7		SANDSTONE: fine to coarse grained, brown and light brown, bedded at 10-20°.	SW	H	1.8 1.1 1.5 1.6 1.9 1.3		(6.42m) XWS, 11°, Un, Vr, Clayey sand, 5mm.t		Hawkesbury Sandstone
		15 14 13 12	8 9 10		END OF BOREHOLE AT 7.10 m				600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 33532S

Borehole No: BH5

Depth: 4.3m - 7.1m

JKGeotechnics

BOREHOLE LOG

Borehole No.
6
1 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Method: SPIRAL AUGER
R.L. Surface: ~21.3 m
Date: 18/9/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: B.Z./T.C.

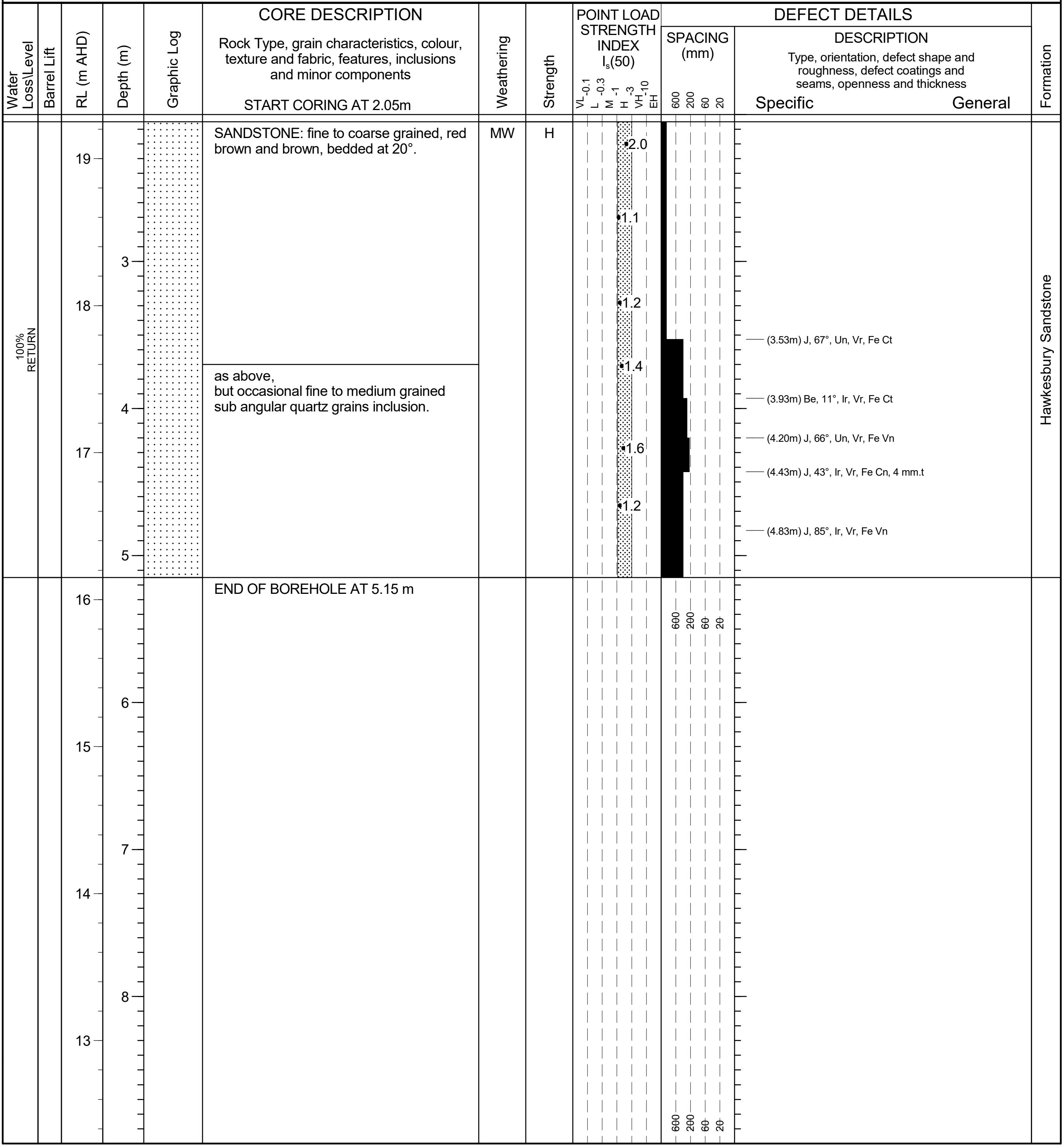
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING										CONCRETE: 120mm.t				NO OBSERVED REINFORCEMENT
						21			-	FILL: Silty sand, fine to medium grained, dark brown with sand and fine to medium grained sub angular igneous gravel.	M			
					N = 5 4,3,2	20	1			FILL: Clayey silty sand, fine to medium grained, red brown and brown, low to medium plasticity clay, with fine to medium grained sub angular cemented sand nodules.	M	L		APPEARS POORLY COMPACTED
					N = 4 2,2,2		2			FILL: Silty sand, fine to medium grained, brown, trace of clay.				
						19 18 17 16 15	3 4 5 6			REFER TO CORED BOREHOLE LOG				MODERATE TO HIGH 'TC' BIT RESISTANCE BEDROCK ENCOUNTERED

CORED BOREHOLE LOG

Borehole No.
6
2 / 2

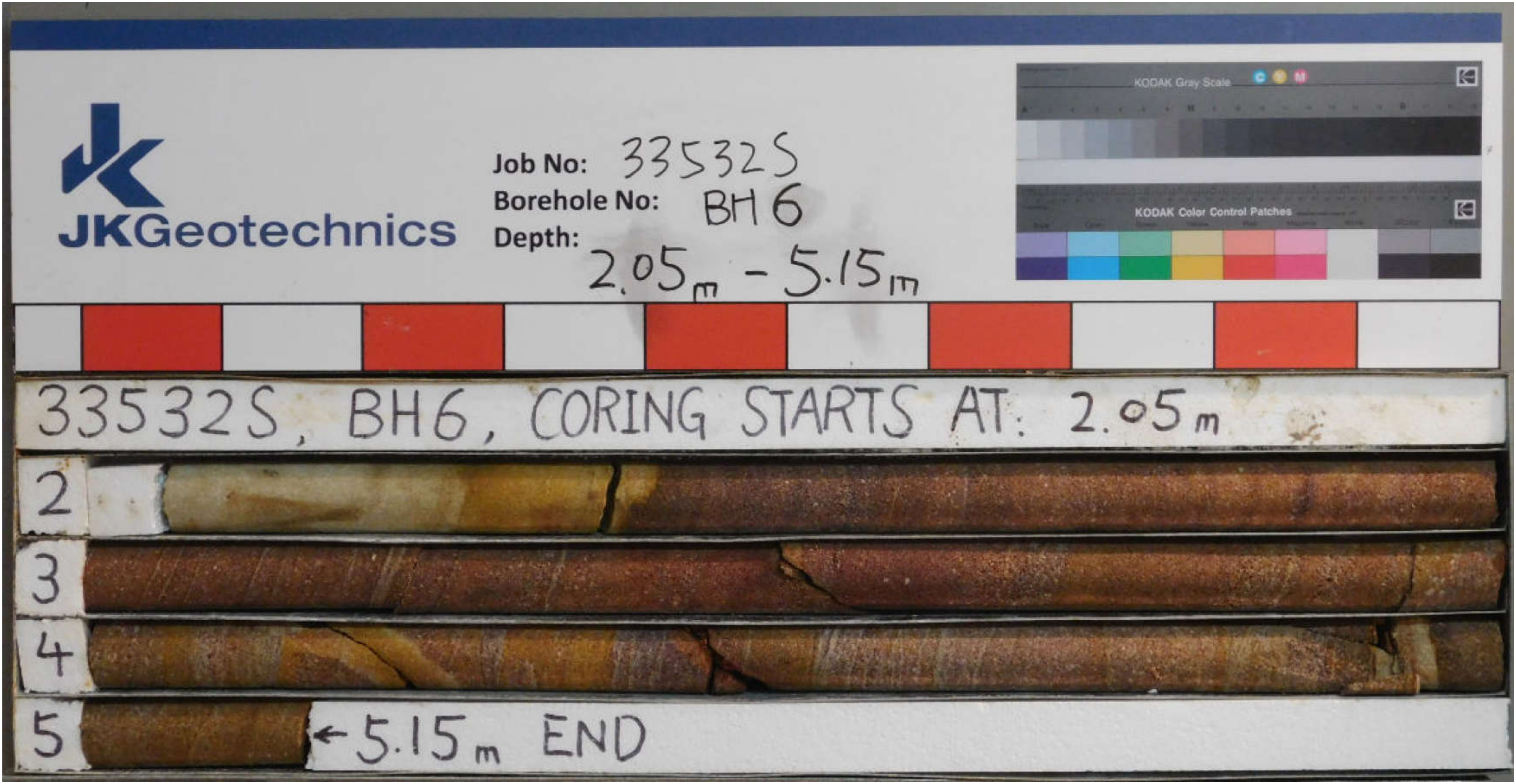
Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Core Size: NMLC
R.L. Surface: ~21.3 m
Date: 18/9/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: B.Z./T.C.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components START CORING AT 2.05m	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL-0.1 L-0.3 M-1 H-3 VH-10 EH	SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
100% RETURN		19 18	3		SANDSTONE: fine to coarse grained, red brown and brown, bedded at 20°.	MW	H	2.0 1.1 1.2		(3.53m) J, 67°, Un, Vr, Fe Ct		Hawkesbury Sandstone
		17	4 5		as above, but occasional fine to medium grained sub angular quartz grains inclusion.			1.4 1.6 1.2		(3.93m) Be, 11°, Ir, Vr, Fe Ct (4.20m) J, 66°, Un, Vr, Fe Vn (4.43m) J, 43°, Ir, Vr, Fe Cn, 4 mm.t (4.83m) J, 85°, Ir, Vr, Fe Vn		
		16 15 14 13	6 7 8		END OF BOREHOLE AT 5.15 m				600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



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Job No: 33532S
Borehole No: BH6
Depth: 2.05m - 5.15m

33532S, BH6, CORING STARTS AT: 2.05m

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BOREHOLE LOG

Borehole No.
7
1 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S **Method:** SPIRAL AUGER **R.L. Surface:** ~20 m
Date: 18/9/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** B.Z./T.C.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING								■		ASPHALTIC CONCRETE: 50mm.t over GRAVELLY ROADBASE: 50mm.t				
	■							▨	-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to coarse grained sub angular and angular igneous gravel.	M			APPEARS POORLY COMPACTED
	■				N = 3 2,1,2			▨		FILL: Silty clayey sand, fine to medium grained, brown and grey, low to medium plasticity clay, with fine to coarse grained sub angular sandstone gravel.				
	■			■		19	1	⁘	SP	SAND: fine to medium grained, brown, with silt fines and low plasticity clay.	M	(L)		RESIDUAL
										REFER TO CORED BOREHOLE LOG				
						18	2							
						17	3							
						16	4							
						15	5							
						14	6							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33532S PARRAMATTA.GPJ <<DrawingFile>> 16/10/2020 16:43 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

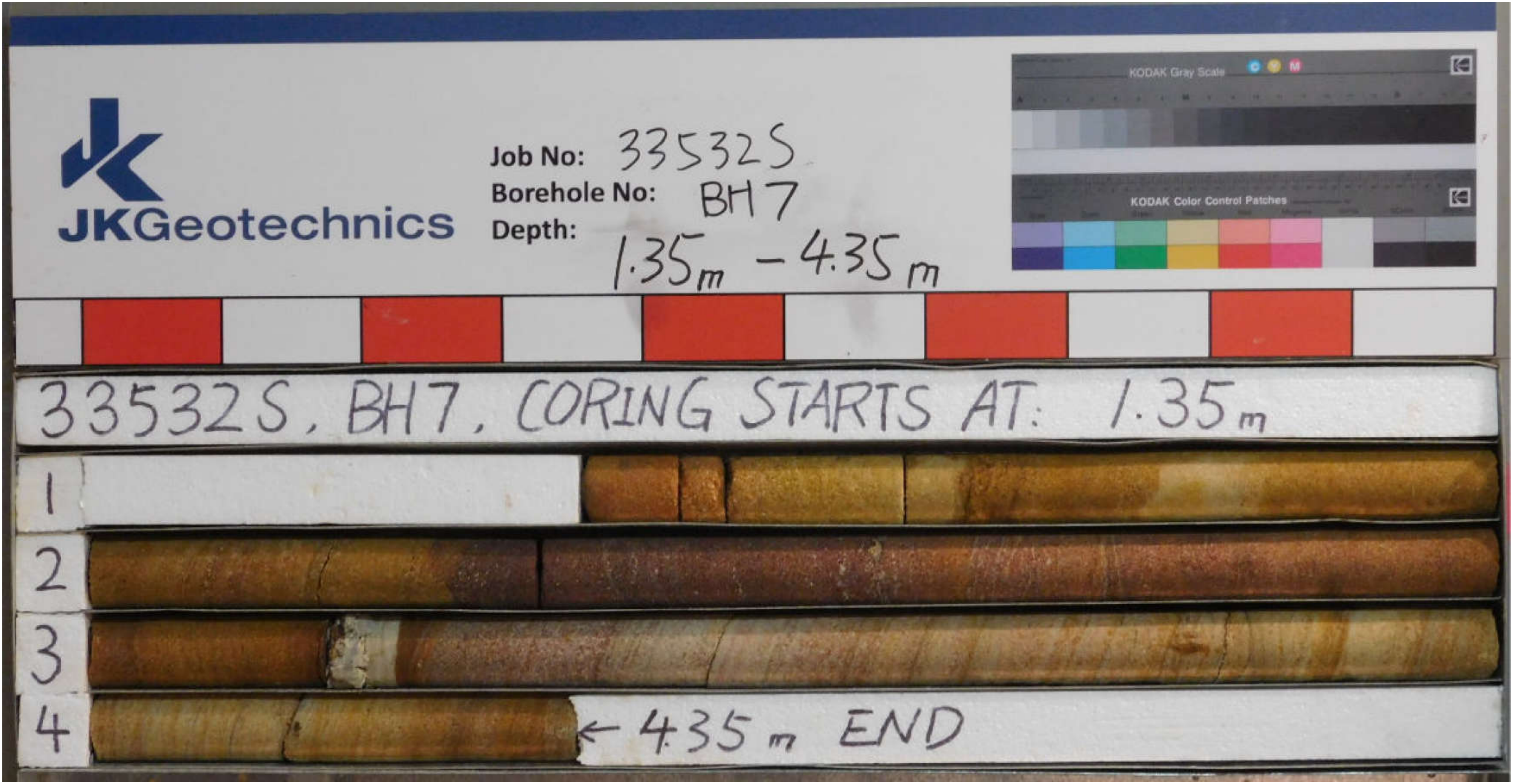
Borehole No.
7
2 / 2

Client: WEST END MAZDA C/O GREENWICH PROJECTS
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 574-584 CHURCH STREET, PARRAMATTA, NSW

Job No.: 33532S
Core Size: NMLC
R.L. Surface: ~20 m
Date: 18/9/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: B.Z./T.C.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600, 200, 60, 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					START CORING AT 1.35m							
100% RETURN		18	2		SANDSTONE: fine to coarse grained, brown, massive.	MW	L - M M - H	0.20 0.70 1.3				Hawkesbury Sandstone
		17	3		as above, but red brown, with occasional fine to medium grained, sub angular quartz grain inclusion.			1.4				
		16	4		SANDSTONE: fine to coarse grained, light brown and brown, bedded at 15-25°.			1.0 1.2 1.7		(3.16m) XWS, 5°, Un, R, 26 mm.t, Sandy clay (4.14m) Be, 15°, P, Vr, Sand, Ct		
		15 14 13	5 6 7		END OF BOREHOLE AT 4.35 m				600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



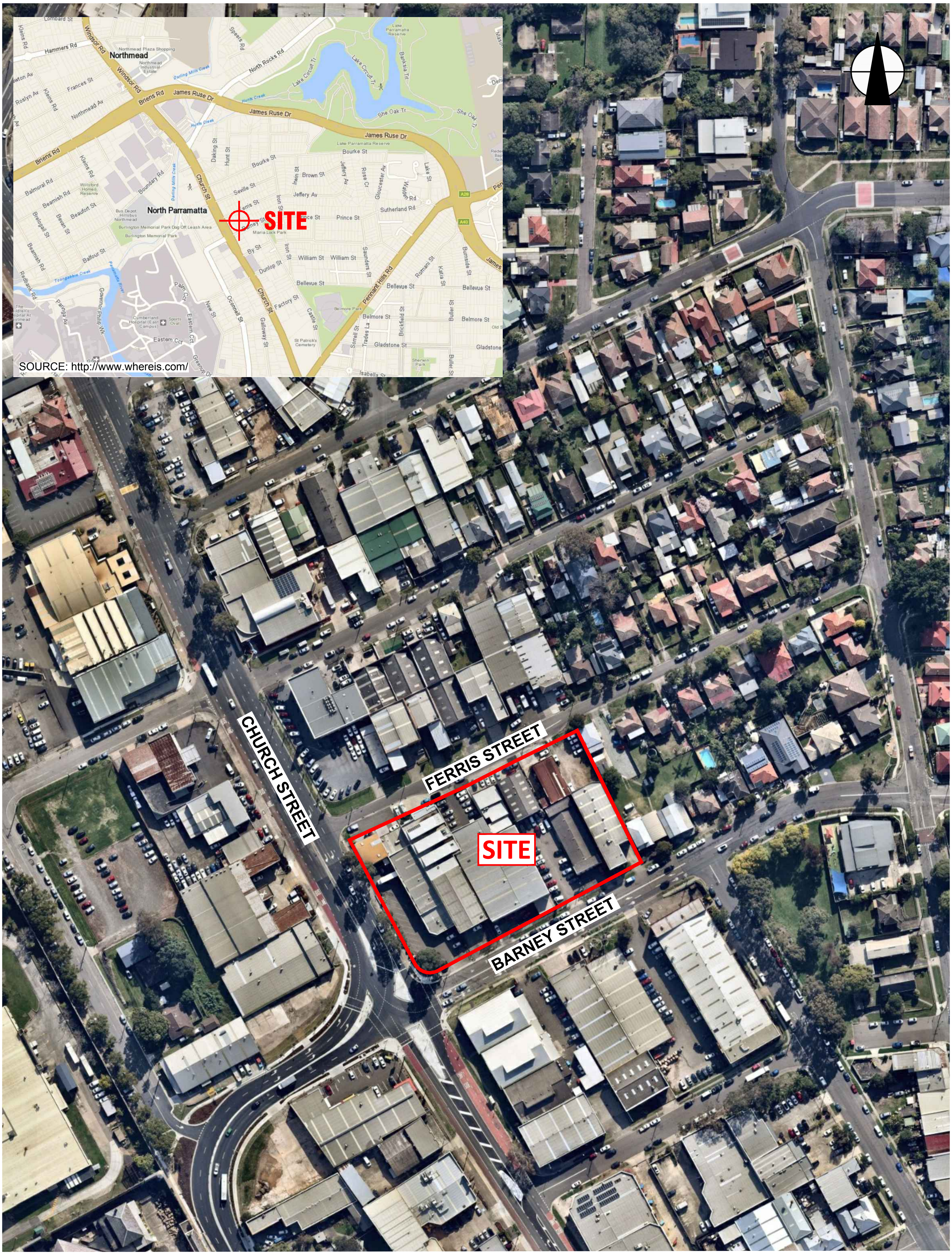
JKGeotechnics

Job No: 33532S

Borehole No: BH7

Depth: 1.35m – 4.35m

33532S, BH7, CORING STARTS AT: 1.35m



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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:

SITE LOCATION PLAN

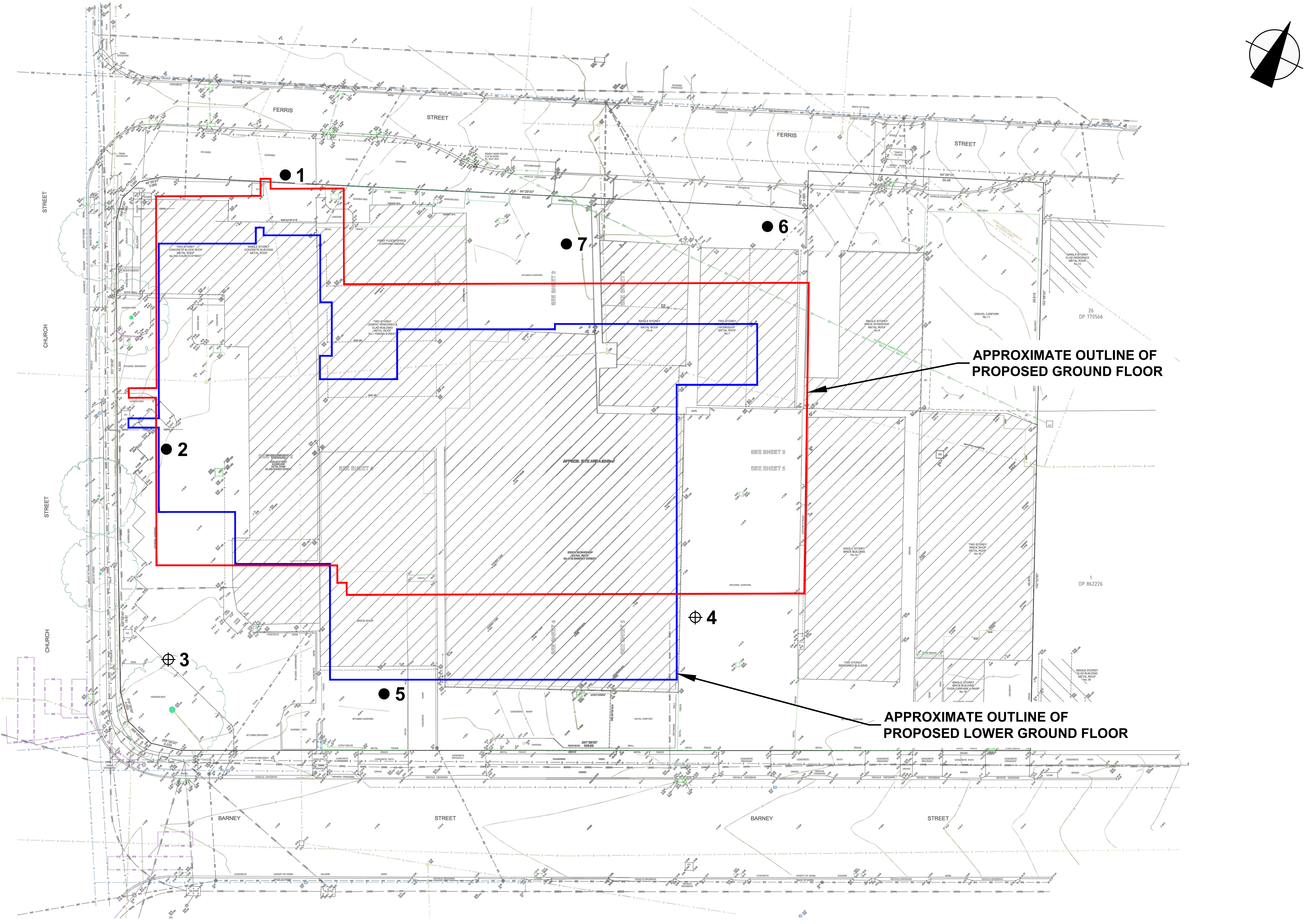
Location: 574-584 CHURCH STREET,
NORTH PARRAMATTA, NSW

Report No: 33532S

Figure No: 1

JKGeotechnics

PLOT DATE: 28/09/2020 4:07:32 PM DWG FILE: Y:\33000'S\33532S NORTH PARRAMATTA\CAD\33532S.DWG



LEGEND

● BOREHOLE

⌖ BOREHOLE AND GROUNDWATER MONITORING WELL

0 5 10 15 20 25
SCALE 1:500 @A3 METRES

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

Title:
INVESTIGATION LOCATION PLAN

Location: 574-584 CHURCH STREET,
NORTH PARRAMATTA, NSW

Report No: 33532S
Figure No: 2

JKGeotechnics

PLOT DATE: 28/09/2020 4:10:00 PM DWG FILE: Y:\33000'S\33532S NORTH PARRAMATTA\CAD\33532S.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 1726:2017 *'Geotechnical Site Investigations'*. 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 soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

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 (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	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 fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

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 shrink-swell behaviour, 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 methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock 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 a large 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. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, 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 limited 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 cuttings. 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 assessed 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. 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, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, 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 NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom 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 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

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 63.5kg 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.

A modification to the SPT 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.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

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. The CPT does not provide soil sample recovery.

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. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- 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.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

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 reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised 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 assess 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 Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. 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.

Reasonable 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.
- Details of the development that the Company could not reasonably be expected to anticipate.

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 rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

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

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 and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- ▩ FILL
- ≋ TOPSOIL
- ▨ CLAY (CL, CI, CH)
- ▥ SILT (ML, MH)
- ░ SAND (SP, SW)
- ⁘ GRAVEL (GP, GW)
- ▨ SANDY CLAY (CL, CI, CH)
- ▨ SILTY CLAY (CL, CI, CH)
- ▨ CLAYEY SAND (SC)
- ░ SILTY SAND (SM)
- ▨ GRAVELLY CLAY (CL, CI, CH)
- ▨ CLAYEY GRAVEL (GC)
- ▥ SANDY SILT (ML, MH)
- ⁂ PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- ⁘ CONGLOMERATE
- ░ SANDSTONE
- ▤ SHALE/MUDSTONE
- ▥ SILTSTONE
- ▤ CLAYSTONE
- ■ COAL
- ▥ LAMINITE
- ▦ LIMESTONE
- ≋ PHYLLITE, SCHIST
- ▤ TUFF
- ⁖ GRANITE, GABBRO
- + DOLERITE, DIORITE
- ∨ BASALT, ANDESITE
- ≈ QUARTZITE

OTHER MATERIALS

- ▦ BRICKS OR PAVERS
- △ CONCRETE
- ■ ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory **Classification** Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

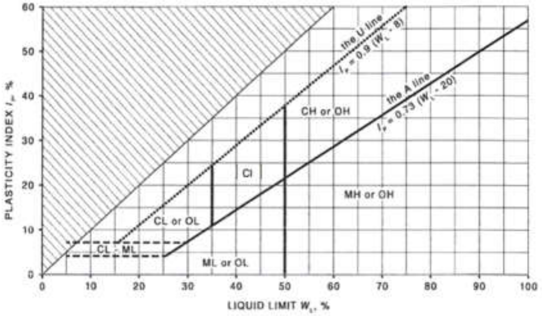
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	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 analysis.
	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. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	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° 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 (Fine Grained Soils)	*w* > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained 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 ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
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 and ≤ 35 | 4 – 10
	MD	MEDIUM DENSE | > 35 and ≤ 65 | 10 – 30
	D	DENSE | > 65 and ≤ 85 | 30 – 50
	VD	VERY DENSE | > 85 | > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered		XW		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately Weathered		MW		The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres