SPECIALIST ADVICE TO

SUSTAINABLE DEVELOPMENT GROUP LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MIXED USE DEVELOPMENT

AT

461 CHAPEL ROAD, BANKSTOWN, NSW

Date: 28 March 2025
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ATTACHMENTS

1 INTRODUCTION

This specialist advice report presents the results of a geotechnical investigation for the proposed residential development at 461 Chapel Road, Bankstown, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Richard Huynh of Sustainable Development Group Limited via email dated 19 November 2024. The commission was on the basis of our fee proposal, Ref. 'P70355PE' dated 11 October 2024.

Based on the supplied architectural drawings (Job No. 20451, Drawing Nos. PLA-DA-10B1^{11}, PLA-DA-1000^{11} to PLA-DA-1003^{11}, PLA-DA-1005^{11}, PLA-DA-1009^{11}, PLA-DA-1010^{11}, PLA-DA-1011^{11}, PLA-DA-1023^{11}, PLA-DA-2000^{11} to PLA-DA-2003^{11}, and PLA-DA-3000^{11} to PLA-DA-3002^{11}, all dated 26 March 2025) prepared by Plus Architecture, we understand the proposed development will comprise demolition of the existing structures followed by construction of an twenty-three storey building overlying a single level basement. Due to the sloping nature of the site down to the south, excavation to a maximum depth of between approximately 2.6m (south-eastern corner) and 4.4m (north-western corner) will be required to achieve the basement finished floor level (FFL) at RL26.8m. Locally deeper excavations, say in the order of between approximately 1m and 2m below the basement FFL, are anticipated for any lift overrun pits and services.

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

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at three borehole locations, and to use this as a basis for providing our comments and recommendations on site preparation, excavation, shoring design, drainage issues, footing design, the basement floor slab, earthquake design parameters, exposure classification, and additional geotechnical input.

This geotechnical investigation was carried out in conjunction with a preliminary environmental site investigation completed by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref. 'E37149PL', for the results of the environmental investigation.

This report provides specialist advice for use by the structural and civil designers in preparing their designs and no part of this report is considered or intended to form a regulated design in accordance with the Design and Building Practitioners Act 2020.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out between 2 and 4 December 2024 and comprised the drilling and testing of three boreholes (BH1 to BH3) using our track mounted JK308 drilling rig. The boreholes were drilled to depths between approximately 2.7m (BH1) and 5.8m (BH2) below existing grade using spiral augering techniques. Regular Standard Penetration Tests (SPT) were carried out in the soil materials. All three boreholes were subsequently extended to their final depth of approximately 12.0m using NMLC diamond coring techniques, with water flush.

Prior to the commencement of the fieldwork, a specialist sub-consultant reviewed available 'Before You Dig Australia' information and electro-magnetically scanned the borehole locations for buried services.

The fieldwork for the investigation was carried out in the full time presence of our geotechnical engineer (Christoper Rooke), who set out the borehole locations, supervised the electro-magnetic scanning, nominated testing and sampling, and guided the groundwater monitoring well installations. The borehole logs are attached to this report, together with a glossary of terms and symbols used.

The borehole locations, as shown on the attached Figure 2, were set out using taped measurements from existing surface features. The surface RL's shown on the borehole logs were obtained by interpolation between spot heights shown on the provided survey plan (Ref. 01301, Sheet No. 1, dated 1 September 2023) prepared by Survtech. The survey plan forms the basis of Figure 2. The datum is the Australian Height Datum (AHD).

The strength of the natural soils was assessed from the SPT 'N' values, together with hand penetrometer readings on residual clay soils recovered in the SPT split-spoon sampler, and by tactile examination. The strength of the underlying bedrock was initially assessed by observation of auger penetration resistance when using a tungsten carbide (TC) drill bit and examination of the recovered rock cuttings. The strength of the underlying bedrock which was core drilled was assessed by examination of the recovered rock cores, together with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) test results.

Groundwater observations were made in the boreholes during and on completion of drilling. Groundwater monitoring wells were installed within the boreholes and comprise a 50mm diameter Class 18 PVC standpipe. Prior to their installation, each borehole was 'reamed out' with a Polycrystalline Diamond (PCD) drill bit of approximately 100mm diameter to form a sufficient annulus between the monitoring well casing and sides of the boreholes. The annulus between the boreholes and slotted lengths was backfilled with 2mm sand within the response zone. Above the sand backfill, the boreholes were sealed with bentonite. A cast-iron 'Gatic' cover was concreted flush with the ground surface to protect the top of the groundwater monitoring wells. The installation details for the monitoring wells are presented on the relevant borehole logs.

During a return visit to site on 9 December 2024, the groundwater was purged from the monitoring wells and 'mini-diver' data loggers installed to continually measure groundwater levels. Whilst on site on 10 March 2025, approximately three months after the fieldwork was completed, the data loggers were downloaded and the groundwater monitoring results for BH3 only, presented as groundwater RL (mAHD) and daily rainfall (mm) versus time plots, are shown in Figure 3 (BH3). We note the groundwater monitoring wells installed in BH1 and BH2 remained 'dry' during the monitoring period; that is, no groundwater was recorded within approximately 7.5m of surface levels at these two locations. The plotted rainfall data shown in Figure 3 was obtained from the Bureau of Meteorology's rainfall records for their monitoring station at Bankstown Airport (Station No. 066137), which is located approximately 4.3km to the south-west of the site.

For details of the investigation techniques employed, and their limitations, reference should be made to the attached Report Explanation Notes.

The recovered rock cores were returned to our laboratory where they were photographed and Point Load Strength Index tested. The rock core photographs are enclosed with the relevant borehole logs. The Point Load Strength Index test results are plotted on the borehole logs and summarised in the attached Table A. The Unconfined Compressive Strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table A.

Three selected soil samples were returned to a NATA accredited analytical laboratory, Envirolab Services Pty Ltd, for soil pH, sulphate, chloride and resistivity testing. The results are presented in the attached Envirolab Services 'Certificate of Analysis 368570'.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located toward the toe of a south facing hillside which slopes gently down at an average of approximately 3°. The site itself is rectangular in plan, slopes down with the natural topography, and has a maximum elevation relief of approximately 1.6m between the northern and southern boundaries. French Avenue and Chapel Road bound the site to the north and west, respectively.

At the time of the fieldwork, the site was occupied by several single storey buildings of various construction, including a brick church, a brick residence, a weatherboard and clad hall, a demountable amenities block, and two small weatherboard and brick structures (i.e. garage and vestry building). An asphaltic concrete (AC) driveway/parking area was located along the eastern boundary, and a concrete driveway was located between the church and hall within the south-western portion of the site. The remainder of the site comprised concrete pavements, grass areas, and small to medium sized bushes and trees.

The neighbouring property to the east (14 French Avenue) contained a two storey brick and rendered building underlain by a single (assumed) level basement which abutted the common boundary. The building/basement was located within the southern portion of the site and a concrete paved carpark occupied the northern end of the site. The driveway down to the basement level was located along the western boundary of the neighbouring property. Based on our observations along French Avenue, we anticipate the basement walls provide between approximately 2m and 3m of support to the subject site along the central and southern portions of the common boundary, although this could not be confirmed from the street frontage.

The neighbouring property to the south (457 Chapel Road) contained a two storey brick building which abutted the common boundary. The building was located within the eastern portion of the site and a concrete paved carpark occupied the western end of the site. The building did not appear to be underlain by a basement level and surface levels across the common boundary were similar.

The buildings and structures within the subject site and neighbouring properties appeared in relatively good external condition, with the exception of the AC pavements within the subject site which were in a poor condition with extensive cracking and pot holes evident.

3.2 Subsurface Conditions

The 1:100,000 series geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site is underlain by Ashfield Shale of the Wianamatta Group comprising black and dark grey shale and laminite, but is located close to the contact with the higher lying Bringelly Shale. Groundwater was encountered below 6.7m depth in BH3 only. Reference should be made to the attached boreholes logs for specific details at each location. A summary of the subsurface conditions encountered at the borehole locations is presented below.

Pavements
AC pavement was encountered from surface level in BH1 and was 20mm thick.

Fill
Fill comprising silty clayey gravel and silty clay was encountered below the pavement in BH1 and from surface level in the remaining boreholes, and extended to depths of 0.4m (BH2) and 0.6m (BH1 and BH3). The fill contained various proportions of fine to medium grained igneous and alluvial gravel, glass, ash, fragments of ceramic, plastic, concrete and brick, and roots and root fibres.

Natural Soils
Residual silty clay was encountered below the fill in all boreholes and extended to the surface of the underlying weathered siltstone and sandstone bedrock. The residual clays were of medium to high plasticity and of stiff to hard strength, and contained traces of fine to medium grained ironstone gravel, roots and root fibres.

Weathered Bedrock
Weathered siltstone, sandstone and laminite (interbedded siltstone and sandstone) bedrock was encountered below the residual soils in BH1, BH2 and BH3 at respective depths of 1.9m (RL28.0m), 1.8m (RL27.1m), and 2.3m (RL27.2m). On first contact, the weathered bedrock was highly weathered and of very low and low strength.

Based on the rock core lengths, the bedrock was predominantly siltstone with bands of sandstone and laminite up to approximately 1.2m thick. The bedrock quality was initially relatively poor but improved with depth. The upper sequence of bedrock which extended to depths between approximately 7m and 10m was extremely, highly, moderately and slightly weathered and generally of 'soil' strength and very low to medium rock strength. The bedrock contained a number of defects including sub-horizontal bedding partings, inclined joints and extremely weathered/clay seams and bands up to approximately 0.2m thick. Several 'no core' zones were encountered within the boreholes and were between approximately 0.11m and 0.6m thick. The 'no core' zones have been interpreted to represent either extremely weathered bedrock, clay seams or fractured bands of bedrock.

The bedrock within the basal 2m to 3m of each borehole was generally assessed as being slightly weathered to fresh, and of medium and high strength. The lower sequence of rock contained fewer defects than the overlying poorer quality rock, though numerous sub-horizontal bedding partings, steeply dipping joints and extremely weathered seams up to approximately 0.08m thick were still present.

An indicative rock classification of the bedrock has been carried out for each borehole (in accordance with 'Classification of Sandstone and Shales in the Sydney Region: A Forty Year Review' by Pells et al., Australian Geomechanics, June 2019), as tabulated below:

Borehole	Approximate Surface RL (mAHD)	Indicative Engineering Classification of Siltstone Bedrock Depths (m)[1] [RL of Unit (mAHD)]			
		Class V	Class IV	Class III	Class II
1	29.9	4.2 – 6.9 [25.7 – 23.0]	-	6.9 – 8.6[2] [23.0 – 21.3]	8.6 – 12.0 [21.3 – 17.9]
2	28.9	1.8 – 7.8 [27.1 – 21.1]	7.8 – 10.3[2] [21.1 – 18.6]	-	10.3 – 12.0 [18.6 – 16.9]
3	29.5	5.7 – 9.0 [23.8 – 20.5		-	9.0 – 12.0[2] [20.5 – 17.5]

Notes.
1. The above classification is for siltstone bedrock only as the laminite/sandstone bands were relatively narrow.
2. Depending on the pile depth, Class V bedrock was encountered between 8.2m and 8.4m in BH1 and between 9.7m and 10.1m in BH2, and Class III bedrock was encountered between 11.1m and 11.5m in BH3. Further geotechnical review of the rock classes must be completed (after receipt of the structural drawings) as the founding depth and footing width/diameter influence the rock classification.

Groundwater

All boreholes were 'dry' during and on completion of auger drilling. As water is injected into the boreholes during core drilling, the groundwater levels shown on the borehole logs immediately after drilling are most likely elevated and not representative of actual groundwater levels. The results of the groundwater monitoring carried out between 9 December 2024 and 10 March 2025 are summarised below.

Groundwater Monitoring Well	Approximate Surface Level / Base of Standpipe (RL mAHD)	Monitored Groundwater RL (mAHD)		
		Shallowest Groundwater Level	Deepest Groundwater Level	Average Groundwater Level[1]
BH1	29.9 / 22.1	Dry to 7.8m depth (RL22.1m)		
BH2	28.9 / 21.4	Dry to 7.5m depth (RL21.4m)		
BH3	29.5 / 20.5	22.9	21.7	22.0

Note
1. The sum of the groundwater levels was divided by the number of groundwater readings throughout the monitoring period to determine the average groundwater levels.

From the above table, attached borehole logs and groundwater RL versus time plot for BH3, groundwater was only encountered in BH3 and the shallowest groundwater level recorded over the monitoring period was approximately 3.9m below the proposed basement FFL at RL26.8m. In addition, there was no increase in groundwater level due to the relatively large single (i.e. 54mm of rainfall on 16 January 2025) or cumulative (i.e. 126mm of rainfall between 7 and 16 January 2025) rainfall events. Further comments and recommendations on groundwater seepage are presented in Section 4.2.4 below.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores correlated well with our field logging assessment of bedrock strength. The estimated UCS's, based on a correlation provided in AS1726:2017 'Geological Site Investigations' (i.e. UCS = 20 x $I_{S(50)}$), ranged between 1MPa and 32MPa, with an average of approximately 15MPa (medium strength). Within the upper poorer quality rock sequence, the average bedrock strength was approximately 9MPa, whereas the average bedrock strength within the underlying better quality rock (i.e. below 6.9m in BH1, 8.5m in BH2, and 9.0m in BH3) was 18MPa.

The soil pH test results were 7.0, 7.5 and 8.7 which showed the samples were neutral to moderately alkaline. The sulphate and chloride test results were less than or equal to 450mg/kg, which indicate low sulphate and chloride contents. The resistivity test results were between 1,900 ohm.cm and 23,000 ohm.cm indicating moderate to high resistivity.

4 COMMENTS AND RECOMMENDATIONS

4.1 Sydney Water

Based on an available and current 'Before You Dig Australia' drawing (Sequence No. 246673111) provided by Sydney Water Corporation (Sydney Water), a 150mm diameter Cast Iron Cement Lined (CICL) water main is located approximately 3m beyond the western site boundary. Several other water mains and sewer pipes are located more than 10m beyond the northern and western site boundaries and these assets appear to be outside the zone of influence or 'dig zone' of the proposed basement excavation.

As a Sydney Water asset (i.e. 150mm diameter CICL water main) is located in close proximity to the site, we recommend that a Water Services Coordinator (WSC) be engaged very early in the design process to determine whether the proposed development will be subject to a 'Specialist Engineering Assessment' (SEA), which requires an estimate of pipe deflections, using finite element modelling (FEM) software, as a result of the basement excavation. This information is then used as an input, by others, to complete the SEA of the effect of the proposed basement on the Sydney Water infrastructure. JK Geotechnics can assist with the numerical modelling after the WSC has been engaged and the shoring/structural design has been completed.

4.2 Site Preparation and Excavation

4.2.1 Dilapidation Surveys

Prior to demolition and excavation commencing, we recommend that detailed dilapidation reports be prepared for the neighbouring buildings to the east (14 French Avenue) and south (457 Chapel Road). The dilapidation survey reports can be used as a benchmark for assessing possible future damage claims arising from the works. As dilapidation survey reports are relied upon for the assessment of potential future damage claims, they must be carried out thoroughly with all defects rigorously described (i.e. defect type, defect location, crack width, crack length etc.) and defects photographed where practical.

The respective owners of the adjoining properties should be asked to confirm in writing that the dilapidation survey reports on their property presents a fair assessment of the existing conditions. We note that Council may also require that dilapidation reports be prepared for any adjoining Council assets.

4.2.2 Demolition and Excavation

All excavation should be carried out with reference to the most recent 'Excavation Work – Code of Practice' by Safe Work Australia.

The outline of the proposed single level basement is indicated on the attached Figure 2. The basement will extend to the site boundaries and require excavation to a maximum depth of between approximately 2.6m (south-eastern corner) and 4.4m (north-western corner). Locally deeper excavations, probably in the order of between 1m and 2m below the proposed basement FFL at RL26.8m, will be required for the lift overrun pits. Excavation below existing surface levels will extend through the surficial fill and residual clays, and penetrate the underlying siltstone and sandstone bedrock.

Site preparation works will include demolition of the existing buildings and pavements, and excavation of any deleterious or contaminated fill. Reference should be made to the JKE report on the offsite disposal of soils. Care must be taken during these works and subsequent bulk excavation not to undermine or remove support from adjacent buildings or site boundaries. In this instance, the work will need to be completed using suitably experienced (and insured) contractors.

Prior to bulk excavation commencing, the footing details for the adjacent buildings to the east and south must be confirmed by review of available 'as-built' structural drawings, though judging by their age it is likely the drawings will not be available. The purpose of the review is to confirm the founding level of the neighbouring basement (14 French Avenue) and floor level (457 Chapel Road), and whether additional strengthening or underpinning works are required. If the drawings are not available, several test pits should be excavated in an attempt to assess the footing depth and foundation materials below the neighbouring building to the south. Permission from the owners to complete the works must be sought and the test pits inspected by JK Geotechnics and the project structural engineer. Test pits below the base of the neighbouring basement level will be relatively difficult (due to their anticipated depth) and provision of a survey plan which shows the basement floor level may assist in this regard (i.e. in lieu of any structural drawings or test pits).

Excavation of the soils and weathered bedrock profiles ranging from 'soil' strength to very low rock strength is expected to be readily achievable using conventional techniques such as the buckets of medium to large sized hydraulic excavators, with ripping of the rock using tynes.

'Hard rock' excavation conditions should be expected for the low and any medium strength bedrock, and in the unlikely case of high strength bedrock. Hydraulic rock hammers fitted to excavators will be required for effective removal of any medium or higher strength bedrock, if encountered. However, considering the quality of the bedrock above and slightly below the basement FFL, it will likely be possible to rip the bedrock with a tyne of a large, say 30 Tonne, tracked excavator. Grid sawing the bedrock would also facilitate excavation and would assist in dampening the transmission of ground borne vibrations to the nearby buildings and/or buried services, provided the base of the saw slot is maintained well below excavation level. Dust suppression by spraying with water should be carried out whenever rock saws are being used.

If rock hammers are to be used, they should be relatively small hammers fitted on small excavators to reduce the risk of vibration induced damage; refer to Section 4.2.3 below for a discussion on monitoring vibrations.

4.2.3 Potential Vibration Risks

Where percussive excavation techniques are adopted, there is the possibility that there will be direct transmission of ground borne vibrations to the surrounding buildings. As such, continuous vibration monitoring should be carried out whenever hydraulic rock hammers are used during demolition and excavation, as a precaution against possible vibration induced damage.

Vibration monitors should be set up on the neighbouring structures to the east and south prior to demolition and excavation commencing. The monitors should be fitted with a warning system (i.e. flashing lights, audible alarm etc.) which is set to trigger when the permissible vibration limit has been recorded. The locations of the monitors (i.e. geophones) should be assessed following review of the dilapidation survey reports, and should be jointly nominated by JK Geotechnics, the project structural engineer and acoustic consultant. The vibrations on the neighbouring buildings should be tentatively limited to a peak particle velocity (PPV) of 5mm/sec, subject to review of the dilapidation reports by the project structural engineer and their confirmation that the buildings have no particular sensitivity that would require a lower threshold.

4.2.4 Seepage

Following the three months of monitoring, the groundwater level within the standpipe installed in BH3 ranged between approximately 6.6m (RL22.9m) and 7.8m (RL21.7m) below existing surface levels. The standpipes installed in BH1 and BH2 were ‘dry’ for the duration of the monitoring period. As such, groundwater levels appear to be a minimum of 3.9m below the proposed basement FFL of RL26.8m. On this basis, groundwater levels are not expected to be adversely affected by the basement excavation to the extent there will be any noticeable (or measurable) impact on nearby water bodies (if any) or neighbouring properties.

Notwithstanding the above, minor seepage into the proposed basement excavation may occur as local seepage flows at the fill/natural soil interface, at the soil/bedrock interface, and through joints/bedding partings/seams within the bedrock profile, particularly after heavy or prolonged rainfall. If seepage does occur, it is likely to be the result of local infiltration, intermittent and of small flowrate, and readily controlled during construction by sump and pump methods to divert it to Council’s stormwater system for disposal.

Provided WaterNSW approves a ‘drained’ basement, drainage will need to be provided behind the shoring walls and below the basement slab to intercept any ephemeral seepage and dispose of this directly to Council’s drainage system. The underfloor drainage should comprise a strong, durable, single-sized aggregate such as ‘blue metal’ gravel.

The completed excavation should be inspected by the hydraulic consultant to assess if the designed drainage system is adequate for actual seepage flows.

4.3 Retention

4.3.1 Shoring

For the encountered subsurface conditions generally comprising fill and natural clay soils over weathered siltstone and sandstone bedrock, we consider a suitable shoring system may comprise soldier pile walls which are progressively anchored, or internally propped, as excavation proceeds (where necessary). The shoring wall piles must extend the full depth of the excavation and be founded below bulk excavation level. Where the excavation is less than approximately 3m deep and no movement sensitive structures are located within the influence zone of the exaction, cantilevered piles could be considered. The influence zone is defined as a horizontal distance of 2H (where 'H' is the depth of the excavation in metres) behind the shoring wall. The effect of ground movements on all structures and services that lie within the influence zone of the excavation must be taken into account by the shoring designer. Pouring using tremie methods is recommended for pile holes greater than about 4m depth.

We note that depending on the founding level of the neighbouring basement to the east (14 French Avenue), shoring may not be required along the southern portion of the eastern boundary. However, in the first instance, the shoring should be designed to extend around the entire perimeter until it is confirmed, either by review of the neighbouring 'as-built' structural drawings, survey plan or test pits, that shoring is not required in any specific areas. Approval from the neighbouring land owners would be required prior to the installation of anchors below their property.

4.3.2 Lateral Earth Pressures

The major consideration in the selection of earth pressures for the design of retaining walls is the need to limit deformation occurring outside the excavation. The following characteristic earth pressure coefficients and soil and bedrock parameters may be adopted for the design of the retention system.

- For propped or anchored shoring walls which are highly sensitive to movements (which we expect to be on all sides of the proposed basement), we recommend the use of a trapezoidal earth pressure distribution with a lateral earth pressure of 8HkPa for the soil and bedrock, where H is the retained height in metres. The maximum pressure should be applied over the central 50% of the support system, tapering to zero at the crest and toe. The lateral earth pressure assumes a horizontal retained surface.
- Cantilevered walls of not more than 3m in height which will be propped or retrained in the long term by the structure, should be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient (K_0) of 0.55 for the soil and weathered bedrock. The lateral earth pressure assumes a horizontal retained surface.
- A bulk unit weight of 20kN/m^3 and 22kN/m^3 should be adopted for the soil and bedrock, respectively.

- Any surcharge affecting the basement walls (i.e. traffic loading, nearby buildings etc.) must be allowed for in the design.

- The basement shoring walls must be designed as 'drained' and measures taken to provide permanent and effective drainage of the ground behind the walls. The lift pits should be designed to withstand full hydrostatic uplift pressures (i.e. tanked) with a design groundwater level equivalent to the proposed basement level. Alternatively, the potential groundwater pressures may be alleviated by providing the lift pit with drainage and a pump-out system.

- Lateral restraint of the fully penetrating shoring wall piles may be achieved by embedding the piles into weathered bedrock below bulk excavation level. At this stage, an allowable lateral resistance of 150kPa could be adopted for the siltstone and sandstone bedrock of at least very low strength (not extremely weathered bedrock) below bulk excavation level, though the upper 0.5m of the socket must be ignored to allow for disturbance or possible over excavation. For piles embedded into bedrock below bulk excavation levels, a minimum embedment depth (ignoring the 0.5m allowance above) of 1.0m should apply. Care is required not to over excavate in front of the piles, and all excavation in front of the walls such as for footings, buried services etc. must be taken into account in the wall design.

- If adopted, anchors bonded into consistent bedrock of at least very low strength may be designed on the basis of an allowable bond strength of 100kPa. All anchors should be proof loaded to at least 1.3 times their working load and then locked off at approximately 85% of their working load. Proof loading should be carried out in the presence of an engineer independent of the anchoring contractor. Anchors must be bonded behind a line drawn up at 45° from the base of the excavation, with all anchors having a free length and bond length of at least 3m each. Lift off tests should be carried out on at least 25% of all anchors to confirm they are maintaining their loads. Permission must be obtained from the neighbouring property owners prior to installation of anchors within their property. If permission is not granted to install anchors within the neighbouring properties, the shoring walls could be appropriately braced/propped from with the subject site.

4.4 Footing Design

On completion of excavation, poor quality siltstone and sandstone bedrock of 'soil' strength to very low and low rock strength is expected to be exposed at bulk excavation. Due to the anticipated high column loads and limited strength bedrock likely to be exposed at bulk excavation level, high level pad/strip footings founded within the poorer quality bedrock are not considered feasible to support the proposed twenty-three storey building. The more appropriate foundation system is considered to be either piled footings founded in the underling Class II bedrock or the use of a stiffened raft slab. Further details on these options are outlined below.

Piled Footings

Bored piles which are socketed at least 0.3m into Class II bedrock may be designed for an allowable end bearing pressure of 4.5MPa provided that: 1) a supplementary geotechnical investigation is carried out following demolition to assess the depth to Class II bedrock within the central and north-western portions of the site, and 2) all foundation piles are inspected by a geotechnical engineer. For adequate site coverage, the geotechnical investigation should include a minimum of two cored boreholes. Depending on the necessary socket lengths, deeper coring (i.e. below the current investigation depths) must be completed to show the bedrock continues to be of at least Class II quality with depth.

For the length of pile rock socket longer than this 0.3m, an allowable shaft adhesion of 450kPa in compression and 225kPa in tension (i.e. uplift) may be adopted provided the socket is satisfactorily cleaned and roughened. The shaft adhesion values are on condition that the pile shaft is roughened to a Roughness Class equivalent to at least R2, defined as the grooves of depth 1mm to 4mm, widths greater than 2mm, and at a spacing of 50mm to 200mm.

For preliminary design, any contribution to the capacity of the piles from the overlying poorer quality bedrock should be ignored due to strain incompatibility between the weaker material and much stiffer Class II bedrock. In addition, should load bearing shafts longer than 2.5m be proposed, detailed pile analysis must be completed as the load transfer between the pile and the bedrock becomes non-uniform. Further design advice (i.e. ultimate bearing pressures, geotechnical strength reduction factors etc.) can be provided if this approach is adopted.

The allowable bearing pressures provided above are based upon serviceability criteria of deflections at the pile base of less than 1% of the pile diameter. We note these settlements will be of an elastic nature and are expected to occur as construction proceeds.

Perimeter shoring piles founded at least 0.3m into Class II bedrock may also be designed for the allowable end bearing pressure and shaft adhesion values provided above. If preferred, the shoring piles could be terminated at a shallower depth within Class V (or better) bedrock. Provided the shoring piles are socketed a minimum of 0.3m into Class V bedrock below bulk excavation level, the piles may be designed for an allowable end bearing pressure of 700kPa. For the length of pile rock socket longer than this 0.3m, an allowable shaft adhesion of 70kPa in compression and 35kPa in tension (i.e. uplift) may be adopted provided the socket is satisfactorily cleaned and roughened, as outlined above.

Bored piles must be inspected by a geotechnical engineer during drilling, cleaned out, and poured on the same day as drilling. Depending on the load carrying capacity of the shoring wall piles, geotechnical inspections may only be required during the initial stages of piling to confirm that a satisfactorily bearing stratum is being achieved, provided that the piling contractor is adequately drilling and cleaning out each pile hole. Where the shoring wall piles have been designed to be founded in the better quality bedrock (i.e. Class II), all shoring piles must be inspected by a geotechnical engineer, similar to the foundation piles.

Raft Foundation

As weathered bedrock will be exposed at bulk excavation level, the use of a thickened raft slab may be considered. Detailed analysis of the raft would be required to estimate the total settlement and possible differential settlements in areas underlain by poorer quality bedrock, and the contact pressures below the raft. The preliminary design of the raft may be completed using the elastic parameters provided in the table below.

Parameters	Siltstone Bedrock				
	Extremely Weathered Siltstone	Class V	Class IV	Class III	Class II
Unit Weight (kN/m^3)	20	22	23	23	24
Elastic Modulus (MPa)[1]	50	80	300	700	1200
Poisson's Ratio	0.3	0.3	0.2	0.2	0.15
Note. 1. Appropriate sensitivity analysis using upper and lower bound parameters will need to be carried out and further advice will need to be sought from JK Geotechnics in this regard.					

The design of a heavily loaded raft is complex and requires complex analysis procedures for soil-structure interaction, such as with the use of three-dimensional finite element (FE) analysis software (e.g. Plaxis 3D). Analysis software modelling the rock as equivalent springs must not be used for this analysis. We expect the design of the raft will be an iterative process with both the geotechnical and structural engineers having input throughout the design. The first pass of the analysis, which could be completed using two-dimensional FE analysis software, will demonstrate the potential of the concept and identify the parameters critical to the design. JK Geotechnics can assist with the detailed geotechnical analysis of the raft using our FE software, once the initial raft design details are supplied by the structural engineer.

4.5 Basement Floor Slab

The proposed basement is expected to overlie weathered siltstone and sandstone bedrock. As such, we recommend the on-grade floor slab be provided within underfloor drainage. The underfloor drains should comprise a high strength, durable, single sized washed aggregate, such as 'Blue Metal', so as to lead any ephemeral seepage to a sump with an automatic pump out system for pumping to the stormwater system. Joints in the on-grade floor slab should incorporate dowels or keys to transfer shear forces but not bending moments.

4.6 Earthquake Design

The following parameters should be adopted for earthquake design in accordance with AS1170-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia' (including Amendments 1 & 2):

- Hazard Factor (Z) = 0.08; and
- Site Subsoil Class (Retention and Footing Design) = C_e

4.7 Soil Aggression

The soil aggression test results have indicated neutral to moderately alkaline conditions, as well as low sulphate and chloride contents. In accordance with Table 6.4.2. of AS2159-2009 'Piling-Design and Installation' and Table 4.8.1 of AS600-2018 'Concrete Structures', the exposure classification to concrete is 'Non-Aggressive' and 'A1', respectively.

4.8 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Supplementary geotechnical investigation to assess the depth to Class II bedrock within the central and north-western portions of the site.
- Finite element analysis of the basement floor slab, if required.
- Review of 'as-built' structural drawings or survey plans to assess the founding levels of the neighbouring buildings. If these drawings/plans are not available, excavation of several test pits should be completed along the southern site boundary to attempt to assess the footing details and foundation materials below the neighbouring building (457 Chapel Road).
- Geotechnical pile inspections.
- Proof loading and lift-off tests of temporary rock anchors (if adopted).
- Monitoring of seepage inflows during bulk excavation.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the design and construction phases of the project. In the event that any of the advice presented in this report is 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.

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Sustainable Development Group Limited	**Ref No:**	37148PE
Project:	Proposed Residential Development	**Report:**	A
Location:	461 Chapel Road, BANKSTOWN, NSW	**Report Date:**	6/12/24

Page 1 of 2

BOREHOLE NUMBER	DEPTH (m)	$I_{S(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	3.66 - 3.68	0.03	1	A
	3.84 - 3.86	0.02	<1	A
	5.25 - 5.28	0.04	1	A
	5.56 - 5.58	0.2	4	A
	6.40 - 6.43	0.5	10	A
	6.82 - 6.84	0.2	4	A
	7.17 - 7.20	0.9	18	A
	7.58 - 7.60	1	20	A
	7.87 - 7.90	1.5	30	A
	8.31 - 8.35	1.1	22	A
	8.46 - 8.49	0.9	18	A
	8.92 - 8.95	0.8	16	A
	9.17 - 9.20	0.7	14	A
	9.67 - 9.70	1.6	32	A
	10.11 - 10.14	1.4	28	A
	10.46 - 10.49	0.9	18	A
	10.89 - 10.92	0.5	10	A
	11.12 - 11.16	0.9	18	A
	11.71 - 11.75	1.1	22	A
	12.00 - 12.03	1.5	30	A
2	6.43 - 6.45	0.5	10	A
	6.81 - 6.83	0.9	18	A
	7.04 - 7.06	1.5	30	A
	7.85 - 7.88	0.7	14	A
	8.52 - 8.55	0.8	16	A

NOTE: SEE PAGE 2

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Sustainable Development Group Limited **Ref No:** 37148PE

Project: Proposed Residential Development **Report:** A

Location: 461 Chapel Road, BANKSTOWN, NSW **Report Date:** 6/12/24

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	8.88 - 8.92	1.4	28	A
	9.16 - 9.19	1.1	22	A
	9.80 - 9.83	1	20	A
	10.24 - 10.28	0.3	6	A
	10.40 - 10.42	0.4	8	A
	10.96 - 10.99	0.7	14	A
	11.37 - 11.40	0.5	10	A
	11.80 - 11.82	0.8	16	A
	12.01 - 12.03	0.4	8	A
3	5.68 - 5.72	0.2	4	A
	5.94 - 5.97	0.5	10	A
	6.14 - 6.17	0.5	10	A
	6.91 - 6.93	0.3	6	A
	7.20 - 7.24	0.8	16	A
	7.78 - 7.82	0.8	16	A
	8.11 - 8.14	0.3	6	A
	8.95 - 8.99	0.3	6	A
	9.16 - 9.19	0.5	10	A
	9.58 - 9.61	0.8	16	A
	9.88 - 9.92	1.2	24	A
	10.07 - 10.10	0.6	12	A
	10.57 - 10.60	1.1	22	A
	10.97 - 10.99	0.4	8	A
	11.31 - 11.34	0.6	12	A
	11.70 - 11.74	0.6	12	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
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 based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{S(50)}$.

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CERTIFICATE OF ANALYSIS 368570

Client Details

Client	JK Geotechnics
Attention	Christopher Rooke
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**37148PE Bankstown**
Number of Samples	3 Soil
Date samples received	10/12/2024
Date completed instructions received	10/12/2024

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.

Report Details

Date results requested by	17/12/2024
Date of Issue	16/12/2024

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		368570-1	368570-2	368570-3
Your Reference	UNITS	BH1	BH2	BH3
Depth		1.5-1.9	0-0.2	3.2-3.6
Date Sampled		04/12/2024	04/12/2024	04/12/2024
Type of sample		Soil	Soil	Soil
Date prepared	-	06/12/2024	06/12/2024	06/12/2024
Date analysed	-	12/12/2024	12/12/2024	12/12/2024
pH 1:5 soil:water	pH Units	7.5	7.0	8.7
Chloride, Cl 1:5 soil:water	mg/kg	450	<10	220
Sulphate, SO4 1:5 soil:water	mg/kg	240	10	63
Resistivity in soil*	ohm m	19	230	38

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode. 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: 37148PE Bankstown

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			06/12/2024	[NT]	[NT]	[NT]	[NT]	06/12/2024	[NT]
Date analysed	-			12/12/2024	[NT]	[NT]	[NT]	[NT]	12/12/2024	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	103	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	107	[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.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

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.

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BOREHOLE LOG

Borehole No.
1
1 / 3

Client: SUSTAINABLE DEVELOPMENT GROUP LIMITED
Project: PROPOSED RESIDENTIAL BUILDING
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Job No.: 37148PE **Method:** SPIRAL AUGER **R.L. Surface:** ~29.9 m
Date: 2/12/24 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** C.A.R./M.E.

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: 20mm.t	M			
										FILL: Silty clayey gravel, fine to medium grained igneous, dark grey.				
										FILL: Silty clay, medium plasticity, grey and orange brown, with fine to medium grained igneous gravel.	w~PL			
					N = 4 1,2,2	29	1		CH	Silty CLAY: high plasticity, grey and red brown, trace of root fibres.	w>PL	St	140 150 160	RESIDUAL
ON COMPLETION OF CORING ▽					N = 16 3,5,11	28				Silty CLAY: high plasticity, grey and light brown, trace of root fibres.		VSt	300 330 280	
							2		-	SILTSTONE: dark grey, with occasional iron indurated bands.	DW	L		ASHFIELD SHALE LOW 'TC' BIT RESISTANCE
						27 26 25 24 23	3 4 5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 7.8m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.8m TO 7.8m. CASING 0m TO 2.8m. 2mm SAND FILTER PACK 2.5m TO 7.8m. BENTONITE SEAL 0m TO 2.5m. COMPLETED WITH A CONCRETED GATIC COVER.

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1

Client: SUSTAINABLE DEVELOPMENT GROUP LIMITED
Project: PROPOSED RESIDENTIAL BUILDING
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Job No.: 37148PE
Core Size: NMLC
R.L. Surface: ~29.9 m
Date: 2/12/24
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: C.A.R./M.E.

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	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 2.80m							
90% RETURN		27	3		NO CORE 0.60m							
		26	4		Extremely Weathered siltstone: silty CLAY, medium plasticity, grey brown and dark grey, with occasional very low strength siltstone bands and iron indurated bands.	XW	Hd	0.030 0.020				
		25	5		SILTSTONE: grey, bedded at 0-10°.	HW	VL			(4.35m) J, 70°, P, S, Fe Sn (4.47m) J, 90°, P, S, Fe Sn (4.72m) XWS, 0°, 140 mm.t (4.84m) Be, 0°, P, S, Clay FILLED, 8 mm.t (4.97m) XWS, 0°, 50 mm.t		Ashfield Shale
					SANDSTONE: fine to medium grained, grey brown, with occasional siltstone lenses, bedded at 0-5°.	MW		0.040		(5.38m) XWS, 0°, 25 mm.t		
					SILTSTONE: dark grey, bedded at 0-5°.	SW	L	0.20		(5.44m) XWS, 0°, 20 mm.t		
		24	6		NO CORE 0.50m							
		23	7		SILTSTONE: dark grey and grey, bedded at 0-5°.	SW	L - M	0.50 0.20 0.90		(6.60m) XWS, 0°, 20 mm.t (6.79m) XWS, 0°, 30 mm.t (6.90m) XWS, 0°, 80 mm.t		Ashfield Shale
		22	8				M - H	1.0 1.5 1.1 0.90		(7.64m) Be x 2, 0°, P/Ir, R, Clay Ct (7.82m) Be x 2, 0°, P, R, Cb Sn (8.17m) XWS, 0°, 40 mm.t (8.23m) XWS, 0°, 30 mm.t (8.40m) XWS, 0°, 75 mm.t (8.53m) J, 80°, P, R, Cn (8.59m) Be, 5°, P, R, Clay Ct		
		21			LAMINITE: fine grained, grey and dark grey, thinly to moderately bedded at 0-10°.	FR		0.80				

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 37148PE BANKSTOWN.GPJ <<DrawingFile>> 29/01/2025 10:35 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

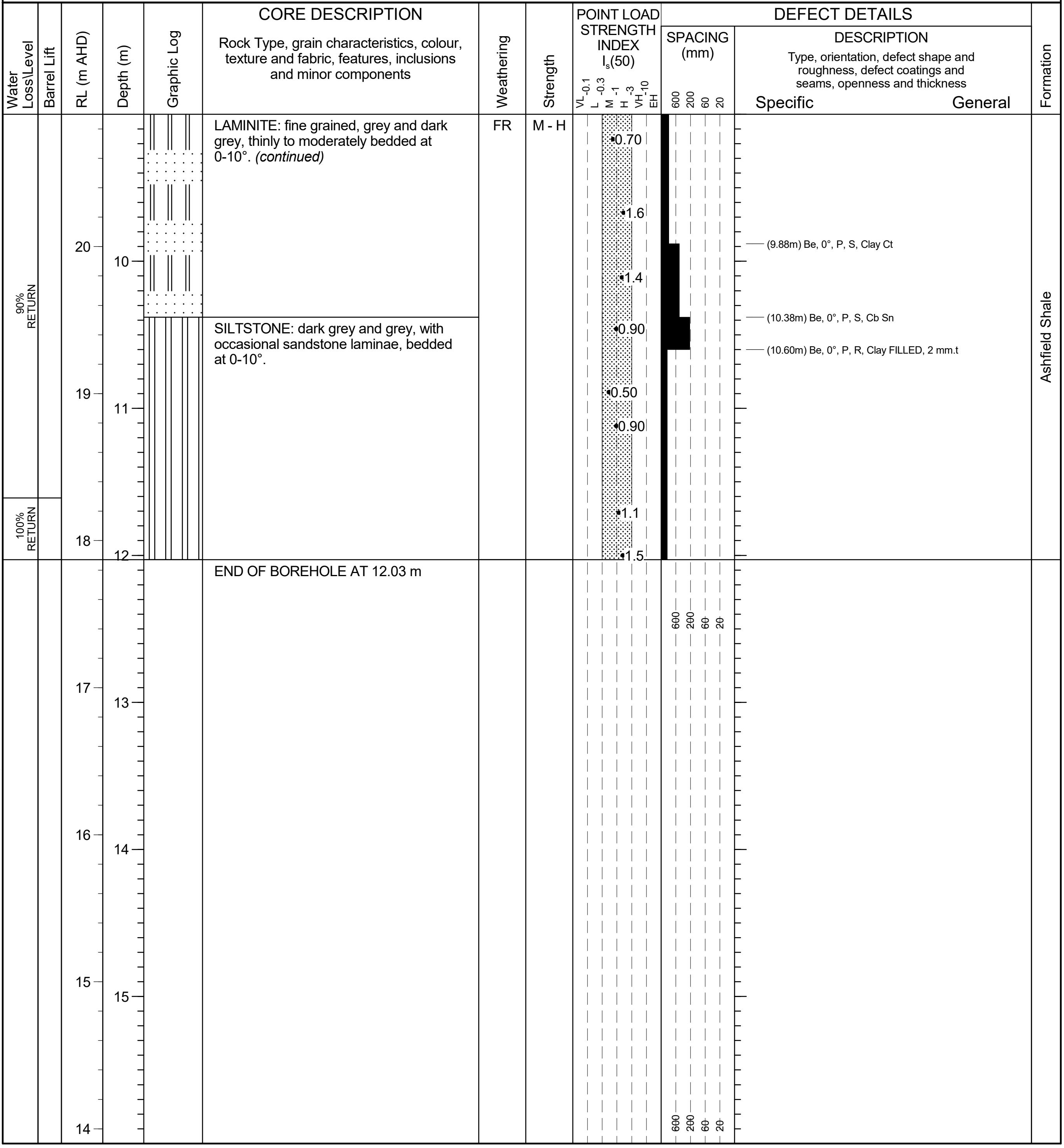
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

CORED BOREHOLE LOG

Borehole No.
1
3 / 3

Client: SUSTAINABLE DEVELOPMENT GROUP LIMITED
Project: PROPOSED RESIDENTIAL BUILDING
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Job No.: 37148PE
Core Size: NMLC
R.L. Surface: ~29.9 m
Date: 2/12/24
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: C.A.R./M.E.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	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 DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
90% RETURN		20	10	LAMINITE: fine grained, grey and dark grey, thinly to moderately bedded at 0-10°. *(continued)*	FR	M - H	0.70 1.6 1.4	(9.88m) Be, 0°, P, S, Clay Ct		Ashfield Shale
		19	11	SILTSTONE: dark grey and grey, with occasional sandstone laminae, bedded at 0-10°.			0.90 0.50 0.90	(10.38m) Be, 0°, P, S, Cb Sn (10.60m) Be, 0°, P, R, Clay FILLED, 2 mm.t		
100% RETURN		18	12				1.1 1.5			
		17 16 15 14	13 14 15	END OF BOREHOLE AT 12.03 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 37148PE
Borehole No: BH1
Depth: 2.80m to 11.00m

37148PE BH1 START CORING AT 2.80m

JKGeotechnics

Job No: 37148PE
Borehole No: BH1
Depth: 11.00m to 12.03m

BOREHOLE LOG

Borehole No.
2
1 / 2

Client: SUSTAINABLE DEVELOPMENT GROUP LIMITED
Project: PROPOSED RESIDENTIAL BUILDING
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Job No.: 37148PE **Method:** SPIRAL AUGER **R.L. Surface:** ~28.9 m
Date: 3/12/24 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** C.A.R./M.E.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					FILL: Silty clay, low plasticity, dark grey, trace of fine to medium grained igneous gravel, glass, ceramic and concrete fragments, ash and roots and root fibres.	w>PL			GRASS COVER
	N = 7 2,3,4	28	1	CH	FILL: Silty clay, medium to high plasticity, grey brown, trace of fine grained igneous gravel, brick and ceramic fragments, and ash.	w>PL	St - VSt	180 260 210	RESIDUAL
					Silty CLAY: high plasticity, light grey, red brown and light brown, trace of fine to medium grained ironstone gravel.				
	N = 25 3,7,18				as above, but light grey.		VSt - Hd	330 300 410	
ON COMPLETION OF CORING ▽		27	2	-	SANDSTONE: fine to medium grained, grey brown, with iron indurated bands.	DW	VL - L		ASHFIELD SHALE VERY LOW TO LOW 'TC' BIT RESISTANCE
					SILTSTONE: grey and grey brown, with iron indurated bands.				
		26	3				L		LOW RESISTANCE
		25	4						
		24	5						GROUNDWATER MONITORING WELL INSTALLED TO 7.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.5m TO 7.5m. CASING 0m TO 2.5m. 2mm SAND FILTER PACK 2.8m TO 7.5m. BENTONITE SEAL 0m TO 2.8m. COMPLETED WITH A CONCRETED GATIC COVER.
		23	6		REFER TO CORED BOREHOLE LOG				
		22							

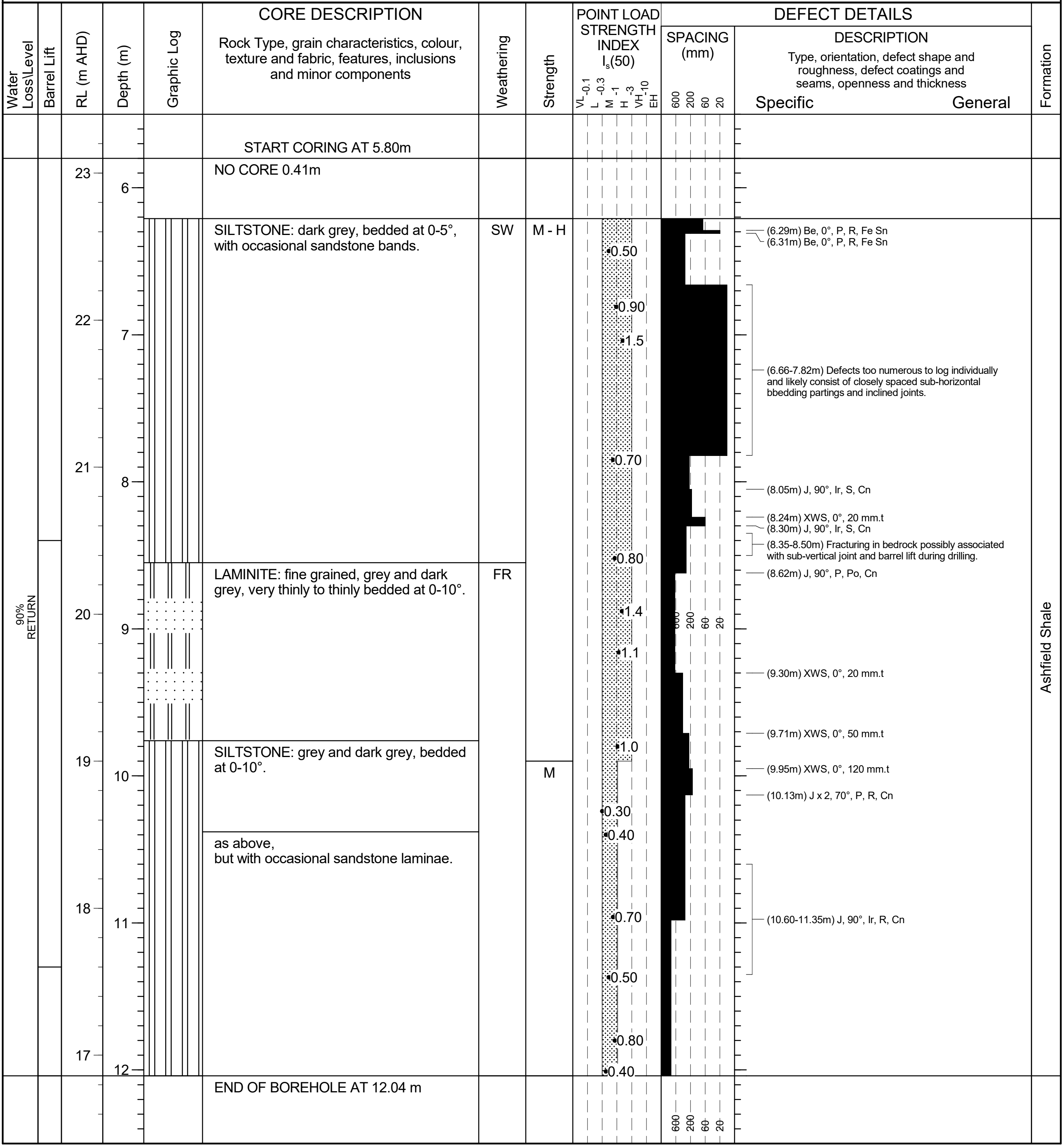
SAMPLES: ES U50 DB DS

CORED BOREHOLE LOG

Borehole No.
2
2 / 2

Client: SUSTAINABLE DEVELOPMENT GROUP LIMITED
Project: PROPOSED RESIDENTIAL BUILDING
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Job No.: 37148PE
Core Size: NMLC
R.L. Surface: ~28.9 m
Date: 3/12/24
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: C.A.R./M.E.



CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength
START CORING AT 5.80m		
NO CORE 0.41m		
SILTSTONE: dark grey, bedded at 0-5°, with occasional sandstone bands.	SW	M - H
LAMINITE: fine grained, grey and dark grey, very thinly to thinly bedded at 0-10°.	FR	
SILTSTONE: grey and dark grey, bedded at 0-10°.		M
as above, but with occasional sandstone laminae.		
END OF BOREHOLE AT 12.04 m		

DEFECT DETAILS

DESCRIPTION
Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness
Specific General

- (6.29m) Be, 0°, P, R, Fe Sn
- (6.31m) Be, 0°, P, R, Fe Sn
- (6.66-7.82m) Defects too numerous to log individually and likely consist of closely spaced sub-horizontal bbedding partings and inclined joints.
- (8.05m) J, 90°, Ir, S, Cn
- (8.24m) XWS, 0°, 20 mm.t
- (8.30m) J, 90°, Ir, S, Cn
- (8.35-8.50m) Fracturing in bedrock possibly associated with sub-vertical joint and barrel lift during drilling.
- (8.62m) J, 90°, P, Po, Cn
- (9.30m) XWS, 0°, 20 mm.t
- (9.71m) XWS, 0°, 50 mm.t
- (9.95m) XWS, 0°, 120 mm.t
- (10.13m) J x 2, 70°, P, R, Cn
- (10.60-11.35m) J, 90°, Ir, R, Cn

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 37148PE
Borehole No: BH2
Depth: 5.80m to 12.04m

37148PE BH2 START CORING AT 5.80m

END OF BOREHOLE AT 12.04m

BOREHOLE LOG

Borehole No.
3
1 / 3

Client: SUSTAINABLE DEVELOPMENT GROUP LIMITED
Project: PROPOSED RESIDENTIAL BUILDING
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Job No.: 37148PE **Method:** SPIRAL AUGER **R.L. Surface:** ~29.5 m
Date: 3/12/24 TO 4/12/24 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** C.A.R./M.E.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		29			FILL: Silty clay, low plasticity, dark brown, trace of igneous and alluvial gravel, fine to medium grained sand, and concrete, glass, plastic and ceramic fragments.	w>PL			GRASS COVER
	N = 7 2,3,4		1	CH	Silty CLAY: high plasticity, light brown, grey and red brown, trace of fine grained ironstone gravel, and root fibres.	w>PL	St - VSt	200 220 210	RESIDUAL
	N = 8 1,4,4	28	2		as above, but light grey and light brown, and without root fibres.			190 200 210	
1 DAY AFTER CORING ▽		27	3	-	SANDSTONE: fine to medium grained, grey brown, with iron indurated bands and extremely weathered bands.	DW	VL - L		ASHFIELD SHALE VERY LOW TO LOW 'TC' BIT RESISTANCE
		26			SILTSTONE: dark grey, with extremely weathered bands.		L		LOW RESISTANCE
		25 24 23	4 5 6		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 9.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.0m TO 9.0m. CASING 0m TO 3.0m. 2mm SAND FILTER PACK 3.0m TO 9.0m. BENTONITE SEAL 2.0m TO 3.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

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CORED BOREHOLE LOG

Borehole No.
3
2 / 3

Client: SUSTAINABLE DEVELOPMENT GROUP LIMITED
Project: PROPOSED RESIDENTIAL BUILDING
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Job No.: 37148PE
Core Size: NMLC
R.L. Surface: ~29.5 m
Date: 3/12/24 TO 4/12/24
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: C.A.R./M.E.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	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
		26		START CORING AT 3.67m						
90% RETURN		25 24	4 5	Extremely Weathered siltstone: silty CLAY, medium plasticity, dark grey and grey brown, with very low strength siltstone bands and iron indurated bands.	XW	Hd				Ashfield Shale
		23	6	SILTSTONE: dark grey and grey, bedded at 0-5°.	MW	L - M	0.20 0.50 0.50	60 20	(5.72m) XWS, 0°, 70 mm.t (5.83m) Be, 0°, P, R, Cn (5.86m) Be, 0°, P, S, Clay FILLED, 2 mm.t (6.03m) Be, 0°, P, S, Cn (6.34m) XWS, 0°, 80 mm.t (6.45m) Be, 0°, P, S, Clay FILLED, 5 mm.t (6.48m) Be, 0°, P, S, Clay FILLED, 5 mm.t	
ON 10/12/24				Extremely Weathered siltstone, silty CLAY, medium plasticity, dark grey and grey.	XW	Hd				
		22	7 8	SILTSTONE: dark grey and grey, bedded at 0-5°.	SW	M	0.30 0.80 0.80 0.30		(6.95m) XWS, 0°, 10 mm.t (7.15m) Be, 0°, P, S, Clay FILLED, 2 mm.t (7.63m) Be, 5°, P, R, Cn (7.71m) Be, 0°, P, S, Clay FILLED, 8 mm.t (8.03m) XWS, 0°, 25 mm.t	
				NO CORE 0.11m						
		21		SILTSTONE: dark grey and grey, with occasional sandstone laminae, bedded at 0-5°.	SW	M			(8.32-8.94m) Defects too numerous to log individually and likely consist of closely spaced sub-horizontal bedding partings and inclined joints	Ashfield Shale
			9		FR		0.30 0.50			
		20				M - H	0.80 1.2	200 60 20	(9.32m) Be, 0°, Ir, R, Cn (9.45m) Be, 0°, Ir, R, Cn (9.97m) Be, 0°, P, R, Clay Ct	

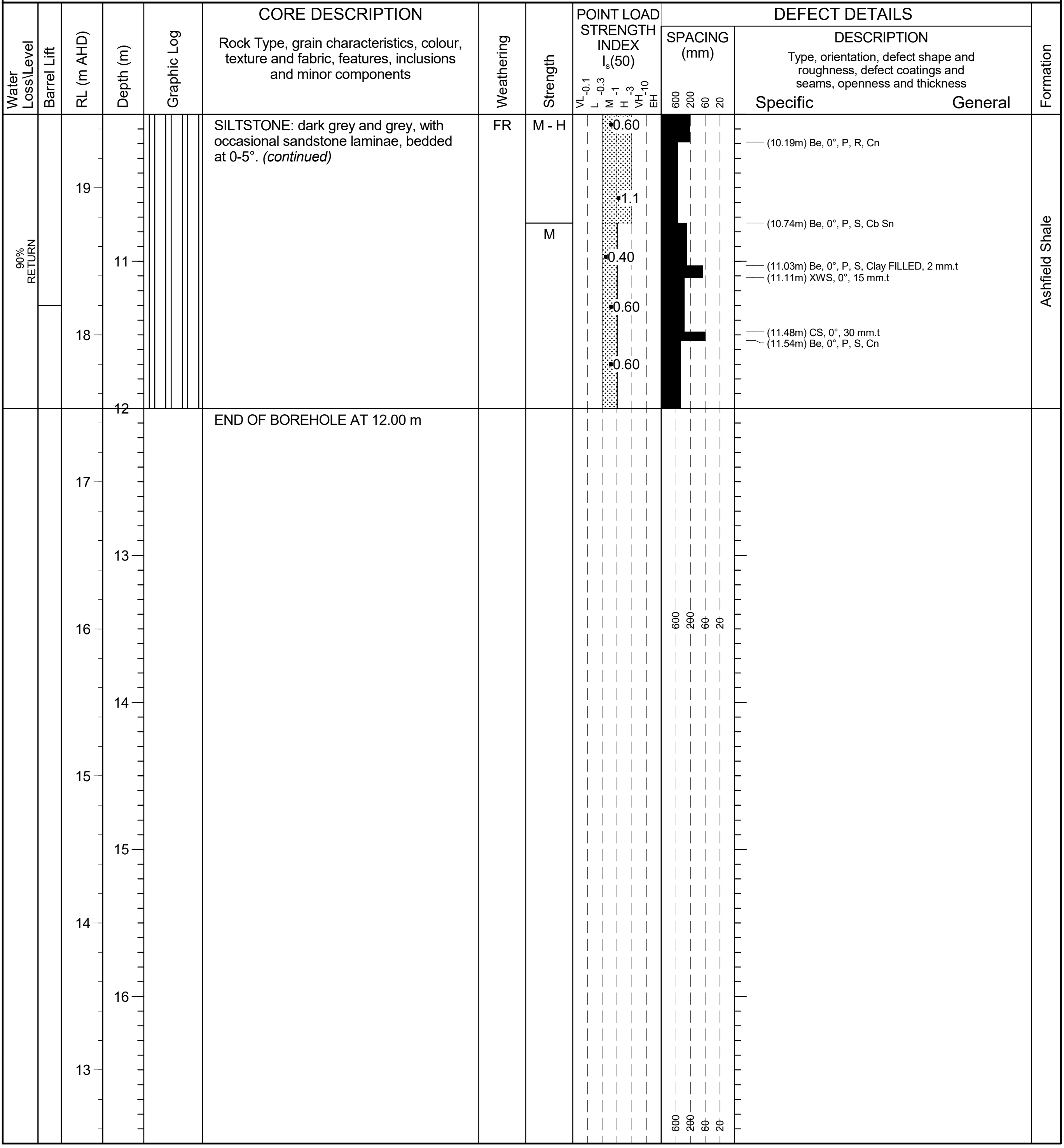
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

CORED BOREHOLE LOG

Borehole No.
3
3 / 3

Client: SUSTAINABLE DEVELOPMENT GROUP LIMITED
Project: PROPOSED RESIDENTIAL BUILDING
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Job No.: 37148PE
Date: 3/12/24 TO 4/12/24
Plant Type: JK308
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: ~29.5 m
Datum: AHD
Logged/Checked By: C.A.R./M.E.



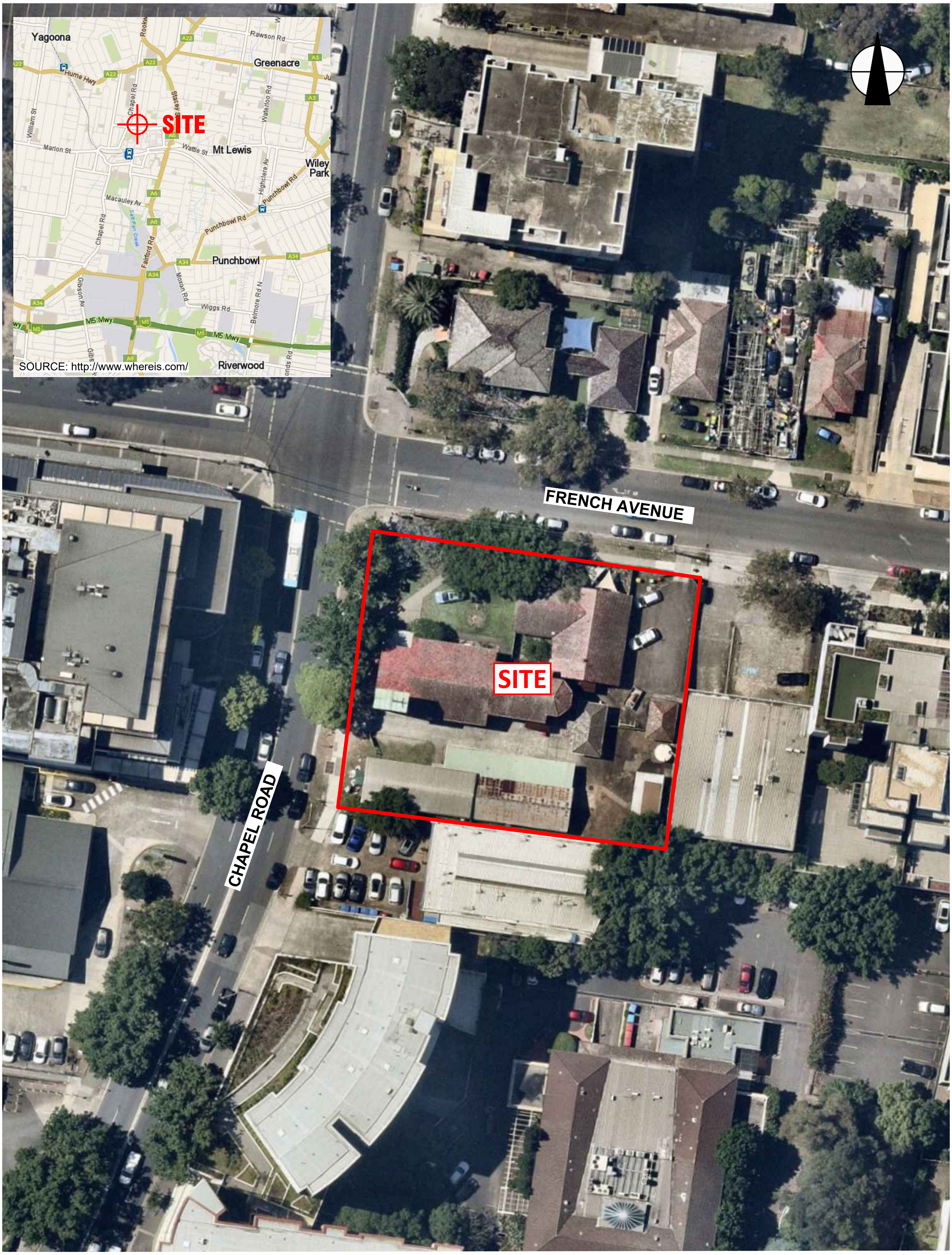
Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
to 12	SILTSTONE: dark grey and grey, with occasional sandstone laminae, bedded at 0-5°. *(continued)*	FR	M - H M	(10.19m) Be, 0°, P, R, Cn (10.74m) Be, 0°, P, S, Cb Sn (11.03m) Be, 0°, P, S, Clay FILLED, 2 mm.t (11.11m) XWS, 0°, 15 mm.t (11.48m) CS, 0°, 30 mm.t (11.54m) Be, 0°, P, S, Cn	Ashfield Shale
12	END OF BOREHOLE AT 12.00 m				

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 37148PE
Borehole No: BH3
Depth: 3.67m to 12.00m

37148PE BH3 START CORING AT 3.67m

END OF BOREHOLE AT 12.00m



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

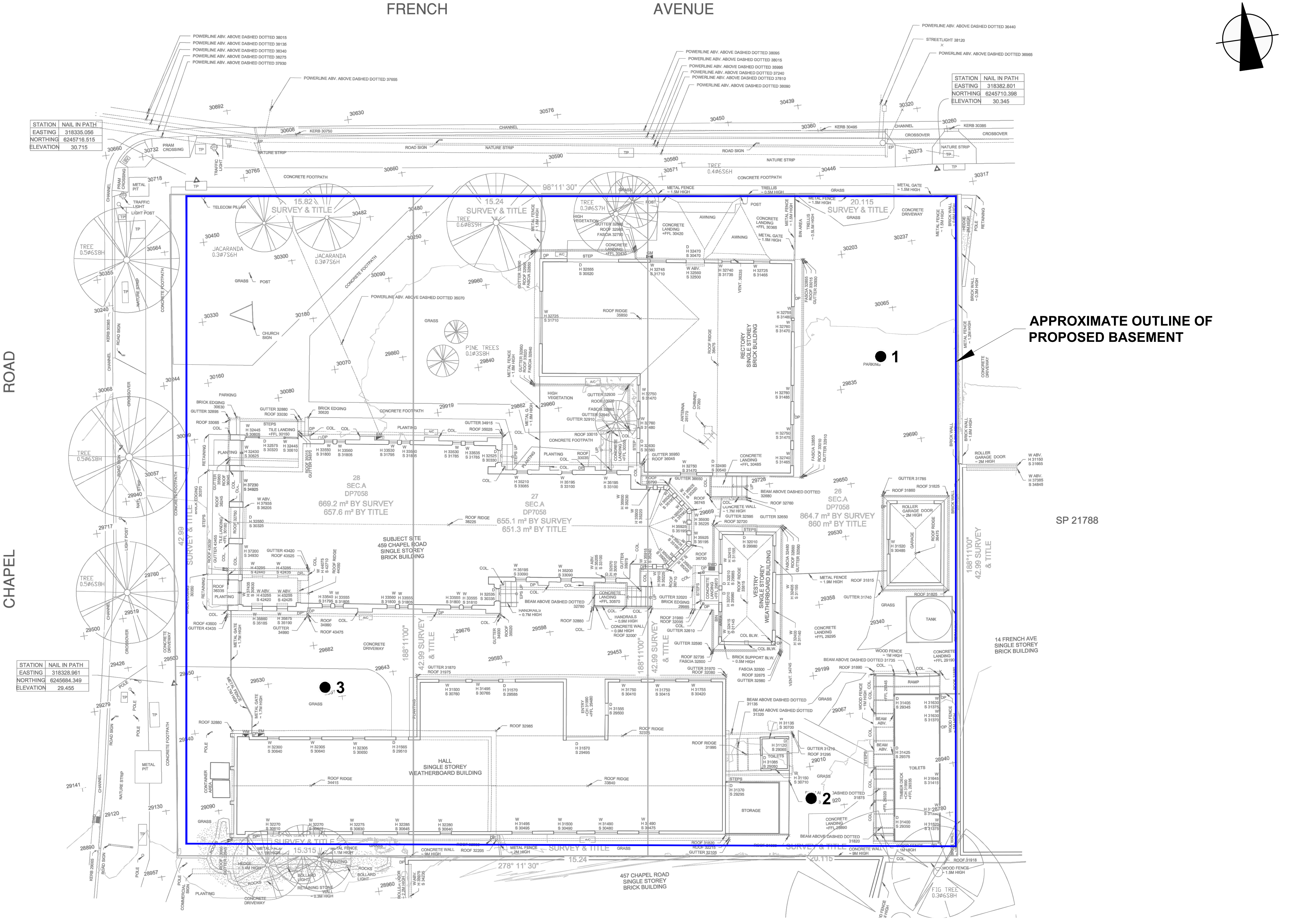
Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Report No: 37148PE

Figure No: 1

JKGeotechnics

PLOT DATE: 13/12/2024 11:08:28 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\37000'S\37148PE BANKSTOWN\CAD\37148PE.DWG



0 2.5 5 7.5 10 12.5

SCALE 1:250 @A3 METRES

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

Title:

BOREHOLE LOCATION PLAN

Location: 461 CHAPEL ROAD, BANKSTOWN, NSW

Report No: 37148PE

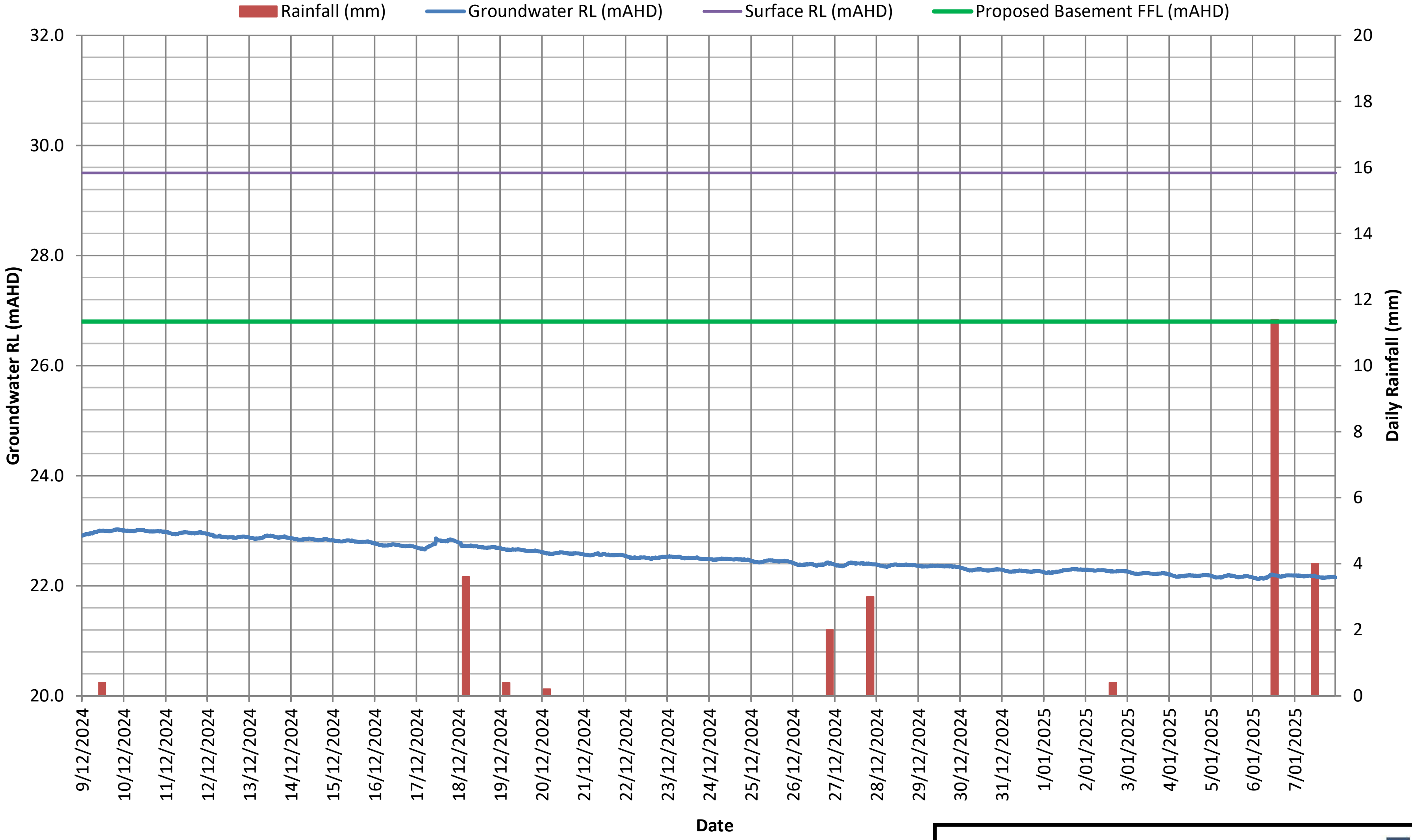
Figure No: 2

JKGeotechnics

PLOT DATE: 8/01/2025 1:25:18 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37148PE BANKSTOWN\CAD\37148PE.DWG

Groundwater Level and Daily Rainfall -v- Time Plot

BH3



Rainfall data from Bankstown Airport, Station No. 066137

JKGeotechnics

Report No. 37148PE

Figure No. 3

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)
ᴪ 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:

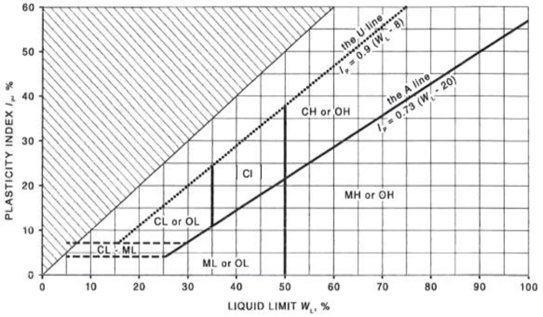
$$C_U = \frac{D_{60}}{D_{10}} \qquad and \qquad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

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

<table>
<tr><th>Log Column</th><th colspan="2">Symbol</th><th colspan="3">Definition</th></tr>
<tr><td rowspan="3">Groundwater Record</td><td colspan="2">▼</td><td colspan="3">Standing water level. Time delay following completion of drilling/excavation may be shown.</td></tr>
<tr><td colspan="2">C</td><td colspan="3">Extent of borehole/test pit collapse shortly after drilling/excavation.</td></tr>
<tr><td colspan="2">▶</td><td colspan="3">Groundwater seepage into borehole or test pit noted during drilling or excavation.</td></tr>
<tr><td rowspan="7">Samples</td><td colspan="2">ES</td><td colspan="3">Sample taken over depth indicated, for environmental analysis.</td></tr>
<tr><td colspan="2">U50</td><td colspan="3">Undisturbed 50mm diameter tube sample taken over depth indicated.</td></tr>
<tr><td colspan="2">DB</td><td colspan="3">Bulk disturbed sample taken over depth indicated.</td></tr>
<tr><td colspan="2">DS</td><td colspan="3">Small disturbed bag sample taken over depth indicated.</td></tr>
<tr><td colspan="2">ASB</td><td colspan="3">Soil sample taken over depth indicated, for asbestos analysis.</td></tr>
<tr><td colspan="2">ASS</td><td colspan="3">Soil sample taken over depth indicated, for acid sulfate soil analysis.</td></tr>
<tr><td colspan="2">SAL</td><td colspan="3">Soil sample taken over depth indicated, for salinity analysis.</td></tr>
<tr><td rowspan="6">Field Tests</td><td colspan="2">N = 17
4, 7, 10</td><td colspan="3">Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. ‘Refusal’ refers to apparent hammer refusal within the corresponding 150mm depth increment.</td></tr>
<tr><td rowspan="3">N_c =</td><td>5</td><td colspan="3" rowspan="3">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.</td></tr>
<tr><td>7</td></tr>
<tr><td>3R</td></tr>
<tr><td colspan="2">VNS = 25</td><td colspan="3">Vane shear reading in kPa of undrained shear strength.</td></tr>
<tr><td colspan="2">PID = 100</td><td colspan="3">Photoionisation detector reading in ppm (soil sample headspace test).</td></tr>
<tr><td rowspan="5">Moisture Condition
(Fine Grained Soils)</td><td colspan="2">w > PL</td><td colspan="3">Moisture content estimated to be greater than plastic limit.</td></tr>
<tr><td colspan="2">w ≈ PL</td><td colspan="3">Moisture content estimated to be approximately equal to plastic limit.</td></tr>
<tr><td colspan="2">w < PL</td><td colspan="3">Moisture content estimated to be less than plastic limit.</td></tr>
<tr><td colspan="2">w ≈ LL</td><td colspan="3">Moisture content estimated to be near liquid limit.</td></tr>
<tr><td colspan="2">w > LL</td><td colspan="3">Moisture content estimated to be wet of liquid limit.</td></tr>
<tr><td rowspan="3">(Coarse Grained Soils)</td><td colspan="2">D</td><td colspan="3">DRY – runs freely through fingers.</td></tr>
<tr><td colspan="2">M</td><td colspan="3">MOIST – does not run freely but no free water visible on soil surface.</td></tr>
<tr><td colspan="2">W</td><td colspan="3">WET – free water visible on soil surface.</td></tr>
<tr><td rowspan="8">Strength (Consistency)
Cohesive Soils</td><td colspan="2">VS</td><td colspan="3">VERY SOFT – unconfined compressive strength ≤ 25kPa.</td></tr>
<tr><td colspan="2">S</td><td colspan="3">SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.</td></tr>
<tr><td colspan="2">F</td><td colspan="3">FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.</td></tr>
<tr><td colspan="2">St</td><td colspan="3">STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.</td></tr>
<tr><td colspan="2">VSt</td><td colspan="3">VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.</td></tr>
<tr><td colspan="2">Hd</td><td colspan="3">HARD – unconfined compressive strength > 400kPa.</td></tr>
<tr><td colspan="2">Fr</td><td colspan="3">FRIABLE – strength not attainable, soil crumbles.</td></tr>
<tr><td colspan="2">( )</td><td colspan="3">Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.</td></tr>
<tr><td rowspan="7">Density Index/
Relative Density
(Cohesionless Soils)</td><td colspan="2"></td><td></td><td>Density Index (I_D) Range (%)</td><td>SPT ‘N’ Value Range (Blows/300mm)</td></tr>
<tr><td colspan="2">VL</td><td>VERY LOOSE</td><td>≤ 15</td><td>0 – 4</td></tr>
<tr><td colspan="2">L</td><td>LOOSE</td><td>> 15 and ≤ 35</td><td>4 – 10</td></tr>
<tr><td colspan="2">MD</td><td>MEDIUM DENSE</td><td>> 35 and ≤ 65</td><td>10 – 30</td></tr>
<tr><td colspan="2">D</td><td>DENSE</td><td>> 65 and ≤ 85</td><td>30 – 50</td></tr>
<tr><td colspan="2">VD</td><td>VERY DENSE</td><td>> 85</td><td>> 50</td></tr>
<tr><td colspan="2">( )</td><td colspan="3">Bracketed symbol indicates estimated density based on ease of drilling or other assessment.</td></tr>
<tr><td>Hand Penetrometer
Readings</td><td colspan="2">300
250</td><td colspan="3">Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.</td></tr>
</table>

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

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>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.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>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.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>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.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>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.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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 $Is_{(50)}$ (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